FLOOD PLAIN ROAD DESIGN ANALYSIS IN LAKSAR AREA USING GEOMATICS TECHNIQUES

A

DISSERTATION

submitted in partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(with specialization in Geomatics Engineering)

By

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CANDIDATE’S DECLARATION

It hereby declare that the work which is being presented in the dissertation entitled ‘**FLOOD PLAIN ROAD DESIGN ANALYSIS IN LAKSAR AREA USING GEOMATICS TECHNIQUES’** towards the partial fulfilment of the requirement for the award of the degree of **Master of Technology** in **Civil Engineering** submitted in the Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee (India) is an authentic record of my own work carried out under the guidance of **Dr. Kamal Jain**(Professor), Department of Civil Engineering, Indian Institute of Technology Roorkee.

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

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CERTIFICATE

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

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ABSTRACT

Flood plains may be regarded place nearby riverside which are prone to frequent floods when the river is flowing at high stage. In these conditions the areas along the riverside get submerged under the influence of floods, consequently these areas get waterlogged for a considerable period of time an if any mode of transportation such as a roadway in the flood plains, the same also get affected and become dysfunctional during flood. Therefore special design considerations are required to be taken in account while designing the roads passing through flood plains. For this, the region under consideration was divided into various sub catchments and peak discharge (maximum surface run-off) for each sub catchment was estimated for maximum rainfall of return period 50 years and corresponding to the peak discharge, high flood level for each sub catchment was estimated. The peak discharge was calculated from different methodologies such as synthetic unit hydrograph method, rational method, empirical relationships and maximum of the peak discharge obtained from the different methodologies was in the calculation of high flood level. The peak discharge estimated in sub catchments through which the Laksar Purquaji major district road was passes were 179.7 m^3/sec, 357.5 m^3/sec, 427.3 m^3/sec, 174.3 m^3/sec and corresponding high flood levels of sub catchments were 236.03 m, 232.35 m, 230.8 m, 230.94 m. A total stretch of road length of about 17 km was found to be below the high flood level that is road under submergence during floods and was designed as per IRC SP 34:2011 specifications to be properly functional during floods of 50 year return period. At the stream and road crossing, in various sub catchment a cross-drainage of suitable size was designed for surface runoff to recede from the catchment without affecting the road. The design bridge opening after providing appropriate vertical clearance as per IRC SP 13:2011 for various sub catchments was estimated as 123.54 m^2, 229.0 m^2, 294.6 m^2, 115.0 m^2. Another region taken into consideration was the proposed NH58 bypass from Mangalore towards Rehmadpur which has a crossing at Solani river. The peak discharge estimated for the sub-catchment in the region corresponding to maximum rainfall of return period 50 years was 255.33 m^3/sec and the inlet discharge in sub catchment was added to the peak discharge to obtain the total discharge at the outlet of the sub catchment .The total discharge hence obtained was 2522 m^3/sec. The high flood level corresponding to the observed discharge in the stream was estimated as 252.9 m and a length of about 3 km of proposed path for the bypass was observed below high flood level that is the region under submergence during floods. The elevation of road subgrade in this region under submergence was designed as per IRC SP 34:2011 recommendations and the size of design bridge opening beneath the road for surface run off to recede from the sub catchment during floods after providing appropriate vertical clearance(between water level and bottom of bridge deck) as per IRC SP 13:2011 estimated was 1078.3 m^2.

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CHAPTER-1

INTRODUCTION

1.1 General

A well-knit and coordinated system of transport plays an important role in the sustained economic growth of a country. Transport routes are the basic economic arteries of the country. Transport system is regarded as the controller of the national economy and provides a very important link between production and consumption. The amount of traffic moving in a country is a measure of its progress.
In India, the importance of transport is more because of its vastness as well as varied nature of geographical conditions. In India, it is also a source of national integration. The present Indian transport system comprises several modes including rail, road, coastal shipping, air transport, etc. Transport has recorded a substantial growth over the years both in terms of length and output of the system.

India has a **road network** of over 4,689,842 lakh kilometers, the second largest road network in the world. The quantitative density of India's road network is 0.66 km of roads per square kilometer of land. Qualitatively India's roads are a mix of modern highways and narrow, unpaved roads, and are being improved. About 54 percent (2.53 million kilometers) of Indian roads are paved.

The history of roads in India is as old as civilization. Roads have been existing in India for the last 5,000 years. Modern developments of roads started in India after the First World War. During this period it was found that there is rapid rise in motor transport. So need for better roads became a necessity. Hence, in 1927, the Government appointed a committee called Road development Committee with Mr.M.R. Jayakar as the chairman (Jayakar committee).
Jayakar committee laid stress on long term planning program, for a period of 20 years. It also recommended holding of periodic road conferences to discuss about road construction and development. This led to the establishment of a semi-official technical body called Indian Road Congress (IRC) in 1934.
The committee recommended a dedicated research organization should be constituted to carry out research and development work. This resulted in the formation of Central Road Research Institute (CRRI) in 1950.
The Nagpur road conference (The Nagpur plan) of the chief engineers of the province convened by the government in 1943 drafted first twenty year planned development program for the period (1943-1963) with objective to construct 2 lakh kilometers of road across the country within 20 years, to achieve a road density of a road density of 16

kilometers per 100 sq.km and inclusion of star and grid pattern of roads throughout the country.

Under this plan the roads were first classified into four classes:

National highways which would pass through states, and places having national importance for strategic, administrative and other purposes.

State highways which would be the other main roads of a state.

District roads which would take traffic from the main roads to the interior of the district . According to the importance, some are considered as major district roads and the remaining as other district roads.

Village roads which would link the villages to the road system.

Some important objectives and recommendations of the committee were to construct

The Bombay road conference, held in 1961, second 20 year road plan (1961-1981) with an objective to construct total road length of about 10 lakh km and to increase the road density to changed 32kms/100 sq.km. The construction of 1600 km of expressways was also then included in the plan.

The Lucknow road conference, held in 1981 was the third 20 year plan was drafted with major objectives to construct a road length of 12 lakh kilometres by the year 2001 resulting in a road density of 82kms/100 sq.km, to improving the transportation facilities in villages, towns etc. such that no part of country is farther than 50 km from National Highway.

India, thus has a huge network of **National highways**. There are 228 national highways comprising a total length of 92851 km. Indian highways cover 1.7% of the total road network of India and carry 40% of the total traffic. The highway network of India is managed by the National Highway Authority of India which is responsible for development and maintenance of highways. The longest highway in India is NH 7 which stretches from Varanasi in Uttar Pradesh to Kanyakumari in the southern most point of Indian mainland. The shortest highway is NH47A which stretches from Ernakulam to Kochi and covers total length of 4 Km.

State highways in India are numbered highways that are laid and maintained by the state governments. The state highways are usually roads that link important cities, towns and district headquarters within the state and connect them with National Highways or highways of neighbouring states. These highways provide connections to industries or places from key areas in the state making them more accessible.

Major District Roads are important roads within a district connecting areas of production with markets and connecting these with each other or with the State Highways & National Highways. It also connects Taluka headquarters and rural areas to District headquarters within the state.

Road transport is vital to India's economy. It enables the country's transportation sector to contribute 4.7 percent towards India's gross domestic product, in comparison to railways that contributed 1 percent. India's road network carries over 65 percent of its freight and about 85 percent of passenger traffic.

Hence, there are various specifications for National Highways to work exceptionally well even under extreme conditions. One of the most important specifications is efficiently functioning of National highways in areas under the influence of floods, waterlogging.

The provision of suitable embankments or bridges is therefore included in the design and construction of highways.When the ground generally slopes from one sides to another, the embankment intercepts natural flow of rainwater and required crossing must be designed accordingly. In order to develop the road and railway formation level which get least affected due there are suitable provisions specified in Indian roads congress (IRC) codes for the design of embankments or bridge carrying the road or railway routes. Some of the specifications are finished road level must be at least 0.6m above ground level, the bottom of subgrade must be 1.0 m above the high flood level and .6 m in case of old existing roads (IRC SP 84-2014). As per IRC 34:2011 for highway construction in area prone to waterlogging minimum height of subgrade above HFL (Highest flood level) shall be 1.5 m.

1.2 Need of the study

Normally the IRC specifications are followed in the design of road or rail route. However it is seen that many stretches of National Highways, State highways and Major district roads get submerged due to floods in rainfall season as the water in the field flow from one side of the of the road/rail route to the other side making the road unusable and also it will damage the road.

The road/rail routes nearby flood plains are further prone to frequent floods due to which the region gets affected and consequently the mode of transportation also gets affected. Roorkee and Laksar areas are situated along riverside (Solani and Ganges river) may be regarded as flood plains hence subjected to frequent floods.

In flood plains during any rainfall event discharge collected at outlet of any catchment which subsequently flows into the river. But during the floods rivers also flow at full stage and hence water does not flow into the river. In this condition water becomes stagnant and the region starts to get submerged as the duration and intensity of the storm increases and high flood level is attained.

In addition to it, ground water infiltration during the rainfall in the flood plains is also very less or negligible as the ground water is recharged up to its capacity in flood plains.

Due to these factors the road in these regions and the surrounding area get flooded and waterlogged, not being usable for transportation with no alternate route.

Since the water is in the stagnant condition road/rail route in the region gets submerged and becomes dysfunctional for 5 to 7 days.

In view of this there is a need to design the formation level of the road line considering the highest flood level of the region so that it gets least affected due to floods and remains functional during floods of respective return period and to design the waterway beneath the road for water to recede during and after the floods. A casual approach may result in improper design being unsuitable corresponding to the magnitude of floods in the regions or may uneconomic structure designed for floods of very high return period.

1.3 Objective of the study

- To estimate the impacts of floods affecting the roads and the surrounding area, causing submergence in the flood plains.
- To study the surface run-off and rise in water levels in the study area due to heavy rainfall events.
- To obtain a methodology for the design of roads in these flood plains so that these gets least affected due to floods and remain functional during floods of appropriate return period.
- To design the size of waterway suitable to pass the surface run-off (of appropriate return period) beneath the road without affecting the functioning of roads.

1.4 Organization of thesis

The thesis is divided into six chapters. The content of each chapter are briefly described below:

Chapter 1 Gives brief general introduction about the importance and development of road networks in India, various specifications regarding highway design, description of problem definition, and objective of the study.

Chapter 2 includes basic theory about highway alignment and surveys, IRC specifications for the embankment design in flood plains, about cross drainage structures and IRC specifications for the construction of cross drainage structures.

Chapter 3 discusses about Watershed, various methodologies to compute surface run-off at outlet of watershed and relevance of high flood level. This chapter also includes the case study relevant to analysis.

Chapter 4 includes description of study area, acquisition of data and methodology adopted in the current analysis.

Chapter 5 includes observation, analysis and discussion of results obtained as per the methodology discussed in Chapter 4.

Chapter 6 includes summary of the results obtained in chapter 5 and conclusion of the analysis.

CHAPTER-2

GENERAL HIGHWAY AND CROSS DRAINAGE DESIGN SPECIFICATIONS

2.1 Highway Alignment and Surveys

2.1.1 Factors Controlling Alignment

The various factors which control the highways alignment in general area-

a) **Obligatory points**: There are control points governing the alignment of the highways. Obligatory points through which the road alignment has to pass may cause the alignment to often deviate from the shortest or easiest path. The various examples of this category may be bridge site, intermediate town, a mountain pass or a quarry.

b) **Traffic**: The alignment should suit traffic requirements, Origin and Destination study should be carried out in the area and the desire lines be drawn showing the trend of traffic flow. The new to be aligned should keep in view the desired lines, traffic flow patterns and future trends.

c) **Geometric Design**: It may be necessary to make adjustment in the horizontal alignment of roads keeping in view the minimum radius of curve and the transition curves. Alignment should be finalised in such a way that the observations to visibility do not cause restriction to the right distance requirements.

d) **Economy**: The alignment finalized based on the above factors should also be economical. The initial cost of construction can be decreased if high embankments and deep cuttings are avoided and the alignment is chosen in a manner to balance the cutting and filling.

e) **Other Considerations**: Various other factors which may govern the alignment are drainage consideration hydrological factors, political considerations and monotony.

3.1.2 Engineering surveys for Highway Locations:

Before a highway alignment is finalised in highway project, the engineering surveys are to be carried out. The stages of the engineering surveys are

a) Map Study

By careful study of maps, it is possible to have an idea of several possible alternate routers so that further details of these may be studied later at the site. The possible alignment can be located on the map from the following details available on the map.

a) Alignment avoiding valleys, ponds or lakes.

b) When the road has to cross a new of hills, possibility of crossing through a mountain pass.

c) Approximate location of bridge site for crossing rivers, avoiding bend of the river, if any.

d) When a road is to be connected between two stations, one on the top and other on the foot of the hill, then alternate routes can be suggested keeping in view the permissible gradient of the hill, then alternate routes can be suggested keeping in view the permissible gradient.

b) Reconnaissance

The second stage of surveys for highways location is the reconnaissance to examine the general character of the area for deciding the most feasible routes for detailed surface. A field surveys of land along the proposed alternative routes of the map in the field as studied. All relevant details not available in the map are collected and noted down. Some of the details to be collected during reconnaissance are given below:

i. Valleys, ponds, lakes, marshy land, ridge, hills permanent structures and other obstructions along the route which are not available in the map.

ii. Approximation values of gradient, length of gradients and radius of curves of alternate alignments.

iii. Number and type of cross drainage structures, maximum flood level and natural ground water level along the possible routes.

iv. Soil types along the routes.

v. Sources of construction materials.

From the details collected during the reconnaissance the alignment prepared after study may be altered or even changed completely.

c) Preliminary survey

The main objectives of preliminary survey are:

i) To survey the various alternate alignments proposed after the reconnaissance and to collect all the necessary information and details of topography, drainage and soil.

ii) To compare the different proposals in view of the requirements of a good alignment.

iii) To estimate quantity of earth work and to work out the cost of alternate proposals.

iv) To finalise the best alignment from all consideration.

The procedure of preliminary survey is given in following steps:

i) Primary traverse

ii) Topographical features

iii) Levelling work

iv) Drainage studies and Hydrological data.

v) Soil survey

vi) Material survey

vii) Traffic survey

viii) Determination of final centre line.

d) Final Location and Detailed Survey

The location finalized at the design office after the preliminary survey is to be first located on the field by establishing the centre line.

Next detailed survey should be carried out for collecting the information necessary for the preparation of plains and construction details for the highway project.

e) Location

The centre line of the road finalized in the drawings is to be translated on the ground during the location survey.

f) Detailed Survey

Temporary bench marks are fixed at intervals of about 250 meters and at all drainage and under pass structure. Levels along the final centre line should be taken earth work calculation and drainage details are to be worked out from the level routes. The cross section level are taken up to the desired width at interval of 50 to 100 meter in plain. The data during the detailed survey should be elaborate and complete for preparing detailed plains, design and estimates of the project.

2.2 Design specifications for embankments as per IRC

Specifications for embankment construction for National highways (IRC SP 84-2014**)**

- The height of the embankment shall be measured with respect to the finished road levels.
- No section of the road is overtopped. The finished road level shall be at least 0.6m above ground level(figure 2.1).
- The bottom of subgrade is generally 1.0 m above the high flood level/high water table. However, in the case of existing old roads where it may be difficult to fulfill

this criterion, the criteria is reduced to the bottom of subgrade 0.6 m above High Flood Level (HFL).

As per IRC 34:2011 for highway construction in area prone to waterlogging minimum height of subgrade above HFL shall be 1.5 m.

The provision of Causeways and submersible bridges must be strictly avoided at National Highways (IRC SP: 82-2008).

2.3 Cross Drainage structures

Construction of a road embankment unavoidably obstructs and interferes with the natural overland flow and flow through the natural streams. Whenever, streams have to cross the roadway, facility for cross drainage is to be provided. Also often the water from the side drain is taken across by these cross drain in order to divert the water away from the road, to a water course or valley. The cross drainage structures commonly in use are culverts and small bridges. When a small stream crosses a road with a linear waterway less than about the meter, the cross drainage structure provided a called culvert, for higher values of linear waterway, the structure is called as bridge.

Constructing bridges and culverts under road in high embankment is a better proposition than providing so many dips and causeways leading to disruption in traffic movement during flood season. The common types of culverts in use are, Slab Culvert, Box Culvert, Arch culvert, Pipe culvert.

2.3.1 Specification of the cross drainage structures across the stream

- The width of the waterway should be such that the opening is able to pass the maximum flows that may be expected without affecting the structure by scour, without creating major maintenance problems, without causing unacceptable backwater effects upstream, and without causing currents, waves, or turbulence unacceptable to navigation or other legitimate interests. Lacey's width of water way (W) for the regime channel for no silting and scouring is given by $W = 4.75Q^{.5}$ (Q is the discharge passing through the stream).
- The height of the formation level or the deck should be designed such that the superstructure does not get affected by the action of flowing water, floating debris, or waves, and the roadway is not rendered impassable.Height of the formation level shall be designed according to the designed high flood level for the given region.
- IRC SP 13:2011 recommendation for a cross drainage structure (culverts and small bridges) are,
  - Must be situated on a straight reach of stream, sufficiently downstream of bends.
  - Must be sufficiently away from the confluence of large tributaries as to be beyond their disturbing influence.
  - They must have well defined banks.
  - If possible it must offer a square crossing.

- Inspection should also include taking notes on channel conditions from which the silt factor and the co-efficient of rugosity can be estimated.
- IRC SP 13:2011 also prescribes minimum vertical clearance between the bottom of the bridge and the water level based on the discharge passing beneath the stream as shown in table 2.1.

Table 2.1 Discharge and minimum vertical clearance (IRC SP 13:2011)

Discharge in m^3/s	Minimum vertical clearance in mm
Upto 0.30	150
Above 0.3 and upto 3.0	450
Above 3 and upto 30	600
Above 30 and upto 300	900
Above 300 and upto 3000	1200
Above 3000	1500

- Specifications for Bridges and culverts as per IRC SP : 20 – 2002
  - The overall width of the culvert should be equal to the formation width of the road.
  - The cross drainage works should generally be sited on the straight alignment of a road.
  - If the road at the culvert is in gradient, the same gradient of road may be provided for deck slab of the culvert.
  - From the consideration of maintenance of culverts, it is preferable that the clear waterway of slab culvert is a minimum of 1.5 m and minimum diameter of pipe in case of pipe culvert is 1000mm (900 mm internal diameter).
  - Minimum height of the soffit of the slab should be 1.5 m above the lowest bed level from the consideration of inspection and maintenance.
  - Culverts are provided at the rate of one per 500m length of road to avoid water logging.
  - The minimum height of the formation level of the road from the bed level is 1.75m
- Specifications of cross drainage structures as per IRC SP 34:2011
  - IRC 34:2011 recommends "Sufficient number of cross drainage structures must be provided for movement of water across the embankment. For free flow of water across the embankment atleast 2 culvert/km must be provided.
  - In cases of roads passing through the region of flooding and waterlogging recommendation prescribed by IRC 34:2011.It recommends to do a careful examination of the structure above, and an embankment of such height may be provided that the bottom of the sub grade remains at least 1.5 m above the highest flood water level (Figure 2.1).

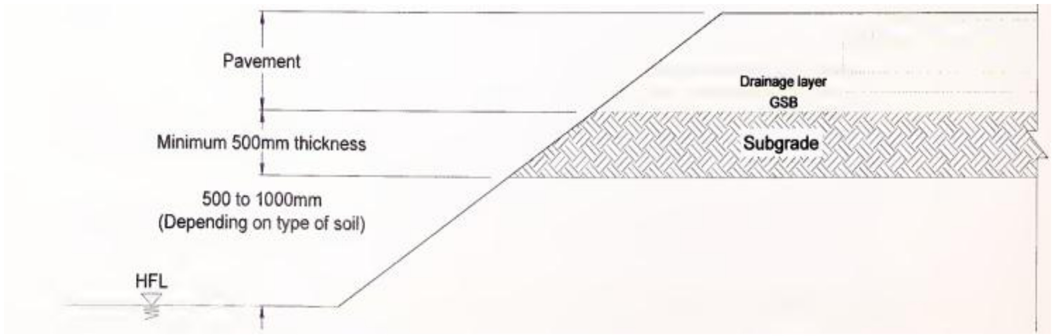

Figure 2.1 minimum height of subgrade above high flood level, HFL (IRC SP 34:2011)

- Provision of capillary cutoff.A capillary cutoff could be provided to arrest the capillary rise of water in the embankment. The capillary cutoff may be a layer of coarse or fine sand and graded gravel.
- The cut-off should be placed at least 0.15m above the ground level or the standing water level, whichever is higher, as illustrated in Figure 2.2. But in no case it shall be positioned higher than 0.6 m below the top of the sub grade. Material used in the capillary cut-off must be coarse grained material such as sand or gravelly sand to check the capillary rise in the embankment.

Table 2.2 Minimum thickness of capillary cutoff (from IRC SP 34:2011)

Sr.No.	Situations	Minimum thickness of granular layer (mm)		
		Fine Sand (425 micron to 2 mm)	Coarse Sand (2 mm to 4.75 mm)	Graded Gravel l (4.75 mm to 20 mm)
1)	Subgrade 0.6 – 1.0m above HFL (Pl>5)	350	150	150
2)	Subgrade 0.6 – 1.0 m above HFL, the subgrade soil being sandy in nature.	300	100	100

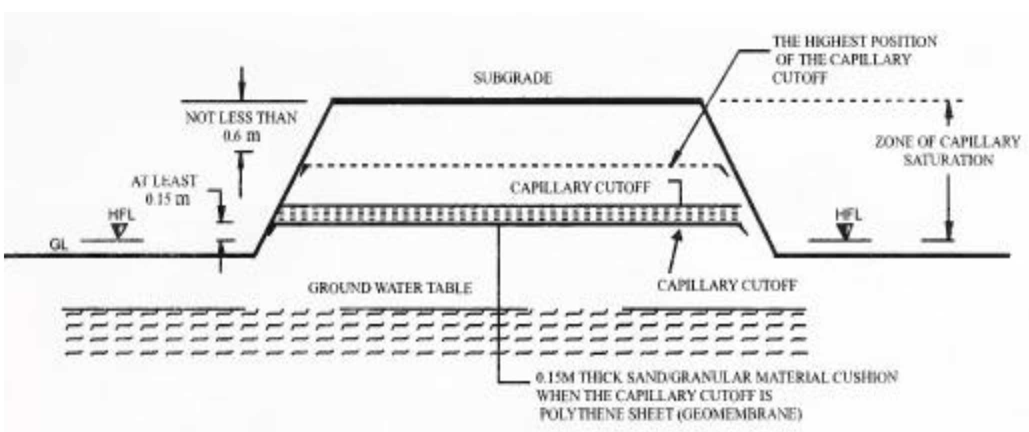

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Figure 2.2 Illustration of position of capillary cutoff for Preventing the Rise of Capillary Moisture (IRC SP 34:2011).

CHAPTER-3

HYDROLOGICAL CONSIDERATIONS AND CASE STUDIES

3.1 Watershed

The watershed through which the road is passing significantly affects the design characteristics of the road. In watershed streams of the watershed control the elevation of the roads. The size of the streams directly depends on the area of the watershed. The topography of the region, rainfall, infiltration, drainage density also impacts the design aspects of the road.

3.1.1 Catchment area and its characteristics

A **drainage basin** or **catchment area** is an extent or an area of land where surface water from rain, melting snow, or ice converges to a single point at a lower elevation, at the exit of the basin, where the waters join another waterbody, such as a river, lake, reservoir, estuary, wetland,sea or ocean. Figure 3.1 shows general overlay of a watershed

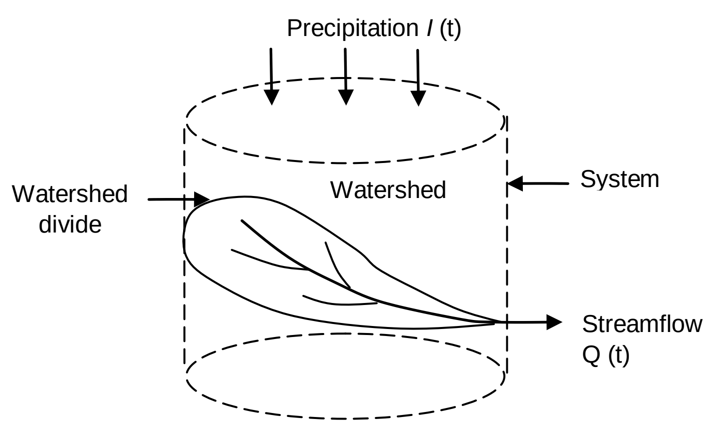

Figure 3.1 Watershed (from Chow et al)

3.1.2 Importance of drainage basins

In hydrology, the drainage basin is a logical unit of focus for studying the movement of water within the hydrologic cycle because the majority of water that discharges from the basin outlet originated as precipitation falling on the basin. A portion of the water that enters the groundwater system beneath the drainage basin may flow towards the outlet of another drainage basin because groundwater flow directions do not always match those of their overlying drainage network. Measurement of the discharge of water from a basin may be made by a stream gauge located at the basin's outlet.

Rain gauge data is used to measure total precipitation over a drainage basin, and there are different ways to interpret that data. If the gauges are many and evenly distributed over an area of uniform precipitation, using the arithmetic mean method will be useful in giving good results. In the Thiessen polygon method, the drainage basin is divided into polygons with the rain gauge in the middle of each polygon assumed to be representative for the rainfall on the area of land included in its polygon. These polygons are made by drawing lines between gauges, then making perpendicular bisectors of those lines form the polygons. The isohyetal method involves contours of equal precipitation are drawn over the gauge on a map. Calculating the area between these curves and adding up the volume of water is time consuming.

Isochrone maps can be used to show the time taken for runoff water within a drainage basin to reach a lake, reservoir or outlet, assuming constant and uniform effective rainfall.

3.1.3Catchment factors

The catchment is the most significant factor determining the amount or likelihood of flooding. Catchment factors are: topography, shape, size, soil type, and land use land cover. Catchment topography and shape determine the time taken for rain to reach the river or stream, while catchment size, soil type, and development determine the amount of water to reach the river or stream.

Topography

Generally, topography plays a big part in how fast runoff will reach a river. Rain that falls in steep mountainous areas will reach the primary river in the drainage basin faster than flat or lightly sloping areas (e.g., > 1% gradient).

Shape

Shape will contribute to the speed with which the runoff reaches a river. A long thin catchment will take longer to drain than a circular catchment.

Size

Size will help determine the amount of water reaching the river, as the larger the catchment the greater the potential for flooding. It also determined on the basis of length and width of the drainage basin.

Soil type

Soil type will help determine how much water reaches the river. Certain soil types such as sandy soils are very free-draining, and rainfall on sandy soil is likely to be absorbed by the ground. However, soils containing clay can be almost impermeable and therefore rainfall on clay soils will run off and contribute to flood volumes. After prolonged rainfall even free-draining soils can become saturated, meaning that any further rainfall will reach the river rather than being absorbed by the ground. If the surface is impermeable the precipitation will create surface run-off which will lead to higher risk of flooding; if the ground is permeable, the precipitation will infiltrate the soil.

Land use

Land use can contribute to the volume of water reaching the river, in a similar way to clay soils. For example, rainfall on roofs, pavements, and roads will be collected by rivers with almost no absorption into the groundwater.

3.2 Hydrographs

A plot of the discharge in the stream plotted against the time chronologically is called a hydrograph. Figure 3.2 shows general shape of a hydrograph and its elements. Depending upon the unit of time types of hydrograph are

- Annual hydrograph: It shows variation of daily weekly or mean flow of any number of successive days over a year.
- Monthly hydrograph: It shows variation of the daily mean flow over a month.
- Seasonal hydrograph: It shows variation of discharge in a particular season, such as monsoon season or dry season.
  The above three hydrographs are called long term hydrographs.
- Flood hydrographs or hydrograph due to a storm: It shows a stream flow due to storm over a catchment.
- Flood hydrograph on the other hand is used to study the flooding characteristics of a stream which is a short term study.

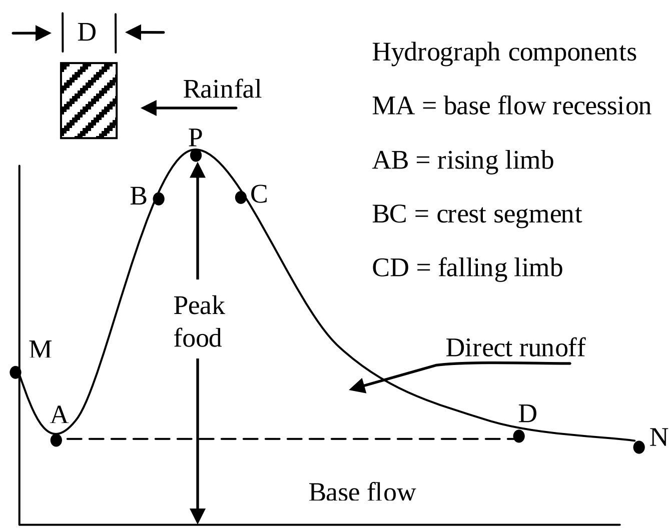

Figure 3.2 Elements of hydrograph

3.2.1 Components of hydrograph

The essential components of a hydrograph are:

1) The rising limb
2) The crest segment, and
3) The recession limb

Rising limb

The shape of the rising limb depends mainly on the duration and the intensity distribution of rainfall and to same extent on the antecedent condition and the shape of the time area diagram of the basin.

Crest Segment

The peak discharge, included in the crest segment, represents the highest concentration of runoff from the basin. It occurs usually at a certain time after the rainfall has ended and this time depends on the serial distribution of rainfall. The point of inflection at the end of crest segment commonly assumed to mark the time at which surface inflow to the channel system or the overload flow cases.

Recession Limb

The recession limb extends from the point of inflection at the end of the crest segment to the commencement of the natural groundwater flow. It represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph. The point of inflection represents the condition of maximum storage. Since the depletion of storage takes place after the cessation of rainfall, the shape of this part of the hydrograph is independent of storm characteristics sand depends entirely on the catchment characteristics.

3.2.2Factors Affecting Runoff Hydrograph

There are various factors which affects the shape of runoff hydrograph. The factors that affect the shape of the hydrograph can be broadly grouped into climatic factors and physiographic factors. Each of these two groups contains a host of factors and the important ones are listed in table 3.1. Generally, the climatic factors control the rising limb and the recession limb is independent of storm characteristics, being determined by catchment characteristics only. Many of the factors listed in table 3.1 are interdependent. Further, their effects are very varied and complicated as the factors are interdependent. As such only important effects are listed below in qualitative terms only.

Table 3.1 Factors affecting flood hydrograph

Physiographic factors	Climatic factors
1. Basin characteristic: (a) Shape (b) Size (c) Slope (d) Nature of the valley (e) Elevation (f) Drainage density 2. Infiltration characteristics: (a) Land use and cover	1. Strom characteristics: precipitation, intensity, duration, magnitude and movement of storm. 2. Initial loss 3. Evapotranspiration

(b) Soil type and geological conditions 3. Channel characteristics: cross-section, roughness and storage capacity	

3.2.3 Unit Hydrograph

- To predict the flood hydrograph from a known storm in a catchment, one of the method is the use of unit hydrograph concept.
- The unit hydrograph of a drainage basin is defined as a hydrograph of surface runoff resulting from 1 cm of effective rainfall applied uniformly over the basin area at a uniform rate during, a specified period of time for example 6 hour unit hydrograph or 12 hour unit hydrograph.
- A D-hr unit hydrograph will have an effective rainfall intensity of 1/D cm/hr.
- The effective rainfall intensity means the rainfall which will produce only runoff. In the D-hr unit hydrograph, D should not be more than time of concentration for the catchment.
- Volume of water contained inside the unit hydrograph (i.e. area of unit hydrograph) is equal to (1 cm × catchment area).
- Unit hydrograph can be used in extension of flood-flow records based on rainfall records and development of flood forecasting and warning systems based on the rainfall.

  Limitations
- Unit hydrograph cannot be used for catchment area greater than 5000 km^2 because similar rainfall distribution from storm to storm over a large area is rare.
- Unit hydrograph cannot be used for catchment <200 hr or 2 km^2.
- Precipitation must be from rainfall only. Snow-melt runoff cannot be satisfactorily represented by unit hydrograph.
- The catchment should not have unusually large storage in terms of large flood bank storage etc. Which effect the linear relationship between storage and discharge.
- If the precipitation is non-uniform, unit hydrograph cannot be expected to give good results.

3.3 Runoff characteristics of stream

On the basis of hydrograph studies a stream can be classified as

1) Perennial Stream 2) Intermittent Stream 3) Ephemeral Stream

1) Perennial stream : It is the one which always carries some flow throughout the year. Even during dry seasons, the water will be above the bed of the stream. Thus, considerable amount of the ground water flow during non-precipitation period.

2) Intermittent Stream : Intermittent stream has limited contribution from the groundwater. Stream remains dry for most part of the dry month. Base flow occurs significantly during wet season. During dry seasons the water table drops to a level lower than that of the stream bed.

3) Ephemeral Stream : An ephemeral stream is one which does not have any base-flow contribution. The stream becomes dry soon after the end of storm flow. An ephermeral stream does not have any well-defined channel. Most river in arid stones does not have any well-defined channel.

3.4 Flood definition

"A flood is an unusually high stage in river – normally the level at which the river over flows its banks and inundates adjoining area" (subramanya, 1997). Generally flood is a sudden overflow of water due to overtopped or levee break of river. It may be seasonal due to intensive rainfall, influenced by dam discharge or due to storm surge in the coastal area.

i) **Single storm flood:** This type of event is common in Indian scenario along the major river channels.

ii) **Multiple event flood:** Due to bad weather or several close storm events, this type of flood occurs is a common event in the Ganges-Brahmaputra basin during south-west monsoon season in India. During the second and onward events with less time span, almost total amount of precipitation comes to the channel due to less or no initial loss in the watershed.

3.5 Peak Discharge and its estimation

It is the maximum discharge or runoff collected at the outlet of the catchment area for a particular design period.

- Various methods to calculate peak discharge for the catchment area
  - Empirical method
    - 1. Dickens formula
    - 2. Ryves formula
    - 3. Intensity duration frequency relations
  - Rational method
  - Unit hydrograph method
  - Flood frequency studies

3.5.1 Empirical formula

The empirical formulae used for the estimation of the flood peak are essentially regional formulae based on statistical correlation of the observed peak and important catchment properties. To simplify the form of the equation, only a few of the many parameters affecting the flood peak are used. Generally formulae use the catchment area as a parameter affecting the flood peak and the most of them neglect the flood frequency and intensity of rainfall as a parameter. In view of these, the empirical formulae are applicable only in the region from which they were developed and when applied to other areas they can at best give approximate values.

The maximum flood discharge Q_p from a catchment area A is

$$Q_p = f(A) \qquad \text{(Eq. 3.1)}$$

While there are a vast number of formulae of this kind proposed for various parts of the world, only a few popular formulae used in various parts of India are given below

DICKENS FORMULA

$$Q_p = C_D A^{3/4} \qquad \text{(Eq. 3.2)}$$

Where,

Q_p = maximum flood discharge (m^3/s)

A = catchment area (km^2)

C_D = Dickens constant with value between 6 to 30

Dickens formula is used in the central and northern parts of the country. Some guidelines for selecting the value of C_D are mentioned in table 3.2.

Table 3.2 Guidelines in selecting the value of C_D(Subramanya, k)

	Value of C_D
North-Indian plains	6
North-Indian hilly regions	11-14
Central India	14-28
Coastal Andhra and Orissa	22-28

RYVES FORMULA

$$Q_p = C_R A^{2/3} \qquad \text{(Eq. 3.3)}$$

Where,

Q_p= maximum flood discharge (m^3/s)

A = catchment area (km^2)

C_R = Ryves coefficient

This formula originally developed for the Tamil Nadu region, is in use in Tamil Nadu and parts of Karnataka and Andra Pradesh. The values of C_R recommended by Ryves for use are:

C_R = 6.8 for areas within 80 km from the east coast

= 8.5 for areas which are 80-160 km from the east coast

= 10.2 for limited areas near hills.

3.5.2 Rational method

The method to give the peak flow Q_p is:

$$Q_p = C\ i\ A \qquad \text{(Eq. 3.4)}$$

Where C is the coefficient of runoff (dependent on catchment characteristics), i is the intensity of rainfall for time period equivalent to time of concentration t_c which is the time required for rain falling at the farthest point of the catchment to flow to the measuring point of the river and A is the area of catchment.

Peak discharge depends upon the amount of direct surface runoff.

Direct surface runoff is a part of total amount precipitation

Abstraction from Total precipitation are-

- Evaporation
- Evapotranspiration
- Interception losses
- Depression Storage
- Infiltration-

  Factors affecting infiltration-

  Infiltration capacity

  Rainfall characteristics

  Characteristics of land cover

  Characteristics of infiltratory water

Depending on these factors the coefficient of runoff is decided. Table 3.3 shown lists the values of coefficient of runoff for different various land use and land cover.

Table 3.3 Coefficient of runoff, C for different areas (Subramanya,K)

Types of area	Values of C
A. Urban area (P = 0.05 to 0.10)	
Lawns: Sandy-soil, flat, 2%	0.05-0.10
Sandy-soil, steep, 7%	0.15-0.20
Heavy-soil, average, 2.7%	0.18-0.22
Residential areas:	
Single family areas	0.30-0.50
Multi units, attached	0.60-0.75
Industrial:	
Light	0.50-0.80
Heavy	0.60-0.90
Streets	0.70-0.95
B. Agriculture Area	
Flat: Tight clay; cultivated	0.50
woodland	0.40
Sandy loam; cultivated	0.20
woodland	0.10
Hilly: Tight clay; cultivated	0.70
woodland	0.60
Sandy loam; cultivated	0.40
Woodland	0.30

3.5.3 Peak Discharge calculation as per IRC SP 13:2011

IRC SP 13:2011 modifies the rational formula considering various factors. IRC SP 13:2011 recommends various relationships for the calculation of important parameters like time of concentration, intensity of the rainfall, duration of storm.

Relation between intensity and duration of the storm is given for an individual storm, if F cm of rain falls in T hours and intensity may not be uniform for the whole dutaion of storm as shown in figure 3.3, then over the whole interval of time T, the mean intensity will be given as F/T cm per hour. In the given storm for a smaller time interval less then T it is required to calculate intensity and the intensity is not uniform throughout the storm the mean intensity reckoned over the time interval t (smaller then T the total duration of storm) which will be higher than the mean intensity taken over the whole period. As per the code the methodology to calculate the intensity I for the duration t is given by

$$I = \left(\frac{T+1}{t+1}\right)\frac{F}{T} \qquad \text{(Eq. 3.5)}$$

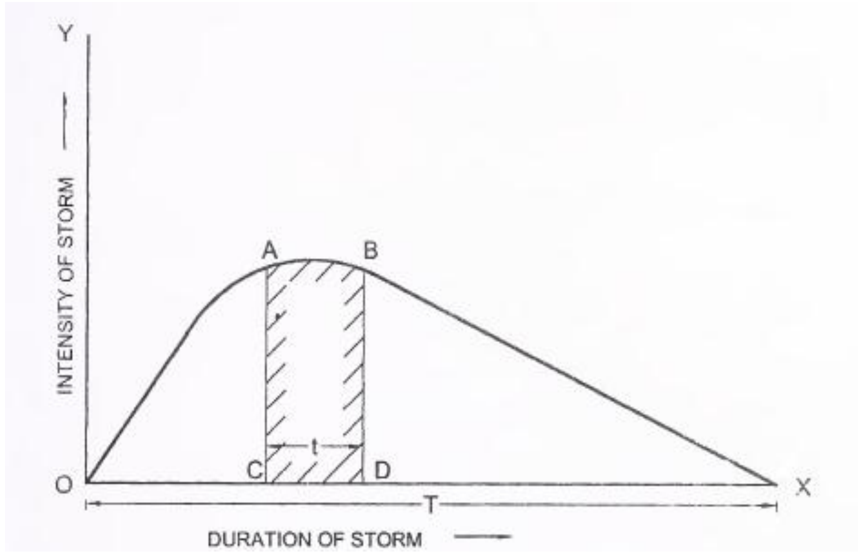

Figure 3.3 Intensity duration relationship (IRC SP 13:2011)

If it is required to calculate the rainfall for one hour duration that is the intensity

Then t = 1 hour, I = I_o (Intensity of rainfall of one hour duration).

$$I_o = \frac{F\left(1+\frac{1}{T}\right)}{2} \quad \text{(Eq. 3.6)}$$

Time of concentration

As per codal provisions the time of concentration for the catchment depends

(1) The distance from the critical point to the structure

(2) The average velocity of flow

As per the code time of concentration is given by

$$t_c = \left(.87 \times \frac{L^3}{H}\right)^{.385} \quad \text{(Eq. 3.7)}$$

t_c = the concentration time in hours

L = the distance from the critical point to the structure in km.

H = the fall in level from the critical point to the structure in m.

As the intensity of the storm increase as the duration of storm decreases so it is safe to take shorter duration of storm. But the duration must be equal to the time of concentration because at the time of concentration whole area starts in contribution to the flow so the duration of storm is equal to the time of concentration. Hence intensity is calculated corresponding to the time of concentration.

The intensity of storm(I_c) corresponding to the time of concentration given by

$t = t_c$

$$I_c = \frac{\left(\frac{T+1}{t+1}\right)F}{T} \qquad \text{(Eq. 3.8)}$$

Runoff at the outlet given as per the code,

For a precipitation of rainfall intensity I_c over an catchment area of A hectares runoff is given by

$$Q_{peak} = .028PAI_c \qquad \text{(Eq. 3.9)}$$

Where,

Q, max. run-off in m^3/s.

A, area of catchment in hectares.

I_c, critical intensity of rainfall in cm per hour.

P, co-efficient of run-off for the catchment characteristics.

The discharge also depends on the position of the rain gauge. Rain gauge stations provide point rainfall and it required to obtain the rainfall for the overall catchment area so a factor '*f*' is introduced in the calculation of discharge which takes into its consideration. Figure 3.4 shows the relationship between the factor *f* and catchment area prescribed by IRC SP 13:2011

The runoff is given by

$$Q_{peak} = .028fPAI_c \qquad \text{(Eq. 3.10)}$$

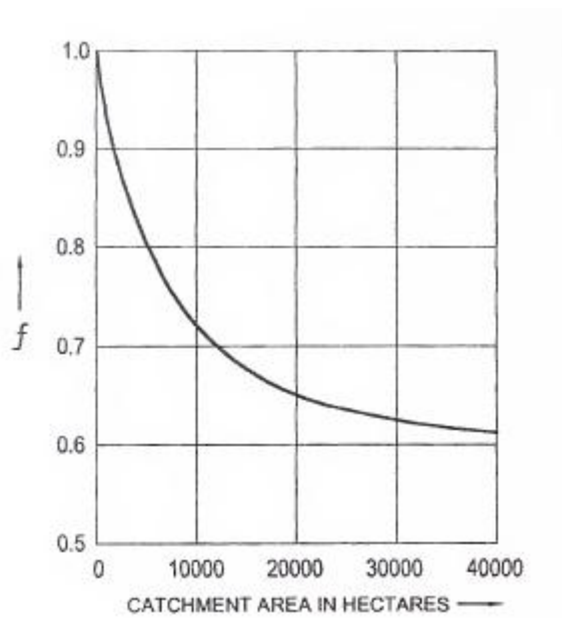

Figure 3.4 Graphical relationship of point to areal rainfall reduction factor vs. catchment area (IRC SP 13:2011).

3.5.4 Unit hydrograph method

The unit hydrograph can be used to predict the peak-flood hydrograph if the rainfall producing the flood. For design purposes, extreme rainfall situations are used to obtain the design storm. The hydrograph of the catchment is then operated upon by the design storm to generate the flood hydrograph.

In this method maximum effective rainfall (input) is estimated as per the storm duration and is applied to the unit hydrograph (transfer function) to obtain a design flood hydrograph (output).

Synthetic unit hydrograph

Unit hydrographs can be derived if rainfall and runoff records are available for the basin under consideration. But there are many basins, which are not gauged and for which unit-graph may be required. Hence some method of deriving unit hydrograph for un-gauged basins is necessary.

This is usually done by relating the selected basin physiographic characteristics such length of stream bed slope etc. to the unit hydrograph shape. Once such relations are established between the basin parameters and unit hydrograph parameters for the basins having sufficient data, the same relations are applied to obtain the unit hydrograph of un-gauged basins in the same hydro meteorologically homogeneous area. The unit hydrograph thus obtained is known as Synthetic unit hydrograph.

For preparation of Synthetic unit hydrograph of design storm duration for various regions in India a detailed study of hydro meteorological characteristics of different climatic zones of India was carried out by committee of engineers under the chairmanship of Dr. A.N. Khosla to develop a rational approach for the determination of flood discharge. The committee prepared flood estimation reports for different climatic subzones of India which are hydro meteorologically similar. The reports describe full methodology for the preparation of synthetic unit hydrograph and estimation of peak discharge for small and medium catchments. The country has been divided into 26 hydro meteorologically homogeneous sub-zones as shown in figure 3.5 and the report publication of hydro meteorological data for 21 subzones has been done. Table 3.4 lists these 21 subzones. The study area falls in the region of subzone 1(e) (Upper Indo-Ganga Plains subzone). This subzone is shaded in figure 3.5 in political map of India.

In the report, for the preparation of synthetic unit hydrograph various empirical relationships between physiographic parameters of catchment such as length and slope of stream etc. and unit graph parameters such as discharge per unit area, time of concentration etc. are mentioned. Figure 3.6 shows the synthetic unit hydrograph and its various parameters as described in CWC flood estimation reports. Table 3.5 list various empirical relationships between physiographic parameters and the unit graph parameters of the catchment area for the preparation of synthetic unit hydrograph for sub zone 1(e).

Figure 3.5 Image of political map of India divided into 26 hydrometeorological similar subzones(CWC flood estimation report).Study area fall in the region shaded in the figure.

Table 3.4 Classification of subzones

S.No.	Name of Sub-Zone	Sub-Zone No.
1.	ChambalUpper Indo-Ganga Plains sub--zone sub-zone	1(b)
2.	Betwa sub-zone	1(c)
3.	Sone sub-zone	1(d)
4.	Upper Indo-Ganga Plains sub-zone	1(e)
5.	Middle Ganga Plains sub-zone	1(f)
6.	Lower Gangetic Plains sub-zone	1(g)
7.	North Brahmaputra basin sub-zone	2(a)
8.	South Brahmaputra basinsub-zone	2(b)
9.	Mahi and Sabarmati sub-zone	3(a)
10.	Lower Narmada and Tapi sub-zone	3(b)
11.	Upper Narmada and Tapi sub-zone	3 (c)
12.	Mahanadi sub-zone	3 (d)
13.	Upper Godavari sub-zone	3 (e)
14.	Lower Godavari sub-zone	3 (f)
15.	Krishna and Pannar sub-zone	3 (h)
16.	Kaveri river sub-zone	3 (i)
17	Eastern Coast sub-zones	4(a), 4 (b) and 4 (c)
18.		
19.		
20.	West coast region sub-zones	5 (a) and 5 (b)
21.		

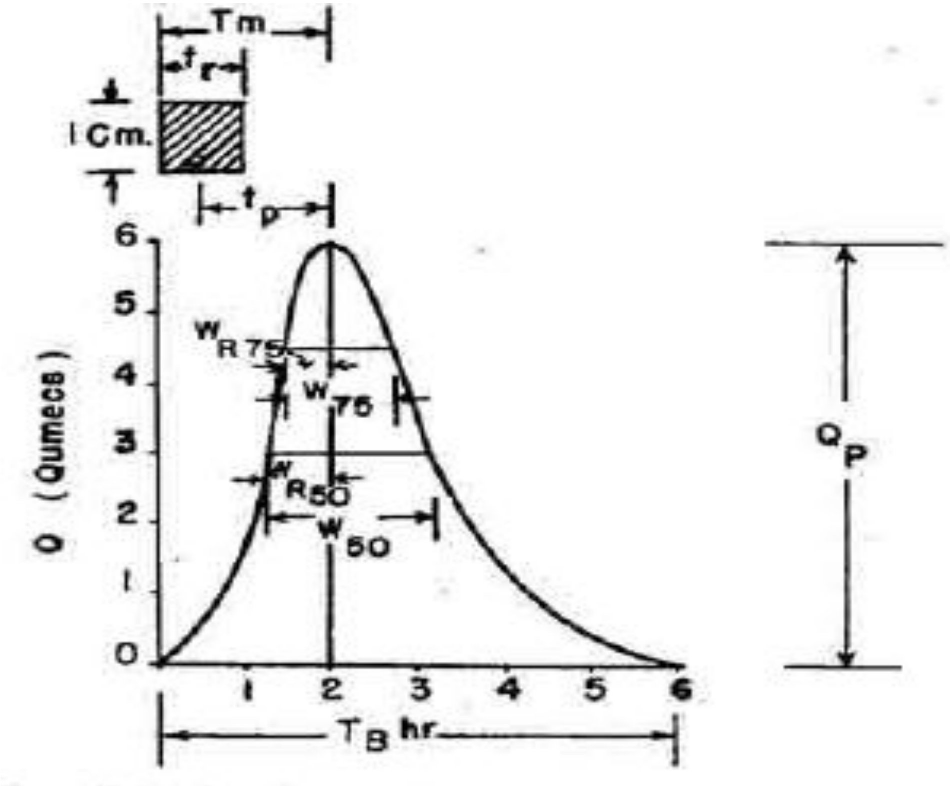

Figure 3.6 Synthetic unit hydrograph and its parameters(CWC flood estimation report).

U.G = Unit Graph

t_r = Unit Rainfall Duration adopted in a specific study (hr.)

T_m = Time from the start of rise to the peak of the Unit graph (hr.)

Q_p = Peak Discharge of unit hydrograph (cumecs)

t_p = Time from the center of the rainfall excess duration to the unit graph peak (hr.)

W_{50} = Width of the unit graph measured at 50% of peak discharge ordinate (hr.)

W_{75} = Width of the unit graph measured at 75% of peak discharge ordinate (hr.)

W_{R50} = Width of the rising limb of unit graph measured at 50% of peak discharge ordinate (hr.)

W_{R75} = Width of the rising limb of unit graph measured at 75% of peak discharge ordinate (hr.)

T_B = Base width of the unit graph

A = Catchment Area (Sq. km.)

q = Q_p/A, Cumecs per sq.km.

$t_p + t_r/2$ = time of concentration = minimum duaration of storm to attain peak discharge

Table 3.5 Relationship between physiographic and unitgraph parameters(for subzone 1(e), CWC flood estimation report for Upper Indo-Ganga Plains)

$q=\frac{2.030}{\left(\frac{L}{\sqrt{S}}\right)^{.649}}$	(Eq. 3.11)
$t_p=\frac{1.858}{q^{1.038}}$	(Eq. 3.12)
$W_{50}=\frac{2.217}{q^{.99}}$	(Eq. 3.13)
$W_{75}=\frac{1.477}{q^{.876}}$	(Eq. 3.14)
$W_{R50}=\frac{0.812}{q^{.907}}$	(Eq. 3.15)
$W_{R75}=\frac{0.606}{q^{.791}}$	(Eq. 3.16)
$T_B=7.744(t^{.779})$	(Eq. 3.17)

3.5.5 Calculation of peak discharge as per relationship prescribed by CWC for design flood for 50 year return period

The magnitude of the peak discharge for a flood of return period 50 year in cumecs for small and medium catchments as per the Central water commission flood estimation report 1(e) (Upper Indo Ganga plains) is

$$Q_{50}= K \times A \times R \times \frac{S^{.324}}{L^{.649}} \qquad \text{(Eq. 3.18)}$$

Where,

Q_{50}= Peak discharge from flood of return period 50 year.

A = Catchment area in sq.km.

R = 50-year return period point rainfall data of given storm duration in cm.

S = Equivalent stream slope m/km.

L = Length of the longest stream in km.

k = A coefficient, the value of which is dependent on the catchment. Table 3.6 shows the relationship of catchment area and the coefficient as per CWC flood estimation report for Sub zone 1(e)

Table 3.6 Catchment area and corresponding value of coefficient

Catchment area. (sq. Km.)	K. Value
25	1.70
100	1.51
500	1.32
1000	1.15
2500	1.00

The K value corresponding to the catchment area under study falling in between the above range of catchment areas is interpolated linearly between the K values corresponding to that particular range of catchment areas.

3.6 Computation of discharge in a stream as per Manning's equation (area velocity method)

The discharge in the stream as per Manning equation,

$$Q = \frac{AR^{\left(\frac{2}{3}\right)}S^{\left(\frac{1}{2}\right)}}{n} \qquad \text{(Eq. 3.19)}$$

A, Area of cross section

R, the hydraulic mean radius

S, Bed slope of the stream.

n, Rugosity or roughness co-efficient

The hydraulic radius is given by

R = Area of cross-section/wetted perimeter.

3.7 High Flood Level

It is required to determine the design high flood level corresponding to adopted design flood for the design bridge and the cross drainage structures. The elevation is of very important aspect in the analysis of foundation, scour, freeboard, formation levels, hydraulic forces.

The high flood level (HFL) corresponding to design flood of appropriate return period is generally established with help of stage(depth of flow) discharge relationships which are represented by stage vs. discharge curves for a stream at the required point of study. Stage discharge relationships depend on various catchment characteristics such cross sectional area, shape, slope and roughness of bed and banks. The gauge discharge curves can be extrapolated if necessary. The stage discharge readings can be obtained through,

- Observation of gauges and discharges covering satisfactorily the lower and upper elevation ranges.
- In absence of gauge discharge readings gauge discharge relationship is prepared by area velocity method (discharge calculation through manning's equation) using the stream slope and nature of cross section.

3.8 Case study

3.8.1 Road crossings in Belize City

This analysis was published in Caribbean Handbook on Risk information management. The overall objective of the analysis is to develop a methodology for the design of a road where it crosses water stream, particularly in locations that are prone to floods.

As flow naturally varies in a watercourse due to various factors such as seasons, rainfall conditions and land use etc., which must be allowed in order to design road crossing for very high flow conditions. If the crossing cannot allow all water through the structure without holding some back then water will build up behind the structure and create flooding problems, including possibly overtopping the road and eroding the banks/road support. Figure 3.7 shows areas affected and damages of road due to floods. So it was required to know the flood of extreme magnitude in order to design a crossing of sufficient size.

The areas under consideration are the critical locations for stream crossing the roads are two sites in Belize (Central America). The first location is the George Price Highway crossing of Mount Pleasant Creek crossing near Belmopan (steep terrain) and the second is the Ladyville area near the west coast and Belize City (flat terrain).

Figure 3.7 Image showing damages caused due to floods

Mount Pleasant Creek

George Price Highway crosses the Mount Pleasant Creek the flow in the creek annually exceeds the capacity of the culvert underneath the roads, causing the water to back up behind the road embankment and then flow over the top of the road. This road is also the only evacuation route from Belize City and therefore has a high importance for the country. For the road crossing, the culvert's hydraulic capacity was observed insufficient during floods. Figure 3.8 shows the google earth image of the area in which the George Price Highway crosses the Mount Pleasant Creek. The blue lines in the image indicate the streams in the region and the yellow line indicates the highway passing through the region.

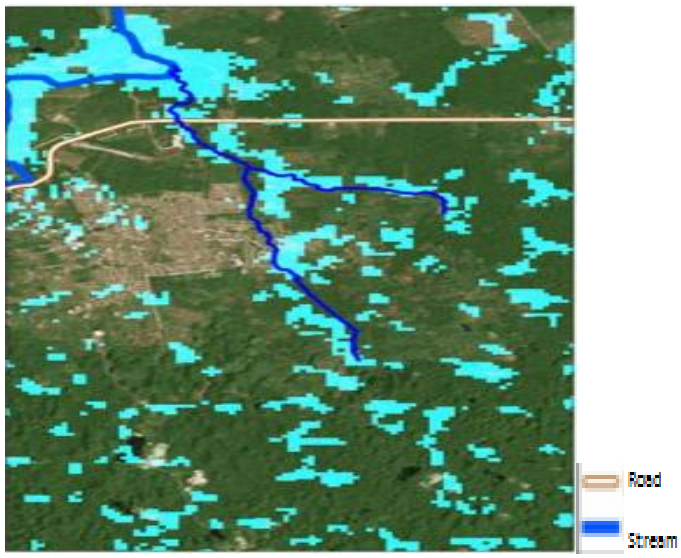

Figure 3.8 Google earth image Mount Pleasant Creek with road and stream overlaid

Data used for analysis was 90 m resolution SRTM DEM and derived contours from DEM. Measured flow data for the water course were not available. However from observation of history of flooding with pictures and eyewitness accounted for limiting culvert capacity.

In methodology the water network and highway crossing was located in GIS. The catchment was delineated from the SRTM DEM data and the longest flow path was located. There were no rainfall intensity duration frequency curves for the region. So rainfall intensity data of Miami which has similar conditions to Belize was used. From the GIS the longest flow path in the catchment observed was 12 km and using the Kirpich formulae, time of concentration was estimated for the catchment. From the corresponding time concentration the rainfall intensity was derived. The rational method was then used to calculate the design flow from culverts. From the design discharge the water levels were calculated. Finally it was found that the culvert was undersized for even the peak annual expected rainfall and may be exacerbated by future growth of Belmopan. Annual design return period was chosen in such a way that the culvert can pass the annual high flow.

Flat Terrain - Ladyville area

The coastal Ladyville area, just to the west of Belize City, illustrates another example of crossing a river as it approaches the sea, where floodplains are wide and flat. The terrain is very flat, with very little in the way of drainage slope to drive flow through culverts. The elevation is between zero and five metres above sea level, and therefore is subject to coastal storm surges as well as swampy conditions, particularly south of the Belize River. Here water network was observed to have poor drainage characteristics resulting in standing water for long periods. Figure 3.9 shows google earth image of the Ladyville area with over of blue line showing Belize river and yellow lines showing roads.Crossing of road and river is marked as red in figure. Green dots on the yellow line denote the positions of culverts along the roads.

Figure 3.9 Google earth image of Ladyville area road and stream overlaid.

Building the road above flood levels results in road embankment obstructing natural drainage which causes prolonged standing water which affects the road substructure even if a road surface is not flooded. Also during flood events the river in which the run-off flows is also flowing at high flood level, which therefore extends the length of the flood event.

Under hydrological study of the natural water systems in this area gauged flow analysis is done which more sophisticated than Rational method. Next a detailed hydraulic assessment of the crossing was carried out. In this case, the location area is very flat and is low lying which cause backwater effects at the crossing location due which there is increase in the water levels upstream at the crossing. Next step was to design the bridge of sufficient hydraulic capacity to pass the design flow while not disturbing the superstructure. The whole road section was elevated above the water levels expected in the river and floodplain in this flat area to remain passable because of backwater effects. A multiple span bridge over the whole floodplain was proposed, allowing water to pass freely under the road for the entire width of the floodplain.

3.8.2 Waterway design for railway bridges (Chennai-Gudur Section, Southern railways)

This analysis was carried out by Mr. D. Stanley, Deputy executive engineer South Central railways division, India and Mr. S. Shanmughom, Executive engineer, Southern railways division, India.

Heavy rains and over flowing of water above the danger level at railway bridges in Chennai – Gudur section caused the cancellation / partial cancellation / diversion of various trains. This is another case where during floods due to lack of proper design the breach of railways tracks and bridges happened. Figure 3.10 area of water way beneath the railway track capable of receding water during floods of specific return period.

Figure 3.10 Railway bridge opening (water way)

Due to flash floods occurred in October 2001(rainfall 245mm, 24 hour), at Chennai Central Gudur section(114-118 km) water levels of surface runoff touched bottom of the girder, crossed danger level, at various bridges (Latitude = 13°57' N , Longitude 79°47' E),therefor surpassing the discharge of which the waterway was designed to recede discharge during floods. Hence redesign of the bridge section was carried out. The criterion of design flood discharge was based on catchment area. For catchment area less than 25 sq. km rational methodology as prescribed by RDSO (Research design standards organization, Indian Railways) was adopted and for catchment area greater than 25 sq. km synthetic unit hydrograph technique was used (Sub zonal reports as prescribed by Central water commission). The catchment area was delineated from the toposheet of the area. Maximum rainfall data was obtained from the iso-pluvial map of the region as published in CWC sub zonal report. The high flood level of the region was thus estimated and compared with the elevations at the bridge opening which showed raised water level above danger level and redesign of the bridge was proposed. At some of the crossings breach of embankment took place due to heavy discharge as shown in figure 3.11.Figure 3.12 shows the derailing of the train from the tracks due to subgrade erosion of embankment due to floods.

Figure 3.11 Embankment breach of railway track due to floods.

Figure 3.12 Image showing train derailed due to subgrade erosion of embankment

CHAPTER 4

STUDY AREA, DATA ACQUISITION AND METHODOLOGY

4.1 STUDY AREA

The road/rail routes nearby flood plains are susceptible to frequent floods due to which the region gets affected and consequently the mode of transportation also gets affected. The study areas lie in Roorkee and Laksar which have similar characteristics as they being situated along riverside (Solani and Ganges river) hence can be regarded as flood plains hence subjected to frequent floods.

For present study, the catchment area from where the major district road passes through Laksar-Dabki-Kalan-Gordpur-Jhabarpur-Purquaji and finally connects to NH58 towards Delhi is under consideration. This major district road covers around 30 km of the path. The spatial extent of the study area falls in UTM 43R and 44R from 29° 39' 36.62" N-29° 44' 43.6" N and 77° 51' 37.66" E-78° 01' 02.07" E. The area mainly constitutes agriculture fields with a very small amount area constituting urban area.

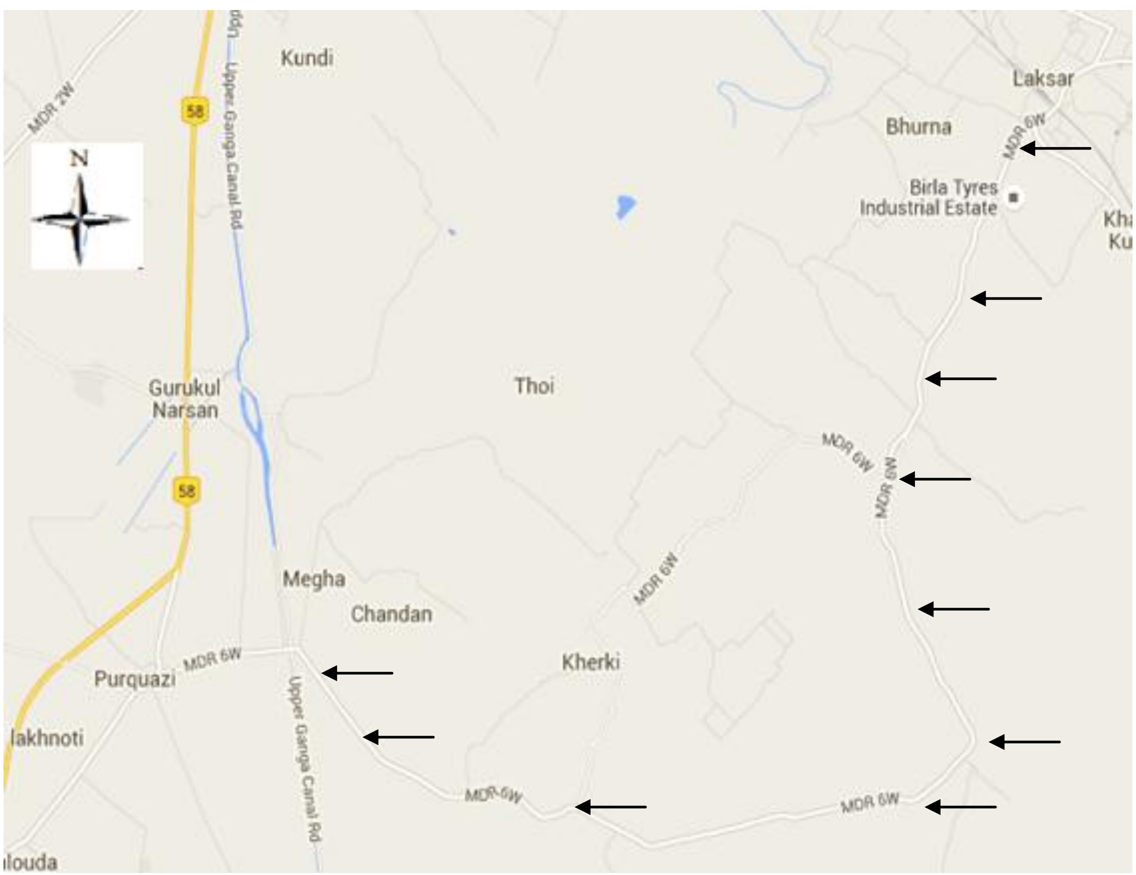

Figure 4.1 Map of the study area and the major district road passing (Laksar area)(google map).The arrows shown in the image highlight the roadway under consideration in the map.

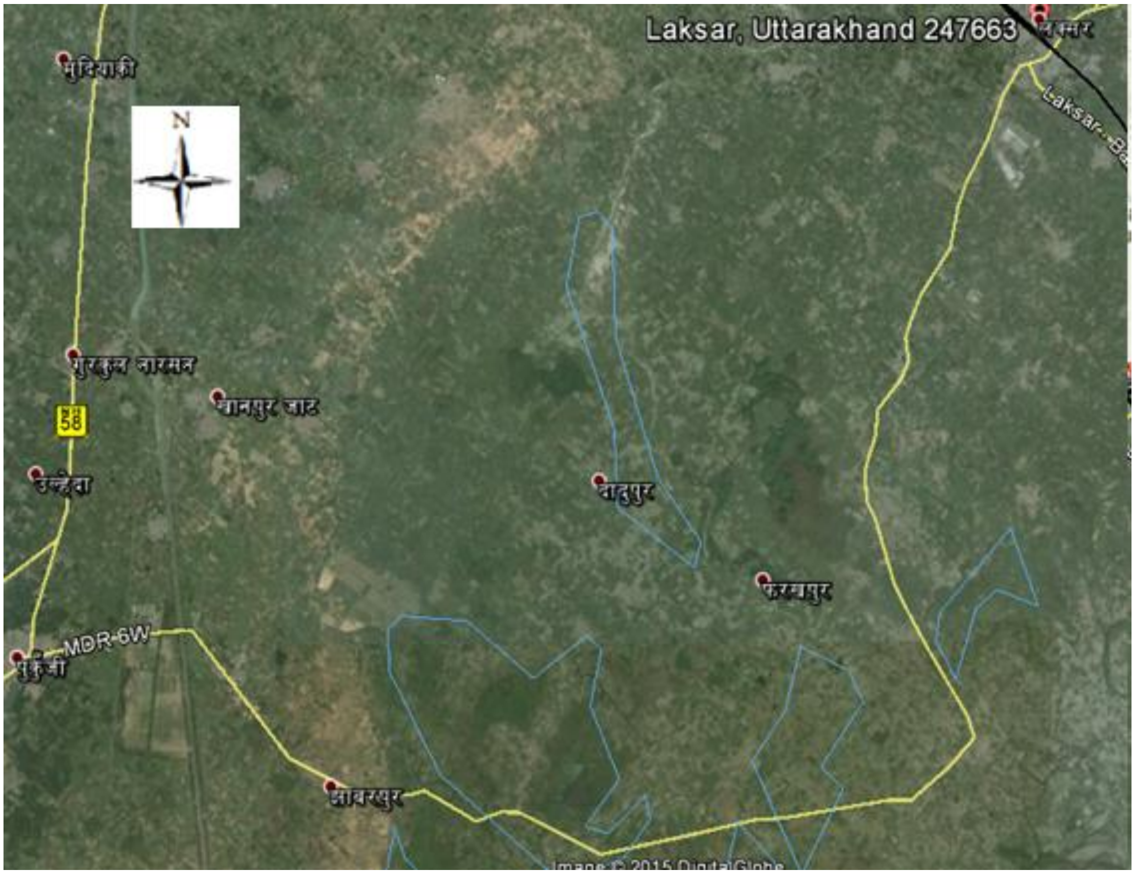

Figure 4.2 Google earth image Study Area (Laksar area). The yellow line from Laksar to Purquaji indicates the road under consideration.

Another study area is the bypass proposed to be constructed from Mangalore to Rehmadpur which diverges from NH 58 at Mangalore towards Rehmadpur and again at Rehmadpur connects to the NH58 and in between crosses Solani river.The bypass covers around 16 km of the path.The spatial extent of the study area fall in UTM 43R from 29° 48' 31" N - 29° 54' 00" N and 77° 52' 33" E - 77° 58' 04" E. The area mainly constitutes of agricultural fields and urbanized area.

Both the region falls between 280mm to 300mm isochrones of maximum rainfall in iso-pluvial maps as published by Central water commission. The temperature of the regions fall between 22.5°C - 25°C.The regions consist mainly of alluvial soil or tarai soil.

Figure 4.3 Map of the study area (NH 58 bypass)(google map)

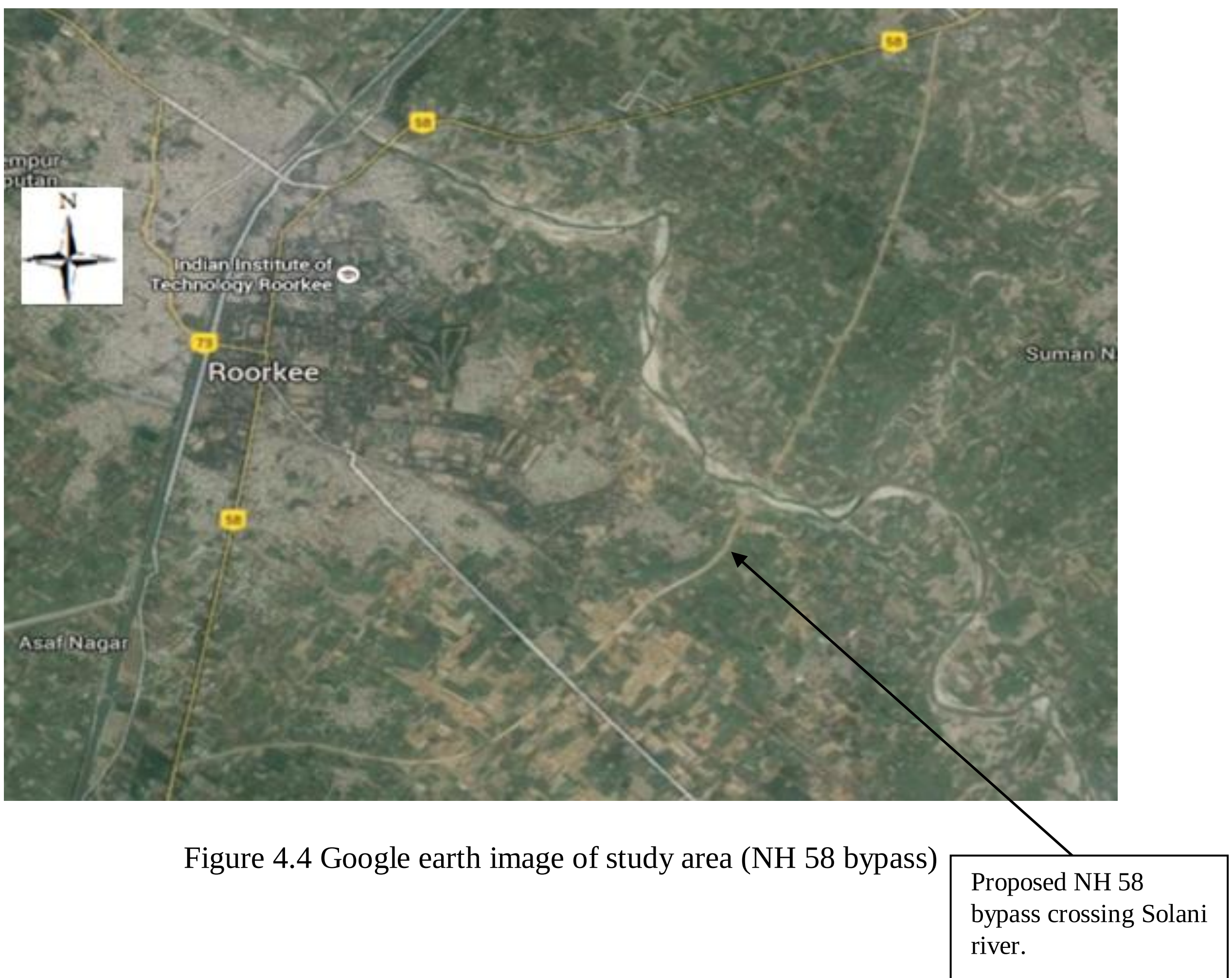

Figure 4.4 Google earth image of study area (NH 58 bypass)

4.2 Acquisition of datasets

- The elevation data of the Laksar area and NH 58 bypass drainage basin is obtained from the ASTER DEM of the region which covers the surface between 29° N-30° N and 77° E-78° E. ASTER DEM has horizontal resolution of 1 arc second (30 m). The catchment is delineated from the DEM as shown in figure 4.5 and 4.6.

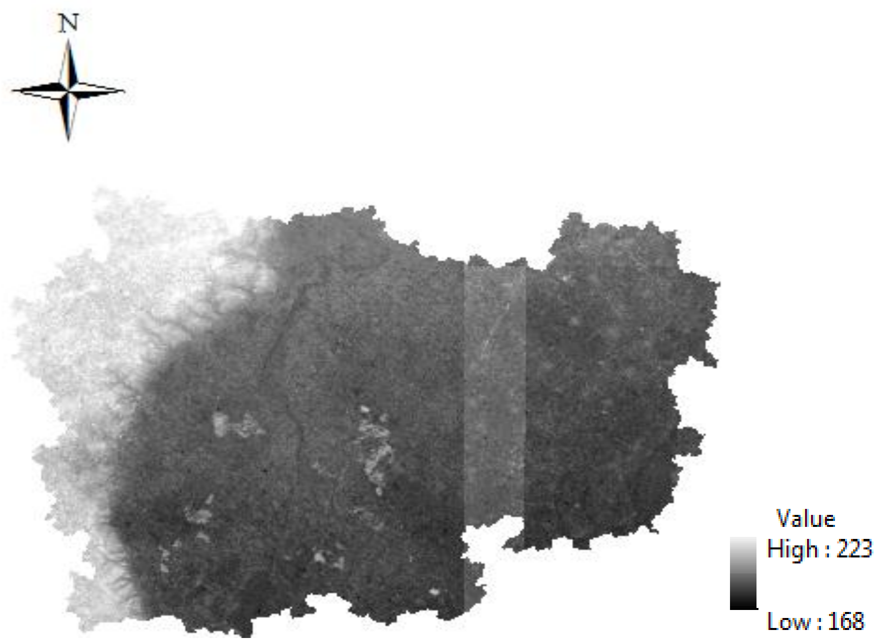

Figure 4.5 ASTER DEM of the Laksar area

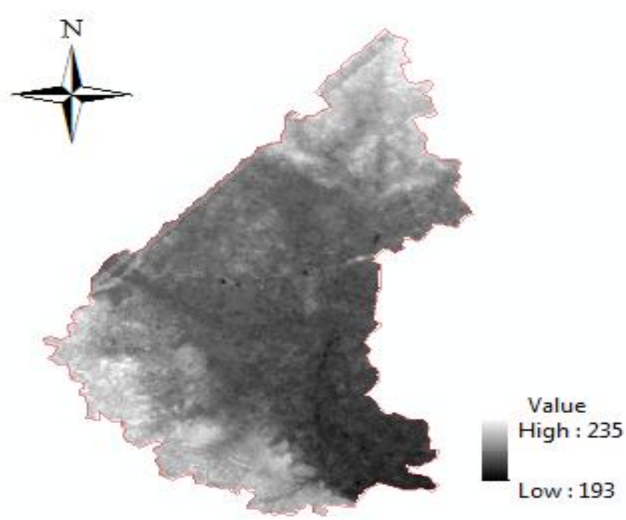

Figure 4.6 ASTER DEM of the NH 58 bypass drainage basin.

- The maximum point rainfall data for a return period of 50 year for the study area has been obtained from Indian meteorological department (IMD) website for duration of 24 hours. The same has been verified from the isopluvial maps of the region published by CWC (flood estimation report, CWC) as shown in figure 4.7.

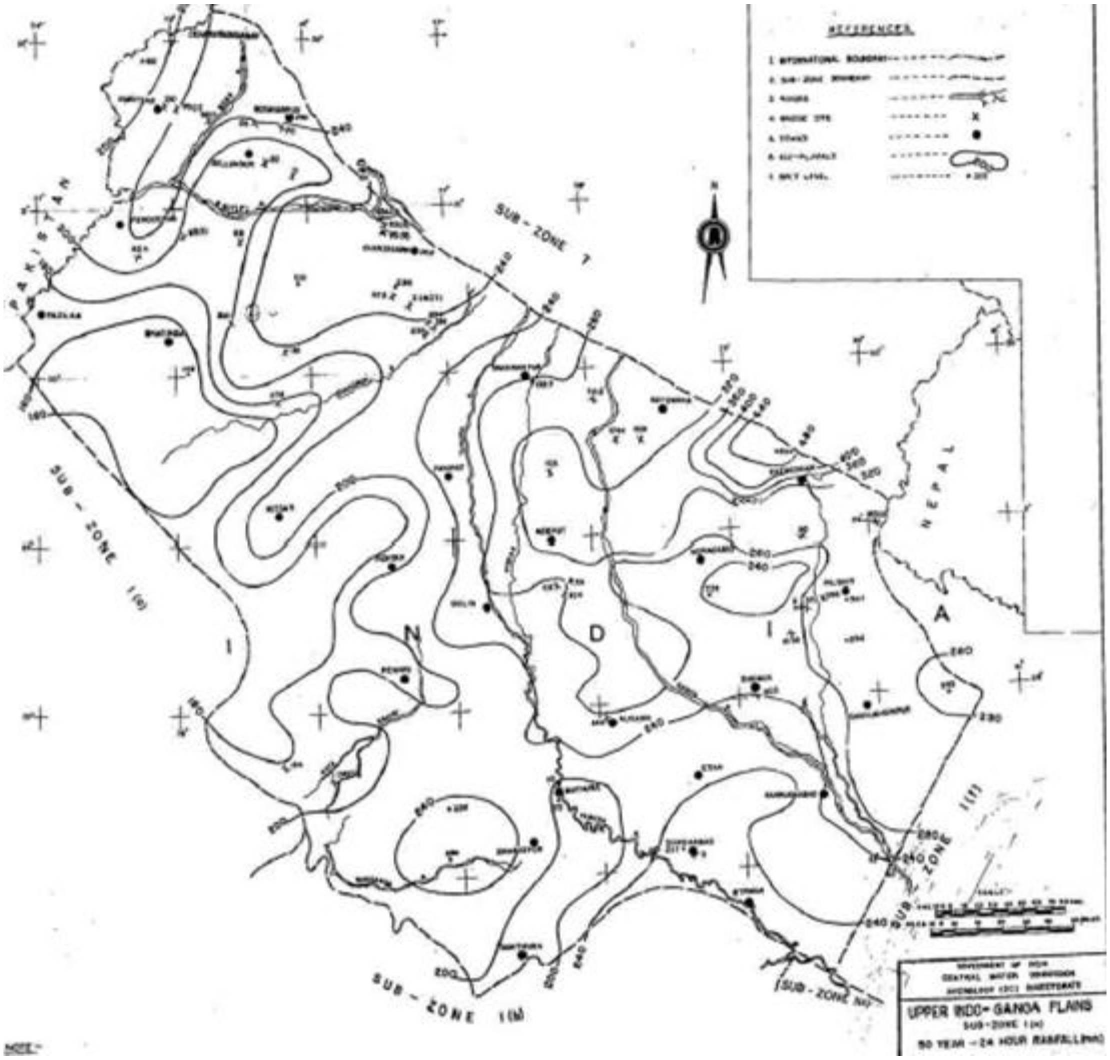

Figure 4.7 Isopluvial map of the region under consideration (CWC flood estimation report, subzone 1(e))

4.3 Methodology

- The ASTER GDEM of the whole catchment through which the road passes was processed in Arc GIS and divided into various sub catchments in GIS.
- Various information about the physiographic data for each sub catchment was extracted through DEM e.g. area for each sub-catchment lowest elevation point in the sub-catchment, slope of stream, length of stream in GIS. The same was verified from google earth.

The result hydrograph is derived from the 2 hr. unit hydrograph depending upon the duration of the storm in the catchment area.

The data of rainfall which is used to calculate peak discharge is for maximum rainfall of 24 hour duration of 50 year return period. So it is required to derive rainfall data for a particular storm event from 24 hour rainfall data.

- The 24 hour point rainfall data was converted to the point rainfall data of specific storm duration as per the graphical interrelationships obtained CWC flood estimation report as shown in figure 4.8.
- The storm duration point rainfall was converted into areal rainfall data by multiplying it with a factor as per the graphical relations in the CWC flood estimation report (figure 4.9).
- Further the rainfall in the total storm duration may not be uniform. The amount of rainfall in successive intervals of total storm duration was calculated as per the guidelines of CWC flood estimation report (figure 4.10).
- Figure 4.11 shows steps to obtain effective rainfall in the form of flow chart.

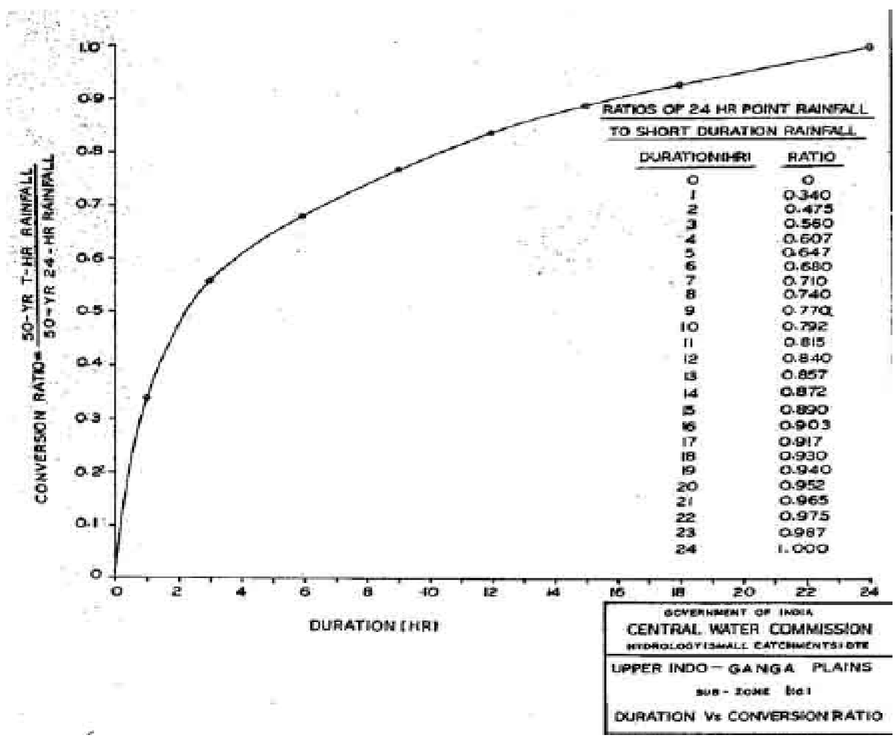

Figure 4.8 24 hour rainfall to Storm duration rainfall conversion graph of region under consideration (CWC flood estimation report, subzone 1(e))

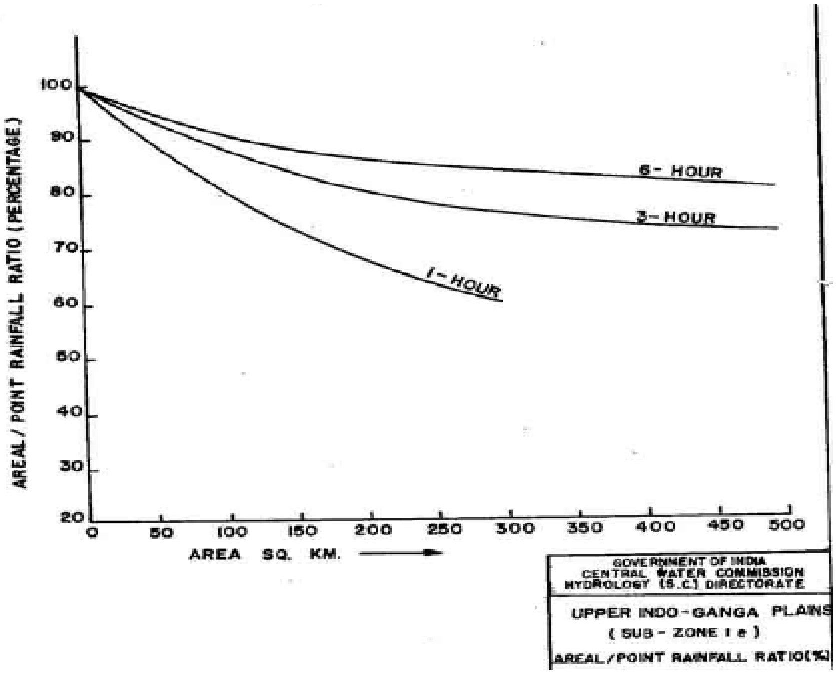

Figure 4.9 Point to areal rainfall conversion graph region (CWC flood estimation report subzone 1(e))

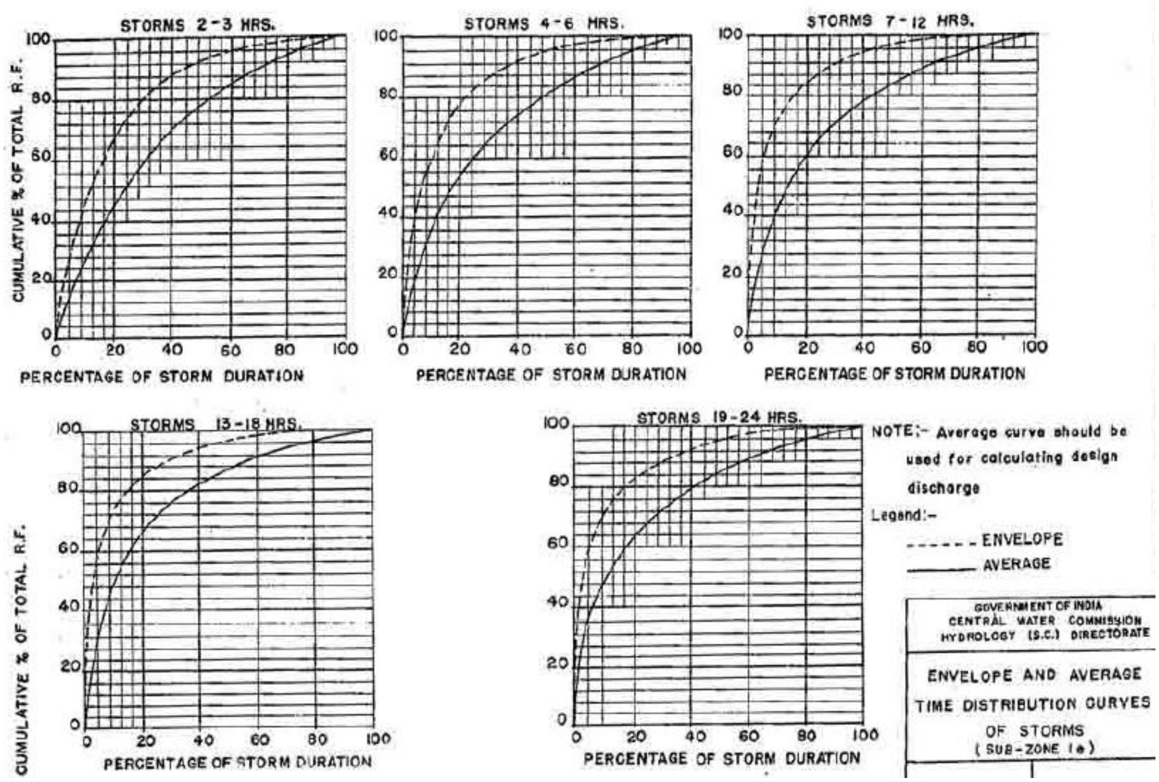

Figure 4.10 Time distribution of rainfall curves for various storm period region (CWC flood estimation report, subzone 1(e)).

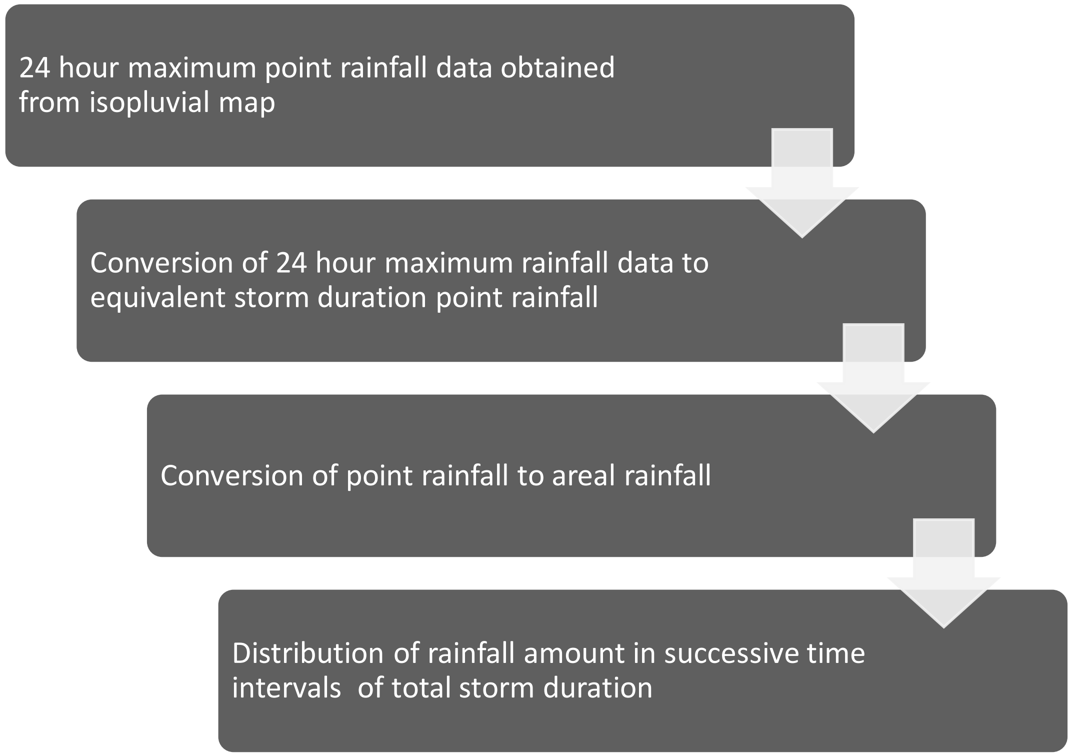

Figure 4.11 Flow chart of derivation of effective rainfall.

- Runoff calculation using synthetic unit hydrograph technique from Central water commission flood estimation reports provides methodology for derivation of synthetic unit hydrograph for 2 hour Duration.
- Unit graph parameters of the catchment area for 2 hour duration were obtained were from the empirical relationships of unit graph parameters and physiographic parameters of the sub catchment.
- The peak ordinate of unit hydrograph was obtained by multiplying the discharge per unit area by the catchment area.
- The duration of the storm was obtained from the time at which the peak discharge is achieved which is corresponding to the time of concentration.
- The unit hydrograph prepared from the calculated unit graph parameters was of unit rainfall of 2 hour duration and was further modified in order to obtain hydrograph of required storm duration
- This was achieved by adding the same unit hydrograph lagged by 2 hours in succession till the final hydrograph of required storm duration is achieved.
- The respective unit hydrographs are multiplied by the amount of rainfall in its respective time interval.
- The resultant hydrograph was obtained by the summation of ordinates of all the hydrographs obtained in succession.
- Figure 4.12 shows steps for the preparation of runoff hydrograph for any flood event using synthetic hydrograph method in the form of flow chart.

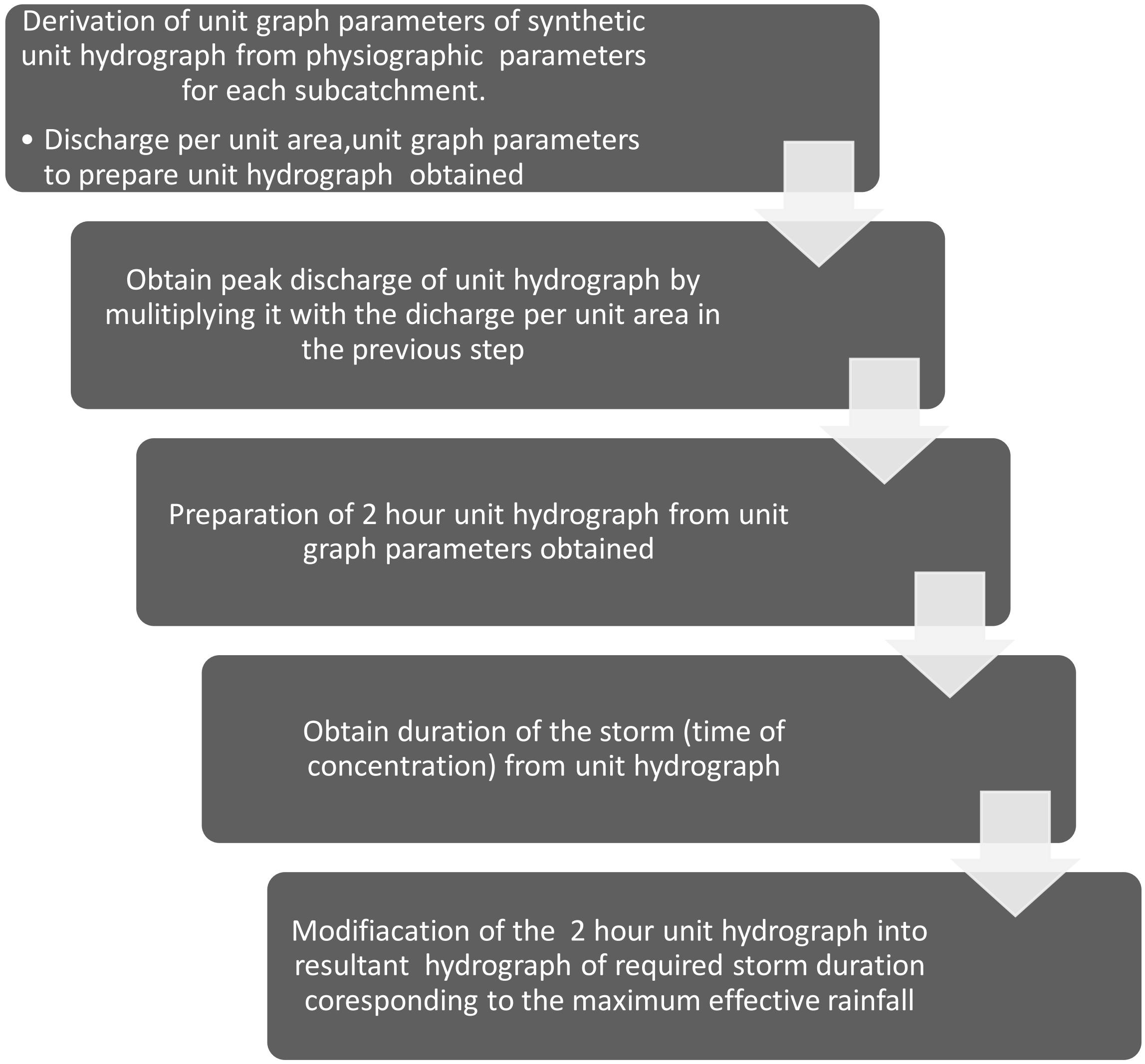

Figure 4.12.Flow chart for the preparation of synthetic run-off hydrograph.

- For peak discharge calculation as per IRC SP 13:2011, time of concentration for each sub catchment was obtained as per the empirical relationships in IRC SP 13:2011 which depends on length of longest stream and elevation difference between the highest and lowest point in the sub-catchment.
- Intensity corresponding to the time of concentration obtained as per the relationship given in IRC SP 13:2011.
- Point to areal reduction factor (*f*) was taken depending upon the catchment area as per the graphical inter relationship between *f* and catchment area mentioned in IRC SP 13:2011.
- Runoff coefficient was taken as .5 which was determined from literature considering various factors such as land use, slope of terrain. Land use was monitored using Google earth.

- For peak discharge calculation as per formula prescribed by CWC for the study area required parameter such as area of catchment, slope and length of the stream was extracted through GIS.

- K, coefficient of catchment dependent on the catchment was obtained as per guidelines of CWC flood estimation report.
- Peak discharge obtained from 3 methods were compared for each sub-catchment and the highest of the three was considered as design discharge and was used in calculation of high flood levels for the respective sub-catchment.
- From area velocity method (Manning's equation) discharge in the stream at various depths were obtained and depth corresponding the design discharge was obtained.
- Hydrodynamic parameters such as manning's coefficient were assigned as 0.026 obtained from literature for alluvial soils.
- In case of drainage basin for NH 58 bypass the inflow discharge in river Solani was added to the design discharge in order to get the total discharge at the outlet.
- The inflow discharge obtained as 2267 cumecs in the Solani river was obtained from Irrigation department website, Government of Utter Pradesh.
- Figure 4.13 shows steps to estimate regions of submergence from the calculated peak discharge in the form of flow chart.
- In figure 4.14 flow chart of overall methodology is shown.

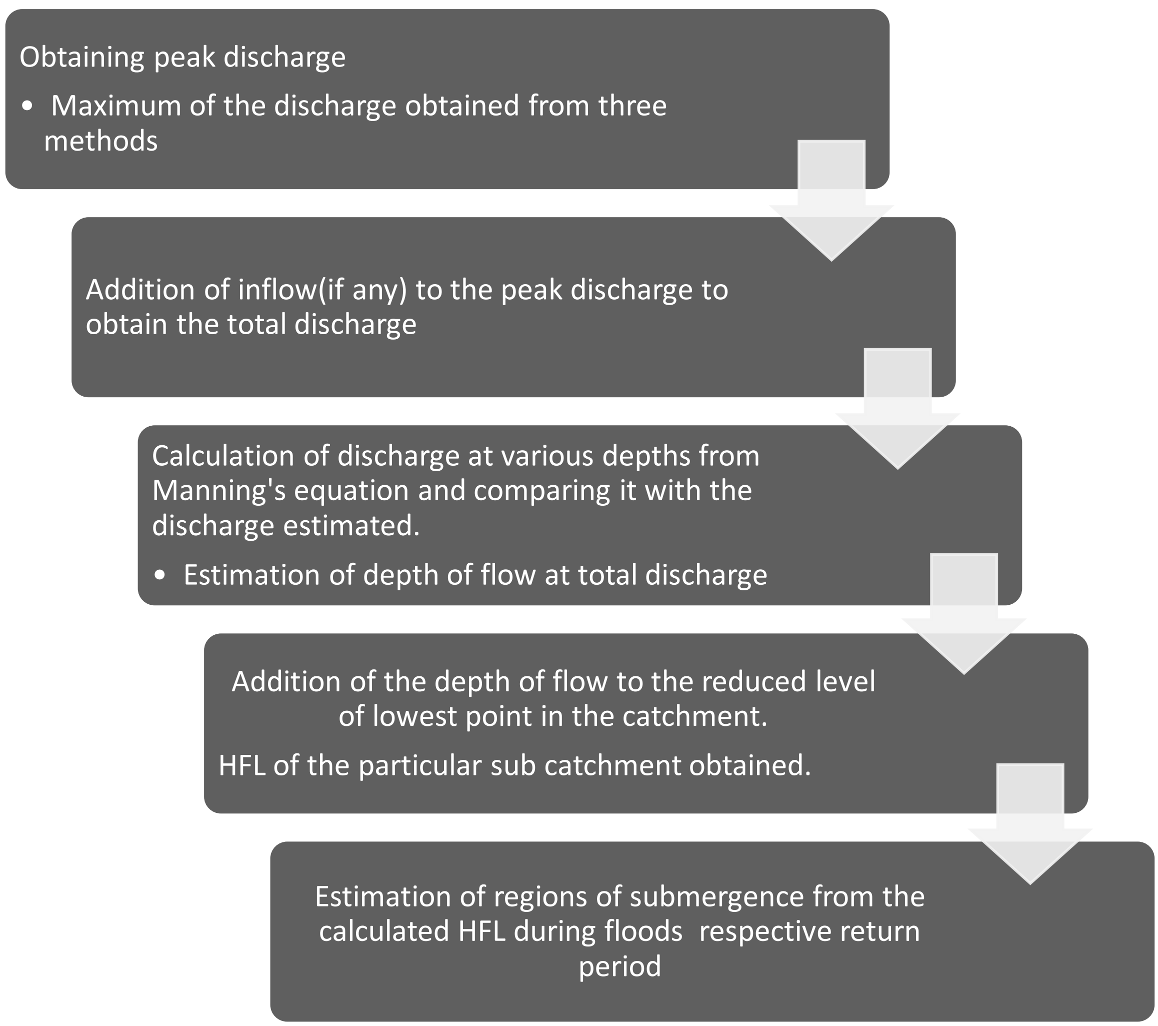

Figure 4.13 Flow chart for estimation of regions under submergence

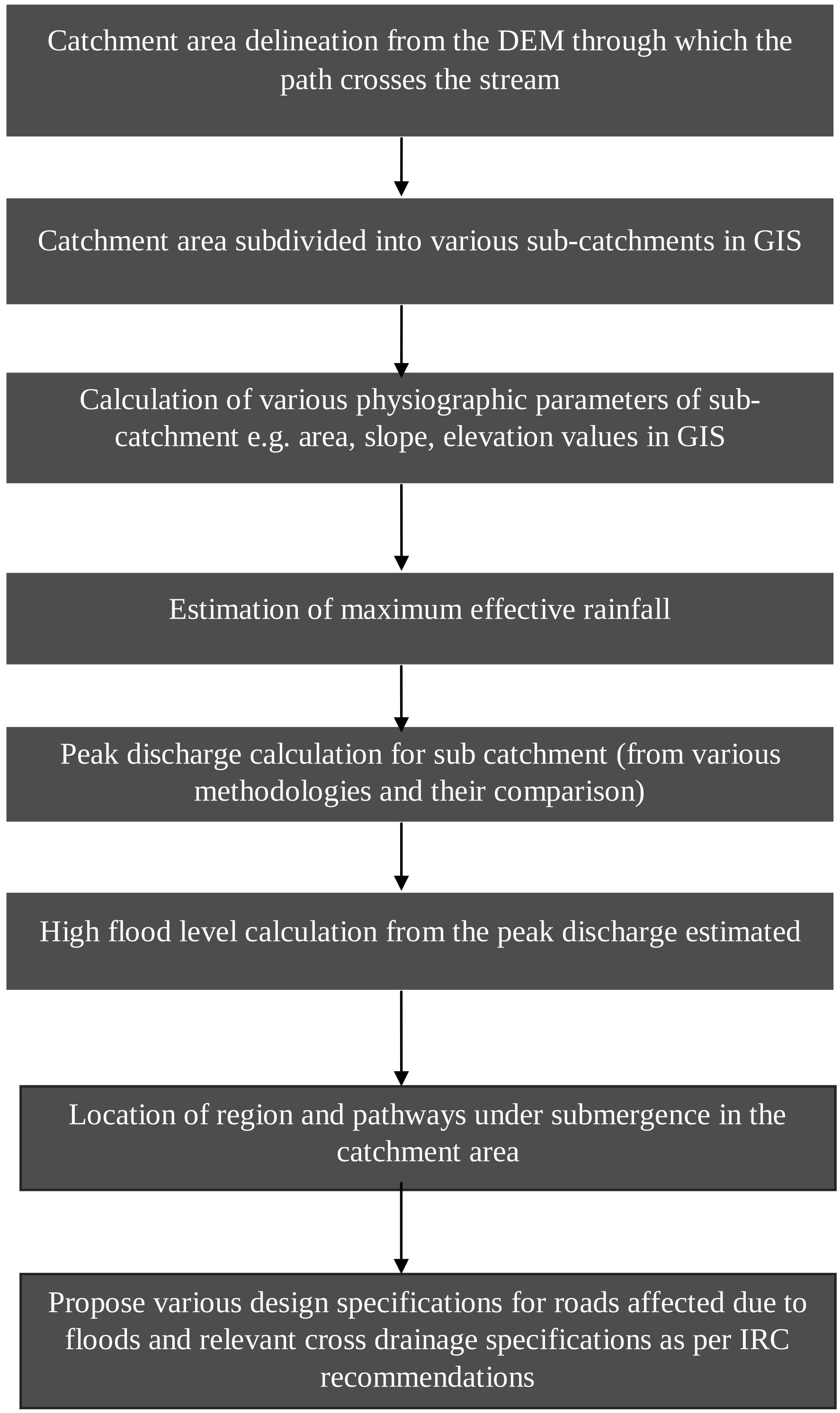

Figure 4.14 Flow chart of overall procedure adopted

CHAPTER 5

OBSERVATIONS AND RESULTS

5.1 Delineation of stream network and sub-catchments in Arc GIS

Digital elevation model is imported into arc GIS environment. Then digital elevation model is processed using Arc hydro tool. Some sequential steps which are listed below followed using arc hydro tool to generate different hydrological units and drainage network. In Figure 5.1 shown below DEM overlaid. Blue lines are showing drainage lines, polygons with dark purple boundary are showing sub catchments from where the peak discharge is obtained. The red line is showing major district road in this region. Each sub catchment is numbered. Stream network and sub catchments are in the form of shape file. The same area is shown in the google earth with sub catchments and drainage lines overlaid in Figure 5.2.

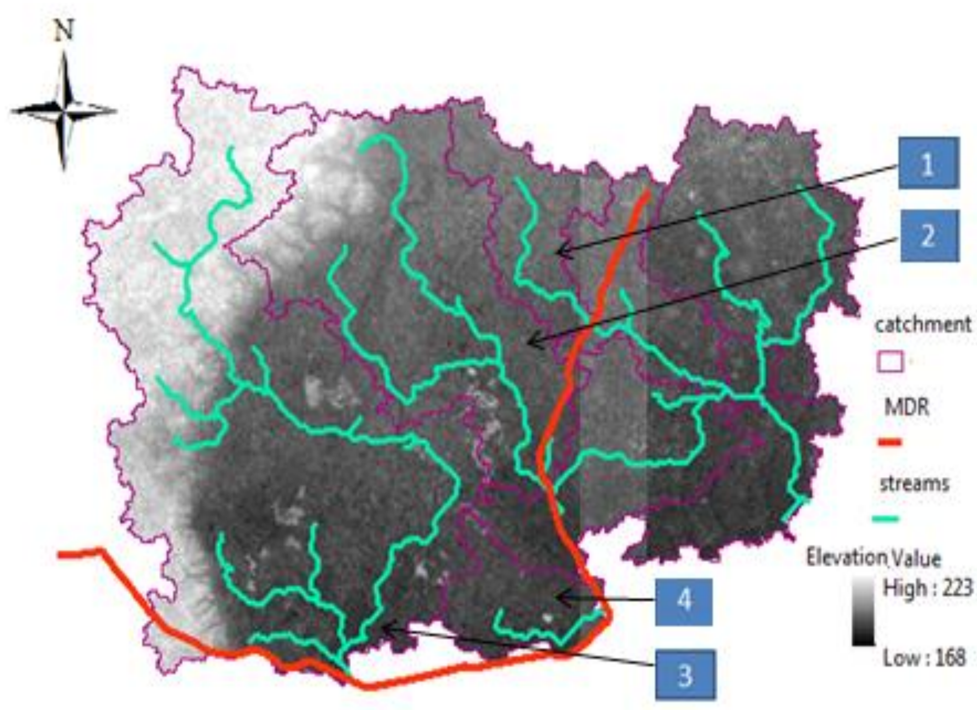

Figure 5.1 DEM with drainage basins and drainage lines overlaid (Laksar area)

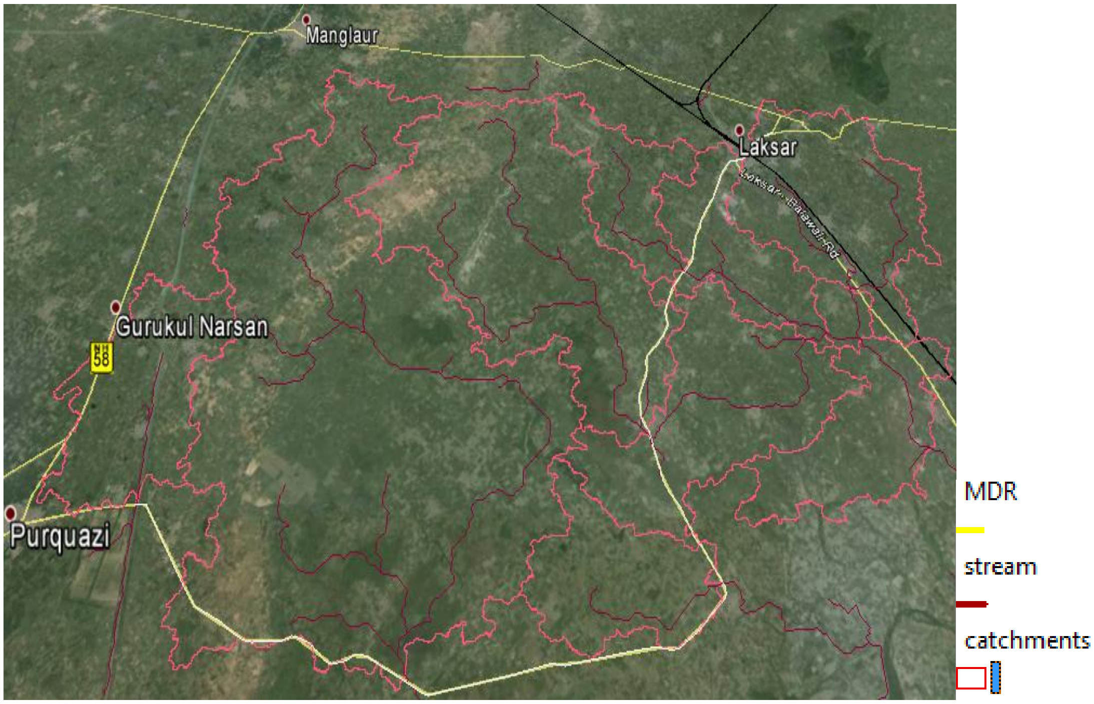

Figure 5.2 Drainage basin and drainage lines overlay on google earth (Laksar area)

Similarly the DEM of the NH 58 bypass drainage basin as shown in figure 5.3 was delineated in Arc GIS. The yellow lines show drainage lines, polygon of red boundary showing sub catchment. The catchment and drainage line overlaid on google earth image of the same region is shown in Figure 5.4.

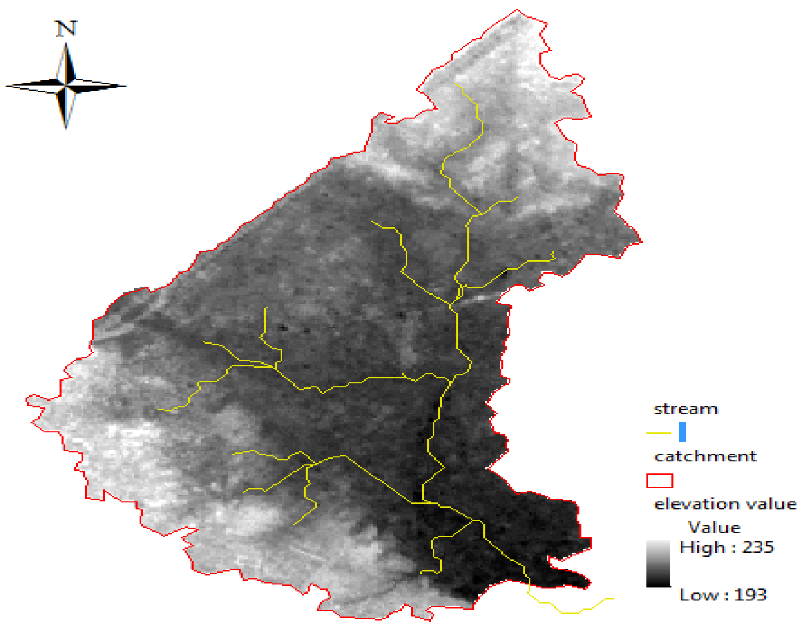

Figure 5.3 DEM with drainage basin and drainage lines overlaid (NH 58 bypass drainage basin)

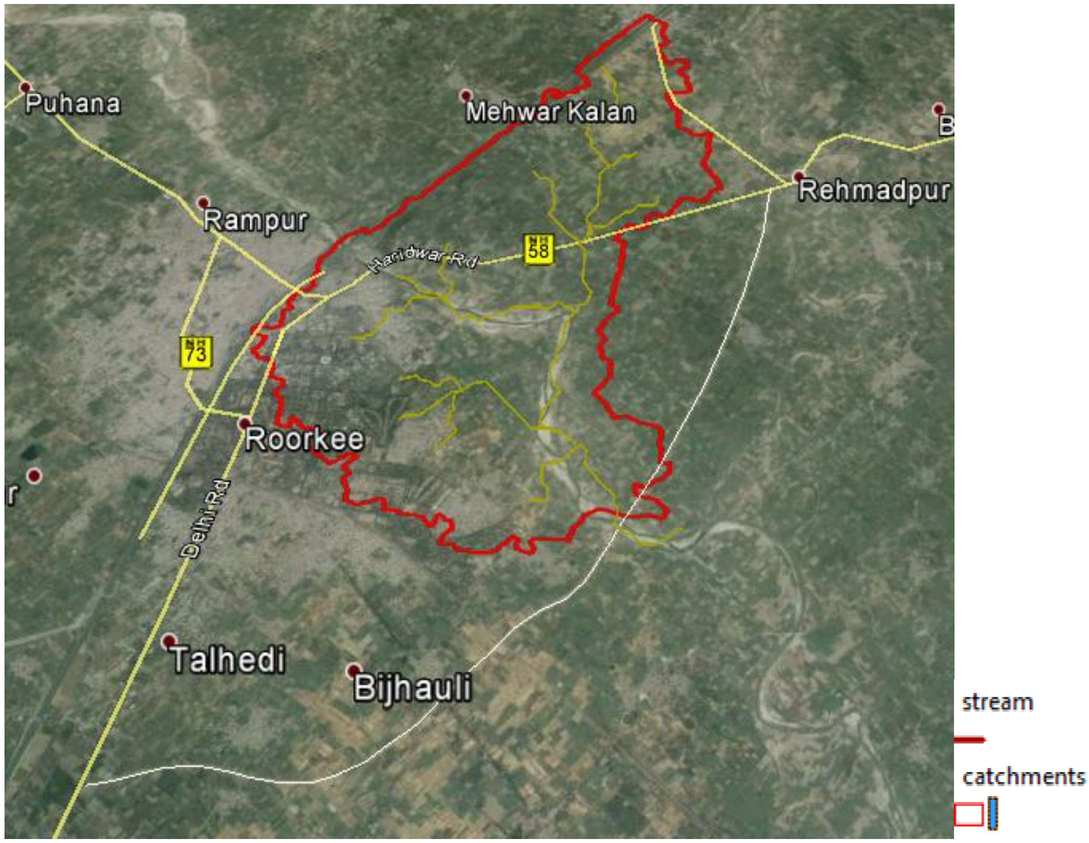

Figure 5.4 Drainage basin and drainage lines overlay on google earth (NH 58 bypass drainage basin)

Some sequential steps in Arc GIS, fill sink, flow direction, flow accumulation, and catchment polygon processing were followed to delineate the catchment.

5.2 Laksar-Purquaji road Drainage basin

5.2.1 Peak discharge calculation by unit hydrograph method

From the digital elevation model of the study area processed in arc GIS various physiographic parameters such as length of longest stream of catchment, stream slope, area of the sub-catchment are derived. The physiographic parameters of the sub catchments thus obtained are shown in table 5.1.

Table 5.1 Observed physiographic parameters of the Sub catchment

Sub catchment	Length of longest stream, L(km)	Stream slope, S(m/km)	Area, A(sq. km)
1	5.72	6	12.01
2	12.9	4	46.05
3	21.4	2	90.09
4	3.9	6	9.13

From the physiographic parameters, unit graph parameters for each sub-catchment are derived from the empirical relationships as per the flood estimation reports (from table 3.9) for the preparation of unit hydrograph for each sub-catchment. The various unit graph parameters estimated for unit hydrograph of 2 hour duration (t_r=2 hour, figure 3.6) for each sub catchment are listed in table 5.2.

Table 5.2 Unit graph parameters derived from physiographic parameters (from table 3.9)

Sub Catchment	$q_p = \frac{2.030}{\left(\frac{L}{\sqrt{S}}\right)^{.649}}$	Q= $q_p \times A$	$t_p = \frac{1.858}{q^{1.038}}$	Storm duration (hr), $t_p + t_r/2$	$W_{50} = \frac{2.217}{q^{.99}}$	$W_{75} = \frac{1.477}{q^{.876}}$	$W_{R50} = \frac{0.812}{q^{.907}}$	$W_{R75} = \frac{0.606}{q^{.791}}$	$T_B = 7.744(t^{.779})$
1	1.17	14.07	1.5	2.3	1.8	1.5	.86	.53	11.04
2	.61	27.8	3.12	4.02	3.64	2.29	1.28	.90	18.8
3	.347	31.31	5.57	6.07	6.3	3.7	2.12	1.39	29.5
4	1.5	13.69	1.22	2.22	1.48	1.03	.56	.43	9.03

Form the unit graph parameters obtained for each sub-catchment the coordinates (time vs. discharge) of the unit hydrograph are estimated and from the coordinates thus obtained unit hydrograph for each sub catchment is plotted. Table 5.3 shows coordinates for each sub-catchment and the plotted unit hydrograph for sub-catchment 1 is shown in Figure 5.5, for sub-catchment 2 is shown in Figure 5.6, for sub-catchment 3 is shown in Figure 5.7, for sub-catchment 4 is shown in Figure 5.8.

Table 5.3 Unit hydrograph Coordinates for various sub-catchments

Sub Catchment 1 (Figure 5.6)		Sub Catchment 2 (Figure 5.7)		Sub Catchment 3 (Figure 5.8)		Sub Catchment 4 (Figure 5.9)	
Time (hour)	Discharge (cumecs)	Time (hour)	Discharge (cumecs)	Time (hour)	Discharge (cumecs)	Time (hour)	Discharge (cumecs)
0	0	0	0	0	0	0	0
0.5	2.5	0.5	4.37	0.5	3.12	1.29	6.82
1	5	1	8.13	1.5	8.75	1.62	10.27
1.39	7.03	1.5	12.5	2	11.87	2.21	13.69
1.5	8.1	2	16.25	2.35	15.6	2.65	10.27
1.99	10.56	2.73	20.91	4	22.6	2.78	6.82
2.57	14.07	4.12	27.88	4.22	23.48	9.03	0
3.12	10.56	4.5	22.5	6.57	31.31		
3.28	7.03	5.02	20.9	7.5	25.62		
5	5.48	5.40	13.9	7.96	23.48		
7	4.16	9	10.15	8.69	15.6		
9	1.84	12	7.03	10	14.6		
11.04	0	15	3.91	15	10.86		
		18.82	0	20	7.11		
				25	3.36		
				29.51	0		

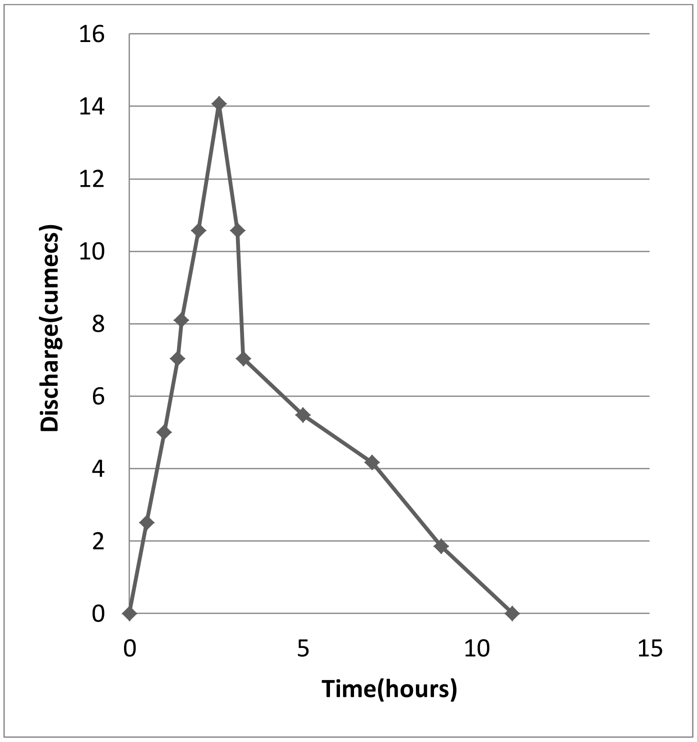

Figure 5.5 Unit hydrograph of 2 hour duration (Sub Catchment 1)

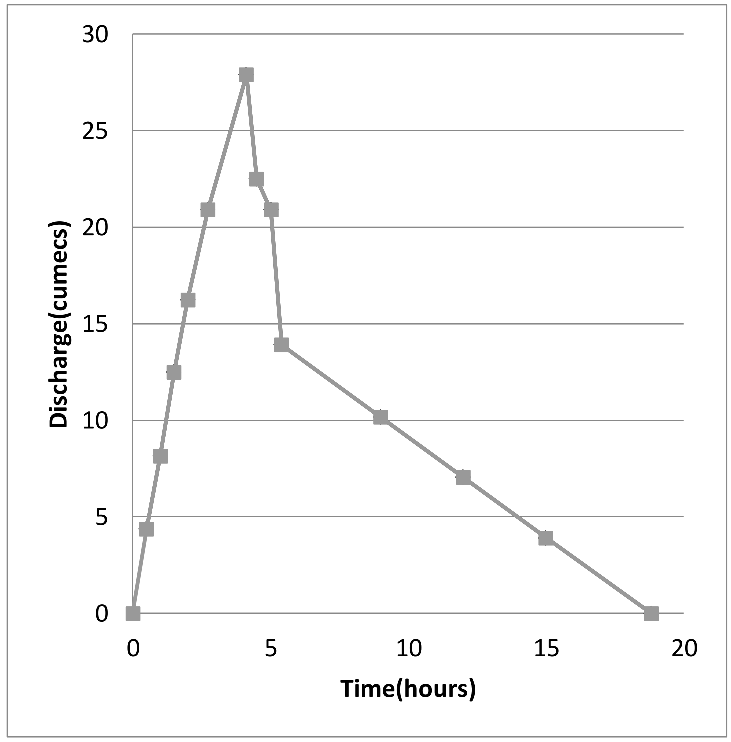

Figure 5.6 Unit hydrograph of 2 hour duration (Sub Catchment 2)

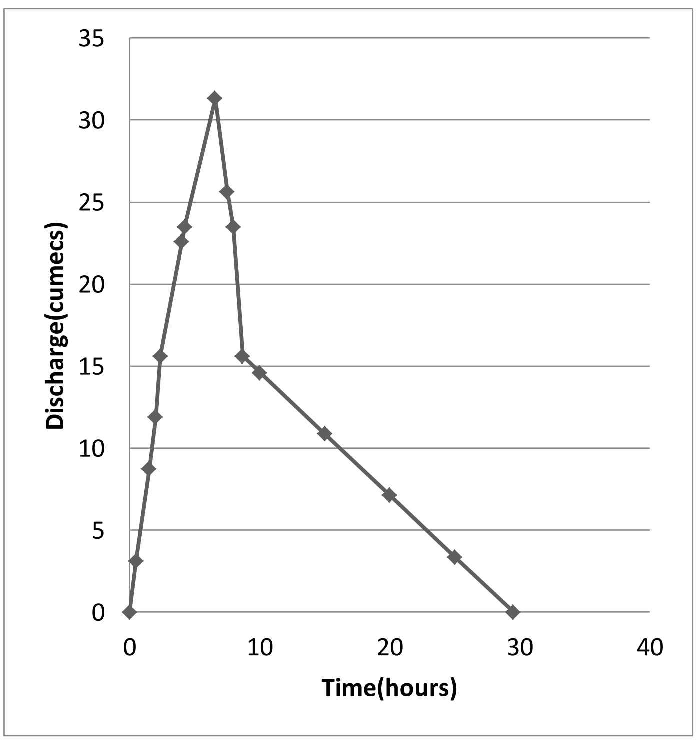

Figure 5.7 Unit hydrograph of 2 hour duration (Sub Catchment 3)

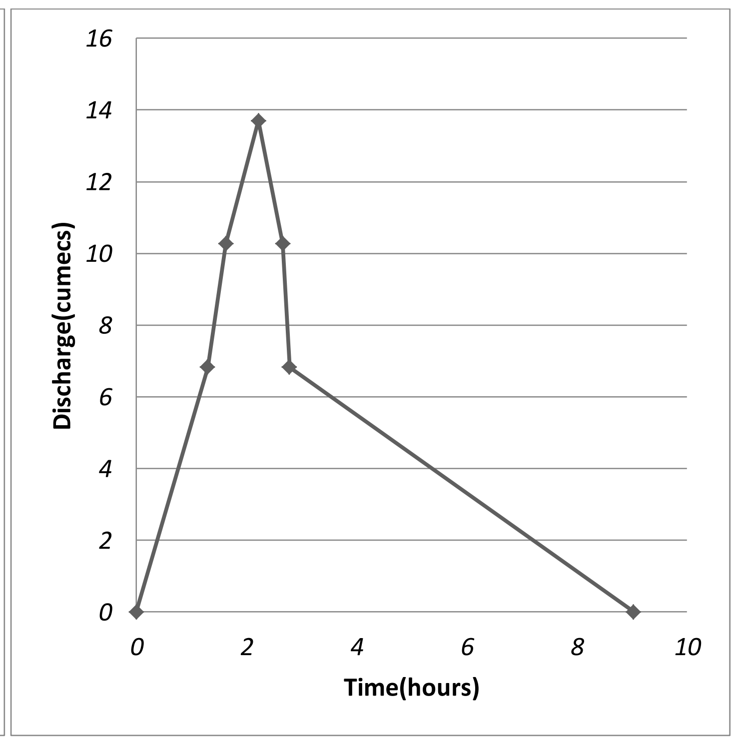

Figure 5.8 Unit hydrograph of 2 hour duration (Sub Catchment 4)

Derivation of maximum effective rainfall

The maximum 24 hour point rainfall data of 50 year return period as obtained from the iso-pluvial map of CWC flood estimation report for the study was converted in to point rainfall corresponding to the storm duration as per the graphical relationship mentioned in CWC flood estimation report to convert 24 hour rainfall to the rainfall of desired storm duration (from figure 4.8).

k = 24 hour point rainfall to required storm duration of rainfall conversion factor

The point rainfall data obtained was further converted to areal rainfall which spread uniformly over the whole catchment as per the graphical relationships between point to areal conversion factor and catchment area (as shown in figure 4.9) as per CWC flood estimation report guidelines

x = point rainfall to areal rainfall conversion factor.

The effective rainfall thus obtained for each sub-catchment is shown in table 5.4.

Table 5.4 Conversion of 24 hour maximum rainfall data to maximum rainfall of required storm duration (as CWC flood estimation report guidelines)

Sub catchment	Storm Duration (hour)	24 hour point rainfall, p (cm)	k	Point rainfall (R) of storm duration= k×p	x	Effective rainfall=p×k×x
1	2.5	28.6	.47	13.4	.95	12.7
2	4.12	28.6	.55	15.73	.95	14.95
3	6.57	28.6	.65	18.59	.85	15.8
4	2.22	28.6	.47	13.4	.95	12.7

Further the areal rainfall is subdivided into the amount of rainfall happened in happened in at various intervals within the total storm duration as per the graphical relationships between amount of rainfall and storm duration (as shown in figure 4.10) from CWC flood estimation report guidelines. The amount of rainfall in each successive time intervals of total storm duration are for each sub-catchment are shown in table 5.5.

Table 5.5 Rainfall Distribution in successive time intervals of storm period.

Sub catchment	Amount of rainfall		
	1st 2 hour interval	Next 2 hour interval	Next 2 hour interval
1	1×12.7=12.7	-	-
2	.8×14.95=11.95	.2×14.95=2.99	-
3	.7×15.8=11.06	.2×15.8=3.16	.1×15.8=1.58
4	1×12.7=12.7	-	-

Resultant runoff hydrograph

The resultant hydrograph for each sub-catchment is obtained by multiplying the unit hydrograph ordinates for each sub-catchment with the effective rainfall estimated for the respective sub-catchment. The resultant hydrograph coordinates thus obtained for each sub-catchment are listed below. From the resultant hydrograph coordinates the hydrograph for the required storm duration was plotted.

(a) Sub Catchment 1

The effective rainfall obtained for Sub-catchment 1 was 12.7 cm .The duration of storm was obtained as 2.3 hours which was assumed to be 2 hours. The unit hydrograph ordinates of 2 hour duration for sub-catchment 1 are multiplied with effective rainfall to obtain resultant hydrograph of sub catchment 1 for 2 hour duration as obtained in table 5.5 which is 12..7 cm. The resultant run-off hydrograph coordinates are mentioned in table 5.6. The resultant hydrograph obtained for sub-catchment 1 is shown in figure 5.9.

Table 5.6 Unit hydrograph coordinates and corresponding run-off hydrograph coordinates for sub catchment 1

Time (hour)	Unit hydrograph ordinate (UHO) (cumecs)	UHO×12.7 (Resultant hydrograph)(cumecs)
0	0	0
0.5	2.5	31.92
1	5	63.85
1.39	7.036	89.85
1.5	8.1	103.43
1.99	10.56	134.85
2.57	14.07	**179.76**
3.12	10.56	134.85
3.28	7.036	89.85
5	5.48	69.97
7	4.16	53.21
9	1.84	23.54
11.04	0	0

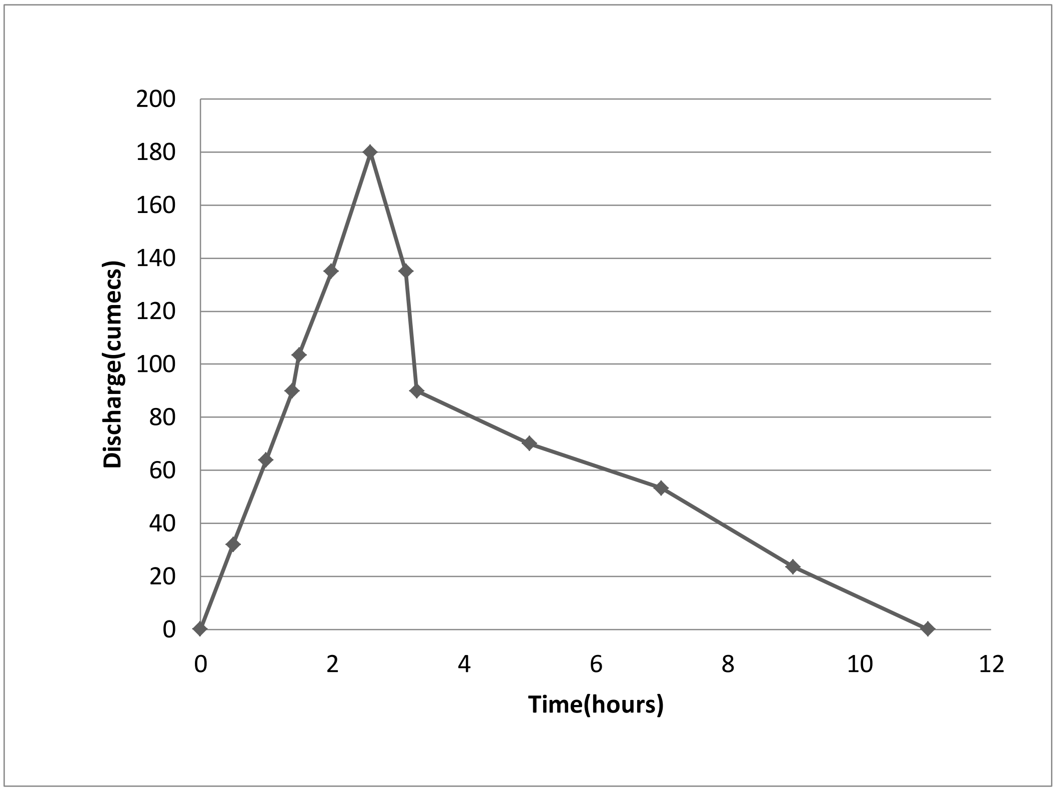

Figure 5.9 Resultant run-off hydrograph of 2 hour duration for Sub catchment 1

(b) Sub Catchment 2

The duration of rainfall obtained for sub-catchment 2 was 4.02 hour, which is assumed to be 4 hours and the effective rainfall obtained for sub-catchment 2 is 14.95cm was divided in amounts of rainfall in successive 2 hour intervals .The unit hydrograph for the sub catchment was of 2 hour duration which converted to 4 hour by lagging the same unit hydrograph by 2 hour. The two hydrographs are thus multiplied with the amount of rainfall in respective time duration obtained in table 5.5(11.95 cm rainfall for Ist hydrograph, 2.99 cm rainfall for second hydrograph) and finally the summation of both hydrographs gives the resultant run-off hydrograph. The resultant run-off hydrograph coordinates are as shown table 5.7.

Table 5.7 Unit hydrograph coordinates and corresponding run-off hydrograph coordinates for sub-catchment 2. UHO mentioned in figure denotes unit hydrograph ordinate

Time	UHO* (cumecs)	UHO×11.95 (Ist hydrograph)	UHO×2.99 (lagged by 2 hr.) (IInd hydrograph)	Resultant Hydrograph ordinates (Hydrograph I +Hydrograph II)
0	0	0	-	0
0.5	4.37	52.28	-	52.28
1	8.13	97.15	-	97.15
1.5	12.5	149.37	-	149.37
2	16.25	194.18	0	194.18
2.73	20.91	249.87	13.081	262.95
4.12	27.88	333.27	24.30	**357.58**
4.5	22.5	268.87	37.37	306.25
5.02	20.9	249.75	48.58	298.34
5.40	13.9	166.10	62.52	228.62
9	10.15	121.36	83.38	204.75
12	7.03	84.08	67.27	151.35
15	3.91	46.79	62.49	109.28
18.82	0	0	41.56	41.56
19.5	-	-	30.36	30.36
20	-	-	21.03	21.03
20.5	-	-	11.70	11.70
20.82	-	-	0	0

The hydrograph I and hydrograph II for sub-catchment 2 are shown in figure 5.10.The resultant hydrograph obtained from the summation of the 2 hydrographs shown in figure 5.11.

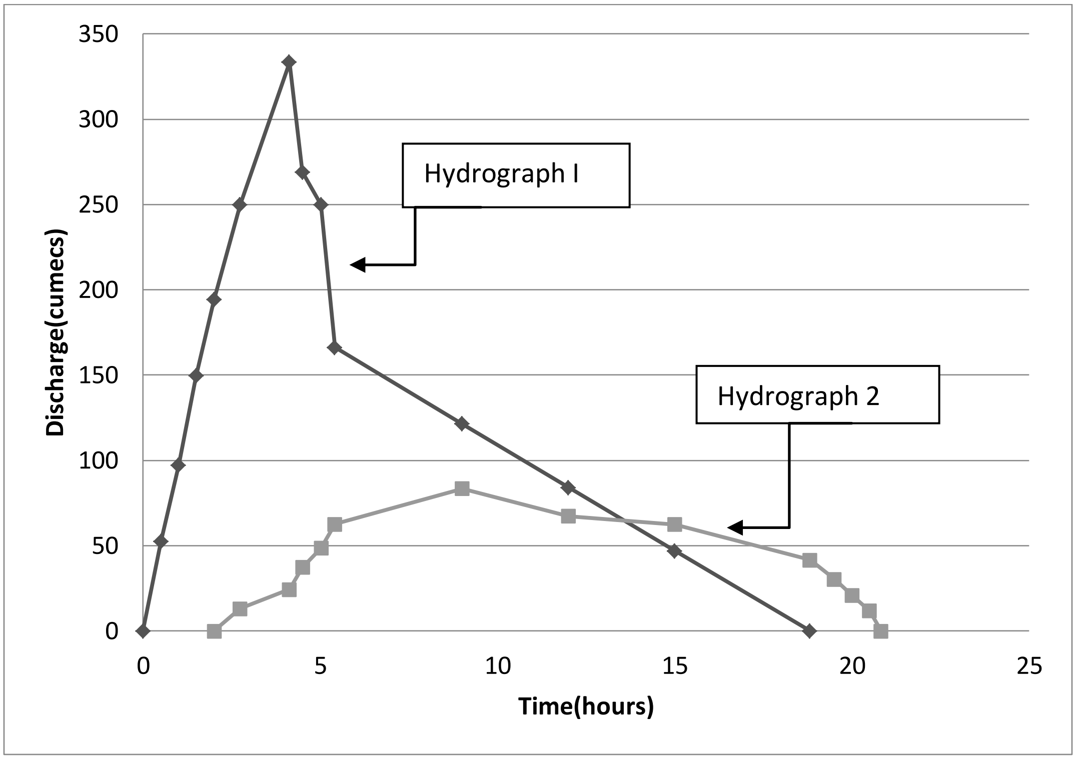

Figure 5.10 Hydrographs of 2 hour duration in succession with corresponding rainfall for sub catchment 2.

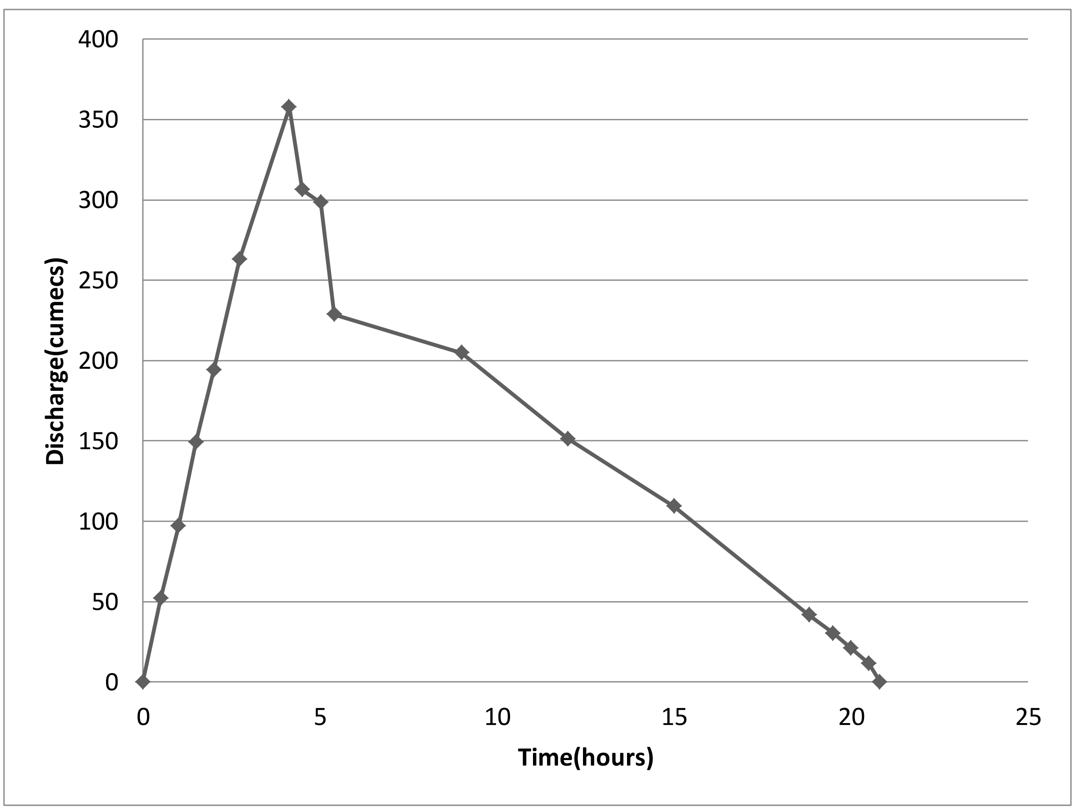

Figure 5.11 Resultant hydrograph of 4 hour duration for Sub catchment 2.

(c) Sub catchment 3

The duration of rainfall obtained for sub-catchment 3 was 6.07 hour, which is assumed to be 6 hours and the effective rainfall obtained for sub-catchment 3 is 15.8 cm was divided in amounts of rainfall in successive 2 hour intervals. The unit hydrograph for the sub catchment was of 2 hour duration which converted to 6 hour by lagging the same unit hydrograph by 2 hour and again the same unit hydrograph by 4 hour. The three hydrographs are thus multiplied with the amount of rainfall in respective time duration as obtained in table 5.5(11.06 cm rainfall for 1st hydrograph, 3.16 cm rainfall for IInd hydrograph and 1.58 cm rainfall for IIIrd hydrograph) and finally the summation of all the hydrographs gives the resultant hydrograph. The resultant hydrograph coordinates are shown below in table 5.8.

Table 5.8 Unit hydrograph coordinates and corresponding run-off hydrograph coordinates for sub-catchment 3.UHO in table denotes unite hydrograph ordinate.

Time	UHO (cumecs)	UHO× 11.06 (1st hydrograph)	UHO×3.16 (lagged by 2 hr.) (IInd hydrograph)	UHO×1.58 (lagged by 4 hr.) (IIIrd hydrograph)	Resultant Hydrograph Ordinates (hydrograph I+II+III)
0	0	0	-	-	0
0.5	3.12	34.56	-	-	34.56
1.5	8.75	96.77	-	-	96.77
2	11.87	131.33	0	-	131.33
2.35	15.6	172.53	9.87	-	182.41
4	22.6	249.95	27.65	0	277.60
4.22	23.48	259.76	37.52	4.93	302.22
6.57	31.31	346.35	49.29	13.82	**409.47**
7.5	25.62	283.40	71.41	18.76	373.57
7.96	23.48	259.76	74.21	24.64	358.63
8.69	15.6	172.53	98.95	35.70	307.20
10	14.6	161.47	80.97	37.10	279.55
15	10.86	120.18	74.21	49.47	243.88
20	7.11	78.71	49.29	40.48	68.49
25	3.36	37.16	46.13	37.10	120.40
29.51	0	0	34.33	24.68	58.98
30	-	-	22.48	23.06	45.55
31.2	-	-	10.61	17.16	27.78
31.51	-	-	0	11.24	11.24
32	-	-	-	5.30	5.30
33.51	-	-	-	0	0

The hydrograph I, hydrograph II and hydrograph III for sub-catchment 3 are shown in figure 5.12.The resultant hydrograph obtained from the summation of all the 3 hydrographs shown in figure 5.13.

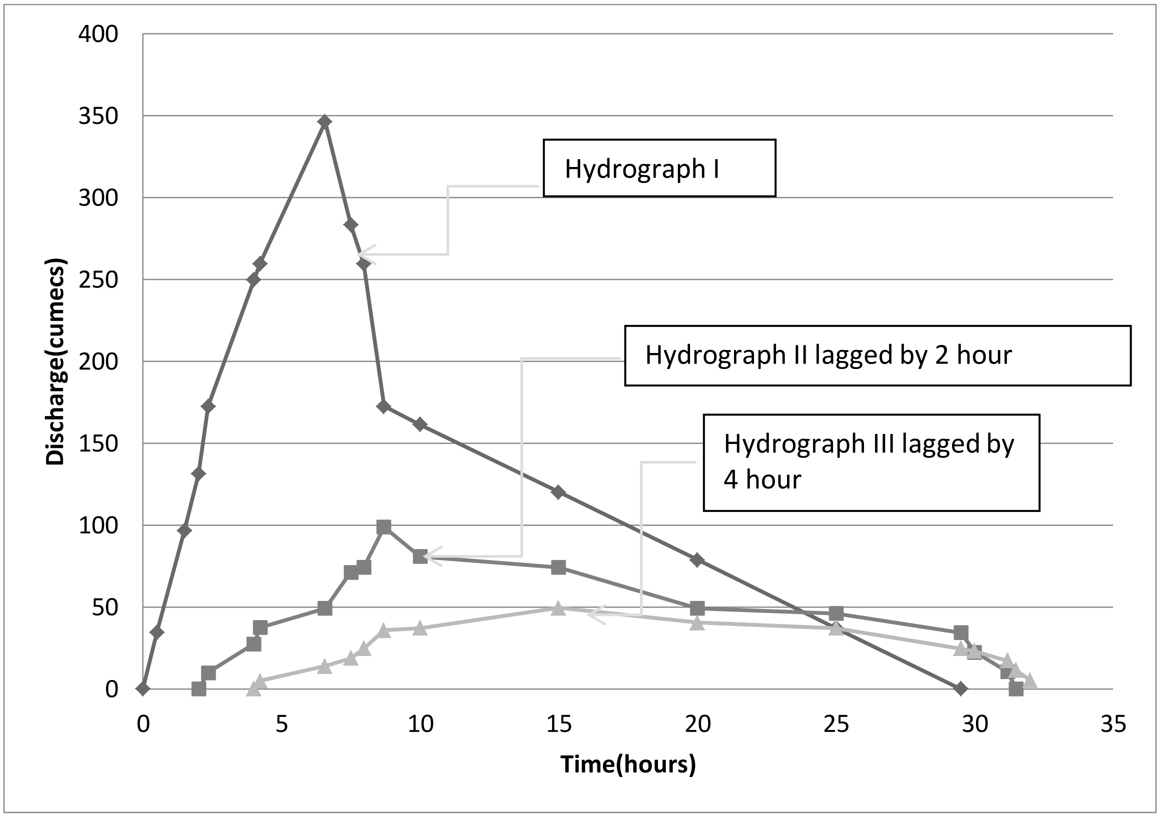

Figure 5.12 Hydrographs of 2 hour duration in succession with corresponding rainfall for sub catchment 3.

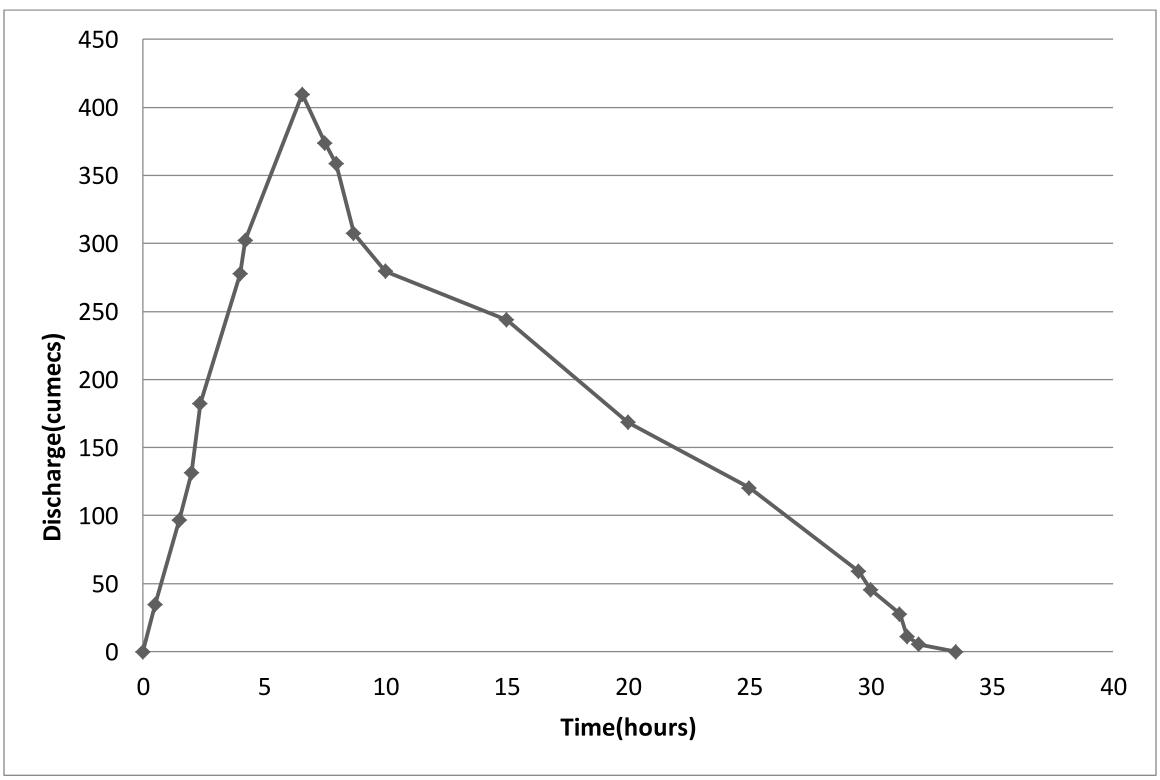

Figure 5.13 Resultant run-off hydrograph of 6 hour duration for Sub catchment 3.

(d) Sub catchment 4

The duration of storm obtained for sub-catchment 4 is 2.22 hours which was assumed to be 2 hours. The ordinates of unit hydrograph of 2 hour duration are multiplied with the effective rainfall which 12.7 cm for sub-catchment 4 to obtain the ordinated of the resultant hydrograph for sub catchment 4.The ordinates of resultant hydrograph for sub-catchment 4 thus obtained are shown below in table 5.9. The resultant hydrograph for sub-catchment 4 is shown in figure 5.14.

Table 5.9 Unit hydrograph ordinates and corresponding resultant hydrograph ordinates for sub-catchment 4. UHO mentioned the figure denotes unit hydrograph ordinate

Time (hours)	UHO* (Cumecs)	UHO×12.7 (Resultant hydrograph)
0	0	0
1.29	6.82	87.18
1.62	10.27	131.19
2.21	13.69	**174.94**
2.65	10.27	131.14
2.78	6.82	87.18
9.03	0	0

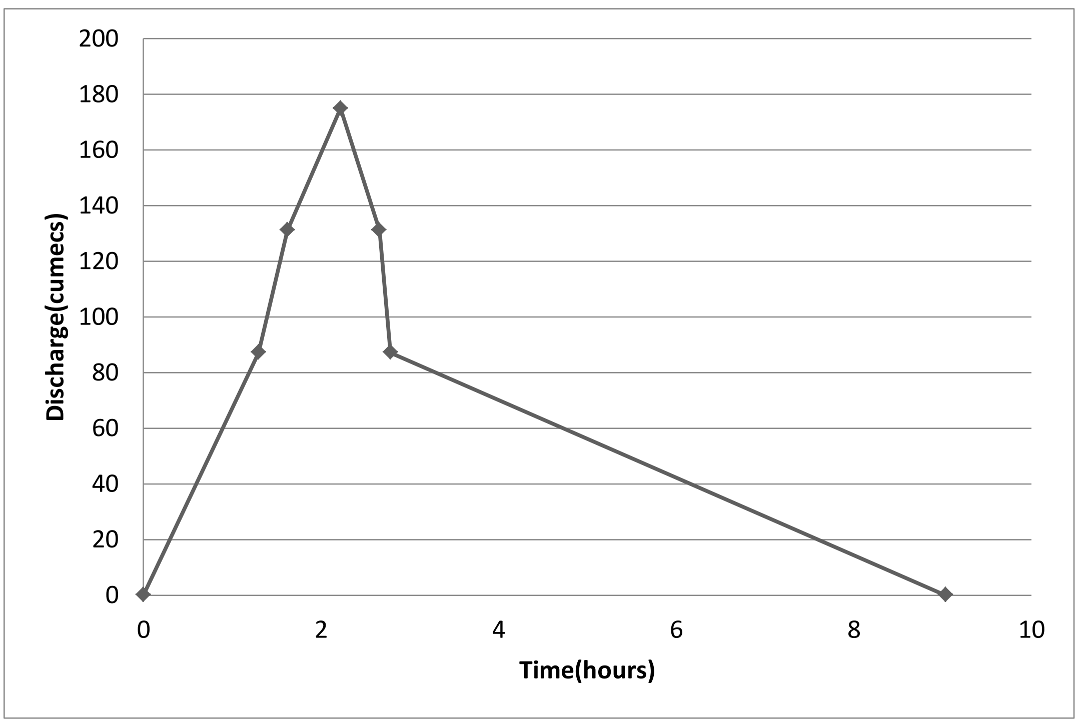

Figure 5.14 Resultant runoff hydrograph of 2 hour duration for sub catchment 4

5.2.2 Peak discharge Calculation as per formula prescribed by CWC for 50 year return period

As per the relationship(equation 2.18) recommended in CWC flood estimation report to calculate peak discharge of 50 year return period, the peak discharge for various sub-catchments is calculated. Coefficient k was interpolated depending upon the catchment area. Point rainfall obtained (R) is used as obtained in table 5.4. The slope of stream, length of stream is used as obtained in table 5.1. All the parameters thus obtained and the peak discharge estimated for each sub-catchment is shown in table 5.10.

Table 5.10 Peak discharge calculation (from eq. 2.18) of various sub catchments using CWC formula

Sub Catchment	Catchment Coefficient K	Catchment Area, A (sq. km)	Point rainfall, R (cm) (From table 5.4)	Stream slope, S(m/km)	Stream length, L(km)	Peak Discharge (cumecs) $Q = K\times A \times R \times \frac{S^{.324}}{L^{.649}}$
1	1.7	12.01	13.4	6	5.72	150.32
2	1.64	46.05	15.73	4	12.9	353.69
3	1.5	90	18.59	2	21.4	427.3
4	1.7	9.13	13.4	6	3.9	147.33

5.2.3 Peak discharge calculation from Rational method as per IRC SP 13:2011

Peak discharge calculation is done as per relationship recommended by IRC SP 13:2011 (equation 3.10) for each sub-catchment. The time of concentration was calculated from equation 3.7 and corresponding critical intensity of rainfall was calculated from equation 3.5 as per IRC SP 13:2011.The length of stream, difference in elevation of highest and lowest points were from GIS (table 5.1).The run-off coefficient was assumed to be .5 considering the area mainly agricultural and alluvial soil which was monitored in google earth. Point to areal reduction factor was obtained as per the graphical relationship given in IRC SP 13:2011 between point to areal reduction factor and catchment area (from figure 3.4). Hence all these parameters for each sub catchment and the corresponding peak discharge obtained is mentioned in table 5.11.

Table 5.11 Peak discharge calculation (from eq. 2.10) of various sub catchments using Rational method

Sub catchment	L^+	H^*	$t_c=\left(.87 \times \frac{L^3}{H}\right)^{.385}$	$I_c=\frac{\left(\frac{T+1}{t+1}\right)F}{T}$	f**	P^{++}	A (hectares)	Peak Discharge Q (cumecs)
1	5.72	10	2.3	8.95	.95	.5	12.1×10^2	144.3
2	12.9	35	4.62	5.2	.95	.5	46.1×10^2	326.18
3	21.4	37	7.94	3.35	.85	.5	90.1×10^2	364.5
4	3.9	5	2.2	8.8	.95	.5	9.2×10^2	123

*H, Difference in the elevation between the highest point and the lowest point at outlet in the catchment area(in metres).

+L, Length of longest stream in the catchment (in km).

++P, Coefficient of catchment.

***f*, Point to areal rainfall reduction factor.

t_c, Time of concentration

I_c, Critical intensity of rainfall corresponding to time of concentration

5.2.4 Results and analysis

Comparison of discharge from different methods

The peak discharge obtained from the above three methods estimated and compared for each sub-catchment as shown in table 5.12. The maximum of the peak discharge obtained for each sub-catchment is used for further calculation.

Table 5.12 Comparison peak discharge of various sub catchments from different methods

Method	Peak discharge(cumecs)			
	Sub-catchment 1	Sub-catchment 2	Sub-catchment 3	Sub-catchment 4
Hydrograph	**179.7**	**357.5**	409.4	**174.9**
CWC formula	150.3	353.6	**427.3**	147.3
Rational method	144.3	326.1	364.5	123.0

The maximum value of peak discharge obtained for sub-catchment 1 is 179.7 cumecs

The maximum value of peak discharge obtained for sub-catchment 2 is 357.5 cumecs

The maximum value of peak discharge obtained for sub-catchment 3 is 427.3 cumecs

The maximum value of peak discharge obtained for sub-catchment 4 is 174.9 cumecs

The peak discharge thus obtained is further utilized in manning's equation (equation 2.19) to obtain the depth of flow of the stream width of the channel is assumed to be given as per the Lacey's width of waterway for regime flow. The hydrodynamic parameter n is taken as .026. Stream slope used as obtained in table 5.1 for each sub-catchment. The depth of flow and the area of waterway obtained for each sub-catchment thus obtained are shown in table 5.13.

The depth of flow in stream for each sub-catchment obtained was added to the lowest elevation level above mean sea level of each sub catchment which was obtained through the DEM of the sub catchment, which gives the high flood level of various sub-catchments in the region as shown in table 5.14. In figure 5.15 google earth image of the study area is shown with elevation profile of the road. From the high flood level obtained, the road stretch having elevation lower then HFL located, which would get submerged during floods. These road stretches are shown in google earth image of the study area in figure 5.16 as well as mentioned in table 5.14. In figure 5.16 the parts of road marked red along the road profile have elevation below the HFL calculated and hence are designated as the regions of submergence. The same region is shaded in the elevation profile of road in figure 5.16. Minimum height of the road subgrade designed as per the IRC SP 34:2011 is 1.5 m above HFL as mentioned below. As per this specification the appropriate elevation of the subgrade level of road for the given study is mentioned in table 5.14.

Table 5.13 Calculation of depth of flow corresponding peak discharge from Manning's equation (from eq. 2.19)

Sub catchment	Peak Discharge, Q (cumecs)	Rugosity Coefficient, n	Stream slope, S	Width, $w=4.75Q^{.5}$ (m)	Depth of flow, d (m)	Area of water way (d×w) m^2
1	179	.026	.006	64.01	1.03	65.93
2	357	.026	.004	89.8	1.35	121.5
3	427	.026	.002	98.19	1.8	176.4
4	174.9	.026	.006	62.5	.94	59.38

Table 5.14 High Flood Level, road submergence and design subgrade level of road

Sub catchment	Depth of flow(d) (in metres)	Z(elevation of lowest point in catchment above mean sea level) (in metres)	HFL(m)= d+Z	Road stretch under submergence (chainage) (km)	Minimum height of sub grade (m) above HFL=HFL+1.5 m (from IRC SP 34:2011)
1	1.03	235	236.03	3.3-7.6	237.56
2	1.35	231	232.35	7.8-11.2	233.85
3	1.8	229	230.8	16.4-22	232.3
4	.94	230	230.94	11.7-14	232.44

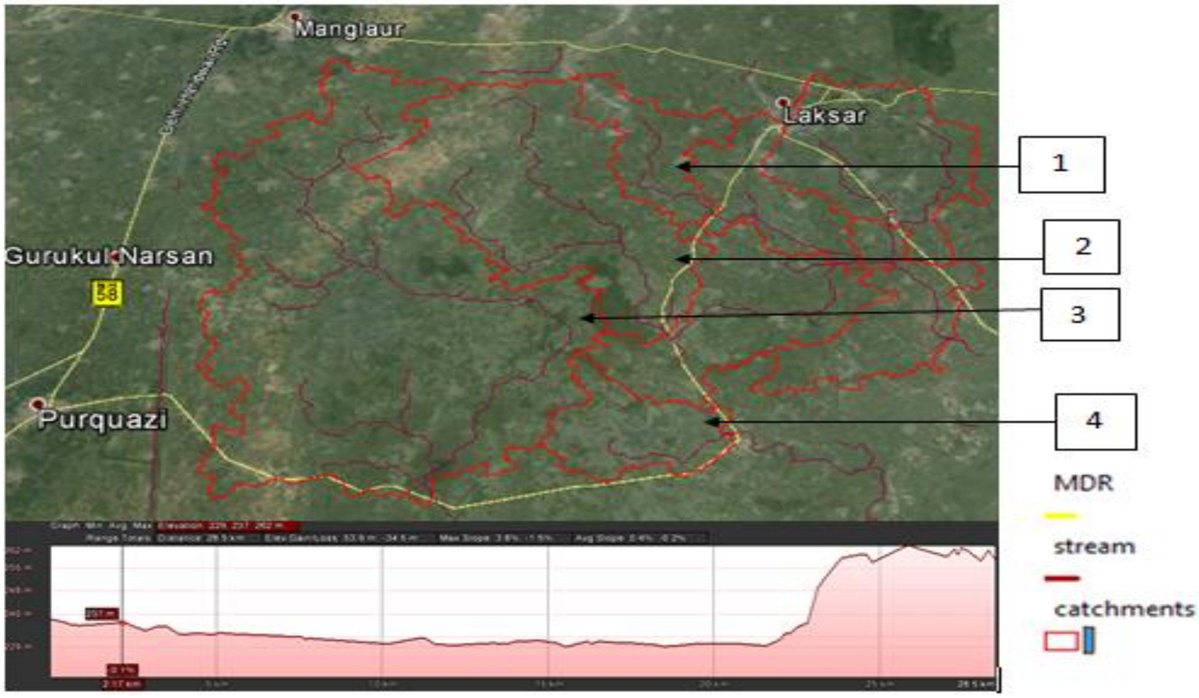

Figure 5.15 The sub catchments overlay on google earth image of Laksar and the elevation profile of the Laksar-purquaji road below the google earth image.

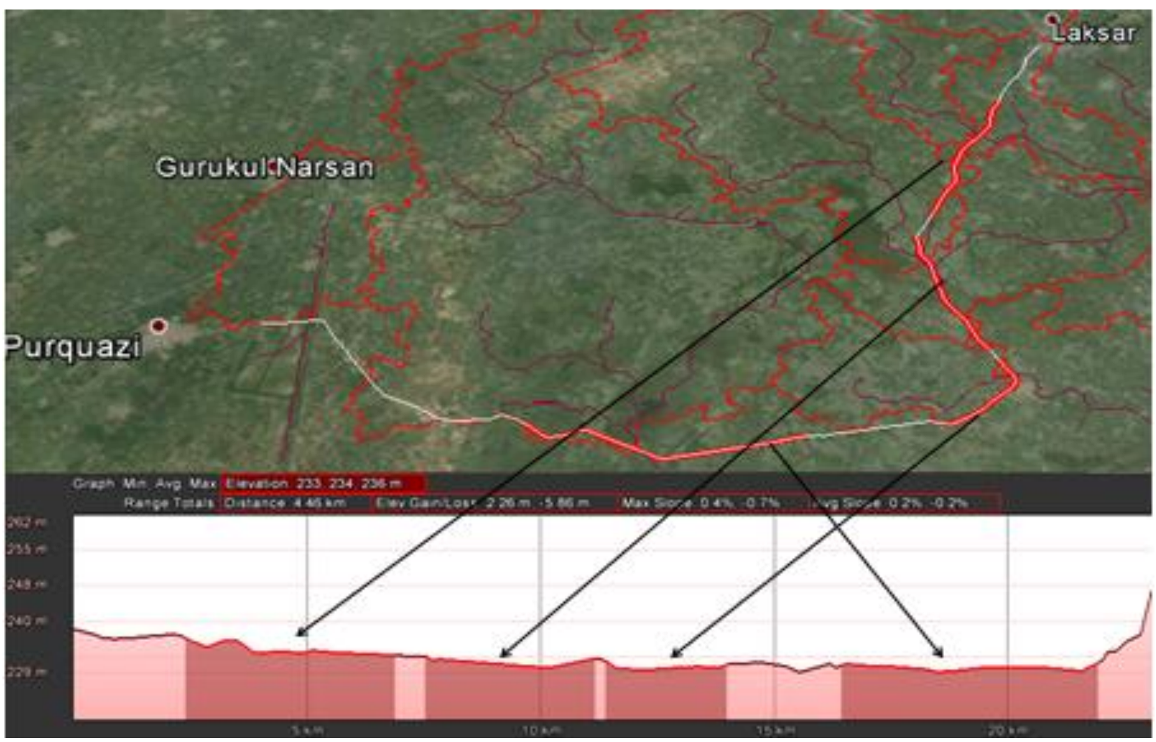

Figure 5.16 Location of regions of submergence during floods on google earth image of Laksar area. The road stretches marked red indicate regions under submergence and the indicated by black arrows in elevation profile of road in respective regions.

As per IRC SP 13:2011 specifications minimum vertical clearance between water way and the soffit depends on the discharge passing through the stream. The minimum vertical clearance obtained for each sub-catchment obtained as per the table 2.1 (from IRC SP

13:2011). The depth of flow was added to minimum vertical clearance which is denoted by T in table 5.15. The value obtained is the height of the soffit (bottom of road above the bed level) which is multiplied by the width of the waterway to obtain design bridge opening(in m^2) for each sub-catchment. The width of the water way is calculated as per Lacey's equation for width of waterway in regime flow conditions which depends on discharge flowing through the stream ($w=4.75Q^{.5}$). The area of bridge opening thus estimated was large for the provision of culverts so it was appropriate to provide clear water way in comparison to culverts. For the protection of embankment against the capillary rise of water during floods the thickness of capillary cut-off provided is 15cm (coarse sand) as per table 2.2(from IRC SP 34:2011). The specifications thus estimated for each sub-catchment are mentioned in table 5.15.

Table 5.15 Cross drainage structure specifications

Sub catchment	Peak Discharge, Q (cumecs)	minimum vertical clearance (m) (from table 2.1)	T (m) = Depth of flow + minimum vertical clearance	Width, $w=4.75Q^{.5}$ (m)	Design opening Beneath road ($T\times w$) (m^2)
1	179	.9	1.93	64.01	123.54
2	357	1.2	2.55	89.8	229.0
3	427	1.2	3.0	98.19	294.6
4	174.9	.9	1.84	62.5	115.0

5.3 NH 58 bypass Drainage basin

5.3.1 Peak discharge calculation by unit hydrograph method

The physiographic parameters are obtained from processing of the DEM of the catchment area in arc GIS

Length of longest stream, L=7 km.

Bed slope, S=1.714m/km.

Catchment area, A=31.01 sq. km

From the physiographic parameters thus obtained unit graph parameters are obtained as per the empirical relation mentioned in CWC flood estimation reports (from table 3.9).The unit graph parameters thus obtained are used for the preparation of unit hydrograph. Table 5.16 mentions the unit graph parameters thus calculated from NH 58 bypass drainage basin.

Table 5.12 Unit graph parameters for NH 58 bypass drainage basin (from table 3.9)

$q=\frac{2.030}{\left(\frac{L}{\sqrt{S}}\right)^{.649}}$	$t_p=\frac{1.858}{q^{1.038}}$	$W_{50}=\frac{2.217}{q^{.99}}$	$W_{75}=\frac{1.477}{q^{.876}}$	$W_{R50}=\frac{0.812}{q^{.907}}$	$W_{R75}=\frac{0.606}{q^{.791}}$	$T_B=7.744(t^{.779})$
.686 cumec/sq. km	2.76 hr.	3.23 hr	2.06 hr	1.147 hr	.819 hr	17.077 hr

Peak discharge ordinate of 2 hour unit hydrograph = discharge per unit area(q)×catchment area

discharge per unit area(q) as obtained from table 5.12 is .686 cumecs

catchment area = 31.01 sq. km

.686×31.01 = 21.173 cumecs

Unit graph parameter t_p = 2.76 hour (form table 5.12)

t_r = duration of rainfall (2 hour)

Duration of the storm (figure 3.6) = $t_p + t_r/2$ = 2.76 + (2/2) = 3.76 hour.

Form the unit graph parameters thus obtained the coordinates of unit hydrograph are calculated and the unit hydrograph for NH 58 bypass drainage basin was plotted. The unit hydrograph coordinates are mentioned in table 5.17 and the unit hydrograph obtained is shown in figure 5.17.

Table 5.17 Unit hydrograph coordinates for NH 58 bypass drainage basin

Time(hour)	Unit hydrograph ordinate (cumecs)
0	0
0.5	1.875
1	4
1.5	6.25
2	8.75
2.6	10.6
2.9	15.87
3.76	21.173
4.1	18.75
4.9	15.87
5.8	10.6
9	7.6
12.2	4.6
15.4	1.6
17.07	0

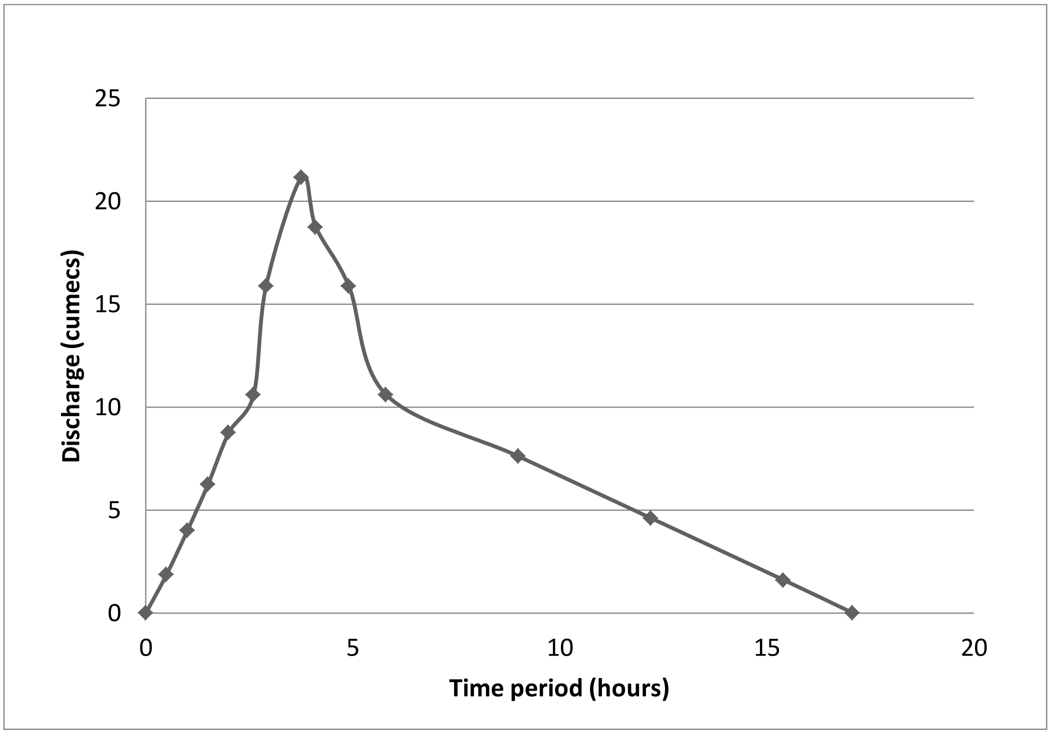

Figure 5.17 Unit hydrograph of 2 hour duration for NH 58 Drainage basin bypass.

Derivation of maximum effective rainfall:

The maximum 24 hour point rainfall data of 50 year return period as obtained from the iso-pluvial map of CWC flood estimation report was converted to rainfall of respective storm duration in the same way as done for the sub-catchments in Laksar area. The storm duration as obtained from unit hydrograph is 3.76 hour which was assumed to be 4 hour. The amount of rainfall obtained is mentioned in table 5.18. In table 5.19 distribution of rainfall in successive intervals of total storm duration is mentioned.

Table 5.18 Derivation maximum effective rainfall for NH 58 bypass drainage basin (as per CWC flood estimation report guidelines) (from figure 4.8)

Maximum rainfall in 24 hour period, R (cm)	Duration of storm (hour)	24 hour point rainfall to 4 hour point rainfall Conversion factor (k)	point rainfall to areal rainfall conversion factor (p)
26.67	4	.55	.95

Table 5.19 Distribution of effective rainfall obtained in table 5.18 in successive intervals of total storm duration (from figure 4.9)

Areal rainfall in 4 hour duration (cm), (R×k×p)	Amount of rainfall in 1st 2 hour interval (cm) (.81×13.9)	Amount of rainfall in IInd 2 hour interval (cm) (.19×13.9)
13.9	11.12	2.78

Resultant hydrograph of 4 hour duration

The unit hydrograph obtained for the study area is of 2 hour duration. The storm duration obtained was 3.76 hour which was assumed to 4 hour and the unit hydrograph of 2 hour duration was converted to 4 hour by lagging the same unit hydrograph by 2 hour .The hydrograph ordinates are further multiplied by the respective amount of rainfall in their time interval and finally added to obtain the resultant hydrograph. The unit hydrograph coordinates and corresponding run-off hydrograph coordinates are mentioned in table 5.20.

Table 5.20 unit hydrograph and resultant run-off hydrograph coordinates for NH 58 bypass drainage basin

Time (hr.)	Unit graph ordinate (cumecs)	lagged by 2 hour	Hydrograph 1(cumecs) Unit graph ordinate × effective rainfall (column 2 × 11.12cm)	Hydrograph 2(cumecs) Unit graph ordinate × effective rainfall (column 3 × 2.78cm)	Resultant(cumecs) (summation of Hydrograph 1, Hydrograph 2)
0	0	-	0	-	0
0.5	1.875	-	20.85	-	20.85
1	4	-	44.48	-	44.48
1.5	6.25	-	69.5	-	69.5
2	8.75	0	97.3	0	97.3
2.6	10.6	1.87	117.87	5.21	123.08
2.9	15.87	4	176.47	11.12	187.59
3.76	21.17	6.25	235.44	20.37	**255.88**
4.1	18.75	8.75	208.5	24.32	232.82
4.9	15.87	10.6	176.47	29.46	205.94
5.8	10.6	15.87	117.87	44.11	161.99
9	7.6	21.17	84.51	58.86	143.37
12.2	4.6	18.75	51.15	52.12	103.27
15.4	1.6	15.87	17.79	44.11	61.91
17.07	0	10.6	0	29.46	29.46
17.57	-	7.6	-	21.12	21.12
18.07	-	4.6	-	12.78	12.78
18.57	-	1.6	-	4.44	4.44
19.07	-	0	-	0	0

Figure 5.18 shows the hydrograph I and hydrograph II corresponding to the amount of rainfall in respective time intervals .Figure 5.19 shows the resultant hydrograph obtained by the summation of the two hydrograph shown in figure 5.18.

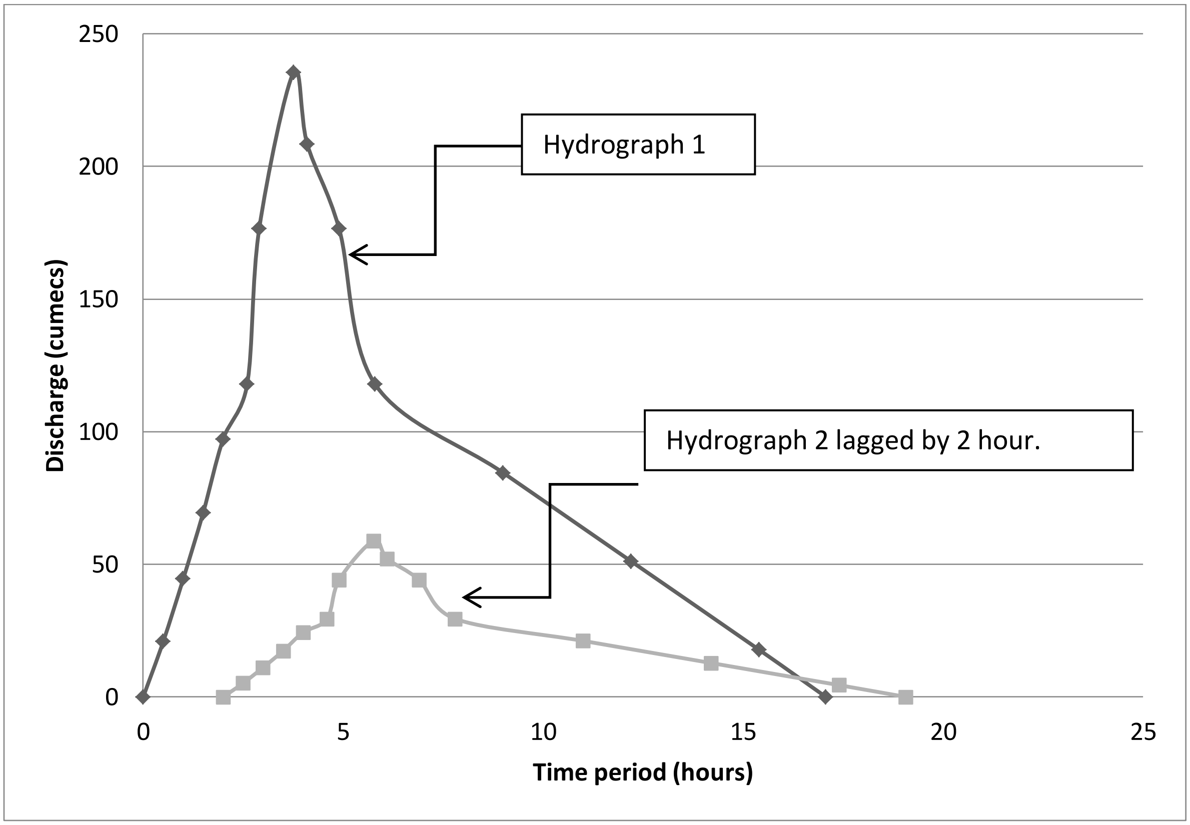

Figure 5.18 Hydrographs of 2 hour duration in succession with corresponding rainfall for NH 58 bypass drainage basin.

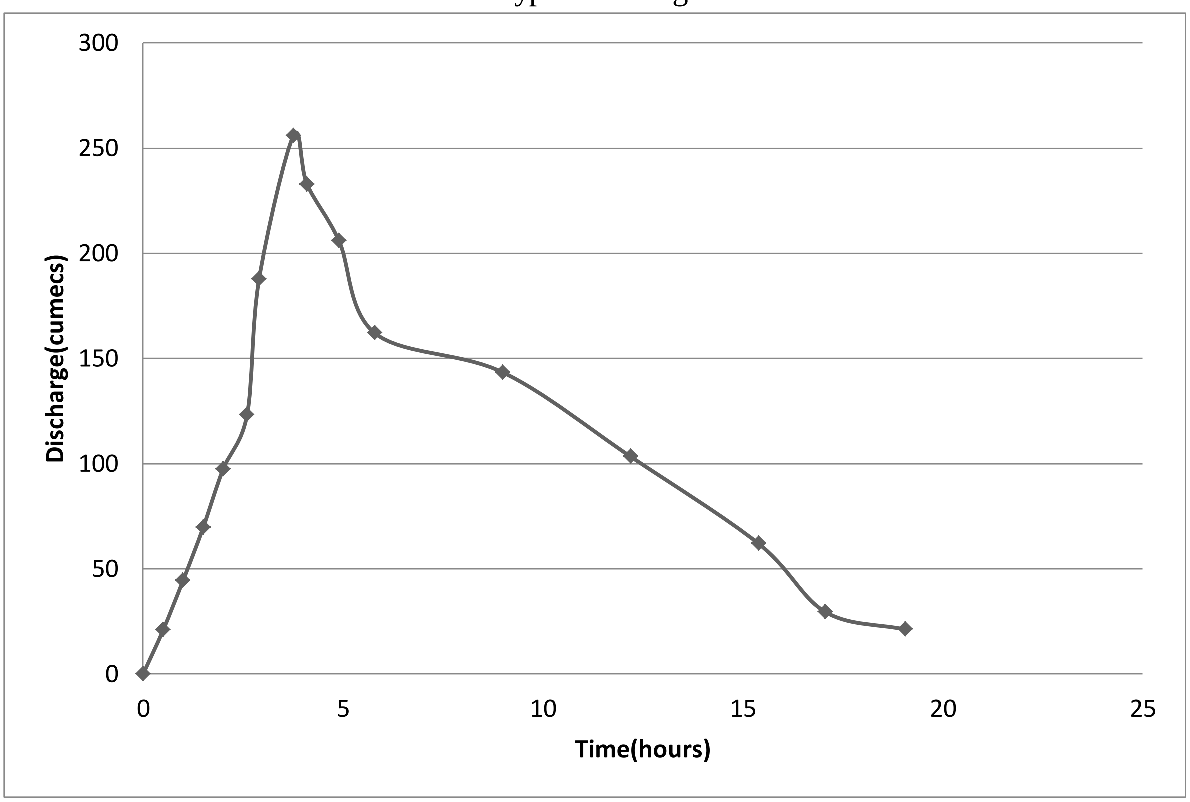

Figure 5.19 Resultant run-off hydrograph of 4 hour duration for NH 58 bypass drainage basin.

Hence Peak Discharge obtained as per resultant Synthetic hydrograph,

Q= 255.334 cumecs.

5.3.2 Peak discharge Calculation as per formula prescribed by CWC for 50 year return period

The parameters required for the calculation of peak discharge by CWC prescribed formula for the NH 58 bypass drainage basin are mentioned below in table 5.21.

Table 5.21 Parameters for the calculation of peak discharge by CWC prescribed formula.

Catchment area, $A(km^2)$	24 hr. point rainfall, R(cm)	Bed slope, S(m/km)	Length of longest stream, L(km)	Catchment Coefficient, K
31	26.67	1.71	7.0	1.65

Hence 50 year return period peak discharge by CWC for NH 58 bypass,

$$Q = K \times A \times R \times \frac{S^{.324}}{L^{.649}} \qquad \text{(from Eq. 3.18)}$$

Q = 252.12 cumecs

5.3.3 Peak discharge calculation from Rational method as per IRC SP 13:2011

The various parameters required in the calculation of peak discharge by rational method are mentioned below in table 5.22.

Table 5.22 Parameters for the calculation of peak discharge by rational method

Catchment area (hectares)	Length of longest stream (km)	Difference between elevation of highest and lowest point of catchment (m)	Point to areal reduction factor, *f*	Time of concentration t_c(hr.) (from eqn	Critical intensity I_c (cm/hr.) (from eqn)
3101	7.0	12.0	0.95	3.446	6.24

Hence, Peak discharge from Rational method as per IRC SP 13:2011,

$$Q_{peak} = .028 f P A I_c \qquad \text{(from Eq. 3.10)}$$

Q_{peak}= 243.77 cumecs

5.3.4 Discharge in a stream calculation as per Manning's equation

$$\text{Discharge, } Q = \frac{A R^{\left(\frac{2}{3}\right)} S^{\left(\frac{1}{2}\right)}}{n} \qquad \text{(from Eq.3.19)}$$

Rugosity coefficient n = .026,

Rectangular channel, Cross sectional Area, A = depth×width,

Width of waterway =276.5m,

Bed slope(S) = 1.74m/km

Hydraulic radius, R= Cross sectional Area/wetted perimeter

The discharge was calculated at various depths and the depth corresponding to the peak discharge obtained is estimated. The discharge at various depths is shown in table 5.23.

Table 5.23 Depth of flow corresponding to discharge in the stream

Depth of flow(d) (metres)	Discharge (cumecs)
2	1409.01
2.7	2535.23
3.2	3075.10
3.5	3567.78

5.3.5 Results and analysis

Peak discharge obtained from various methods are compared and HFL was calculated from the maximum of the three. Table 5.24 shows peak discharge estimated from various methods for NH 58 bypass drainage basin. Q_{max} as mentioned in table 5.24 is the maximum value of peak discharge which was obtained from unit hydrograph method.

Table 5.24 Comparision of peak discharge from different methods

Method	Peak Discharge(cumecs)
Unit hydrograph method	**255.33 ($Q_{max.}$)**
CWC prescribed formula for local region	252.12
Rational formula as per IRC SP 13	243.77

Discharge recorded at the inlet of the catchment in the Solani river stream stream = 2267 cumecs which obtained from the Uttar Pradesh irrigation department website. Hence the total discharge at the outlet of the catchment is obtained by summation of peak discharge contributed by the basin and the discharge at the inlet of the catchment.

$$Q_{outlet} = Q_{inlet} + Q_{max.}$$

$$Q_{outlet} = 2267+255.33 = 2522.33 \text{ cumecs}$$

- The peak discharge obtained 2522.33 cumecs corresponds to the depth of flow of approximately 2.7 metres (from table 5.15).
- The bed level (lowest point in the catchment) obtained from the DEM in the catchment was 250.2m above mean sea level.
- Hence High Flood Level of the region is 250.2+2.7=252.9 m above mean sea level.

From the calculated HFL, the elevation profile of the proposed path was compared and the stretch for which the elevation of pathway is less than high flood level, would be prone to submergence during floods. The stretch for which the path will be submerged is located on google earth image of the study area as depicted in figure 5.20. The white line shows the proposed NH 58 bypass and the part of road under submergence is marked by red line having elevation below HFL (region under submergence during floods)in the figure. The elevation profile of the pathway (which was obtained through digitisation of proposed pathway in google earth) is shown in the same figure(figure 5.20) with shaded region being the region under submergence corresponding portion of pathway marked red. The approximate length of path under submergence during floods comes out to be 3.1 km.

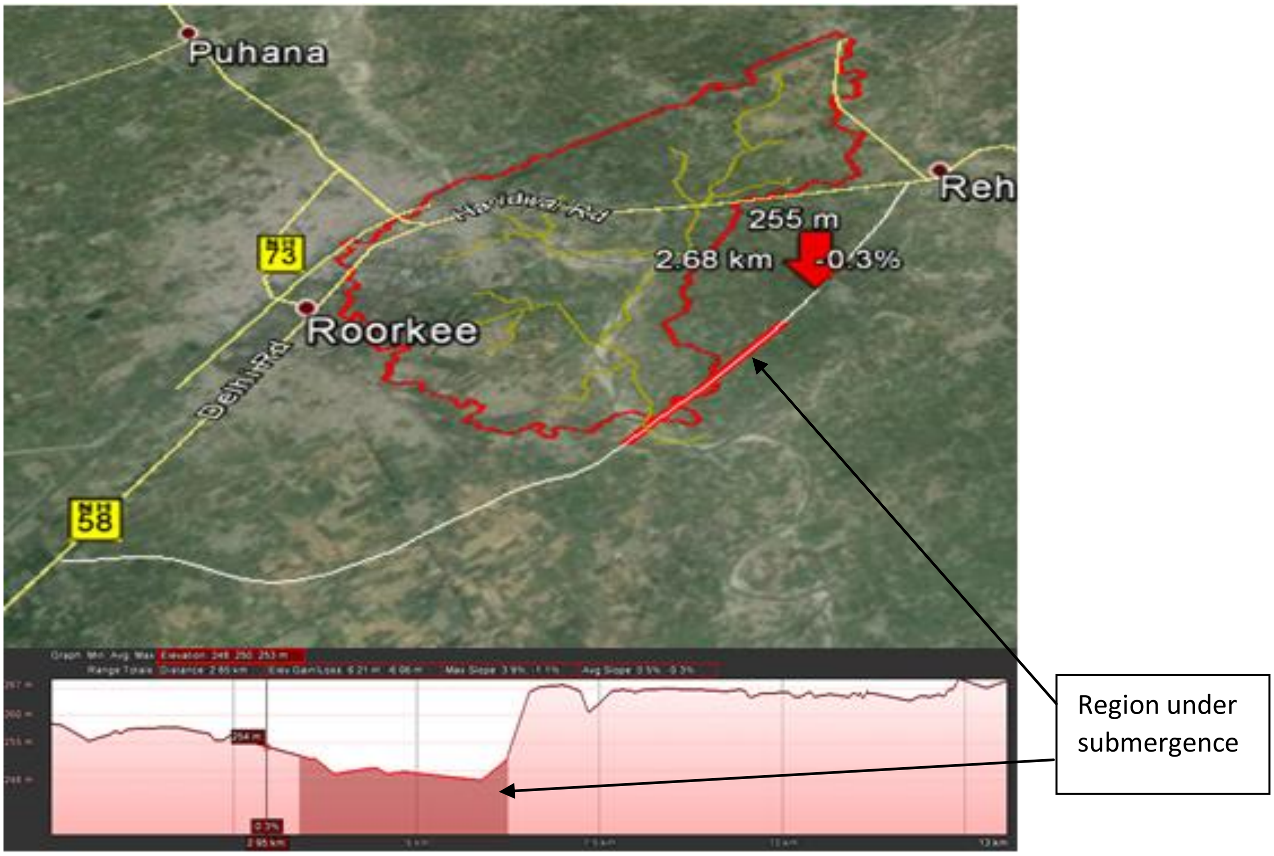

Figure 5.20 Region under submergence shown in google earth image of the study area as well as in elevation profile of the proposed roadway. The roadway marked red indicates region under submergence (NH 58 bypass region).

- For the length of pathway located under submergence elevation level above mean sea level of the subgrade of road above HFL(as per IRC 34:2011) is 252.9+1.5 = 254.4 m.
- Area of water way = width ×depth= 276.5×2.7=746 m^2.
- Minimum vertical clearance as per IRC SP 13:2011 for the discharge 2522 cumecs estimated above is 1.2 m (from table 2.1).
- Hence design bridge opening is,
  (width of the channel)×(depth of flow at HFL+ minimum vertical clearance)
  = 276.5×(2.7+1.2) = 1078.3 m^2.

CHAPTER 6

CONCLUSION

In this study, a methodology was described to estimate regions of road submergence by using the high flood levels and peak discharge for various sub catchments delineated with the help of GIS and to propose suitable measures to avoid submergence of roads during floods.

- This study demonstrates the use of remote sensing and GIS in High flood level calculations and thereby regions of road under submergence.
- Corresponding to maximum rainfall of 50 year return period it was found that a vast amount of area come under submergence for both Laksar area and NH 58 bypass area near Roorkee.
- During floods when water levels in streams flow at high stage that is at high flood level, road stretches for major district road from Laksar to Purquaji which came into submergence were between 3.3 km-7.6 km, 7.8 km-11.2 km, 16.4 km-22 km, 11.7 km-14 km from Laksar to Purquaji covering around 17 kilometres of length as shown in figure 5.17.
- For NH 58 bypass drainage basin proposed road way comes under submergence at a stretch of 3.4 km to 6.26 km from Mangalore to Rehmadpur, covering almost 3 kilometres of length as shown in figure 5.21.
- In order to function efficiently during floods the road stretches which were found to be under submergence during floods required re-design depending upon high flood level of the respective regions as per IRC 34:2011 specifications.
- The size of waterway beneath the road was found to be in proportion with the peak discharge at the outlet of the catchment. In Laksar area the peak discharge for catchment 1,2,3,4 came out to be was came out to be 179.7 cumecs, 357.5 cumecs, 427.3 cumecs, 174.9 cumecs respectively and corresponding area of water way was 64 m^2, 121.5 m^2, 176.4 m^2, 59.38 m^2 respectively, design bridge opening being 123.54 m^2,229.0 m^2,294.6 m^2,115.0 m^2 respectively.
- In NH 58 bypass drainage basin maximum peak discharge obtained was 255.33 cumecs and total discharge at the outlet obtained by adding discharge at inlet of the catchment to the peak discharge was 2522.33 cumecs and corresponding area of waterway observed at NH 58 basin is 746 m^2, design bridge opening being 1078.3 m^2 which were estimated as per the guidelines of IRC SP 13:2011.
- To protect the embankment due to capillary rise of water because of water logging provision of capillary cut-off was required to be provided in the design of road sub-grade as per IRC SP 34:2011. A coarse sand layer of thickness 15 cm is provided as cut off at a distance not less than .6 m below road subgrade level and minimum distance .15 m above HFL (figure 2.2) as per IRC SP 34:2011.

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