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DERIVATION OF GIUH FOR SMALL CATCHMENTS OF UPPER NARMADA AND TAPI SUBZONE (SUBZONE 3C) PART II

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PREFACE

Rainfall-runoff modelling has been an important area of research in the field of hydrology. The phenomenon of rainfall runoff in a watershed is very complex. Our understanding of the physical principles and mathematical formulations to represent them is not yet adequate. Though the instrumentation is being done at a very fast speed, yet, there are vast expanses of land, especially those constituting small to medium sized catchments, which do not have adequate facility for the observation of hydrological variables. This has led to the modelling of ungauged catchments where a very limited amount of information is generally available. Indirect inferences through regionalisation are sought for such types of catchments. Many times this task of regionalising the hydrological parameters becomes very tedious and in certain cases even impossible. Recently, the concept of geomorphological instantaneous unit hydrograph (GIUH) has been introduced wherein the characteristics of the instantaneous unit hydrograph are related to the geomorphological and climatic characteristics of the basin.

The research in the field of fluvial geomorphology has recently picked up and offers some great opportunities in solving many of the problems facing the hydrologists today. A very complicated analysis is required for accurate inferences based on the geomorphological theory. Many investigators have simplified its application to different levels. Also, there have been attempts to relate the parameters of the conventional conceptual models of instantaneous unit hydrograph to the geomorphological characteristics of the catchment.

A mathematical model has been developed at the National Institute of Hydrology which enables the evaluation of the Clark Model parameters using geomorphological characteristics of the basin. Earlier this model was implemented on the Kolar sub-basin of river Narmada and three small catchments of Upper Narmada and Tapi Sub-zone 3c. In this part of the study the model is applied on the remaining fifteen small catchments of the sub-zone 3c. By using this approach the necessity of extensive observed runoff data for the calibration of the Clark model parameters is avoided.

This study has been carried out by Shri Hemant Chowdhary, Scientist 'C' under the guidance of Shri R D Singh, Scientist 'E' of the Surface Water Analysis and Modelling Division of the National Institute of Hydrology, Roorkee. Manual estimation of geomorphological characteristics of these catchments has been done Shri Hemant Chowdhary, Smt. Rama Devi Mehta, Scientist 'B', Shri Mukesh Kumar, 'R A' and Shri Rajesh Agarwal, 'R A' It is expected that this report, on one hand, would be greatly appreciated by the practising engineers and hydrologists, and on the other hand, introduce a new idea for research and its application in the field of fluvial geomorphology. A continuous effort in this regard may result in a better understanding and an easy modelling procedure for rainfall-runoff process using geomorphological approach.

(S M Seth)
Director

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ABSTRACT

The computations of flood hydrographs have always been one of the major concerns of the water resources engineers and scientists. For the purpose of rainfall-runoff process simulation, mathematical modelling is often resorted to. Continued research in this field has resulted in numerous types of rainfall-runoff models. For simulation and design flood evaluation, conceptual models and physically based models are widely used. The linearity principle of unit hydrograph theory has been widely applied for the simulation of rainfall-runoff process, particularly for small and medium sized catchments. Derivation of unit hydrograph has been extensively investigated by many researchers since Sherman gave the principle of unit graph in 1932. For the gauged catchments the unit hydrographs can be derived by analysing the historical rainfall-runoff records. However, for ungauged catchments some indirect approaches have been used for the derivation of the unit hydrographs. Due to scarcity of data, particularly for small and medium sized catchments, physically based models are very difficult to be implemented. Greater emphasis is now being given to the concept of models based on geomorphological characteristics. Geomorphological instantaneous unit hydrograph is one among the various approaches available for the simulation of flood events, especially for the ungauged catchments. Many investigators have tried to relate the parameters of the conceptual models to the geomorphological characteristics of the catchments.

A mathematical model has been developed at the National Institute of Hydrology which enables the evaluation of the Clark Model parameters using geomorphological characteristics of the basin. Earlier this model was implemented on the Kolar sub-basin of river Narmada and three small catchments of Upper Narmada and Tapi Sub-zone 3c. In this part of the study the model is applied on seventeen small catchments of the sub-zone 3c.

Various event based conceptual models and the models for ungauged catchments have been reviewed. The description of the study area alongwith the availability of the data for the present study has also been presented. The methodology is fully explained and analysis has been carried out by using the computer software developed for this approach. Since the data for historical flood events and stream gauging could not be obtained the model is applied to obtain the unit hydrographs for various small catchments corresponding to different velocities of flow. Flood events may be simulated by having an indirect estimate of velocity of flow corresponding to the rainfall intensity of the event. Conclusions drawn have been presented alongwith the suggestions for further work in the direction of improvement of the methodology.

1.0 INTRODUCTION

Simulation of rainfall-runoff process for ungauged catchments is one of the important areas of research in the sphere of surface water hydrology. There are a number of well established techniques like unit hydrograph, conceptual or physically based modelling which are employed for the purpose of rainfall-runoff process simulation for the catchments. All such techniques require a certain amount of historical data for establishing various parameters. However, due to very sparse gauging network available in most of the Indian catchments, particularly for small catchments it becomes very difficult for such techniques to be directly applicable. In such situations of very poor data availability, the options available are, either to go for regionalization of parameters based on the data available for the gauged catchments in nearby hydro-meteorologically similar regions or by using the morphological details available for the ungauged catchments for modelling their hydrological response. Regionalisation of the parameters is, however, a very tedious task to accomplish since the hydrological behaviour of many nearby catchments have to be ascertained before being confident about the values of the parameters. On the other hand, the geomorphological approach has many advantages over the regionalization techniques as it avoids the requirement of flow data and computations in the neighbouring gauged catchments in the region.

As a first step in the direction of using geomorphologic characteristics with the conviction that the search for a theoretical coupling of quantitative geomorphology and hydrology is an area which will provide some of the most exiting and basic developments of hydrology in the future, the concept of Geomorphologic Instantaneous Unit Hydrograph (GIUH) was introduced. This technique, though appears to be tempting to the practitioners for its use in areas of insufficient or inexistent hydrologic data, is very difficult if needed to be applied without making a few assumptions.

A new approach, in which the conceptual modelling of instantaneous unit hydrograph (IUH) is combined with the geomorphologic instantaneous unit hydrograph approach, has been developed at the National Institute of Hydrology. This technique may be applied for the simulation of the flood hydrographs and for the evaluation of the design flood specially for the small to medium sized catchments which are ungauged. By this way, the estimation of parameters of the conceptual model of IUH is not required to be carried out through the tedious regionalisation process. This hybrid approach is developed by linking the Clark's model parameters with the peak characteristics of the geomorphological instantaneous unit hydrograph. The proposed method is called GIUH based Clark model here-in-after in this report.

The methodology was earlier tested by simulating storm events in Kolar sub-basin of river Narmada (NIH, 1993) and three small catchments of Upper Narmada & Tapi subzone (Subzone 3c) - Part I (NIH, 1995) . There are eighteen selected bridge catchments in total located in this sub-zone whose data are available at Central Water Commission. In this II part of the study the model was sought to be applied for simulation of flood events in the remaining fifteen small bridge catchments of Upper Narmada and Tapi Sub-zone (Subzone 3c). Since the flood event and the gauge-discharge data for these fifteen small bridge catchments could not be obtained the model is applied to obtain 1 hour unit hydrographs corresponding to some arbitrary velocities of flow. A comparison is however made with the

1 hour synthetic unit hydrograph recommended for the respective small catchments of the subzone. However, the toposheet pertaining to one of these small catchments namely Br. No. 863 on Sakker river (Itarsi - Jabalpur, Central Railway) could not be obtained, so the same is not included in the study. The three small catchments earlier studied in Part I namely Br. No. 249 on Temur river (Gondia - Jabalpur, South Eastern Railway), Br. No. 930 on Umar river (Itarsi - Jabalpur, Central Railway) and Br. No. 253 on Tyria river (Gondia - Jabalpur, South Eastern Railway) have also been included in this Part II for sake of completeness of the report in itself.

2.0 REVIEW

2.1 GENERAL

The problem of transformation of rainfall into runoff has been a very active area of research throughout the evolution of the subject of hydrology. Through their intuition, many investigators have tried to relate runoff with the different characteristics which affect it. The simplest theoiy proposes to multiply the rainfall with some factor (called the runoff coefficient) to get the runoff. A better way to transform rainfall into runoff is to apply conceptual models in which the various interrelated hydrological processes are conceptualized. More sophisticated procedures are also evolved which are based on the physical concept of the process and try to model this hydrological phenomenon on the basis of physical laws governing them. Never it is inferred that, a particular model is the best for rainfall-runoff transformation. Actually, many more factors, besides the accuracy, e.g., the availability of data, computing facility, time, resources etc. govern the applicability of a model. The search for suitable models for different conditions still continues and thus more and more mathematical models are being suggested.

2.2 REVIEW OF EVENT BASED CONCEPTUAL MODELS

The approaches utilized to develop linear conceptual models of rainfall-runoff relationship may be classified into three groups. The first group employs a differential equation that supposedly governs the operation of a specified system (Kulandaiswamy, 1964; Chow 1964; Shen, 1965; Chaudhry, 1976; Jackson, 1968; Chow and Kulandaiswamy, 1971, 1982; V.P.Singh and Mc Cann, 1979; Mc Cann and V.P. Singh, 1980, 1981; Te and Kay, 1983). The second group utilizes an arrangement of the so-called conceptual elements, including linear channels and linear reservoirs (Nash, 1957; Dooge, 1959, 1977; Chow, 1964; S Bravo et.al., 1970; Maddaus and Eagleson, 1969; Harley, 1967; O'Meara, 1968; V.P. Singh and Mc Cann, 1980a). The third group makes some hypothesis about rainfall-runoff relationship more or less on intuitive grounds (Lienhard, 1964, 1972).

In the second category of the conceptual models Clark (1945) suggested that the unit hydrograph for a watershed due to instantaneous rainfall can be determined by routing its Time-Area-Concentration (TAC) curve through a single linear reservoir. Physically, it is equivalent to Zoch (1934) Model, in which the concept of instantaneous unit hydrograph (IUH) is replaced by one of unit hydrograph. O'Kelly (1955) defined the TAC curve by an iscsceles triangle and routed it through a linear reservoir to produce the instantaneous unit hydrograph for the watershed. Thus, O'Kelly model is equivalent to Clark's model except for the definition of TAC curve.

Nash (1957) developed a model based on a cascade of equal linear reservoirs for derivation of the IUH for a natural watershed. This is one of the most popular and frequently used models in applied hydrology.

Dooge (1959) developed a general unit hydrograph theory, which embraced all previous models as its special cases. The three elements : TAC curves, linear channel and linear reservoir were included in the theory. The basic premise of the Dooge model is that a watershed can be represented by some combination of linear channels and reservoirs. The watershed is drained by a network of channels composed of a complex network of linear channels and linear reservoirs placed in series.

2.3 MODELS FOR UNGAUGED WATERSHEDS

The parameters of the models reviewed in previous section are generally calibrated based on the analysis of rainfall-runoff data for gauged catchments. However, these models can not be calibrated for those catchments which lack such data. Consequently, the parameters of those models for ungauged catchments may be determined from the regional relationships developed by correlating the model parameters with physically measurable catchment characteristics of the gauged catchments. Optimization is one of the most widely used techniques available to calibrate the model for gauged catchments. Frequently, the model parameters are optimized for some selected rainfall-runoff events over a given watershed, using a suitable optimization procedure. The optimized parameter values are then utilized in the model to predict runoff for the rainfall events of interest not used in the calibration process. This approach is obviously not applicable to ungauged watersheds. Further, it has other shortcomings as the optimized parameters can best represent the watershed only for the events used in the calibration. The optimized values change with the change in the events. Also, the extensive amount of data required for calibration is normally lacking and thus prove prohibitive in the widespread use of the model.

The other approach attempts to establish relationships between model parameters and physically measurable watershed characteristics. These relationships are then assumed to hold for ungauged watersheds having similar hydrologic characteristics. Rainfall-runoff relationships for ungauged watersheds have been developed along two complimentary lines: (1) Empirical equations have been developed to relate some individual runoff hydrograph characteristics to watershed characteristics (2) Procedures have been developed to synthesize the entire runoff hydrograph from watershed characteristics. Some of these models are reviewed here under.

Bernard (1935) model is perhaps the first attempt to synthesize the unit hydrograph (UH) from watershed characteristics. It assumes that the peak of the UH is immensely proportional to the time of concentration, which in turn is assumed to be proportional to a watershed factor. A distribution graph establishes relation between the effective percentage area contributing and the watershed factor for different days of the storm.

Snyder (1938) established a set of formulae relating the physical geometry of the watershed to three basic parameters of the unit hydrograph. Mc Carthy (1938) related three parameters of 6-hour UH, including the time of rise, the peak discharge, and the base length, to watershed characteristics such as area, overland slopes expressed as the average slope of the hypsometric curve and stream pattern. Taylor and Schwarz (1952), in addition to the watershed characteristics employed by Snyder (1938), introduced the average slope of the main channel. The method of hydrograph synthesis employed by the Soil Conservation Service (SCS) (1971), U.S. Deptt. of Agriculture, uses an average dimensionless hydrograph derived from an analysis of a large number of natural UHs for watersheds varying widely in size and geographical locations.

As mentioned earlier, the Clark model involves determination of the TAC diagram and the storage coefficient. This storage coefficient has been related with the catchment characteristics. The time of concentration was considered to equal the time interval between the end of rain and the point of contraflexure of the hydrograph recession limb. This time base was measured from the recorded floods and not related to watershed characteristics.

Nash (1960) model has two parameters n and K. Nash showed that these parameters were related to the first and second moments of the IUH about the origin. These moments were then correlated empirically with watershed characteristics.

In early years, in India, the design discharges for very small and medium catchments were used to be calculated by well known empirical formulae viz. Dickens, Ryves, Inglis, Ali Nawaz Jung, etc. Later on, to evolve a method of estimation of design flood peak of desired frequency for small catchments, the unit hydrograph approach has been adopted by the Central Water Commission. For this purpose, the country has been divided into 7 major zones which are sub-divided into 26 hydrometeorologically homogeneous subzones. For most of these sub-zones, Central Water Commission has already developed regional formulae for different sub-zones for the derivation of the synthetic unit hydrograph. The unit hydrograph characteristics such as peak (Q_p), time to peak (t_p), W_{50}, W_{75}, W_{R50}, W_{R75}, time base (t_B) etc. have been computed on the basis of physiographic features. These regional formulae enable computation of unit hydrograph for ungauged catchments of the sub-zones. The reports prepared by CWC for different sub-zones (e.g., CWC, 1983 for sub-zone 3c) may be referred in this regard.

The regional unit hydrograph studies have also been carried out for some of the sub-zones by various research and academic organisations besides Central Water Commission. Singh (1984) developed regional unit hydrograph relationship for lower Godavari sub-zone (3f) relating the parameters of Nash and Clark models with the physiographic characteristics of five gauged catchments in the sub-zone.

National Institute of Hydrology (1985) has carried out a regional unit hydrograph study for Narmada basin based on Clark's approach. In this study the parameters of the Clark model have been derived for each of the sub-basin of Narmada basin using HEC-I package. A regional relationship has been developed in the graphical form relating average value of (t_c + R) for each sub-basin with their respective catchment area. A regional value of R/(t_c + R) along with the graphical relationship has been used to estimate the parameters of the Clark model for ungauged catchment of the Narmada basin.

Huq. et.al. (1982) developed synthetic unit hydrograph relationships using the data of the catchments in Gangetic plains, Mahanadi basin, Krishna basin and Bhramaputra basin. These relationships have been developed relating the parameters of the representative unit hydrograph for gauged catchment with a suitable combination of the physical characteristics of the catchment using regression analysis.

Mathur and Vijay Kumar (1987) related the physical parameters of twenty small and medium catchments in order to arrive at the most effective combination of the physical parameters for the development of the regional unit hydrograph relationships.

Although number of such relations are developed with the hope that they will yield satisfactory results when applied to the ungauged basin, these approaches have following limitations :

(i) The catchment for which data is used in a regional study have to be similar in hydrological and meteorological characteristics. However, it is usually difficult to locate catchments strictly satisfying these requirements.

(ii) While establishing such relations, the inherent limitations of the unit hydrograph theory are also being carried out with it. As a result the prevailing method of predicting the discharge hydrograph for a design storm by using the average unit hydrograph will not be appropriate, since the average unit hydrograph does not necessarily reproduce the actual response due to such inherent limitations.

(iii) The relationship evolved are based upon the gauged observations in number of catchments in the region. It is practically very difficult to always have gauged catchments available in adequate numbers in a region to enable the development of such relationships.

(iv) Generally, the data for intense and short duration storms are not available for the derivation of average unit hydrograph for gauged catchments. Hence the average unit hydrograph derived from minor flood events is considered for the regionalisation. It may result in the under estimation of design flood for ungauged catchments.

Boyd (1978, 1982) developed the linear watershed bounded network (LWBN) model for synthesis of the IUH employing geomorphologic and hydrologic properties of the watershed. The model divides a watershed into sub-areas bounded by watershed lines using large-scale topographic maps. The model has a large number of lumped storage parameters. Most of these parameters are deduced from geomorphologic properties.

Rodriguez-Iturbe and Valdes (1979) developed an approach for derivation of the IUH by explicitly incorporating the characteristics of drainage basin composition (Horton, 1945; Strahler, 1964; Smart, 1972). The approach coupled the empirical laws of geomorphology with the principles of linear hydrologic systems. Rodriguez-Iturbe and his associates have since extended this approach by explicitly incorporating climatic characteristics and have studied several aspects including hydrologic similarity. Gupta, Waymire and C.T.Wang (1980) examined this approach, and reformulated, simplified and made it more general.

The effect of climatic variation is incorporated by having a dynamic parameter velocity in the formulation of Geomorphological IUH (GIUH). This is a parameter that must be subjectively evaluated. It is shown (Rodriguez-Iturbe, et.al., 1979) that this dynamic parameter "velocity" of the GIUH can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin. This transforms the time invariant IUH throughout the event into a time invariant IUH in each storm occurrence.

In the derivation of GIUH one of the greatest difficulties involved is the estimation of peak velocity. This is a parameter that must be evaluated for each flood event. Rodriguez et.al. (1982) rationalised that velocity must be a function of the effective rainfall intensity and duration and proceeded to eliminate velocity from the results. It leads to the development of geomorphoclimatic instantaneous unit hydrograph. The governing equations consists of the terms such as the mean effective rainfall intensity, Manning's roughness coefficient, average width, and slope of the highest order stream.

Janusz Zelazinski (1986) gave a procedure for estimating the flow velocity. It involves the development of the relationship between the velocity and corresponding peak discharge. A methodology based on trial and error procedures has been suggested for estimating the maximum value of the velocity for each flood event.

Panigrahi (1991) estimated the velocity using the Manning's equation. The methodology involves the estimation of equilibrium discharges and subsequently the estimation of the velocity corresponding to it using Manning's equation. It requires the intensity of each rainfall block for the event for the computation of equilibrium discharge. The channel cross-section at the gauging site, longitudinal slope and Manning's roughness are also required during the computation of the velocity. The methodology has been applied to estimate the velocity to derive the Nash model parameters using GIUH approach for the Kolar sub-basin of Narmada basin.

Development of GIUH has potential applications for the estimation of runoff, flood forecasting and design flood estimation, particularly for the ungauged catchments or for the catchments with limited data. Most of the studies available in literature regarding the GIUH approach are synthetic in nature and are in the early stages of research and development. Very few studies are available where its practical applications have been demonstrated. As GIUH approach has many advantages over the traditional method of developing the regional unit hydrograph for the simulation of flood events in the ungauged catchment, it would be appropriate to verify the application of GIUH approach for simulating the flood response of a gauged catchment. In the light of this a new approach of rainfall-runoff modelling based on the geomorphological characteristics has been developed at the National Institute of Hydrology. This technique links the GIUH equations derived by Rodriquez and the parameters of the Clark model. It enables the estimation of parameters of Clark model using the geomorphological characteristics, hydraulic properties of the main stream and storm characteristics. This approach was tested satisfactorily on the Kolar sub-basin of river Narmada (NIH,1993) and on three small catchments of Upper Narmada & Tapi Subzone (Subzone 3c) (NIH, 1995).

3.0 STATEMENT OF THE PROBLEM

The conceptual rainfall-runoff models invariably require calibration of their parameters. This calibration is carried out on the basis of some observed events. For the case of ungauged catchments, where no such observed events are available for the calibration purpose, regionalization on the basis of nearby gauged catchments is resorted to. Such an exercise becomes very tedious considering the computational effort required. Also, it is to be repeated from time to time whenever more observations become available.

Rainfall-runoff modelling based on the geomorphological details of the basin is a new concept in hydrology. Analytical procedures have been established for the derivation of the geomorphological instantaneous unit hydrograph. Such approach may be advantageously applied even for the ungauged catchments as it does not require the observed runoff data. However, these procedures have been tried for basin of smaller stream orders only. For basins of four or higher stream order this type of analytical procedure becomes highly complicated and has not been applied so far. Two formulae for the peak characteristics of the geomorphological instantaneous unit hydrograph (GIUH) have been suggested. But these formulae are not adequate to describe the shape of the instantaneous unit hydrograph (IUH) fully.

A new approach of rainfall-runoff modelling has been developed at the National Institute of Hydrology (NIH, 1993) in which the conceptual modelling has been clubbed with the GIUH approach. This has enabled to determine the complete shape of the IUH by using the formulae given for the peak characteristics of the GIUH. Simultaneously on the other hand, it has been possible to use the conceptual modelling approach without even required to calibrate its parameters on the basis of the observed runoff data. The conceptual model used in this new approach is the Clark model.

In this study the main objective is to apply the new approach developed at the National Institute of Hydrology for the derivation of variable geomorphological instantaneous unit hydrograph for the seventeen small catchments of Upper Narmada and Tapi sub-zone. This approach makes use of geomorphologic details of the catchment while establishing the parameters of Clark's model for the ungauged catchments.

The necessary rainfall-runoff and the stream gauging data for the study could not be obtained and thus the scope of this study is limited only to the development of GIUH and thereby 1 hour unit hydrograph corresponding to a set of arbitrary expected velocities. However, if the required rainfall-runoff data is available any rainfall event may be associated with a unit hydrograph corresponding to the expected velocity of flow for that rainfall event. The methodology for associating any rainfall event with a particular velocity of flow is given in full detail later in the text.

4.0 DESCRIPTION OF THE STUDY AREA

4.1 GENERAL

Taking into account the limitations in adopting the empirical formulae and also substantial progress made in the development of hydrological science Government of India in 1955 constituted a high level committee of Engineers under the Chairmanship of Dr. A N Khosla, to indicate a rational method to determine collection of hydrometeorological data of selected catchments in different climatic zones of India for evolution of revised approach for determination of design flood discharge. Since long term data on small and medium catchments is not available, the planning and Coordination Committee comprising of Central Water Commission, Research Design & Standard Organisation of Ministry of Railways, India Meteorological Department, Ministry of Transport and INC for IHP have adopted the approach of obtaining design flood based on the design storm . The Khosla Committee of engineers recommended two approaches, viz: Long Term Plan and Short Term Plan. Under the Short Term Plan, a method was devised to estimate design flood peak based on unit hydrograph principle and under the Long Term Flood Estimation Plan the country has been divided into 7 major zones which in turn are sub-divided on the basis of river basins and sub-basins into 26 hydro-meteorologically homogeneous sub-zones of moderate sizes (Fig. 4.1).

The sub-zone 3(c) is shown hatched in plate in Fig. 4.1. The location of the selected bridge catchments of the sub-zone 3c is shown in Fig. 4.2. There are eighteen selected bridge catchments in total in the sub-zone for which the data is available. However, the toposheet pertaining to one of these small catchments namely Br. No. 863 on Sakker river (Itarsi - Jabalpur, Central Railway) could not be obtained, so the same is not included in the study.

4.2 GENERAL DESCRIPTION OF UPPER NARMADA AND TAPI SUB-ZONE

4.2.1 River system

The sub-zone 3(c) comprises of upper portion of Narmada and Tapi basins combined and constitutes about 50% of the entire area of the combined Narmada and Tapi basins. Common boundary dividing the two sub-zones falls approximately along a line joining the points at 76° 15′ and 76° 30′ longitudes on the northern and southern boundaries respectively of these two sub-zones. The Narmada, westward flowing river of the peninsula, rises near Amarkantak in the Mailkala range in the Shahdol district of Madhya Pradesh at an elevation of about 1000 metres above sea level. It flows for a length of about 1300 km before it outfalls into the gulf of Cambay in the Arabian sea. The River Tapi rises near Multai in the Betwa district of Madhya Pradesh and like Narmada it flows westward for a length of about 725 Km before outfalling into the gulf of Cambay.

The lengths of main Narmada and Tapi rivers in the upper sub-zone are 813 km and 229 km respectively. The upper sub-zone covers parts of Madhya Pradesh and Maharashtra States. The important tributaries of Upper Narmada and Tapi are Burhnar, Banjar, Sher, Shakkar, Dudha, Tawa, Ganjal and Chhota Tawa along left bank and Hiran, Tendori, Barna,

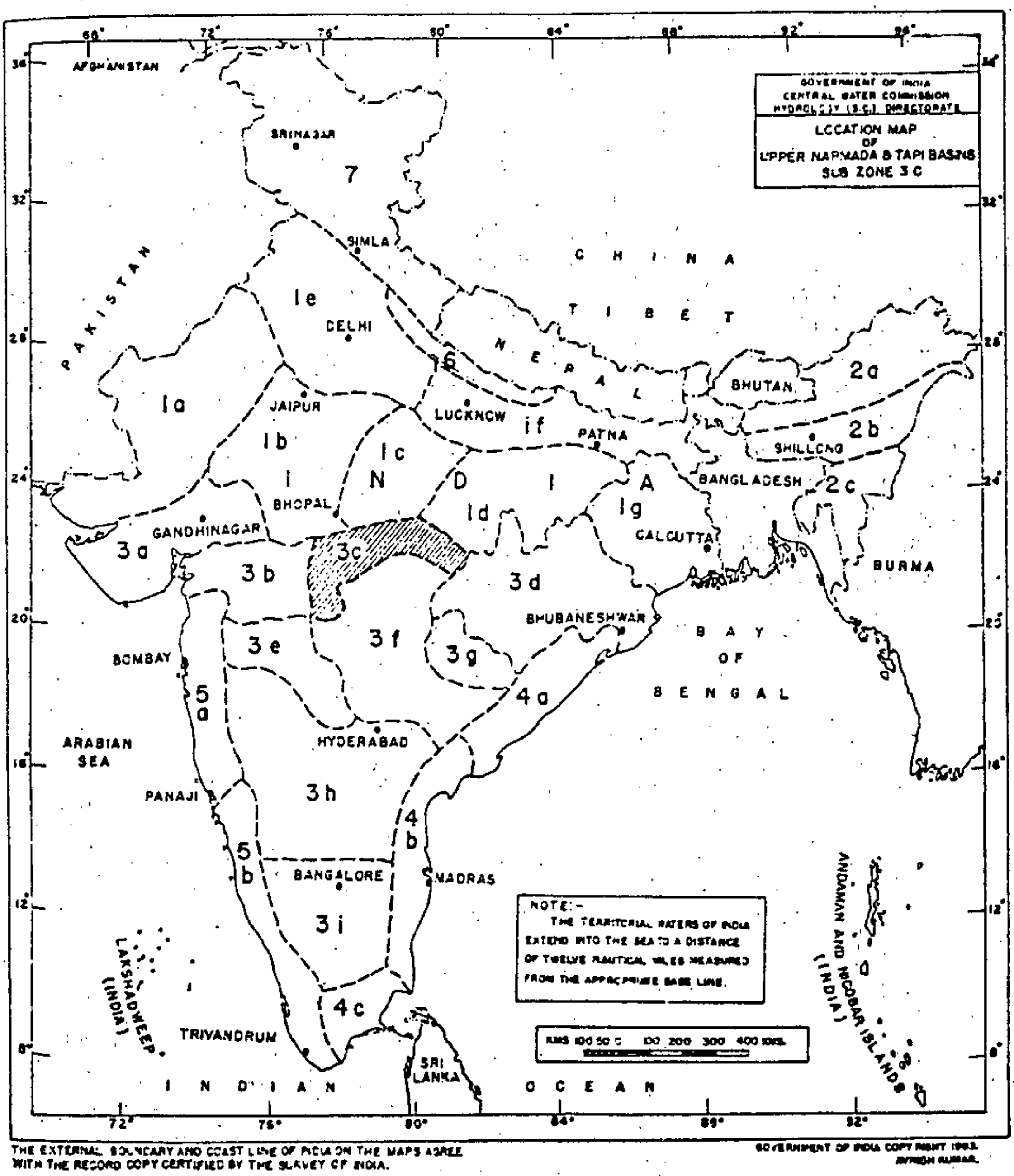

FIG. 4.1 LOCATION MAP OF UPPER NARMADA AND TAPI SUBZONE 3C

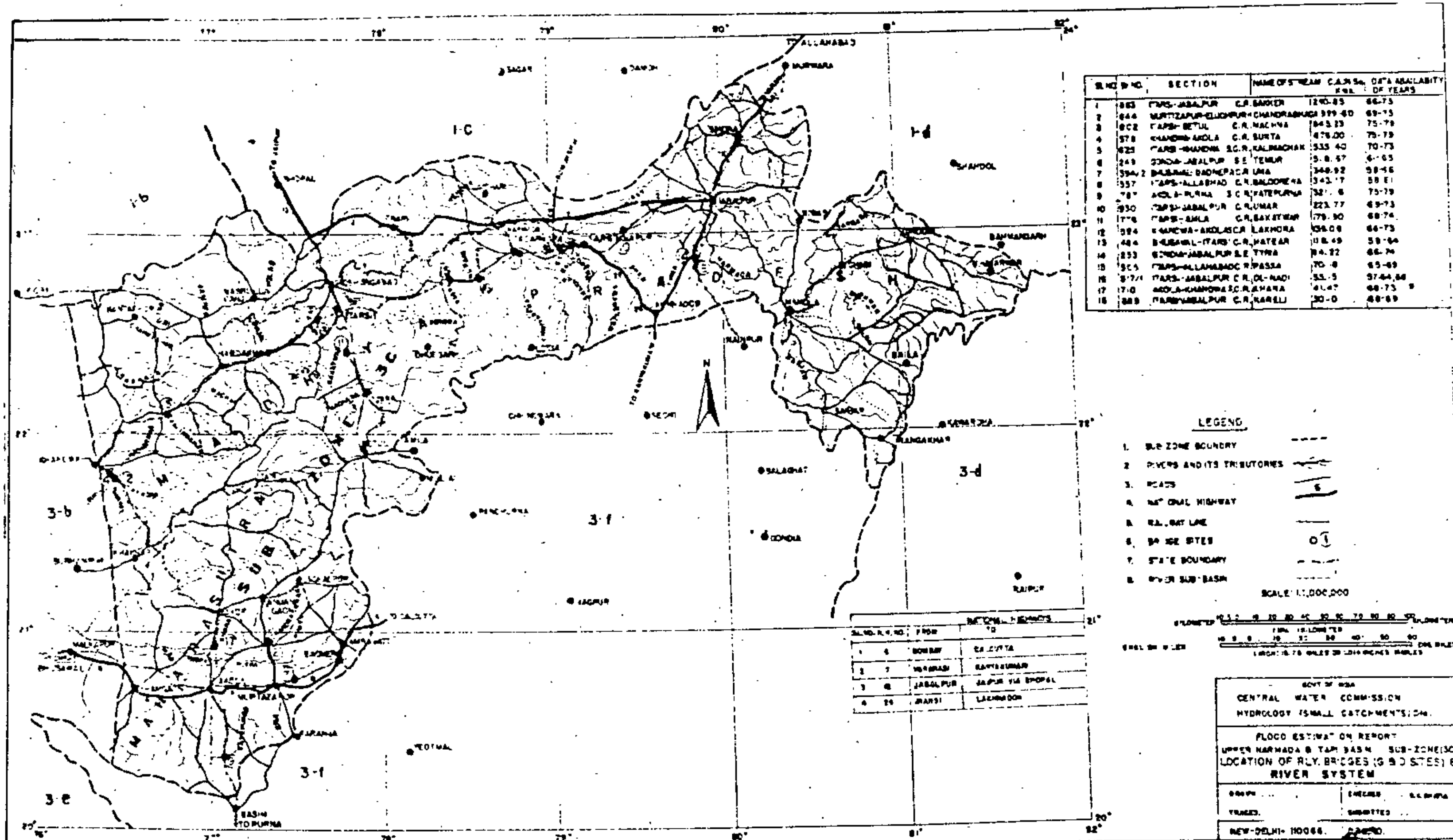

FIG. 4.2 LOCATION OF RAILWAY BRIDGES (G D SITE) AND RIVER SYSTEM IN THE SUBZONE 3C

Kolar, Jamner and Datuni along right bank. Purna is the main tributary of Tapi. Upper parts of Purna fall in the upper sub-zone 3(c). The drainage areas of Upper Narmada and Upper Tapi rivers and their tributaries included in sub-zone 3(c) are given in Table 4.1.

4.2.2 Topography

The Upper Narmada and Tapi sub-zone lies between east longitudes 76° 12′ to 81° 45′ and north latitudes of 20° 10′ to 23°45′. Lying in the northern extremity of the Deccan plateau, the sub-zone covers the states of Madhya Pradesh and Maharashtra. The sub-zone is bounded by Chambal basin 1 (b), Betwa basin 1-(c) and Sone basin 1-(d) on the north, Lower Narmada and Tapi sub-zone 3-(b) on the West, Lower Godavari sub-zone 3-(f) on the south and Mahanadi sub-zone 3-(d) on the East. Important cities and towns within the sub-zone are Mandla, Jabalpur, Narsinghpur, Itarsi, Betul, Hoshangabad, Akola and Amravati.

The Upper Narmada and Tapi sub-zone has a complex relief. High ranges of above 900 m exist over a small area near the source of Narmada river at Amarkantak. Areas varying in height between 600 m to 900 m lie along the eastern and middle portions of the boundary. About 60 percent of the sub-zone varies in height from 300 m to 600 m. Areas varying in height from 150 m to 300 m lie in patches near the western boundary. Fig. 4.3 shows the physiography of the area.

4.2.3 Meteorology and climatology

4.2.3.1. Rainfall

The sub-zone has a continental type of climate. It is very hot in summer and cold in winter and receives most of the rainfall from the South-West monsoon from June to October. Fig. 4.4 shows the normal annual rainfall pattern of the sub-zone. Mean annual rainfall of the sub-zone varies approximately from 800 to 1600 mm. Mean monthly rainfall histogram for typical cities namely Akola, Indore, Bhopal and Jabalpur, in and around the sub-zone are also shown in Fig. 4.4.

4.2.3.2 Temperature

About 50% of the sub-zone on eastern side is having mean annual temperature of 22.5°C to 25°C, while the western side is having mean annual temperature of 25°C to 27°C. The maximum temperature has been recorded in the month of May and minimum temperature has been recorded in the month of December.

4.2.4 Soils

The main soil group of the sub-zone is black soil comprising of different varieties viz., deep black soil, medium black soil and shallow black soil. In addition, mixed red and black soil, red and yellow soil and skeletal soil are also observed in pockets. Of these, deep black soil covers the major portion of the sub-zone. At micro level (i.e. when small and medium catchments are considered), the soil type may vary considerably from the above indicated group.

Table 4.1: Drainage Area of Upper Narmada & Tapi Rivers and their tributaries

Sl. No.	Name of the Basin/ Sub-basin	Drainage Area (sq. km.)
1.	Burhner	4505
2.	Banjar	3855
3.	Sher	2813
4.	Shakkar	2833
5.	Dudhi	1722
6.	Tawa	4555
7.	Ganjal	2072
8.	Chhota Tawa	3825
9.	Hiran	4505
10.	Tendori	1762
11.	Kolar	1302
12.	Purna (only main tributary of Upper Tapi river)	24089
13.	Main Upper Narmada and Tapi and other minor tributaries	28465
Total Area of sub-zone 3(c)		86353

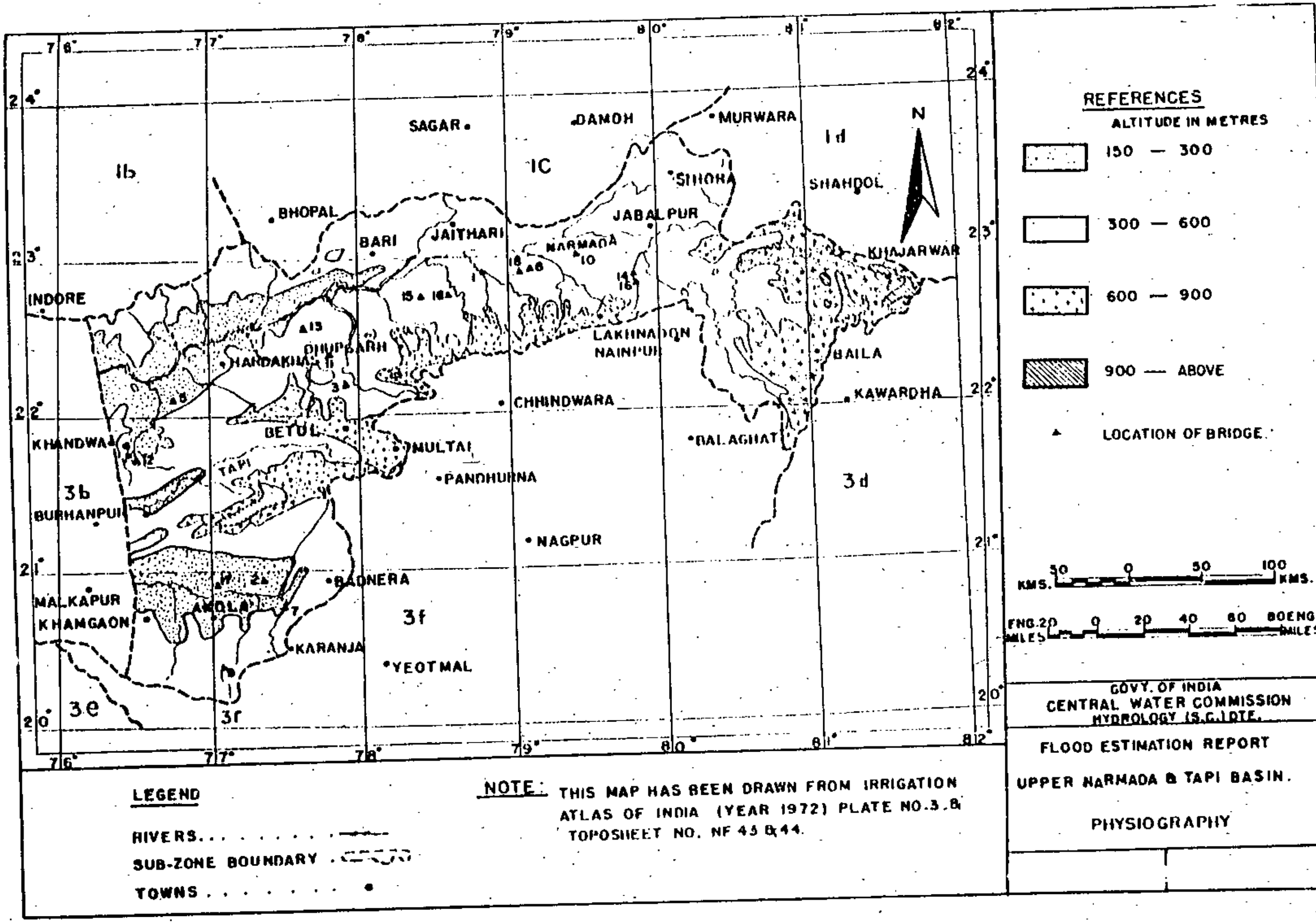

FIG. 4.3 PHYSIOGRAPHY OF SUBZONE 3C

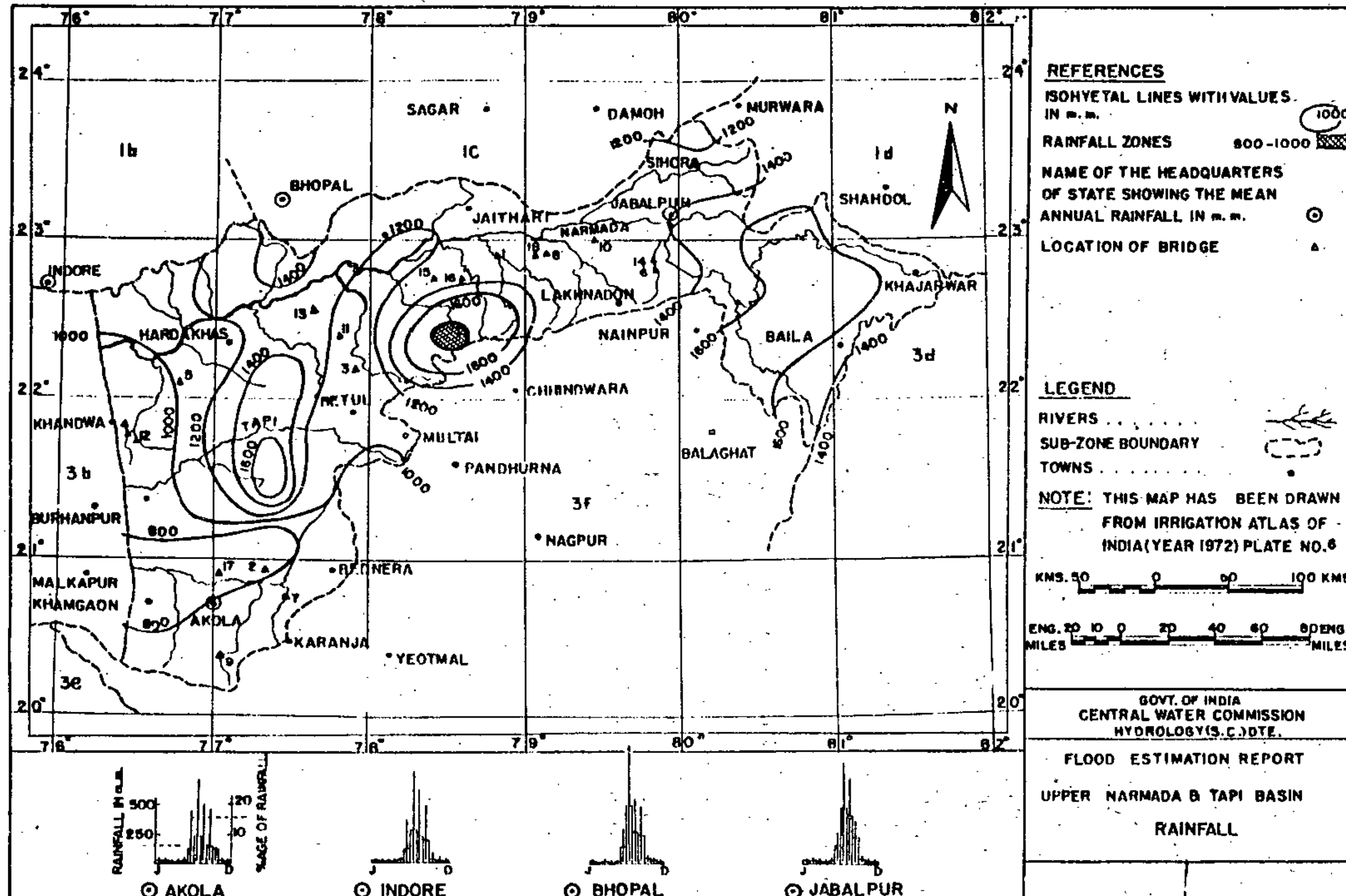

FIG. 4.4 ANNUAL RAINFALL IN SUBZONE 3 C

4.2.5 Land Use

The sub-zone is having extensive area of about 55% under arable land, 40% of area under forest and remaining under wasteland, grassland etc.. Many new projects are proposed to come up in this sub-zone.

4.2.6 Communication

4.2.6.1 Railways

The following railway sections partly or wholly traverse the area of the sub-zone :

1)	Bhusaval - Itarsi - Jabalpur - Katni	C. Railway
2)	Bhopal - Itarsi - Amla	C. Railway
3)	Murtizapur - Achalpur	C. Railway
4)	Bhusaval - Badnera	C. Railway
5)	Khandwa - Akola	S.C.Railway
6)	Gondia - Jabalpur	S.E.Railway

4.2.6.2 Roads

The major highways in the sub-zone are :

(i)	National Highway No. 6	(Bombay to Calcutta)
(ii)	National Highway No. 7	(Varanasi to Kanyakumari via Nagpur
(iii)	National Highway No. 12	(Jabalpur to Jaipur via Bhopal)
(iv)	National Highway No. 26	(Jhansi to Lakhnadon)

5.0 DATA AVAILABILITY FOR THE STUDY

5.1 TOPOGRAPHIC DATA

The topographic maps of all catchments but one are prepared using the Survey of India toposheets on the scale of 1:50,000 scale. However, map of one catchment viz. Br. No 644 on Chandrabhaga river has been prepared on the scale of 1:250,000. This was done because the manual evaluation of geomorphologic characteristics for this catchment on the scale of 1:50,000 would have been very tedious. Since the topographic map for the catchment of Br. No. 863 on Sakker river could not be obtained the same has not been included in this study. In this way a total of seventeen small catchments have been studied in this part II of the study.

5.2 RAINFALL AND DISCHARGE DATA

South Eastern Railways, Central Railways and South Central Railways had observed and collected rainfall, gauge and discharge data for 18 bridge catchments under the guidance and supervision of Research Designs and Standards Organisation, Lucknow. Table 4.2 lists 18 selected bridge catchments alongwith their locations, areas, data available etc.. The data collected for this purpose at the bridge sites consists of the following :

(a) Gauging site details and catchment plans
(b) Hourly rainfall data from raingauge stations in the catchments specially installed for this purpose
(c) Hourly gauge observations at gauging sites
(d) Frequent discharge observations at gauging sites during the day time.

The India Meteorological Department has obtained rainfall data from its own network consisting of both self-recording raingauges and ordinary raingauges, in and around the subzone supplemented by rainfall data collected by South Eastern Railways and Central Railways.

Preliminary scrutiny and analysis of these data have been carried out by CWC, RDSO and IMD under the guidance of FEPCC. However, the data necessary for this study could not be collected from CWC and so the scope of the study has been limited to the development of GIUH for various small catchments corresponding to a set of probable velocities. These probable velocities are the expected velocities during different storm events. For actually simulating the flood events the relation between the intensity of rainfall and the expected velocity have to be obtained on the basis of the observed stream gauging data.

Table 4.2 : Locational and other details of selected railway bridge catchments in Sub-zone 3(c).

Sl. No.	Name of stream	Name of Section where bridge is located, with Railway zone	Bridge No.	G&D site Location		Catchment Area (sq. kms.)	No. of raingauge stations	Years of availability	No. of years
				Latitude	Longitude				
1.	SAKKER	ITARSI - JABALPUR, C.R.	863	22 54'25"	78 48'30"	2110.3	9	66-73	8
2.	CHANDRABHA	MURTIZAPUR - ELLICHPUR, C.R.	644	20 55'35"	77 19'35"	943.5	8	69-73	5
3.	MACHANA	ITARSI - BETUL, C.R.	803	22 12' 0"	77 55' 0"	975.0	9	75-79	5
4.	SUKTA	KHANDWA - AKOLA, C.R.	578	21 45'35"	76 24'55"	640.6	9	75-79	5
5.	KALIMACHAK	ITARSI - KHANDWA, S.C.R	625	22 6'10"	76 46' 5"	562.8	7	70-73	4
6.	TEMUR	GONDIA - JABALPUR, S.E.	249	22 51' 0"	79 50'30"	518.6	7	61-65	5
7.	UMA	BHUSAVAL - BANDNERA, C.R.	394/2	20 45'50"	77 28'58"	353.5	6	58-66	9
8.	BALOORENA	ITARSI - ALLAHABAD, C.R.	897	22 56'55"	79 7'48"	329.9	5	58-61	4
9.	KATEPURNA	AKOLA - PURNA, S.C.R.	787	20 21'40"	77 4'25"	320.1	6	75-79	5
10.	UMAR	ITARSI - JABALPUR, C.R.	930	23 08'36"	79 25'48"	226.2	4	69-73	5
11.	SAKATWAR	ITARSI - AMLA, C.R.	776	22 24'22"	77 49'10"	180.0	3	68-74	7
12.	LAKHORA	KHANDWA - AKOLA, S.C.R.	584	21 44'10"	76 27'15"	154.1	3	67-73	7
13.	HATEAR	BHUSAVAL - ITARSI, C.R.	732	22 34'15"	77 38'45"	115.2	7	58-64	7
14.	TYRIA	GONDIA - JABALPUR, S.E.	253	22 52'36"	79 50' 8"	102.0	2	66-74	0
15.	MACHHWASA	ITARSI - ALLAHABAD, C.R.	813	22 45'25"	78 21'56"	66.5	3	65-69	5
16.	OL-NADI	ITARSI - JABALPUR, C.R.	832	22 46'30"	78 32'30"	54.8	4	57-64,66	9
17.	KHARA	AKOLA - KHANDWA, S.C.R.	710	20 59'25"	77 2'20"	121.6	3	68-73	6
18.	KARELI	ITARSI - JABALPUR, C.R.	889	22 55'40"	79 3'12"	31.1	2	68-69	2

6.0 METHODOLOGY

6.1 COMPUTATION OF EXCESS RAINFALL

When the rainfall occurs over the catchment not all the rain contribute to the direct surface runoff. A part of the rainfall is abstracted as interception, evapotranspiration, surface depression storage and infiltration. The remainder of the rainfall termed as excess rainfall contributes to the direct surface runoff. Thus the computation of excess rainfall is required for the estimation of direct surface runoff by separating the hydrological abstractions from the rainfall hyetographs. Although number of techniques are available for the computation of excess rainfall but the ϕ-index method is one of the simple and most commonly used technique. Among the other techniques SCS curve number method is being widely used for the estimation of the excess rainfall particularly when the catchment is ungauged. In the present analysis the ϕ-index is used to estimate the excess rainfall hyetograph pattern. The volume of the excess rainfall for a given storm event is assumed to be known. It is computed as the volume of direct surface runoff hydrograph for a given event. The direct surface runoff hydrograph is computed by separating the baseflow from the observed hydrograph ordinates. Here the observed direct surface runoff is used only for the estimation of excess rainfall hyetograph and is not used further for the derivation of instantaneous unit hydrograph. However, the use of the observed direct surface runoff for the estimation of excess rainfall has to be avoided for the ungauged catchment as no runoff records would be available for such catchments. In such situations the values of ϕ-index can be estimated by analysing the rainfall-runoff records of flood events of the same period of the neighbouring catchments having similar hydro-meteorological characteristics. Alternatively, other methods such as SCS method may be applied to estimate the excess rainfall provided that the land use, soil type, treatment class, hydrologic condition and antecedent soil moisture condition are known for the estimation of runoff curve number.

6.2 PREPARATION OF TIME-AREA DIAGRAM

Time of travel between any two points in the stream , t ., is considered proportional to $L / \sqrt{S}$

or $$t = K L / \sqrt{S} \tag{1}$$

where:

t ⇒ time of travel
L ⇒ length of the stream. between the two points
S ⇒ slope of the stream between the two points
and K ⇒ proportionality constant.

Using eq.(1) we may get the time of travel between any point in the catchment on the river layout and the outlet of the catchment as:

$$K L / \sqrt{S_A} = K \sum_{i=1}^{NR} (L_i / \sqrt{S_i}) \tag{2}$$

where:

L $\Rightarrow$ the total length of the main stream

$L_1, L_2 \Rightarrow$ the lengths of each individual segments

$S_A \Rightarrow$ average slope of main stream

$S_1, S_2 \Rightarrow$ average slope of individual segment slopes.

NR $\Rightarrow$ no. of segments considered in the main stream.

Assuming some arbitrary value of K, eq.(2) may be used to calculate time of travel between any two points on the river layout in the catchment. Starting from the basin outlet the time of travel of various points over the catchment is thus progressively calculated.

All the values of the time of travels for different points are then denoted on the map at their respective locations. Curves of specified time of concentration called the "Isochrones" are then drawn through these points by making use of linear interpolation and consideration of elevation contour pattern and stream layout.

From this map having contours of equal time of travel the inter isochronal areas may be obtained by using planimeter etc.. The cumulative isochronal area with respect to the cumulative time of travel may thus be obtained. To eliminate the effect of assumed value of K, the each value of time of travel corresponding to cumulative isochronal areas is divided by the largest time of travel to express it in percent form. Thus, a non-dimensional relation between cumulative isochronal area and percent time of travel may be obtained. This may also be expressed in graphical form by plotting percent time of travel on x-axis and cumulative isochronal area on y-axis.

6.3 DERIVATION OF CLARK MODEL IUH AND D-HOUR UNIT HYDROGRAPH

The Clark model concept suggests that the IUH can be derived by routing the unit inflow in the form of time-area diagram, which is constructed from the isochronal map, through a single reservoir. For the derivation of IUH the Clark model uses two parameters, time of concentration (T_c) in hours, which is the base length of the time-area diagram, and storage coefficient (R), in hours, of a single linear reservoir in addition to the time-area diagram.

The governing equation of IUH using this model is given as :

$$u_i = C I_i + (1-C)\, u_{i-1} \tag{3}$$

where;

$u_i \Rightarrow$ ith ordinate of the IUH

C & (1-C) $\Rightarrow$ the routing coefficients.

and C $\Rightarrow \Delta t\ /\ (R+0.5\Delta t)$

$\Delta t \Rightarrow$ computational interval in hours

$I_i \Rightarrow$ the ith ordinate of the time-area diagram

A unit hydrograph of desired duration (D) may be derived using the following equation:

$$U_i = \frac{1}{n}(0.5u_{i-n}+u_{i-n}+u_{i-n+1}\ldots+u_{i-1}+0.5u_i) \tag{4}$$

where;

$U_i \Rightarrow$ ith ordinate of unit hydrograph of duration D-hour and at computational interval Δt hours

$n \Rightarrow$ no. of computational intervals in duration D hrs = $D/\Delta t$

$u_i \Rightarrow$ ith ordinate of the IUH

6.4 USE OF GEOMORPHOLOGICAL CHARACTERISTICS

Rodriquez-Iturbe and Valdes (1979) first introduced the concept of geomorphologic instantaneous unit hydrograph, which led to the renewal of research in hydrogeomorphology.

The expression derived by Rodriquez-Iturbe and Valdes (1979) yields full analytical, but complicated, expressions for the instantaneous unit hydrograph. Rodriquez-Iturbe and Valdes (1979) suggested that it is adequate to assume a triangular instantaneous unit hydrograph and only specify the expressions for the time to peak and peak value of the IUH. These expressions are obtained by regression of the peak as well as time to peak of IUH, derived from the analytic solutions for a wide range of parameters with that of the geomorphologic characteristics and flow velocities.

The expressions are given as:

$$q_p = 1.31\ R_L^{0.43}\ V / L_\Omega \tag{5}$$

$$t_p = 0.44\ (L_\Omega / V)\ (R_B / R_A)^{0.55}\ (R_L)^{-0.38} \tag{6}$$

where;

$L_\Omega \Rightarrow$ the length in kilometers of the main stream

$V \Rightarrow$ the expected peak velocity, in m/sec.

$q_p \Rightarrow$ the peak flow, in units of inverse hours

$t_p \Rightarrow$ the time to peak, in hours

$R_B, R_L, R_A \Rightarrow$ the bifurcation, length and area ratios given by the Horton's laws of stream numbers, lengths and areas respectively.

Empirical results indicate that for natural basins the values for R_B normally ranges from 3 to 5, for R_L from 1.5 to 3.5 and for R_A from 3 to 6 [Smart (1972)].

On multiplying eq. (5) and (6) we get a non-dimensional term q_p x t_p as under.

$$q_{pg} \times t_{pg} = 0.5764\ (R_B / R_A)^{0.55}\ (R_L)^{0.05} \tag{7}$$

This term is not dependent upon the velocity and thereby on the storm characteristics and hence is a function of only the catchment characteristics. This is also apparent from the expression given above.

6.5 DEVELOPMENT OF RELATIONSHIP BETWEEN THE INTENSITY OF THE EXCESS RAINFALL AND THE VELOCITY

For the dynamic parameter velocity (V) , Rodriquez et. al. (1979) in their studies assumed that the flow velocity at any given moment during the storm can be taken as constant throughout the basin. The characteristic velocity for the basin as a whole changes throughout as the storm progresses. For the derivation of GIUH, this can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin. However, for ungauged catchments the peak discharge is not known and so this criteria for estimation of velocity cannot be applied. In such a situation the velocity may be estimated using the relationship developed between the velocity and the excess rainfall. Two approaches for developing this relationship are presented here under.

APPROACH I :

This approach may be utilized when the geometric properties of the gauging section is known and the Manning's roughness coefficient can be assumed with an adequate degree of accuracy.

The steps involved in this approach are as below.

(i) Compute cross sectional area (A), Wetted Perimeter (P)and hydraulic radius (R) on the basis of X-sectional details corresponding to different depths.
(ii) Assume the frictional slope to be equal to the bed slope of the channel.
(iii) Choose an appropriate value of Manning's roughness coefficient (n) from the values given in literature (Chow 1964) for different surface conditions of the channel.
(iv) Compute the discharge (Q) using the Manning's formulae corresponding to each depth.
(v) Plot depth v/s discharge and depth v/s area curves.
(vi) Compute the equilibrium discharge (Q_e) corresponding to an excess rainfall intensity (i in mm/hr) using the relation :

$$Q_e = 0.2778 \, i \, A_c \tag{8}$$

where ; $A_c \Rightarrow$ catchment area in Sq. Kms..
(vii) Compute the depth corresponding to the equilibrium discharge (Q_e) using the depth v/s discharge curve.
(viii) Compute the area corresponding to the depth computed at step (vii) using the depth v/s area curve.
(ix) Compute the velocity V by dividing the discharge (Q_e) by the area computed at step (viii).
(x) Repeat steps (vi) to (ix) to find velocity with respect to different intensities (e.g., 1, 2, 3 mm/hr. etc.) of rainfall excess.

(xi) Develop the relationship between velocity and rainfall excess intensity obtained at step (x) in the form : $v = a\,i^b$, using method of least square.

APPROACH II :

This approach is based on the assumption that the value of the Manning's roughness coefficient is not available but the velocities corresponding to discharges passing through the gauging section at different depths of water flow are known from the observations. The steps involved in this approach are given below.

(i) For different depths of flow the discharge and the corresponding velocities are known by observation.
(ii) Let these velocities and discharges be the equilibrium velocities V_e and the corresponding equilibrium discharges Q_e .
(iii) For these Q_e find the corresponding intensities i of excess rainfall from the expression:

$$i = Q_e / (0.2778\, A_c) \tag{9}$$

(iv) From the pairs of such V_e and i develop the relationship between the equilibrium velocity and the excess rainfall intensity in the form : $v_e = a\,i^b$, using method of least square.

It is to be noted here that this approach though requires the information of discharges and velocities at the gauging site does not necessarily mean that it can be applied for the gauged catchments only. For the ungauged catchments too, this information may be easily obtained by gauging the stream intermittently for all ranges of depth of flow. This type of information may be gathered without incurring much cost and effort.

6.6 DERIVATION OF UNIT HYDROGRAPH USING THE NEW "GIUH BASED CLARK MODEL" APPROACH

A new approach has been developed at the National Institute of Hydrology (NIH, 1993) for the estimation of the parameters of the Clark model through use of geomorphological characteristics.

The step-by step explanation of the procedure to derive unit hydrograph for a specific duration using this approach is given here under :

(i) Excess rainfall hyetograph is computed either by uniform loss rate procedure or by SCS curve number method or by any other suitable method.
(ii) For a given storm the estimate of the peak velocity V using the highest rainfall excess is made by using the relationship between velocity and intensity of rainfall excess (as developed in section 6.5).
(iii) Compute the time of concentration (T_c) using the equation :

$$T_c = 0.2778\ L/V \tag{10}$$

where; L ⇒ length of the main channel and V ⇒ the peak velocity in m/sec.

(iv) Considering this T_c as the largest time of travel find the ordinates of cumulative isochronal areas corresponding to integral multiples of computational time interval with the help of non-dimensional relation between cumulative isochronal area and the percent time of travel. This describes the ordinates of the time-area diagram at each computational time interval.

(v) Compute the peak discharge (Q_{pg}) and of IUH given by equations (5).

(vi) Assume two trial values of the storage coefficient of GIUH based Clark model as R_1 and R_2 . Compute the ordinates of two instantaneous unit hydrographs by Clark model using time of concentration T_c as obtained in step (iii) and two storage coefficients R_1 and R_2 respectively with the help of equation (3). Compute the IUH ordinates at a very small time interval say 0.1 or 0.05 hrs. so that a better estimate of peak value may be obtained.

(vii) Find out the peak discharges Q_{pc1} and Q_{pc2} of the instantaneous unit hydrographs obtained for Clark model for the storage coefficients R_1 and R_2 respectively at step (v).

(viii) Find out the value of objective function , using the relation :

$$FCN1 = (Q_{pg} - Q_{pc1})^2 \tag{11}$$

$$FCN2 = (Q_{pg} - Q_{pc2})^2 \tag{12}$$

(ix) Compute the first numerical derivative FPN of the objective function FCN with respect to parameter R as :

$$FPN = \frac{FCN1 - FCN2}{R_1 - R_2} \tag{13}$$

(x) Compute the next trial value of R using the following governing equations of Newton-Raphson's method :

$$\Delta R = \frac{FCN1}{FPN} \tag{14}$$

and

$$R_{NEW} = R_1 + \Delta R \tag{15}$$

(xi) For the next trial consider $R_1 = R_2$ and $R_2 = R_{NEW}$ and repeat steps (v) and (ix) till one of the following criteria of convergence is achieved.

(a) FCN2 = 0.000001
(b) No. of trials exceeds 200
(c) ABS(ΔR)/R_1 = 0.001

(xii) The final value of storage coefficient (R_2) obtained as above is the required value of the parameter R corresponding to the value of time of concentration (T_c) for the Clark model.

(xiii) Compute the instantaneous unit hydrograph (IUH) using the GIUH based Clark Model with the help of final values of storage coefficient (R), Time of concentration (T_c) as obtained in the step (xi) and time-area diagram.

(xiv) Compute the D-hour unit hydrograph (UH) using the relationship between IUH and UH of D-hour as given by equation (4).

6.7 COMPUTATION OF DIRECT SURFACE RUNOFF USING DERIVED UNIT HYDROGRAPH

The direct surface runoff for a storm event whose excess rainfall values are known at D-hour interval are computed using the convolution based on the D-hour unit hydrograph. The convoluted hydrograph ordinates are given as :

$$Q(t) = \Delta t \sum_{i}^{n} [\, U(D, t-(i-1)) \,] \, I_i \tag{16}$$

where,

U(D,t) ⇒ ordinate of D hour unit hydrograph at time t

I_i ⇒ rainfall intensity at ith interval (i.e., at time = $\Delta t \times i$)

n ⇒ no. of rainfall blocks

Δt ⇒ computational time interval

7.0 ANALYSIS

7.1 DATA PREPARATION

7.1.1 Preparation of time-area diagram:

For all the seventeen bridge catchments the time-area diagrams are prepared according to the methodology explained in section 6.2. The ordinates of the time-area diagrams of these catchments are given in Table 7.1 assuming some arbitrary value of constant of proportionality K. Also, these values are then non-dimensionalised by dividing each time of travel by the largest time of travel for the respective catchments. The time of travel in percent and the cumulative isochronal areas are thus calculated and are also tabulated in Table 7.1

7.1.2 Computation of Excess Rainfall hyetograph.

Since the rainfall-runoff records of these catchments could not be obtained the analysis is restricted to comparing the synthetic unit hydrograph for the catchment with the computed unit hydrographs corresponding to some hypothetical events. These events correspond to various magnitudes of velocities which are expected to be generated due to different storm intensities. However, the simulation of actual flood events for three catchments viz. Br. No. 249, 930 and 253 has been illustrated in Part I of the study (NIH, 1995).

7.1.3 Development of relationship between velocity and intensity of the excess rainfall

Since the rainfall-runoff data could not be obtained the relationship between velocity and intensity of rainfall excess could not be established. Instead, the model could be run for obtaining the unit hydrographs corresponding to different velocities fixed arbitrarily. For any catchment therefore, the instantaneous unit hydrograph and thereby 1 hour unit hydrographs corresponding to velocities of 1.0, 2.0, 3.0, 4.0, 5.0, 6.0 and 7.0 m/sec. are worked out and compared with the 1 hour unit hydrograph recommended on the basis of synthetic regional unit hydrograph approach for the respective catchment.

7.1.4 Estimation of geomorphological characteristics

The topographical maps for seventeen catchments are prepared. For each catchment number of streams, average lengths and average areas for each stream order is found out manually from the topographic maps. These are then plotted against the order of the stream as shown in Fig. 7.1.1 to 7.1.17. Bifurcation, length and area ratios are calculated as the slope of the best fit lines through these plotted points given by the Horton's laws of stream numbers, lengths and areas respectively. The summary of this evaluation of geomorphological characteristics for all the fourteen catchments is given in Table 7.2.

7.2 MODEL APPLICATION

The methodology given in section 6.6 is applied for the seven hypothetical events

Table 7.1 : Time of concentration and isochronal areas for different bridge catchments.

S.No.	Bridge No.	Time of Travel (units)	Isochronal Area (sq.km.)	Time of Travel (%)	Cumulative Isochronal Area (sq.km.)
(1)	(2)	(3)	(4)	(5)	(6)
1.	644	0 - 10	49.90	18.55	49.90
		10 - 20	53.10	37.03	103.00
		20 - 30	82.50	55.55	185.50
		30 - 40	188.40	74.07	373.90
		40 - 45	106.30	83.33	480.20
		45 - 50	269.00	92.59	749.20
		50 - 54	227.20	100.00	976.40
2.	803	0 - 5	30.20	7.14	30.20
		5 - 10	24.60	14.28	54.80
		10 - 15	97.00	21.42	151.80
		15 - 20	79.90	28.56	231.70
		20 - 25	80.60	35.71	312.30
		25 - 30	31.90	42.85	344.20
		30 - 35	126.50	50.00	470.70
		35 - 40	128.80	57.14	599.50
		40 - 45	76.80	64.28	676.30
		45 - 50	70.70	71.42	747.00
		50 - 55	68.30	78.57	815.30
		55 - 60	52.20	85.71	867.50
		60 - 65	49.60	92.85	917.10
		65 - 70	57.90	100.00	975.00
3.	578	0 - 15	18.00	11.11	18.00
		15 - 30	23.10	22.22	41.10
		30 - 45	49.55	33.33	90.65
		45 - 60	129.00	44.44	219.65
		60 - 75	69.50	55.55	289.15
		75 - 90	89.10	66.66	378.25
		90 - 105	95.25	77.77	473.50
		105 - 120	104.10	88.88	577.60
		120 - 135	70.40	100.00	648.00
4.	625	0 - 25	63.50	20.00	63.50
		25 - 50	149.20	40.00	212.70
		50 - 75	124.80	60.00	337.50
		75 - 90	99.40	72.00	436.90
		90 - 100	20.50	80.00	457.40
		100 - 110	43.00	88.00	500.40
		110 - 120	53.30	96.00	553.70
		120 - 125	12.70	100.00	566.40

Table 7.1 Contd...

S.No.	Bridge No.	Time of Travel (units)	Isochronal Area (sq.km.)	Time of Travel (%)	Cumulative Isochronal Area (sq.km.)
5.	249	0 - 10	11.70	11.36	11.70
		10 - 20	32.90	22.72	44.60
		20 - 30	64.00	34.09	108.60
		30 - 40	50.50	45.45	159.10
		40 - 50	46.30	56.81	205.40
		50 - 60	65.00	68.18	270.40
		60 - 70	115.00	79.54	385.40
		70 - 80	117.00	90.90	502.40
		80 - 88	24.00	100.00	526.40
6.	394/2	0 - 20	37.10	25.97	37.10
		20 - 30	64.00	38.96	101.10
		30 - 40	84.00	51.94	185.10
		40 - 50	34.15	64.93	219.25
		50 - 60	46.65	77.92	265.90
		60 - 70	54.20	90.90	320.10
		70 - 77	34.35	100.00	354.45
7.	897	0 - 20	21.00	28.57	21.00
		20 - 30	55.20	42.85	76.20
		30 - 40	128.00	57.14	204.20
		40 - 60	59.40	85.71	263.60
		60 - 70	67.30	100.00	330.90
8.	787	0 - 10	13.50	13.88	13.50
		10 - 15	31.10	20.83	44.60
		15 - 25	46.20	34.72	90.80
		25 - 30	25.15	41.66	115.95
		30 - 45	77.70	62.50	193.65
		45 - 60	79.10	83.33	272.75
		60 - 72	54.00	100.00	326.75
9.	930	0 - 60	21.90	16.00	21.90
		60 - 120	19.00	32.00	40.90
		120 - 180	39.70	48.00	80.60
		180 - 240	32.30	64.00	112.90
		240 - 300	59.70	80.00	172.60
		300 - 360	51.50	96.00	224.10
		360 - 375	3.70	100.00	227.80
10.	776	0 - 10	10.00	22.22	10.00
		10 - 15	20.80	33.33	30.80
		15 - 30	34.10	66.66	64.90
		30 - 35	36.50	77.77	101.40
		35 - 40	41.40	88.88	142.80
		40 - 45	39.50	100.00	182.30

Table 7.1 Contd...

(1)	(2)	(3)	(4)	(5)	(6)
11.	584	0 - 10	16.00	19.23	16.00
		10 - 20	15.60	38.46	31.60
		20 - 30	22.00	57.69	53.60
		30 - 40	47.00	76.92	100.60
		40 - 45	31.20	86.53	131.80
		45 - 52	21.60	100.00	153.40
12.	732	0 - 22	16.80	33.84	16.80
		22 - 30	11.90	46.15	28.70
		30 - 35	9.10	53.84	37.80
		35 - 40	8.45	61.53	46.25
		40 - 50	26.00	76.92	72.25
		50 - 60	32.40	92.30	104.65
		60 - 65	12.10	100.00	116.75
13.	253	0 - 60	15.80	20.00	15.80
		60 - 120	28.30	40.00	44.10
		120 - 180	16.20	60.00	60.30
		180 - 240	23.80	80.00	84.10
		240 - 300	16.30	120.00	100.40
14.	813	0 - 7	11.00	20.00	11.00
		7 - 16	12.70	45.71	23.70
		16 - 30	30.30	85.71	54.00
		30 - 35	13.30	100.00	67.30
15.	832	0 - 6	7.30	30.00	7.30
		6 - 12	17.80	60.00	25.10
		12 - 15	17.30	75.00	42.40
		15 - 17	9.50	85.00	51.90
		17 - 20	6.20	100.00	58.10
16.	710	0 - 5	3.00	7.14	3.00
		5 - 10	3.85	14.28	6.85
		10 - 15	3.30	21.42	10.15
		15 - 20	9.40	28.56	19.55
		20 - 25	8.30	35.71	27.85
		25 - 30	4.70	42.85	32.55
		30 - 35	5.00	50.00	37.55
		35 - 40	3.60	57.14	41.15
		40 - 45	3.90	64.28	45.05
		45 - 50	3.70	71.42	48.75
		50 - 55	25.60	78.57	74.35
		55 - 60	25.10	85.71	99.45
		60 - 65	12.10	92.85	111.55
		65 - 70	7.90	100.00	119.45
17.	889	0 - 7	6.70	20.00	6.70
		7 - 13	7.30	37.14	14.00
		13 - 20	5.50	57.14	19.50
		20 - 26	5.20	74.28	24.70
		26 - 32	4.20	91.42	28.90
		32 - 35	2.30	100.00	31.20

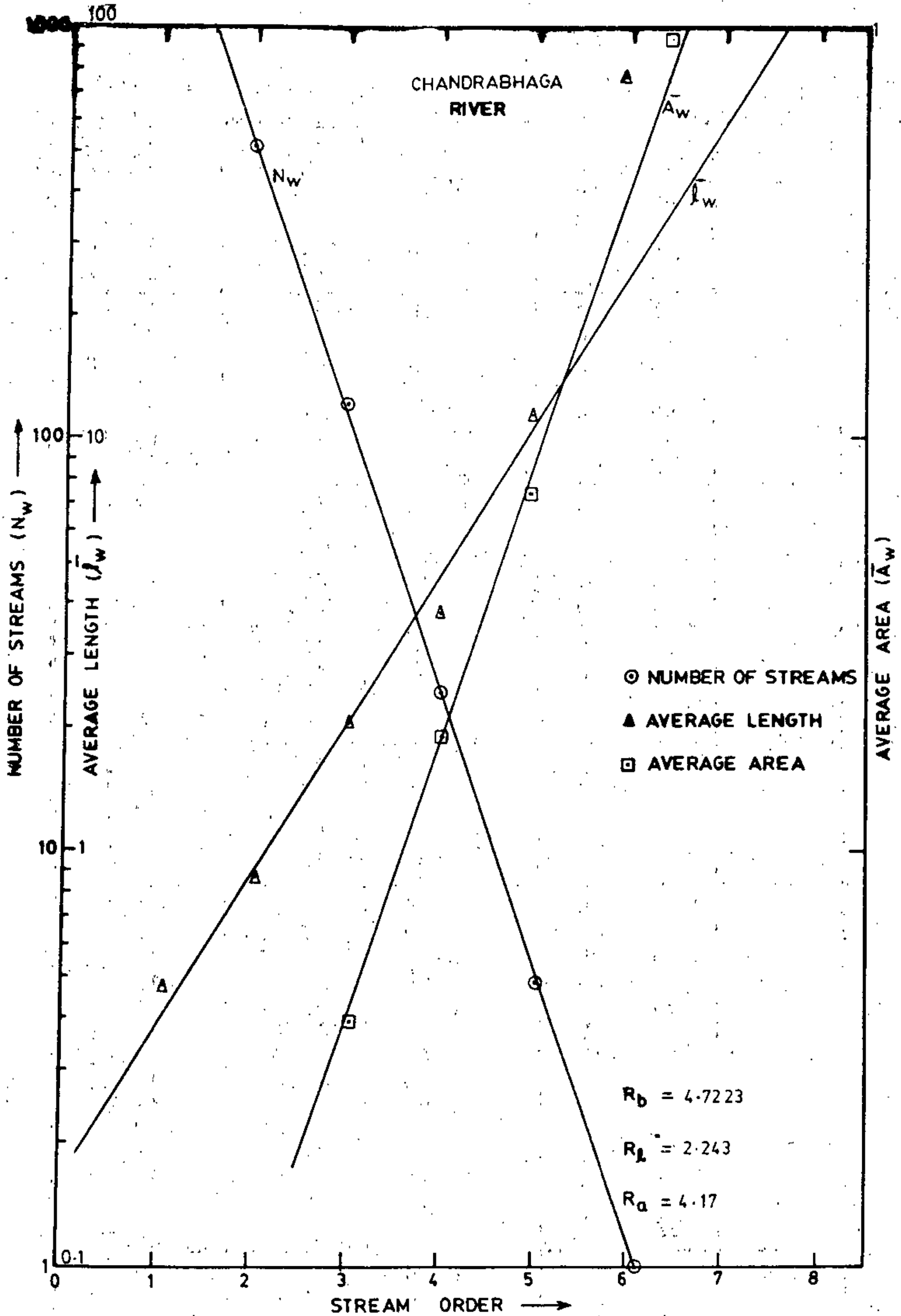

FIG. 7·1.1 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO. 644

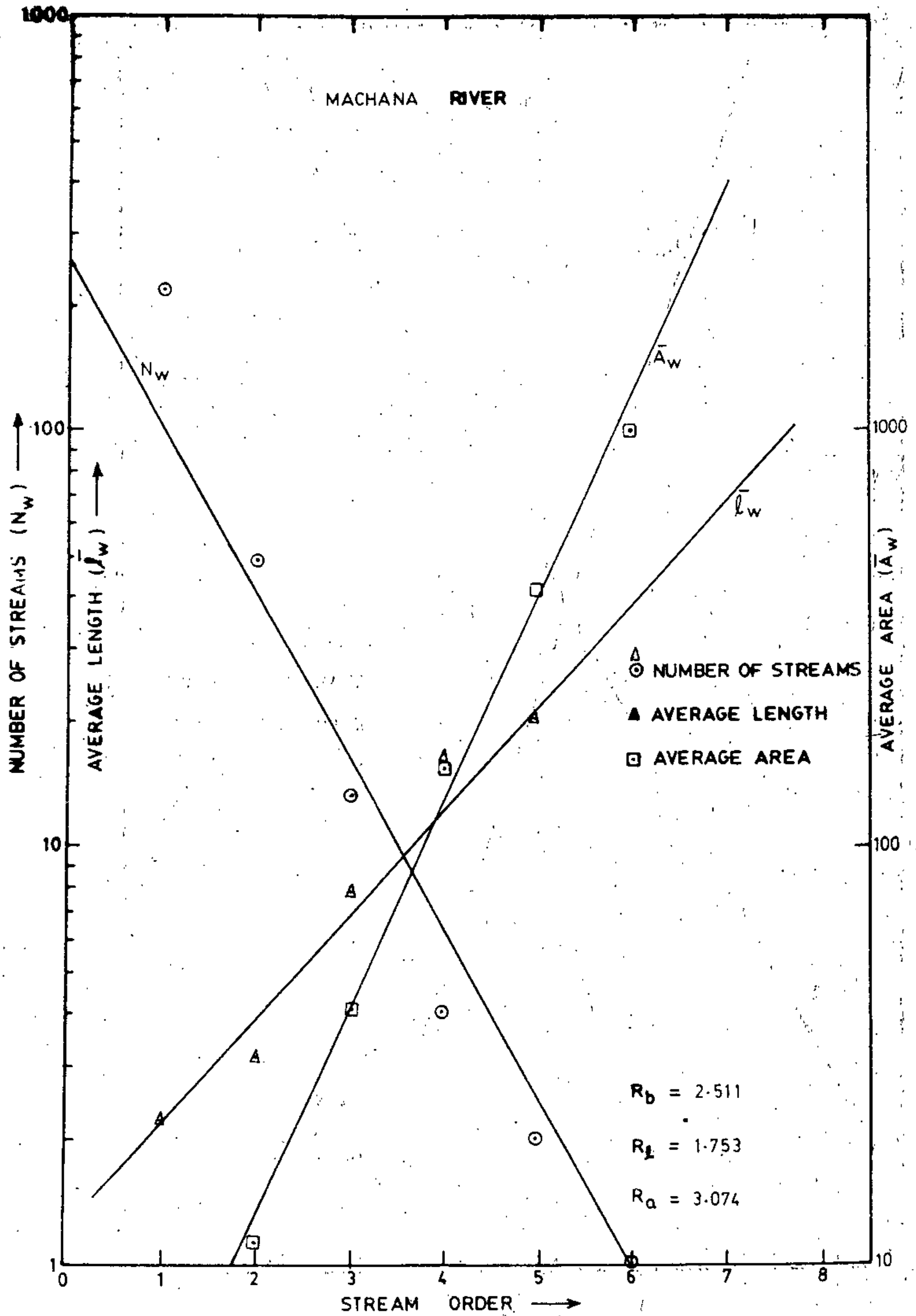

FIG. 7.1.2 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO. 804

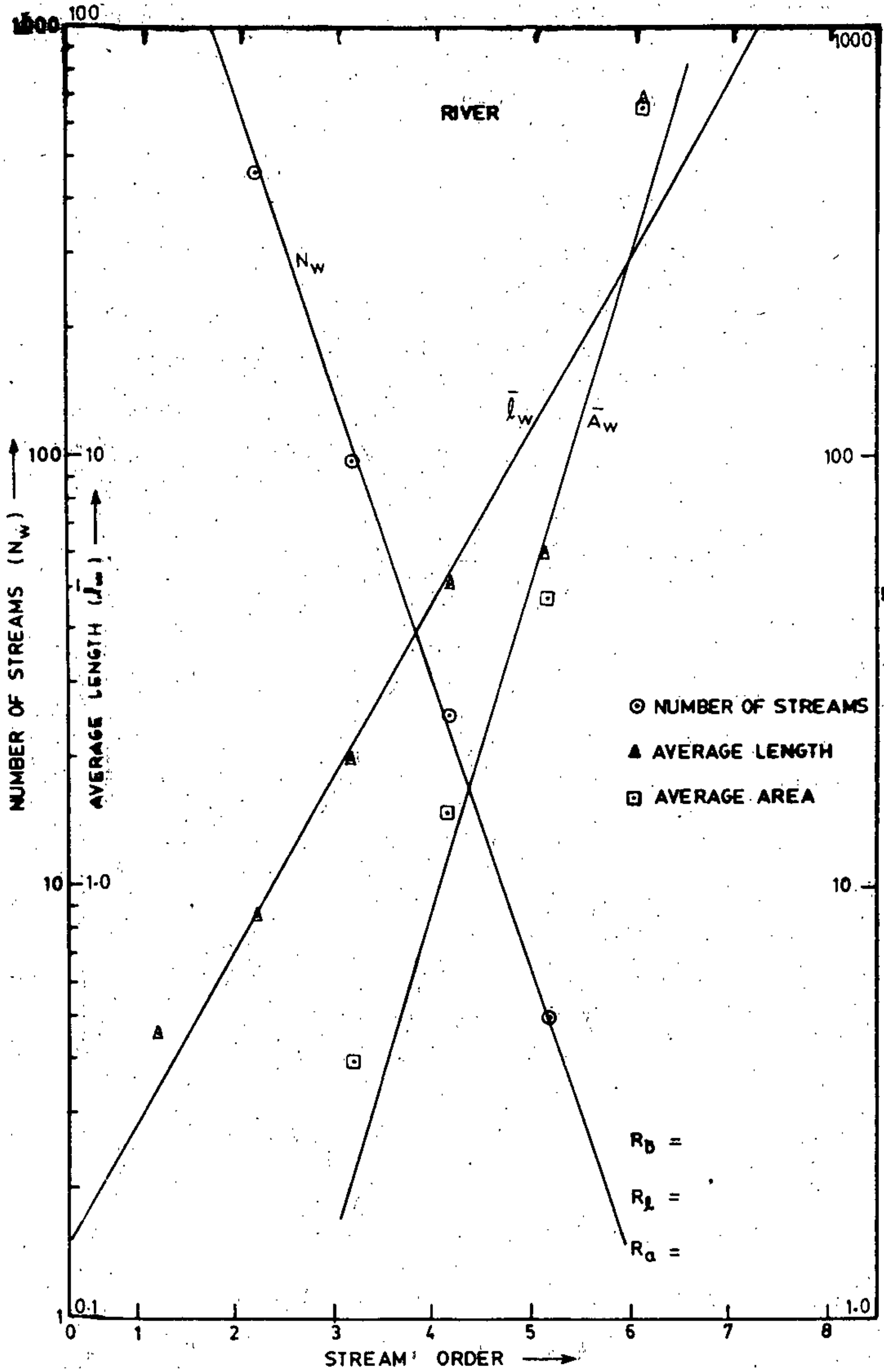

FIG.1.3: PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.578

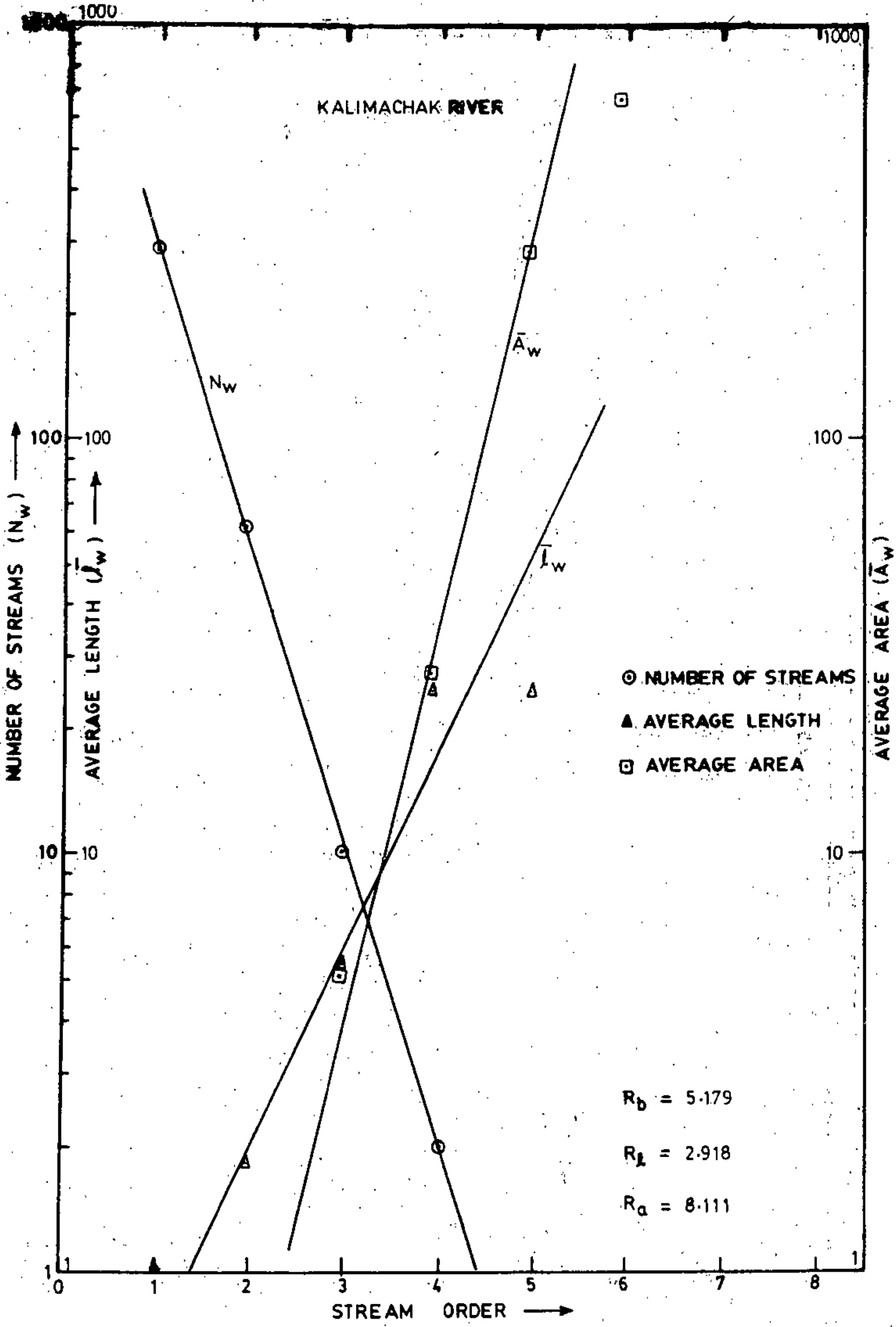

FIG. 7·1·4 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.625

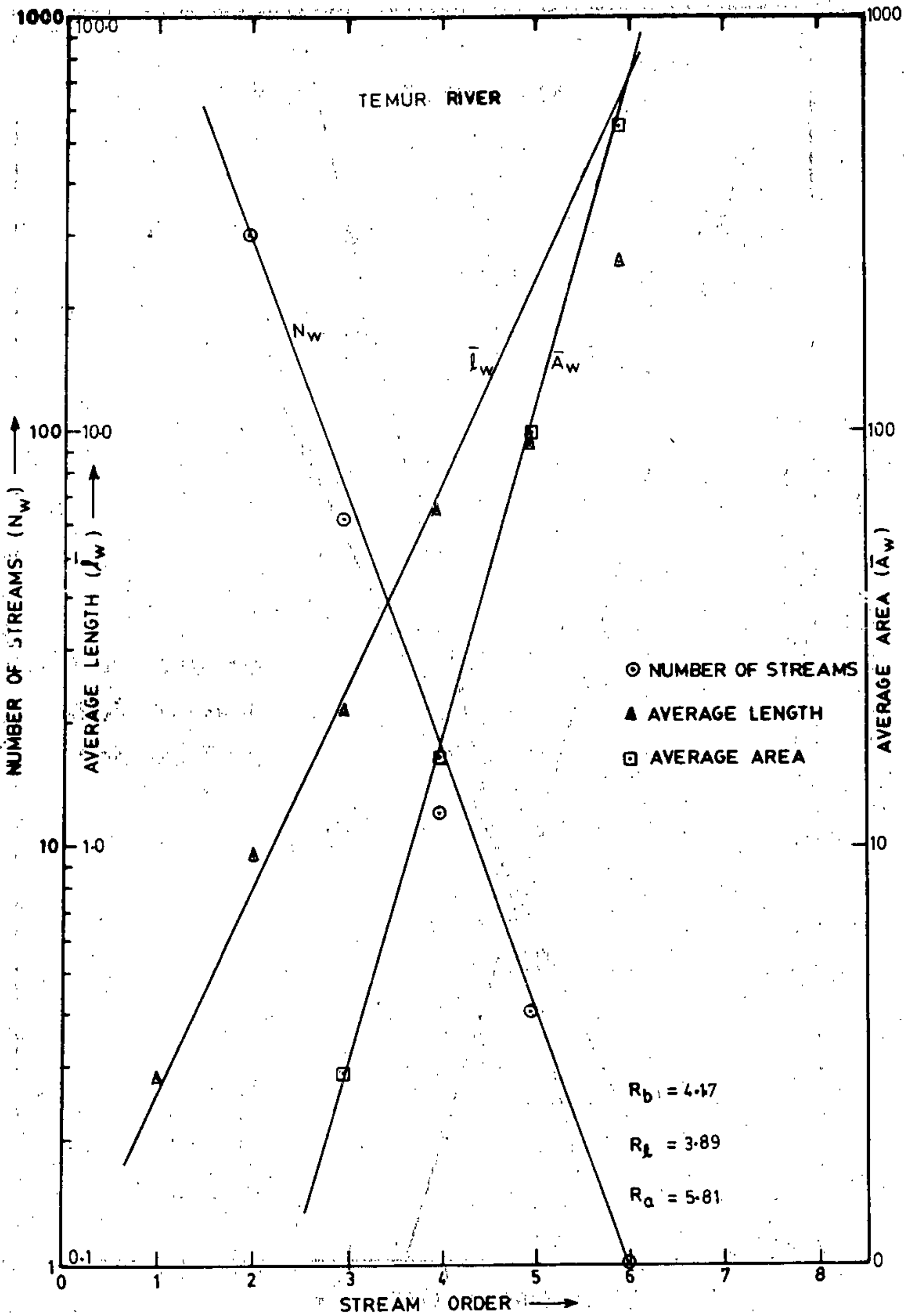

FIG. 7.1.5 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.249

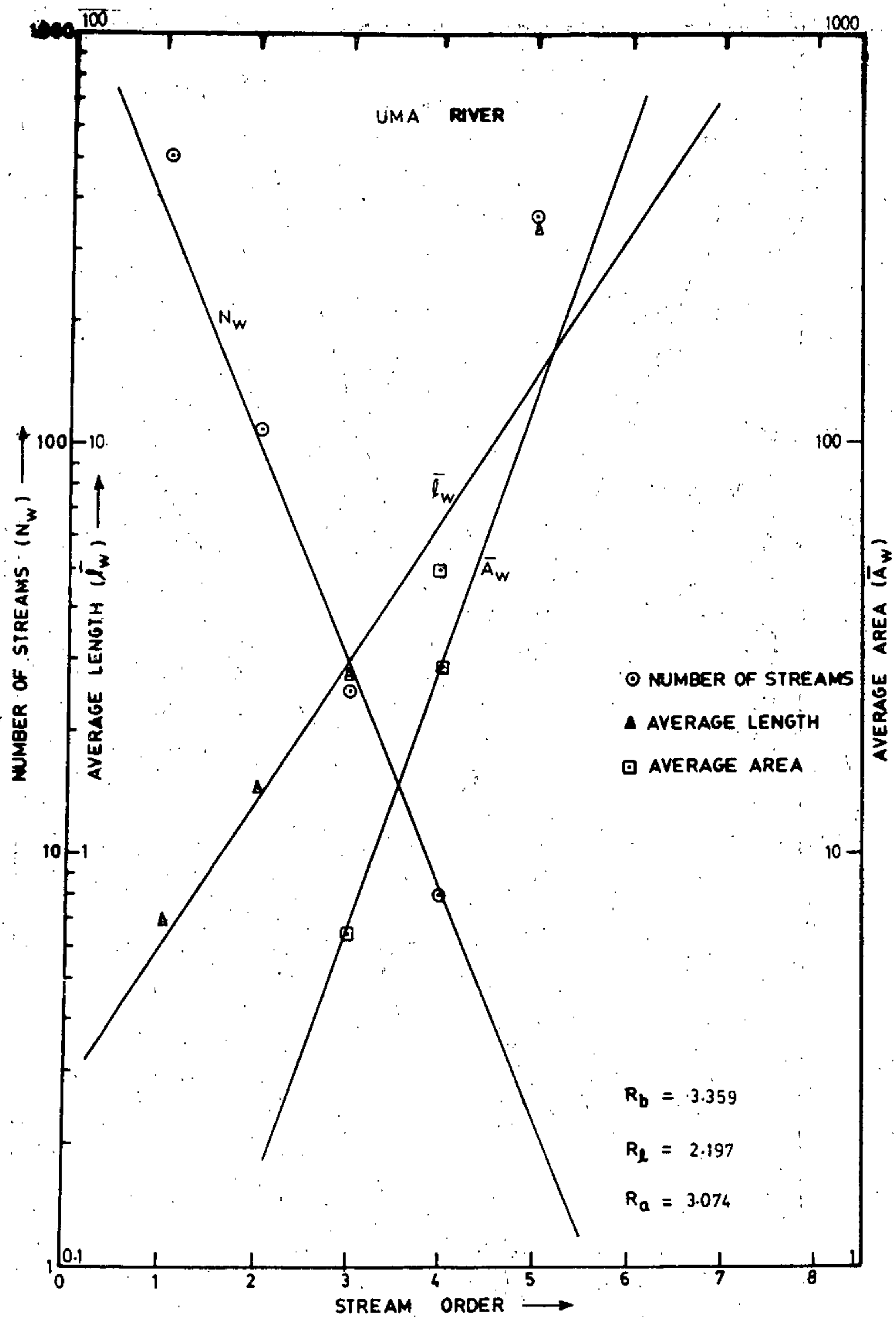

FIG. 7.1.6 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO. 394/2

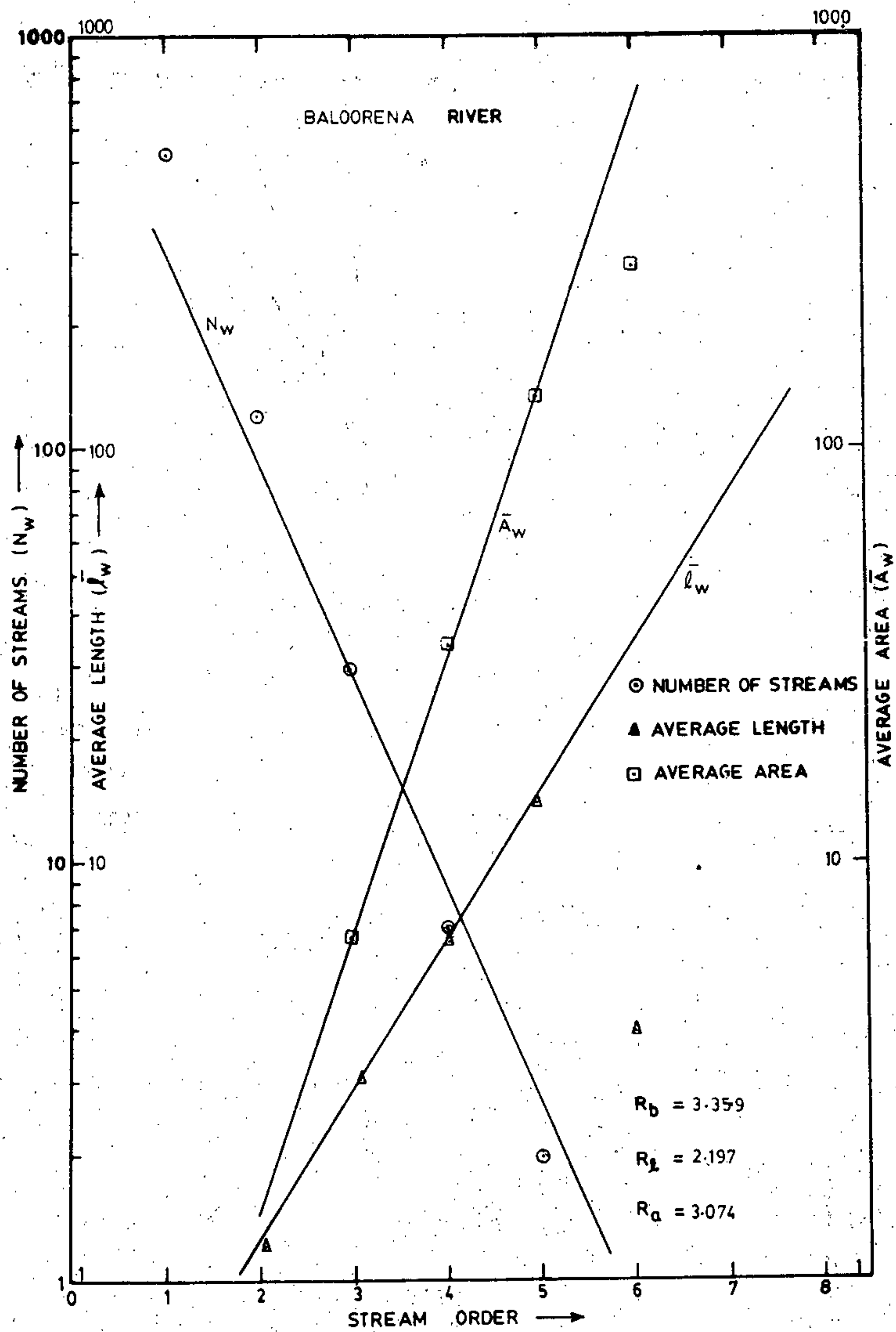

FIG. 7.1.7 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO. 897

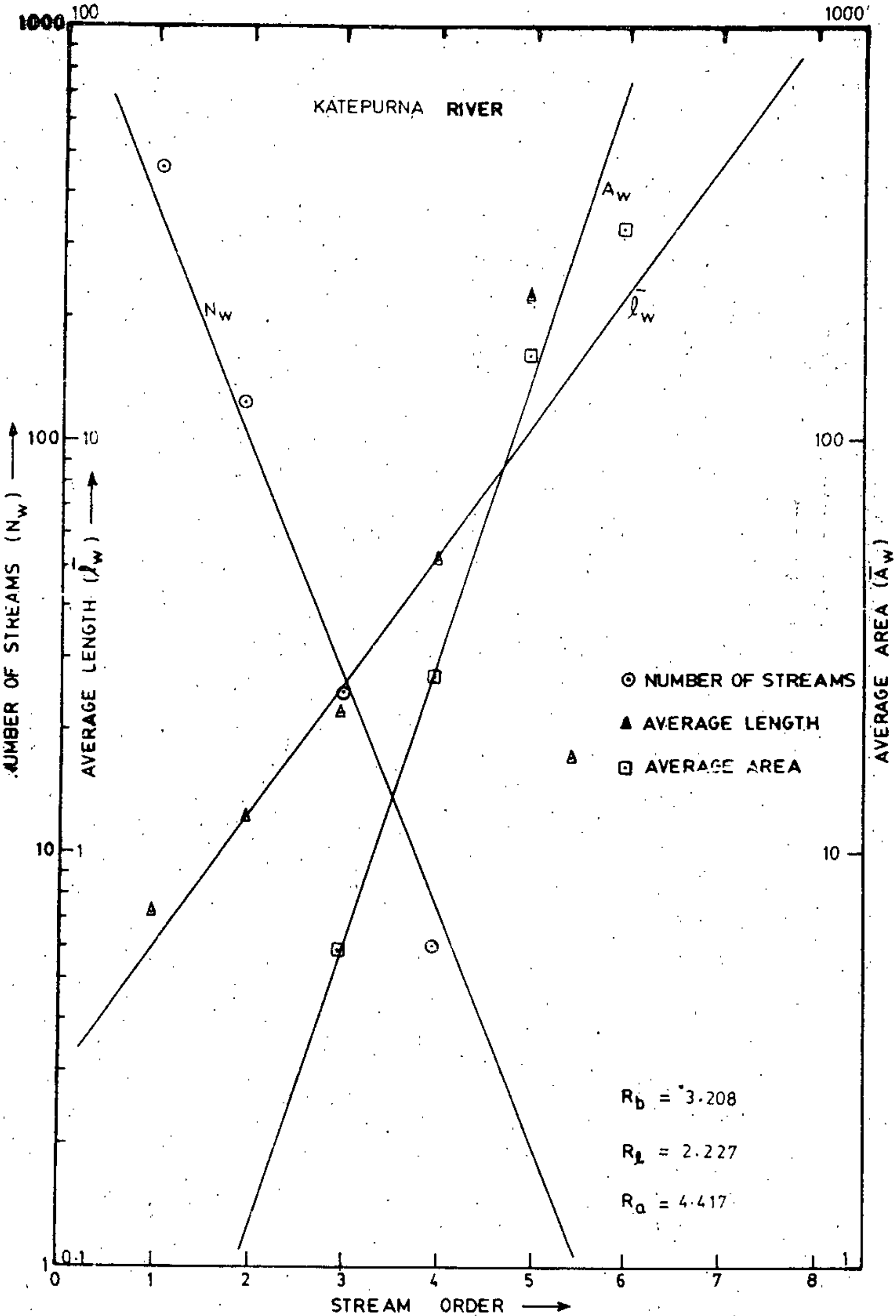

FIG. 7.1.8 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO. 787

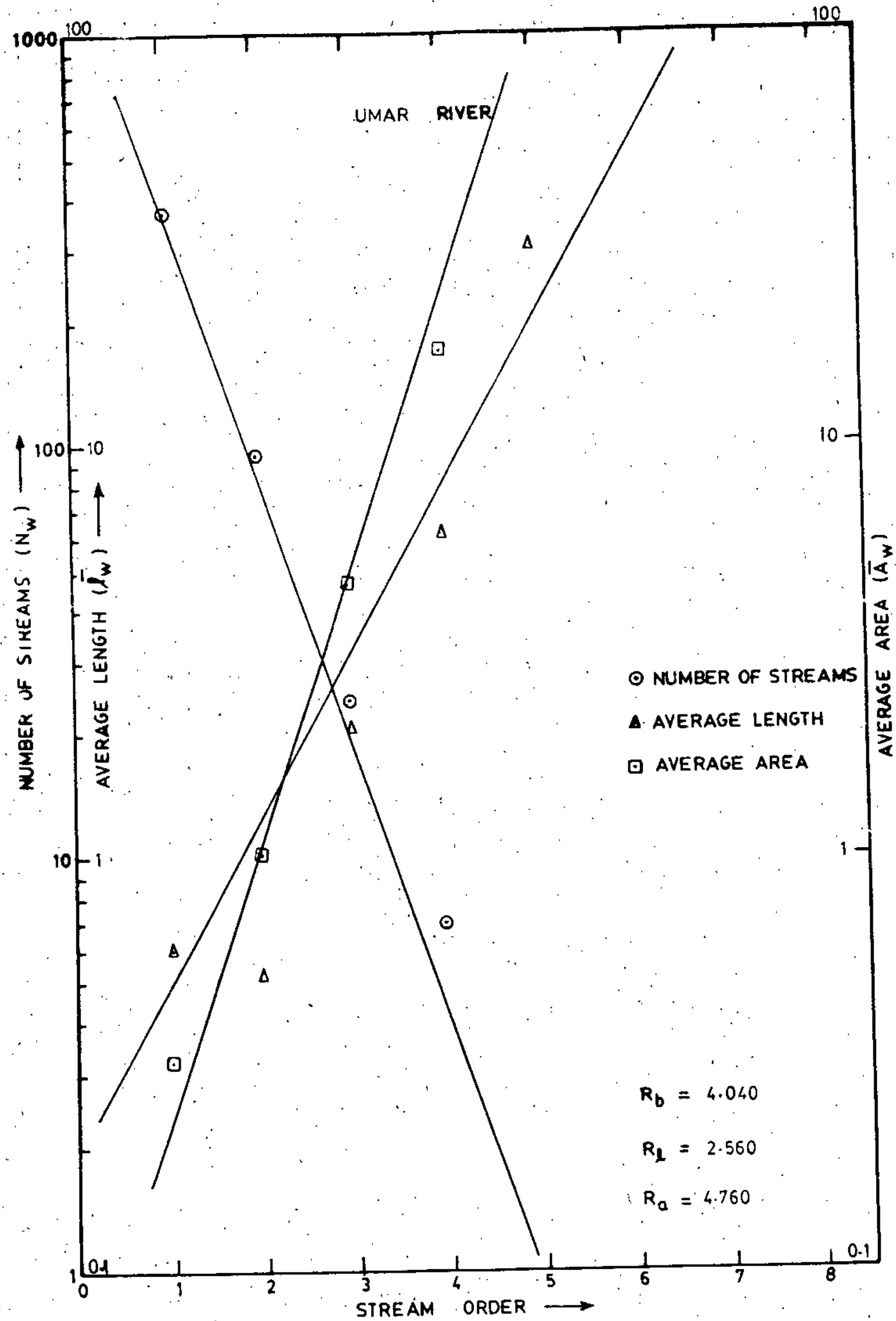

FIG. 7.1.9 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.930

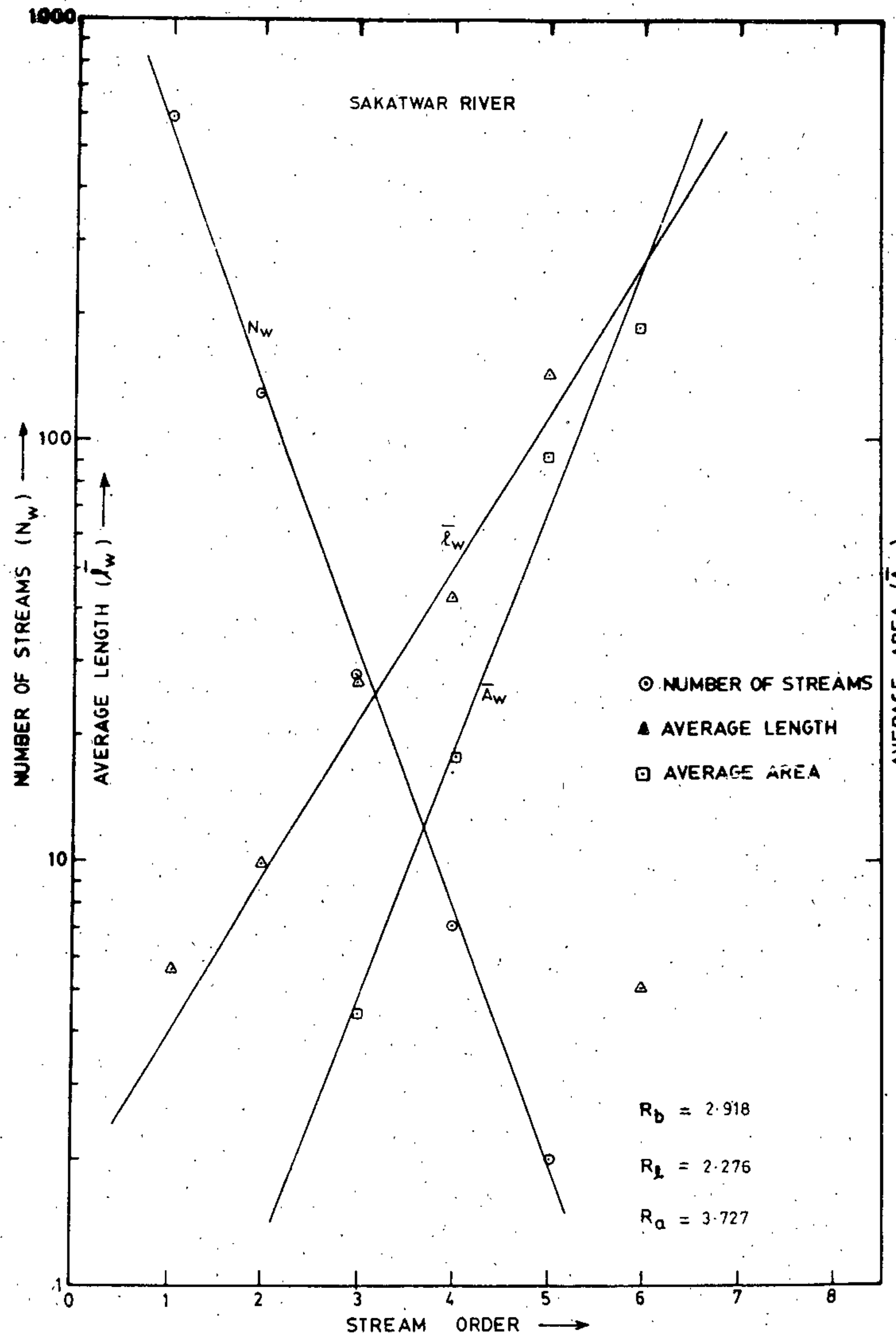

FIG. 7·1.10 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

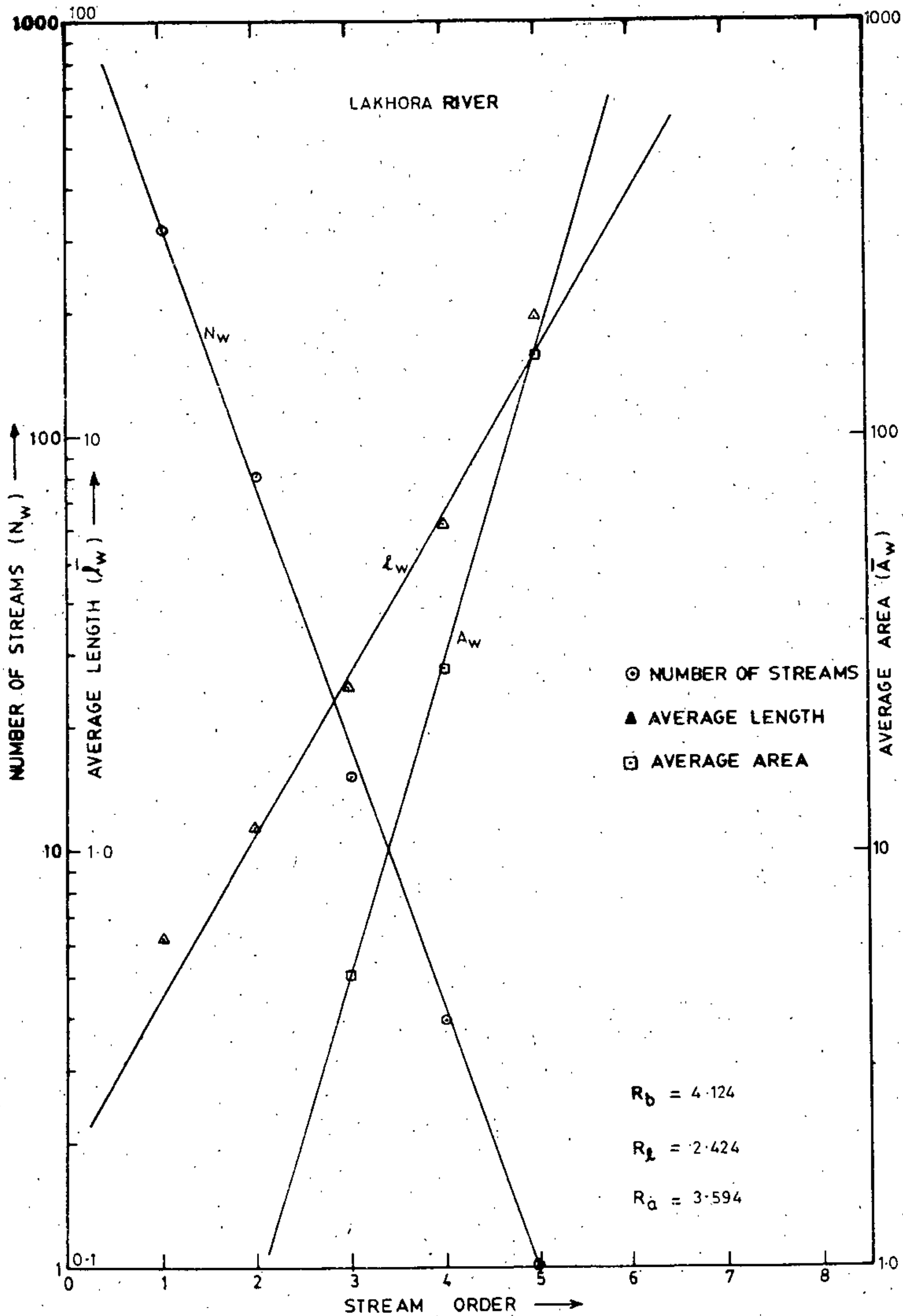

FIG. 7·1·11 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

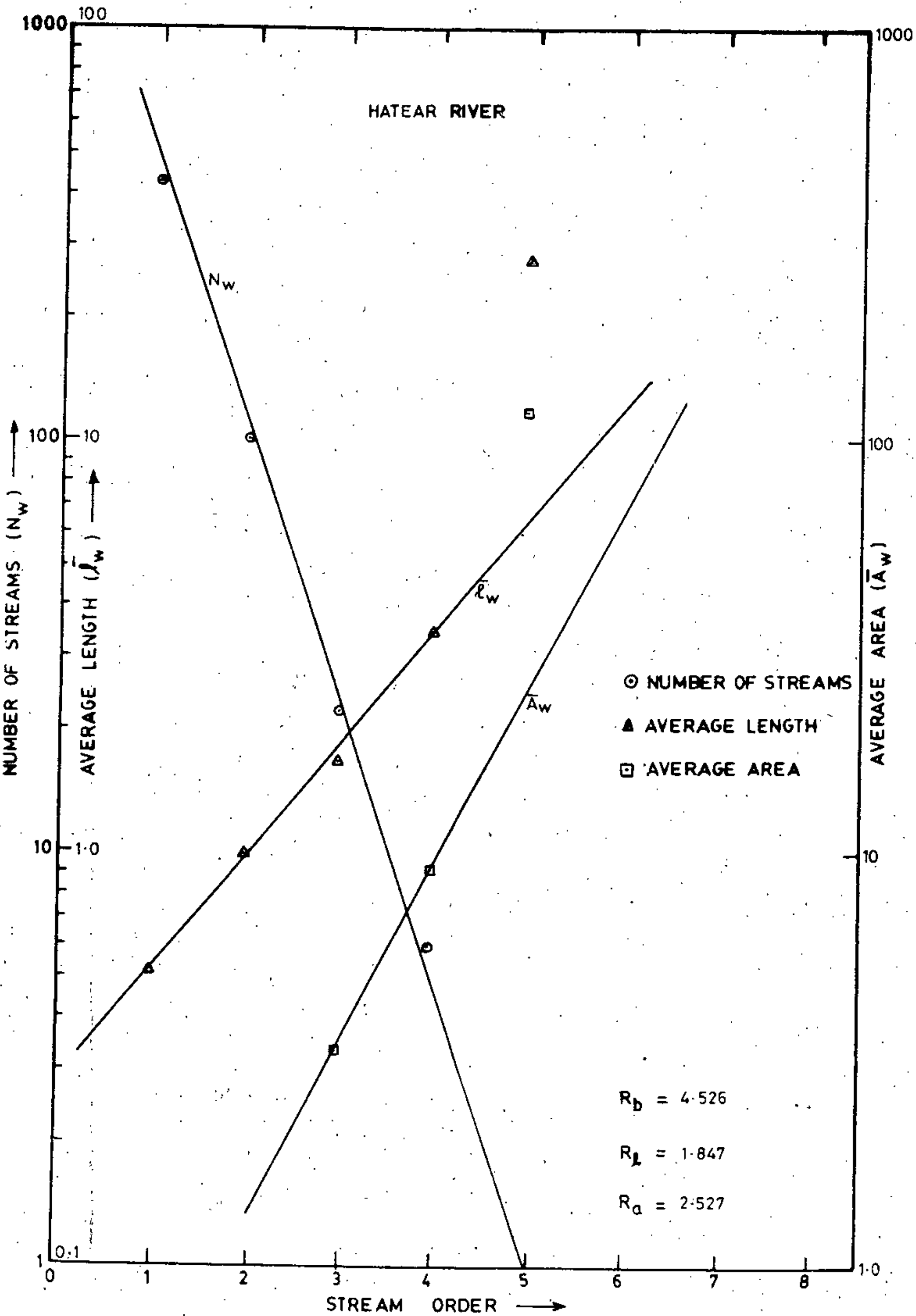

FIG. 7.1.12 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

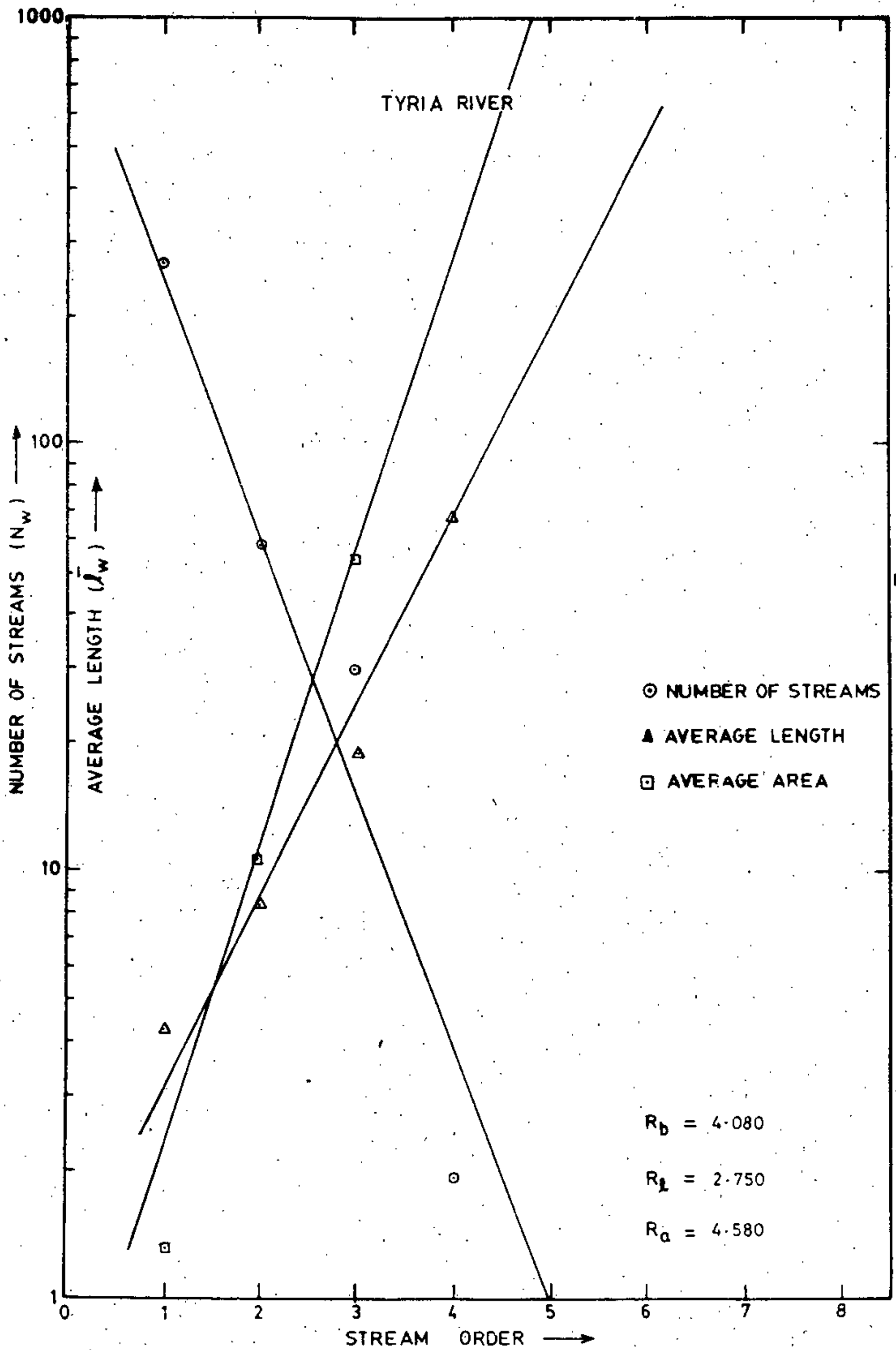

FIG. 7·1.13 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

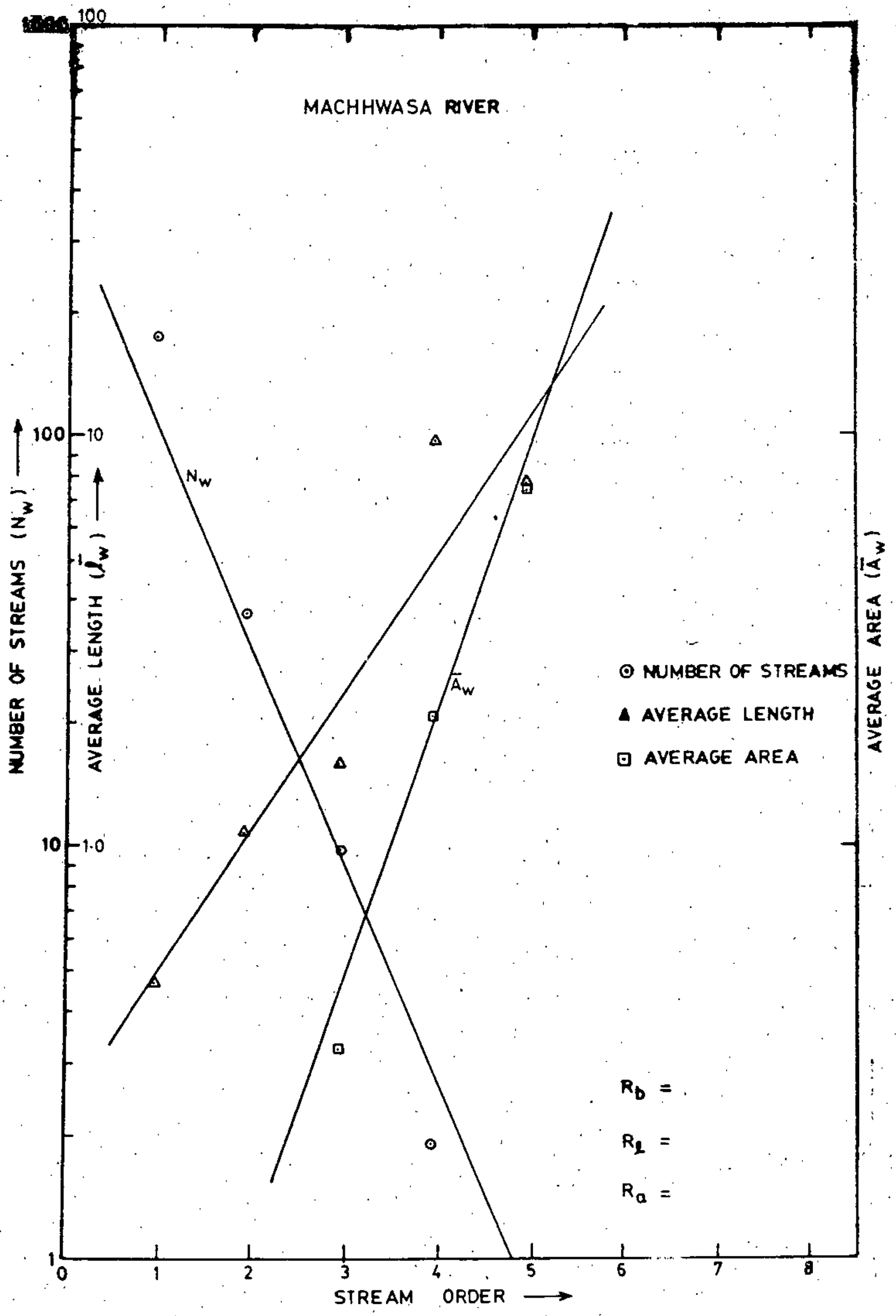

FIG. 7.1.14 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

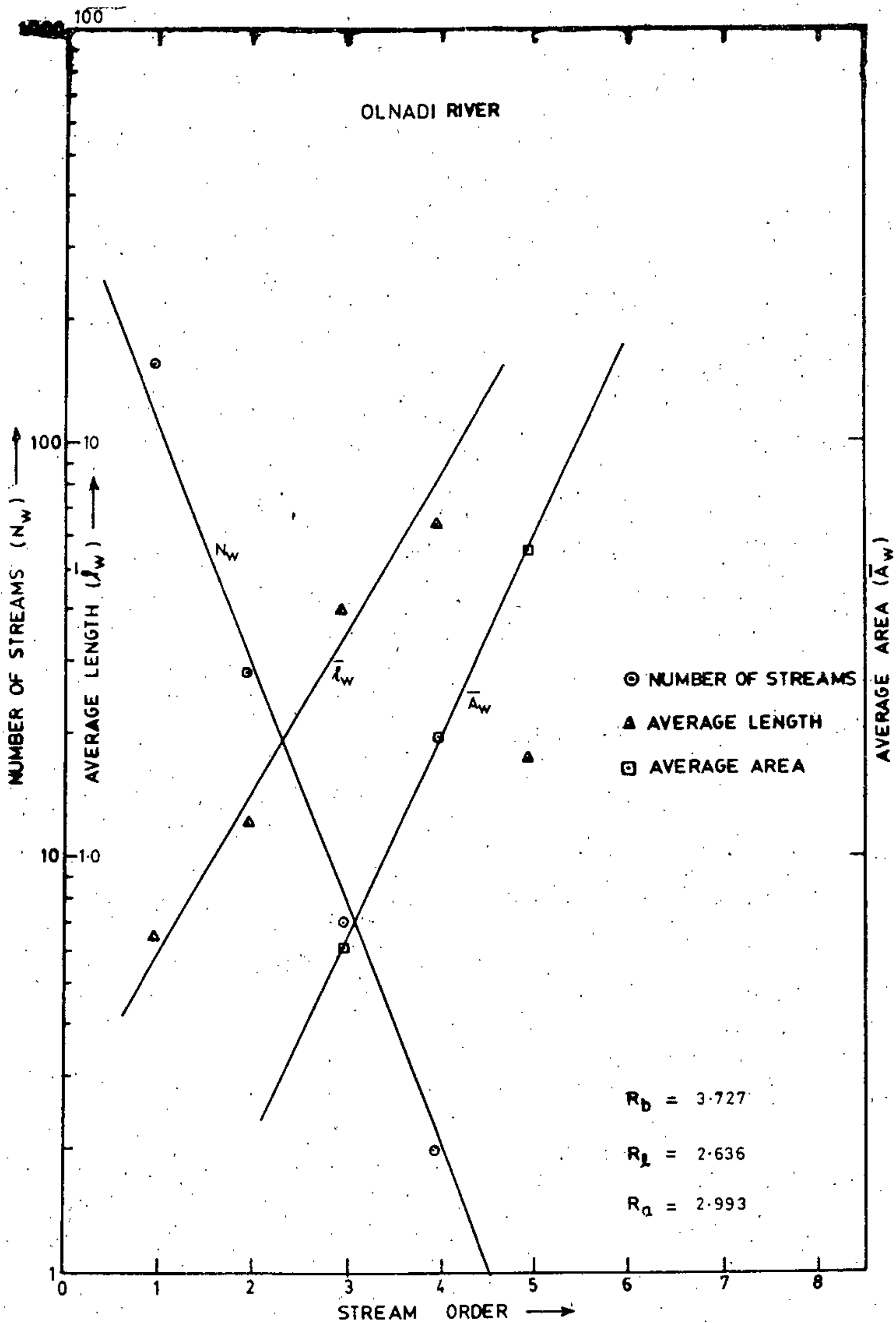

FIG. 7·1.15 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

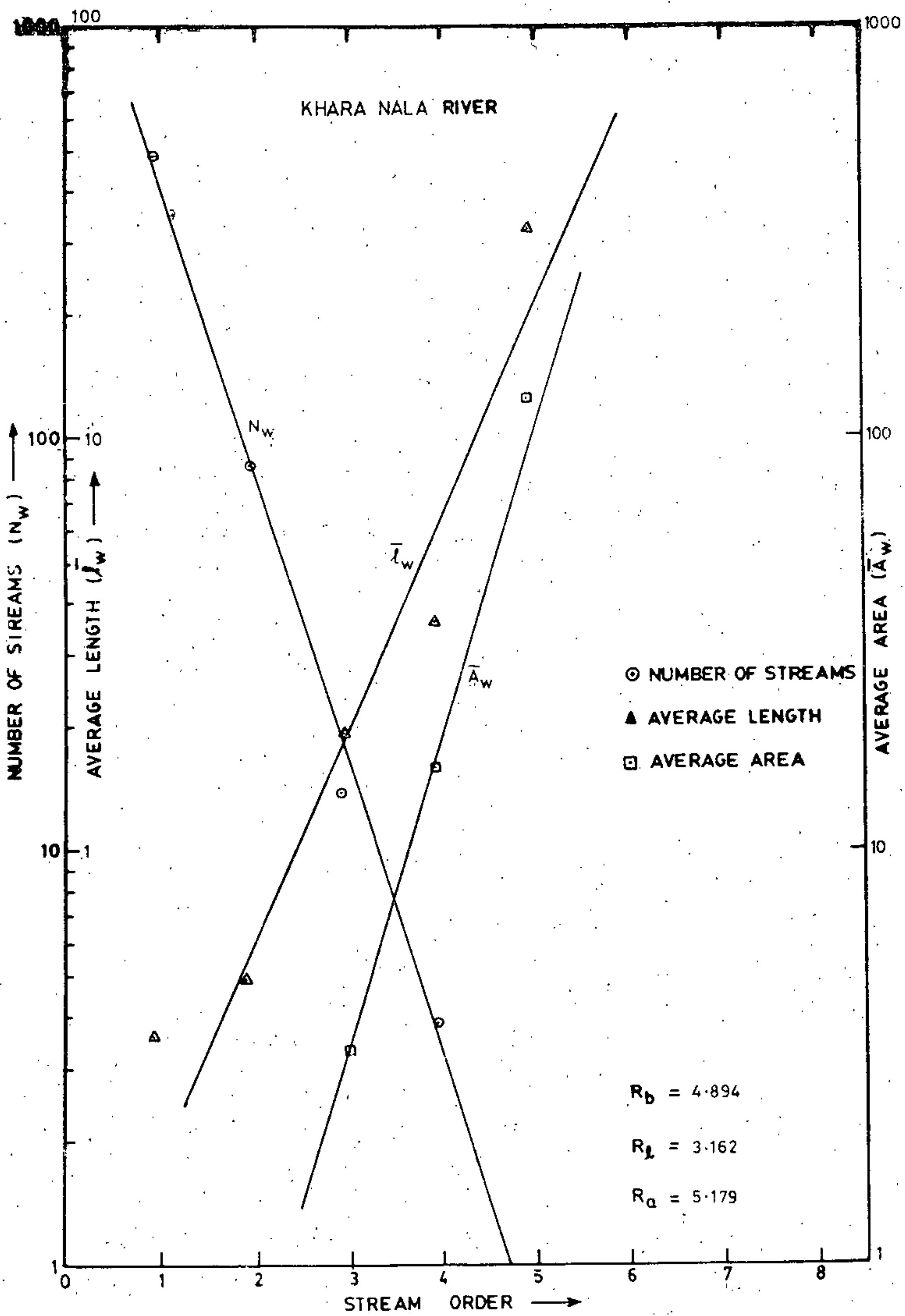

FIG. 7·1.16 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

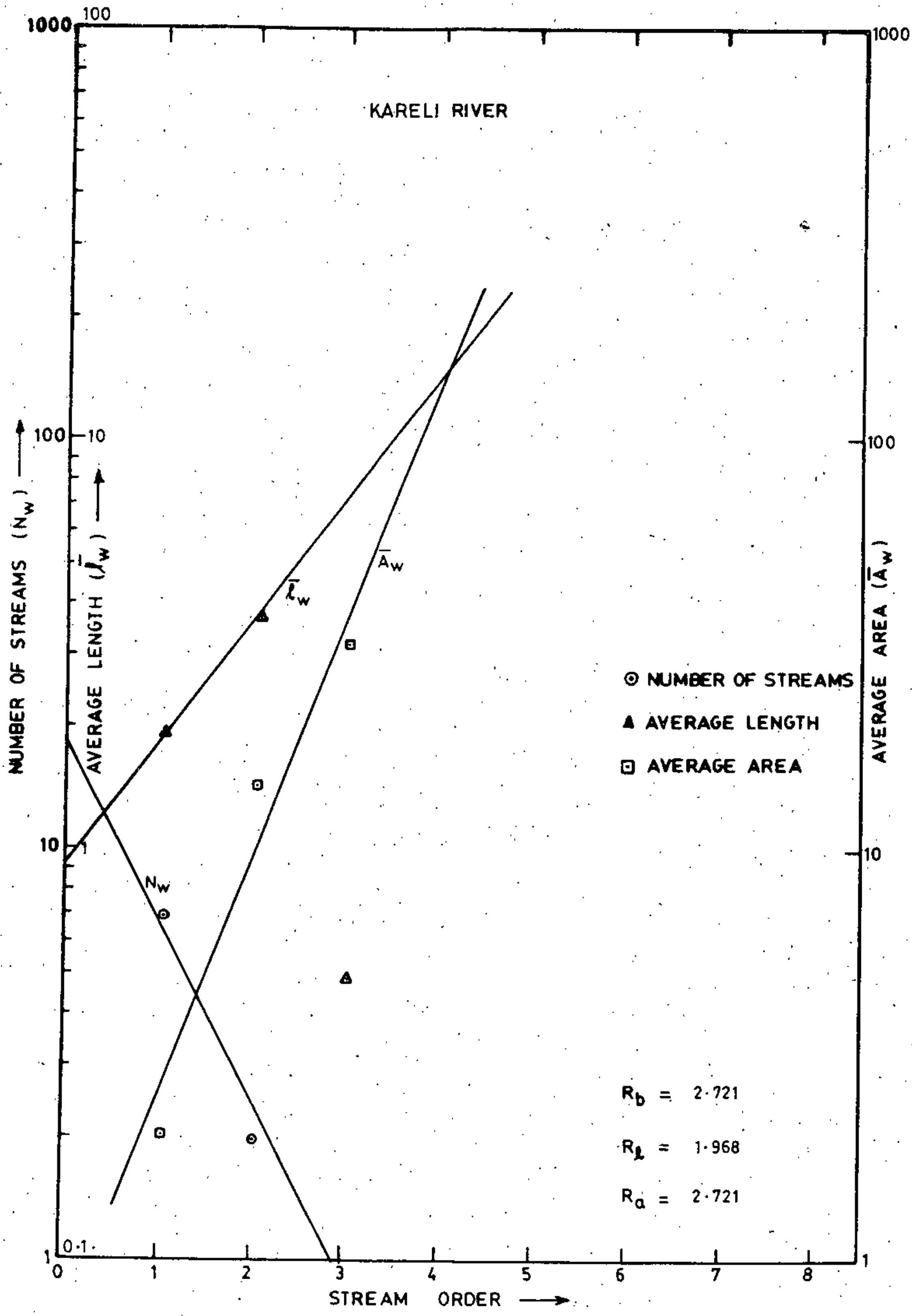

FIG. 7.1.17 : PLOT OF STREAM NUMBERS, AVERAGE LENGTH AND AVERAGE AREA Vs. STREAM ORDER FOR CATCHMENT OF Br. NO.

Table 7.2 : Geomorphological Characteristics of Sub-Catchments of Subzone 3C

S.No.	Bridge No.	River	Order	No. of Streams	Total Length	Average Length (km.)	Total Area	Average Area (sq.km)	Values of Constants
1	644	CHANDRABHAGA	1	2331	1187.0	0.509	-	-	R_b = 4.722
			2	526	485.0	0.922	-	-	R_l = 2.243
			3	123	267.0	2.170	499.4	4.060	R_a = 4.417
			4	25	97.0	3.880	488.2	19.528	L = 87.5km
			5	5	58.5	11.700	373.6	74.720	
			6	1	76.5	76.500	943.5	943.500	
2	803 (802)	MACHANA	1	210	475.0	2.261	-	-	R_b = 2.511
			2	47	148.0	3.148	532.1	11.321	R_l = 1.753
			3	13	100.0	7.692	530.1	40.776	R_a = 3.074
			4	4	65.0	16.250	609.4	152.350	L =100.0km
			5	2	40.0	20.000	810.6	405.300	
			6	1	28.0	28.000	975.0	975.000	
3	578	Sukta	1	1965	925.5	0.471	-	-	R_b = 3.208
			2	442	385.5	0.872	-	-	R_l = 2.445
			3	96	189.0	1.969	376.7	3.924	R_a = 5.505
			4	25	123.5	4.940	369.7	14.788	L = 79.0km
			5	5	29.0	5.800	227.1	45.420	
			6	1	65.5	65.500	640.6	648.000	
4	625	Kalimachak	1	-	-	-	-	-	R_b = 5.179
			2	277	286.0	1.032	-	-	R_l = 2.918
			3	61	116.0	1.902	301.4	5.090	R_a = 8.111
			4	10	54.0	5.400	263.0	26.300	L = 67.0km
			5	2	49.0	24.500	464.0	232.000	
			6	1	24.0	24.000	562.8	562.800	

Table 7.2 Contd...

S.No.	Bridge No.	River	Order	No. of Streams	Total Length	Average Length (km.)	Total Area	Average Area (sq.km)	Values of Constants
5	249	Temur	1 2 3 4 5 6	1432 291 61 12 4 1	403.8 285.7 135.6 75.0 36.7 24.7	0.282 0.982 2.224 6.250 9.187 24.750	120.3 145.8 175.9 199.1 381.9 518.6	0.084 0.501 2.884 16.596 95.499 526.400	R_b = 4.170 R_l = 3.890 R_a = 5.810 L = 52.0km
6	394/2	Uma	1 2 3 4 5	492 108 25 8 1	338.5 161.5 67.5 39.5 33.0	0.688 1.495 2.700 4.937 33.000	- - 161.8 229.5 353.5	- - 6.472 28.687 354.450	R_b = 3.359 R_l = 2.197 R_a = 3.074 L = 41.0km
7	897 (557)	Baloorena	1 2 3 4 5 6	503 118 29 7 2 1	503.0 144.0 90.0 48.0 28.0 4.0	1.000 1.220 3.103 6.857 14.000 4.000	- - 195.0 211.0 320.5 329.9	- - 6.724 30.142 160.250 330.900	R_b = 3.208 R_l = 2.227 R_a = 4.417 L = 43.0km
8	787	Katepurna	1 2 3 4 5 6	448 120 24 6 2 1	322.0 144.0 52.0 31.0 45.0 1.7	0.718 1.200 2.167 5.167 22.500 1.700	- - 139.7 159.3 316.7 320.1	- - 5.821 26.550 158.350 326.750	R_b = 3.593 R_l = 2.077 R_a = 5.179 L = 37.0km
9	930	Umar	1 2 3 4 5	363 94 24 7 1	224.3 49.5 49.2 42.3 30.0	0.618 0.527 2.050 6.050 30.000	125.9 95.2 109.7 118.7 226.2	0.347 1.013 4.572 16.960 227.800	R_b = 4.040 R_l = 2.560 R_a = 4.760 L = 38.0km

Table 7.2 Contd...

S.No.	Bridge No.	River	Order	No. of Streams	Total Length	Average Length (km.)	Total Area	Average Area (sq.km)	Values of Constants
10	776	Sakatwar	1	561	315.6	0.562	-	-	R_b = 2.918
			2	122	121.0	0.992	-	-	R_l = 2.276
			3	27	70.7	2.620	119.7	4.433	R_a = 3.727
			4	7	29.2	4.178	120.7	17.243	L = 24.0km
			5	2	28.2	14.125	176.2	88.100	
			6	1	0.5	0.500	180.0	182.300	
11	584	Lakhora	1	311	197.6	0.635	-	-	R_b = 4.124
			2	79	92.5	1.171	-	-	R_l = 2.424
			3	15	37.2	2.483	76.3	5.086	R_a = 3.594
			4	4	24.7	6.187	111.7	27.925	L = 28.0km
			5	1	19.0	19.000	154.1	153.400	
12	732	Hatear	1	416	217.6	0.523	-	-	R_b = 4.526
	(454)		2	100	99.0	0.990	-	-	R_l = 1.847
			3	22	36.7	1.670	74.3	3.377	R_a = 2.721
			4	6	20.5	3.416	57.4	9.570	L = 36.0km
			5	1	27.0	27.000	115.2	116.75	
13	253	Tyria	1	265	118.7	0.448	36.8	0.139	R_b = 4.080
			2	59	51.1	0.867	67.9	1.151	R_l = 2.750
			3	13	25.7	1.977	71.2	5.475	R_a = 4.580
			4	2	13.8	6.900	94.9	47.49	L = 32.0km
			5	1	4.0	4.000	102.0	100.40	
14	813	Machhwasa	1	173	85.0	0.491	-	-	R_b = 3.257
	(505)	(Passa)	2	38	42.7	1.125	-	-	R_l = 2.182
			3	10	16.2	1.625	33.9	3.395	R_a = 4.417
			4	2	20.0	10.000	42.7	21.385	L = 22.0km
			5	1	8.0	8.000	66.5	67.300	

Table 7.2 Contd...

S.No.	Bridge No.	River	Order	No. of Streams	Total Length	Average Length (km.)	Total Area	Average Area (sq.km)	Values of Constants
15	832 (517/1)	Ol Nadi	1	151	99.7	0.661	-	-	R_b = 3.727
			2	28	33.9	1.212	-	-	R_l = 2.636
			3	7	27.8	3.971	43.3	6.196	R_a = 2.993
			4	2	12.5	6.250	38.8	19.425	L = 16.0km
			5	1	1.7	1.750	54.8	58.100	
16	710	Khara Nala	1	448	165.0	0.368	-	-	R_b = 4.894
			2	85	43.1	0.507	-	-	R_l = 3.162
			3	14	26.9	1.921	45.2	3.228	R_a = 5.179
			4	4	14.5	3.625	64.1	16.025	L = 46.0km
			5	1	33.0	33.000	121.6	119.450	
17	889	Kareli	1	7	13.5	1.935	14.4	2.057	R_b = 2.721
			2	2	7.5	3.750	29.0	14.500	R_l = 1.968
			3	1	0.5	0.500	31.1	31.100	R_a = 2.721
									L = 13.0km

corresponding to velocities of 1.0, 2.0, 3.0, 4.0, 5.0, 6.0 and 7.0 m/sec. for all the fourteen bridge catchments. The computer program is run for all the events separately using the data prepared as explained above and the results obtained are explained here under. However, for a few very small catchments where the time to peak corresponding to higher velocity events is around 1.0 hour the convergence could not be achieved and are thus not reported in the results.

For each event the peak characteristics given by the GIUH theory (i.e., eq.(5) and (6)) and that given by the GIUH based Clark model are tabulated in Table 7.3. The characteristics given by the GIUH based Clark model are given for two computational time intervals. The smaller computational time has been used so that the error due to discretisation in time domain may be reduced to a very low level. The product of peak discharge and time to peak are also given alongwith.

The values of the velocities and GIUH based Clark model parameters derived for all the above mentioned events of the seventeen bridge catchments are tabulated in Table 7.4. The ratio $R/(T_c+R)$ is also calculated for each event and is given alongwith.

The Unit Hydrograph of 1.0 hr. duration is derived from the IUH of the GIUH based Clark Model computed above. The regional relationship for 1.0 hr. unit hydrograph recommended by FEPCC (CWC, 1983) is used to obtain 1.0 hrs. unit hydrograph for all the seventeen bridge catchments. These unit hydrographs are referred to as regional UH hereinafter in the text. The ordinates of these regional unit hydrographs for all the seventeen catchments are given in Table 7.5. Peak discharge and time to peak for one hour regional unit hydrograph and that obtained by GIUH based Clark model for all the events are given in Table 7.6.

Fig.7.2.1 to 7.2.17 gives the plots of the ordinates of 1 hour unit hydrographs by regional unit hydrograph approach and GIUH based Clark model approach for all the assumed velocities of flow for all the seventeen catchments respectively.

Table 7.3 : Comparison of Peak Characteristics of GIUH and GIUH Based Clark Model IUH for Different Bridge Catchments

S.No.	Bridge No.	Velocity	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
		(m/s)	Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
1	644	1.0	5.75	29.38	168.88	5.91	24.30	143.49	5.88	24.00	141.22
		2.0	11.50	14.69	168.88	11.65	12.15	141.49	11.65	12.00	139.84
		3.0	17.24	9.79	168.88	17.36	8.10	140.65	17.38	8.00	139.04
		4.0	22.99	7.35	168.88	23.14	6.10	141.14	23.21	6.00	139.24
		5.0	28.74	5.88	168.88	28.85	4.85	139.90	28.35	5.00	141.76
		6.0	34.49	4.90	168.88	34.60	4.05	140.12	34.36	4.00	137.42
		7.0	40.23	4.20	168.88	40.39	3.45	139.35	40.98	3.00	122.95
2	803	1.0	4.52	31.80	143.66	4.68	27.75	129.84	4.60	28.00	128.73
		2.0	9.03	15.90	143.66	9.14	13.90	127.05	9.06	14.00	126.87
		3.0	13.55	10.60	143.66	13.70	9.25	126.75	13.87	9.00	124.83
		4.0	18.07	7.95	143.66	18.20	6.95	126.49	18.11	7.00	126.80
		5.0	22.58	6.36	143.66	22.69	5.55	125.94	21.57	6.00	129.41
		6.0	27.10	5.30	143.66	27.24	4.65	126.65	26.11	5.00	130.57
		7.0	31.62	4.54	143.66	31.78	3.95	125.51	31.48	4.00	125.94
3	578	1.0	4.38	18.39	80.62	4.46	21.95	97.81	4.42	22.00	97.29
		2.0	8.77	9.19	80.62	8.87	10.95	97.15	8.80	11.00	96.84
		3.0	13.15	6.13	80.62	13.24	7.30	96.66	13.55	7.00	94.87
		4.0	17.54	4.60	80.62	17.64	5.50	97.03	18.61	5.00	93.05
		5.0	21.92	3.68	80.62	22.02	4.40	96.88	23.10	4.00	92.40
		6.0	26.31	3.06	80.62	26.40	3.65	96.34	24.89	4.00	99.56
		7.0	30.69	2.63	80.62	30.61	3.15	96.43	31.67	3.00	95.02

Table 7.3 Contd...

S.No.	Bridge No.	Velocity	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
		(m/s)	Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
4	625	1.0	4.88	15.33	74.76	4.97	17.90	88.98	4.94	18.00	88.84
		2.0	9.75	7.67	74.76	9.84	8.95	88.10	10.06	9.00	90.58
		3.0	14.63	5.11	74.76	14.72	6.00	88.34	15.09	6.00	90.55
		4.0	19.50	3.83	74.76	19.59	4.50	88.14	18.53	5.00	92.65
		5.0	24.38	3.07	74.76	24.32	3.70	89.98	23.10	4.00	92.38
		6.0	29.25	2.56	74.76	29.18	3.00	87.53	29.87	3.00	89.62
		7.0	34.13	2.19	74.76	31.07	2.65	90.28	30.86	3.00	92.59
5	249	1.0	6.61	11.38	75.17	6.67	11.80	78.74	6.60	12.00	79.22
		2.0	13.21	5.69	75.17	13.28	5.90	78.36	13.14	6.00	78.86
		3.0	19.82	3.79	75.17	19.89	3.95	78.58	19.79	4.00	79.16
		4.0	26.43	2.84	75.17	26.51	2.95	78.19	26.18	3.00	78.53
		5.0	33.03	2.28	75.17	32.98	2.35	77.49	29.63	3.00	88.90
		6.0	39.64	1.90	75.17	39.55	2.00	79.10	40.64	2.00	81.28
		7.0	46.25	1.63	75.17	46.19	1.70	78.52	44.37	2.00	88.74
6	394/2	1.0	4.41	14.04	61.98	4.45	11.40	50.78	4.54	11.00	49.92
		2.0	8.83	7.02	61.98	8.89	5.70	50.65	8.55	6.00	51.33
		3.0	13.24	4.68	61.98	13.28	3.80	50.48	12.85	4.00	51.41
		4.0	17.65	3.51	61.98	17.70	2.85	50.44	17.27	3.00	51.80
		5.0	22.07	2.81	61.98	22.03	2.30	50.67	23.30	2.00	46.60
		6.0	26.48	2.34	61.98	26.42	1.90	50.20	25.18	2.00	50.35
		7.0	30.89	2.01	61.98	30.85	1.65	50.90	27.34	2.00	54.68

Table 7.3 Contd..

S.No.	Bridge No.	Velocity (m/s)	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
			Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
7	897	1.0	3.95	11.71	46.25	4.00	11.95	47.77	3.97	12.00	47.62
		2.0	7.90	5.85	46.25	7.94	5.95	47.26	7.89	6.00	47.32
		3.0	11.85	3.90	46.25	11.89	4.00	47.57	11.86	4.00	47.44
		4.0	15.81	2.93	46.25	15.76	3.00	47.29	15.72	3.00	47.17
		5.0	19.76	2.34	46.25	19.71	2.40	47.31	21.52	2.00	43.05
		6.0	23.71	1.95	46.25	23.66	2.00	47.32	23.33	2.00	46.65
		7.0	27.66	1.67	46.25	27.62	1.70	46.96	23.98	2.00	47.95
8	787	1.0	4.40	10.09	44.38	4.44	10.30	45.74	4.51	10.00	45.06
		2.0	8.80	5.04	44.38	8.85	5.15	45.58	8.98	5.00	44.90
		3.0	13.20	3.36	44.38	13.24	3.45	45.68	14.13	3.00	42.38
		4.0	17.60	2.52	44.38	17.56	2.55	44.79	15.88	3.00	47.64
		5.0	22.00	2.02	44.38	21.96	2.05	45.02	22.20	2.00	44.40
		6.0	26.40	1.68	44.38	26.37	1.70	44.82	24.20	2.00	48.40
		7.0	30.80	1.44	44.38	30.77	1.45	44.61	33.28	1.00	33.28
9	930	1.0	3.27	10.69	34.93	3.31	10.15	33.56	3.24	10.00	32.44
		2.0	6.54	5.34	34.93	6.57	5.10	33.51	6.74	5.00	33.70
		3.0	9.80	3.56	34.93	9.77	3.40	33.21	8.92	4.00	35.67
		4.0	13.07	2.67	34.93	13.03	2.55	33.24	11.99	3.00	35.97
		5.0	16.34	2.14	34.93	16.31	2.05	33.43	16.57	2.00	33.14
		6.0	19.61	1.78	34.93	19.58	1.70	33.29	18.33	2.00	36.67
		7.0	22.88	1.53	34.93	22.84	1.45	33.12	19.35	2.00	38.71

Table 7.3 Contd...

S.No.	Bridge No.	Velocity	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
		(m/s)	Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
10	776	1.0	3.94	6.75	26.59	3.96	6.65	26.34	3.83	7.00	26.80
		2.0	7.87	3.38	26.59	7.90	3.35	26.46	8.14	3.00	24.42
		3.0	11.81	2.25	26.59	11.78	2.20	25.92	11.76	2.00	23.52
		4.0	15.75	1.69	26.59	15.73	1.65	25.95	14.38	2.00	28.77
		5.0	19.68	1.35	26.59	19.65	1.35	26.53	19.49	1.00	19.49
		6.0	23.62	1.13	26.59	23.66	1.10	26.02	22.27	1.00	22.27
11	584	1.0	2.92	9.49	27.69	2.94	7.80	22.95	2.89	8.00	23.10
		2.0	5.84	4.75	27.69	5.86	3.90	22.86	5.79	4.00	23.15
		3.0	8.75	3.16	27.69	8.73	2.60	22.71	8.04	3.00	24.12
		4.0	11.67	2.37	27.69	11.64	1.95	22.71	11.60	2.00	23.20
		5.0	14.59	1.90	27.69	14.56	1.55	22.57	12.80	2.00	25.61
		6.0	17.51	1.58	27.69	17.53	1.30	22.79	17.84	1.00	17.84
		7.0	20.42	1.36	27.69	20.45	1.10	22.50	19.94	1.00	19.94
12	732	1.0	1.54	16.60	25.50	1.55	10.00	15.54	1.55	10.00	15.47
		2.0	3.07	8.30	25.50	3.09	5.00	15.46	3.09	5.00	15.43
		3.0	4.61	5.53	25.50	4.62	3.35	15.49	4.80	3.00	14.41
		4.0	6.15	4.15	25.50	6.13	2.50	15.33	5.57	3.00	16.71
		5.0	7.68	3.32	25.50	7.66	2.00	15.33	7.60	2.00	15.21
		6.0	9.22	2.77	25.50	9.21	1.65	15.19	8.47	2.00	16.94
		7.0	10.76	2.37	25.50	10.74	1.45	15.58	11.02	1.00	11.02

Table 7.3 Contd...

S.No.	Bridge No.	Velocity	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
		(m/s)	Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
13	253	1.0	1.76	9.00	15.87	1.78	8.90	15.82	1.76	9.00	15.82
		2.0	3.53	4.50	15.87	3.54	4.45	15.77	3.81	4.00	15.24
		3.0	5.29	3.00	15.87	5.28	2.95	15.56	5.24	3.00	15.73
		4.0	7.06	2.25	15.87	7.04	2.20	15.49	7.42	2.00	14.84
		5.0	8.82	1.80	15.87	8.81	1.75	15.42	8.36	2.00	16.71
		6.0	10.58	1.50	15.87	10.57	1.45	15.33	12.40	1.00	12.40
		7.0	12.35	1.29	15.87	12.36	1.25	15.45	13.63	1.00	13.63
14	813	1.0	1.56	6.09	9.48	1.57	6.10	9.55	1.58	6.00	9.46
		2.0	3.11	3.04	9.48	3.10	3.05	9.47	3.12	3.00	9.37
		3.0	4.67	2.03	9.48	4.66	2.05	9.56	4.73	2.00	9.45
		4.0	6.23	1.52	9.48	6.22	1.55	9.64	5.39	2.00	10.78
		5.0	7.79	1.22	9.48	7.79	1.20	9.35	7.98	1.00	7.98
		6.0	9.34	1.01	9.48	9.35	1.00	9.35	9.18	1.00	9.18
15	832	1.0	2.00	5.50	11.02	2.01	4.45	8.95	2.15	4.00	8.58
		2.0	4.01	2.75	11.02	4.00	2.20	8.80	4.19	2.00	8.38
		3.0	6.01	1.83	11.02	6.00	1.45	8.71	6.46	1.00	6.46
		4.0	8.02	1.37	11.02	8.03	1.10	8.83	7.99	1.00	7.99

Table 7.3 Contd...

S.No.	Bridge No.	Velocity	Peak Characteristics of GIUH			Peak Characteristics of GIUH Based Clark Model IUH					
						Computational Time Interval = 0.05 hrs.			Computational Time Interval = 1.0 hrs.		
		(m/s)	Q_{pg} (cum.)	T_{pg} (hrs.)	$Q_{pg}xT_{pg}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)	Q_{pc} (cum.)	T_{pc} (hrs.)	$Q_{pc}xT_{pc}$ (cuhr)
16	710	1.0	1.55	12.67	19.64	1.57	12.75	19.96	1.54	13.00	20.03
		2.0	3.10	6.33	19.64	3.11	6.40	19.93	3.24	6.00	19.46
		3.0	4.65	4.22	19.64	4.67	4.25	19.83	4.89	4.00	19.57
		4.0	6.20	3.17	19.64	6.22	3.20	19.90	6.43	3.00	19.28
		5.0	7.75	2.53	19.64	7.74	2.55	19.73	6.91	3.00	20.74
		6.0	9.30	2.11	19.64	9.29	2.10	19.51	9.38	2.00	18.76
		7.0	10.85	1.81	19.64	10.84	1.80	19.51	10.36	2.00	20.72
17	889	1.0	1.17	4.42	5.17	1.17	3.60	4.22	1.10	4.00	4.39
		2.0	2.34	2.21	5.17	2.33	1.80	4.20	2.24	2.00	4.47
		3.0	3.51	1.47	5.17	3.51	1.20	4.21	3.87	1.00	3.87

Table 7.4 : Summary of the velocities and GIUH based Clark Model parameters for various velocities for different bridge catchments.

S.No.	Bridge No.	Velocity (m/sec)	Time of concentration T_c (hours)	Storage coefficient R (hours)	Ratio $R/(R+T_c)$
1	644	1.0	24.31	39.93	0.6216
		2.0	12.15	20.17	0.6241
		3.0	8.10	13.51	0.6251
		4.0	6.08	10.11	0.6245
		5.0	4.86	8.14	0.6260
		6.0	4.05	6.77	0.6257
		7.0	3.47	5.83	0.6266
2	803	1.0	27.78	43.35	0.6095
		2.0	13.89	22.13	0.6144
		3.0	9.26	14.78	0.6149
		4.0	6.94	11.11	0.6153
		5.0	5.56	8.92	0.6162
		6.0	4.63	7.40	0.6151
		7.0	3.97	6.39	0.6169
3	578	1.0	21.94	30.67	0.5829
		2.0	10.97	15.43	0.5844
		3.0	7.31	10.33	0.5854
		4.0	5.49	7.72	0.5845
		5.0	4.39	6.18	0.5848
		6.0	3.66	5.18	0.5861
		7.0	3.13	4.43	0.5857
4	625	1.0	18.61	21.56	0.5367
		2.0	9.31	10.88	0.5391
		3.0	6.20	7.27	0.5397
		4.0	4.65	5.47	0.5402
		5.0	3.72	4.43	0.5435
		6.0	3.10	3.67	0.5419
		7.0	2.66	3.15	0.5425
5	249	1.0	14.44	16.54	0.5339
		2.0	7.22	8.30	0.5348
		3.0	4.81	5.53	0.5345
		4.0	3.61	4.16	0.5353
		5.0	2.89	3.32	0.5344
		6.0	2.41	2.76	0.5339
		7.0	2.06	2.37	0.5349

Table 7.4 Contd...

S.No.	Bridge No.	Velocity (m/sec)	Time of concentration T_c (hours)	Storage coefficient R (hours)	Ratio $R/(R+T_c)$
6	394/2	1.0	11.39	16.53	0.5920
		2.0	5.69	8.28	0.5926
		3.0	3.80	5.54	0.5933
		4.0	2.85	4.16	0.5935
		5.0	2.28	3.31	0.5923
		6.0	1.90	2.78	0.5944
		7.0	1.63	2.35	0.5912
7	897	1.0	11.94	17.56	0.5952
		2.0	5.97	8.86	0.5974
		3.0	3.98	5.87	0.5959
		4.0	2.99	4.43	0.5971
		5.0	2.39	3.54	0.5970
		6.0	1.99	2.95	0.5970
		7.0	1.71	2.55	0.5987
8	787	1.0	10.28	15.26	0.5975
		2.0	5.14	7.65	0.5982
		3.0	3.43	5.09	0.5977
		4.0	2.57	3.89	0.6019
		5.0	2.06	3.09	0.6008
		6.0	1.71	2.59	0.6017
		7.0	1.47	2.23	0.6027
9	930	1.0	10.56	14.53	0.5792
		2.0	5.28	7.29	0.5802
		3.0	3.52	4.90	0.5820
		4.0	2.64	3.67	0.5819
		5.0	2.11	2.93	0.5815
		6.0	1.76	2.45	0.5816
		7.0	1.51	2.10	0.5819
10	776	1.0	6.67	10.55	0.6128
		2.0	3.33	5.26	0.6119
		3.0	2.22	3.56	0.6156
		4.0	1.67	2.67	0.6154
		5.0	1.33	2.10	0.6115
		6.0	1.11	1.77	0.6148
11	584	1.0	7.78	11.26	0.5916
		2.0	3.89	5.65	0.5924
		3.0	2.59	3.79	0.5937
		4.0	1.94	2.84	0.5937
		5.0	1.56	2.29	0.5950
		6.0	1.30	1.89	0.5930
		7.0	1.11	1.64	0.5957

Table 7.4 Contd...

S.No.	Bridge No.	Velocity (m/sec)	Time of concentration T_c (hours)	Storage coefficient R (hours)	Ratio $R/(R+T_c)$
12	732	1.0	10.00	17.11	0.6311
		2.0	5.00	8.59	0.6321
		3.0	3.33	5.72	0.6318
		4.0	2.50	4.32	0.6336
		5.0	2.00	3.46	0.6336
		6.0	1.67	2.90	0.6350
		7.0	1.43	2.44	0.6309
13	253	1.0	8.89	10.52	0.5420
		2.0	4.44	5.28	0.5429
		3.0	2.96	3.57	0.5466
		4.0	2.22	2.70	0.5481
		5.0	1.78	2.11	0.5424
		6.0	1.48	1.75	0.5415
		7.0	1.27	1.55	0.5490
14	813	1.0	6.11	9.06	0.5971
		2.0	3.06	4.56	0.5988
		3.0	2.04	3.02	0.5969
		4.0	1.53	2.24	0.5949
		5.0	1.22	1.84	0.6013
		6.0	1.02	1.54	0.6012
15	832	1.0	4.44	6.00	0.5744
		2.0	2.22	3.05	0.5783
		3.0	1.48	2.00	0.5744
		4.0	1.11	1.52	0.5776
16	710	1.0	12.78	16.22	0.5594
		2.0	6.39	8.14	0.5604
		3.0	4.26	5.46	0.5616
		4.0	3.19	4.08	0.5607
		5.0	2.56	3.29	0.5626
		6.0	2.13	2.71	0.5600
		7.0	1.83	2.32	0.5600
17	889	1.0	3.61	5.21	0.5908
		2.0	1.81	2.62	0.5919
		3.0	1.20	1.74	0.5913

Table 7.5 : Ordinates of 1 hr. Synthetic Regional Unit Hydrographs for all the seventeen Bridge Catchments.

Time in hrs	1 hr. Regional Unit Hydrograph Ordinates (cumec.)								
	Br. No. 644	Br. No. 803	Br. No. 578	Br. No. 625	Br. No. 249	Br. No. 394/2	Br. No. 897	Br. No. 787	Br. No. 930
0	.00	.00	.00	.00	0.00	.00	.00	.00	.00
1	1.75	.75	1.25	1.00	1.70	3.00	1.10	2.25	.40
2	5.00	2.00	4.00	2.25	3.70	7.75	2.60	8.50	1.60
3	9.25	5.25	12.50	4.75	6.00	13.25	4.20	18.50	4.40
4	16.25	17.12	30.50	9.75	10.80	20.75	6.00	26.75	14.70
5	25.00	26.25	36.00	20.75	20.50	38.75	8.50	15.00	14.80
6	29.50	31.50	35.75	25.00	23.10	38.75	13.30	6.25	10.90
7	31.75	33.75	28.25	25.00	16.10	33.25	14.40	4.00	6.70
8	32.00	33.75	18.50	21.00	13.80	22.00	12.40	2.75	3.80
9	29.50	29.25	10.00	14.50	11.70	16.50	9.20	2.00	2.30
10	25.00	23.75	5.50	7.75	10.30	13.00	6.50	1.25	1.30
11	20.25	18.25	3.25	5.00	8.80	10.25	4.80	.75	.60
12	15.75	13.37	2.00	4.00	7.30	8.00	3.60	.50	.40
13	11.00	8.25	.75	2.75	5.80	6.25	2.70	.25	.20
14	7.75	5.25	.25	2.00	4.50	4.50	2.10	.25	.05
15	5.50	3.00	.00	1.25	2.80	3.25	1.60	.00	.00
16	3.50	1.75		1.00	1.80	2.00	1.00		
17	2.25	.75		.75	0.60	1.00	.70		
18	1.25	.25		.25	0.00	.00	.30		
19	.87	.00		.00			.20		
20	.37						.10		
21	.00						.00		

Table 7.5 Contd...

Time in hrs.	1 hr. Regional Unit Hydrograph Ordinates (cumec.)							
	Br. No. 776	Br. No. 584	Br. No. 732	Br. No. 253	Br. No. 813	Br. No. 832	Br. No. 710	Br. No. 889
0	.00	.00	.00	0.00	.00	.00	.00	.00
1	3.60	.70	1.30	0.60	.50	1.40	.70	.35
2	13.10	2.60	3.20	1.30	2.00	3.70	1.60	1.80
3	13.20	4.90	4.30	2.40	4.80	4.90	2.90	2.50
4	8.80	7.40	4.80	4.90	5.00	2.50	1.80	1.50
5	5.00	7.30	4.70	5.20	4.00	1.50	1.40	.80
6	2.90	5.30	4.00	4.20	1.80	.80	1.30	.50
7	1.60	3.80	3.30	2.70	.80	.40	1.00	.30
8	.80	2.40	2.40	2.20	.40	.10	.60	.15
9	.30	1.60	1.70	1.80	.20	.00	.30	.05
10	.00	1.00	1.20	1.30	.00		.00	.00
11		.70	.80	0.90				
12		.40	.60	0.50				
13		.30	.40	0.30				
14		.20	.20	0.00				
15		.00	.00					

Table 7.6 : Comparison of the Peak Characteristics of Regional UH and GIUH based Clark Model UH.

S.No.	Bridge No.	Velocity (m/sec)	Peak Characteristics of UH			
			1 hr. Regional UH		1 hr. GIUH Based Clark UH	
			Q_p	T_p	Q_p	T_p
1	644	1.0	32.00	8.00	5.81	25.00
		2.0	32.00	8.00	11.37	13.00
		3.0	32.00	8.00	16.76	9.00
		4.0	32.00	8.00	22.11	7.00
		5.0	32.00	8.00	26.71	6.00
		6.0	32.00	8.00	31.99	5.00
		7.0	32.00	8.00	37.74	4.00
2	803	1.0	33.75	7.00	4.58	28.00
		2.0	33.75	7.00	8.94	14.00
		3.0	33.75	7.00	13.42	10.00
		4.0	33.75	7.00	17.62	7.00
		5.0	33.75	7.00	21.57	6.00
		6.0	33.75	7.00	25.94	5.00
		7.0	33.75	7.00	29.72	4.00
3	578	1.0	36.00	5.00	4.37	22.00
		2.0	36.00	5.00	8.58	11.00
		3.0	36.00	5.00	12.93	8.00
		4.0	36.00	5.00	17.48	6.00
		5.0	36.00	5.00	21.37	5.00
		6.0	36.00	5.00	23.55	4.00
		7.0	36.00	5.00	28.46	4.00
4	625	1.0	25.00	6.00	4.89	19.00
		2.0	25.00	6.00	9.62	10.00
		3.0	25.00	6.00	14.12	7.00
		4.0	25.00	6.00	18.24	5.00
		5.0	25.00	6.00	22.29	4.00
		6.0	25.00	6.00	26.29	4.00
		7.0	25.00	6.00	30.33	3.00
5	249	1.0	17.30	7.00	6.48	13.00
		2.0	17.30	7.00	12.75	7.00
		3.0	17.30	7.00	18.66	5.00
		4.0	17.30	7.00	24.03	4.00
		5.0	17.30	7.00	26.72	3.00
		6.0	17.30	7.00	34.40	3.00
		7.0	17.30	7.00	36.65	3.00
6	394/2	1.0	38.75	5.00	4.41	12.00
		2.0	38.75	5.00	8.26	6.00
		3.0	38.75	5.00	11.81	4.00
		4.0	38.75	5.00	15.41	4.00
		5.0	38.75	5.00	20.24	3.00
		6.0	38.75	5.00	21.34	3.00
		7.0	38.75	5.00	23.90	2.00

Table 7.6 Contd...

S.No.	Bridge No.	Velocity (m/sec)	Peak Characteristics of UH: 1 hr. Regional UH Q_p	T_p	1 hr. GIUH Based Clark UH Q_p	T_p
7	897	1.0	14.40	7.00	3.86	13.00
		2.0	14.40	7.00	7.47	7.00
		3.0	14.40	7.00	10.93	5.00
		4.0	14.40	7.00	14.13	4.00
		5.0	14.40	7.00	18.86	3.00
		6.0	14.40	7.00	19.95	3.00
		7.0	14.40	7.00	21.44	2.00
8	787	1.0	26.75	4.00	4.36	11.00
		2.0	26.75	4.00	8.43	6.00
		3.0	26.75	4.00	12.86	4.00
		4.0	26.75	4.00	15.15	3.00
		5.0	26.75	4.00	19.11	3.00
		6.0	26.75	4.00	20.28	3.00
		7.0	26.75	4.00	27.18	2.00
9	930	1.0	14.80	5.00	3.21	11.00
		2.0	14.80	5.00	6.31	6.00
		3.0	14.80	5.00	8.72	4.00
		4.0	14.80	5.00	10.77	3.00
		5.0	14.80	5.00	14.16	3.00
		6.0	14.80	5.00	15.22	3.00
		7.0	14.80	5.00	16.17	2.00
10	776	1.0	13.20	3.00	3.65	8.00
		2.0	13.20	3.00	7.43	4.00
		3.0	13.20	3.00	10.31	3.00
		4.0	13.20	3.00	12.11	3.00
		5.0	13.20	3.00	15.74	2.00
		6.0	13.20	3.00	17.38	2.00
11	584	1.0	7.40	4.00	2.81	8.00
		2.0	7.40	4.00	5.32	5.00
		3.0	7.40	4.00	7.10	4.00
		4.0	7.40	4.00	9.86	3.00
		5.0	7.40	4.00	10.50	3.00
		6.0	7.40	4.00	14.10	2.00
		7.0	7.40	4.00	15.28	2.00
12	732	1.0	4.80	4.00	1.50	11.00
		2.0	4.80	4.00	2.92	6.00
		3.0	4.80	4.00	4.42	4.00
		4.0	4.80	4.00	4.99	4.00
		5.0	4.80	4.00	6.64	3.00
		6.0	4.80	4.00	7.22	3.00
		7.0	4.80	4.00	9.15	2.00

Table 7.6 Contd...

S.No.	Bridge No.	Velocity (m/sec)	Peak Characteristics of UH			
			1 hr. Regional UH		1 hr. GIUH Based Clark UH	
			Q_p	T_p	Q_p	T_p
13	253	1.0	5.30	5.00	1.73	9.00
		2.0	5.30	5.00	3.48	5.00
		3.0	5.30	5.00	4.69	3.00
		4.0	5.30	5.00	6.26	3.00
		5.0	5.30	5.00	7.23	2.00
		6.0	5.30	5.00	9.64	2.00
		7.0	5.30	5.00	10.30	2.00
14	813	1.0	5.00	4.00	1.49	7.00
		2.0	5.00	4.00	2.82	4.00
		3.0	5.00	4.00	4.06	3.00
		4.0	5.00	4.00	4.65	2.00
		5.0	5.00	4.00	6.28	2.00
		6.0	5.00	4.00	6.93	2.00
15	832	1.0	4.90	3.00	1.98	5.00
		2.0	4.90	3.00	3.60	3.00
		3.0	4.90	3.00	5.17	2.00
		4.0	4.90	3.00	6.01	2.00
16	710	1.0	2.90	3.00	1.53	13.00
		2.0	2.90	3.00	3.06	7.00
		3.0	2.90	3.00	4.48	5.00
		4.0	2.90	3.00	5.73	4.00
		5.0	2.90	3.00	6.00	4.00
		6.0	2.90	3.00	7.92	3.00
		7.0	2.90	3.00	8.53	3.00
17	889	1.0	2.50	3.00	1.09	4.00
		2.0	2.50	3.00	1.96	2.00
		3.0	2.50	3.00	3.00	2.00

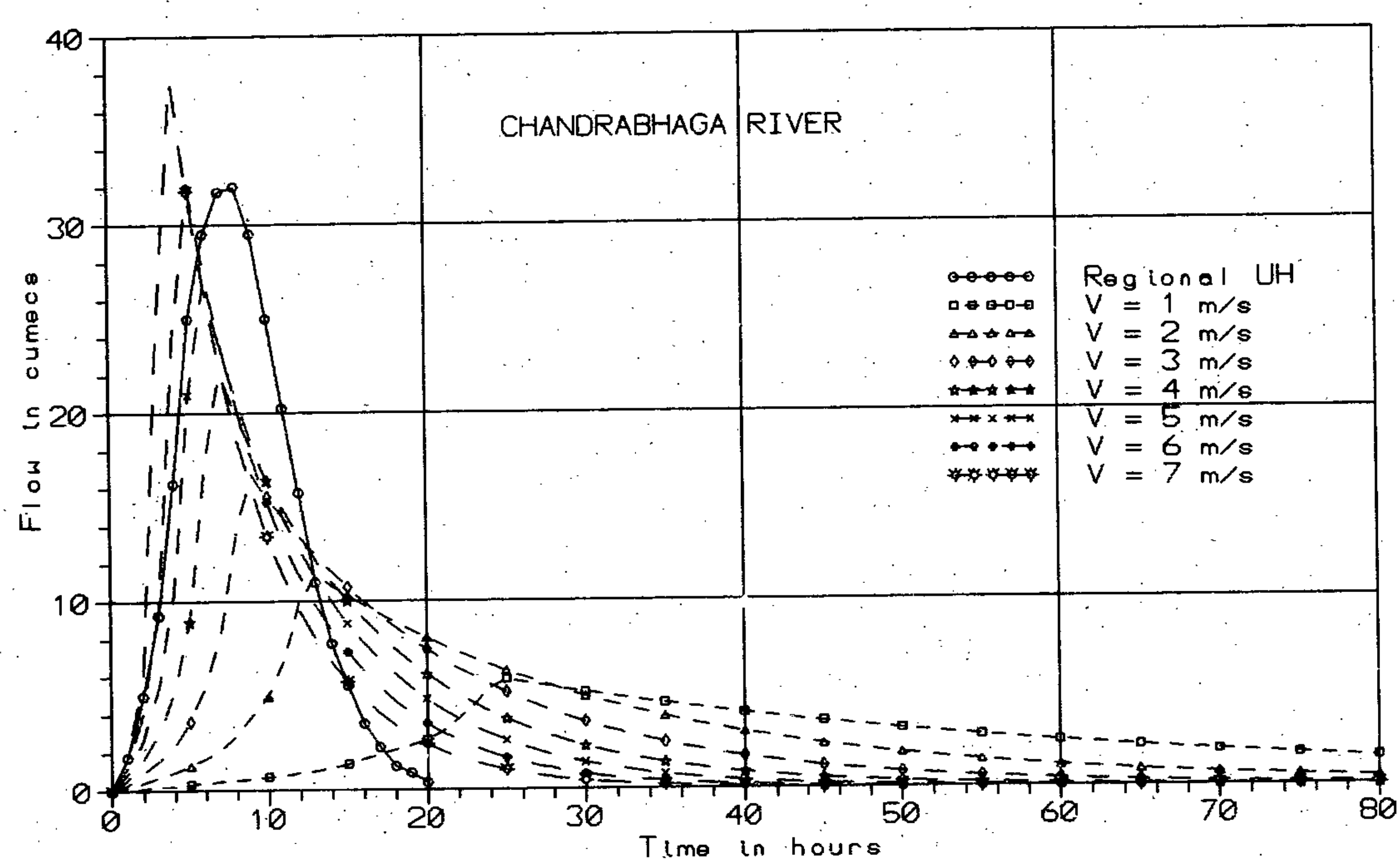

Fig.:7.2.1 : Plot of 1 hr. UHs for Br. catchment No. 644 computed by Regional UH and GIUH based Clark Model approaches.

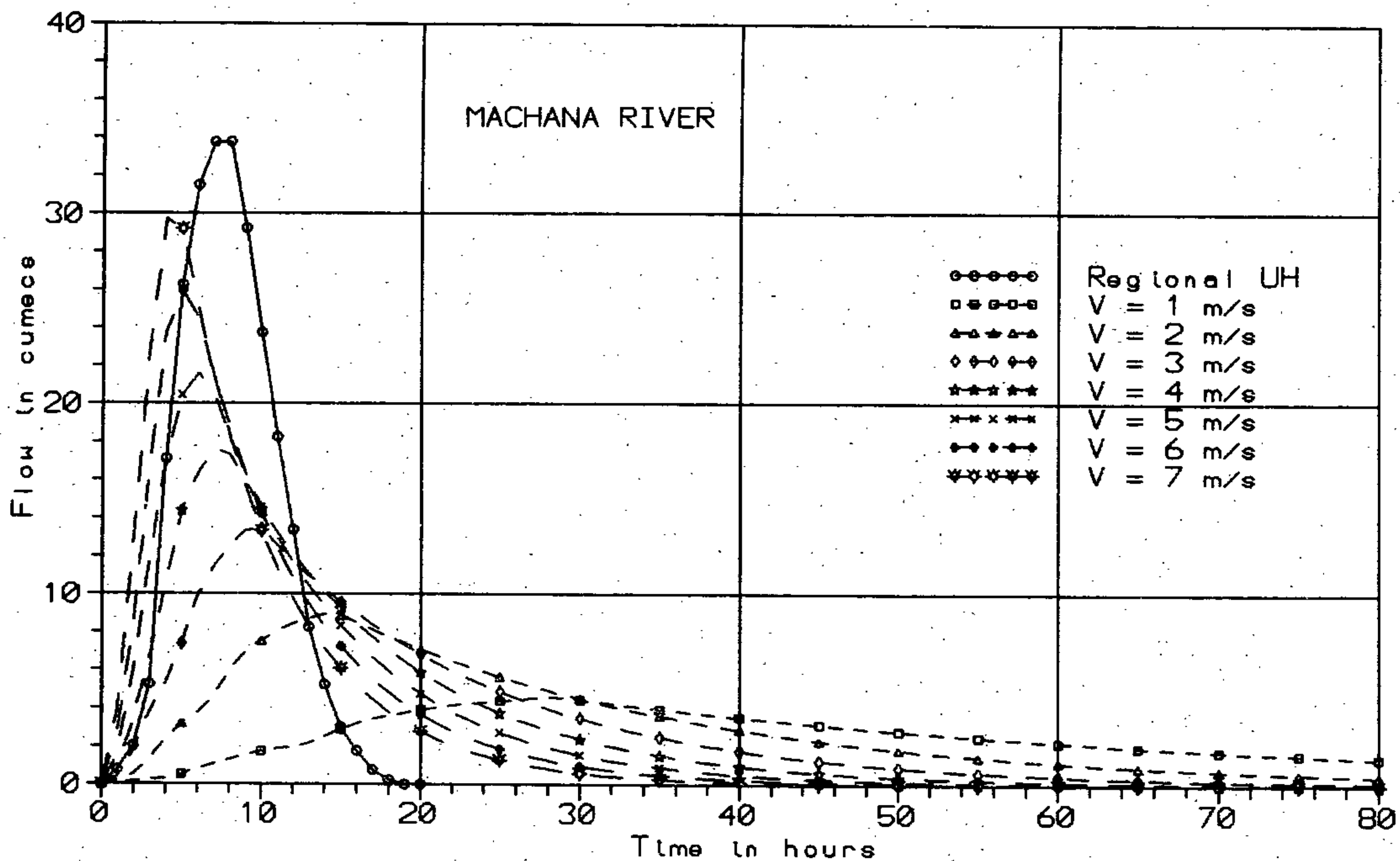

Fig.:7.2.2 : Plot of 1 hr. UHs for Br. catchment No. 803 computed by Regional UH and GIUH based Clark Model approaches.

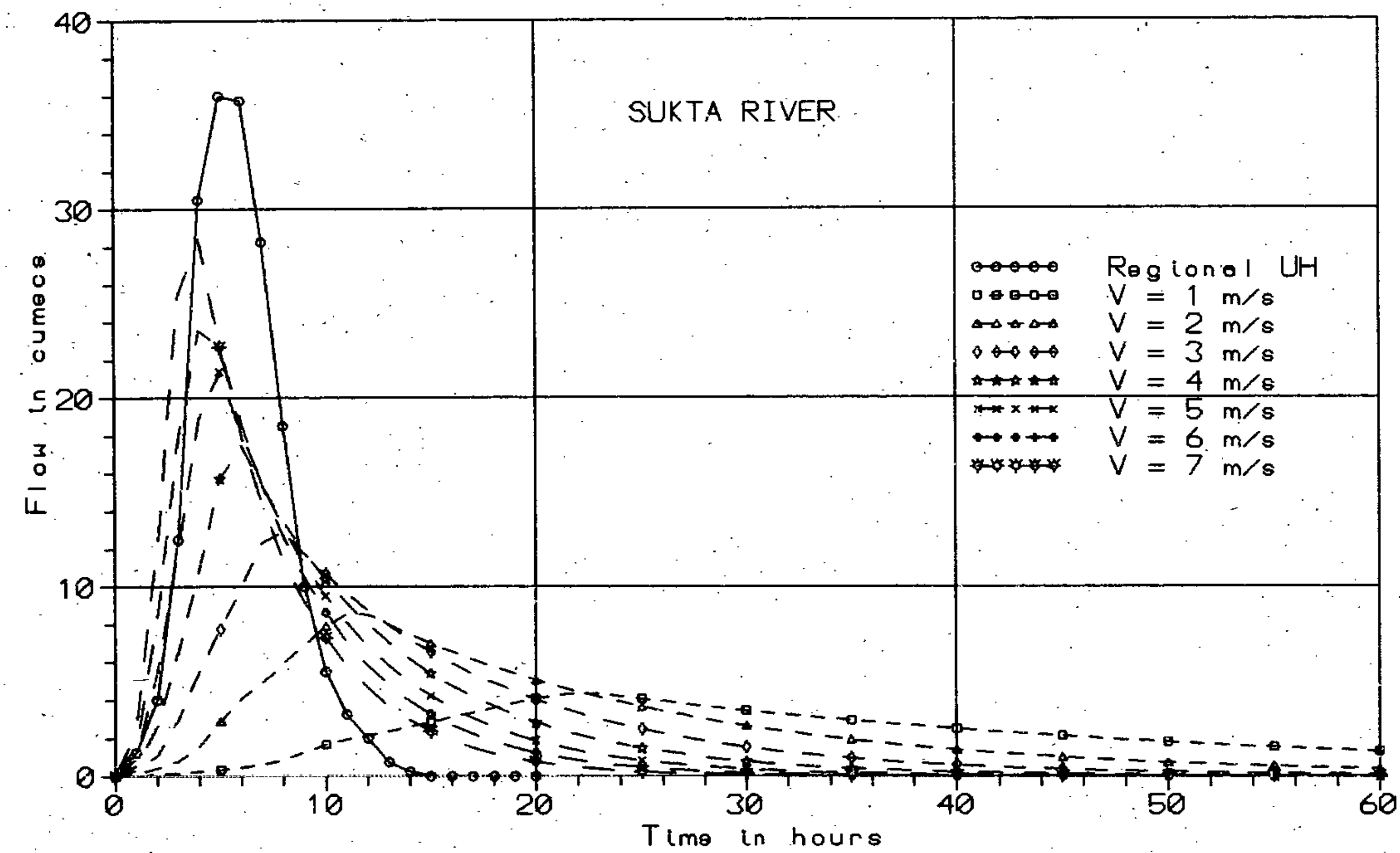

Fig.:7.2.3 : Plot of 1 hr. UHs for Br. catchment No. 578 computed by Regional UH and GIUH based Clark Model approaches.

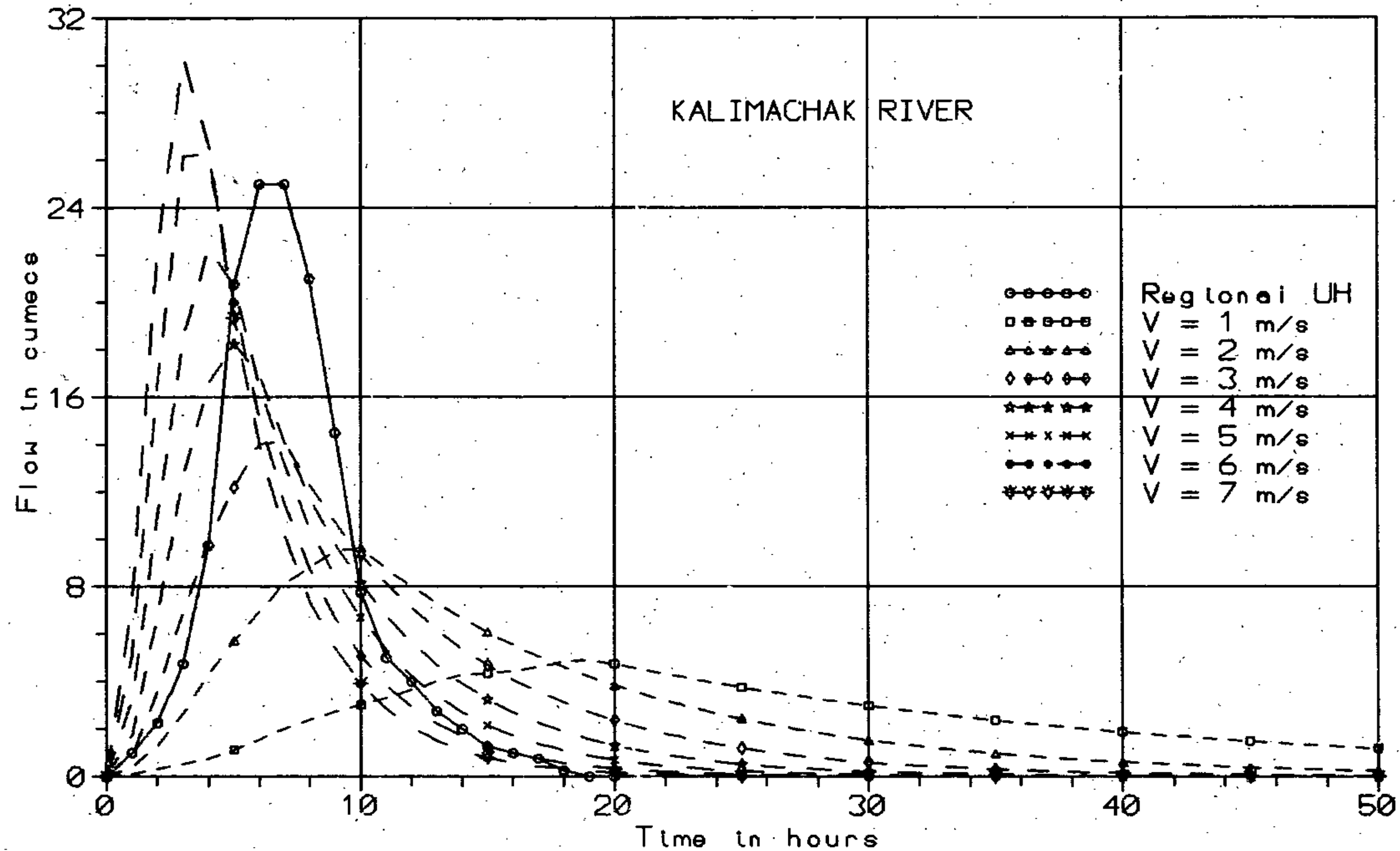

Fig.:7.2.4 : Plot of 1 hr. UHs for Br. catchment No. 625 computed by Regional UH and GIUH based Clark Model approaches.

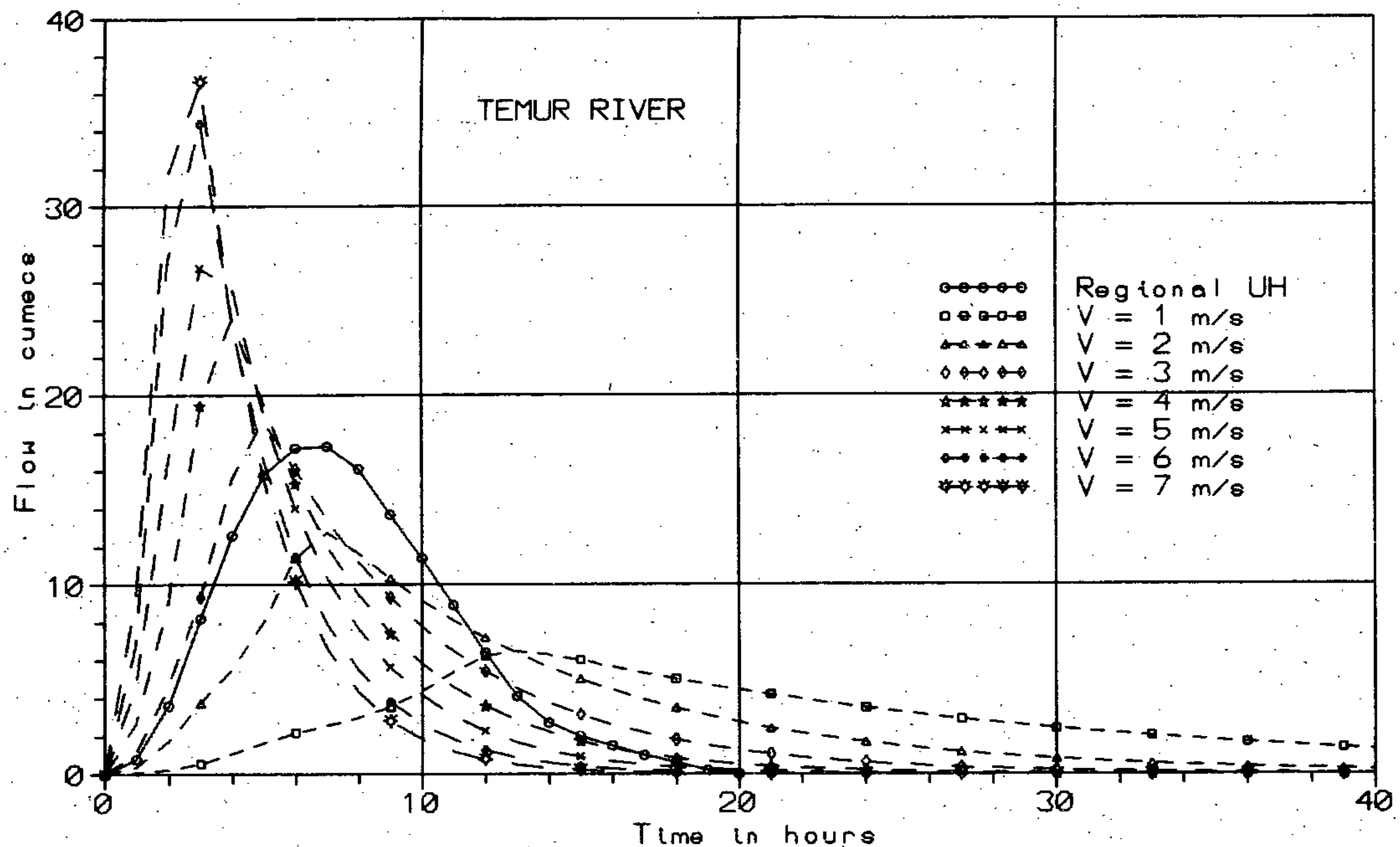

Fig.:7.2.5 : Plot of 1 hr. UHs for Br. catchment No. 249 computed by Regional UH and GIUH based Clark Model approaches.

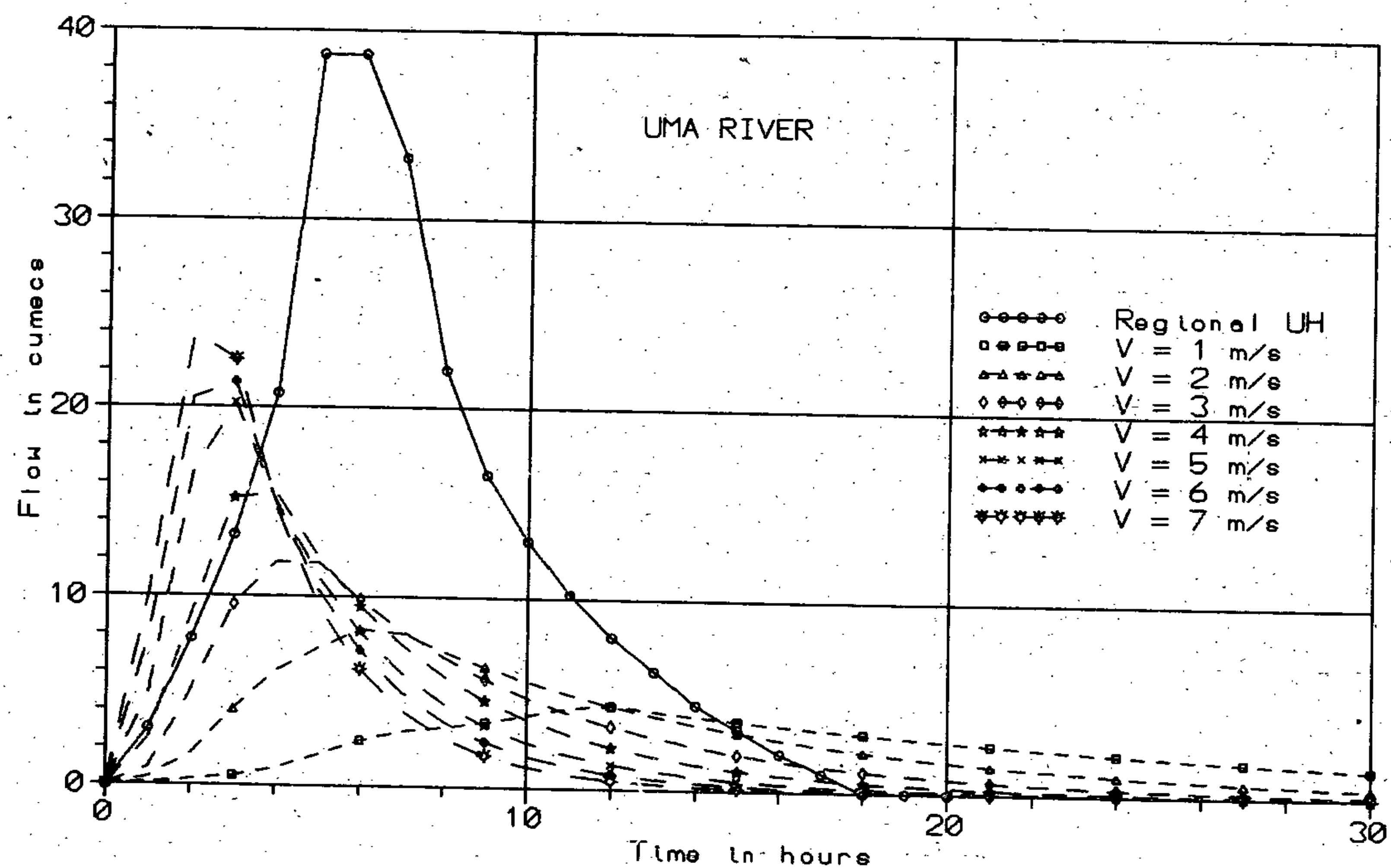

Fig.:7.2.6 : Plot of 1 hr. UHs for Br. catchment No. 394/2 computed by Regional UH and GIUH based Clark Model approaches.

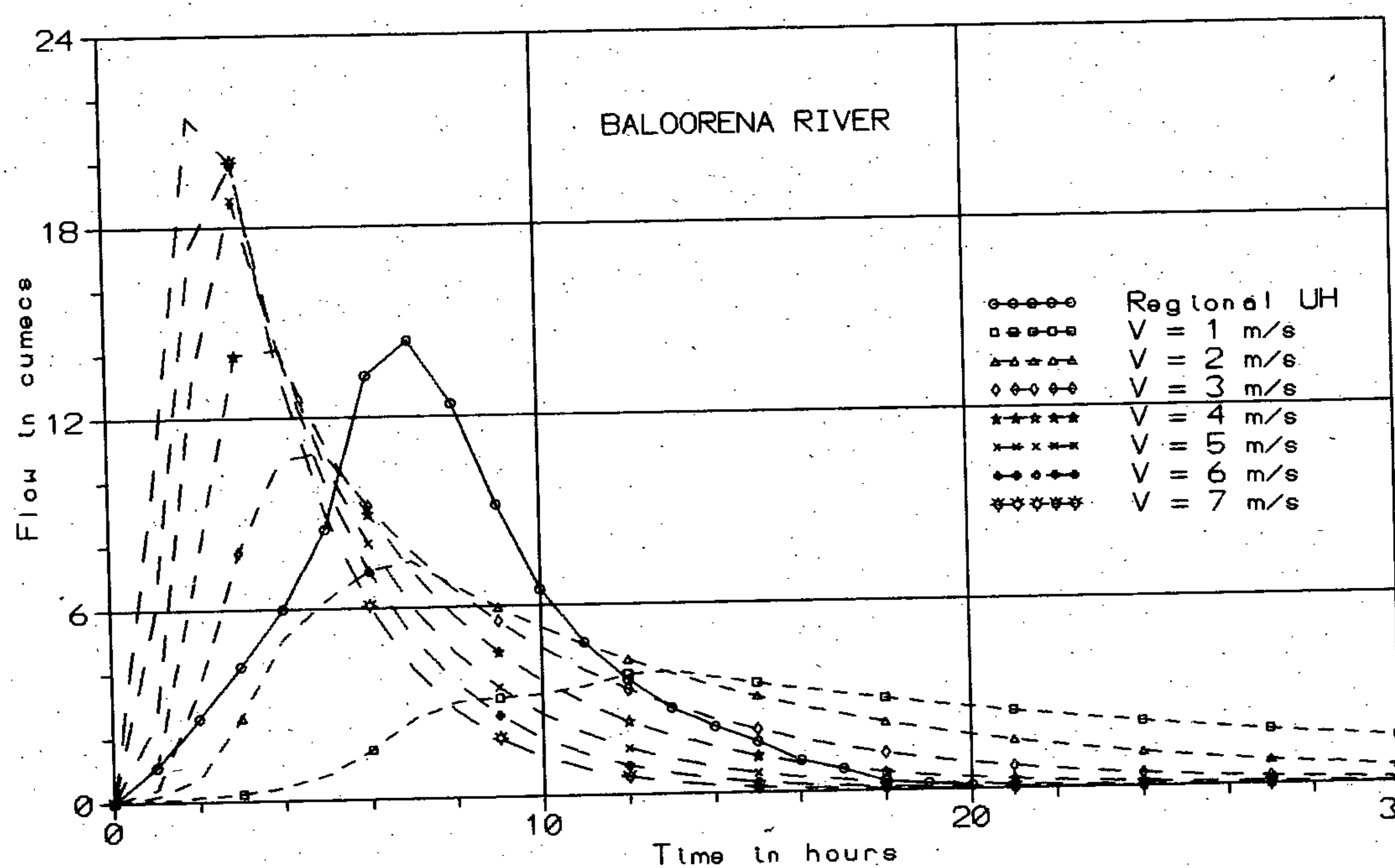

Fig.:7.2.7 : Plot of 1 hr. UHs for Br. catchment No. 897 computed by Regional UH and GIUH based Clark Model approaches.

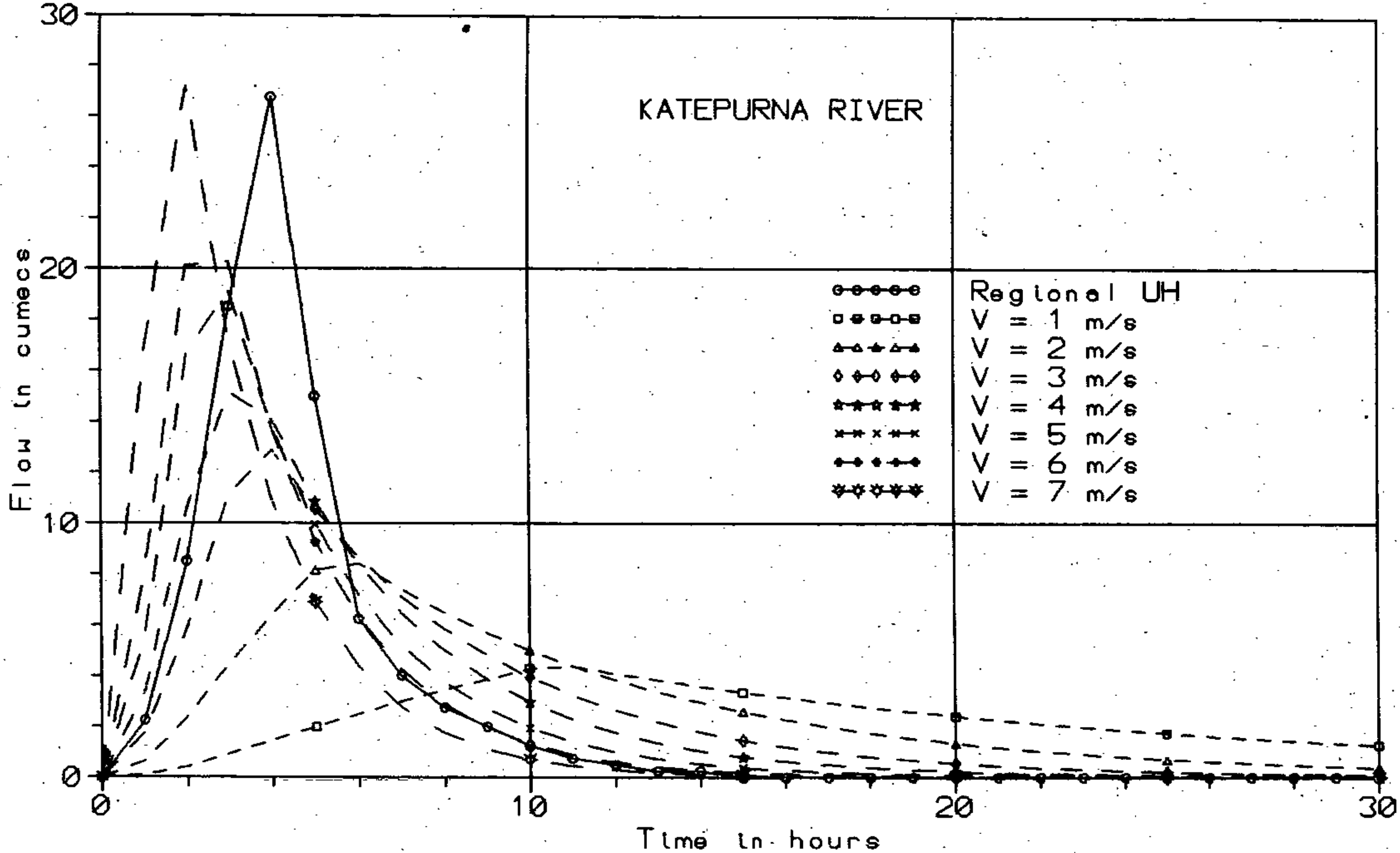

Fig.:7.2.8 : Plot of 1 hr. UHs for Br. catchment No. 787 computed by Regional UH and GIUH based Clark Model approaches.

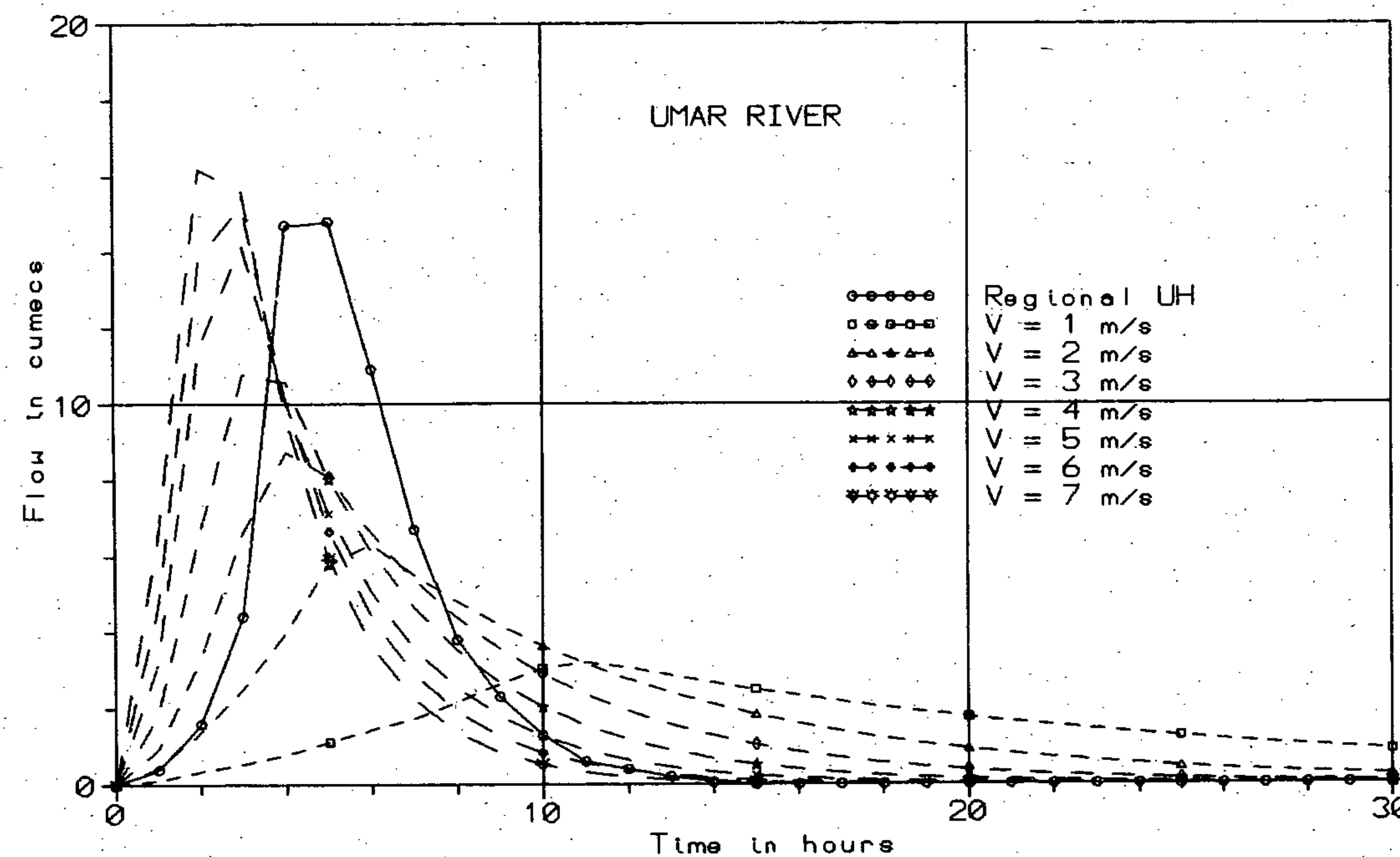

Fig.:7.2.9 : Plot of 1 hr. UHs for Br. catchment No. 930 computed by Regional UH and GIUH based Clark Model approaches.

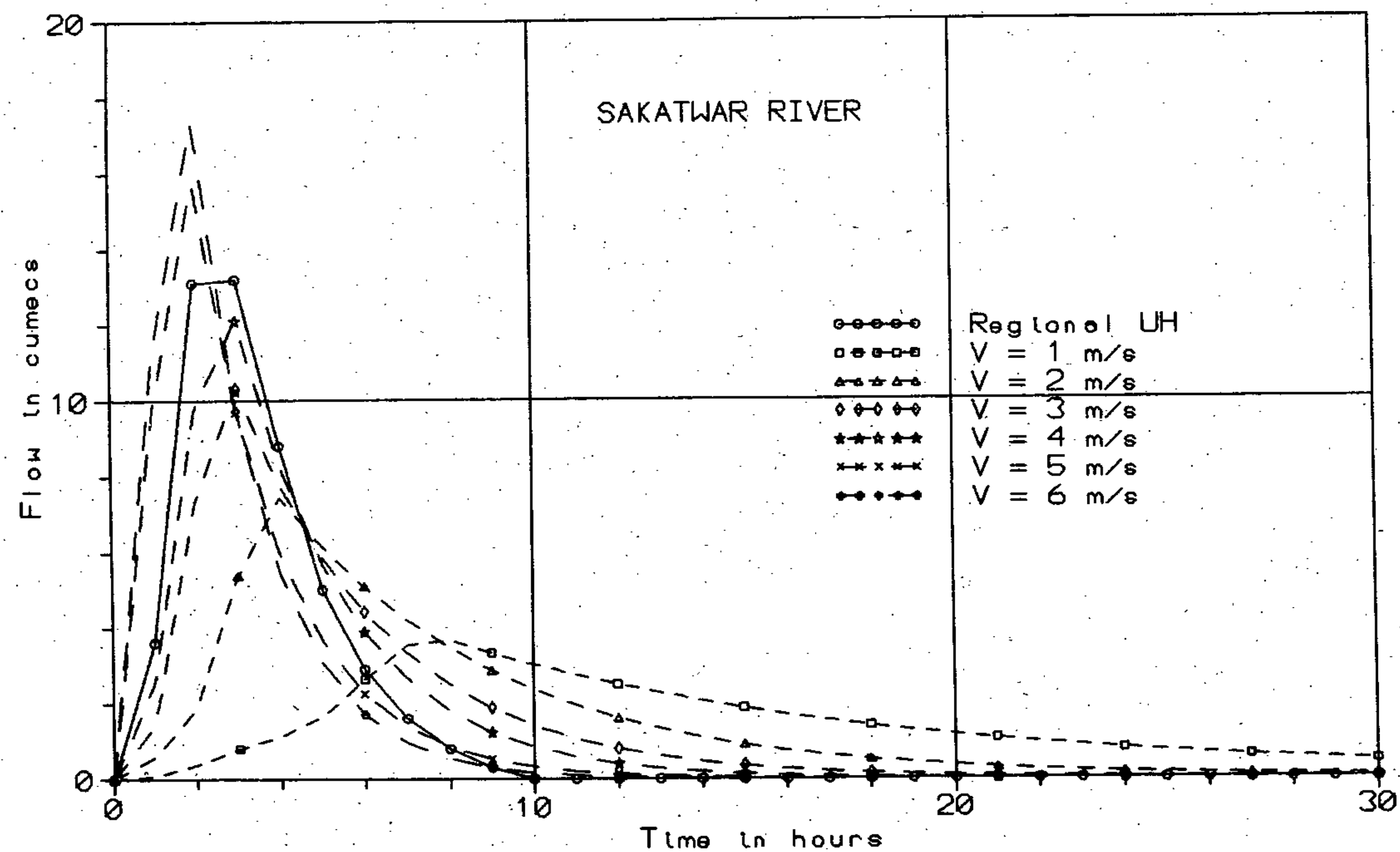

Fig.:7.2.10: Plot of 1 hr. UHs for Br. catchment No. 776 computed by Regional UH and GIUH based Clark Model approaches.

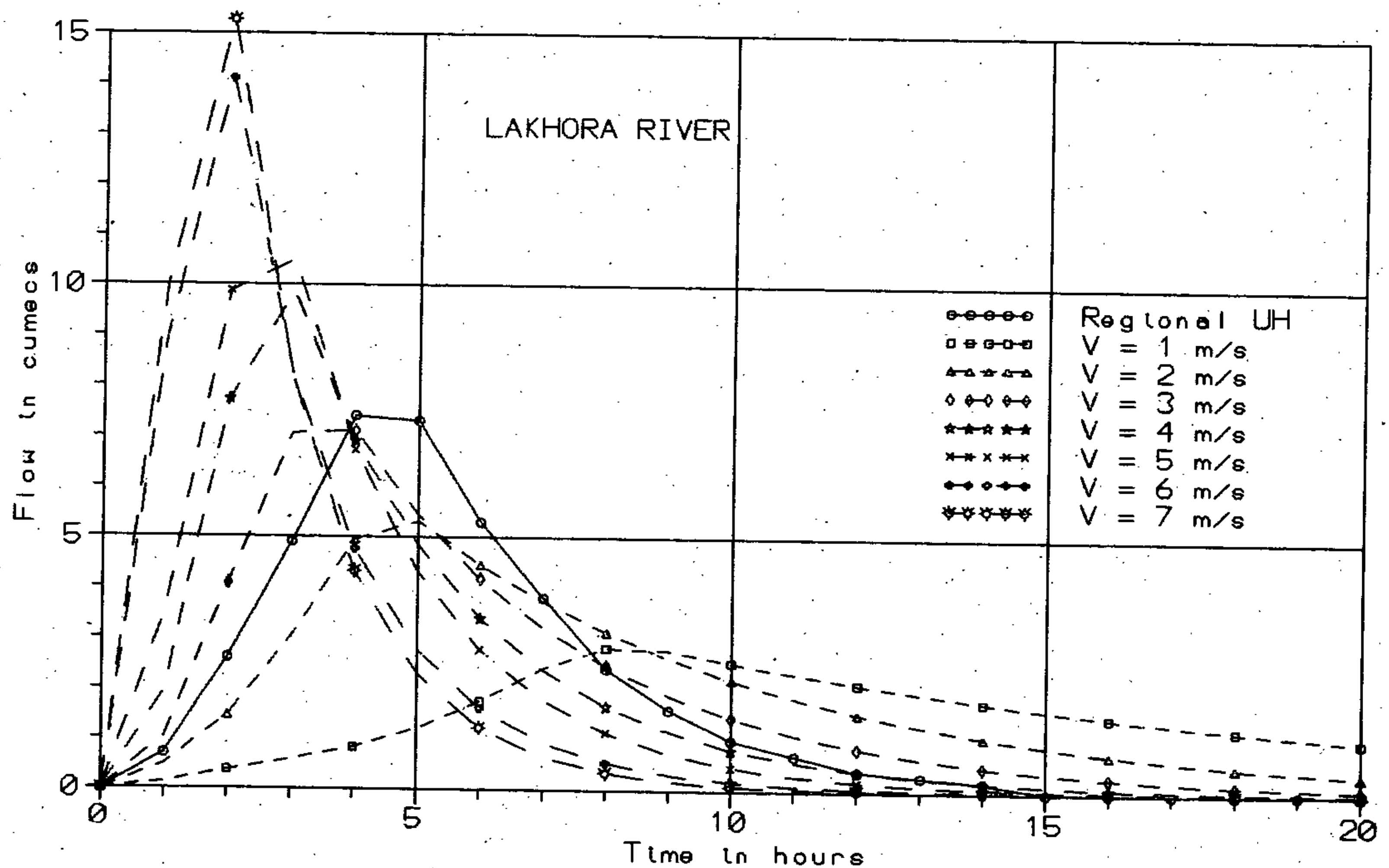

Fig.:7.2.11: Plot of 1 hr. UHs for Br. catchment No. 584 computed by Regional UH and GIUH based Clark Model approaches.

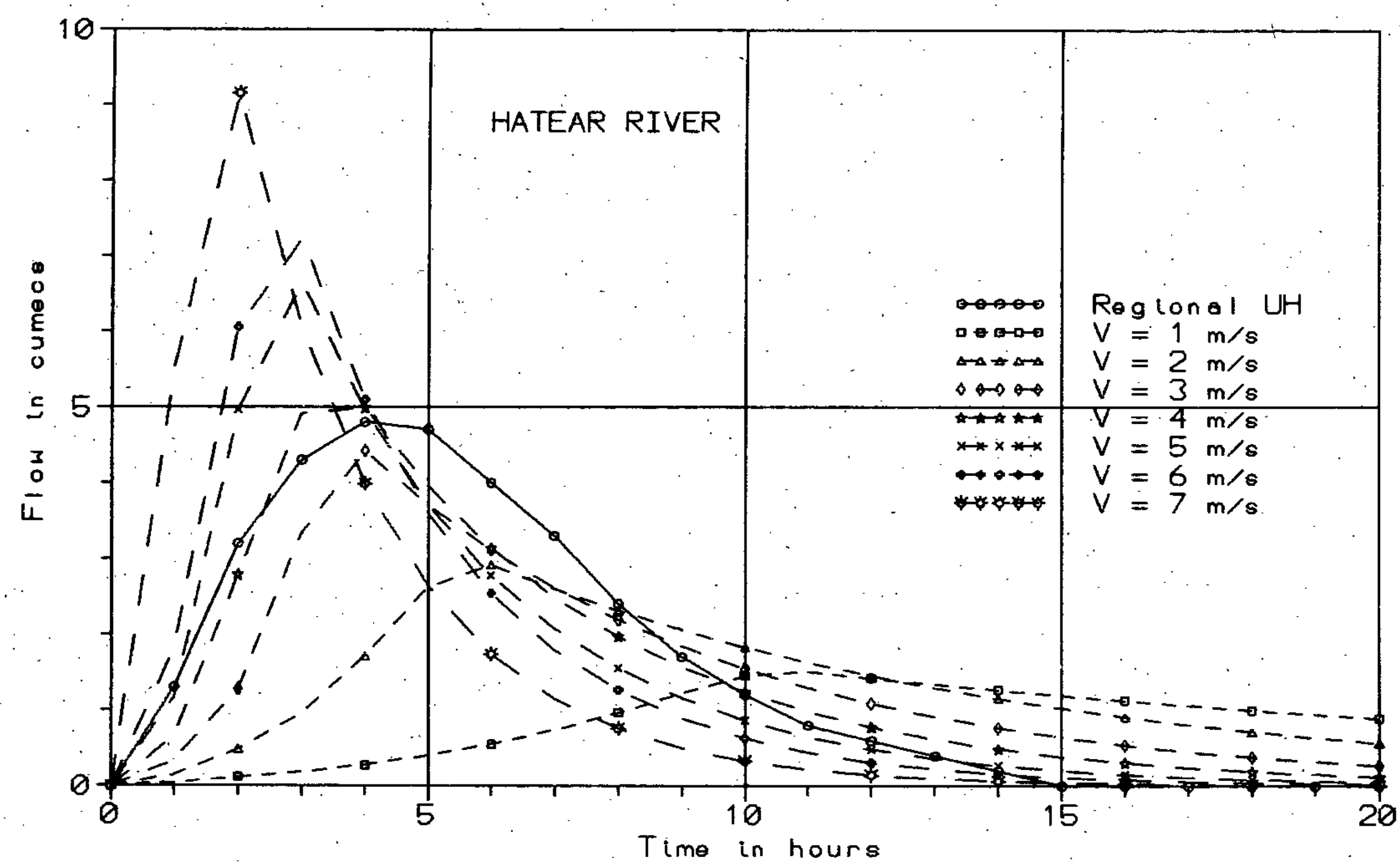

Fig.:7.2.12: Plot of 1 hr. UHs for Br. catchment No. 732 computed by Regional UH and GIUH based Clark Model approaches.

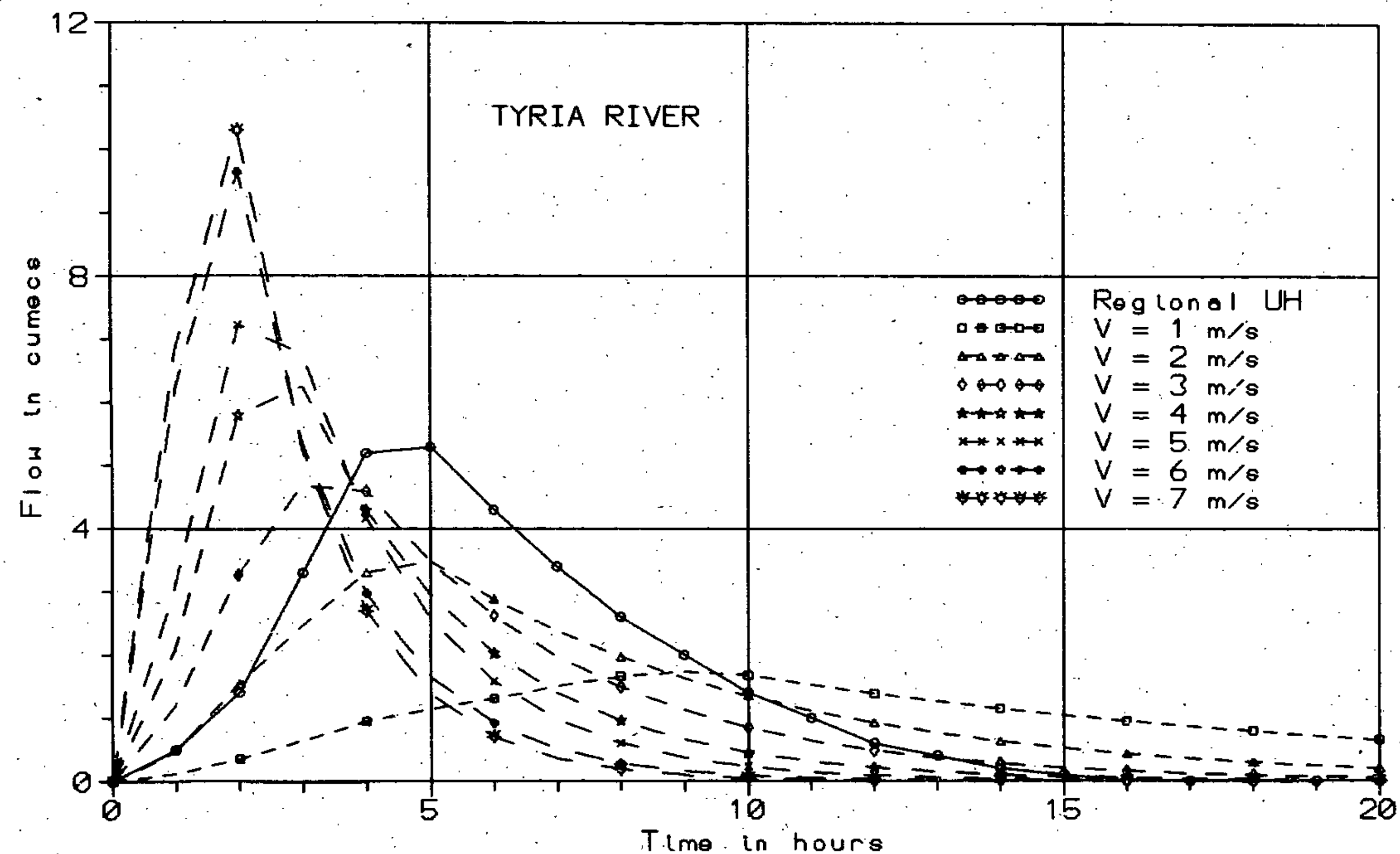

Fig.:7.2.13: Plot of 1 hr. UHs for Br. catchment No. 253 computed by Regional UH and GIUH based Clark Model approaches.

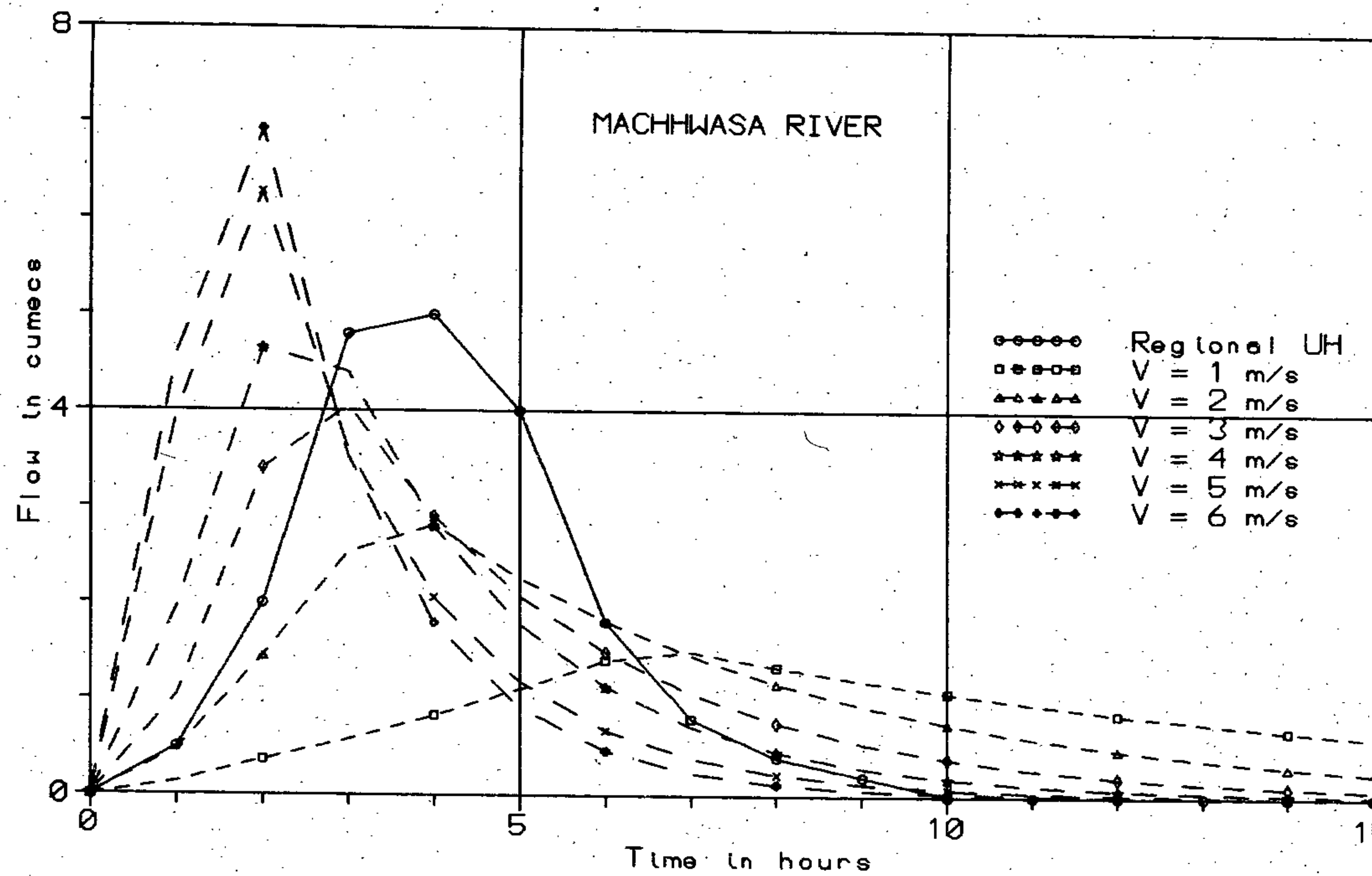

Fig.:7.2.14: Plot of 1 hr. UHs for Br. catchment No. 813 computed by Regional UH and GIUH based Clark Model approaches.

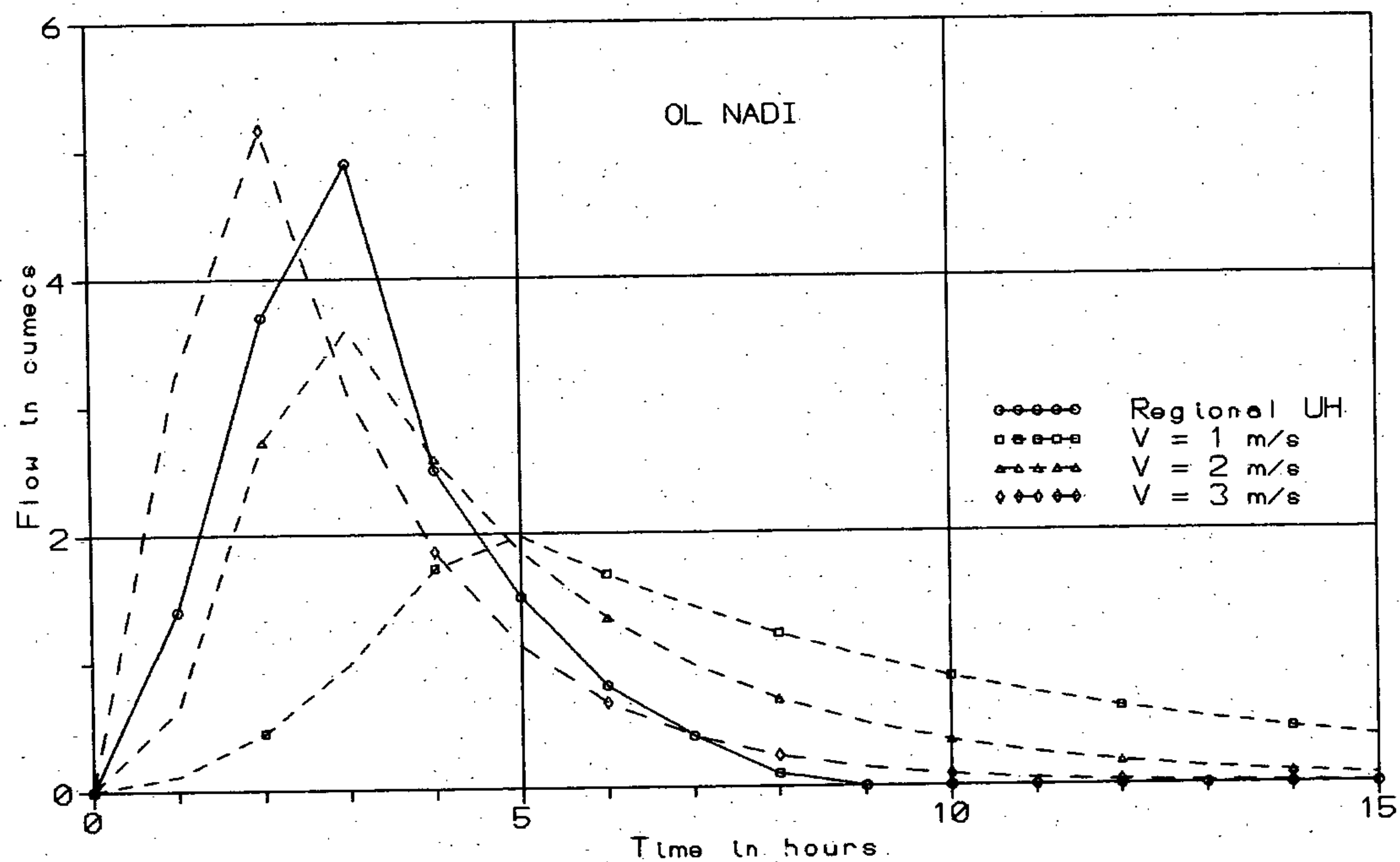

Fig.:7.2.15: Plot of 1 hr. UHs for Br. catchment No. 832 computed by Regional UH and GIUH based Clark Model approaches.

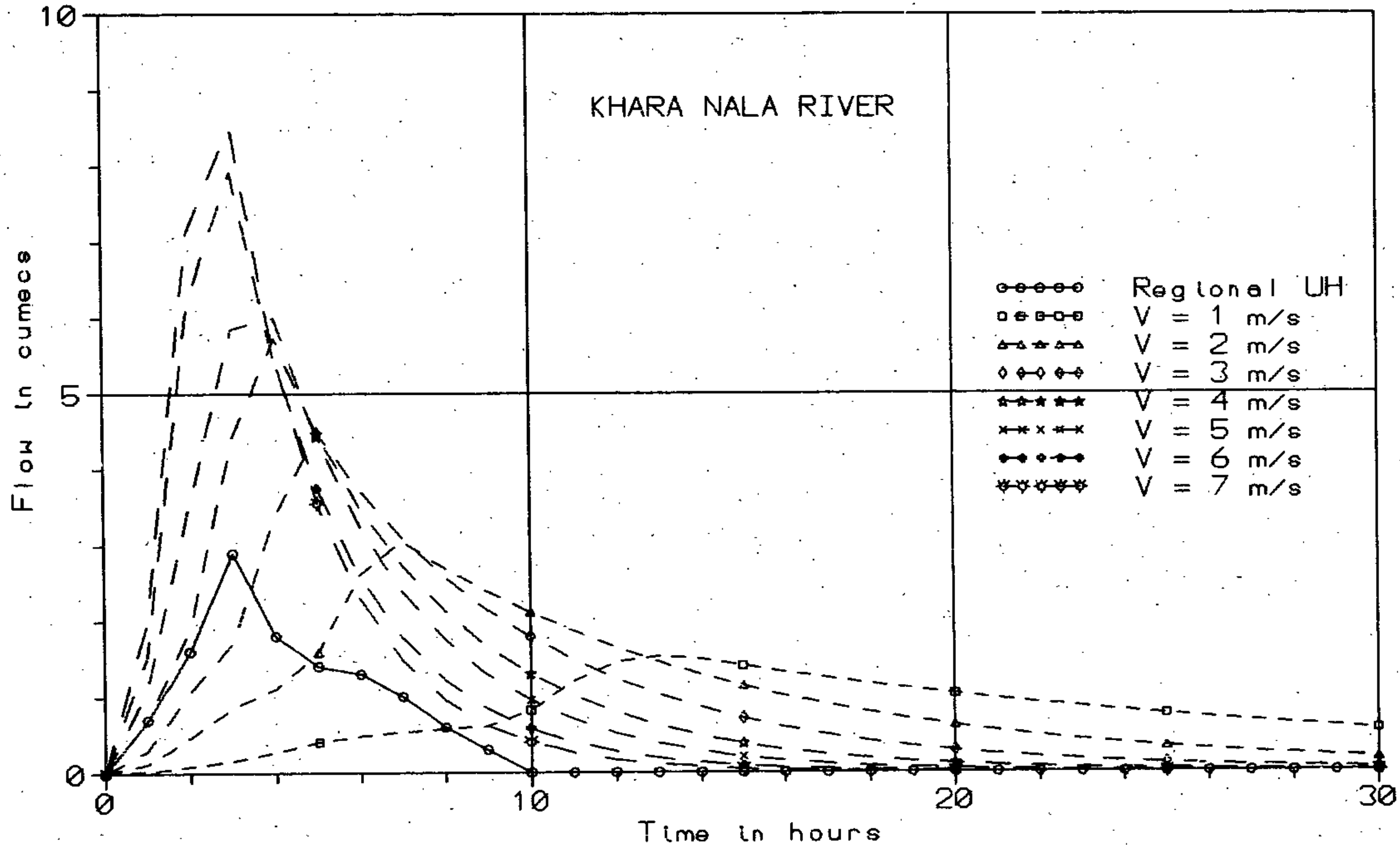

Fig.:7.2.16: Plot of 1 hr. UHs for Br. catchment No. 710 computed by Regional UH and GIUH based Clark Model approaches.

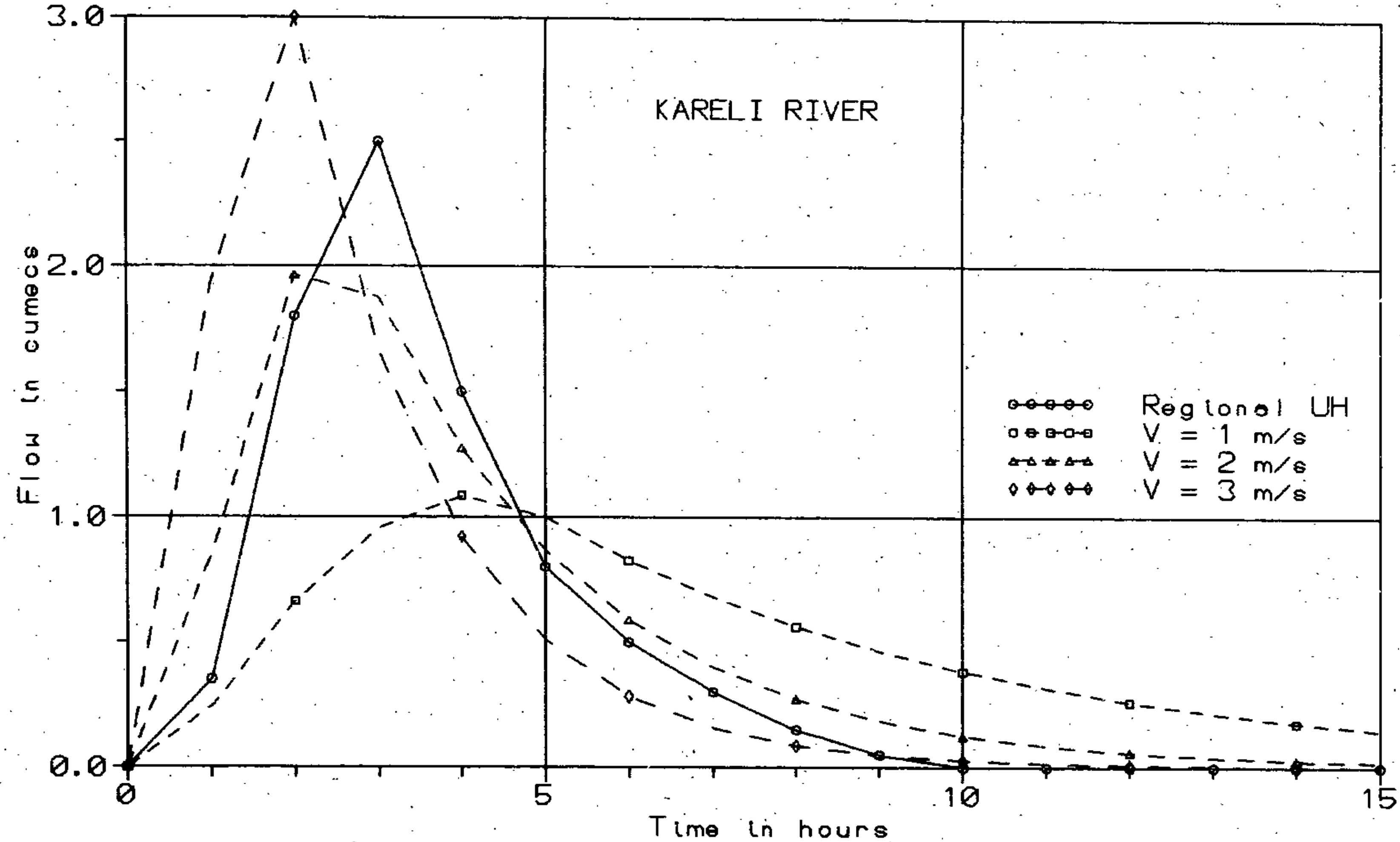

Fig.:7.2.17: Plot of 1 hr. UHs for Br. catchment No. 889 computed by Regional UH and GIUH based Clark Model approaches.

8.0 DISCUSSION OF RESULTS

From Table 7.4 it is observed that the value of T_c and R vary for each event. This may be attributed to the change in storm characteristics from event to event. It may be noted here that this approach is capable of representing the non-linearity in the system caused due to change in storm characteristics. For each individual event the value of the model parameters change even for the same catchment. As is well established, the ratio $R/(T_c + R)$ is constant for each Bridge Catchment.

From the Table 7.3 it is seen that the values of GIUH peak characteristics and Clark Model IUH peak characteristics estimated for computational time interval equal to 0.05 hrs. are very close to each other as compared to that of the Clark Model IUH peak characteristics obtained for the computational time interval is equal to 1.0 hrs. When the computational interval is equal to 1.0 hrs. is considered then the IUH ordinates are obtained at 1.0 hrs. interval only. However, it is not essential that the peak of IUH occurs exactly at the time which is integral multiple of 1.0 hrs. Thus the larger differences in peak characteristics, when interval of 1.0 hrs. is taken, are due to coarser computational interval rather then the incorrect choice of the parameter. The value of T together with the optimum value of parameter R, estimated for minimising the objective function (FCN) evaluated considering the computational interval equal to 0.05 hrs., together with the value of T is used to derive the GIUH based Clark model IUH at a computational interval of 1.0 hrs.

A very important check point in this methodology is to compare the product of peak discharge and time to peak of the IUH obtained by this proposed approach with that given by the eq. (7) which is a non-dimensional characteristic of only the catchment behaviour. This non-dimensional product is thus not dependent on the storm characteristics and is constant for a catchment. It may be seen from the Table 7.3 that the product of the peak discharge and time to peak discharge of the IUH given by the proposed method is very close to the non-dimensional product obtained by eq. (7) for all the bridge catchments. It may be noted here that nowhere in the analysis this non-dimensional number was utilized. Hence, the close conformity of this number with the calculated value proves that the proposed approach is yielding IUH with the correct peak characteristics.

From Table 7.6 it may be seen that the peak and time to peak of regional UH are same for all the events for a bridge catchment, whereas these characteristics change for GIUH based Clark Model UH from storm to storm even for the same bridge catchment. Hence, the unit hydrograph derived by this methodology considers the effect of variable storm characteristics of the events as discussed above.

9.0 CONCLUSION

From this study the following conclusions are drawn :

(1) For each arbitrary storm (represented by expected velocity of flow) the parameters of the proposed GIUH based Clark model could be estimated satisfactorily by using geomorphological characteristics instead of using the observed runoff data, which is not available for the ungauged catchment.

(2) The ratio between storage coefficient (R) and the sum of storage coefficient and the time of concentration (T_c), i.e., $R/(T_c + R)$, has a unique value for a catchment. Thus the value of this ratio may be ascertained for a catchment which may then be used for employing simple Clark model also.

(3) This methodology provides a different unit hydrograph for each event. This shows that the proposed methodology is capable of simulating the non-linear response to different storm events. However, this capability is limited in the sense that the exact relationship between the rainfall pattern and the expected velocity of flow is very difficult to be ascertained.

(4) Further study may be carried out to examine the effects of using the velocity-excess rainfall intensity relationships of the nearby catcnments over the simulation results of various events of different small catchments. Possibility of using a regional velocity-excess rainfall relationship may also be examined.

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