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Sensitivity Analysis of Hydrological Parameters on Flood Hydrograph

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ABSTRACT

In design flood estimation approach using unit hydrograph techniques, the design unit hydrograph parameters are used together with design storm for estimation of design flood. The standard project storm (SPS)/probable maximum precipitation (PMP) depth obtained are in the form of accumulated amounts of the given durations. These are to be distributed in time according to the realistic time distribution which has been observed for the severe storms in the region. In order to obtain the most critical sequence of the rainfall depths, these are arranged in some chronological pattern of rainfall increments characterizing severe storms occurred in a catchment in producing maximum probable peak flood for the involved rainfall amount. In general, it is observed that maximum duration of severe spell of rainfall possible within severe most long duration storms is of the order of 10-15 hours. The arrangement of rainfall in one bell is aimed at arriving the maximum possible peak for the given volume of rainfall excess. Such an extreme possibility of rainfall occurrence in one spell of one, two or three days duration of design storm is not the characteristic of the nature of rainfall and results in higher design flood peaks. Broadly, it can be approximated to one or two bells per day but not over the full period of the long duration storms of high intensity. It has also been mentioned in Central Water Commission (1993) that it is reasonable to consider this aspect in design flood since it is closer to the meteorologic reality.

In this study, the Clark's model has been used for derivation of the 1-hour unit hydrograph for a catchment. Design flood estimate has been obtained using the calibrated parameters of the Clark model together with the time area diagram corresponding to a 48-hour design storm which has been temporally distributed and critically sequenced as a single bell giving due consideration to the ordinates of the unit hydrograph. Sensitivity of the design flood estimate has also been studied considering the different scenarios viz. (i) varying the design unit hydrograph characteristics (ii) considering the design storm as one bell, two bells and four bells for the 48 hour storm, and(iii) varying the design loss rate. An objective methodology has been suggested and applied for estimation of design unit hydrograph parameters after increasing the peak of the average unit hydrograph to account for the hydraulic conditions of the channel and the catchment for the extreme flooding situations, which would occur due to the occurrence of the design flood. The design flood peak is not much sensitive to the change in the design loss rate. The sensitivity runs taken considering the various design storm patterns show that the design flood estimates are very much dependent on the temporal distribution and the critical sequencing of the PMP values.

1.0 INTRODUCTION

Estimation of design flood for hydrological design of various water resources structures, particularly for medium and major water resources schemes, has been one of the most active areas of research for the hydrologists and water resources engineers. Deterministic and statistical approaches are being widely used for this purpose. Unit hydrograph method is a simple and versatile technique which is widely used for prediction and forecasting of the incoming floods. The unit hydrograph for a catchment at a particular site is generally derived using the observed discharge and the rainfall data. However, many of the small catchments are ungauged or even if they are gauged, adequate and reliable record of the required data are rarely available. In such cases, it becomes necessary to use a technique which does not require observed hydrological data for the derivation of the unit hydrograph, at least for the initial stages of planning process. In the absence of rainfall-runoff data, geomorphological characteristics of the basins can be used to synthesise unit hydrograph. The geomorphological characteristics can be easily derived from maps/toposheets having details of stream network as well as contours. Such maps/toposheets are readily available and can be considered to be very reliable. Once a unit hydrograph is estimated, the climatological data can be used for the estimation of the design flood.

The selection of design floods for water resources projects such as dams and spillways involves (i) selecting safety criteria and (ii) estimating the flood that meets these criteria. Depending upon the size of a water resources project, three types of design floods are recognised : (i) probable maximum flood (PMF), (ii) standard project flood (SPF), and (iii) frequency- based flood (FBF). The frequency based design floods are estimated using a frequency analysis of long term stream flow data at the site of interest. Another alternative of estimating the frequency based floods is to perform frequency analysis of rainfall data and couple it with a rainfall-runoff model. However, such data are not always available. Then the design flood is estimated using a rainfall-runoff model; wherein, the design storm forms the main input together with the other design parameters. PMP, SPS and FBS are considered for developing the design storms depending upon the size and type of the structures for which design flood estimates are required to be obtained. For PMF and SPF estimation the main issues are: (i) depth, duration and temporal distribution of PMP and SPS (ii) critical sequencing of the storms obtained from the temporal distribution of PMP and SPS (iii) an appropriate rainfall-runoff model to convert the design rainfall into design flood. Various methodologies are available for resolving these issues considering the type of the project, regional climatic and physiographic characteristics. In this report, design flood estimation using the Clark model of unit hydrograph derivation as well as the sensitivity analysis of the various design parameters on the design flood hydrograph has been studied.

2.0 REVIEW OF LITERATURE

Various deterministic and statistical approaches are used for design flood estimation for different types of water resources schemes, based on the design criteria, data availability etc. The nature of flow producing system-the interaction of atmosphere, land geology and geomorphology, vegetation and soils, and the activities of people is so complex that sole use of theoretical or modelling approaches can provide only generalized estimates of flood regime of a stream or region. Pilgrim and Cordery (1992) state that the choice of the method to be used is the first step in flood estimation. Unfortunately, the choice is made on a largely subjective and intuitive basis. While some subjectivity is always involved, the following considerations provide a sound basis for choice of a flood estimation method:

(i) The form and structure of available methods, the factors they consider, their theoretical basis, and their relative accuracies,

(ii) Whether a deterministic or probablistic estimate is required, and whether a particular method and its parameter values are suited to this application,

(iii) whether a method is capable of calibration with data recorded at the site or, if it is a regional method, whether it has been derived from data recorded in the region,

(iv) The type and importance of the work for which estimate is required, the effects of inaccuracy and exceedance of the estimate, and whether a peak flow or complete hydrograph is required.

(v) The time that that can be spent in estimating the flood,

(vi) The available expertize, as more complex methods generally require greater expertize in their use and interpretation, without which results may be poorer than for simpler methods.

For design at a site where observed flood data are available, a choice must be made between some form of flood frequency analysis or one of the methods based on design rainfall. Flood frequency analysis gives a direct estimate of the flood of the desired return period, but rainfall records are generally longer than flow records, are less variable over time, are available at more locations, and have greater spatial consistency in the surrounding region.

Very little quantitative guidance is available on the choice of flood estimation methods, and rather arbitrary rules are recommended for most of the regions. Bulletin 17 B of the Interagency Advisory Committee on Water Data (1982) of U.S.A. recommends that flood estimates from precipitation should be used only as an alternative method of estimating floods with exceedance probablity of 1 percent (i.e. 100 year flood) or less if the length of available streamflow record is less than 25 years. The U.S. Bureau of Reclamation (1981) gives similar recommendation. The U.K. Flood studies report (1975) recommends that a flood frequency curve should be extrapolated to a return period of 2N years only, where N years is the length of record. Beyond a return period of 4N years, a regional frequency curve is recommended upto a return period of 200 years, and even upto a return period of 500 years with lower accuracy.

Flood plain management and designs for flood control works, reservoirs, bridges and other investigations need to reflect the likelyhood or probablity of such events. Engineers and planners involved in the design of dams, spillways, river channel improvements, storm sewers, bridges, culverts etc. need information on flood magnitudes and their frequencies. Design flood estimation is a first and vital step in the design process for a large variety of water resources development works. It is ia hypothetical event that represents rare occurrence. It need not correspond with any specific event or time as it is essentially a maximum value which could be expected over a long period of time. However, for assigning a magnitude it could be expressed in terms of a probability or some return period. The design flood is generally derived based on two types of approaches viz. (i) flood frequency analysis using the observed annual maximum peak flood data and (ii) from the design storm using some rainfall runoff procedure. The design rainfall comprises of three components viz. design rainfall magnitude, its time distribution and areal pattern.

While trying to estimate the design flood a matter of conjecture is the relative merits of frequency studies of observed floods versus use of design storm. Both the methods are in reality complementary and are not competitive. It is desirable that design flood is estimated directly from observed stream flow data wherever possible. The main advantage of the flood frequency approach is that it allows a direct estimate of the flood peak discharge of a given probability. In practice, this method could not be applied widely especially to small catchments because most of the streams and rivers generally happen to be ungauged

For estimation of floods of various return periods, approaches based on frequency analysis of peak floods and application of one of the methods based on design rainfall e.g. unit hydrograph techniques for converting the excess rainfall of desired frequency to the design direct surface runoff, or watershed modelling are adopted. The methodology for estimation of design storm is discussed in N.I.H. (1984-85). Under the design storm approach, unit hydrograph technique is a simple and versatile technique, which has been widely used for estimation of design flood hydrographs for various hydraulic structures. In this section, the various design flood estimation techniques and some of the studies on data error effects on design flood estimation by unit hydrograph analysis as well as design criteria adopted for some of the hydraulic structutres have been reviewed.

2.1 Methods of Flood Estimation

The following approaches may be used for estimation of design floods depending upon data availability, importance of the study and computation-facilities.

(i) Empirical Formulae and Envelope Curves
(ii) Rational Method
(iii) Flood Frequency Analysis
(iv) Unit Hydrograph Analysis
(v) Geomorphological Instantaneous Unit Hydrograph Approach, and
(vi) Watershed Modelling

2.1.1 Empirical formulae and envelope curves

Whenever, hydrological records are inadequate for flood frequency or unit hydrograph analysis the empirical formulae are only alternative approach for estimation of floods. Empirical formulae used for estimation of the flood peak are regional formulae based on statistical correlation of the observed peaks and important catchment characteristics. However, most of the formulae neglect the flood frequency as a parameter. The empirical formulae are usually based on data obtained for the larger streams because relatively few small streams are gauged in any region. Consequently, the empirical equations are usually applied in computing peak discharges for rivers having small catchment areas where stream flow data are inadequate. Some of the commonly used empirical formulae are Dicken's, Ryve's, Graig, Lillie, Inglis, Ali Nawaz Jung, formulae etc.

In regions having similar climatological characteristics, if the available flood data are scanty, the enveloping curve technique may be used to develop a relationship between the maximum flood flow and catchment area. This method is definitely better than the empirical formulae in the sense that it does not require the selection of coefficients on the basis of judgment as required in empirical formulae. The limitation of these curves lies in the fact that they are based on past records available up to the time such curves are drawn. Such curves, should, therefore be revised from time to time as more and more data become available.

2.1.2 Rational method

The Rational method is applied for estimation of peak floods for small catchments, normally less than 50 square kilometers. This method is based on the principle that if a rainfall of uniform intensity occurs over a catchment for a duration equal to or more than the time of concentration (T_c) of the catchment; then peak flood using this method is computed by multiplying the rainfall intensity for T_c hour duration by catchment area and the runoff coefficient which depends on the land use, soil types and antecedent moisture conditions etc. of the catchment. The runoff coefficients recommended for use in Rational method are given elsewhere (Chow, 1964). This method has been widely used to design the surface drainage systems. Its popularity may be attributed to its simplicity and limited data requirement; although reasonable care is necessary for selecting the runoff coefficient in order to use the method correctly.

2.1.3 Flood frequency analysis

Flood frequency analysis for those gauging sites, where the historical peak discharges are available for sufficiently long period, may be carried out using at-site data. For at-site flood frequency analysis, generally various theoretical frequency distributions are fitted to historical flood records. The parameters of the distributions are estimated using one or more parameter estimation techniques. The best fit distribution is selected on the basis of some goodness of fit criteria. The floods of different return periods are computed using the estimated parameters of the best fit distribution. However, for the ungauged sites or sites with short record lengths, such analysis may not be able to provide consistent and reliable flood estimates. In such a situation, flood frequency analysis may be performed using regional approaches with 'regional and at-site data' or 'regional data' alone. Farquharson et al.(1992) assembled GEV (PWM) based regional flood frequency curves for a number of semi-arid and arid areas of some parts of the world.

Various issues involved in regional flood frequency analysis are testing regional homogeneity, development of frequency curves and derivation of relationship between mean annual peak flood (MAF) and the catchment characteristics. Some of the comparative studies have been conducted by Kuczera (1983), Gries and Wood (1983), Lettenmaier and potter (1985) and Singh (1989). A procedure for estimating flood magnitudes for return period of T years Q_T is robust if it yields estimates of Q_T which are good (low bias and high efficiency) even if the procedure is based on an assumption which is not true (Cunnane, 1989).

Naghavi and Yu(1995) carried out regional frequency analysis if precipitation in Lousiana. A total of 92 raingauges were used to generate 25 synthesized stations with long periods of records. Annual maximum series of rainfall durations of 1, 3, 6, 12 and 24 hour from the 25 synthesized stations were used for various statistical analyses. The mean annual precipitation, geographical locations, and synoptic generating mechanisms were used to identify the three climatological homogeneous in Lousiana. Using the L-moment ratios, the underlying regional probablity distribution was identified to be the generalized extreme value (GEV) distribution. The regional parameters of the GEV distribution were estimated by the indexed probablity weighted moments (PWM). The regional analysis was tested by Monte Carlo simulation. Relative root mean square error and relative bias were computed and compared with those resulting from at-site Monte Carlo simulation. The results show that the regional procedure can substantially reduce the relative root mean square error and relative bias in quantile prediction.

Kumar et al. (1996) developed regional flood frequency curves by fitting the probability weighted moment(PWM) based General Extreme Value(GEV) distribution to the station-year data of annual maximum peak floods of various small catchments of Mahanadi Subzone-3(d). A relationship between mean annual peak floods and catchment area is also developed for the subzone. This relationship is coupled with the developed regional flood frequency curves for derivation of the regional flood formula for the subzone.

2.1.4 Unit hydrograph analysis

Whenever adequate and reliable records on stream flow and rainfall are available for any catchment, the unit hydrograph can be derived from the rainfall-runoff data of storm events. However, most of the small catchments are generally not gauged and many water resources projects are being planned in these catchments. Therefore, it becomes necessary to have estimates of floods at the proposed sites in small ungauged catchments or the catchments with limited data. The unit hydrographs for such catchments have to be derived by using data on climatological, physiographical and other factors of these catchments. This approach for unit hydrograph derivation is popularly known as regional unit hydrograph technique. The procedure involved in developing the regional unit hydrograph requires evaluation of representative unit hydrograph parameters and pertinent physiographic characteristics for the gauged catchments in the region. Then multiple linear regression is performed considering the unit hydrograph parameters one at a time as a dependent variable and various catchment characteristics as independent variables in order to develop the regional relationships for the unit hydrograph. Further, knowing the catchment characteristics for an ungauged catchment in the region from the available toposheets the unit hydrograph for the ungauged catchment can be derived using the relationships developed between unit hydrograph parameters and catchment characteristics for the region.

Small Catchment Directorate of Central Water Commission (C.W.C.), Research Designs and Standards Organizations(R.D.S.O.), Roads Wing of Ministry of Transport and Indaia Meteorological Deaprtment(I.M.D.) have jointly carried out regional unit hydrograph studies for various Indian basins. Specific regions have been identified by dividing the whole of India into 26 hydrometeorological homogeneous sub-zones. Regional unit hydrograph relationships have been developed relating the various unit hydrograph parameters of the gauged catchments with their pertinent physiographic characteristics. Apart from these, various regional unit hydrograph relationships have been developed for some of the regions in India relating the parameters of some well known instantaneous unit hydrograph models such as Nash and Clark models etc. (e.g. Singh and Kumar, 1991).

2.1.5 Geomorphological instantaneous unit hydrograph approach

The concept of Geomorphological Instantaneous Unit Hydrograph (G.I.U.H.) has been introduced in the literature (Rodriguez et al., 1979) for the derivation of unit hydrograph for the ungauged catchments. The geomorphologic approach has many advantages over the regionalization techniques as it avoids the requirement of data and computations for the neighbouring gauged catchments in the region. A hybrid approach by integrating the Clark model and the G.I.U.H. approach for estimation of Clark model parameters using the geaomorphological characteristics and storm pattern has been developed and discussed in N.I.H. (1993-94). This approach avoids the use of extensive rainfall runoff records, which are often not available for calibration of Clark model parameters. The developed approach has been applied for simulation of historical events of Kolar-sub basin of river Narmada.

2.1.6 Watershed modelling

With the advent of high speed computers and improvements in hydrological data base, the mathematical modelling of the hydrological processes becomes an useful tool for accurate estimation of the water resources in space and time. These models can estimate the flood with reasonable accuracy provided the required input data are available for their applications. Many types of models have been developed for estimation of flood hydrographs from excess rainfall. The characteristic that distinguishes these models from unit hydrographs and other transfer function procedures is that they attempt to represent the runoff processes in more detail. The hydraulics of runoff, the routing of runoff through temporary storage within the drainage basin, and the arrangement or topology of the stream network. Computer programmes are available for most of models and are required for practical application. Models that fall into this category include the Corps of Engineers HEC-1 model, the Soil Conservation Service TR-20 model, and similar models. Calculations proceed from upstream to downstream in the basin, and the general modeling sequence is the following.

(i) Subbasin average precipitation,
(ii) Determination of precipitation excess from time-varying losses,
(iii) Generation of the direct surface runoff hydrograph from precipitation excess,
(iv) Addition of a simplified base flow to the surface runoff hydrograph,
(v) Routing of stream flow,
(vi) Reservoir routing
(vii) Combination of hydrographs

In these models, the primary interest is the flood hydrograph, so it is not necessary to calculate evapotranspiration, soil moisture changes during and between storms, or detailed base flow processes.

HEC-1 is a computer model for rainfall-runoff analysis developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. This program develops discharge hydrographs for either historical or hypothetical events for one or more locations in a basin. The basin can be subdivided into many subbasins. Uncontrolled reservoirs and diversions can also be accommodated. The available program options include: calibration of unit hydrograph and loss rate parameters, calibration of routing parameters, generation of hypothetical storm data, simulation of snowpack processes and snowmelt runoff, dam safety applications multiplan/multiflood analysis, flood damage analysis and optimization of flood control system components.

The U.S. Soil Conservation Service TR-20 computer model is a single event rainfall runoff model that is normally used with a design storm as rainfall input. The program computes runoff hydrographs, routes flows through channel reaches and reservoirs, and combines hydrographs at confluences of the watershed stream system. Runoff hydrographs are computed by using the SCS runoff equation and the SCS dimensionless unit hydrograph. Computed flows are routed through channel reaches and reservoirs.

Although, in India, there has been considerable improvement in the data network, their collection and management; even then the data base is usually not adequate for the application of the complex hydrological models for design flood estimation. Thus the conventional methods are still being used for such applications.

Fontaine (1995) states that relatively little is known about the accuracy of conceptual rainfall-runoff model simulations of extreme floods. The author carried a case study to evaluate the accuracy of runoff model simulations of the 100 year flood on the Kickapoo river in sothwest Wisconsin. The accuracy of a simple and quick analysis is compared to that of an eloborate, labour-intensive anasysis. It has been stated by the authors that the more eloborate modelling approach produces more accurate results, although significant errors for the peak discharge and runoff volume are observed in both the approaches. The potential sources of uncertainty in the results are evaluated. Error in the precipitation data used for calibrating the model appears to be the primary source of uncertainty.

The approach of flood estimation using design rainfall has some advantages over the frequency analysis of observed floods. The different parameters affecting the flood runoff could be considered in a more realistic and explicit way and the catchment characteristics of different sub basins contributing to the flood flow in the main river could be determined more thoroughly and added .appropriately. The necessary parameters (unit hydrograph and routing) could be estimated even from a short length of record and the parameters thus derived could be extended to the other ungauged subbasins. The design storm approach also allows for maintenance of consistency in a given geographical area.

Rainfall frequency studies are more advantageous than flood frequency studies because longer records of precipitation are generally available at a larger number of rain gauges more so in case of daily rainfall. Extreme rainfall values are more easily defined from physical consideration.

A number of limitations were noticed in prcatice in spite of the wide spread and continued use of the design storm approach by design engineers. These relate to almost all aspects of the design storm strarting from the approach and risk criteria to the time distribution and others. While some related to inadequacy, others were regarding the inconsistency and inapropriateness of the method.

Eagleson (1978) represented point precipitation by Poisson arrivals of rectangular intensity pulse that have random depth and duration. By assuming the storm depths to be independent and identically gamma distributed, the cumulative distributin function for normalized annual precipitation is derived in terms of two parameters of storm sequence, the mean number of storms per year and the order of the gamma distribution. In comparison with long-term observations in subhumid and an arid climate it is demonstrated that when working with only 5 years of storm observations this method tends to improve the estimate of the variance of the distribution of the normalized values over that obtained by conventional hydrologic methods which utilize only the observed totals.

The major criticism of the design rainfall approach is that in the process of deriving design flood from design storm a series of steps are involved which would introduce some error and, therefore, may not provide the expected results. Thus, a design rainfall of a given frequency might not produce flood of the desired frequency. Besides, some of the limitations of fitting a frequency distribution to the flood data apply equally well to the extreme rainfall values too.

2.2 Hydrologic Design Criteria

Design criteria refers to standards and practices laid down for judging whether a project has been properly designed to deliver the anticipated outputs. If different criteria are adopted, different engineering decisions may result. The need for criteria and standardisation arises whenever choices between alternatives are to be made in a systematic and scientific manner. Further, the areas of activity which address the problems involved with complexities of nature, have to necessarily depend on experience and judgment, and thus need adequate standards and criteria to guide the practicing engineers and decision makers. The design criteria for some of the hydraulic/water resources structures as mentioned by Sharma(1991).

2.3 Unit Hydrograph Techniques and Data Error Effects in Unit Hydrograph Derivation

Unit hydrograph method is a simple and versatile technique which is widely used for prediction and forecasting of flood peaks. The unit hydrograph for the catchment can be derived when adequate and reliable records of rainfall and runoff data are available.

2.3.1 Unit hydrograph (UH)

The unit hydrograph (also known as the unit graph) may be defined as the direct runoff (outflow) hydrograph resulting from one unit of effective rainfall which is uniformly distributed over the basin at a uniform rate during a specified period of time known as unit time or the unit duration.

The unit hydrograph theory assumes the principles of the time invariance i.e., there are no changes either in the catchment characteristics or the storm patterns or its movements. It also assumes the principles of linearity, superimposition or proportionality.

As indicated in the definition of the unit hydrograph, the rainfall must be uniformly distributed both in space and time. In order to meet the conditions of uniform rainfall excess in time, it is desirable that the storms are of short duration & intense and that the resulting hydrograph should be single peaked and of short time base. Similarly for the uniformity of rainfall excess in space, the catchment should be small (generally less than 5,000 sq.kms.). In case of large catchment, it should be sub-divided into smaller catchments and unit hydrographs should be derived for each of the sub-catchments.

2.3.2 Instantaneous unit hydrograph (IUH)

If the unit duration of a unit hydrograph is shortened but the depth is kept constant, the peak flow will increase and the base length will decrease. If the unit duration tends to zero and depth remain the same, the intensity of the rain becomes infinite. This corresponds with the instantaneous application of a sheet of water over entire catchment area. This water drains off by gravity and the resulting unit hydrograph is the instantaneous unit hydrograph (IUH).

The instantaneous unit hydrograph is a purely theoretical concept and represents the unit hydrograph obtained when the unit excess rainfall volume occurs instantaneously. IUH represents a more characteristics curve of a catchment area than a T-hours unit hydrograph as it is not affected by the duration. It is a useful tool in regional unit hydrograph studies.

There are various methods for the determination of an IUH from the given effective rainfall hyetograph and direct runoff hydrograph. But the most common are the models suggested by Clark (1945) and Nash (1957). Clark (1945) suggested that the instantaneous unit hydrograph (IUH) can be derived by routing the unit inflow in the form of time area concentration curve, which is constructed from isochronal map, through a single linear reservoir. Nash proposed a conceptual model by considering a drainage basin as `n' identical linear reservoirs in series. By routing a unit inflow through the reservoirs a mathematical equation for IUH can be derived. The relationship between the IUH (U(0,t)) and the T-hours unit hydrograph (U (T,t)) both with the same unit depth is available in literature (e.g. NIH, 1988-89).

As discussed earlier, the observed rainfall and runoff data are required for the derivation of the unit hydrograph or the instantaneous unit hydrograph. However, the observed data are not available at all the points along the river reach. This is more so in case of smaller river systems or the tributaries. Even in case of gauged rivers, the observed data may not be available at the desired interval or may not be representative of the conditions which are essential for the derivation of the unit hydrograph or the instantaneous unit hydrograph. If runoff data are inadequate or not available, it becomes necessary to adopt techniques in which geomorphological characteristics of the basin, the hydrometeorological features of the region and other factors are used to derive the unit hydrographs. Such unit hydrographs are termed as regional unit hydrograph or synthetic unit hydrograph or geomorphological unit hydrograph or geomorphological instantaneous unit hydrograph.The geomorphological characteristics can be easily derived from maps/toposheets having details of stream network as well as contours. Such maps/toposheets are readily available and are very reliable.

The ordinates of the unit hydrograph are derived with help of a number of storm events resulting in floods. Generally, wide variations are observed in the values of the ordinates from event to event. However, the normal practice is to estimate the average values of the ordinates of the unit hydrograph for use in design flood estimation. The ordinates of the unit hydrograph are also sometimes found through solution of a set of linear algebraic equations involving matrix operations. However, the accuracy in the estimation of the ordinate value considerably depends on the separation of effective rainfall from the total rainfall and that of direct runoff hydrograph from the observed flood hydrograph.

While solving the field problems, it has been observed on many occasions that the shape of unit hydrographs are somewhat unconventional and do not match with the general concept of a smooth unit hydrograph shape. This along with the considerable number of parameters to define the shape of the hydrograph causes practical problems in relating the unit hydrograph characteristics with catchment characteristics.

As a result, efforts have been made to define the unit hydrograph with least possible number of parameters. Earlier approaches for derivation of synthetic unit hydrographs with the help of geomorphological characteristics involved considerable personal judgment. In effort to represent the unit hydrograph through lesser number of parameters, the concept of a triangular unit hydrograph or double-triangular unit hydrograph have also been used. But they have their own limitations.

Thus, the IUH is a purely theoretical concept and represents the unit hydrograph obtained, when the unit excess rainfall volume occurs instantaneously.

As the rainfall duration term is eliminated, the IUH indicates storage characteristics of a catchment and it is unique for a catchment. It is very useful tool for regional unit hydrograph studies.

Many conceptual models for derivation of IUH such as Nash Model(1957), Clark model(1945), Zoch model(1934,1936) and Dooge model(1973) are in vogue. Apart from the above, the hydrologists namely, Laurenson(1964), and Diskin(1972) etc. have also proposed models for derivation of IUH.

For derivation of unit hydrograph or IUH by any of the above discussed techniques, rainfall runoff data at short intervals are needed for a few representative storm events. Runoff records are usually limited for most of the basins to suffice derivation of unit hydrograph and therefore, there has been very urgent need for defining hydrological response of a basin in terms of geomorphological and climatological characteristics of the basin.

2.3.3 Data Error Effects in Unit Hydrograph Derivation

Laurenson and O'Donnell (1969) studied of the sensitivity of various methods of unit hydrograph derivation to errors in the data, five factors were originally defined for the study: (i) Method of derivation; (ii) Type of error; (iii) Magnitude of error; (iv) Shape of rainfall-excess hyetograph; and (v) Shape of true unit hydrograph.

To simplify the investigation, factor 3 has not been studied in detail quantitatively, although derivations have been made both with and without data error. The general effect of increasing data errors is, of course, to increase the unit hydrograph error. The authors mention that after analysis, further simplification of the interpretation of results became possible, as factor 5 proved to have little effect on the error. Even with the elimination of two of the five factors, the drawing of general conclusions has proved difficult because the remaining three factors interact so strongly with one another.

The combination of the Laguerre method of derivation with the particular form of late peaked hyetograph used resulted in such large errors that the Laguerre method could not be recommended for general use in its present form, even though its performance under most conditions was as good as or better than that of the other three methods.

If the Laguerre method is excluded, the shape of the hyetograph does not have a major effect on the unit hydrograph error. There still exists some noticeable and inconsistent interaction between type of error and shape of hyetograph, but this is not of major proportions, and the average effect of hyetograph shape is small. Therefore, attention will be concentrated on the types of error and the remaining three methods of derivation.

The error due to lack of synchronization between the clocks of different raingauges on a catchment could be a serious source of unit hydrograph error. Also, the lack of synchronization between the rainfall and runoff records, and the assumption of a uniform loss rate instead of varying losses throughout the storm both could be potential sources of significant error. However, the former source results only in a time lag error in the unit hydrograph with the harmonic, Larguerre, and lease squares methods. Reasonable errors in the estimation of total rainfall, in the discharge rating curve, and in base flow separation result in surprisingly low unit hydrograph errors.

Concerning the harmonic, least squares, and gamma methods of derivation, each method has its own strengths and weaknesses, and the authors state that none can be recommended in preference to the other without further research.

2.4 Design Storm Parameters for Design Flood Estimation

The estimation of design storm value and its temporal as well as spatial distribution is the most critical element in the analysis. Design storm studies for different projects have been carried out by various investigators with widely varying results due to subjectivity involved in the procedures. Large variations in the results obtained by different/agencies point to the need for effective standard procedures in this field (CWC, 1993). To meet this objective, a workshop was organised in 1993 at CWC, Delhi to discuss the various issues involved and recommended guidelines for estimation of design storm parameters for design flood estimation so as to minimise variance in the results of individual analysts. In the approach paper prepared by the Hydrology Organisation of CWC, issues to be debated for standardising design storm analysis were identified and existing practices discussed and attempts were made to evolve appropriate procedure for estimation of design floods in Indian tropical region (CWC, 1993).

3.0 STATEMENT OF THE PROBLEM

Estimation of design flood is one of the important components of planning and design of any water resources project. Deterministic and statistical approaches are being widely used for this purpose. The selection of design floods for water resources projects such as dams and spillways involves (i) selecting safety criteria and (ii) estimating the flood that meets these criteria. For the design of many types of hydraulic structures different design criteria have been evolved, as described in Section 2. Depending upon the size of a water resources project, three types of design floods are recognised:

(i) probable maximum flood (PMF),

(ii) standard project flood (SPF), and

(iii) frequency-based flood (FBF).

Application of one of the methods based on design rainfall e.g. unit hydrograph techniques for converting the excess rainfall of desired frequency to the design direct surface runoff, or watershed modelling are adopted. In the second approach it is presumed that the frequency of the flood peak is same as the frequency of the design rainfall.

The frequency based design floods are estimated using a frequency analysis of long term stream flow data at the site of interest. Another alternative of estimating the frequency based floods is to perform frequency analysis of rainfall data and couple it with a rainfall-runoff model. However, such data are not always available. Then the design flood is estimated using a rainfall-runoff model; wherein, the design storm forms the main input together with the other design parameters. PMP, SPS and FBS are considered for developing the design storms depending upon the size and type of the structures for which design flood estimates are required to be obtained. For PMF and SPF estimation the main issues are : (i) depth, duration and temporal distribution of PMP and SPS (ii) critical sequencing of the storms obtained from the temporal distribution of PMP and SPS (iii) an appropriate rainfall-runoff model to convert the design rainfall into design flood. Various methodologies are available for resolving these issues considering the type of the project, regional climatic and physiographic characteristics.

For estimation of floods of various return periods, two types of approaches viz. (i) frequency analysis of peak floods and (ii) application of one of the methods based on design rainfall e.g. unit hydrogrraph techniques for converting the excess rainfall of desired frequency to the design direct surface runoff, or watershed modelling are adopted. In the second approach it is presumed that the frequency of the flood peak is same as the frequency of the design rainfall.

The objectives of this study are:

(a) Derivation of 1-hour unit hydrograph for the catchment using the Clark model,

(b) Computation of hourly excess rainfall for the design storm of 48 hours using the 2 days PMP value.

(c) Estimation of design flood by convoluting the 48 hour design storm with the unit hydrograph.

(d) Examination of the effects of sensitivity of the design parameters on design flood hydrograph.

4.0 DATA AVAILABILITY FOR THE STUDY

The design flood estimation and its sensitivity to design flood parameters studies have been carried out for the catchment of the river Tons upto Kishau. The catchment area upto Kishau is 4755 square kilometers. The time area diagram of the catchment has been derived using the toposheets of the study area. The parameters of the Clark model viz. time of concentration (Tc) and storage coefficient (R) have been derived using the observed rainfall-runoff data of three storms. The 2 days PMP values as well as their time distribution are also available for the catchment.

5.0 METHODOLOGY

The direct surface runoff and effective rainfall are assumed to be linearly related with unit hydrograph ordinates. Thus, the ordinates of the unit hydrograph are found through a solution of the set of linear algebraic equations which involves the matrix operations. Even small errors in effective rainfall and direct surface runoff make the general linear solution numerically ill conditioned resulting in unrealistic shape of the unit hydrograph with fluctuating ordinates. The above approach becomes impracticable while relating the unit hydrograph parameters with catchment characteristics. Therefore, it becomes necessary to postulate a general linear hydrologic model (conceptual model) which can be represented by a limited number of parameters. This implies choosing a fixed form or equation for the instantaneous unit hydrograph (IUH) keeping in view the various constraints imposed by the nature of the hydrologic system. However, the number of parameters which are used to define the fixed form of IUH is limited by the number of independent significant relations which can be established with the catchment characteristics. If there are more parameters than this, the extra parameters can not be evaluated from the catchment characteristics for ungauged catchments and must either be given fixed values or related to the other parameters. In either case, the effective number of degrees of freedom is reduced to the number of independent relationship established.

Thus, the following requirements must be fulfilled while choosing a general IUH equation:

(i) The IUH ordinates are all positive

(ii) The shape of the IUH is preserved

(iii) The errors in input data should not be amplified during the IUH derivation.

(iv) The number of parameters of the chosen form is limited to the number of independent relation ships established between the responses and the catchment characteristics.

(v) The form should reflect, as far as possible, the physical elationship between the input and output.

5.1 Clark Model Concept

Clark (1945) suggested that the instantaneous unit hydrograph (IUH) can be derived by routing the unit inflow in the form of time area concentration curve, which is constructed from isochronal map, through a single linear reservoir. The isochronal map a watershed is shown in Fig. 1. Fig. 2 shows the time area diagram prepared from

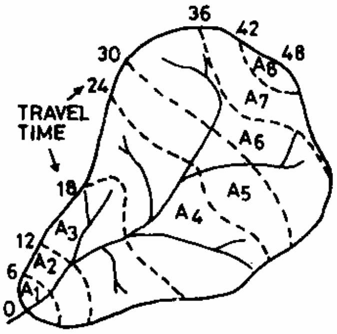

FIG.1: ISOCHRONAL MAP OF A WATERSHED

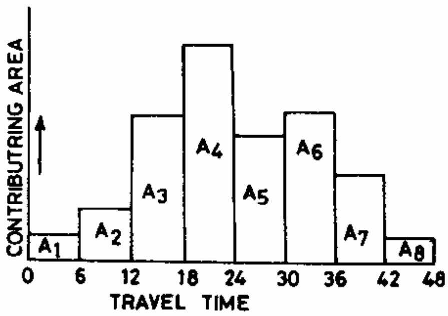

FIG. 2: TIME AREA CURVE CONSTRUCTED FROM ISOCHRONAL MAP

the isochronal map. The detailed concept of Clark Model for IUH derivation (Fig. 3) is available in literature (e.g. NIH, 1988-89). The procedure foı derivation of unit hydrograph using Clark Model, is given below.

5.2 Estimation of Effective Rainfall and Direct Surface Runoff

The effective rainfall and direct surface runoff may be estimated using the standard procedures (e.g. NIH, 1988-89).

5.3 Estimation of Parameters for Clark Model

The Clark model uses two parameters viz. time of concentration (T_c), storage coefficient (R) and a time area relation to define the IUH as described below.

(i) Time of concentration (T_c): This represents the travel time of a water particle from the most upstream point in the basin to the out flow location. An initial estimate of this lag time is the time from the end of effective rainfall (plus snowmelt if any) over the basin to the inflexion point on the recession limb of the surface runoff hydrograph as shown in Fig. 4. This time of concentration is used in developing the time area relation.

(ii) Storage coefficient (R): This is an attenuation constant which has the dimension of time. This parameter is used to account for the effect which storage in the river channel has on the hydrograph. This parameter can be estimated by dividing the flow at the point of inflexion of the surface runoff hydrograph by the rate of change of discharge (slope) at the same time (Fig. 4). Another technique for estimating R is to compute the volume remaining under the recession limb of the surface runoff hydrograph following the point of inflexion and divide by the flow at the point of inflexion. In either case, R should be an average value determined by using several hydrographs.

(iii) Time-area diagram: The other necessary item to compute an IUH using Clark model is the time-area relation. When Tc has been determined the basin is divided into incremental runoff producing areas that have equal incremental travel times to the outflow location. The computational steps involved, in constructing time-area curve, are:

- Measure the distance from the most upstream point in the basin to the outflow location along the principal water course.

- Estimate the time of travel as the ratio of $L/\sqrt{S}$ along the water course where L is the length of a segment and S is the slope of the segment.

- Lay out the isochrones representing equal times of travel to the outflow

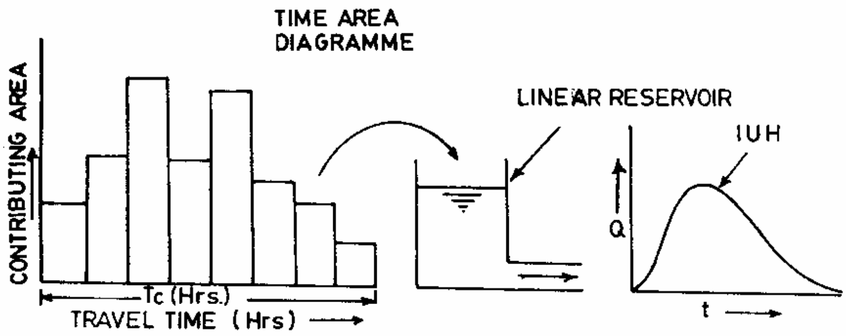

FIG. 3 CLARK MODEL

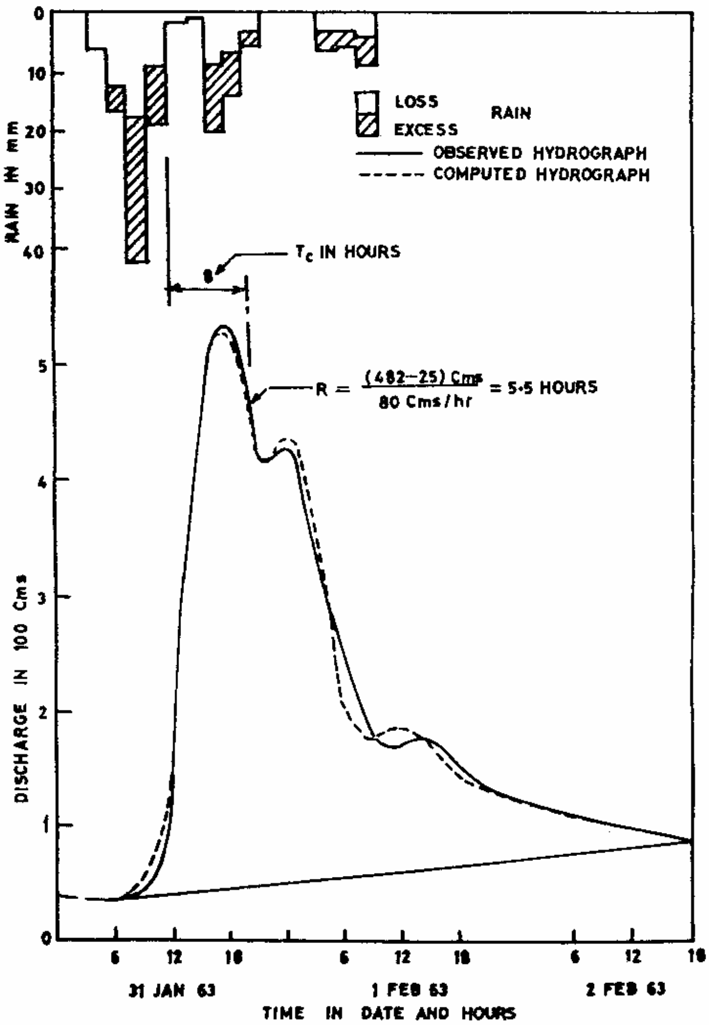

FIG.4: DETERMINATION OF CLARK COEFFICIENTS FROM A TYPICAL HYDROGRAPH

location after establishing the location of lines using the ratio $L/\sqrt{S}$ of different segments.

- Measure the area between the isochrones and tabulate them in upstream segments versus the corresponding incremental travel time for the each incremental area.

5.4 Computation of IUH

The resulting shape of time area diagram is routed through a linear reservoir to simulate the storage effects of the basin and the resulting outflow represents the IUH. Before going for linear reservoir routing the runoff from the contributing areas (between the isochrones) which would be translated to the outflow location, should be expressed in proper unit. The conversion to proper unit of discharge can be made through the following relationship.

$$I_i = K\, a_i \,/\, \Delta t \qquad (1)$$

where,

I_i = Ordinate in proper units of discharge of the translation hydrograph at the end of period.

a_i = Ordinates in units of area-depth of excess of the translation hydrograph at the end of period i.

K = Conversion factor to convert a_i to I_i

Δt = Time period of computation interval in hours.

Then the linear reservoir routing is accomplished using the general equation as described below.

Clark (1945) was the first to show that the routing of a flood wave in a reach could be successfully accomplished by translating the wave into a time equal to the travel of the reach and then routing in through an amount of reservoir, storage equivalent to that in the reach. From this, Clark conceived at this approach could be used to derive the IUH for a basin by routing the time area curve of a basin through a given amount of linear reservoir storage hypothetically, this simply infers placing a reservoir at the outlet of a stream that has storage characteristics such that S = RO. Hence the countinuity equation for linear storage is:

$$\frac{I_1+I_2}{2} - \frac{Q_1+Q_2}{2} = \frac{R(Q_2-Q_1)}{\Delta t} \qquad (2)$$

$$O_2 = \frac{0.5\Delta t}{R+0.5\Delta t} I_2 + \frac{0.5\Delta t}{R+0.5\Delta t} I_1 + \frac{R-0.5\Delta t}{R+0.5\Delta t} \times O_1 \qquad (3)$$

For practical purposes $I_1 = I_2$

$$O_1 = \frac{\Delta t}{R+0.5\Delta t} I_1 + \frac{R-0.5\Delta t}{R+0.5\Delta t} O_1 \qquad (3)$$

$$O_2 = C\, I_1 + (1 - C)\, O_1 \qquad (4)$$

$$\text{where, } C = \frac{\Delta t}{R+0.5\Delta t} \qquad (5)$$

or, $$U_i = CI_i + (1\text{-}C)\, U_{i-1} \qquad (6)$$

where,

C and (1-C) are routing coefficients,

U_i is the IUH at the period i,

U_{i-1} is the IUH at the period i-1.

5.5 Derivation of Unit Hydrograph

The hydrograph that results from routing these flows from the incremental areas is the IUH. This IUH can be converted to a unit hydrograph of unit rainfall duration Δt by simply averaging the two ordinates of IUH spaced at an interval Δt apart as follows:

$$UH_i = 0.5\ (U_i + U_{i-1}) \qquad (7)$$

The IUH can be converted to a unit hydrograph of some unit rainfall duration other than Δt, provided that it is an exact multiple of Δt, by the following equation:

$$UH_i = 1/n\ [0.5\ U_{i-n} + U_{1-n+1} + \ldots\ldots + U_{i-1} + \ldots\ldots 0.5\ U_i]$$
(8)

where,

UH_i= Ordinate at time i of unit hydrograph of duration D-hour and computational interval Δt.

$$n = \frac{D}{\Delta t}$$

where,

D = Unit hydrograph duration (hours),
Δt = Computational interval (hours) and
U_i = Ordinate at time i of IUH

The computational steps involved in the derivation oh unit hydrograph by the Clark Model are mentioned below.

(i) Make first estimate of Clark Model parameters, T_c and R from the excess rainfall hyetograph and direct surface runoff hydrograph computed using the procedure described above.

(ii) Construct the time-area curve, taking the T_c value obtained from step (i), using the procedure described in 5.1.3.

(iii) Measure the area between each pair of isochrones by planimeter.

(iv) Plot the curve of time versus cumulative area. The abscissa is expressed in percent of $T_{c.}$ Tabulate increments between points that are at computational interval Δt apart.

(v) Convert the units of inflow using the Eq. (1).

(vi) Route the inflow obtained from step (v) using the Eq. (5) and (6) to get IUH ordinates.

(vii) Compute the unit hydrograph of the excess rainfall duration using Eq. (7) and (8).

6.0 ANALYSIS AND RESULTS

The results of the analysis carried out for design flood estimation and sensitivity analysis of the design flood estimates obtained using the Clark model to the various design parameters are discussed below.

6.1 Design Flood Estimation

For estimation of design flood, 1-hour unit hydrograph has been derived using the Clark model, as described in Section 5. The dimensionless plot of the time area diagram for the catchment is shown in Fig. 5. The Clark model parameters have been estimated as Tc = 9 and R = 12. The unit hydrograph which is derived based on the principle of linearity, has been applied to convert the excess rainfall hyetograph into direct surface runoff hydrograph. In order to estimate design flood using the unit hydrograph principle, various design parameters viz. design unit hydrograph, design storm, design loss rate and design base flow are required. In this study, 2-days probable maximum precipitation (PMP) has been considered for the catchment for developing the design storm patterns. For this purpose, a 48-hour time distribution has been adopted to convert the 2-days PMP values into 48 hour rainfall values at hourly interval. The critical sequencing of the 48-hours PMP has been carried out by arranging the hourly values corresponding to the magnitude of the unit hydrograph and reversing the sequence in order to obtain the single bell of the design storm at hourly interval. The design loss rate of 1-mm/hour has been adopted for computation of design excess rainfall hyetograph from the design storm values. Fig. 6 shows the critically sequenced design excess rainfall hyetograph as a single bell. The hourly values of the design excess rainfall hyetograph have been convoluted with the 1-hour unit hydrograph for estimation of the design direct surface runoff hydrograph. An estimate of the design base flow of 340 cumec has been added with the ordinates of design direct surface runoff hydrograph for estimation of design flood hydrograph. The peak of the design flood hydrograph (Qp) is estimated as 24,359 cumec and the time to peak (Tp) is computed as 49 hours. Fig. 7 shows the flood hydrograph by considering the 48 hour storm as single bell. This has been referred as the reference run in the following sections of the report. Sensitivity of peak characteristics of the unit hydrograph has been studied as described here under.

For the purpose of sensitivity analysis, the following scenarios have been formulated to study the sensitivity of the peak characteristics of the design flood hydrograph.

(i) Variation in the parameters of the Clark's model,

(ii) Consideration of design storms as one bell, two bells and four bells (Case-1 and Case-2)

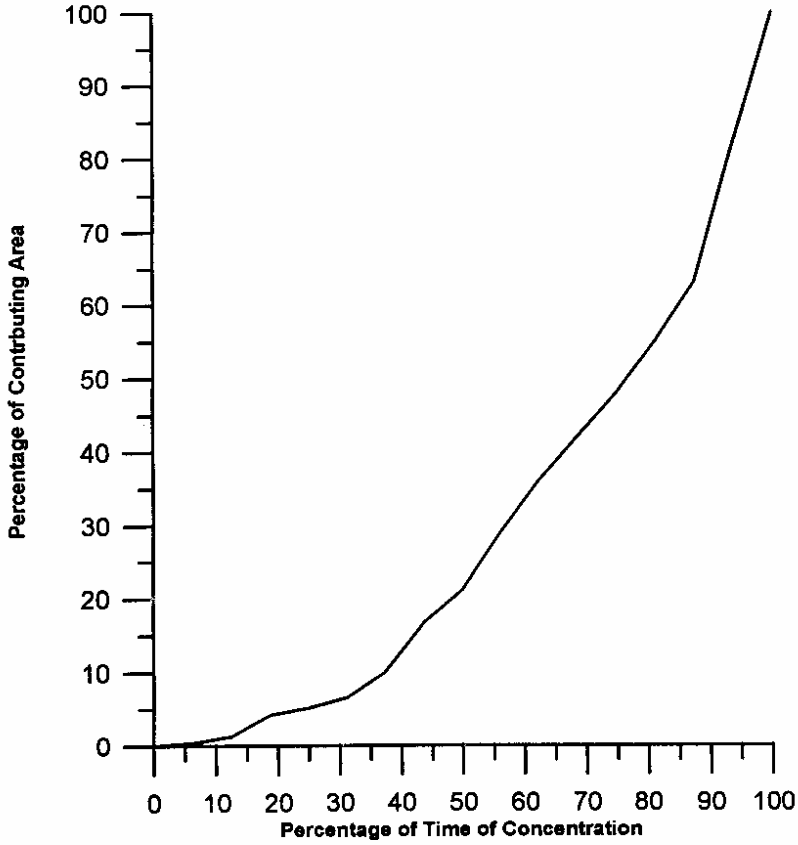

Fig.5 The Time Area Diagram for the Catchment

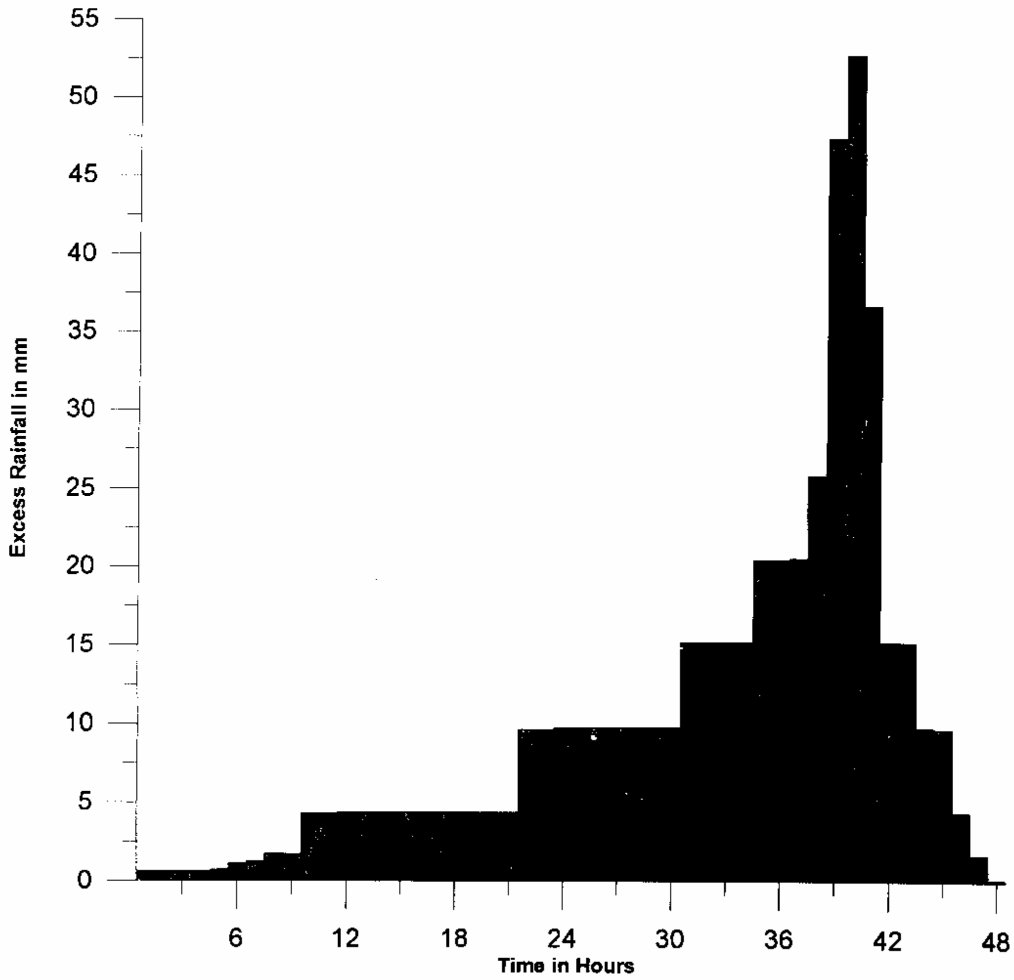

Fig. 6 Critically sequenced design excess rainfall hyetograph as single bell (used for reference run)

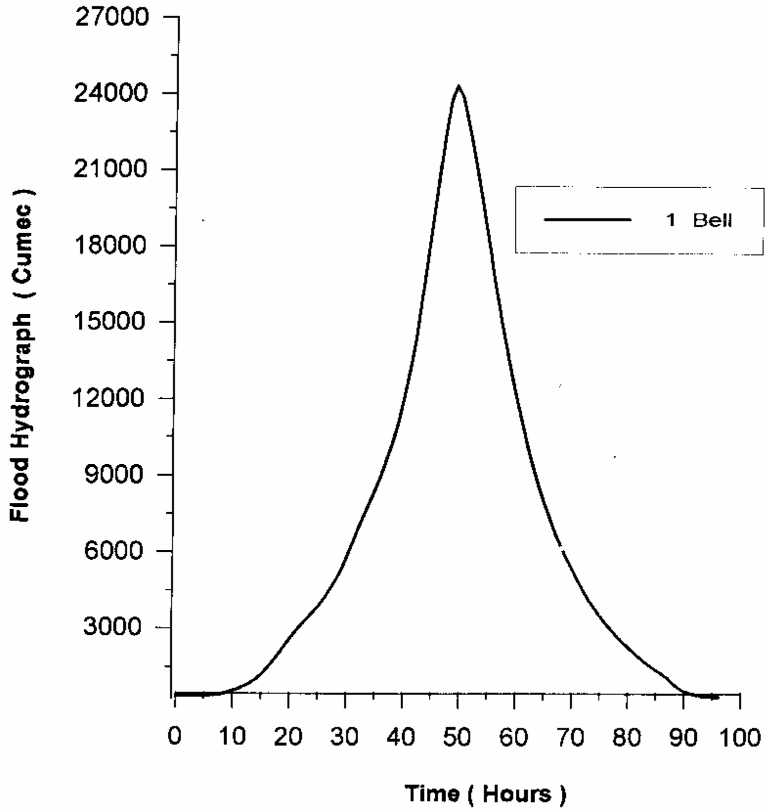

Fig. 7 Flood hydrograph by considering 48 hour storm as one bell

(iii) Variation in the design loss rate.

(iv) Sensitivity of temporal distribution pattern of design storm

6.2 Sensitivity of the Parameters of the Clark Model

The parameter Tc of the Clark model has been varied from 4 hours to 16 hours and the parameter R was varied from 6 hours to 18 hours. The peak (Qp) and time to peak (Tp) of the 1-hour unit hydrograph have been evaluated corresponding to different sets of parameter values and the same are given in Table-1 and Table-2 respectively. Fig. 8 shows the variation of peak of unit hydrograph with time of concentration (Tc) for different values of storage coefficients. It may be observed from the figure that peak of the unit hydrograph decreases with time of concentration. Also, for a given value of time of concentration, the peak of unit hydrograph decreases with increase in values of storage coefficient (R). Fig. 9 shows the variation of time to peak (Tp) of unit hydrograph with time of concentration (Tc) for different values of storage coefficients. It is seen from Fig. 9 that time to peak (Tp) increases with increase in time of concentration. However, there is no change in the time to peak values (Table 2) for different values of storage coefficient corresponding to a time of concentration. Variation of Qp*Tp with time of concentration (Tc) for different values of storage coefficients (R) is shown in Fig. 10.

When the design unit hydrograph is derived from the average unit hydrograph normally the shape of the unit hydrograph is modified after increasing the peak of the average unit hydrograph by 25% to 50% to account for the hydraulic conditions of the catchment and the channel for the severe flooding conditions. Fig. 8 provides an objective methodology for estimating the unit hydrograph parameters Tc and R corresponding to the increased peak of the unit hydrograph. To study the impact of increase in peak of the unit hydrograph on the design flood peak, different unit hydrographs have been applied with the design storm of one bell (considered to be reference design storm). Fig. 11 shows variation of percentage increase in the peak of the design flood hydrograph with percentage increase in the peak of the unit hydrograph. It may be observed from the Fig. 11 that for 50% increase in the peak of unit hydrograph there is an increase of about 25% in the design flood peak.

Table 1 : Variation of peak of 1-hour unit hydrograph (Qp) with Clark model parameters (Tc and R)

S.No.	Tc (Hours)	Peak of 1-hour unit hydrograph (Cumec)						
		R = 6	R = 8	R =10	R =12	R =14	R =16	R =18
1	4	163.2	130.6	108.8	93.1	81.5	72.3	65.0
2	5	156.6	126.4	105.9	91.0	79.8	71.1	64.0
3	6	150.9	122.9	103.5	89.3	78.5	70.7	63.1
4	7	145.2	119.3	100.8	87.3	77.0	68.8	62.2
5	8	140.5	116.0	98.7	85.7	75.7	67.8	61.4
6	9	134.7	112.2	96.0	83.7	74.2	66.6	60.4
7	10	130.0	109.0	93.7	82.0	72.8	65.5	59.5
8	11	125.0	105.9	91.4	80.2	71.5	64.4	58.6
9	12	121.3	103.0	89.3	78.7	70.3	63.4	57.8
10	13	117.3	100.3	87.3	77.1	69.0	62.4	57.0
11	14	113.8	97.7	85.3	75.6	67.9	61.5	56.2
12	15	110.4	95.3	83.5	74.2	66.7	60.6	55.4
13	16	107.4	93.0	81.8	72.9	65.7	59.7	54.7

Table 2 : Variation of time to peak (Tp) of 1-hour unit hydrograph with Clark model parameters (Tc and R)

S.No.	Tc (Hours)	Time to peak (Tp) of 1-hour unit hydrograph (Hours)						
		R = 6	R = 8	T =10	R =12	R =14	R =16	R =18
1	4	5	5	5	5	5	5	5
2	5	6	6	6	6	6	6	6
3	6	7	7	7	7	7	7	7
4	7	8	8	8	8	8	8	8
5	8	9	9	9	9	9	9	9
6	9	10	10	10	10	10	10	10
7	10	11	11	11	11	11	11	11
8	11	12	12	12	12	12	12	12
9	12	13	13	13	13	13	13	13
10	13	14	14	14	14	14	14	14
11	14	15	15	15	15	15	15	15
12	15	16	16	16	16	16	16	16
13	16	17	17	17	17	17	17	17

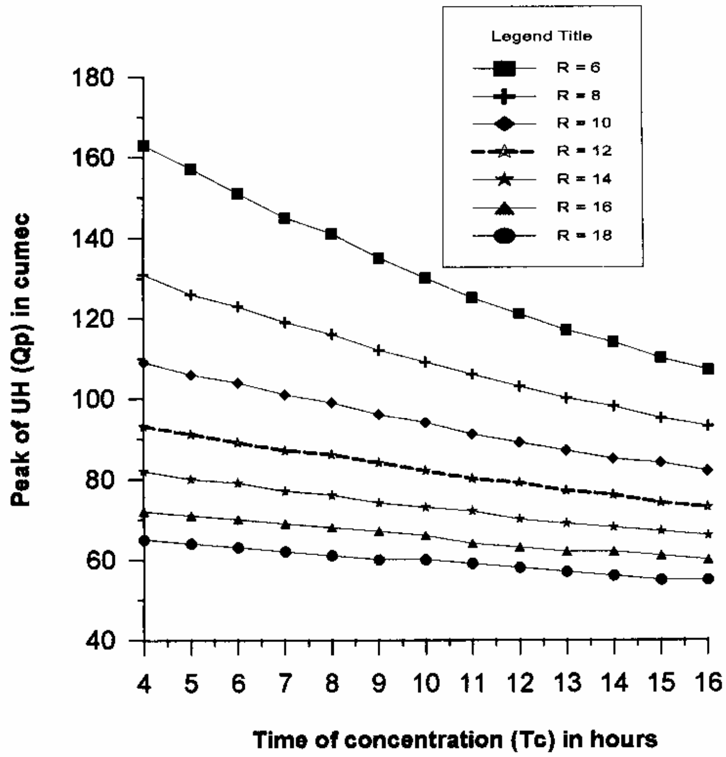

Fig.8 Variation of peak of UH (Qp) with time of concentration (Tc) for different values of storage coefficient (R)

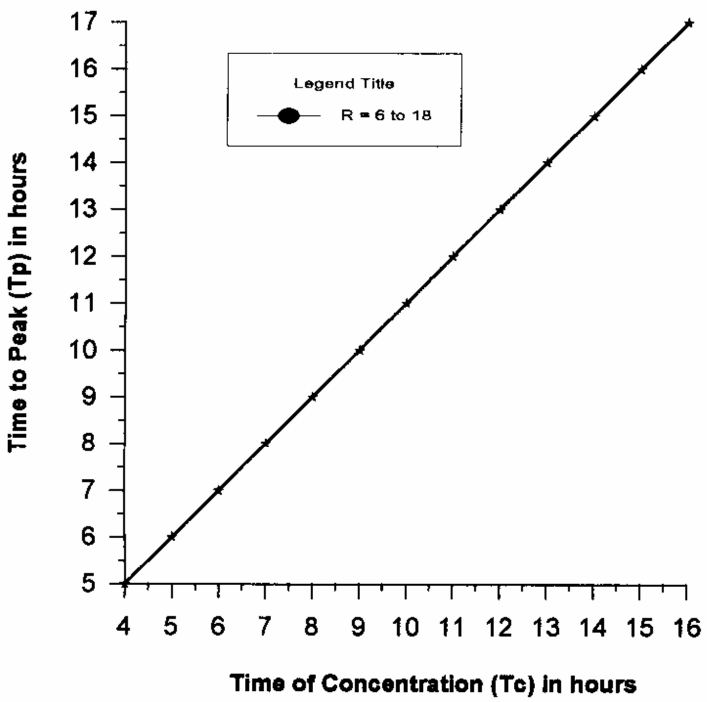

Fig. 9 Variation of time to peak (Tp) with time of concentration (Tc) for different values of storage coefficient (R)

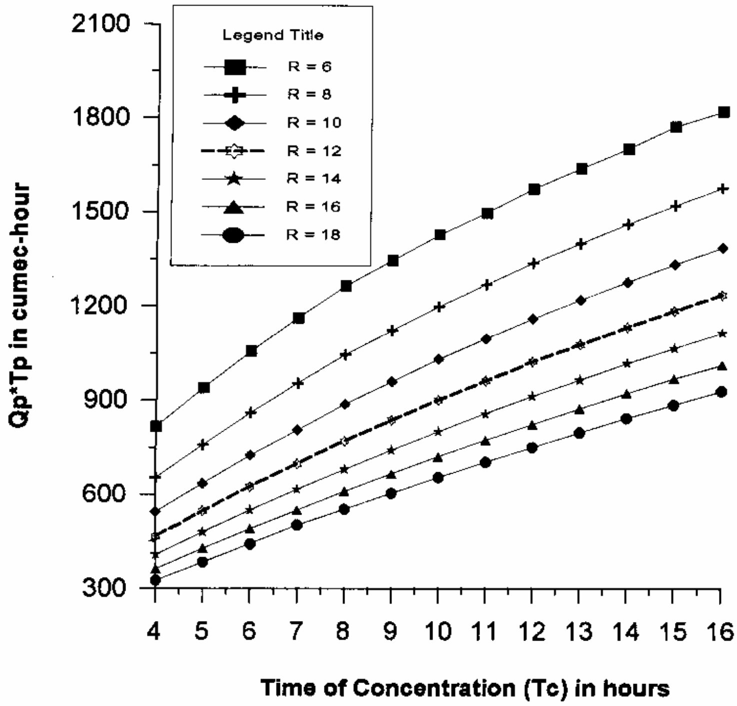

Fig.10 Variation of Qp * Tp with time of concentration (Tc) for different values of storage coefficient (R)

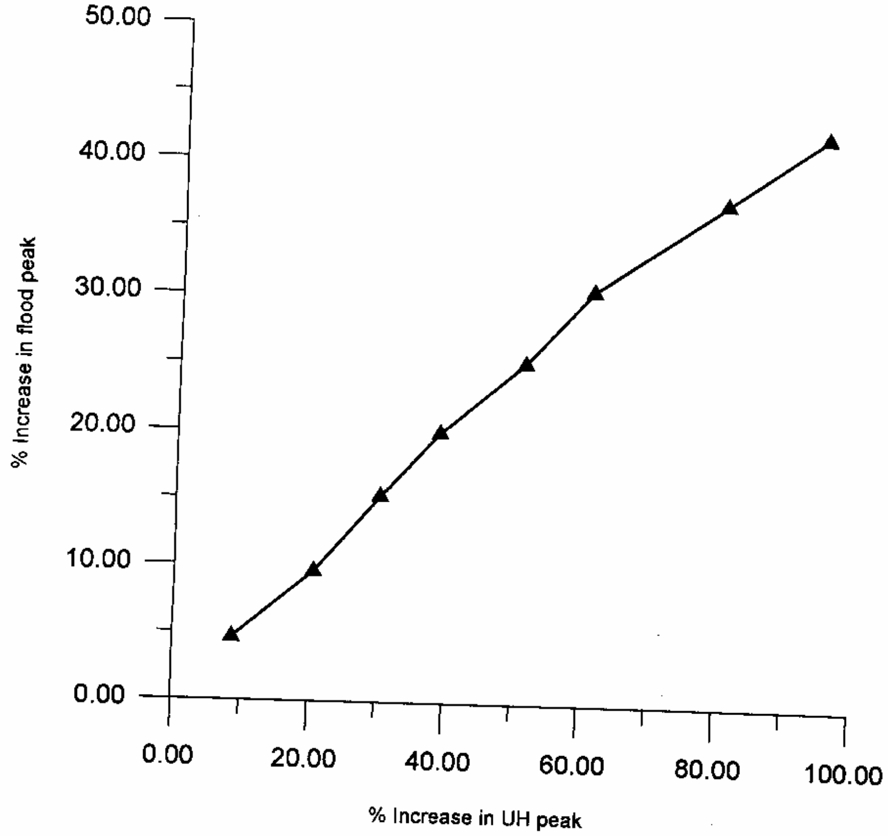

Fig.11 Variation of percentage increase in peak of flood hydrograph with percentage increase in peak of UH

6.3 Sensitivity of Design Storm

The magnitude of design flood is most significantly dependent on the design storm. In this study, the sensitivity of design storm on design flood estimate has been conducted by considering the following four procedures of arranging the 48 hour design storm:

(i) Single bell design storm is critically sequenced by placing the maximum ordinate of the excess rainfall against the maximum ordinate of the unit hydrograph, and second maximum against the second maximum and so on; and then reversing the sequence of the design storm. (Reference run as discussed in Section 6.1 and shown in Fig. 6)

(ii) Considering the design storm in two bells (as shown in Fig. 12),

(iii) Considering the design storm in four bells as shown in Fig. 13 (Case 1), and

(iv) Considering the design storm in four bells as shown in Fig. 14 (Case 2).

Fig. 15 shows the comparison of design flood hydrographs corresponding to the design storms of one bell, two bells, and four bells. The design flood hydrograph estimated using the design storm considering one bell shows the maximum peak and time to peak as compared to the design flood hydrograph resulting due to the design storm of the two and four bells. For the computation of the design storm arranged in one bell, the practice recommended by CWC (1973) for the critical sequencing has been followed. However, for computing the design storms of two and four bells the recommendation made by CWC (1993) have been adopted. For the four bells of the design storm, two storm patterns have been considered and they are referred as case-1 and case-2. The design storm patter four bells (Case-1) results in double peak of the design flood hydrograph. The corresponding peak flood is the lowest. However, for the design storm pattern of the four bells (Case-2), efforts have been made to arrange the rainfall depths in such a way that a single peak design flood hydrograph can be obtained without exceeding the 24-hour PMP value for the first day. Table-3 shows the design flood peaks and time to peaks and the percentage variation with respect to the values of the reference run (one bell design storm). From the Table-3, it is observed that there is reduction in peak of the order of 28.7% when the four bell (Case-1) design storm has been considered. Whereas, reduction in design flood peaks is 8% and 13.3% for design storms of two bells and four bells (Case-2) respectively. The percentage variation in time to peak is about 61% for four bell (Case-1) design storm and about 29% for both two bells as well as for four bells (Case-2) design storms. Thus, the sensitivity analysis results show that the storm pattern has significant impact on the peak and time to peak of the design flood hydrograph.

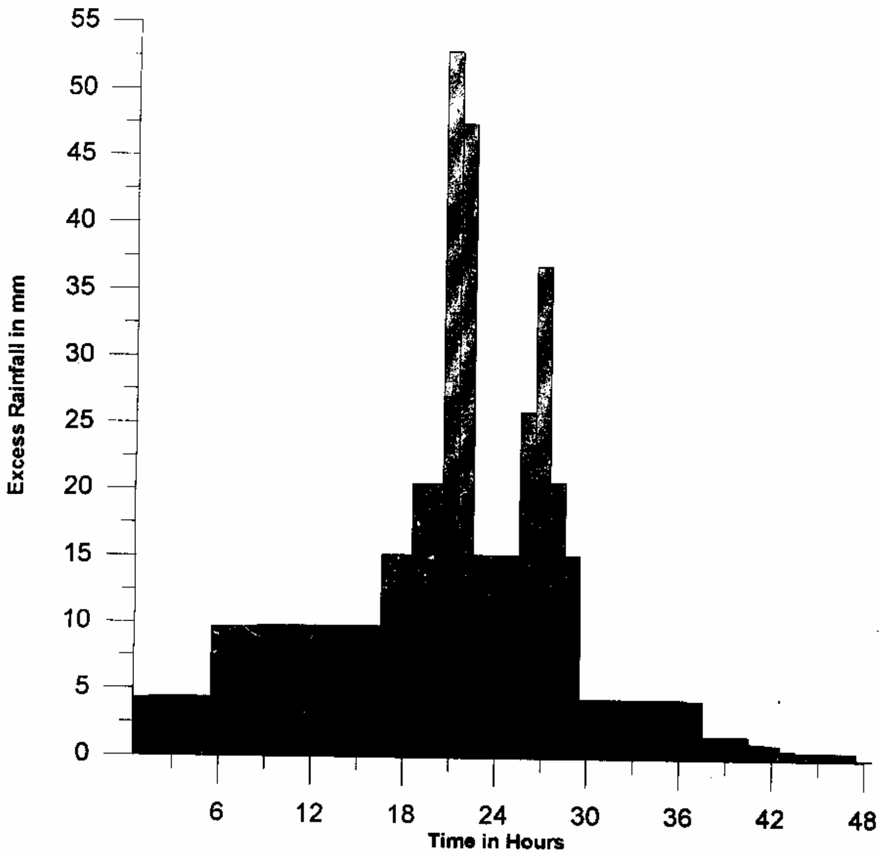

Fig.12 Design excess rainfall hyetograph considered as 2 bells

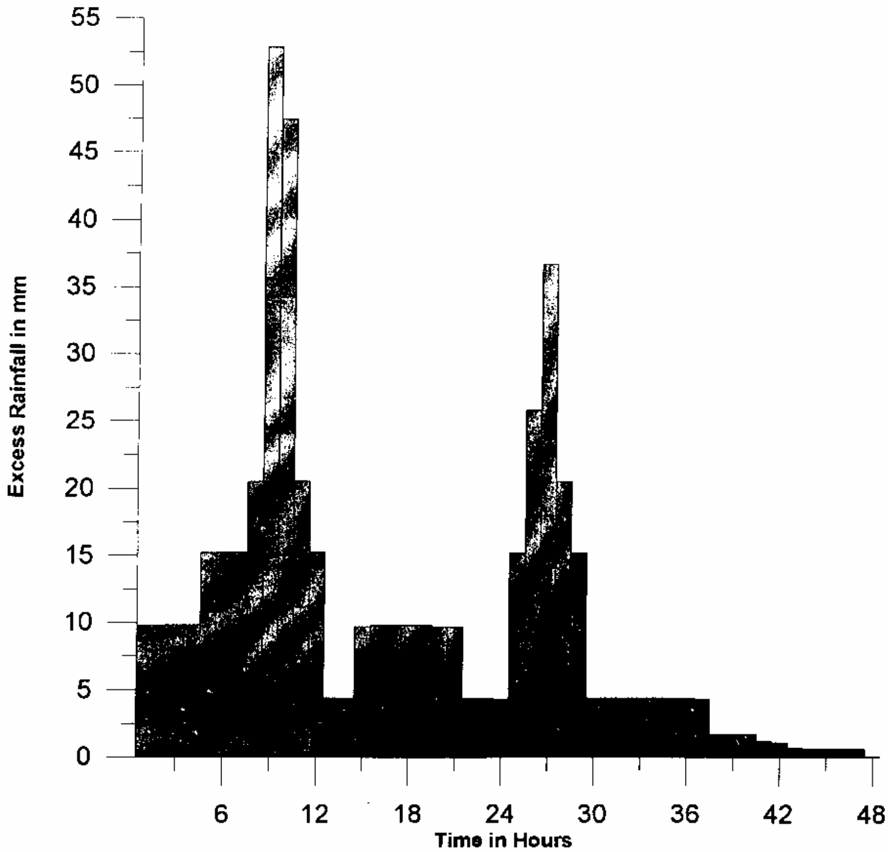

Fig.13 Design excess rainfall hyetograph considered as 4 bells (Case 1)

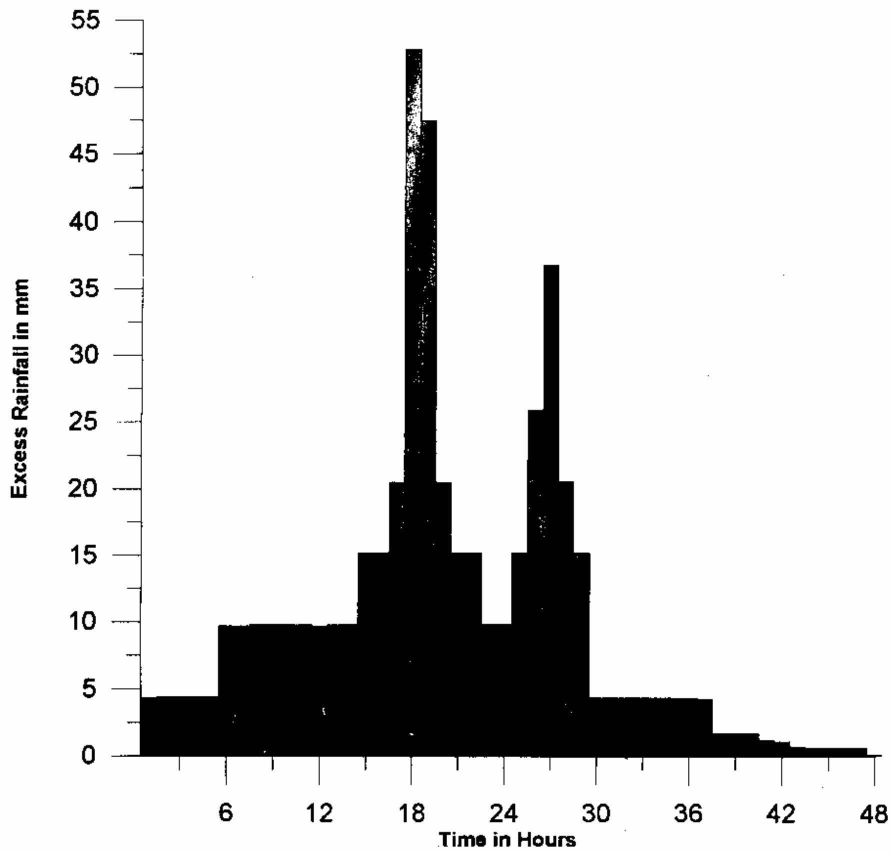

Fig.14 Design excess rainfall hyetograph considered as 4 bells (Case 2)

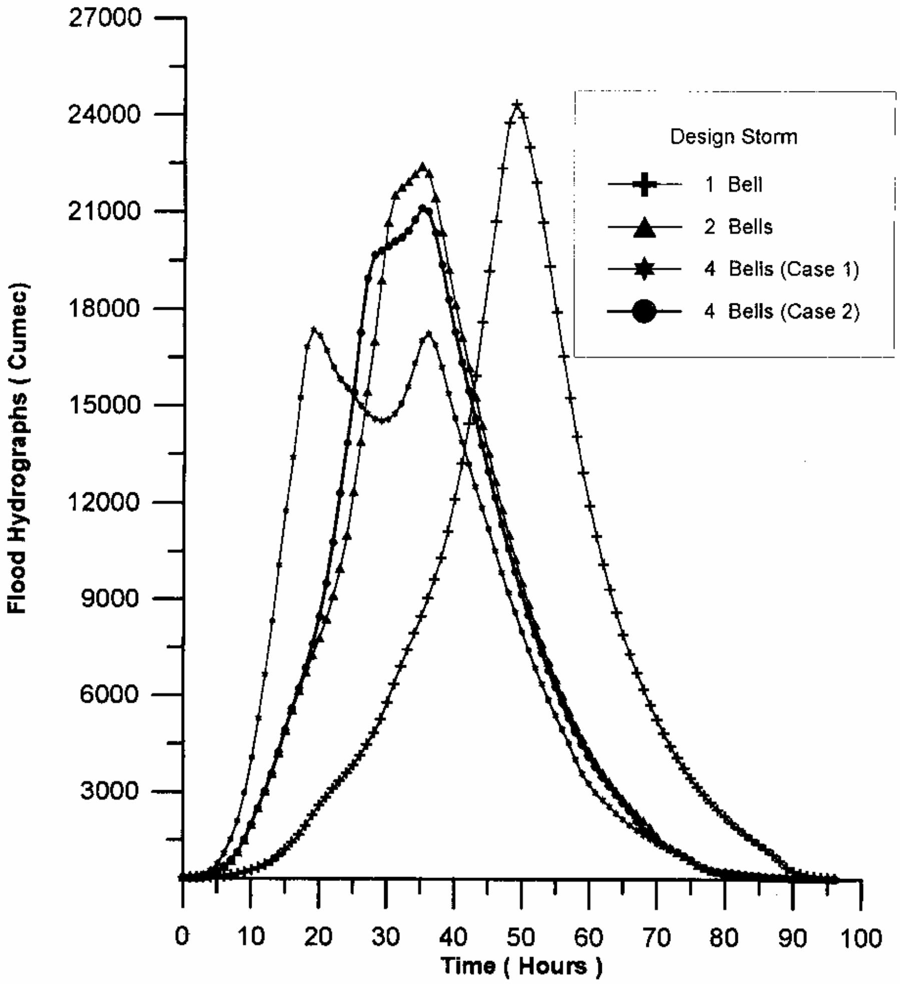

Fig.15 Design flood hydrographs for various storm patterns

Table 3 : Design flood peak (Qp) and time to peak (Tp) and the percentage variation with respect to the values of the reference run for various storm patterns

S.No.	Design storm Patterns	Peak (Qp) (Cumec)	Percent Variation in Qp	Time to Peak (Tp) (Hours)	Percent variation in Tp
1	One Bell(Reference run)	24359	Reference run	49	Reference run
2	2 Bells	22407	-8.0	35	-28.6
3	4 Bells (Case-1)	17369	-28.7	19	-61.2
4	4 Bells (Case-2)	21121	-13.3	35	-28.6

6.4 Sensitivity of the Design Loss Rate

In order to study the sensitivity of the design flood peak to change in loss rate values sensitivity run have been taken up for the uniform loss rates of 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5 and 4.0 mm/hour, considering other design parameters same as for the reference run. Fig. 16 shows the variation of percentage decrease in peak of the flood hydrograph with increase in loss rate. Variation of peak (Qp) and time to peak (Tp) of the design flood hydrograph with loss rate is shown in Table-4. It is seen from the Table-4 that with increase in loss rate the peak of the design flood hydrograph decreases.

Table 4 : Variation of percent decrease in peak (Qp) and time to peak (Tp) of the design flood hydrograph with increase design loss rate

S.No.	Loss rate (mm/hr)	Peak of flood hydrograph (Cumec)	Percent decrease in peak	Time to peak (Hours)	Percent increase in time to peak
1	0.0	25609	-5.1	49	0
2	0.5	24985	-2.6	49	0
3	1.0	24359	Reference run	49	Reference run
4	1.5	23736	2.6	49	0
5	2.0	23120	5.0	49	0
6	2.5	22510	7.6	49	0
7	3.0	21907	10.1	49	0
8	3.5	21308	12.5	49	0
9	4.0	20709	15.0	49	0

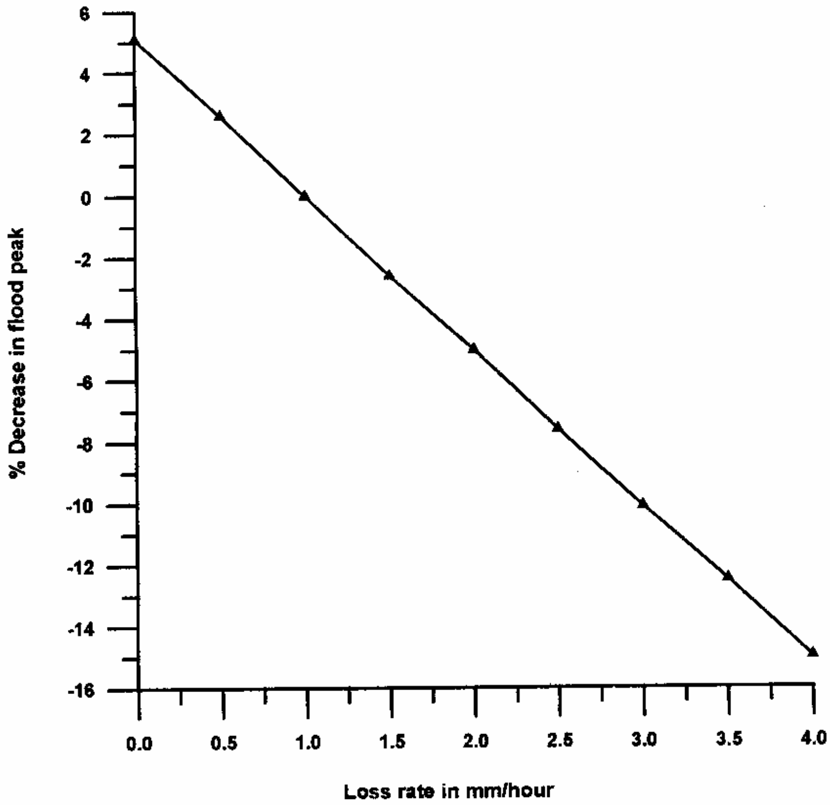

Fig. 16 Variation of percentage decrease in peak of flood hydrograph with loss rate

The percentage decrease corresponding to loss rate of 4 mm/hour (400% increase) is only 15%, with respect to the reference run. It indicates that the design flood hydrograph is not much sensitive to the loss rate.

6.5 Sensitivity of Temporal Distribution Pattern of Design Storm

The pattern of temporal distribution of design storm also plays a significant role in determining the peak of the design storm hydrograph. Whenever, the problem is referred to IMD, the IMD provides the values of the PMP along with its temporal distribution within bells to be used for design flood estimation as per the recommendations of WMO. Whenever, there is concurrent data of the storm under consideration for design the time distribution of areal rainfall over the catchment is recommended (CWC, 1993). In this study, the sensitivity of the temporal distribution of storm pattern has been conducted by considering two more distribution patterns apart from the storm pattern available for the design storm. The design storm distribution adopted for the reference run as well as the two other distribution patterns considered in the study are shown in Fig. 17.

The design flood hydrographs of the design storm distributions of the reference run as well as for the two sensitivity runs are shown in Fig. 18. It is seen from the figure that the design flood peak decreases from 24,359 (reference run/distribution pattern 1) to 20,023 cumec i.e. by 17.8% for the design storm distribution pattern 2. Whereas, for the design storm pattern 3, the peak increases to 27,745 viz. 13.9%. The time to peak does not get effected for the above cases. It shows that the design storm pattern affects the peak of the design flood hydrograph very significantly.

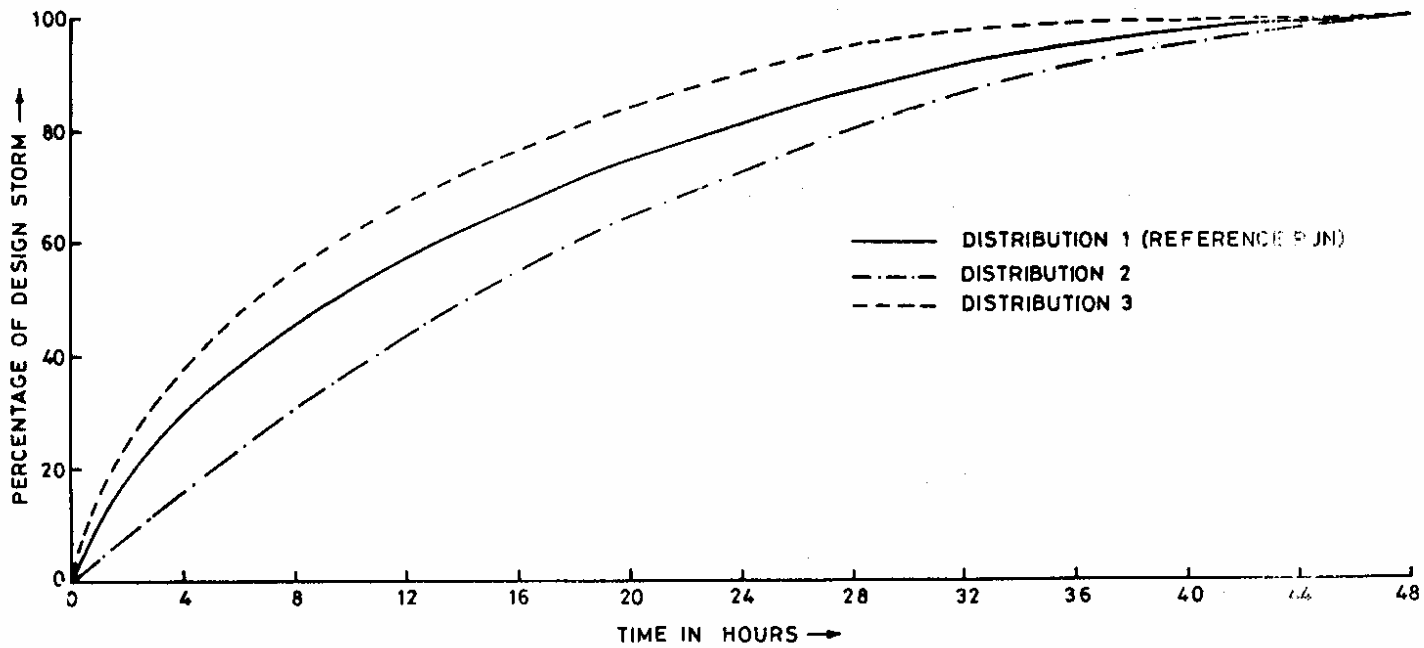

FIG. 17 VARIOUS PERCENTAGE DESIGN STORM DISTRIBUTION PATTERNS

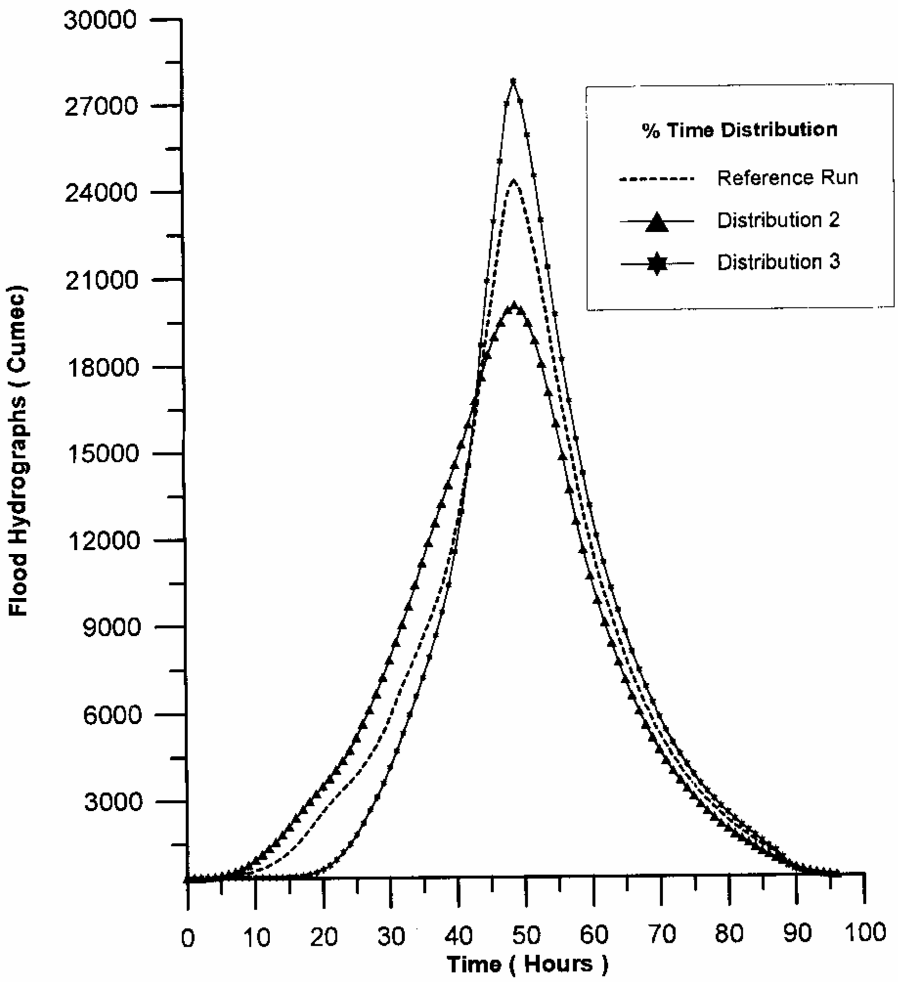

Fig.18 Design flood hydrographs for various percentage design storm distribution patterns

7.0 CONCLUSIONS

The following conclusions are drawn from this study.

(i) In this study, unit hydrograph has been derived using the Clark model. The design unit hydrograph together with other design parameters has been used for estimation of design flood hydrograph.

(ii) An objective methodology has been suggested and applied for estimation of design unit hydrograph parameters after increasing the peak of the average unit hydrograph to cater for the hydraulic conditions of the channel and the catchment for the extreme flooding situations, which would occur due to the occurrence of the design flood.

(iii) Design flood peak has increased by about 25% for an increase in design unit hydrograph peak of 50%.

(iv) The design flood peak is not much sensitive to the change in the design loss rate as even 400% increase in the design loss rate results in decrease of flood peak by only 15%.

(v) The sensitivity runs taken considering the various design storm patterns show that design flood estimates are very much dependent on the temporal distribution and the critical sequencing of the PMP values.

(vi) It is seen from the figure that the design flood peak decreases from 24,359 (reference run/distribution pattern 1) to 20,023 cumec i.e. by 17.8% for the design storm distribution pattern 2. Whereas, for the design storm pattern 3, the peak increases to 27,745 viz. 13.9%. The time to peak does not get effected for the above cases. It shows that the design storm pattern affects the peak of the design flood hydrograph very significantly.

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