FLOOD ESTIMATION
IN
MUMBAI METROPOLITAN REGION

A DISSERTATION

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

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled, **FLOOD ESTIMATION IN MUMBAI METROPOLITAN REGION** in partial fulfillment of the requirements for the award of the degree of **Master of Technology** in Hydrology submitted in the **Department of Hydrology, Indian Institute of Technology Roorkee,** is an authentic record of my own work carried out during a period from July 2007 to June 2008, under the supervision of **Dr. N. K. Goel**, Professor and Head, Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee.

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

Dipankar Chakraborty.

Dated: 30.06.08.

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

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SYNOPSIS

A dependable estimation of flood in terms of their magnitude as well as their occurrence is very much relevant in developing countries like India. Mumbai Metropolitan Region having a large potential for future growth has been suffering from flood threats at different locations. The present study is an attempt to estimate the design flood for the aforesaid area keeping in view the national importance of the same. In order to carryout the regional flood frequency analysis for design flood estimation, the annual peak flow data of 12 gauged sites lying in and around the target area have been used for this study. L-moments approach which is the most advanced and reliable technique in this field has been adopted for the analysis. Pearson Type-3 distribution is found to be best fit distribution for the area and growth factors are derived for different return periods after estimation of best fit distribution parameters. The design floods are directly estimated for gauged catchments as a product of growth factor and mean annual peak flood. A relationship between catchments area and mean annual peak flood has been developed for obtaining the design floods at ungauged catchments of the region. Further more design floods with 25 yrs, 50 yrs and 100 yrs return periods have been estimated for four gauged catchments of MMR with relatively short flow record by Regional Unit Hydrograph Approach, given by Central Water Commission, Govt. of India for the hydrometeorologically homogeneous sub-zone No-5(a). The results obtained from different approaches have been compared and the findings are depicted. The study has been concluded by giving the recommendations regarding the adoption of different approaches depending on various possible circumstances within the area.

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LIST OF ABBREVIATIONS

Symbol	Description
ARF	Areal Reduction Factor
CWC	Central Water Commission
CV	Co-efficient of Variation
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
GEV	General Extreme Value
GIS	Geographic Information System
GLO	Generalised Logistic
GPA	Generalised Pareto
GN	Generalised Normal
IMD	India Meteorological Department
IUH	Instantaneous Unit Hydrograph
MMR	Mumbai Metropolitan Region
MMRDA	Mumbai Metropolitan Regional Development Authority
NERC	Natural Environmental Research Council
PE3	Pearson Type-3
PMP	Probable Maximum Precipitation
PWM	Probability Weighted Moments
ROI	Region of Influence
SDSC	State Data Storage Center
SRTM	Shuttle Radar Topography Mission
SUH	Synthetic Unit Hydrograph
UH	Unit Hydrograph

CHAPTER 1
INTRODUCTION

1.1 GENERAL

Mumbai, the financial capital of India is one of its major industrial & most populous cities. Unprecedented extreme rainfall and flooding events on 26th & 27th July, 2005 in Mumbai have focused the attention of urban planners on urban flood mitigation and management methods. The growth and development of the city due to population pressure, reclamation of land from water bodies without catering to proper drainage of storm water and protection of natural water conveyance channels often leads to extreme flooding of the low lying areas. In the context of Mumbai, very high daily rainfall is quite common during the onset phase of the monsoon resulting severe flooding incident in the city along with the greater Mumbai region and sometime it turns to be a catastrophic kind like 2005 leading to major loss of life and property. One interesting feature of Mumbai city is that the population growth rate of the city is declining significantly which reflects the Government's successful programmes to build a series of new towns in New Mumbai. More over outside of New Mumbai, some towns belonging to greater Mumbai region e.g. Thane, Kalyan are having their population growth rate even higher than that of the other metropolis of India. This fact reveals that the future growth of the other parts of the Mumbai Metropolitan Region (MMR) is inevitable due to the booming expansion of Mumbai City region. In addition to above, the past records of extreme rainfall exhibit that the severity of the condition prevails in MMR as a whole entirely lying in the Konkan coastal belt and will have to be ready in respect of disaster preparedness. The entire Greater Mumbai region including its expansion within the MMR region in a wild fast rate acts like foundation pillar of Indian economy. Due to its manifold eye-catching features like its leading role in the country's international trade, high-tech infrastructure, strategic location in respect of global markets, the region's contribution for facilitating the integration of the country's economy with the rest of the world is highly significant.

From the above discussion it is crystal clear that leaving aside the major disaster, even a flooding incident of a small extent in Mumbai region may have a long range impact on the socio-economic condition of the country. Therefore the flood threats as well as the national importance of the region justify for paying a great attention to the region in terms of its disaster safely and this study is an Endeavour to highlight this problem and to analyze the same with the help of some technical approaches in pertaining to its possible solutions.

The present study mainly concentrates on the estimation of design floods at gauged and ungauged catchments of MMR by adopting different regional approaches. Hydrological regionalisation is concerned with spatial extension of hydrological records and has been used as a standard tool to facilitate extrapolation of hydrological data from gauged sites to ungauged sites. The need for regionalised hydrological data has increased manifold in recent times as most of the catchments either do not have any hydrological record at all or have a record for very short period. The economic considerations, many a times do not justify the detailed hydrological and meteorological investigations at every new site on a large scale and on a long - term basis. The hydrologic engineering evaluation and decisions cannot be delayed for non-availability of systematic records or to obtain longer record. For these reasons hydrologists have resorted to regionalisation, i.e. combining the hydrological data of the gauged sites from a homogeneous zone or region. This leads to a realistic estimation of hydrological parameters, which may be extrapolated to the basins where short records are available or where records are not at all available. Again hydrologists are interested in accurate estimation of some hydrological parameters because of their relative importance (e.g. the magnitude and frequency of rare flood events). Limitations of a single site systematic record are that a sequence may be too short to represent the population of the e adequately. In addition critical values in the records may be subjected to serious err measurement. This has led the hydrologists to search for records outside the at-site syste records to improve upon their estimates. Regionalised hydrological data are used in cases for a more realistic estimation of hydrological parameters.

The need and its appealing traits as stated above justify the application of regionalis approaches in Mumbai region. The specific objectives of the study are pinpointed in section and the approaches adopted are described elaborately in chapter 5.

1.2 OBJECTIVES OF THE STUDY

The objectives of the study are summarized below.

(i) Estimation of flood quantiles with different return periods at available gauged of MMR through at site regional flood frequency approach.

(ii) Establishing a general flood frequency formula for ungauged catchments of M through the regional flood frequency approach.

(iii) Employing the Central Water Commission approach for derivation of Reg Unit Hydrograph as well as estimation of design flood of 25 yrs, 50 yrs & 1C return periods in MMR. In order to this, four gauged sites with relatively flow records have been selected for this study and design flood for specific r periods as mentioned earlier have been estimated.

(iv) Carrying out a comparative study between the results obtained from diff approaches at various sites.

(v) Making general recommendations regarding the adoption of various approach estimation of design flood for the region.

CHAPTER 2

REVIEW OF LITERATURE

2.1 GENERAL

Floods are principal natural hazards in many parts of the world. Flood is perhaps the most discussed hydrological phenomenon involved with a great uncertainty in terms of both magnitude as well as occurrence and when catastrophic in nature either due to specific flooding incident or any unusual sequence of floods, can lead to a great human misery and loss of national property. Therefore it is evident that flood mitigation measure is one of the major findings for the hydrologists as well as a crucial task as the economy is another concern to be dealt with. Flood mitigation measures whether structural or nonstructural, the diagnosis involves with the estimation of design flood. Here the enough reason lies why the estimation of design flood has become such a thrust area in the field of hydrology as well as a fascinating study area for the researchers.

Different methods for design flood estimation for ungauged catchments are based on regional flood frequency analysis and regional unit hydrograph analysis. As the two approaches are distinct and the literatures available for them are also not interrelated, these have been reviewed separately under this study.

2.2 FLOOD FREQUENCY ANALYSIS

In the first part of review of Literature the significant earlier studies related with general flood frequency analysis are briefly documented. In the second part the regional studies are discussed. As the present study concentrates on L-moments technique as to be elaborated in chapter 5, the literatures related with the same are discussed as a separate part.

2.2.1 General about Flood Frequency Analysis

The concept of hydrologic data as independent and identically distributed was given by Gore and Thomson (1909). The note of them shows a clear understanding of the essential randomness in hydrological time series and if represents the dawn of Stochastic Hydrology (Wolf.P.O, 1980).

Todd (1957) discussed the basic principles of frequency analysis of stream flow data and outlined computational procedures.

Due to its large economical and environmental impact, flood frequency analysis remains a subject of great importance and literature related to it is vast and growing. In design and planning concerned with future events whose time or magnitude can not be forecast, we must resort to statements of probability or frequency (Linsley et.al, 1975).

The extreme value distribution is very much relevant with flood frequency distribution. The extreme value type-1 distribution was developed by Gumbel (1941); type-2 & type-3 distributions were given by Frechet (1927) and Weibull (1939) respectively. Three limiting forms of distribution were combined in a single distribution called the General Extreme Value (GEV) distribution by Jenkinson (1955). Historically GEV distributions have been widely used for flood frequency analysis. The flood studies report, U.K adopted the GEV distribution to describe regional flood growth. The log normal distribution was applied in hydrologic time series by Chow (1954), where as Pearson type-3 distribution was first applied in hydrology by Foster (1924) to describe the probability distribution of annual maximum flood peaks. Pearson type-3 distribution is a very flexible distribution assuming a number of different shapes as λ, β & ϵ vary (Bobee and Robitaille, 1977). The frequency factor (k-T) relationship for the distributions with the probability distribution function not being invertible was given by Chow (1951).

The log Pearson type-3 distribution is the standard distribution for frequency analysis of annual maximum flood in the United States of America (Benson, 1968). As per the recommendation of U.S.Water Resources Council the log Pearson type-3 distribution may be used as a base distribution for flood flow frequency studies.

Cunnane (1989) framed a proper definition of return period. Flood peaks do not occur with any fixed pattern in time and magnitude and time intervals between them vary. The return period is the average of these inter event times between flood events.

The plotting positions specify the positions at which particular data points are to be plotted on the frequency axis. Numerous empirical methods have been proposed for plotting position. The first compromising formula was given by Hazen (1930). Weibull formula in this regard is more logical. U.S. Research Council gave the formula which finds its application in log Pearson type-3 distribution. Gringorten (1963) gave the formula for plotting position and generally recommended for General Extreme Value distribution (Cunnane, 1978). Cunnane (1989) compiled the significant plotting position formulas and studied their suitability for different distribution.

The standard errors related with flood frequency studies was first introduced by Kaczmarek (1957). The formulas for the same were given by Kite (1977) for Normal and EVI distributions.

Cunnane (1986) traced an outline of development of flood frequency analysis methods in three phases under selected headings. In early phase if was recognized that there is no such thing as a single design flood but rather a choice of different return period floods depending on circumstances. In middle phase extreme value theory, algebric development of alternative parametric model types and estimation schemes were introduced. In recent phase the models which have been stimulated by the outcome of simulation experiments were developed.

Apart from the methods based on distribution some significant contributions towards distribution free models are notable. A method of estimating the frequency of annual peaks by using monthly peaks was developed by Whisler and Smith (1957). Adamowski (1985) introduced the nonparametric kernel estimation method for flood frequency analysis.

Langbein (1949) first attempted to link between the annual maximum model and peak over threshold model. Kavvas (1982) developed a trigger type cluster model related to peak over threshold method.

Baker (1986) outlined the definition of paleoflood hydrology as the study of past or ancient flow events which occurred prior to direct measurement by modern hydrologic procedures. Benson (1950) introduced graphical procedure for flood frequency analysis by using historical data. Chen et al. (1975) used historical flood data for China. In order to generate information to design numerous major dam projects, the people's republic of China embarked on a national survey of historical flood marks (Chen et al., 1974). Baker et al. (1986) carried out paleoflood hydrologic analysis at ungauged sites of central and northern Australia. Hosking and wallis (1986a, b,) assessed the value of historical information in flood frequency analysis by computer simulation. The usefulness of historical data was also evaluated by Stedinger and Cohn (1985).

2.2.2 Regional flood frequency analysis

Regional analysis can be used to make flood frequency estimates at sites that have no flow records (ungauged sites) or can be used to improve estimates at sites with relatively short records (Tasker, 1986).

Cunnane (1988) enlisted some important methods related to regional analysis as given below.

(i) Station year method

(ii) USGS method

(iii) Methods based on dimensionless moments

(iv) NERC method

(v) Record extension methods

(vi) United states Water Resources Council method

(vii) Bayesian methods

(viii) Method based on standardized Probability Weighted Moments (PWMs)

(ix) Two Component Extreme Value (TCEV) methods

(x) Regional application of Box Cox transformation

(xi)Threshold and censored sample methods

(xii) Simultaneous at site and regional parameter estimation.

Dalrymple (1960) acted as a pioneering role of the regional flood frequency analysis through an index flood technique and is better known as USGS method which reveals that the data at different sites in a homogeneous region follow the same distribution except for scale. Mean annual floods are correlated with the basin characteristics for regional analysis. The ratio of floods of other recurrence intervals to the mean annual flood has usually little variability in a given region and these ratios are defined by long term records of the basic stream flow network. If needed, the ratios may be varied with size of the drainage area or other basin characteristics that have found to affect them in some regions (Dalrymple, 1961). The derivation of regional distribution for estimation of design flood at ungauged catchments as per Index flood method (Dalrymple, 1960) consists of eight sequential steps as given below.

(i) Selection of gauged catchments within the region having similar characteristics.

(ii) Determination of the time base period to be used for the study.

(iii) Establishment of flood frequency curves for data at each gauging sites using EVI distribution probability paper.

(iv) Estimation of the mean annual flood at each station.

(v) Testing of homogeneity of data using homogeneity.

(vi) Establishment of relationship of mean annual flood and catchment characteristics usually drainage area at each station.

(vii) Ranking of the ratios of selected return period flows to the mean annual flood at each station.

(viii) Computation of median ratio for each of the selected return period of step (vii), multiply the estimated mean annual flood of the gauged catchment and plot them against return periods on Gumbel's probability paper, which gives the flood frequency curve at ungauged catchment.

Cunnane (1988) reviewed these methods and concluded that the index flood method with a regional wakeby distribution is the best available regional procedure.

The index flood technique was also adopted by Natural Environmental Research Council (NERC, 1975) in Flood Studies Report (FSR), for regional flood frequency analysis in U.K. in a different manner. In the FSR, the index flood is used to link the flood frequency and regional quantile i.e. the flood frequency curve is obtained by multiplying the index flood and the regional quantile. The mean annual flood QBAR was used as the index flood. The best fit equation for QBAR with catchment descriptors is determined by multiple regression method. Regional quantile is estimated as (Q_i/ QBAR) by plotting against different return periods. The FSR framed the detail catchments descriptors for regional flood frequency studies at ungauged catchments. A method was presented for estimating the mean annual flood for catchments without flow data by means of a multiple linear regression of mean annual flood on catchments characteristics (Miller and Newson, 1975, Flood Studies Conference on FSR). The FSR divided the entire U.K. in to several areas. For all areas except three, six variable equations were recommended. These six variables were AREA, STM FRQ (Stream frequency), S1085 (stream slope), SOIL (Soil Index), RSMD (Net one rainfall with 5yrs return period) and LAKE (An index of lake).

Flood Estimation Handbook (1999) followed a slightly different approach to the FSR for flood frequency analysis in U.K. The main difference lies in that the index flood used in the FEH is Median Annual Peak flood (QMED) with the replacement of QBAR. With the advancement of the age, the catchments descriptors have been measured digitally. Another significant difference is that, a single equation has been derived in the FEH to describe all the rural catchments of the U.K. (FEH, vol-3, 1999).

One of the most useful methods of regional flood frequency analysis is regional regression of flood characteristics on basin characteristics. The significant earlier studies were carried out by Potter (1957), Benson (1962), Mc.Cuen et al. (1977) and others. Benson (1962) used the ordinary least square (OLS) regression in his regional study. OLS estimators are proved to be appropriate and statistically efficient if the flow records are equally reliable, the natural variability of flows at each site is the same and concurrent flows at any pair of stations are independent (Tasker, 1986). Stendinger and Tasker (1985) developed a method of estimating the regional regression parameters that takes into account the length of record available at gauged sites and the between-site cross correlation among concurrent flows. They used a generalized least square (GLS) regression and pointed out the merits of GLS method over OLS method as,

(i) Provided estimates of regression parameters with smaller mean square errors than the OLS estimates.

(ii) Provided relatively unbiased estimates of the variance of the regression parameters compared to OLS.

(iii) Provided a more accurate estimate of the regression model error than OLS.

Rossi et al. (1984) developed the Two Components General Extreme Value (TCEV) regionalization procedure based on a four parameter distribution for annual maximum floods. The study was carried out with Italian flood series where they observed that Italian rainfall and flood series contain occasional outliers due to unusual synoptic and catchments conditions. They modeled the occurrence of maxima by two parallel Poisson processes with exceedence rates λ_1 and λ_2 for the basic and outlier events. The model assumes that flood peaks of extreme events and ordinary events come from different distributions. On this basis they derived a cumulative distribution function for the annual maximum distribution coming from different parent distributions. The resultant distribution is the maximum of two EVI distributions. Arnell and Beran (1986) tested the suitability of TCEV distribution for regional flood estimation in U.K. Its fit to UK regional and national flood data displays other desirable features; it has good ability to encompass outlying floods and to replicate observed sample skewness (Arnell and Beran, 1986). There is evidence that the annual maximum population distribution in some countries is thick tailed. Traditional distributions may not be able to mimic that behavior, leaving only wakeby and TCEV distributions as genuine contenders (Cunnane, 1986).

Probability Weighted moments (PWMs) were first given by Greenwood et al. (1979). The at-site sample PWMs are calculated first by the formulas given by Greenwood et al. (1979) as,

$$M_{1,r,0} = \frac{1}{N}\sum_{i=1}^{N} W_i^r Q(i,N)$$

$$M_{1,0,S} = \frac{1}{N}\sum_{i=1}^{N} (1-W_i)^S Q(i,N)$$

Where, Q (i, N) represents sample values ranked from the smallest to the largest by using plotting position formula as,

$$W_i = \frac{(i-0.35)}{N}$$

The general formula consists of the symbol $M_{p,r,s}$ where, p, r & s are real numbers; when r & s are equal to zero, the moment represents the conventional moments of order p. The PWMs method of parameter estimation consists of deriving expression for $M_{1,r,0}$ or $M_{1,0,S}$ in terms of the parameters of the assumed parent distribution. Equating as many of these as there are unknown parameters, to sample PWM values calculated from data and solving the resulting equations, parameters of the unknown population can be obtained. This procedure is similar in structure to the estimation by ordinary moments but results in parameters estimates, which are superior to those obtained by moments (Landwehr et al., 1979).

For regional flood frequency analysis, Wallis (1980) proposed that at-site values of PWMs be standardized with division by the at-site mean $M_{1,0,0}$ and the resulting standardized values be averaged across the sites in the region. Each site contribution is weighted according to its record length. These regional weighted average values can be used to estimate parameters of distributions which are invertible in nature.

Cunnane (1986) recommended that regional index flood estimation techniques which pool all standardized statistics in a region together in order to estimate the parameters of a standardized distribution, particularly those based on PWMs are generally more efficient than at site estimators which are based on single sample.

The performance of the regional flood frequency analysis strongly depends on identifying of homogeneous regions or grouping of sites into homogeneous regions.(U.S. National report to I.U.G.G, 1991-1994). The earlier prominent method for identification of homogeneous regions was USGS method originally used by Dalrymple (1960). In this method the ratio of the 10 yr flood to mean discharge is used to measure homogeneity of the discharge data from gauged catchments. For each station of the region the return period (T) corresponding to a discharge equal to the regional average flood ratio times the mean flood is computed. An analysis of the record years of at-site data gives a Q-T relationship and T value for each case is determined. If T value falls outside the theoretically determined confidence level, the site is rejected with respect to the region.

Wiltshire (1985) divided the entire group of catchments into two regions initially based on any of the variables like catchments area, wetness of basins or annual average rainfall. Subsequently four way partitions on two variables were performed. Measures of the variability of the observed annual maximum data from the regional distribution were given as SSQ5. Wiltshire (1986) further used co-efficient of variation (CV) instead of SSQ5 to perform a systematic search along catchments characteristics axis and introduced CV based test. Wiltshire (1986) applied this procedure to the data of 376 sites in U.K. and found five significantly different regions which are internally homogeneous with respect to CV at various significant levels.

The attempt for identifying flood regions not in geographical space but in a data space defined either by flood statistics or catchments characteristics was made by Mosley (1981). Mosely used Cluster analysis to form groups of basins characterized by specific mean annual flood and the co-efficient of variation (CV). He performed this analysis with data of some selected catchments of New Zealand.

Acreman and Sinclair (1986) proposed a method of identifying homogeneous regions by using a Clustering algorithm depending on the catchments characteristics in Scotland.

Wiltshire & Beran (1986) proposed multivariate techniques for the identification of homogeneous flood frequency regions. One of the major requirements for the successful identification of homogeneous regions is a method of grouping hydrologically similar basins into homogeneous regions. Homogeneous collections of basins can be formed by Clustering in a bivarate data space of co-efficient of variation and specific mean annual flood. The allocation of an ungauged basin to a cluster can be achieved through the application of multivariate discriminate analysis to the basin characteristics data within the cluster (Wiltshire & Baran, 1986).

Wiltshire & Beran (1986) developed a significant test for regional homogeneity based upon the non exceedence probabilities of individual flood maxima which are termed as G-points. The power of the test is investigated in relation to region size and distribution choice. This test was used to investigate the homogeneity of the geographical region adopted in flood studies report of U.K.

Fiorentino, Gabriele, Rossi & Versece (1986) proposed a hierarchical approach for regional flood frequency analysis, which is characterized by identification of homogeneous regions wherein the skewness co-efficient is assumed not to vary from site to site and identification of homogeneous sub-region wherein the co-efficient of variation is assumed to be constant too.. The approach adopted the use of TCEV distribution and the validity of the approach was demonstrated using 28 Italian annual flood series. The hierarchical approach exploits, better than the classical index flood method, the hydrologic information (Fiorentino et al, 1986).

Burn (1990) and Zrinji & Burn (1994) developed one of the most significant approaches regarding the regionalization of area and popularly termed as Region-of-Influence method. In this method the site of interest is located at the centre of gravity in a space of relevant flood or catchments characteristics, each weighted properly according to its relevance. The method also involves the choice of a distance threshold (Burn, 1990); only sites whose distance to the target site does not exceed this threshold are included in the region of influence. Zrinji & Burn (1994) replaced the somewhat subjective choice of threshold with a statistical test. The advantages of region of influence were discussed by using data from New found land, Canada (Zrinji & Burn, 1994). A hierarchical feature was subsequently introduced as a refinement to the ROI (Zrinji and Burn, 1996).

Flood estimation handbook, U.K. (1999) used ROI method for forming the pooling groups. It is a flexible method in which the pooling group is specifically tailored to the site of interest (FEH, 1999).

"Regions are dead, long live pooling groups"- the subheading was inspired by Acreman and Wiltshire (1989) as they recommended for pooling the gauge data on the basis of hydrometeorologically homogeneous region irrespective of fixed geographical location for obtaining the pooled growth curve instead of regional quantile estimates.

It has been suggested (Jakob et al., 1999) that a region should contain 5T station-years of data in order to provide an effective estimate for the flood event with a return period of T years. As the size of a region is increased, there is a tendency for the homogeneity of the collection of catchments forming the region to decrease. There is thus a trade-off between the first and third required characteristics for a region. FEH (1999) also supported the statement and adopted this rule in the pooled frequency analysis.

Flood magnitudes and return periods may be estimated by the frequency analysis of continuous synthetic flow data generated by a rainfall runoff model. This approach has been demonstrated by number of authors using various hydrological models (see e.g. Brass et al. 1985; Beven, 1987; Bradley and Potter, 1992; Blazkova and Beven, 1997; Cameron et al. 1999; Lamb, 1999). Use of model with continuous water balance accounting enables the representation of dynamic factors affecting runoff implicitly (Lamb 1999). This also avoids the base flow separation as in case of event based modeling. By using a continuous rainfall series (either observed or generated via a stochastic rainfall model), running a hydrological model continuously and going on to analyze the flood peaks of the simulated flow series, it is no longer necessary to make the assumptions behind the design event approach or simplifying assumptions of derived distribution approach pioneered by Eagleson (Cameron et al. 1999).

Bradley and Potter (1992) proposed peak to volume approach for making flood frequency estimates using flows simulated by rainfall runoff model. The approach is based on separately estimating a probability distribution of runoff volume and a distribution of peak discharge conditioned on volume. The peak to volume approach appears to be promising method for making physically reasonable estimates of flood frequencies for hydrologic analysis and design. This was demonstrated by Goel (1990) and Goel et al. (1998) through multivariate stochastic modeling of flood flows of river Narmada at Garudeshwar (India). Blazkova and Beven (1997) applied a newer version of stochastic rainfall simulator (Beven, 1987) with TOPMODEL and applied it to three small catchments of Czech republic. Lamb (1999) uses probability distributed model (PDM) of Moore (1985, 1993) and his study advances previous work using the continuous simulation method by exploring the model calibration problems.

Cameron et al. (1999) explored flood frequency estimation for a gauged catchment through the use of continuous rainfall runoff simulation (using TOPMODEL) within a uncertainty framework that avoids the idea that there is an optimal set of model parameters values. It has been established by the study that within current hydrological modeling limits, parameter sets could be found that were acceptable for both flood frequency estimation and hydrograph simulation for the study catchments. This opens up several avenues of research, notably with respect to uncertainty in flood frequency estimation and the question of consistency of model parameterizations for both continuous flow series and flood frequency simulation.

Goel et al. (2001) furnished the recent advances in flood frequency analysis especially in the decade of nineties in a very systematic manner. They documented the different approaches involving deterministic and/or hydraulic components for flood frequency estimation. The regional flood frequency formulae and the regional unit hydrograph relationships for various sub zones of India have been summarized. The review of literature depicted by them indicates that the future studies in the area of flood frequency analysis should include physics of flood phenomena also besides the refinement of existing statistical methods.

2.2.3 'L'-moments based regional flood frequency analysis

The most eye-catching paper regarding flood frequency analysis was published by Hosking (1990), where the L-moments were introduced. Since then the L-moments methods have been proved as a unique method in regional flood frequency analysis and unanimously accepted by the researchers and hydrologists. It can be shown that L-moments are linear function of PWMs and hence for certain applications, such as the estimation of distribution parameters, serve identical purposes (Hosking, 1986).

The L-moments calculation proceeds via estimation of PWMs and was shown in the book published by Hosking and Wallis (1997). An alternative calculation procedure for sample L-moments was presented by Wang (1996).

The advantages of L-moments over PWMs (Hosking, 1990) have been discussed in the methodology chapter under this study. Vogel and Fennessey (1993) framed the manifold advantages of L-moments technique. Fill and Stendinger (1995) compared the relative performance of different tests for identifying homogeneous regions and concluded that L-moments technique is always powerful than other tests.

Hosking and Wallis (1997) proposed the discordancy measure and heterogeneity measure for checking the homogeneity of the region.

The goodness of fit measures for selecting the best-fit distribution for flood frequency curve was given by Hosking and Wallis (1997). In this context the L-moment ratio diagrams were given by Hosking (1990). Some useful relationships for constructing the L-moment ratio diagrams for some common distribution were given by Hosking (1990 &1991).The method of moments for distribution fitting can give poor results when data are strongly skewed because sample estimates of Skewness become unreliable (Hosking & Wallis, 1997).

Flood Estimation Handbook, U.K. (1999) adopted L-moments approach for flood frequency studies in U.K. The FEH L-moments approaches are slightly different from classical L-moments. Both the approaches give flood frequency curves that are identical except for a scaling factor. This scaling factor corresponds to the ratio of the fitted median to Median Annual Maximum Flood (QMED) or equivalently, as the ratio of QBAR to the fitted means (FEH, 1999).

In India regional flood frequency studies based on L-moments technique have been carried out for different basins. Mostly the studies are concerned with fixed geographical region concept; the studies related to pooling group's concept are still in an infancy stage in the country.

One of the effective studies was made by Jaiswal et al. (2002) on L-moment based flood frequency modeling in the Beas basin, which belongs to Indus river system and frequently affected by flash floods. The best fit distribution for flood frequency was found to be GEV distribution and separate formulas were established for quantile estimation at gauged & ungauged catchments.

Kumar & Chatterjee (2005) proposed flood frequency formulas for gauged and ungauged basins of North Brahmaputra region of India. The GEV distribution was found to be best fit distribution after meeting the homogeneity criteria of the region. When data of all sites were considered without meeting the criteria of regional homogeneity the best fit distribution was found to be Pearson type-3 distribution.

2.3 REGIONAL UNIT HYDROGRAPH ANALYSIS

The first conceptual approach of the development of regional unit hydrograph (U.H.) may be recognized as Bernard's approach (1935) and since then an appreciable number of approaches have been reported in the literature. On the basis of the basic approaches towards the development of regional U.H. it can be broadly classified into two categories namely Synthetic Unit Hydrograph (SUH) approach and Instantaneous Unit Hydrograph (IUH) approach which are discussed in the following sections.

2.3.1 SUH Approach

The stream flow and rainfall data are available in a few locations on a stream or in a basin where the U.H. can be developed from direct rain fall runoff relationship. In order to develop the unit hydrograph in other locations of the stream or in near by catchments, empirical equations of regional validity relating the U.H. characteristics to basin characteristics are derived. This kind of approaches is called synthetic unit hydrograph approach. There are three types of synthetic unit hydrographs: 1) those relating hydrograph characteristics to Watershed Characteristics, 2) those based on a dimensionless U.H. and 3) those based on model of watershed storage (Chow et al.).

Snyder (1938) analyzed a large number of watersheds located in the Appalachian Mountain Region in U.S.A ranging in size from 30 to 30,000 sq km and found synthetic relations for standard unit hydrograph parameters. Snyder selected three parameters for the development of U.H. namely base width (T), peak discharge (Q_P) and basin lag (t_p) and derived empirical relationships for them. Standard U.H. was defined as one with a rainfall duration of t_r which is related to the basin lag t_p as,

$$t_r = \frac{t_p}{5.5}$$

And

$$t_p = C_t\,(LL_c)^{0.3}$$

Where L & L_c are the length of the main stream and the distance from the outlet to a point on the stream nearest to the centroid of the watershed respectively. Peak flow Q_P is given as,

$$Q_p = C_p \frac{A}{t_p}$$

Time base is given as,

$$\mathrm{T} = 3 + 3\left(\frac{t_p}{24}\right)$$

C_t and C_p are the empirical constants with a different range of values as given by Snyder. He considered that the shape of the U.H. is likely to be affected by the basin characteristics like area, topography, drainage density, shape, slope and channel storage. It is worth noting that the parameters like L, L_c introduced by Snyder are still relevant and important for derivation of regional U.H.

Linsley et al (1958) developed synthetic U.H. by giving a new expression for time tag in terms of basin characteristics (L, L_c & slope). They introduced one co-efficient for determination of time tag in U.S. and assigned different values of it with respect to topographical features of the region.

The Snyder's approach was further modified with the inclusion of two additional parameters by U.S. Army Corps of Engineers (1959). These parameters are the widths of the U.H. at 50% and 75% of the peak flow ordinate respectively (W_{50} & W_{75}).

Taylor and Scwarz (1952) analyzed 20 drainage basins of size 50 sq km to 4000 sq km in the north and middle Atlantic States in U.S.A and gave the empirical relationships for C_t and time base. Espey et al (1977) further innovated in the uses of Snyder's method by developing a set of generalized equations for the construction of 10 minutes U.H. They studied with 41 catchments located at different states of U.S. with a different sizes and impervious percentages. The approach was proved to be very effective for their study region with having a better shape of U.H.

The dimensionless U.H. is a form of Synthetic U.H. where discharge is expressed by the ratio of q to peak discharge q_p and the time by the ratio of time t to the time to peak t_p. From dimensionless U.H. of gauged basins, the peak flow and the shape of U.H. can be derived for ungauged basins located at same hydrometeorologically homogeneous region. Thus the dimensionless U.H. eliminates the effect of basin size. A typical study of dimensionless U.H. was carried out by Soil Conservation Service, U.S. (1972) and is well known as SCS dimensionless U.H. method. They used a large number of U.H. resulting from different watersheds from different locations and having different range of areas. They suggested that the time of recession may be approximated as 1.67 t_p irrespective of the size and shape of the catchments.

Natural Environmental Research Council (1975) represented the dimensionless 1hr triangular U.H. for rural as well as urbanized catchments areas. The U.H. was completely defined in terms of time to peak (t_p).

In India the Central Water Commission (CWC) developed Regional U.H. relationships in the preparation of flood estimation reports under short term and long term plan. Under short term plan CWC (1973) derived the empirical relationship for estimating the design flood utilizing hydro-meteorological data available for 60 bridge catchments spread through out the country. The method was recommended for the catchments with area ranging from 25 to 500 sq km.

Under long term plan of CWC, the entire country was divided into 26 numbers of hydrometeorologically homogeneous sub-zones. For preparing the flood estimation report, systematic collection of hydrometeorological data was carried out with the co-operation of the ministry of Railway and the ministry of the Transport, Govt. of India. Synthetic unit hydrograph relationships were established for different sub-zones and were recommended for the catchments ranging from 25 to 5000 sq km in area. The details of this approach are discussed in chapter 5.

2.3.2 IUH Approach

Instantaneous Unit hydrograph (IUH) is a fictitious conceptual unit hydrograph which represents the surface runoff due to an excess rainfall of unit amount with an infinitesimally small duration. The concept of IUH was first proposed by Clark (1945). This cannot be realized in actual watersheds but it is useful because the IUH characterizes the watershed's response to rainfall without reference to the rainfall duration (Chow et al).

Clark (1945) proposed a model where the instantaneous unit hydrograph is derived by routing the time area diagram of the catchment through a single linear reservoir. The time of concentration has been defined as the time that elapses between the instants when the rainfall excess ends to the second point of contraflexure of the hydrograph. The model requires knowledge of T and K in addition to the time area diagram of the catchment, which can be derived using topographical characteristics.

O'Kelly (1955) suggested that Instantaneous Unit Hydrograph could be obtained by routing an isosceles triangular inflow of the unit volume and having a base width of T hours through a single linear reservoir having a storage coefficient of K hours. Therefore T and K are the two parameters, which describe the shape of the instantaneous unit hydrograph based on O'Kelley's approach.

Nash (1957) derived the IUH by routing the unit impulse input through 'n' linear reservoirs of equal storage coefficient 'K'. Therefore the two parameters 'n' and 'K' define the complete shape of the IUH.

A very powerful approach of development of instantaneous unit hydrograph based on geomorphologic and geomorphoclimatic factors is gaining tremendous importance now days. This is an attempt to discover simplifying principles of geomorphological equilibrium that produce simple regularities at the catchment scale. The basis of this is followed by the studies on quantitative geomorphology by Horton (1945). The quantitative analysis of drainage networks led the way for a theoretical foundation of Horton's well-known empirical laws and provided a new perspective for many other problems in fluvial geomorphology. Thereafter, an attempt was made by Rodriguez-Iturbe and Valdes (1979) for a unifying synthesis of the hydrological response of a catchment to surface runoff and it is attempted by linking the (IUH) with the geomorphologic parameters of the basin.

CHAPTER 3
STUDY AREA

3.1 GENERAL

The present study area is Mumbai Metropolitan Region (MMR), the administrative boundary of which was demarcated by the Mumbai Metropolitan Regional Development Authority (MMRDA). MMRDA was originally set up on 1st March, 1975. The type of analysis related to this study needs a clear picture about the regional settings of the area which has been depicted in this chapter.

Further this is a hydrological study which is basically the study of nature's water cycle through various phases and scientific procedures. The kind of study in any region requires a thorough idea about the meteorological behavior as well as the hydrological setting of the same. These are collectively termed as hydro-meteorological characteristics and are discussed in this chapter for MMR.

In any hydrological study the quantative study of the surface land form is used to arrive at measures of geometric similarity among watersheds, especially among their stream networks (Chow et al., 1988). This study is called geomorphology and is extremely important when catchment related studies are carried out. This chapter also covers a brief description of the fundamental concepts about river system and the river network of Mumbai Metropolitan Region.

3.2 THE REGION

The manifold aspects of the regional settings for MMR are picturised in the following subsections.

3.2.1 Geographical settings

The Geographical extent of the region is between 18° 33″ and 19° 31″ north latitude and between 72° 45″ and 73° 28″ east longitude. The boundaries of MMR are marked as Vaitarna Creek and Tansa River in the north, foothills of Sahyadri in the east, administrative boundaries of Pen and Alibag tehsils of Raigad district in the south, and Arabian sea in the west. The area is narrower in its north east portion and the boundary of the same is defined as the administrative boundaries of Kalyan and Bhiwandi tehsils of Thane district.

The entire region extends over an area of 4355 sq km as recorded by Mumbai Metropolitan Region Development authority (MMRDA). The region consists of Mumbai City District, Mumbai Suburban District, part of Thane district and part of Raigad district.

From the Hydrological point of view the entire region lies within the same hydro-meteorologically homogeneous sub-zone no-5(a) namely Konkan coast sub-zone as defined by the Central Water Commission, Govt. of India. The area under sub-zone no-5(a) also indicating the location of MMR within it is presented in Fig. 3.1.

3.2.2 General topography

The region lies in North Konkan and in the west of Sahyadri hills. The region has got wide variation in terms of elevation. In west the coastal belts elevation ranges from 0 to 100 m. In one hand the region comprises of an appreciable number of low land areas and in the other hand the hills having a mean elevation of more than 300 m are also present, giving dual topographical features of the area.

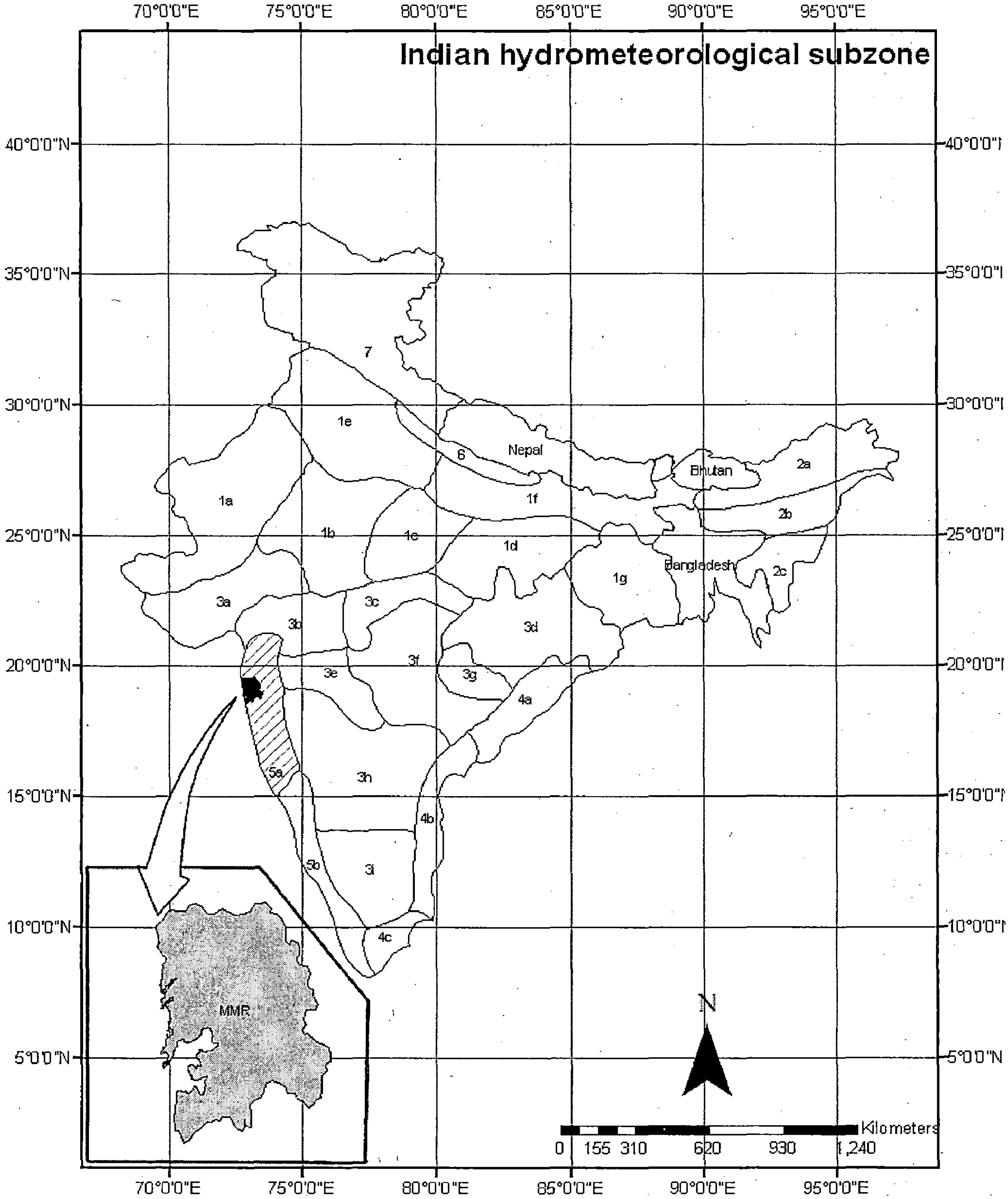

Figure 3. 1 MMR within hydrometeorologically homogeneous sub- zone no 5 (a) of India

3.2.3 Land use

The existing land use plan of Mumbai Metropolitan Region has been prepared by MMRDA in three categories of level among which the level-(i) shows the six major land use classes while the level-(ii) and level-(iii) depict the other details. As per level-(i) category the classes are Urban built up land, Agriculture, Forest, Waste Land, Coastal Wetland and Water bodies. The existing land use pattern at level-I category is shown in the fig. 3.2. The built up portion accounts 12.68% of the total area of the region. Most of the region's urban growth was confined to the greater Mumbai region till 1968 but in post 1968 period the growth took place in Thane, Kalyan, and Navi Mumbai along with the Suburban region. The built up region is divided into above two parts as pre 1968 & post 1968 period as shown in the map. The urbanization mostly occurred by acquisition of agricultural land and by reclamation of wetland. Despite rapid urban sprawl, the agricultural area is still the maximum and accounting 42% of the total area. The forest area is 27% of the total area and some of the sparse forest region has undergone with urbanization, but percentage wise the figure is insignificant. The other two categories namely coastal wet land and water bodies account for 8.45% and 2.80% of total area respectively.

In any hydrological study the impact of urbanization on the change of hydrological behavior within the region, presence of water bodies, densely populated area etc, are very relevant and this justifies the necessity of knowing land use pattern for any region before carrying out the study.

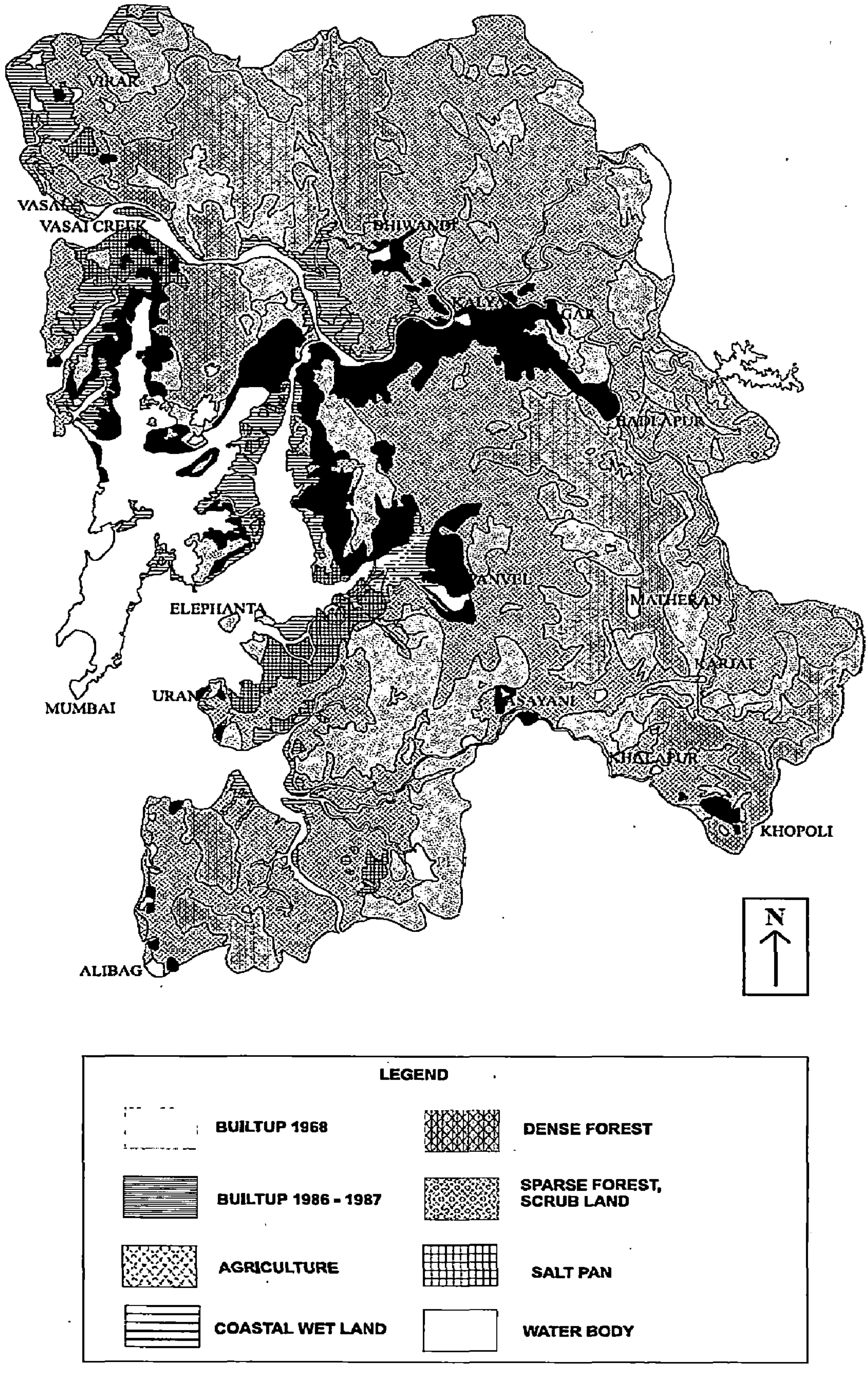

Figure 3. 2 Land use pattern of MMR at level-I category (source- MMRDA)

3.3 METEOROLOGICAL CHARACTERISTICS

3.3.1 Temperature

The region is neither too hot in summer and nor too cold in winter and maintains a moderate range of temperature. The presence of Arabian Sea minimizes the fluctuations in temperature but also responsible for the high relative humidity of the region. In general the mean daily minimum temperatures in the region is 17° C in winter and mean daily maximum temperature is 34° C in summer and maximum relative humidity reaches up to 95%.

3.3.2 Rainfall

The south-west and north-east monsoon causes the rainfall in the region. The monsoon generally appears in the month of June and continues up to early October. The normal annual rainfall varies from 2000 mm to 5000 mm within the region. Matheran in Raigad district receives maximum normal annual rainfall which is more than 5000 mm. The interior part of the region receives more precipitation in comparison with the coastal part of the region. The heaviest rainfall month is July as recorded by almost all the stations. It clearly appears with the normal annual rainfall values, that the region lies in the heavy rainfall zone. But when the flood and its protection are concerned, the local intensity of the precipitation or the maximum daily precipitation is more relevant, and for Mumbai region the later is more severe in nature. This fact is reflected from the instance of the heavy downpour conditions witnessed on 26th July, 2005 in Mumbai city with a maximum precipitation recorded as 1011 mm in Vihar lake, which indicates one half of the normal annual rainfall value has been received in a single day.

The very heavy rainfall criterion as per Indian Meteorological Department (IMD) is a rainfall of more than 130 mm a day. As per this very heavy rainfall in Mumbai region is almost the event of every year.

The extreme rainfall events do occur in nature rarely. The extreme rainfall criteria as per Indian Institute of Tropical Meteorology Pune are the precipitation values exceeding .the threshold value which is defined as Normal rainfall + 5 times the standard deviation. Another concept is the Probable Maximum Precipitation (PMP) was estimated by IIT, Mumbai for Mumbai Region, which implies the theoretical upper limit of one day precipitation. The Central Water Commission also presented the 24hr. rainfall at different return periods in the isopluvial maps prepared for hydro- meteorologically homogeneous sub-zone no-5(a). All the above figures computed for the extreme rainfall events were exceeded by the 2005, 26th July rainfall. The 1989 one day's rainfall event at Raigad district also exceeds the other values except PMP value.

Hydro meteorologists studying the Indian phenomenon detected that the Mumbai downpour of 26th July, 2005 was the result of a combination of synoptic scale weather systems which have a span of 1000-2000 km and the Meso scale system localized over 20-30 km. The above conditions for the extreme event could be met in future also. Moreover the global warming studies by Intergovernmental panel on climate change (IPCC) have concluded that the extreme rainfall events are likely to be more frequent in many areas of India. In addition to that a rise in sea level may occur due to increased snow melting processes caused by Global Warming. All the above discussions reveal that the entire MMR is to be prepared as well as protected from the calamitous condition caused by extreme rainfall events.

3.4 HYDROLOGICAL FEATURES

Some of the important hydrological features of MMR are discussed below.

1) The natural hydrological process of any region is greatly affected by its urbanization and hence careful attention should be paid on rainfall - runoff relationships in this area. Moreover the run off response of the area also depends on the level and extent of the

urbanization. The runoff is greatly influenced by the topography, soil type, vegetative cover of the area.

Due to urbanization paved area of the region increases which results into the reduction of vegetative cover. The effect of this is the low rate of infiltration and evapo-transpiration which ultimate the increased runoff responses of the area. Many a times the natural slope of the area is also altered which also influences on the runoff pattern of the area.

As per the recommendation of Brihanmumbai Storm Water Drainage (BRIMSTOWAD), the co-efficient of runoff for built up portion of Mumbai is nearly equal to one which implies that almost entire portion of precipitation is converted into runoff in these areas. Even besides the urbanized portion, the Konkan area as a whole is having a run-off co-efficient of around 0.75, which is much higher in comparison with other basins of India.

2) The time of concentration of a catchment is the time taken by the flow of rain water from the remotest point to its outlet. This parameter is very important for the estimation of the flood. In small urbanized catchments the time of concentration is very low, which results in the reduction of the time to peaks. Moreover, in hilly catchments of MMR, due to high natural slope, time of concentration is also less. A large catchment with flat terrain has got a higher value of time of concentration. It also depends on the shape of catchments as more in case of oblong catchments and less in case of fan shaped catchments. All these factors are to be kept in mind while estimating the flood or in its mitigation approaches.

3.5 STREAM NETWORK

3.5.1 An overview

The quantitative study of stream network was originated by Horton (1945). The study develops a system for ordering stream networks and imparts the concept regarding the catchment shapes, drainage density etc. with the help of number & length of streams of different order. Later the stream ordering system was slightly modified by Strahler (1964).

As per ordering of stream network the finger tips streams are given order one. Generally these are the smallest recognizable channel in a stream network system. When two streams of same order join the resulting downstream is designated as higher order. If a stream of lower order joins the higher order, the resulting stream follows the order of the higher of the two.

The Bifurcation ratio (R_B) for any stream network is the ratio of the number of streams in an order to the number of the streams in the next higher order. Horton (1945) found that for different order the Bifurcation ratios are nearly equal for a given stream network. This is known as Horton's law of stream numbers. The theoretical minimum value of Bifurcation ratio is two. The catchments having R_B values as 2 to 3 are generally fan shaped catchments, those having the values from 3 to 5 are normal shaped catchments and the higher values of the same indicate the oblong catchments.

Another vital concept of stream network system is drainage density and was given by Smart (1972). The drainage density is the ratio of the total length of the stream network of a catchment to its drainage area. This is a very important physiographic parameter of the catchments as the runoff responses are significantly more with the higher values of same.

3.5.2 Stream network of MMR

The river system of MMR consists of five major rivers along with their numerous tributaries. The rivers are all west flowing rivers and drain the entire region into the Arabian Sea. The river system of the region can be divided into three categories according to their Geographical location and is discussed below. The river network of MMR is shown in fig. 3.3.

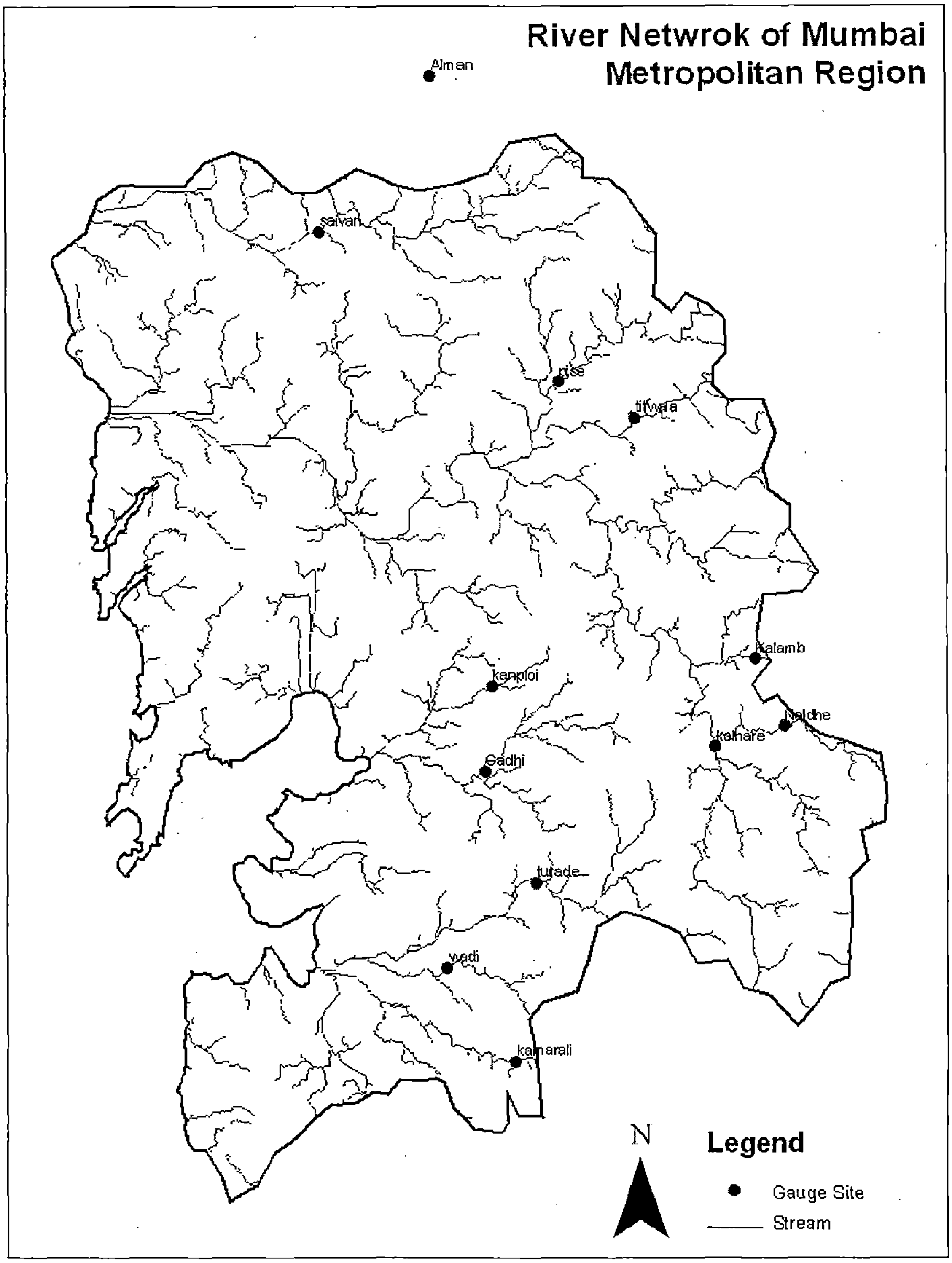

Figure 3. 3 River network of MMR indicating gauged site location

Rivers of Northern Part

The longest river of MMR is Ulhas (135km) lying in the northern part of the region, drains a considerable area of the same. Ulhas originates from the rainy ravines of Bhorghat and bound by the steep scarps of the Sahyadri and Matheran rigde flows northwards. A dam has been constructed near Badalpur on this river for drinking water supply at nearby area. The river has two tributaries namely Kalu and Bhatsai confluence at east of Kalyan. The river then turns west and flows through Mumbai ridge. Later it turns north for a few kilometers and again by turning westward finally joins the Arabian Sea south of Vasai. The river is perennial and tidal up to Kallyan. The other tributanies of this river are Poshir and silhar.

The river Tansa is a tributary of river Vaitarna, which flows beyond the limits of the region. Tansa River rises in the Sahapur uplands and flows westward till the Bhiwandi hills. Then it turns north and finally joins Vaitarna River.

Rivers of Central Part

Panvel River lying in the central part of the region rises from northern part of the Matheran hills. It mostly flows through the Panvel tehsil and finally joins the Thane Creek. The tributaries of Panvel River are Kalundri, Navdi and Kasadi all draining the central part of the region.

Rivers of Southern Part

The main & longest river of this part is Patalganga which is perennial in nature. It rises in the western scarps of Materan uplands and branches off from the main ridge near Khopoli. Then if flows in a general westward direction and finally joins the Dharamtar Creek. The different tributaries of the river are Bhogeshwari and Balganga traverse the southern part of the region.

The other two important rivers in southern part are Amba and Bhogwati and ends at the Dhanamtar Creek. Amba is perennial in nature and navigable, flows through the extreme south portion of the region.

Rivers of Mumbai City and Suburban Region

Mumbai &Mumbai suburban districts are drained through four rivers namely Mithi, Dahisar, Poishar and Oshwara along with the storm water drainage system. The longest river among these is Mithi having a length of only 17.9 km. It originates from the overflow of Vihar and Powai dams, flows in south west direction and finally joins the Mahim Creek near Bandra. The name of Mithi River has been highlighted after the devastating deluge of 2005 in Mumbai City as the numerous problems with the river have been detected as one of the major responsible factors for the same.

3.6 CONCLUSION

A careful study describing the region as a whole, its topography, climatic behavior, land use pattern, hydrological features and river network has been taken up in this chapter. The study under discussion in MMR is greatly involved with the regionalization approaches which require the knowledge of the hydro meteorological characteristics of the region. Hence the information acquired and discussion as the outcomes of this chapter are precious for carrying out the regional analysis of the problem.

In general the highest orders of the main rivers of the region at their outlet are 3 to 4. The moderate values of Bifurcation ratio for most of the catchments under this study indicate the normal shape of the catchments. The drainage densities of the catchments in the region as a whole are quite high resulting in an increased runoff response.

CHAPTER 4
DATA AVAILABILITY

4.1 GENERAL

Hydrological analysis and estimates are based on data. The data are used to better understand the hydrologic processes and as a direct input into hydrologic simulation models for design, analysis and decision making (Chow et al). Hydrologic processes vary in space and time and are random or probabilistic in nature. Thus the uncertainties involved with the process create the requirement of data. Hence proper collection and processing of data are prerequisite to any hydrological analysis. Moreover, the accuracy and precision of any hydrological estimates are dependent on the availability and the amount of data collected, to a large extent.

The collection and storing of hydrological data worldwide were notably encouraged from the time of Hydrologic decade (1965-1974). Nevertheless the scenario of our country in terms of the number of hydrologic measurement stations or the collection of the data and their availability for the analysis purpose is not yet satisfactory.

4.2 A BRIEF OVERVIEW OF HYDROLOGICAL DATA

The hydrological data can be broadly categorized into three classes namely Space- oriented data, Time-oriented data and Relation-oriented data and brief description of these categories are presented under this section.

4.2.1 Space-oriented data

This kind of data includes all the information related to spaces which are generally static in nature with the change of time. The following data belong to this category.

(a) Catchment data: - It comprises of catchment boundary, its physical characteristics, geographical area, river network etc.

(b) River data: - It consists of cross sections, longitudinal profiles, bed characteristics of the Rivers or the streams and semi-static in nature.

(c) Lake/Reservoir data: - This includes the elevation, area, capacity of Lake& Reservoir present in particular area.

(d) Station data: This includes a wide variety of information associated with gauging stations like station name, type, drainage boundaries, catchment area, latitude and longitude, administrative units, elevation etc.

4.2.2 Time-oriented data

These data are point measurement data which have an associated time of observation. The measurement may be either of an instantaneous value, a cumulative value, a constant value or an averaged value. The resulting data series from this kind of data measurement forms a time series which is subjected to statistical analysis. Time-oriented data includes the following.

(a) Meteorological Data:- All the climatic variables like precipitation (Rainfall), evaporation, temperature, atmospheric pressure, humidity, wind speed & direction, sunshine etc. come under this class of data.

(b) Hydrological Data:- Mainly the time series of stage and discharge in surface water bodies form the hydrologic data. Many a times the discharge data are derived from the stage data by utilizing the pre-established stage-discharge relationship. These are termed as derived hydrological data.

(c) Water Quality Data:- This kind of data includes the water quality parameters and is measured with a regular time-interval.

4.2.3 Relation-oriented data

The data obtained from the mathematical relationship between two or more hydrological variables are relation-oriented data. Sediment data derived from flow sediment relationship is a typical example of this kind.

4.3 DATA REQUIRED

The present study involves the estimation of design flood for gauged & ungauged catchments of Mumbai Metropolitan Region (MMR) by means of Flood Frequency analysis and Regional Unit Hydrograph analysis. In order to carry out those analyses, the requisite data may be classified as catchment descriptors, Meteorological data and Hydrological data.

4.3.1 Catchment descriptors

The catchments descriptors are related with the physical properties of the catchments and are essential for carrying out the regional analysis for estimation of design flood. It is worth noting that these are space-oriented data. The various data of this kind required for this study are mentioned below in a specific manner.

(i) The station name, Administrative units and location (latitude & longitude) of the available gauge & discharge sites of the region.

(ii) Catchment area plans showing the river network as well as name of the rivers, streams or tributaries where the gauge & discharge sites are located.

(iii) Catchment area plans showing the contours or elevation at different point along the longitudinal section of the longest stream of the catchments.

(iv) Area of representative catchments.

(v) Land use pattern within the catchments area.

4.3.2 Meteorological data

The various Meteorological data required are as follows,

(i) The annual rainfall data of different rain gauge stations in the region.

(ii) Maximum daily rainfall for 25yrs, 50yrs and 100yrs return periods at different representative catchments of the region.

4.3.3 Hydrological data

The time series of annual maximum discharge which comprises the largest flood peak in each year are required for all the available gauge and discharge sites of the MMR region.

4.4 DATA COLLECTED

The catchments descriptors as mentioned in sec 4.3.1 are collected in systematic way. The annual rainfall data for 5 numbers of rain gauge stations located at different points of the region have been collected. The details of the rain gauge stations and the period of availability of concurrent rainfall data are presented in table 4.1. The Isopluvial maps of 24hrs rainfall with 25, 50, & 100 yrs return period respectively for Konkan coastal region, i.e. hydro meteorologically homogeneous sub-zone no. 5(a) have been collected. These maps give the isohyets for maximum daily rainfall with specified return periods for the entire region where the present study area lies.

The annual maximum discharge series have been collected for 12 catchments for a period ranging from 8 to 31 years, out of which 11 catchments are located within MMR and another station namely Alman is located slightly beyond the northern boundary of the region. The location of gauge sites and the areas of representative catchments ranging from 31 sq km to 1040 sq km are collected. The locations of gauge & discharge sites along with the rivers or stream on which they are located are shown in fig. 3.3. The details of the gauge & discharge site, their representative catchments & area, length of data period etc, may be observed at a glance from Table 4.2.

Table 4. 1 Details of raingauge stations and period of data availability

Sl No.	Station Name	District	Geographical Location		Data Period	Record Length (years)	Normal annual Rainfall (mm)
			Latitude (In full degree)	Longitude (In full degrees)			
1	Kolaba	Mumbai city	18	72	1901-1996	96	1972
2	Santacruze	Mumbai city	19	72	1950-1996	47	2338
3	Alibag	Raigad	18	72	1933-1996	64	2162
4	Poddar	Mumbai city	19	72	1962-1978	17	1878
5	Bhira	Raigad	18	73	1962-1996	33	4275

Table 4. 2 Details of gauge sites

Station Name	District	Tahsil / Taluk	Latitude	Longitude	River	Tributary	Stream	Catchment Area (sq km)
Alman	Thane	Wada	19°37'49"	73°05'28"	Vaitarna	Vaitarna	Vaitarna	647.5
Gadhi	Raigad	Panvel	18°59'54"	73°08'17"	Panvel	Kalundri	Kalundri	125.22
Kalamb	Raigad	Karjat	19°05'33"	73°23'31"	Ulhas	Poshir	Poshir	155.07
Kamarli	Raigad	Pen	18°43'28"	73°10'08"	Patalganga	Bhogeshwari	Bhogeshwari	79.11
Kanpoli	Raigad	Panvel	19°04'02"	73°08'31"	Panvel	Navdi	Kasadi	31.35
Kolhare	Raigad	Karjat	19°00'40"	73°20'25"	Ulhas	Ulhas	Ulhas	306.05
Naldhe	Raigad	Karjat	19°01'55"	73°24'44"	Ulhas	Shilhar	Shilhar	94.41
Pise	Thane	Bhiwandi	19°21'25"	73°12'03"	Ulhas	Bhatsa	Bhatsa	844.51
Saivan	Thane	Vasai	19°29'00"	72°59'00"	Vaitarna	Tansa	Tansa	519.55
Titwala	Thane	Kalyan	19°19'10"	73°16'02"	Ulhas	Bhatsa	Kalu	1040.38
Turade	Raigad	Panvel	18°53'16"	73°10'50"	Patalganga	Patalganga	Patalganga	317.57
Wadi	Raigad	Pen	18°48'25"	73°06'07"	Patalganga	Balganga	Balganga	138.26

4.5 SOURCES OF DATA

The major Maharashtra River gauging authority is State Data Storage Centre (SDSC), Nasik under the water resources Department of Maharashtra Govt. The daily maximum discharge data series of 12 gauge & discharge sites were provided by SDSC, Nasik from which the annual maximum discharge series have been extracted. The name of the gauge sites, their location along with the name of rivers or streams, catchments area were also supplied by the sane authority.

The other catchment descriptors have been derived from S.R.T.M. data with 90 meter resolution downloaded from the respective sites. The Shuttle Radar Topography Mission (SRTM) is an international project spearheaded by the U.S. National Geospatial-Intelligence Agency (NGA) and the U.S. National Aeronautics and Space Administration (NASA). This is an international research effort that obtained digital elevation models on a near-global scale from 56 °S to 60 °N, to generate the most complete high-resolution digital topographic database of Earth to date. . SRTM consisted of a specially modified radar system that flew onboard the Space Shuttle Endeavour during the 11-day STS 99 mission in February of 2000. To acquire topographic (elevation) data, the SRTM payload was outfitted with two radar antennas. One antenna was located in the Shuttle's payload bay, the other on the end of a 60-meter (200-foot) mast that extended from the payload bay once the Shuttle was in space. The technique employed is known as Interferometric Synthetic Aperture Rader. The resolution of the cells of the source data is one arc second, but 1" (approx. 30 meter) data have only been released over United States territory; for the rest of the world, only three arc second data (approx. 90 meter) are available. Each one arc second tile has 3,601 rows, each consisting of 3,601 16 bit bigrndian cells. The dimensions of the three arc second tiles are 1201 x 1201. The elevation models derived from the SRTM data are used in Geographic Information Systems (GIS). They can be downloaded freely over the internet, and their file format (.hgt) is supported by several software developments.

The annual rainfall data were collected from the rain gauge stations managed by Indian Meteorological Department. The Isopluvial maps for 5(a) sub-zone were prepared by Central Water Commission, Govt. of India and are available with CWC hand book for estimation of Design flood for sub-zone no 5(a) & (b).

CHAPTER 5
METHODOLOGY

5.1 GENERAL

A design flood is the magnitude of flood at a desired return period for a specified structure. The rainfall-runoff modeling is one of the two basic approaches of design flood estimation which links rainfall as an input to resulting runoff as an output and sometimes better known as deterministic approach. Another approach namely statistical approach based on the frequency analysis is more direct approach as design flood is derived from the gauge records and if the same are available, the priority of choice is generally given to this approach. The primary objective of frequency analysis is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions (Chow, 1988). Generally the hydrologic data employed for this approach is annual flood series and assumed to be independent and identically distributed. The simplest approach of this type is 'At site flood frequency analysis' using data of the subject site only. In general as recommended by Flood Estimation Handbook (1999), single site analysis is used when there is a reliable and long record at the site of interest and when the target return period T is not too long. Single site analysis is not usually appropriate if the record length is shorter than T.

The approaches mentioned above require data of subject site on a large scale and a long term basis for estimation of design flood with a desired return period. But economic constraints do not justify detailed hydrological and meteorological investigations at every new site for collection of the same. This basically generates the necessity of regionalization which has been illustrated in chapter1. Broadly two types of regional approaches namely regional flood frequency and synthetic unit hydrograph approaches (CWC approach) are open for adoption in India depending on the availability of data. The first one needs long term annual flood series for the representative catchments for subjecting to statistical analysis to develop a regional flood frequency model. The other approach needs concurrent storm rainfall and runoff data of the representative catchments over a period of 5 to 10 years to develop a "regional rainfall-loss rate-runoff (UH) model" and long term rainfall records at a large number of stations to develop "design storm values". The present study adopts both of the aforesaid approaches, the methodologies of which are discussed in the subsequent sections. In the first part, the methodologies for the regional flood frequency analysis with 'L' moments technique are described in a sequential manner. An attempt has been made to pay a special attention towards the illustration of the 'L' moments theory. In the next part the steps of the regional unit hydrograph approach are discussed. In both the cases certain assumptions have been made prior to the commencement of the discussion.

Data processing is the manipulation of data into a more useful form with the objective of evaluating the same for its accuracy. Hydrological and related meteorological data collected from different source are generally in raw or semi-processed form. Therefore further processing of data is necessary before it is put to use in hydrological analysis. Thus the preliminary analysis of data followed by its processing is carried out and the general statistics of the time series are prepared before the main part of flood frequency analysis.

5.2 PROCESSING AND PRELIMARY ANALYSIS OF RAINFALL DATA

The methodology applied for executing the various steps involved in the processing and preliminary analysis of rainfall data are furnished below.

5.2.1 Preliminary scrutiny

Before storing the data manual scrutiny of data is necessary. Checking of the reasonableness of data, presence of any outliers is also the part of this step.

5.2.2 Estimation of missing data

Many a time rainfall data series contain gaps or inconsistent values and if so, the blanks in data series are denoted as zero in computer while carrying out the analysis. Therefore the estimation of these missing data for filling up the gaps in data series is a primary task prior to the analysis. The technique applied for this purpose under this study is Normal Ratio Method.

Normal ratio method

In this method the annual rainfall at station 'G' containing the gap is estimated as a function of the Normal Annual rainfall of the same and those of the neighboring stations along with their annual rainfall for the year under question.

Thus,

$$R_G = \frac{1}{n}\sum_{i=1}^{n}\frac{NR_G}{NR_i} * R_i \qquad (5.1)$$

Where,

R_G = estimated annual rainfall at station G

R_i = annual rainfall at neighboring station

NR_G = Normal annual rainfall at station G

NR_i = Normal annual rainfall at station i

n = number of the neighboring station

5.2.3 Checking for short-term dependence

The meteorological time series frequently exhibit significant short-term serial correlation. This should be verified prior to commencement of the analysis. Moreover a slow continuous variation of rainfall pattern in increasing or declining mode may also be present which influences in the resulting runoff responses of the catchments. Early detection of this is must by carrying out the trend analysis. Most water resources systems have been designed and operated based on the assumption of stationary hydrology. If this assumption of stationarity is not valid, current systems may be under or over designed. In order to check this stationarity or nonstationarity of Hydrological time series, trend analysis plays a predominant role. The two wings of the trend analysis namely Parametric and Nonparametric tests have got their own advantages a Features of both include:

- Allows easy statistical testing using different tests
- Supports various time series data input formats
- Provides simple statement of test result
- Displays test statistic and critical values for various statistical significance levels
- Performs resampling analysis to determine critical test statistic values
- Allows easy retrieval of test results.

Based on the WMO/UNESCO Expert Workshop on Trend/Change Detection and on the CRC for Catchment Hydrology publication *'Hydrological Recipes: Estimation Techniques in Australian Hydrology'* by Grayson et al.; TREND (Trend, Change & Randomness) has 12 statistical tests as following.

- Mann-Kendall (non-parametric test for trend).
- Spearman's Rho (non-parametric test for trend)
- Linear Regression (parametric test for trend)
- Distribution-Free CUSUM (non-parametric test for step jump in mean)
- Cumulative Deviation (parametric test for step jump in mean)
- Worsley Likelihood Ratio (parametric test for step jump in mean)

- Rank-Sum (non-parametric test for difference in median from two data periods)
- Student's t (parametric test for difference in mean from two data periods)
- Median Crossing (non-parametric test for randomness)
- Turning Points (non-parametric test for randomness)
- Rank Difference (non-parametric test for randomness)
- Autocorrelation (parametric test for randomness).

It is worth noting that the nonparametric tests tend to ignore the magnitude of the observations in favour of the relative values or the rank of the data. As these tests have fewer and less stringent assumptions to be met with, the tests are less powerful compare to parametric one, however, to reduce the number of underlying assumptions required nonparametric tests are typically employed. Indeed Nonparametric tests are usually adopted for using environmental impact assessment because the statistical characteristics of the often messy environmental data make it unwise to use parametric methods. The above stated logic definitely justifies the frequent use of nonparametric test for detection of trend in hydro meteorological Time Series. Among the Nonparametric Trend tests for hydro meteorological Time Series, whatever may be the reason Spearman's Rho test is rarely used, where as the Mann-Kendall (MK) test has been evergreen choice of the researchers of this field since its formulation. Moreover some research papers related with Power of test also justify the use of MK test rather than Spearman's Rho test. The study under discussion adopts Turning point test and Anderson's correlogram test for checking of randomness and MK test for detection of trend; a brief discussion of each is given below.

The Turning point test (Kendall & Stuart, 1976) is based on a binary series. If $x_{i-1} < x_i > x_{i+1}$ or $x_{i-1} > x_i < x_{i+1}$, then x_i is assigned a value of one; otherwise it is assumed to be zero. If $x_{i-1} < x_i > x_{i+1}$, then x_i is termed as peak and when $x_{i-1} > x_i < x_{i+1}$, x_i is termed as trough (Wallis & Moore). The number of ones, m as termed as turning points, is approximately normally distributed, i.e.

$$m \approx N\left\{\frac{2(n-2)}{3}, \left[\frac{(16n-29)}{90}\right]^{1/2}\right\} \tag{5.2}$$

The test statistics Z is calculated as,

$$Z = \frac{m - E(m)}{\sigma(m)} \tag{5.3}$$

A level of statistical significance α is then chosen. A value of $Z_{1-\alpha/2}$ is determined using a standard normal distribution table. If $|Z| < Z_{1-\alpha/2}$, the series is said to be random significant to α.

Anderson (1942) developed a parametric test for checking of randomness and well known as Anderson's correlogram test. The autocorrelation function, r_k, is assumed to be approximately normally distributed and given as

$$r_k \approx \left\{\frac{-1}{(n-k)}, \left[\frac{(n-k-1)^{1/2}}{(n-k)}\right]\right\} \tag{5.4}$$

The test statistics Z is calculated as,

$$Z = \frac{r_k - E(r_k)}{\sigma(r_k)} \tag{5.5}$$

A level of statistical significance α is then chosen. A value of $Z_{1-\alpha/2}$ is determined using a standard normal distribution table. If $|Z| < Z_{1-\alpha/2}$, the series is said to be random significant to α. The present study considers the autocorrelation coefficient of lag one (k=1) i.e. r_1.

MK Test (Mann, 1945; Kendall, 1975) is based on the test statistics 'S' defined as follows,

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$$S = \sum_{i=1}^{n-1} \sum_{j=i+1}^{j=n} \operatorname{sgn}(P_i - P_j) \tag{5.6}$$

Where,

$$\operatorname{sgn}(P_i - P_j) = \begin{cases} 1 \ \textit{if} \ P_i > P_j \\ 0 \ \textit{if} \ P_i = P_j \\ -1 \ \textit{if} \ P_i < P_j \end{cases} \tag{5.7}$$

Generally, if a dataset displays a consistently increasing or decreasing trend, S will be positive or negative respectively, with a larger magnitude indicating the trend is more consistent in its direction. By using the 'sgn' function, the algorithm is able to detect trends featuring either large or small increase steps from year to year equally. Under the null hypothesis H_0 that there is no trend displayed by the time series, the distribution of S is then expected to have a zero-mean and variance.

$$\operatorname{var}(S) = \frac{n(n-1)(2n+5)}{18} \tag{5.8}$$

Where n is the number of years under consideration. After determination of var(S), the test statistic Z is calculated as,

$$Z = \begin{cases} \dfrac{S-1}{\sqrt{\operatorname{var}(S)}} \ \textit{if} \ S > 0 \\ 0 \ \textit{if} \ S = 0 \\ \dfrac{S+1}{\sqrt{\operatorname{var}(S)}} \ \textit{if} \ S < 0 \end{cases} \tag{5.9}$$

A level of statistical significance α is then chosen. A value of $Z_{1-\alpha/2}$ is determined using a standard normal distribution table. If $|Z| > Z_{1-\alpha/2}$, the series is said to display a trend significant to α ; a positive value of Z indicates rising trend and a negative value for the same indicates falling trend.

5.2.4 Checking for long-term dependence

Many a time the meteorological time series exhibit no or very little short-term dependence while, the long-term dependence may be present in the same series. Long-term dependence of a series indicates a significant long-term serial correlation. A long-term dependence present in a time series is linked with a long term non stationary behavior of the series. Therefore short- term dependence results from a climatic variability over a short period where as a long-term dependence indicates climate change over the region under which a long term alteration is occurring.

Long-term dependence can be measured by Hurst co-efficient (Hurst, 1951) k and is given by,

$$\mathrm{k} = \frac{\log\left(\frac{R}{S}\right)}{\log\left(\frac{n}{2}\right)} \tag{5.10}$$

Where,

R = Range of cumulative departures from mean

S = Standard deviation

n = length of the series

The parameter (R/S) is called rescaled range. For the series of purely independent data the value of k is theoretically 0.5. As the 'k' value increases the long-term dependence increases up to its maximum possible value 1.0.

To check whether the long-term dependence is significant or not, the percentage points of Hurst's k for serially independent data at different probability levels are required, which vary with the sample length. The percentage points of Hurst's 'k' (for normally distributed independent data) were obtained and tabulated by Monte Carlo simulation procedure (Lye and

Lin, 1993). If the k value for a time-series is greater than the empirical percentage points of Hurst 'k' at a given significance level for a given sample length, the series is long-term dependent. The test has been performed for the stations with an available data period of 20 yrs or more under this study.

5.3 PROCESSING AND PRELIMINARY ANALYSIS OF ANNUAL FLOOD SERIES

Floods are events of the streamflow regime. The most important data to be collected and processed for flood studies belong to this category. The annual flood series have been extracted from daily discharge series collected from the concerned authority. The various steps & the methodology associated for processing and preliminary analysis of the annual flood series data are discussed below.

5.3.1 Consistency check of the data

The annual flood series may exhibit the non-stationary behavior which results in an inconsistent data set. A data series is non stationary if some of the underlying properties of data change over time. A series with trend is a type of non-stationary data. The others non-stationary behaviors are steps jumps & fluctuations.

5.3.1.1 Causes of Non-stationarity

The causes of non-stationarity present in a data set can be grouped as,

(a) Changes with the catchment physiography like change in land use pattern, drainage diversion, construction of reservoir etc. These changes occur especially through human activities.

(b) Climatic variability over the time.

(c) Problems with the data records like rebuilding of weirs and recording stations, abrupt changes in the rating equations etc.

5.3.1.2 Statistical Test for Detecting Non-stationarity

Prior to accepting the flood series data for the flood frequency analysis the statistical test must be performed to check whether the non-stationarity behavior or trend in the data set are present or not. If they are present, the cause should be detected for the same and subsequent treatment or allowance for non-stationarity should be made in interpretation of the results. The statistical tests performed for this purpose under this study are checking of randomness by Turning Point test & Anderson's correlogram test and detection of trend by Kendall's rank correlation test, the methodologies for which except the last one are same as applied for rainfall data and may be referred in section 5.2.3.

The Kendall's tau (τ) or rank correlation coefficient (Kendall, 1938) is given as,

$$\tau = \frac{4P}{n(n-1)} - 1 \tag{5.11}$$

Where P is the number to be counted in all pairs of observations ($x_i, x_j, j > i$) when $x_j > x_i$. Under the null hypothesis H_0 that there is no trend displayed by the time series, the distribution of τ is then expected to have a zero-mean and variance,

$$\text{var}(\tau) = \frac{2(2n+5)}{9n(n-1)} \tag{5.12}$$

Where n is the number of years under consideration. After determination of var (τ), the test statistic Z is calculated as,

$$Z = \frac{\tau - E(\tau)}{\sigma(\tau)} \tag{5.13}$$

A level of statistical significance α is then chosen. A value of $Z_{1-\alpha/2}$ is determined using a standard normal distribution table. If $|Z| > Z_{1-\alpha/2}$, the series is said to display a trend significant to α ; a positive value of Z indicates rising trend and a negative value for the same indicates falling trend.

5.3.2 Preliminary analysis of data

As the data set are checked for consistency and found to be suitable for the analysis, the elementary statistical parameters for the same are determined. The statistical parameters are estimated in normal as well as log domain. The parameters estimated are mean (μ), extreme values of the series, standard deviation (σ), co-efficient of variation (c_v), co-efficient of skewness (c_s), co-efficient of Kurtosis (c_k) and auto correlation co-efficient for lag 1, lag 2 & lag 3 series (r_1, r_2, r_3).

5.4 REGIONAL FLOOD FREQUENCY ANALYSIS

In India except the large and independent size of dams, frequency analysis is used in everywhere for estimation of design flood. The different approaches related with regional flood frequency analysis can be carried out using following approaches.

(i) At site regional flood frequency analysis using data of the subject site in combination with the data of the other gauged sites lying in the same hydrometeorologically homogeneous region or data pooled from the sites hydrologically similar with the subject site irrespective of their geographical location.

(ii) Regional flood frequency analysis using the data of available gauged sites located in the same hydrometeorologically homogeneous region.

The main difference between at site regional & regional analysis is that in regional analysis mean annual flood is calculated from the catchment characteristics while in the case of at-site regional analysis it is calculated from the observed data of the site. Therefore in case of regional analysis the regional flood quantiles are derived whereas the flood frequency curves relating flood size to its frequency are directly obtained from at- site analysis. Moreover the latest technique related to at site analysis is pooled frequency analysis which gained an extensive popularity in recent past. In this technique the data are pooled from the sites selected on the basis of similarity distance which is also termed as Euclidian distance and pooled growth curve is derived for the subject site. In a broad sense two major deeds in relevance with the both types of analysis are carried out. One is identifying of homogeneous region and another is derivation of flood quantiles or flood frequency curve.

The catchments considered in this study are rural catchments and obliges the essential criteria for the Regional flood frequency analysis. Therefore it is worth to mention that neither the study comprises of any urban catchment within the MMR nor the resulting relationship of this study is valid for those same.

The three major steps of the present study are as follows,

(1) Regional Homogeneity Test based on L-moments.

(2) Selection of regional distribution based on L-moment ratio diagram and Z statistics.

(3) Development of regional flood formula for MMR.

5.4.1 Regional homogeneity test

5.4.1.1 L-Moments Theory

L-moments theory was first introduced by Hosking (1990) and is indisputably considered as a major turning point in the area of flood frequency analysis.

It is based on the linear combinations of probability weighted moments and 'L' signifies the linearity. The first 'L' moment refers the location and is known as L-mean (l_1). The second 'L' moment is a measure of scale and dispersion and termed as L- scale (l_2). The third and fourth 'L' moments are measure of symmetry and peakedness respectively (l_3 & l_4).

The different estimates in pertaining with 'L' moments approach are furnished in steps as follows (Hosking and Wallis (1997),

(a) At first the unbiased probability weighted moment estimators (Landwehr et.al, 1979) are calculated as,

$$b_0 = \frac{1}{n}\sum_{j=1}^{n} x_j \tag{5.14}$$

$$b_1 = \frac{1}{n}\sum_{j=2}^{n} \frac{(j-1)}{(n-1)} x_j \tag{5.15}$$

$$b_2 = \frac{1}{n}\sum_{j=3}^{n} \frac{(j-1)(j-2)}{(n-1)(n-2)} x_j \tag{5.16}$$

$$b_3 = \frac{1}{n}\sum_{j=4}^{n} \frac{(j-1)(j-2)(j-3)}{(n-1)(n-2)(n-3)} x_j \tag{5.17}$$

Where, n = sample size

$x_j = j^{th}$ element in ascending order

(b) The sample L- moments are estimated by,

$$l_1 = b_0 \tag{5.18}$$

$$l_2 = 2b_1 - b_0 \tag{5.19}$$

$$l_3 = 6b_2 - 6b_1 + b_0 \tag{5.20}$$

$$l_4 = 20\,b_3 - 30\,b_2 + 12b_1 - b_0 \tag{5.21}$$

(c) L-moment ratios are the dimensionless versions of the sample 'L' moments and defined by Hosking (1990) as,

L-moment ratios	Expressions	Range of values
L- CV	$t_2 = \frac{l_2}{l_1}$	$0 \leq t_2 < 1$
L-skewness	$t_3 = \frac{l_3}{l_2}$	$-1 \leq t_3 \leq 1$
L- kurtosis	$t_4 = \frac{l_4}{l_2}$	$-1 \leq t_4 \leq 1$

'L' moments have significant advantages over ordinary Probability Weighted Moments (PWM) and notably enlightened by Hosking and Wallis (1997) as well as Vogel and Fennessey (1993).

The appealing traits of L-moments approach are as follows,

(i) L-moments estimators as well as L-moments ratio estimators have small bias and variance as compared to conventional moments approach.

(ii) L-moments can be directly interpreted as measures of scale and shape of probability distributions.

(iii) Regarding the fitting of distribution the conventional method of moments give poor results when data are strongly skewed (Hosking and Wallis 1997), where as L- moment ratios are particularly good at identifying the distributional properties of highly skewed data (Vogel and Fennessey,1993). As the flood series often exhibits the high skewness, the use of L-moment ratios is justifiable in flood frequency analysis.

(iv) L-moments weigh each element of a sample according to its relative importance (Hosking 1997).Moreover as the higher power is absent in case of linear approach of L- moments, it is less sensitive to the presence of outliers if any.

(v) The values of L-moment ratios are bound in either way and Hosking (1990) identified this fact as an advantage for easier interpretation of the measures.

It is also interesting to note, in spite of having so lengthy list of advantages, the computational hazards involved with L-moments approach is quite less in comparison with other PWM methods.

5.4.1.2 Testing for Discordancy

This is the preliminary tool for detecting an unusual site (Discordant) within a group of sites and was introduced by Hosking and Wallis (1997). A site is termed as discordant, if the L-moment ratios of the same significantly mismatches with those of the group of sites as a whole.

The Discordancy is formally defined as follows. Let M be the number of the sites present in the region and u_i be a vector of the L-moment ratios at the site i,

$$u_i = (t_2, t_3, t_4)^T \tag{5.22}$$

Where, 'T'denotes the transpose of a vector.

Defining,

$$U = \frac{1}{M} \sum_{i=1}^{M} u_i \tag{5.23}$$

$$A = \sum_{i=1}^{M} (u_i - U)(u_i - U)^T \tag{5.24}$$

Then the Discordancy measure 'D_i' for site i is given as,

$$D_i = \frac{1}{3} M \, (u_i - U)^T A^{-1} (u_i - U) \tag{5.25}$$

Where,

M = number of sites with a minimum of 7 (Hosking& Wallis, 1997)

If 'D_i' for any site is greater than the critical value dependent on the number of sites present in the group, the site will be treated as discordant. The maximum critical value for discordancy is 3.0 for 15 sites or more. The critical values for discordancy statistic D_i for various numbers of sites are given in table 6(Hosking & Wallis, 1997).

5.4.1.3 Testing for Heterogeneity

This is the more important statistics for homogeneity test of the sites. It checks whether the significant inter-site variations in terms of L-moment ratios are there or not and if not the group of the sites is said to be homogeneous. In other words the homogeneous region indicates that the quantile distributions are broadly similar at all the sites.

The inter-site variability (V_i) is expressed either of the following ways,

(i) V_1 based on L-CV (t_2 statistics)

(ii) V_2 based on L-CV & L-skewness (t_2 & t_3 statistics)

(iii) V_3 based on L-skewness & L-kurtosis (t_3 & t_4 statistics)

Hosking and Wallis (1997) evaluated the heterogeneity measures. Multiple random samples are generated by assuming the region as homogenous based on a generalized 4-parameter Kappa Distribution. Now for each simulation the variability measures V_1, V_2 & V_3 are calculated. After carrying out the entire simulation task the mean and standard deviation (μ_{v_i}, σ_{v_i}) are determined. In case of ideally homogenous region the variability observed in the subject group of sites should differ insignificantly with μ_{v_i}. Thus, heterogeneity measures (H_1 , H_2 & H_3) are given as,

$$H_i = \frac{Vi - \mu_v}{\sigma_v} \quad (5.26)$$

The recommended value for H_i is less than one for the region to be acceptably homogeneous (Hosking and Wallis). If $1 \leq H_i \leq 2$, the region is possibly heterogeneous and if $H_2 > 2$, the region is heterogeneous.

Hosking recommended H_1 as more powerful statistics for identifying Homogeneous Region. Some recent studies (FEH et al) considered H_2 as the measuring statistics for heterogeneity test. The validity of the later one remains on the fact that H_2 statistics are based on L- CV and L-skewness which are required for the estimation of regional quantiles with three parameter distributions. Moreover as most of the flood frequency studies find their quantile distributions as GEV, GLO or PE3 distribution (three parameter distribution), the acceptance of the H_2 along with the H_1 as heterogeneity measures may be justifiable.

A FORTRAN programming given by Hosking (1997) for heterogeneity test has been used for this study for determination of H_1 , H_2 & H_3 statistics.

5.4.2 Regional distribution

5.4.2.1 L-Moment Ratio Diagram

The L-moment ratio diagram is a graphical representation of one L-moment ratio against another. In the context of the present study for identification of the best fit distribution, the particular diagram of L-skewness versus L-kurtosis is of interest. A two parameter distribution is represented as a point in this diagram, where as a three parameter distribution is represented by a line on the same. The relationships for obtaining the L-moment ratio diagrams for different distributions were given by Hosking.

5.4.2.2 Selection of the Best Fit Distribution

Initial Judgement

This is worked out by plotting the Regional L-kurtosis and L-skewness values on the L-moment ratio diagram and to match with the nearest point or line corresponding to a theoretical distribution.

Goodness of Fit Measure

Hosking and Wallis (1997) developed this measure to identify the best fit distribution specifically appropriate for comparing 3-parameter distributions. The goodness of fit is judged by the difference between regional L-kurtosis values and the theoretical L-kurtosis values and is given by,

$$Z^{Dist} = \frac{t_4^R - \tau_4^{Dist}}{\sigma_4} \tag{5.27}$$

Where,

t_4^R = Regional L-kurtosis

τ_4^{Dist} = Theoretical L-kurtosis for the fitted distribution

σ_4 = Sample variability of t_4^R

For small record lengths (n<20) which is relevant in the present study, a bias correction factor β_4 is used for t_4^R. The values of β_4 and σ_4 are obtained by simulation procedure (generally 500 simulations).

The values of Z^{Dist} sufficiently close to zero indicate a good fit. However Hosking proposed as $|Z^{Dist}| \leq 1.64$ for the fitting criteria of any distribution. If more than one distribution meet the above criteria, then the distribution corresponding to the minimum $|Z^{Dist}|$ value is generally considered as the best fit.

A FORTRAN programming given by Hosking& Wallis (1997) for goodness of fit test has been used for this study for determination of $|Z^{Dist}|$ statistics for different distributions.

5.4.2.3 Estimation of Regional Quantile Parameters

Regional Quantile parameters namely location, scale and shape parameters for 3 parameter distributions are obtained from the best fit distribution. The regional quantile $(Q_T/\overline{Q})$ in the form of expressions or the growth factors can be derived by using these parameters. The growth factors may be calculated for the best fit distribution by using FORTRAN programming.

5.4.3 Flood quantile estimation

5.4.3.1 Gauged Catchments

An index flood is a typical magnitude of flood expected at a given site. In the traditional method of regional flood frequency analysis as defined in Flood Studies Report, U.K. (1975) the mean annual peak flood is considered as the index flood for a given site.

The statistical approach gives the flood frequency curve or flood quantile(Q_T) as the product of the index flood and the growth factor. Therefore the mean annual peak flood $\overline{Q}$ can be estimated with the available flood data and flood quantile (Q_T) can be expressed as the product of Regional Quantiles ($Q_T/\overline{Q}$) and $\overline{Q}$.

5.4.3.2 Ungauged Catchments

In this case as direct measurement of $\overline{Q}$ is not possible due to absence of flow data, a regional relationship is developed with the help of catchment physiographic data of the gauged cathments such as catchments area, drainage density, overland & channel slope, station elevations etc. In this study due to absence of other physiographic parameters the aforesaid regional relationship has been established between $\overline{Q}$ and area of the catchment (A) of gauged sites by regression procedure.

Thus,

$$\overline{Q} = f(A)$$

And $$Q_T = \left(\frac{Q_T}{\overline{Q}}\right) * f(A) \tag{5.28}$$

5.5 REGIONAL UNIT HYDROGRAPH ANALYSIS

After a detailed study under long term plan of Central Water Commission, the country has been divided in 26 hydro-meteorologically homogeneous sub-zones. After identification of homogeneous sub-zones, representative unit hydrographs have been derived for a few selected catchments based on actual rainfall runoff data. The different unit hydrograph parameters for describing the shape of the representative U.H. have been defined. These parameters have been derived from the representative U.H. of the gauged sites and correlated with their corresponding catchments physiographic parameters. Then equations have been developed to derive the synthetic unit hydrograph using only to catchments characteristics instead of rainfall runoff data. These SUH relationships are documented in various flood studies reports brought out by CWC. The reports deal with the estimation of design flood of 25 yrs, 50 yrs & 100 yrs return periods and are generally recommended for the catchments varying in areas from 25 to 5000 sq km. The 1hr SUH parameters with their corresponding notations as described by CWC are as under.

t_p = Time from the centre of Unit rainfall duration to the peak of unit hydrograph in hours

t_m = Time to peak of the unit hydrograph in hours.

Q_P = Peak discharge of unit hydrograph in m^3 /sec

t_r = Unit rainfall duration adopted in a specific study

T_B = Base width of the unit hydrograph in hours.

W_{50}= Width of unit hydrograph measured at discharge ordinate to 50% of Q_P in hours

W_{75} = Width of unit hydrograph measured at discharge ordinate equal to 75% of Q_P in hours.

WR_{50}=Width of the rising side of the unit hydrograph measured in hours at discharge ordinate equal to 50% of Q_P in hours.

WR_{75} =Width of the rising side of the unit hydrograph measured in hours at discharge ordinate equal to 75% of Q_P in hours.

The present study considers four gauged catchments of MMR with relatively short flow record which lies in hydro-meteorologically homogeneous sub-zone No. 5(a) as mentioned earlier. The methodologies for deriving the Synthetic Unit hydrograph and estimating the design flood of various return periods (25 yrs, 50 yrs, & 100 yrs,) are available with the flood estimation report for Konkan (5a) and (5b) coastal sub-zones which are recommended for ungauged and inadequately, gauged catchments in the sub-zones.

It is worth to mention the assumptions made by CWC prior to carry out the analysis. These are as follows,

(i) It is assumed that 50 yrs, return period storm rainfall produces 50 yrs, flood similar is the case for 25 yrs, flood and 100 yrs flood.

(ii) A generalized conclusion regarding the base flood and loss rate are assumed to hold good during the design flood event.

Various steps necessary to carry out the aforesaid study are furnished in subsequent sections.

5.5.1 Determination of cathcment physiographic parameters

The various physiographic parameters required for the estimation of SUH parameters are catchments are (A), Length of the main stream (L), Length of the main stream from a point near to the centre of gravity of the catchments to its outlet (Lc) and equivalent stream slope (S). As any of the equations for SUH parameters does not consist of Lc, the determination of the same is of no use under this study.

5.5.1.1 Catchment area (A)

The S.R.T.M. data for the study region has been downloaded from the definite website. The gauging sites are located with the help of their latitudes & longitudes. The drainage networks have been identified for each of the gauging sites. Then the catchments areas delineated and calculated with the help of Arc-GIS software.

5.5.1.2 Length of the main stream (L)

This implies the longest length of the main river from the farthest watershed boundary of the catchment area to the gauging site. The longest rivers for each catchments area have been digitized up to the outlet points with the help of Arc-GIS software and by using the necessary tools, the lengths have been determined in km.

5.5.1.3 Equivalent stream slope (S)

This implies one single slope of the longitudinal section of the main river representing the resultant effect of all segmental slopes. Either of the methods graphical or analytical may be employed for the determination of the equivalent slope. Under the present study analytical method has been adopted by using Arc-GIS software. The longitudinal section of the main stream has been divided in to several segments representing the broad ranges of the slopes of the segments. The length of the segment was kept relatively smaller in upstream side. At each section the elevations were found by the software tools and equivalent slope (S) has been determined by the following formula,

$$S = \frac{\sum L_i \left[D_{i-1} + D_i \right]}{L^2} \tag{5.29}$$

Where,

S is in m/km

L_i = Length of the ith segment in km,

D_{i-1}, D_i = Elevations of river bed at ith intersection points of contours reckoned from the bed elevation at points of interest considered as datum and D_{i-1} & D_i are the heights of successive bed location at intersections.

L = Length of the longest stream in km.

5.5.2 Derivation of 1hr synthetic unit hydrograph

The SUH parameters are computed using relationships with its known physiographic characteristics like A, L and S. The relationships are as under:

(i) $q_p = 0.9178 \left(\frac{L}{S} \right)^{-0.4313}$ (5.30)

(ii) $t_p = 1.5607 \left(q_p \right)^{-1.0814}$ (5.31)

(iii) $W_{50} = 1.9251\left(q_p\right)^{-1.0896}$ (5.32)

(iv) $W_{75} = 1.0189\left(q_p\right)^{-1.0443}$ (5.33)

(v) $WR_{50} = 0.5788\left(q_p\right)^{-1.1072}$ (5.34)

(vi) $WR_{75} = 0.3469\left(q_p\right)^{-1.0538}$ (5.35)

(vii) $T_B = 7.3801\left(t_p\right)^{0.7343}$ (5.36)

(viii) $T_M = t_p + \left(\frac{t_r}{2}\right)$ (5.37)

(ix) $Q_p = q_p \times A$ (5.38)

Estimated SUH parameters have been plotted on graph paper. The plotted points have been joined with a smooth Curve which gives the synthetic unit hydrograph for the catchment. The discharge ordinates at one hour interval have been summed up and multiplied by one hour which gives the total volume of direct runoff. The theoretical volume of direct runoff for 1c.m. depth has been found out by the formula,

$$Q = \frac{A \times d}{t_r \times 0.36} \quad (5.39)$$

Where,

d = 1 cm depth

t_r = 1 hr unit duration

Both theoretical volume and volume obtained from SUH are compared and in case of mismatch, the falling limb of SUH has been suitably adjusted without altering the points corresponding to SUH parameters.

5.5.3 Estimation of design storm

5.5.3.1 Design storm duration (T_D)

The duration of the storm rainfall which results in maximum discharge in a catchment is called the design storm duration. The design storm duration for this study has been adopted as 1.1 t_p as per CWC recommendation.

5.5.3.2 Estimation of point rainfall

The catchments under study have been placed on the 24hr Isopluvial maps for 25 yrs, 50 yrs & 100 yrs return periods and the corresponding point rainfalls for 24 hrs with various return periods were obtained. Conversion factors are used for converting 24 hrs point rainfall into short duration rainfall and are available with CWC Flood Estimation Report. Conversion factors were read for the design storm duration (T_D) hrs as obtained in the previous section and were multiplied by the 24 hrs point rain fall to obtain T_D hrs point rain fall for various return periods.

5.5.3.3 Estimation of areal rainfall

The areal reduction factors (ARF) were found by interpolation for the selected catchment sizes and for design storm duration from the table given in CWC Flood Estimation Report. The T_D hrs point rainfalls have been converted to T_D hrs areal rainfalls by multiplying these with the corresponding ARFs.

5.5.3.4 Estimation of effective rainfall hyetograph

The T_D hrs areal rainfalls for various return periods have been split into 1 hourly rainfall increments using the time distribution co-efficient given in the CWC Flood Estimation Report. At first the cumulative percentages of total rainfall were found from time distribution table and cumulative values of rainfall were obtained in every 1hr time interval. After this1hr areal rainfall increments were computed by subtraction of successive 1hr cumulative values of rainfall.

Design loss includes the losses due to infiltration,evapotranspiration,soil moisture and filling up the depressions. Direct runoff is the end product of the storm rainfall in excess of this design loss. Therefore a design loss rate of 0.19 cm/hr as recommended by CWC has been subtracted from 1 hr areal rainfall increments to obtain 1 hr effective rainfall increments which give the effective rainfall hyetograph.

5.5.4 Estimation of design flood

The design floods of 25 yrs, 50 yrs & 100 yrs, return periods have been estimated for the selected catchments by carrying out the steps mentioned in the subsequent section.

5.5.4.1 Computation of direct runoff peak

The effective rainfall increments for a given return period as calculated in previous section have been re-arranged against the SUH ordinates such that the maximum effective rainfall is placed against the maximum ordinate, and the next lower value of the maximum effective rainfall is placed against the next lower value of the ordinate and so on up to T_D hrs duration. Then the products of the UH ordinates and the corresponding effective rainfall increments were computed in a tabulation form. The products so calculated were summed vertically which gives the direct runoff peak.

5.5.4.2 Estimation of design flood peak

Central Water Commission recommends for considering a design base flow of 0.15 Cumecs per sq km for the sub-zone under consideration. The selected catchments areas were multiplied with this design base flow rate and base flow contributions to the design flood were obtained.

Finally the base flow contributions have been added to the direct runoff peak to obtain the design flood peak.

CHAPTER 6

RESULTS AND DISCUSSION

6.1 RAINFALL DATA

The missing data were very few in number & subsequently estimated by normal ratio method.

The test results related to short term dependence and long term dependence are presented in table 6.1. The results interpret that the rainfall data series of all the stations are random in nature as obtained from either of the tests i.e.. Turning Point test & Anderson's correlogram test.

Trend analyses were performed by applying Man Kendall's test. As auto-correlations are absent for all the stations, the modified Man-Kandell test proposed by Hamed & Rao (1997) is not required.

It has been observed from the test results that only the Kolaba station exhibits a significant rising trend for the period of 1901-1996 at 1%, 5% and 10% significance level. For other stations blended results have been found as some station shown the rising tendency & some show the falling.

Therefore a further analysis of Kolaba rainfall has been carried out for the later period (1950-1996) which results as no significant trend.

Long-term persistence tests were carried out for four stations having a data length of more than 20. The Hurst's Co-efficient (k) have been checked against empirical percentage points from the table. The result shows that the Long-term persistence is only present in Kolaba rainfall data at 5% significance level, but is absent at 1% significance level.

Table 6. 1 Results of short-term and long-term dependence of the rainfall time series

Sl No	Station name	r_1	Anderson's Correlogram test		Turning point test		Mann-Kendall test		Long Term Persistence Test	
			$\lvert Z \rvert$ value	Remarks	$\lvert Z \rvert$ value	Remarks	Z value	Remarks	Hurst's (k)	remarks
1	Kolaba	.07	.784	Random	1.058	Random	+3.11	Rising trend	.744	Long term persistent
2	Santacruz	.17	1.28	Random	.236	Random	-1.01	No significant trend	.710	Not persistent
3	Alibag	-.05	.298	Random	.208	Random	+1.41	No significant trend	.6230	Not persistent
4	Poddar	-.21	.620	Random	.609	Random	-.032	No significant trend		
5	Bhira	.15	1.01	Random	.863	Random	-2.32	Falling trend	.7090	Not persistent

Note: All the test results are given on the basis of 5% significance level

The rainfall data analysis shows in general a stationary behavior of the data and implies that there is no influence of climate change on the stream flow pattern for the region.

6.2 ANNUAL FLOOD SERIES DATA

The tests for randomness & trend were performed for the annual flood series of 12 gauge & discharge sites and the subsequent results are shown in table 6.2. Only the data series of station Alman has been found to be non-random by carrying out both the tests for checking randomness but simultaneously no trend has been detected for the same. Other data series are found to be random and trend free. Although a non-stationary behavior has been observed in the Alman data series, but with remembering its short data length the effect could be ignored.

The elementary statistical parameters for each data series have been estimated for normal as well as log domain and are presented in table 6.3.

Table 6. 2 Results of randomness and trend tests of the annual flood series

SL NO.	Name of gauge site	Turning Point Test		Anderson's Correlogram test		Kendall's Rank Correlation Test	
		$\|Z\|$ value	Remarks	$\|Z\|$ value	Remarks	Z value	Remarks
1	Alman	2.04	Not Random	3.26	Not Random	.1.26	No significant trend
2	Gadhi	0.95	Random	0.02	Random	0.73	No significant trend
3	Kalamb	0.15	Random	0.06	Random	-0.29	No significant trend
4	Kamarli	0.68	Random	1.77	Random	-1.26	No significant trend
5	Kanpoli	0.95	Random	0.88	Random	0.49	No significant trend
6	Kolhare	0.78	Random	0.28	Random	1.79	No significant trend
7	Naldhe	0.35	Random	1.37	Random	-0.37	No significant trend
8	Pise	0.00	Random	0.33	Random	1.24	No significant trend
9	Saivan	0.95	Random	1.44	Random	2.22	Rising trend (No trend at 10% S.L.)
10	Titwala	0.47	Random	0.99	Random	0.00	No significant trend
11	Turade	0.78	Random	1.21	Random	-1.17	No significant trend
12	Wadi	0.68	Random	1.64	Random	0.60	No significant trend

Note: All the test results are given on the basis of 5% significance level

Table 6. 3 Elementary statistical parameters for annual flood series

Station name	Original series					Log series				
	Mean	Std. Dev.	C_s	C_k	r_1	mean	Std. Dev.	C_s	C_k	r_1
ALMAN	686.13	530.56	1.32	4.28	.791	6.28	.73	.33	2.76	.608
GADHI	333.51	145.91	.67	3.41	-.089	5.72	.46	-.37	4.01	.003
KALAMB	146.61	66.24	.65	2.95	.395	4.89	.45	.08	2.43	.348
KAMARALI	88.02	47.07	.36	2.59	.161	4.33	.6	-.37	2.74	.294
KANPOLI	441.86	196.76	.69	2.99	-.022	5.99	.46	-.21	2.76	.064
KOLHARE	1153.74	614.08	.45	2.49	-.185	6.91	.55	.08	2.11	-.123
NALDHE	233.97	94.79	1.15	4.52	-.34	5.38	.38	.21	3.15	-.392
PISE	993.06	542.79	.63	3.24	-.259	6.77	.56	.15	2.92	-.237
SAIVAN	970.04	380.38	.17	3.35	.362	6.8	.42	-.4	3.58	.534
TITWALA	1997.68	596.79	.33	3.68	-.357	7.56	.31	-.31	3.07	-.404
TURADE	954.58	451.23	.41	2.7	.262	6.75	.5	-.15	2.69	.382
WADI	206.74	191.4	2.42	9.13	.352	5.08	.65	1.4	4.85	.385

For shorter data period of annual flood series a non-stationary behavior present in a single station does not imply any major climatic variability or change in Land-use pattern and sometimes may be present with no obvious cause. Therefore if may be concluded that the data are free from any non-stationary behavior or strong trend and the subsequent flood frequency analysis will not be influenced by the same.

6.3 REGIONAL FLOOD FREQUENCY ANALYSIS

The results of at site flood frequency analysis for gauged sites as well as regional flood frequency modeling for ungauged sites are presented and discussed under this section.

6.3.1 Regional homogeneity test

6.3.1.1 Discordancy Measures

The L-CV, L-skewness & L-kurtosis for each site are calculated and presented in Table 6.4. The discordancy measures D_i for 12 sites are also shown in the Table 6.4 and finds the maximum D_i as 2.62 at Wadi site. The critical value of D_i for 12 numbers of sites as per Hosking and Wallis (1997) is 2.757, which is not even exceeded by the site with maximum one. This clearly indicates that all the sites are non-discordant.

6.3.1.2 Heterogeneity Measures

The heterogeneity measures H_1 , H_2 & H_3 are found as 1.90, -0.14 & 0.04. A higher value of H_1 is perhaps due to the unusual flooding events of Mumbai region in 2005 and exhibited in flood series at some of the sites. In respect to flood frequency analysis the extreme flood date are precious in nature and elimination of these data for removing a little heterogeneity will not be wise. Moreover the value of H_2 statistics appears quite low with favouring the consideration of the region as homogeneous. Therefore the region has been treated here as homogeneous region and further analysis has been carried out.

Table 6. 4 L-moments and discordancy measures

Sl. No.	Station Name	Record length (n)	L-CV	L-skew	L-kurt	Discrdancy measure (D_i)
1	Alman	14	.4165	.3653	.1442	2.57
2	Gadhi	13	.2523	.1929	.1408	.15
3	Kalamb	31	.2523	.1684	.1164	.14
4	Kamarli	14	.2614	.1816	.0135	.38
5	Kanpoli	13	.3154	.1092	-.0076	1.02
6	Kolhare	11	.3113	.1827	-.1150	1.26
7	Naldhe	22	.2215	.2345	.1972	.64
8	Pise	8	.3226	.2403	-.0621	1.08
9	Saivan	8	.2393	.0529	.0088	.50
10	Titwala	13	.1727	.0473	.1469	1.42
11	Turade	11	.2801	.1352	-.0259	.21
12	Wadi	14	.3099	.5341	.3938	2.62
WEIGHTED MEANS			***.2741***	***.2087***	***.1008***	

6.3.2 REGIONAL DISTRIBUTION

6.3.2.1 Selection of the Best Fit Distribution

Initial Judgement

The Regional values of L-skewness and L-kurtosis as 0.2087 and 0.1008 respectively are plotted as a point on the standard L-moment ratio diagram as shown in the fig 6.1 which gives the judgement in favour of Generalized Pareto Distribution (GPA), Pearson Type-3 (PE3) distribution.

Goodness of Fit Measure

The Z^{Dist} statistics for different distributions are computed and are shown in table 6.5. Two distributions namely GPA and PE3 distributions are satisfying the fitting criteria with $|Z|$ values as 0.55 and 1.15 respectively, which matches with the initial judgement. As the GPA distribution is useful for analysis of peaks-over-threshold data and normally is not recommended for annual peak data, the PE3 distribution has been adopted as the best fit distribution.

Table 6. 5 Goodness of fit measures for various distributions

Distribution	L-kurtosis	Z^{Dist}
General Logistic (GLO)	.203	3.17
General Extreme Value (GEV)	.167	2.05
General Normal (GN)	.157	1.75
Pearson Type 3 (PE 3)	.137	1.15 *
General Pareto (GPA)	.082	-.55 *

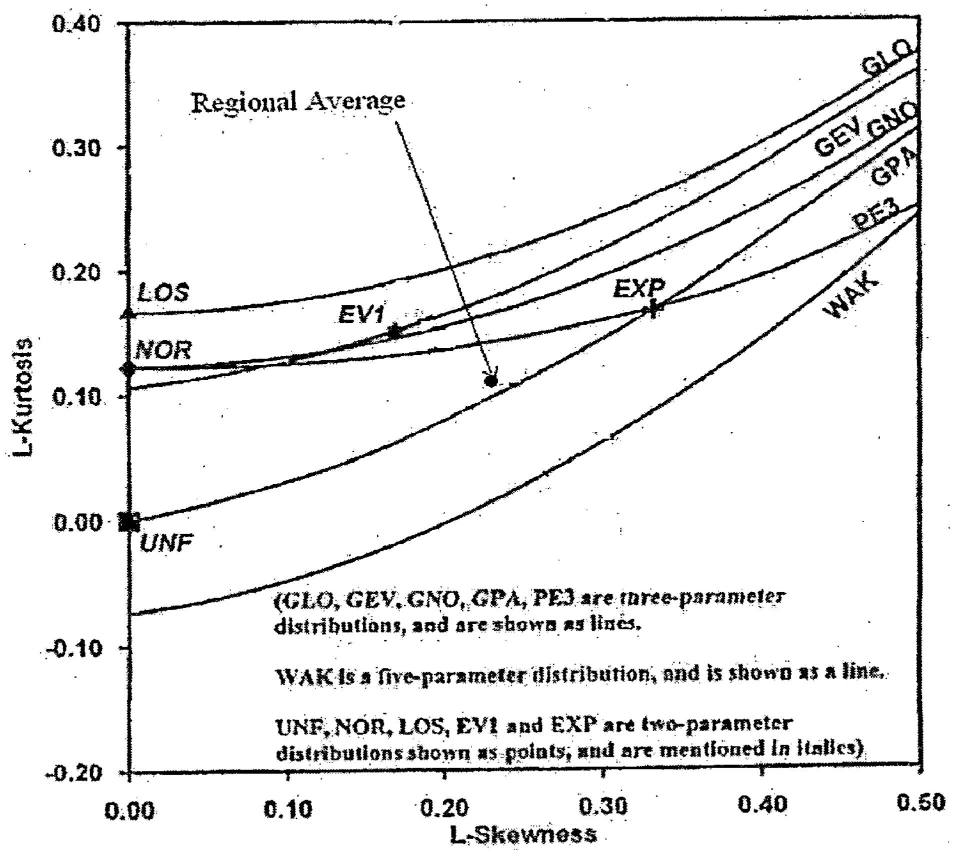

Figure 6. 1 Initial Judgement from L-Moment Ratio Diagram

6.3.2.2 Estimation of Regional Quantile Parameters

The regional parameters for various distributions are presented in Table 6.6. The regional quantile parameters for PE3 distribution are found as 1.000 (location parameter,u), 0.510 (scale parameter,α)& 1.262 (shape parameter,k). As the PE3 distribution can not be expressed in inverse form, the derivation of regional quantile equation is complicated. Therefore as an alternative the growth factors with PE3 distribution at different return periods are calculated and presented in Table 6.7.

Table 6. 6 Regional parameters for various fitted distributions

Distribution	Parameters of the distributions
Pearson Type 3 (PE 3)	μ = 1.00 α = .51 k = 1.262
General Pareto (GPA)	ξ = .367 α =.829 k = .309
Wakeby distribution	$\hat{\varepsilon}$ = .00 $\hat{\alpha}$ = 43.4 $\hat{\beta}$ = 111.56 $\hat{\gamma}$ =.78 $\hat{\delta}$ = -.269

Table 6. 7 Growth factors ($Q_T/\overline{Q}$) at different return periods with PE3 distribution

Return Period (years)	2	5	10	25	50	100	200	500	1000
Growth Factor	0.90	1.35	1.68	2.07	2.37	2.63	2.93	3.26	3.50

6.3.3 Flood quantile estimation

6.3.3.1 Gauged Catchments

For gauged catchments the flood quantiles can be estimated as,

$$Q_T = \text{(growth factor at return period T)} * \overline{Q} \tag{6.1}$$

The flood quantile values at different gauged sites of the study area at different return periods are presented in Table 6.8.

6.3.3.2 Ungauged Catchments

The areas and mean annual peak floods for available gauged catchments are plotted on the graph paper and trend line has been drawn as shown in fig 8.2. The resulting relationship between mean annual peak flood ($\overline{Q}$) in cumecs and area of catchments (A) in sq km are obtained as,

$$\overline{Q} = 5.1848\,(A)^{0.8334} \tag{6.2}$$

The correlation coefficient 'r' for the above equation is found to be 0.93. Flood quantile for ungauged catchments can be expressed as,

$$Q_T = (Q_T/\overline{Q})*[5.1848*(A)^{0.8334}] \tag{6.3}$$

Fig 6.2 shows the graph resulting from regression analysis which represents the relationship between mean annual maximum flood and catchment area.

Table 6. 8 Flood Quantiles at gauged sites with different return periods

Station Name	Flood Quantiles At Various Return Perids (cumecs)				
	T = 5 yrs	T = 10 yrs	T = 25 yrs	T = 50 yrs	T = 100 yrs
ALMAN	926	1153	1420	1626	1805
GADHI	450	560	690	790	877
KALAMB	198	246	303	347	386
KAMARALI	119	148	182	209	231
KANPOLI	597	742	915	1047	1162
KOLHARE	1558	1938	2388	2734	3034
NALDHE	316	393	484	555	615
PISE	1341	1668	2056	2354	2612
SAIVAN	1310	1630	2008	2299	2551
TITWALA	2697	3356	4135	4735	5254
TURADE	1289	1604	1976	2262	2511
WADI	279	347	428	490	544

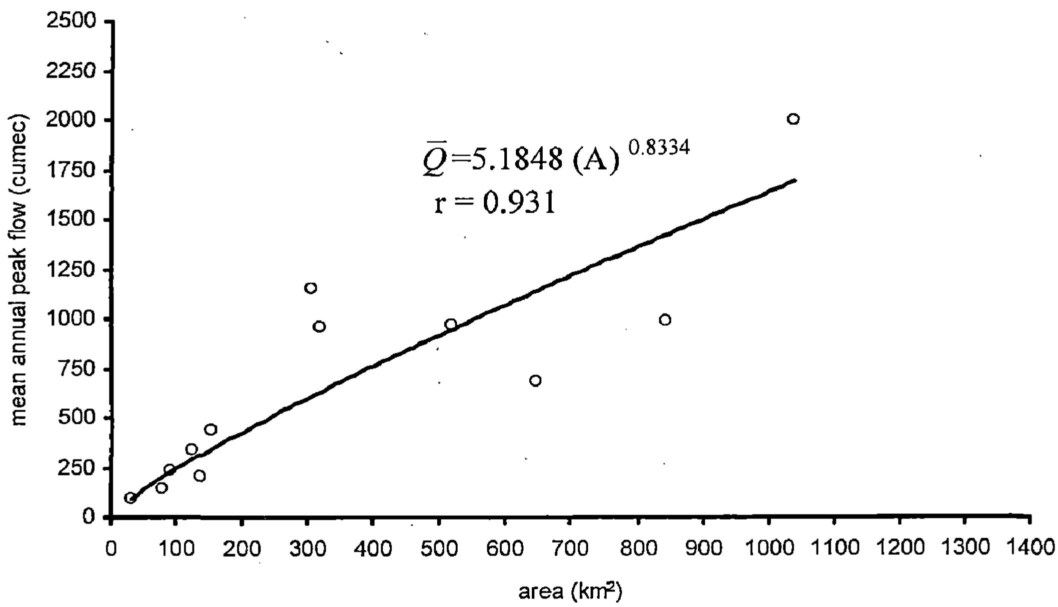

Figure 6. 2 Relationship between mean annual peak flood and catchment area of gauged sites

6.4 REGIONAL UNIT HYDROGRAPH ANALYSIS

6.4.1 Physiographic parameters

The four numbers of catchments under this analysis have been delineated and are shown from fig 6.3 to fig 6.6. The catchments are mostly oblong shaped. The catchments areas calculated were found to be matched with the area data supplied by the state storage data centre, Nasik. The parameters L, Lc & S have also been estimated and are presented with their corresponding catchments figures.

6.4.2 Derivation of 1hr synthetic unit hydrograph

SUH parameters for all the catchments are computed using equations as mentioned in sec 5.5.2 and are presented in Table 6.9. In computation of the parameters t_p and T_B the results have been rounded off to their nearest value.

The ordinates of the resulting unit hydrograph have been adjusted in their falling limb for equalizing the volume obtained from the U.H. with the volume of 1.00 cm direct runoff depth over the catchments as calculated and shown in the table numbering from 6.10 to 6.13 along with the corresponding unit hydrograph figure. The modified SUH for all the catchments are shown in the figure numbering from 6.7 to fig 6.10 along with their adjusted ordinate values.

Table 6. 9 One hour regional SUH parameters

Station Name	AREA (sq km)	t_p (hrs)	q_p ($m^3/s/km^2$)	Q_p (m^3/s)	t_r (hrs)	T_B (hrs)	T_M (hrs)	W_{50} (hrs)	W_{75} (hrs)	W_{R50} (hrs)	W_{R75} (hrs)
SAIVAN	519	8.50	0.208	108.07	1.00	36.00	9.00	10.64	5.25	3.29	1.81
PISE	844	9.50	0.196	165.18	1.00	37.00	10.00	11.38	5.60	3.52	1.94
KOLHARE	306	5.50	0.335	102.64	1.00	26.00	6.00	6.33	3.19	1.94	1.10
TURADE	317	6.50	0.258	81.84	1.00	30.00	7.00	8.42	4.19	2.59	1.45

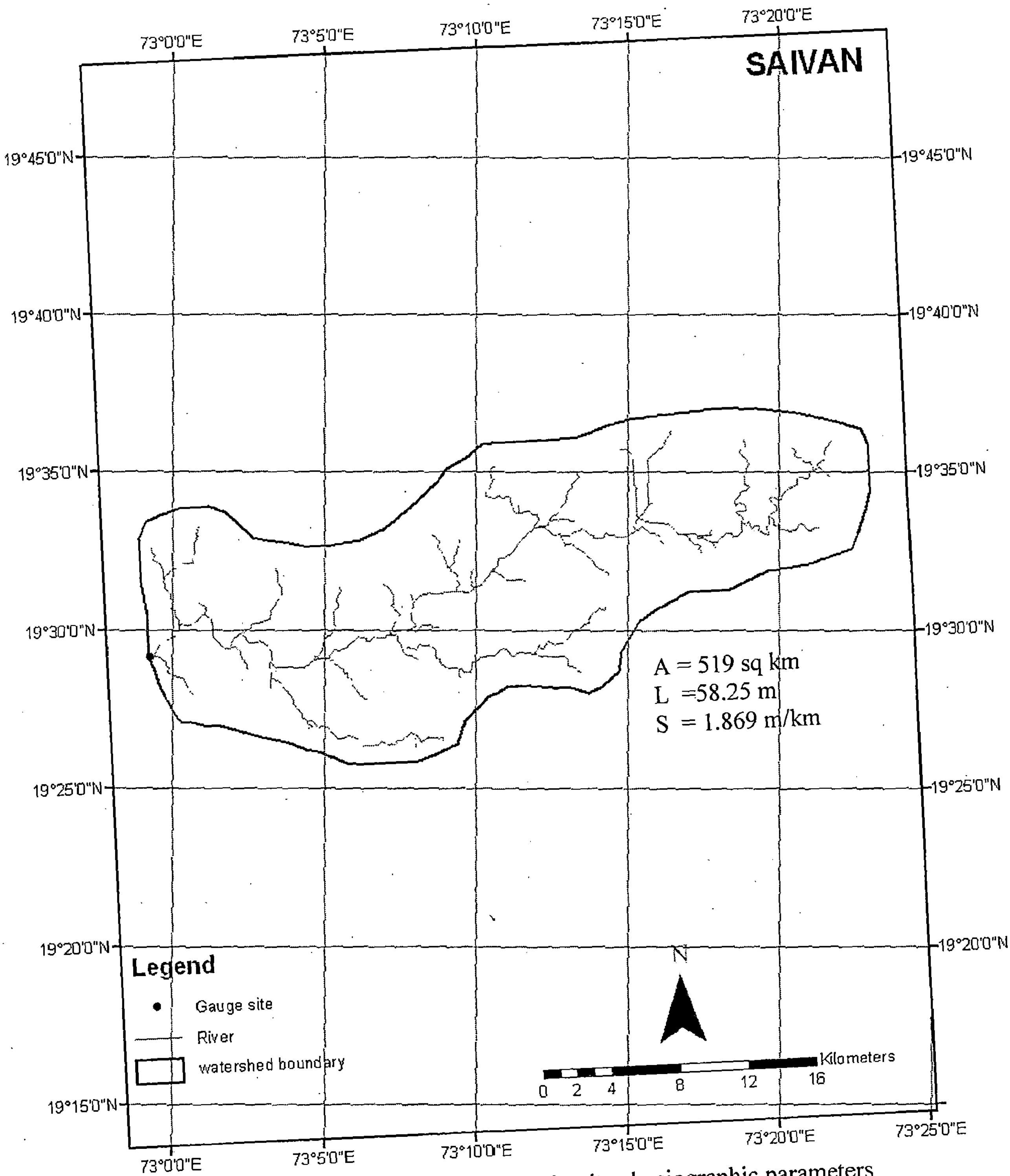

Figure 6. 3 Catchment Saivan showing its physiographic parameters

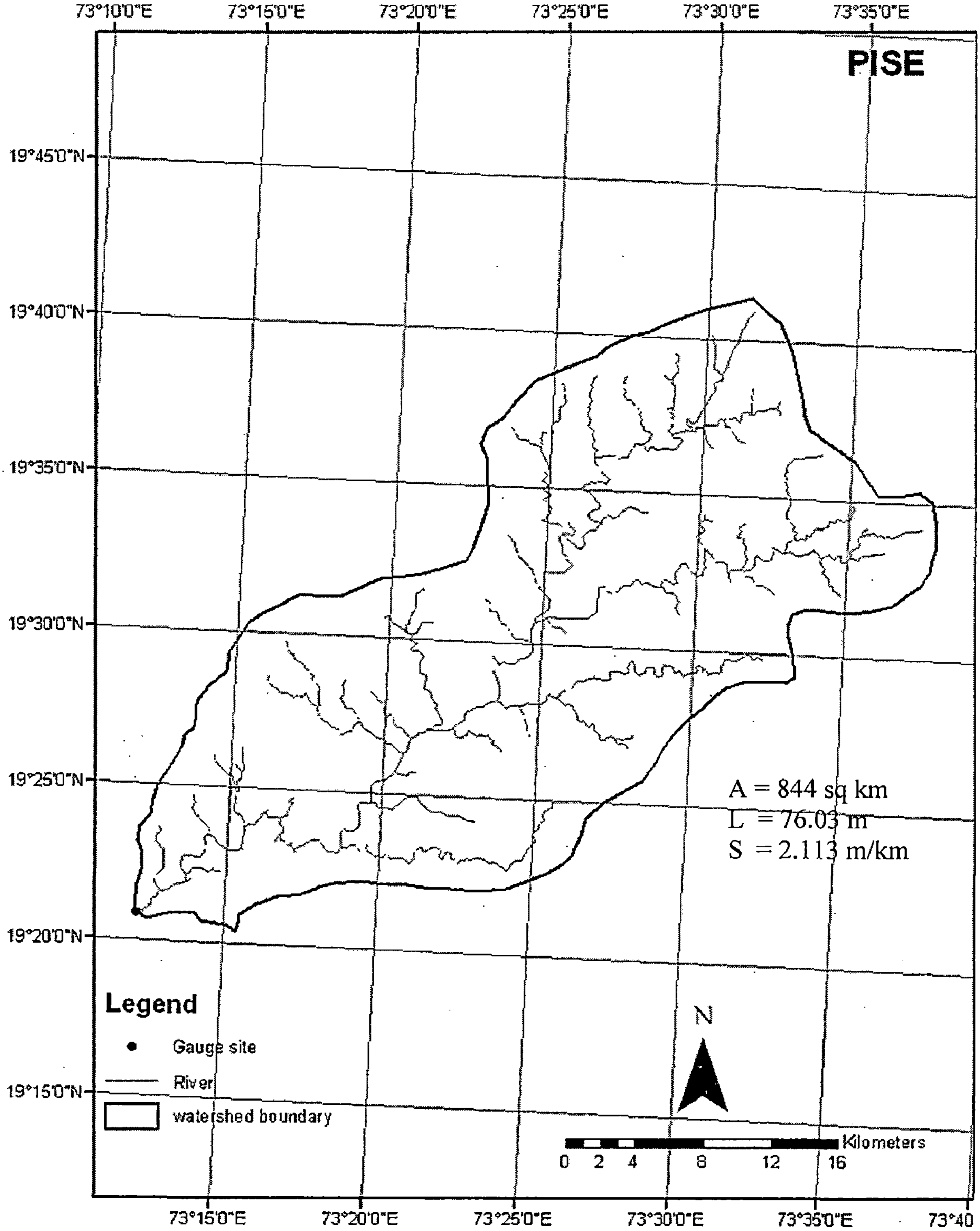

Figure 6. 4 Catchment Pise showing its physiographic parameters

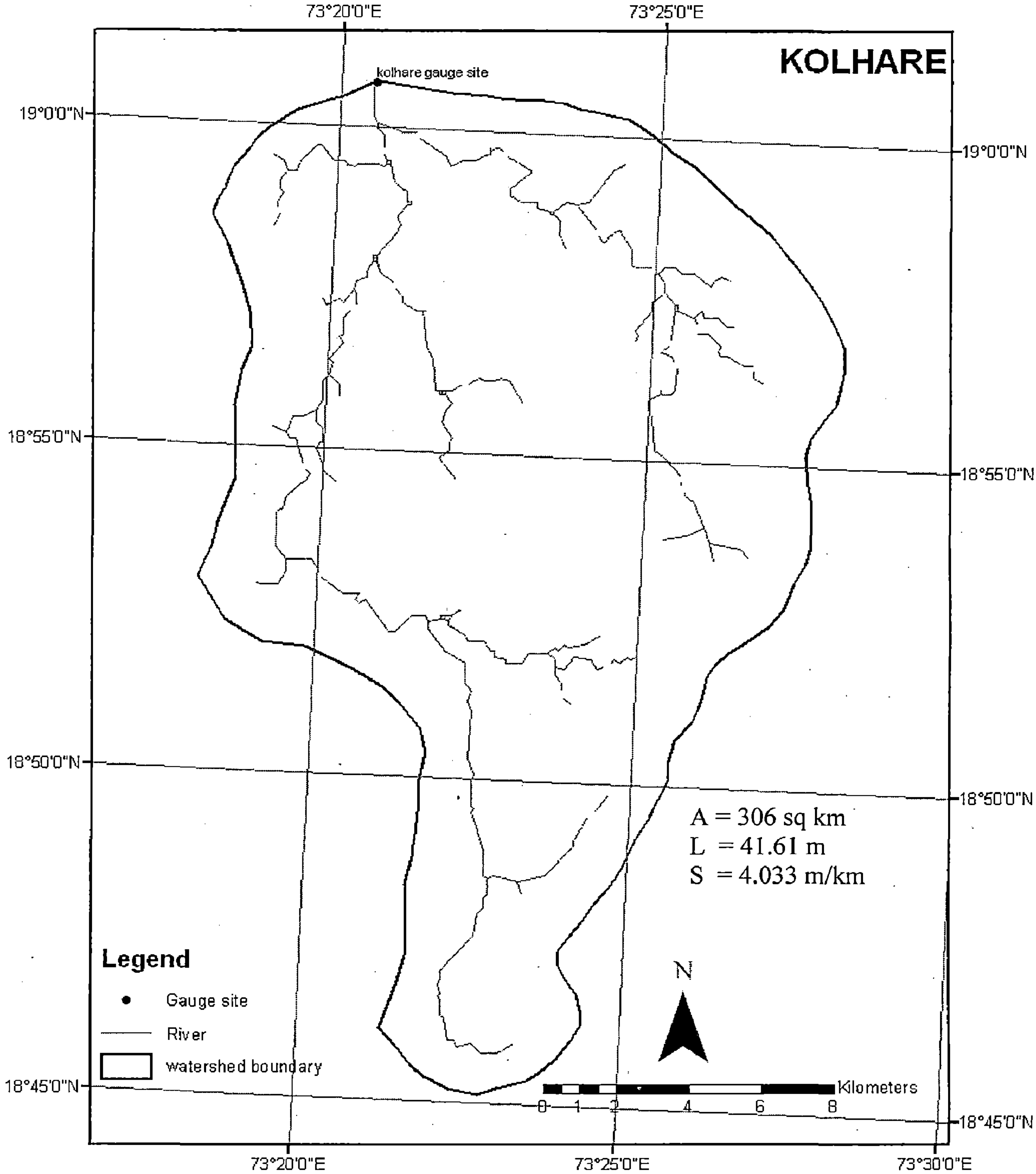

Figure 6. 5 Catchment kolhare showing its physiographic parameters

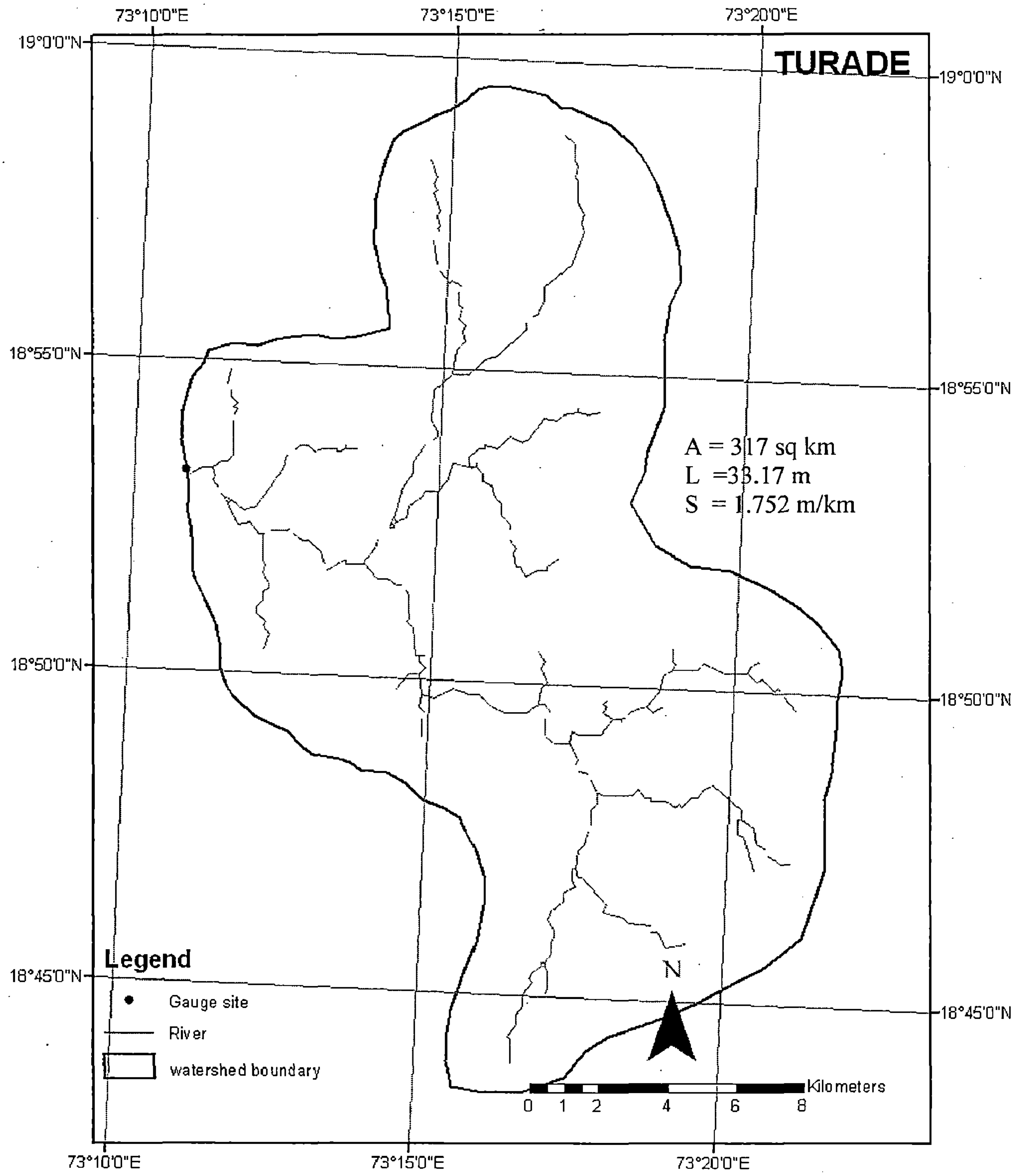

Figure 6. 6 Catchment Turade showing its physiographic parameters

Table 6. 10 One hour SUH ordinates for Saivan catchment (CWC method)

Time (hr) (i)	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Ordinate 1hr UH (ii)	0	8	17	25.5	33.5	44	56	70.5	96	108	103	95	86	76.5
(i)	.....	14	15	16	17	18	19	20	21	22	23	24	25	26
(ii)	.....	70.5	63	56	50.5	45	40.5	36.5	33	30	27.5	25.5	23.5	21.5
(i)	.....	27	28	29	30	31	32	33	34	35	36			
(ii)	.....	19.5	17.5	15	13	11	9	7	5	2.5	0			

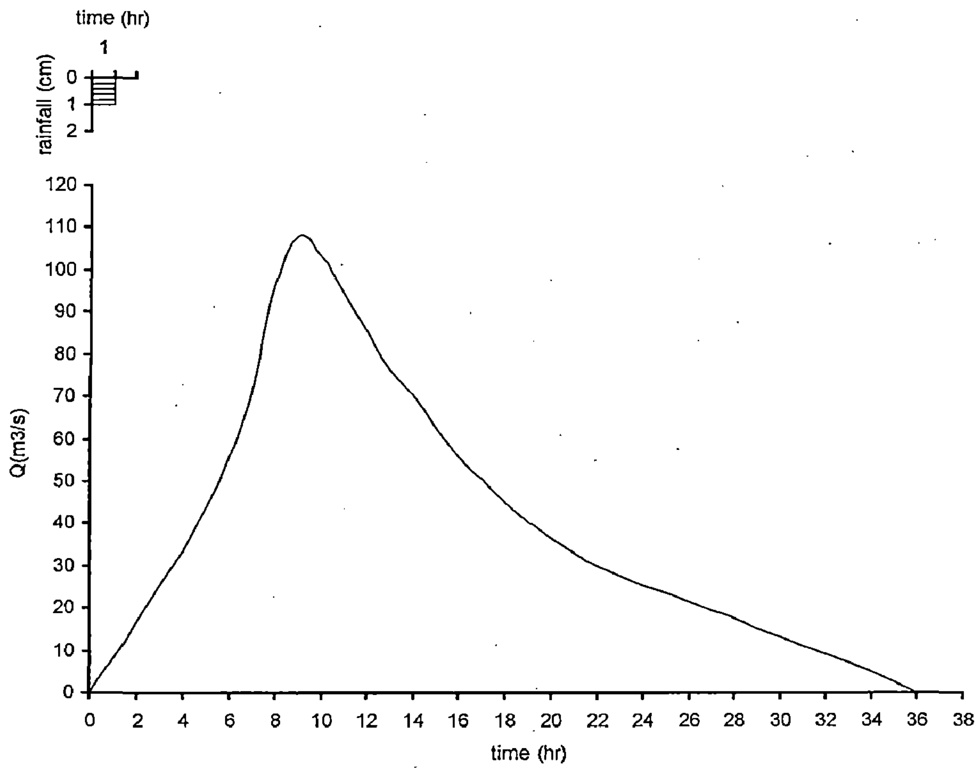

Figure 6. 7 One hour SUH for Saivan catchment (CWC method)

Table 6. 11 One hour SUH ordinates for Pise catchment (CWC method)

Time (hr) (i)	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Ordinate 1hr UH (ii)	0	6	15.5	26.5	39	52	66.5	90	124	151	165	159	144	131.5

(i)	....	14	15	16	17	18	19	20	21	22	23	24	25	26
(ii)	....	119	107.5	95.5	87.5	80.5	75	71	66.5	62	57	52.5	48	44

(i)	....	27	28	29	30	31	32	33	34	35	36	37
(ii)	....	39	35	31	26.5	22.5	18.5	14.5	11	7	3.5	0

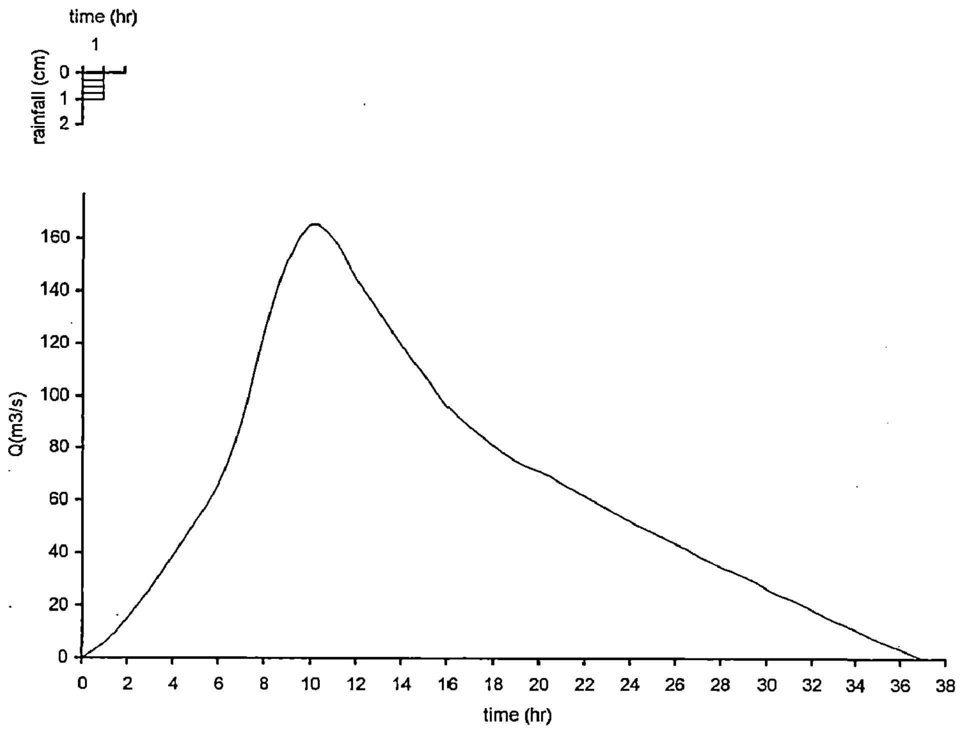

Figure 6. 8 One hour SUH for Pise catchment (CWC method)

Table 6. 12 One hour SUH ordinates for Kolhare catchment (CWC method)

Time (hr) (i)	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Ordinate 1hr UH (ii)	0	6	15.5	27	45.5	81	102.64	91	76.5	63.5	53	46	40	35

(i)	14	15	16	17	18	19	20	21	22	23	24	25	26
(ii)	30	26	23	20	17	14	11	8.5	7	5	4	2	0

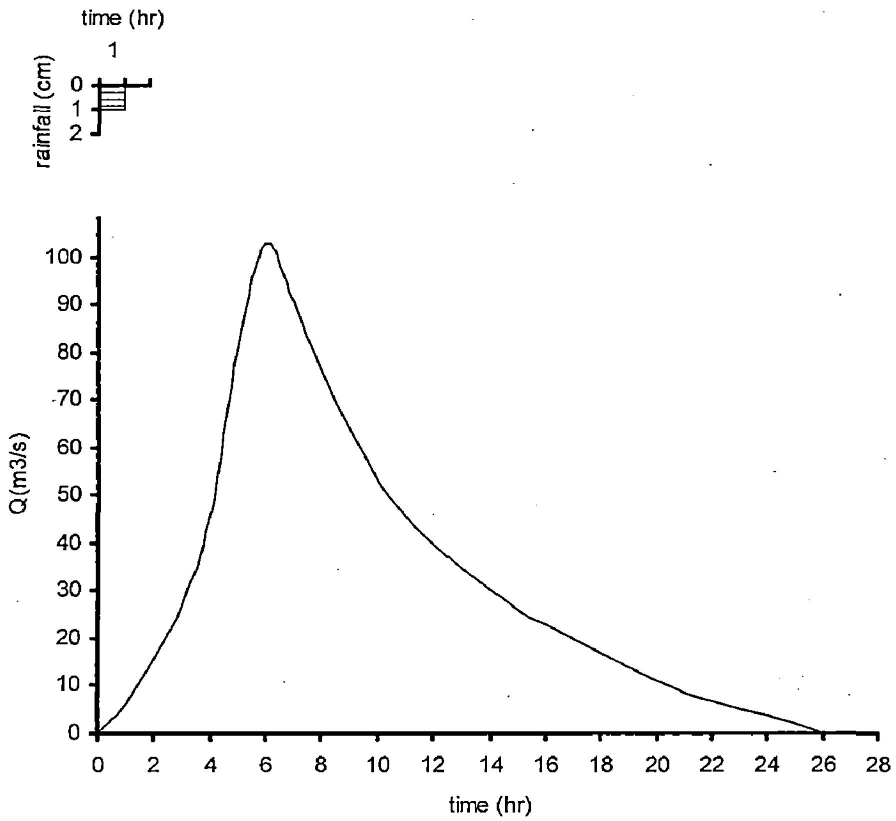

Figure 6. 9 One hour SUH for Kolhare catchment (CWC method)

Table 6. 13 One hour SUH ordinates for Turade catchment (CWC method)

Time (hr) (i)	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Ordinate 1hr UH (ii)	0	6	13	22.5	33.5	53	74	82	74.5	66	59	52	45	39.5

(i)	14	15	16	17	18	19	20	21	22	23	24	25	26
(ii)	35	31.5	28.5	25.5	22.5	20	18	16	14	12	10.5	9	7

(i)	27	28	29	30
(ii)	5	3.5	2	0

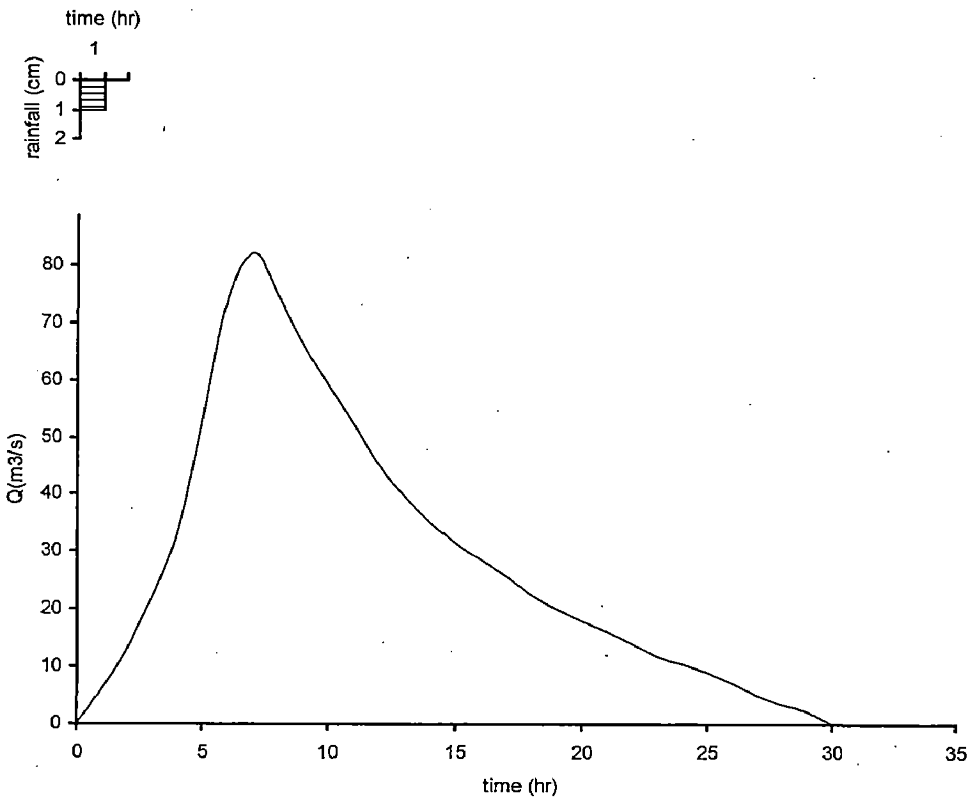

Figure 6. 10 One hour SUH for Turade catchment (CWC method)

6.4.3 Design storm

The design storm durations for the catchments have been adopted as 1.1 t_p and the computed values have been rounded off to its nearest hours.

The design storm duration, point & areal rainfalls for 25 yrs, 50 yrs &100 yrs, return periods for the catchments under analysis are tabulated in Table 6.14 in a sequential manner.

The effective rainfall increments have been found out by using time distribution co-efficient and by considering design loss at the rate of 0.19 cm/hr.

6.4.4 Design flood

The design flood values for the catchments for 25 yrs, 50 yrs & 100 yrs, return periods have been calculated by following the procedure mentioned in the previous chapter. The design flood values can be referred from Table 6.15.

6.5 REGIONAL UNIT HYDROGRAPH ANALYSIS VERSUS REGIONAL FLOOD FREQUENCY ANALYSIS

To check the performance of regional U.H. approach in case of flood predictions in ungauged basins of MMR, the design floods have been calculated for the catchments under consideration by using regional flood frequency formula for ungauged catchments of MMR as derived under this study for 25 yrs, 50 yrs, & 100 yrs, return periods.

The comparative values of design flood with 25 yrs, 50 yrs & 100 yrs return periods by various approaches are given in Table 6.15 and are presented with the help of the Bar Charts as shown in the fig 6.11, 6.12 & 6.13.

It is found that for the catchments Saivan & Pise having the areas of 519sqkm & 844sq km respectively, the results obtained from both the approaches are approximately same with negligible deviations. Although the results of Pise are found to be overestimated in comparison with the flood quantiles of the respective sites obtained from flood frequency approach for gauged sites, whereas for Saivan site all the three results are very much comparable with each other. The catchments Kolhare & Turade having almost equal areas (306sq km & 317sq km) reflect some how different features. For both the catchments the regional flood quantiles obtained from direct flood frequency approach are more then the same obtained from both regional SUH approach & by using flood frequency formula. Further the flood frequency formula approach gives the lower result in compare to regional SUH approach. The above statements are more prominent in case of the station Kolhare due to its much higher equivalent slope.

The above findings clearly indicate the following,

(i) In case of the catchments with higher values of area, the regional SUH approach and regional flood frequency formula both give over estimation of the results. This is because of the direct influence of the higher magnitude of area for both the approaches.

(ii) In case of moderate area catchments like Saivan the results may be expected to be comparable as obtained from all the three approaches as the direct influence of the area value is reduced here.

(iii) In case of the catchments with comparatively less area due to the less influence of area magnitude the results obtained from regional SUH approach and by using regional flood frequency formula are underestimated.

Table 6. 14 Point and Areal rainfall for the catchments

Station name	Design Storm Duration (T_D) hrs	T yrs 24hrs point Rainfall (mm)			Conversion factor	T yrs T_D hrs point Rainfall (mm)			Areal Reduction Factor (ARF)	T yrs T_D hrs Areal Rainfall (mm)		
		T=25	T=50	T=100		T=25	T=50	T=100		T=25	T=50	T=100
SAIVAN	9	405	440	490	0.70	283.50	308.00	343.00	0.7825	222	241	268
PISE	10	365	410	450	0.73	266.50	299.30	328.50	0.7933	211	237	261
KOLHARE	6	450	490	560	0.61	274.50	298.90	341.60	0.8076	222	241	276
TURADE	7	420	440	510	0.64	268.80	281.60	326.40	0.8118	218	229	265

Table 6. 15 comparative values of design floods with various approaches

Station name	Design Flood Peak(Q_p) (Cumecs)								
	Regional SUH approach			Using Regional Flood Frequency Formula			At site Regional Flood Frequency Analysis		
	T = 25yrs	T = 50yrs	T =100yrs	T = 25yrs	T = 50yrs	T =100yrs	T = 25yrs	T = 50yrs	T =100yrs
SAIVAN	2044	2224	2481	1966	2251	2498	2008	2299	2551
PISE	2940	3317	3665	2948	3375	3745	2056	2354	2612
KOLHARE	1961	2132	2448	1266	1449	1608	2388	2734	3034
TURADE	1545	1625	1887	1303	1492	1656	1976	2262	2511

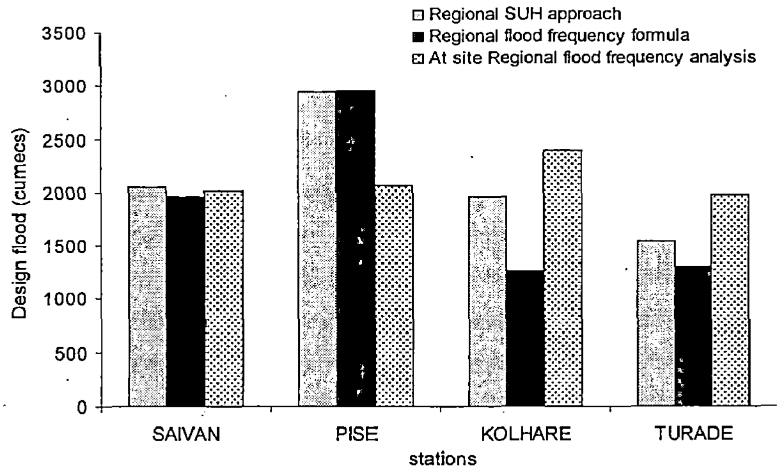

Figure 6. 11 Bar chart of design floods with 25 yrs return period at four sites obtained from three approaches

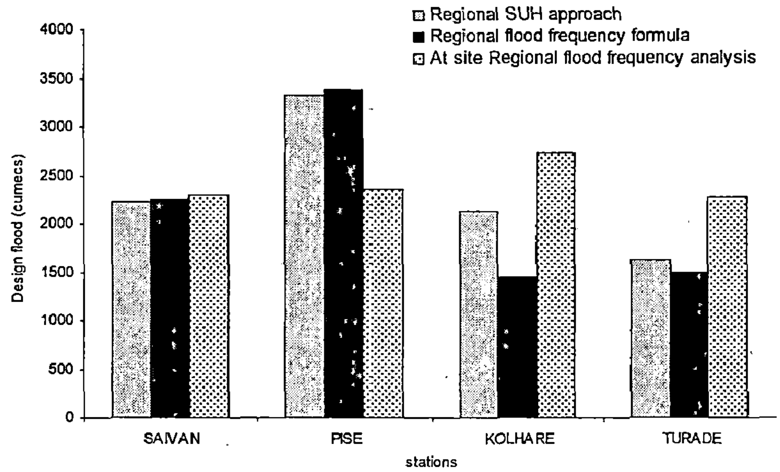

Figure 6. 12 Bar chart of design floods with 50 yrs return period at four sites obtained from three approaches

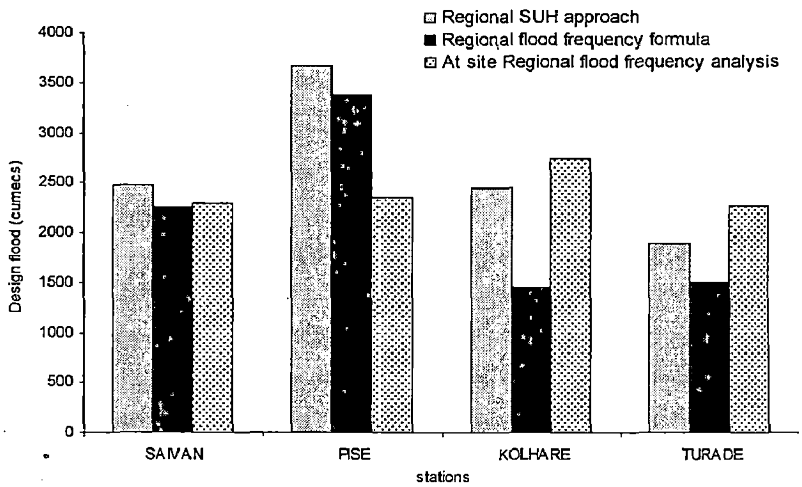

Figure 6. 13 Bar chart of design floods with 100 yrs return period at four sites obtained from three approaches

(iv) Further in case of smaller area catchments SUH approach gives a comparatively higher value of design flood and hence gives more reliable results as these appear to be closer with the results obtained from frequency analysis. The possible reason behind this is that the SUH approaches consider equivalent slope parameter in its analysis whereas the derived flood frequency formula does not consist of the same in it. The greater value of equivalent slope in Kolhare catchment is responsible for its much higher value of flood quantile as obtained from the flood frequency approach for gauged catchments. In case of Turade site the outcome between the approaches are not so prominent because the equivalent slope with its lower value has not significant effect which may create any appreciable difference.

(v) There is no significance difference between the 24 hrs point rainfall between the catchments under discussion as observed from the Isopluvial maps. However the Kolhare site is having a bit of higher value of the same. This is another deciding factor which is also partially responsible for producing higher flood quantile as obtained from the flood frequency analysis, as this factor has not been considered while deriving the flood frquency formula.

The above findings are strongly supported by the results obtained from the site considered in the analysis of Central Water Commission in their Flood Estimation Report for West Coast Region (sub zones 5a & 5b). The SUH parameters were derived by analyzing 13 bridge sites out of which one is situated at the southern boundary of MMR. The name of the section where the bridge is located is Kolad-Panvel denoted as MOT (2) in Flood Estimation Report. The site is located on the stream Amba with latitude & longitude of. 18°32″ & 73°12″ respectively. The catchment area is 310sq km which is approximately same as the areas which Kolhare or Turade have. The design floods obtained by SUH approach for the site are

1823.95 Cumecs, 2042.47 Cumecs, & 2313.16 Cumecs with return periods of 25 yrs, & 100 yrs respectively. The values of design floods indicate that these are in betwe values as obtained from Turade & Kolhare site. The equivalent slope having a value c also lies in between the same for Turade & Kolhare sites. Further the design flood valı the aforesaid site obtained by using flood frequency formula as derived for ung catchments are computed as 1279.39 cumecs, 1464.81 cumecs & 1625.51 cumecs with periods of 25 yrs, 50 yrs, & 100 yrs, respectively. The results again reflect that the considerably less than the same obtained from the SUH approach.

CHAPTER 7
CONCLUSIONS

7.1 GENERAL

Mumbai city and its surroundings lying under Mumbai Metropolitan Regional Development Authority (MMRDA) is the cream region of the country. Due to extreme rainfall the entire region has been unfortunately suffering with massive flood threats. The severity of the condition is likely to be further deteriorated due to fast growing urbanization of the entire region. This study was an attempt to estimate the design flood for various gauged and ungauged catchments of the area by adopting different regional approaches. The results, outcome and recommendations of the study may assist in design of various structural and nonstructural measures for flood protection. The conclusions of the study are as under.

(i) The rainfall data analysis does not reflect any significant non stationary behavior which may affect on the stream flow pattern of the region.

(ii) The annual flood series data of 12 gauged sites are mostly free from any non-stationary behavior though for small record length the detéction of same is not very significant.

(iii) The design flood for different return periods at 12 number of gauged sites have been estimated by at site regional flood frequency analysis with L-moments approach. For this purpose, the analysis has been carried out with 176 station years data of 12 gauged sites of MMR. The Pearson type-3 distribution was found to be best fit distribution for the region.

(iv) For ungauged catchments of the region a relationship between mean annual peak flood and catchment area has been established. The flood quantiles are to be estimated as the product of the mean annual peak flood obtained from known area and the growth factors for the desired return periods.

(v) The regional synthetic unit hydrograph approach has been adopted for four catchments namely Saivan, Pise, Kolhare and Turade having comparatively less number of record lengths. The design floods have been estimated for 25 yrs, 50 yrs, & 100 yrs, return periods.

(vi) Statistical approach based on frequency analysis is a direct approach and will generally be the first choice of method when there is a sufficient record of gauged floods close to the site of interest are available.

(vii) The flood quantiles estimated for gauged sites with at site regional approach are superior as compared to any other approache. Even in case of other gauged sites with inadequate record length which have not been considered in this study, the same approach may be adopted.

7.2 RECOMMENDATIONS

The results obtained from different approaches were compared and discussed in results and discussion chapter. The general recommendations on the basis of the findings of the analysis are as follows..

(i) If long term data are available with any gauged site within the MMR region, it is recommended to estimate the design flood by adopting the at site regional analysis. The steps of the method are given below for ready reference.

(a) Estimate mean annual peak flood ($\overline{Q}$) from annual flood series data of the given site.

(b) Get growth factor ($Q_T/\overline{Q}$) for the desired return period from the table given below.

Return Period (years)	2	5	10	25	50	100	200	500	1000
Growth Factor	0.90	1.35	1.68	2.07	2.37	2.63	2.93	3.26	3.50

(c) Get design flood (Q_T) by multiplying the growth factor ($Q_T/\overline{Q}$) by $\overline{Q}$.

It is worth noting that the design floods with different return periods of gauged sites of MMR considered under this study are presented in table 6.8.

(ii) When there is a very short record available at the subject site which overlaps a much longer record nearby, it may be fruitful to extend the record by a regression method and corresponding design flood may be computed with the at site regional approach. Even then the results are expected to be more reliable than that which may be obtained from the other approaches by treating the site as ungauged.

(iii) The design flood (Q_T) for ungauged catchments with no or ignorable urbanization may be computed by either of the regional synthetic unit hydrograph approach or by using regional flood frequency formula derived under this study. The recommendations in this context are as follows.

(a) For the catchments having an area of more than 500 sq km, the use of regional flood frequency formula is recommended for computation of design flood due to its simplicity & reliability for this area range. The formula is given as under.

$$Q_T = (Q_T/\overline{Q}) * [5.1848 * (A)^{0.8334}]$$

(b) SUH approach is recommended for catchments having less area.

(c) Further for the catchments with an appreciable equivalent slope, SUH approach is recommended

(iv) A considerable region of MMR is heavily urbanized and therefore it is worth noting that any of the statistical approaches is not recommended for those regions. In these cases the Rational method may be applied for flood computation. The Rational Formula is the most commonly used method of determining peak discharges from small drainage areas. This method is traditionally used to size storm sewers, channels and other stormwater structures.

The Rational Formula is expressed as per (IRC-SP-50)

$$Q_P = 2.78\ CiA$$

Where,

Q_P = peak run-off rate (m^3/s)

C = run-off coefficient

i = rainfall intensity (cm/hr.) of a storm whose duration is equal to the time of concentration of the basin.

A = Area of watershed (km^2)

7.3 LIMITATIONS OF THE STUDY

The various approaches for estimation of flood in MMR taken up in the study under discussion have got a few limitations which are as follows.

(i) The approaches are not applicable in case of moderate to heavily urbanized area of the region. The analysis for gauged catchments with moderate urbanization may be practical but requires careful study and special treatment for the site by considering the effect of the same. In case of ungauged urban catchments neither the resulting relationship nor the SUH approach is valid for estimation of flood for those same.

(ii) The gauged catchments with few years of data period may suffer from the over or under estimation of its mean annual peak flood and hence its design flood. The logic behind this is that one or two exceptional value of annual flood peak may appreciably increase its mean annual peak flood as it occurred in 2005 in most of the regions of Mumbai. Therefore the results of the analysis for gauged catchments with a small record length may be more reliable with the availability of data in future.

(iii) The results of the regional flood frequency analysis could be refined with the availability of more number of gauged sites.

(iv) In addition to area, other physiographic parameters of the catchments such as drainage density, overland and channel slopes etc can be used to obtain the mean annual peak flood for ungauged catchments by multi regression analysis. In the present study due to lack of data only area of the catchments has been used.

(v) The regional SUH relationships were established by using 7-bridge catchments in sub-zone 5(a) and 6-bridge catchments in sub-zone 5(b) which have a combined effect of both the zones. The results can be more realistic with developing the separate relationships for 5(a) zone by collecting the data for more number of catchments.

(vi) The regional SUH approach was developed mostly covering the catchments with flat to moderate slope. Therefore the design flood computed for the catchments with higher slope is likely to be under estimated by adopting this approach.

7.4 SCOPE OF FURTHER STUDIES

The regionalization approach for flood estimation being a thrust area in the field of hydrology has been fascinating the young researchers from the recent past. The approach is important because a reliable estimation of flood is obtained which is the primary task for the disaster preparedness. The approach is interesting because it requires the proper understanding of physics along with the application of mathematics. Therefore the new developments and modification of the approaches are very frequent in this eye-catching field. Keeping in view the above some of the works in pertaining with this study which may be taken up in future are briefly enlisted below.

(i) The flood frequency relationship for ungauged catchments can be improved by computation of other physiographic parameters of the gauged sites after collecting the proper data required.

(ii) Along with the 12 number of gauged sites data considered under this study, more numbers of gauged sites data for 5(a) region can be collected. Separate and improved relationships of SUH parameters for 5(a) zone can be developed.

(iii) The pooled site analysis for the gauged sites can be carried out by forming the pooling groups consisting of the similar catchments irrespective of their geographical location.

(iv) The different non-structural measures like flood plain zoning, flood risk mapping and flood warning system can be studied and developed with the assistance of flood frequency prediction as analyzed under this study.

(v) Flood computation may be carried out by using rational formula for urbanized catchments of MMR.

(vi) There is also a need to develop early flood warning system for the region. This is recommended to be taken up by the Mumbai Municipal Corporation or MMRDA on priority basis.

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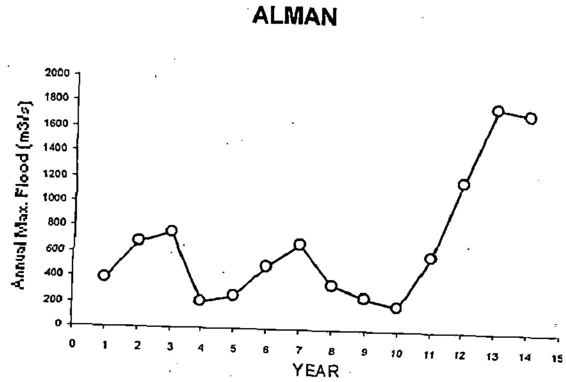
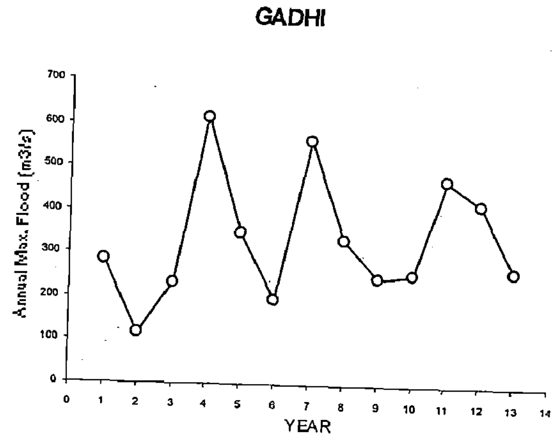
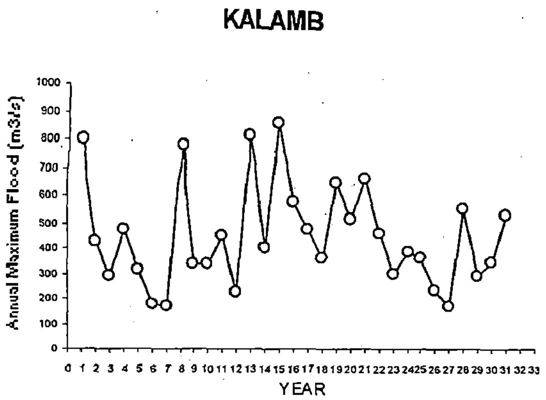
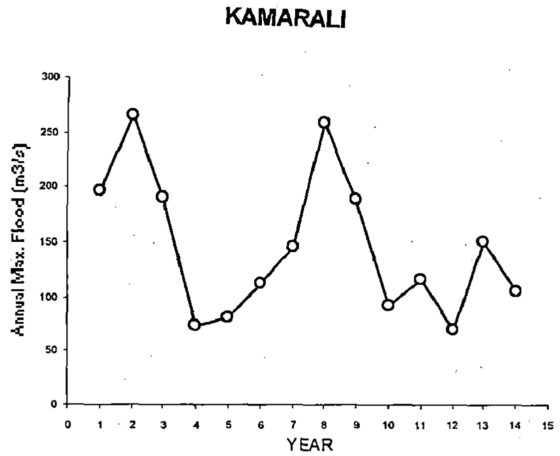
APPENDIX I

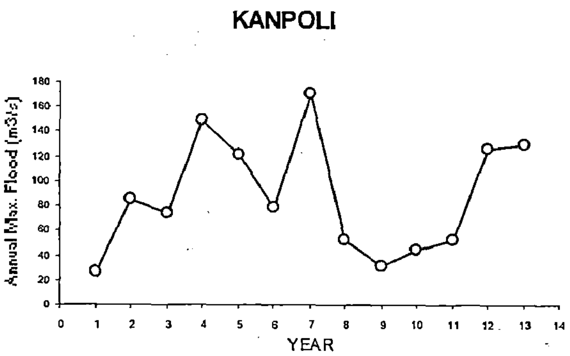
Time Series Plots for Annual Flood Series of 12 number of gauged sites considered under this study are annexed below.

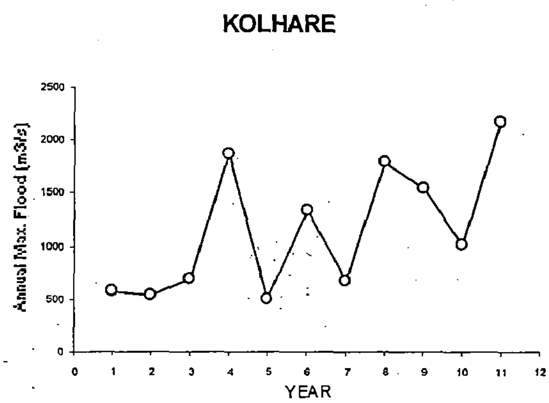












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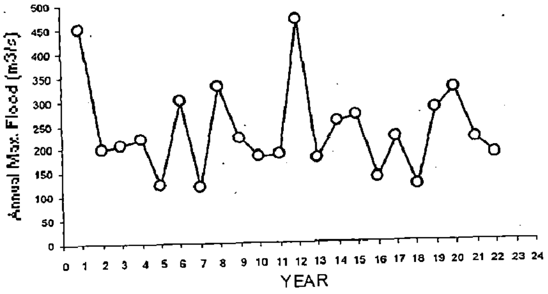

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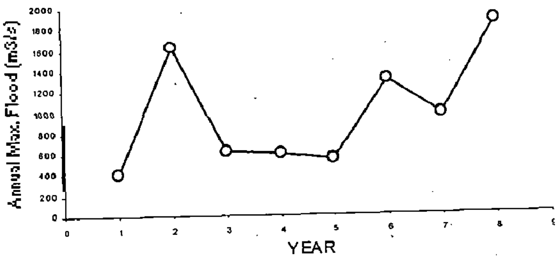

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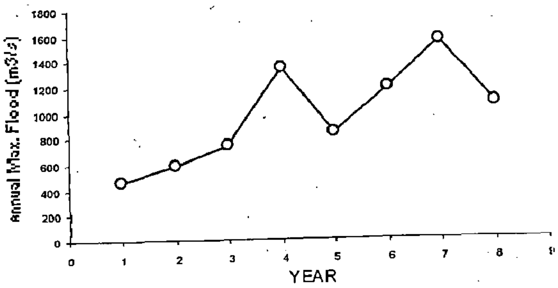

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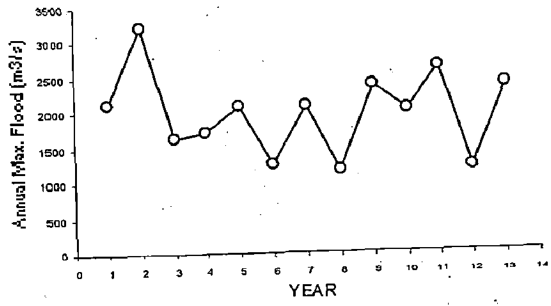

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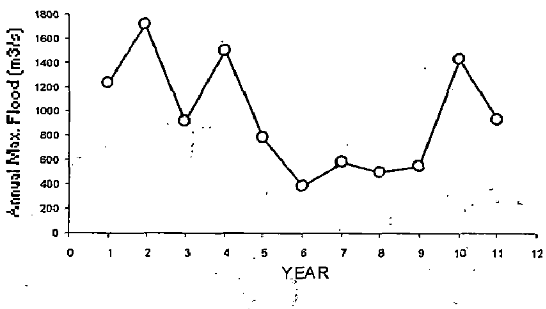

WADI