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Development of rainfall – runoff model for extreme storm events in the Bagmati River Basin, Nepal

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Abstract

This study is based on the Bagmati river basin that flows along with the capital city, Kathmandu which is a small and topographically steep basin. Major flood occurring in 1993 and 2002 as stated in the report of DWIDP shows that the basin is subjected to water-induced disaster in monsoon season affecting people and property. This study focuses on the development of a rainfall-runoff model for Bagmati basin in HEC-HMS using the Synthetic Unit Hydrograph (SUH) with Khokana as the outlet. The coefficients for SUH like Lag time coefficient (Ct), peak discharge coefficient (Cp), unit hydrograph widths at 50% and 75% of peak and base time were determined calibrating the Synder's equation where Ct varies from 0.244 to 1.016 and Cp varies from 0.439 to 0.410. The rainfall-runoff model in HEC-HMS has been calibrated from daily data of 1992-2013 and validated from hourly data for July 2011, August 2012, and July 2013. Furthermore, the model has been tested to compare the discharge for various return periods with the observed ones which are in close agreement. The determination of Peak Maximum Flood (PMF) using the calculated Peak Maximum Precipitation (PMP) is also another application of the model which can be used to design various hydraulic structures. Thus the values of coefficients, Ct and Cp can be used to construct unit hydrograph for the basin. Moreover, the satisfactory performance of the model during calibration and validation proves the applicability of the model in flood forecasting and early warning.

Keywords: *Bagmati basin, event-model, HEC-HMS, unit hydrograph*

1. Introduction

While majority of the basins worldwide are ungauged (Hrachowitz et al., 2013), hydrological behavior in ungauged basins are poorly known. It has implications on estimating runoff, water availability, flood events, and inundation depth/extent. A rainfall-runoff model usually produces the runoff hydrograph as a response to a rainfall hyetograph as input (Beven, 2012). The actual shape and timing of the response hydrograph for a particular watershed is a function of many physiographic, land use, and climatic variables (Chow, 1988).

Several efforts are made over the years to overcome the prediction and forecasting challenges in ungauged basins. They include but not limited to developing a wide variety of data acquisition techniques: as improving rainfall-runoff measurements using real-time monitoring, weather radars, and satellites (Li & Shao, 2010) and enhance the capability of hydrological modeling by using geographic information system (GIS) (Jain et al., 2004). Though those efforts have resulted in a set of new tools and methods, putting the approaches of prediction in ungauged basins to practice is still a challenge (Efstratiadis et al., 2014).

A variety of simulation models, ranging from simple empirical models (e.g. transfer function, data-driven, regression, etc.) to complex physically-based distributed models are available for generating a watershed response in both gauged and ungauged basins. Simulation using physically–based models are generally better compared to lumped and empirical models (Hughes, 2010), as they can account for the system heterogeneity and simulate the hydrological process in a watershed (Cibin et al., 2014). However, in the case of ungauged locations, a synthetic UH is a simple and effective method of the rainfall-runoff simulation (Fedorova et al., 2018). The UH can then be used as a transfer function to transfer rainfall into runoff using appropriate hydrological models (Saghafian, 2006).

When there is no observed long-term discharge, prediction of runoff is more challenging to practical applications such as design of drainage infrastructure, flood forecasting, and for watershed management tasks such as water allocation and climate impact analysis. In the case of ungauged river basin, one of the options is to develop Synthetic UHs for runoff prediction, flood forecasting as well as early warning system (Sherman, 1932). To develop UH to a catchment, detailed information about the rainfall and the resulting flood hydrograph is needed. However, such information would be available only at a few locations, and in a majority of catchments especially those which are at remote locations, the data would normally be very limited. In order to construct UHs for such areas, empirical equations of regional validity which relate the hydrograph characteristics to the basin characteristics are to be available. Unit hydrographs derived from such relationships are known as synthetic – unit hydrographs (Bhunya, 2011). Some of the physical characteristics of the watershed to develop hydrograph includes peak flow rate (Qp), time to peak (tp), time base (tb), and width of unit hydrograph at 0.5Qp and 0.75Qp (i.e., W0.5 and W0.75), respectively. In addition, simultaneous adjustments are required for the area under the synthetic UH to be unity. There are three types of synthetic UHs. One among them relates hydrograph characteristics (peak flow rate, base time, etc.) to watershed characteristics (Snyder UH), while another one is based on a dimensionless unit hydrograph (Soil Conservation Service), and last one is based on the model of watershed storage (Clark Unit hydrograph). However, these relations are quite empirical and as such cannot be expected to be universally applicable. Their applications, in general, should be restricted to the region in which they have derived Hoffmeister et. al. (1977) developed a synthetic UH for an un-gauged basin in New Zealand and Sudhakar et al. (2015) tested three different methods viz. Snyder method, Common's dimensionless method, SCS dimensionless hydrograph for India. The study concluded that Snyder UH method gives the best results as compared to later ones. Due to lack of hourly measured rainfall and runoff long term data our work also applies the Snyder UH. It requires only basin characteristics as input. The basin characteristics of the watershed are extracted from the HCE – GeoHMS model, a data preparation tool for HEC – HMS (Hydraulic Engineering Center – Hydrologic Modelling System) model.

For the city basin like Baghmati, the density of the observation network is spare but issues of flooding and inundation prevail due to extreme rainfall, topography, blockage of natural drainage, and poor planning/ design/construction practices. Such issues are more frequent in highly urbanized Kathmandu Valley, the capital city of Nepal. In such a case, we can develop a synthetic hydrograph by calibrating the continuous rainfall data and validating with different rainfall events to make the model applicable for predicting flooding/runoff at several ungauged locations in the urban area. The objectives of this study are: i) to develop a synthetic UH for an ungauged basin; ii) to develop the rainfall-runoff model using HEC-HMS and iii) evaluate the applicability of the model for estimating flood of different return periods as well as probable maximum flood using the developed model.

2. Materials and Methods

2.1 Study Area

The Kathmandu Valley, located in the upper part of the Bagmati River Basin is chosen as a system or basin boundary. It originates at Shivpuri Mountain (2679m) and draining the Kathmandu Valley, the river flows through the Middle Mountains and Siwalik range before entering into Terai (see Fig.1). The basin area above Khokana is 612 km2 and the elevation of the basin varies from 1260 m to 2679 meters above the mean sea level (masl). The average length of the river up to Khokana is 35 km and the average gradient is 0.0025. There are several tributaries of different orders originating from the middle mountains that feed the Bagmati River. These are Manahara, Dhobi Khola, Bishnumati, Balkhu Khola, Hanumante Khola, and Nakkhu Khola. The basin always faces the problems of flash floods and inundation during the rainy season, which cause severe human and property losses. The real-time rainfall and runoff observations are therefore essentially required in the basin to conduct flood and inundation predictions. This study focused on tributaries of the upper urban reach of the Bagmati basin and developed a rainfall-runoff model and analyzed/ interpreted the simulated results.

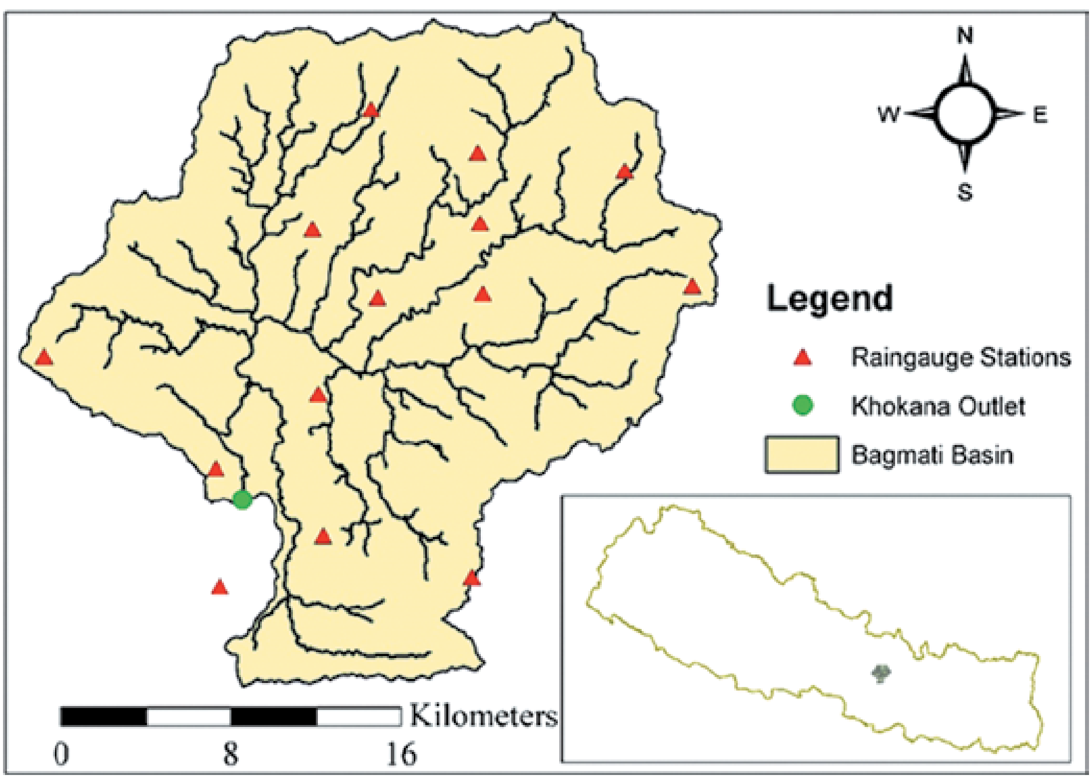

Figure 1: Location and associated details of the Kathmandu Valley watershed in the uppermost part of the Bagmati River Basin

2.2 Development of Synthetic UH

As shown in the overall methodological framework (see Fig.2), Synthetic UH methods were utilized to determine runoff hydrograph for ungauged sites. The physical parameters of sub-basins (i.e., area of sub-basin, length of the river, centroidal length, the slope of the river) were extracted from HEC – GeoHMS and processed further in MS –Excel program for generation of synthetic UH.

The synthetic UH of Snyder is based on relationships between the characteristics of a standard UH and basin morphology. Snyder formulates an equation for the effective rainfall duration (Tr), the peak direct runoff rate (qp), and the basin lag time (Tl). From these relationships, the following five characteristics of a required UH for a given effective rainfall duration was calculated (Chow, 1988): the peak discharge (qp), the basin lag (Tl), the base time (Tb), and the widths, W (in time units) of the UH at 50 and 75 percent of the peak discharge. In this particular study, the model parameters we seek to estimate are Snyder's equations coefficient, namely, coefficient of slope (Ct) and coefficient of peak (Cp). The method calibrates the Snyder coefficients (i.e., Ct and Cp) such that direct runoff must be 1cm or within 10% relative error and the observed hourly maximum discharge and calculate peak discharge within 10% difference. The calculated peak discharge is obtained by multiplication of ordinate of Synthetic UH and point rainfall depth. In addition, this method determines the peak discharge, lag time, and time to peak by using characteristic features of the watershed. Also, the lag time is compared with the time of concentration (Tc) such that Tl range from 50 to 75 percentage of Tc. It is generally taken as 60% of Tc (USACE, 2005). The Tc is calculated from the Kripich formula. Snyder's equations are described as follows:

- Basin Lag (tp): $tp = 0.75Ct\ (L \times Lc)^{0.3}$; where L is Length of stream, and Lc is centroidal length of stream.
- Peak Discharge (Qp): $Qp = \dfrac{2.78 \times Cp \times A}{Tp}$; where Cp is the Coefficient of the peak, A is Basin area and Tp is time to peak.
- Base period (Tb): $Tb = 5\ (Tp + D/2)$; where Tp is time to peak and D is unit duration.
- Unit Duration (Tr): $Tr = Tp / 5.5$
- Correction for actual Duration: $Tl1 = Tp + (D - Tr)/4$
- Width at 50% and 75% of peak discharge (W50 and W75): $W50 = 5.9 / (qpl)^{1.08}$ & $W75 = 3.4 / (qpl)^{1.08}$

2.3. Hydrological Model Development

2.3.1 Selection of a hydrological model

The HEC HMS model was chosen for developing the hydrological model as it has the capability of simulating the rainfall-runoff process for dendritic watersheds in space and time (Oleyiblo & Li, 2010). Since the old version of HEC – HMS being lump model, 4.2.1 version of HEC – HMS model is used as a semi-distributed model. This model is used widely across the globe to simulate runoff to a wide variety of watersheds. The semi-distributed models evaluate basin response by partially representing spatial heterogeneity by dividing the basin into several sub-basins, depending upon the resolution of available input data (HMS, 2000). HEC – HMS model has been used successfully in different parts of the world, including Nepal, for catchment modeling. Some examples include determining hydropower potential (Prajapati, 2015), climate change impact assessment (Babel et al., 2014), and development of the rainfall-runoff model (Khadka & Bhaukajee, 2018).

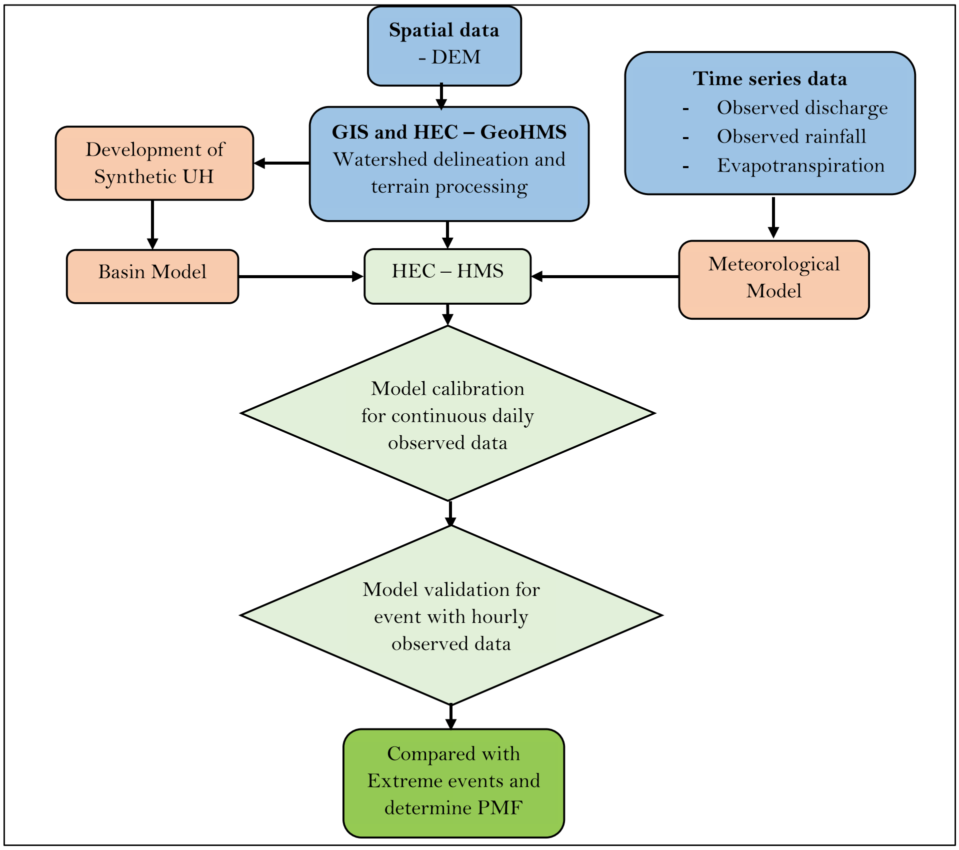

Figure 2: Methodological framework adopted in this study

2.3.2 HEC – HMS model set – up

Basically, HEC-HMS requires four model components: Basin model, meteorological model, control specifications, and input data (time series, paired data, and gridded data). The Basin model describes different elements of the hydrological system (subdivision, reaches, junction, sources, sinks reservoir, and diversion) and their connectivity that represent the movement of water through the drainage system (Guide & Manual, 2008).

As present in Fig.2 the HEC – GeoHMS extension in ArcGIS was selected to derive topography-related inputs to HEC-HMS. It usages the spatial analyst extension in ArcGIS to develop hydrologic parameters as input data for HEC – HMS (Vizina & Hradilek, 2012). Analyzing the digital terrain information, HEC – GeoHMS transforms the drainage paths and watershed boundaries into a hydrologic data structure that represents the watershed response to precipitation. The spatial analyst extension was used for terrain pre-processing based on digital elevation model (DEM) and stream data. A terrain model was used as an input to derive eight additional data sets that collectively describe the drainage patterns of watersheds and allows for a stream of sub-basin delineations. The first five data sets in grid representation for the flow direction are DEM reconditioning, fill sink, flow accumulation, stream definition, and stream segmentation. The next two data sets are watershed polygons and the drainage line processing. The last one is aggregated watershed. Outputs after terrain preprocessing serve as a spatial database for the study.

After terrain processing, the HMS project is generated for the study area. The stream gauging station at Khokana (550.05) is considered as the control point for the project generation. Then the sub-basins automatically generated during terrain processing are merged and added at major river junctions. The study area is divided into 13 sub-basins as indicated in Fig. 3. The spatial-temporal precipitation distribution was calculated by the gauge weight method. The Thiessen polygon technique was used to determine the gauge weights, the area is shown in Table 1 and the following input data were used for the meteorological module: daily precipitation, daily temperature, elevation, and long-term mean monthly actual potential evapotranspiration. And finally, the HMS basin model was used to generate HEC – HMS model including a background map (see Fig. 3).

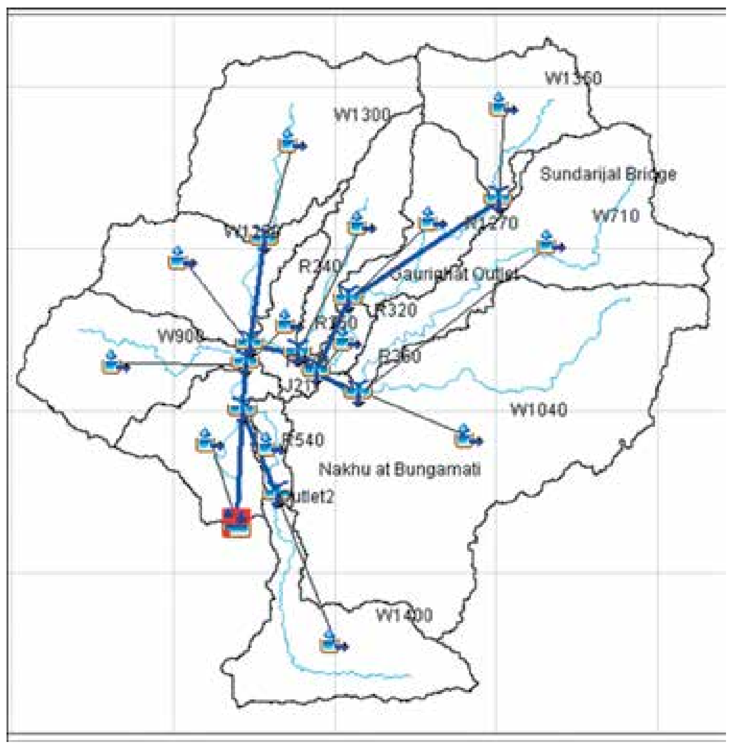

Figure 3: HEC - HMS model set-up

HEC – HMS uses a separate model to represent each component of the runoff process. For every sub-basin and reach all the hydrological parameters have been initially estimated and simulated the models for observed boundary conditions to compute out either the watershed runoff hydrograph or a channel outflow hydrograph. Then the computed hydrograph is compared with the observed hydrograph to evaluate how well the model "fits" the real hydrological system. The model parameters were adjusted until satisfactory results are obtained.

While setting up the HEC-HMS model, the following methods were selected for various hydrological processes.

Loss Method: After the occurrence of precipitation, the loss method controls the partitioning between intercepted water, infiltrated, and the water that leaves the catchment as direct runoff. Water that survives a loss method leaves the catchment as quick flow. The loss method used in this study is the Deficit and Constant, which is a quasi-continuous model of precipitation loss where initial loss can recover after a prolonged period of no rainfall and is most suitable for continuous simulation (Majidi & Shahedi, 2012). The parameters of this method are constant rate, initial deficit, and maximum deficit.

Table 1: Thiessen area of rain gauge stations

Rain gauge station	Thiessen Area sq. km	Thiessen Weightage
1015	45	0.074
1022	48	0.079
1029	53	0.087
1030	30	0.049
1035	40	0.066
1039	73	0.120
1043	41	0.067
1052	24	0.039
1059	61	0.100
1060	48	0.079
1071	59	0.097
1073	32	0.052
1074	44	0.072
1075	12	0.020

Direct runoff method: The transformation method controls the channel surface runoff concentration time. Water concentration is recorded in a hydrograph, thus transformation methods attempt to build the right hydrograph using catchment characteristics (Halwatura & Najim, 2013). In this study, the Snyders user-specified UH method was selected. Objective functions of percent error in peak and volume were used to determine the best fit between observed and simulated hydrographs. The objective function of percent error in peak only shows the well-fitting of peak discharge of simulated and observed hydrographs. The objective function of percent error in volume calculations is based on just volumes of observed and simulated hydrographs.

Base flow method: Base flow is influenced by groundwater and is closely related to watershed characteristics. The recession model has been used often to explain the drainage from natural storage in a watershed (Linsley et al., 1982). It defines the relationship of Qt, the base flow at any time t, to an initial value as:

Qt = Qokt Error! Bookmark not defined.

Where Qo = initial base flow and k = an exponential decay constant.

In the HMS model, the variables for the base flow method by the recession are initial discharge, ratio to peak, and recession constant.

Reach routing: A channel or reach s an element with one or more inflow and only one outflow. Inflow comes from other elements in the basin model. The routing models included in HEC – HMS program are the fundamental equations of open channel flow (the momentum equation and the continuity equation). Together the two equations are known as the St. Venant equations or the dynamic wave equations. The momentum equation accounts for forces that act on a body of water in an open channel. In simple terms, it equates the sum of gravitational force, pressure force, and friction force to the product of fluid mass and acceleration. In one dimension, the equation is written as:

Sf = So - dydx- Vgdvdx- 1gdvdt

To solve this equation we have used the Muskingum - Cunge method (Barry & Bajracharya, 1995).

2.3.3 Model calibration and validation

Automatic calibration in conjunction with manual calibration was used to determine a practical range of the model parameter values preserving the hydrograph shape and minimum error in volume. Nelder and Mead optimization method (Nelder & Mead, 1965) were used as automatic calibration algorithm, which aims to minimize a specific objective function, such as the sum of the absolute error, the sum of the squared error, percent error in peak and peak weighted root mean square error (Guide & Manual, 2008). This study selected the sum of squared error objective function for automatic calibration.

Daily river discharge data for the period of 1992 - 2013 at Khokana (550.05) was used for model calibration. And the model was validated with storm event by real-time hourly data in the years 2011, 2012, and 2013. The objectives of the model calibration and validation were to match simulated volumes, peaks, and timing of hydrographs with the observed ones.

2.4 Rainfall Frequency Analysis

For evaluating the applicability of UH, various scenarios were evaluated using rainfalls of different return periods. Rainfall frequency in the study area was analyzed based on rainfall data at 15 stations spread across the basin. Hydrognomon software was used for frequency analysis (Kozanis et al., 2010). With the best fit statistical method 5, 10, 25, 50, 100, 500,1000,10,000 year return period rainfall data have been calculated taking input as daily time series data. Extreme Value (EV) 1 – Max (Gumbel), Log Pearson – III, Extreme value method, and Lognormal distribution were used to determine extreme values. From Kolmogorov – Smirnov Test in Hydrognomn software, EV1 – Max (Gumbel) gave a good performance (98.6%) than other methods. Therefore, results from the Gumbel method were finally adopted.

2.5 Probable Maximum Precipitation and Flood Estimation

According to WMO (2009), probable maximum precipitation (PMP) can be defined as the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area a particular time of year, with no allowance made for long – term climate trends. Several techniques of PMP estimation are available based on the availability of rainfall data, catchment size and location, and meteorological conditions responsible for extreme rainfalls. A simplified statistical method developed by Hershfield (1961), recommended by the WMO (2009), and used widely across the globe is adopted in this study to estimated PMP. As per this method, PMP is estimated as;

$$PMP = \bar{X}n + K \times Sn$$

Where $\bar{X}n$ is the mean of maximum daily annual rainfall sample, Sn is standard deviation, and K is a factor depending on $\bar{X}n$ and estimated using the following equation:

The PMP thus estimated was subject to adjustments of mean to maximum rainfall, standard deviation (SD) to maximum rainfall, and mean and SD to the length of a data record as per the guidelines in WMO (2009a). In this case, the adjustment factor for mean was 1.01, adjustment factor of SD was 1.06, and adjustment for area reduction factor was 1.1.

The probable maximum flood (PMF) value, as well as hydrograph, was estimated by feeding PMP as rainfall input to the event model. In addition, floods of different year return periods were also estimated by feeding rainfall of different return periods.

2.6 Data and Sources

Both geo-spatial and time-series data were used in this study as elaborated in Table 2.

Table 2: Data and sources used in this study

Data set Unit	Data type	Data description/ Properties	Data Source	Resolution (Time Frame)
Terrain (m)	Spatial Grids	Digital Elevation Model (DEM)	EarthExplorer (usgs.gov)	30m × 30m grids
Stream (m)	Spatial vectors	Stream network and its physical properties (e.g. Length, gradient)	Generated from DEM	
Precipitation (mm)	Time - Series	Daily observed precipitation Hourly observed precipitation	Department of Hydrology and Meteorology (DHM), Nepal	15 stations (1992 - 2013) 5 stations (2011 - 2013)
Evapotranspiration (mm per day)	Time - Series	Monthly evapotranspiration data downloaded from CLIMWAT 2.0 tools	http://www.fao.org/nr/water/infores_databases_climwat.html	3 Stations (Mean monthly)
River Discharge (m^3 per sec)	Time - Series	Daily Observed discharge and instantaneous maximum Discharge Hourly observed discharge	DHM	1 Station (1992 - 2013) 1 Station (2011 - 2013)

3. Results and Discussion

3.1 Unit Hydrograph for the Study Area

The calibrated Ct and Cp depend on the physical parameters of the watershed and are also defined as regional coefficients. After satisfying the required criteria, the value of Ct and Cp were calibrated for each sub-basin. The physical parameters extracted from HEC – GeoHMS for each sub-basins and calibrated Ct and Cp for each sub-basin are shown in Table 3. The calibrated value of Ct ranges from 0.244 to 1.016 and Cp ranges from 0.410 to 0.439. According to Subramanya (2017) and Acanal (2021) the coefficient Ct and Cp in the range of 0.3 to 6.0 and 0.31 to 0.93, respectively. In our research, the sub-basins having greater slope responds to smaller Ct value and in the case of small slope, the Ct value is larger than others. In the case of Hanumante river (sub-basin W1040), is the largest basin among all. But due to the longest flow path is largest for Manahara river (Sub – basin W710), the basin lag time is greater for Manahara river with comparison to Hanumante river. But due to largest basin, the peak discharge is greater for Hanumante river. The basin lag, time to peak, base time and peak discharge varies across the sub basins. This depends on the regional coefficients Ct and Cp and physical parameters of the watershed. The sub-basin having longest flow path and mild slope result in longer time to generate peak flow. UH for selected sub-basins representing major tributaries are shown for sample only (see Fig. 4). All these graphs have been used to routing the direct runoff from each sub-basin.

Table 3: Calibrated coefficient and summary parameters of Snyder Unit Hydrograph

Name of Sub-Basin	L km	Lc km	Area km²	Slope	Ct	Cp	Qpeak m³/s	Tl hr	Tp hr	Tb hr	W50%	W75%
W710	29.04	16.73	73.7	0.04	0.4	0.42	41.47	1.9	2.56	11.51	3.98	2.27
W720	16.48	8.02	26.67	0.06	0.31	0.43	26.1	1.02	1.72	6.82	2.19	1.25
W900	18.23	10.49	44.28	0.06	0.3	0.43	41.01	1.09	1.79	7.19	2.33	1.33
W930	9.88	3.93	14.74	0.005	0.79	0.42	8.81	1.78	2.45	10.87	3.73	2.13
W1040	26.8	10.64	184.01	0.03	0.46	0.42	104.92	1.87	2.54	11.36	3.93	2.24
W1150	12.3	5.09	31.39	0.002	1.02	0.41	12.94	2.64	3.27	15.42	5.57	3.18
W1240	5.96	2.61	5.85	0.01	0.6	0.43	5.7	1.03	1.73	6.85	2.2	1.26
W1290	12.47	3.88	41.95	0.01	0.81	0.42	23.09	1.94	2.61	11.74	4.08	2.32
W1300	15.02	5.95	65.44	0.09	0.27	0.44	80.15	0.77	1.49	5.51	1.72	0.98
W1340	17.31	8.47	31.03	0.03	0.39	0.43	24.63	1.3	1.99	8.31	2.75	1.57
W1350	11.72	5.99	37.03	0.09	0.24	0.44	51.56	0.66	1.38	4.88	1.5	0.85
W1390	8.41	3.77	7.32	0.02	0.47	0.43	7.31	0.99	1.7	6.69	2.14	1.22
W1400	20.67	12.67	49.04	0.06	0.3	0.43	41.88	1.2	1.89	7.76	2.54	1.45

3.2 HEC – HMS Model Performance

The model performance in calibration and validation was evaluated by visual inspection of the calculated and observed hydrographs and Nash – Sutcliffe Efficiency (Nash & Sutcliffe, 1970). The continuous data from 1992 to 2013 was used for calibration (see Fig. 5). Results indicate that the simulated hydrograph is comparable with the observed one, responds well to rainfall, and it can reproduce the overall hydrological pattern. The NashSutcliffe Efficiency in calibration is obtained as 0.753. Due care was given during calibration to ensure peak discharge and baseflows are reasonably reproduced. The model therefore can be considered as acceptable for further application.

The developed UH and other calibrated parameters were fed with the model as input along with real-time data. The model was then applied to simulate the flood hydrograph at the outlet of Khokana. Available real-time hourly hydro-meteorological data for the storm event of July 2011, August 2012, and July 2013 were used for validation. From Fig. 6, it can be easily observed that the simulated discharge follows the similar trend of recorded discharge and also observed the Nash – Sutcliffe Efficiency was 0.90, 0.637, and 0.731 for 2011, 2012, and 2013 respectively which were in acceptable range as per Moriasi et al. (2007). Here, in the case of 2011, the rainfall was started from 9:00 AM July 30, and maximum precipitation recorded as 10.06 mm at 5:00 AM Jul 31. It seems 20 hours of regular rainfall with small fluctuations generate the peak flow of 461.6 m^3/sec at 10:00 AM July 31. And the model also simulates the rainfall and gives the peak discharge of 480.4 m^3/sec at the same time. However in the case of the event of 2012, the rainfall starts from 5:00 PM Aug 2, but the intense rainfall of 27.7 mm within 3 hours was recorded on 5:00 AM Aug 3. In this event, the peak discharge of 188.7 m^3/sec was recorded at 3 Aug 11:00 AM and the model gives the peak value of 185 m^3/sec at 7:00 AM 2 Aug. This is because of short duration instance rainfall. Also in the case of 2013 the rainfall starting from 5:00 PM and peaks after 12 hours up to 6.66 mm/hour. This precipitation gives rise to 175.7 m^3/sec of flow at 9:00 AM but the model pretends the flow of 182.0 m^3/sec at 8:00 AM.

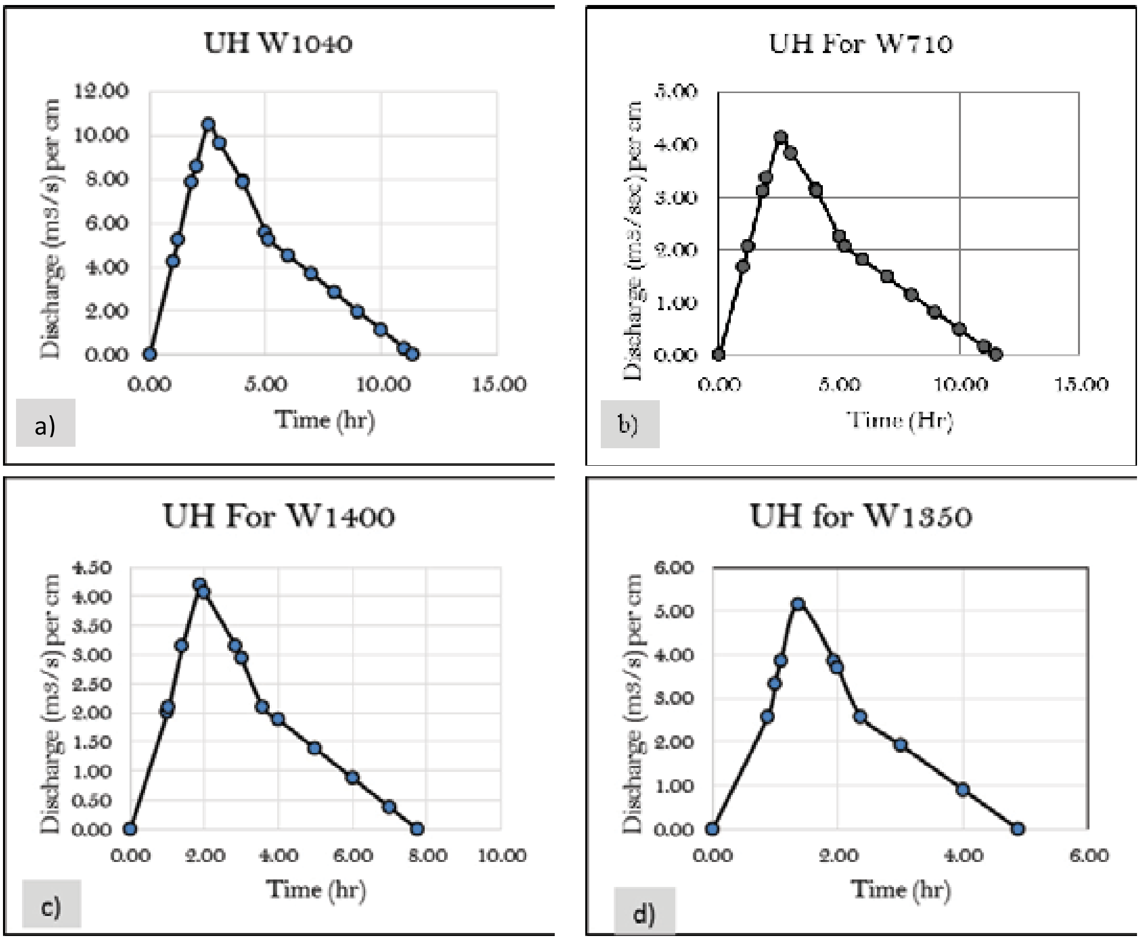

Figure 4: Unit hydrograph for selected sub-basins. a) For sub-basin W1040 which represents the Hanumante river. b) For sub-basin W710 which represents the Manahara river. c) For sub-basin W1400 which represents the Nakhu river. d) For sub-basin W1350 which represents the upper part of Bagmati river.

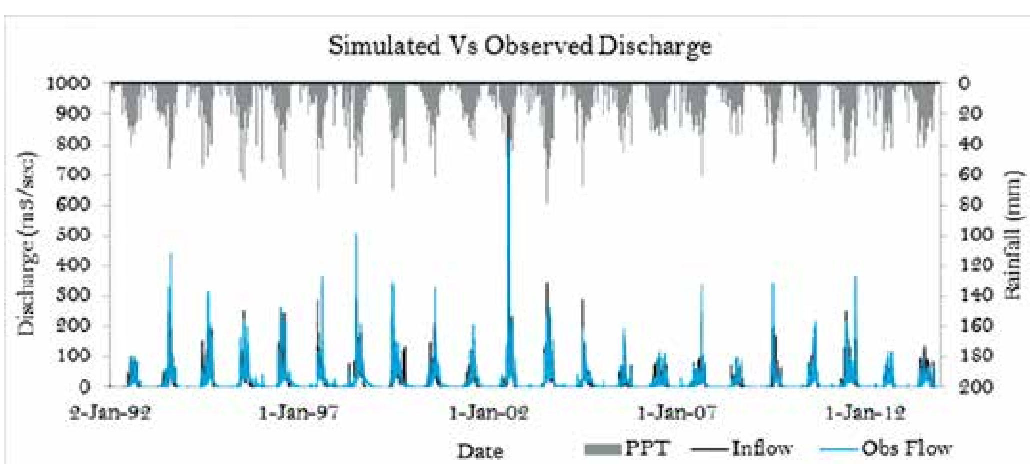

Figure 5: Comparison of observed versus simulated daily stream flows for the continuous-time period at Khokana outlet (index (530.05)) from 1992 to 2013.

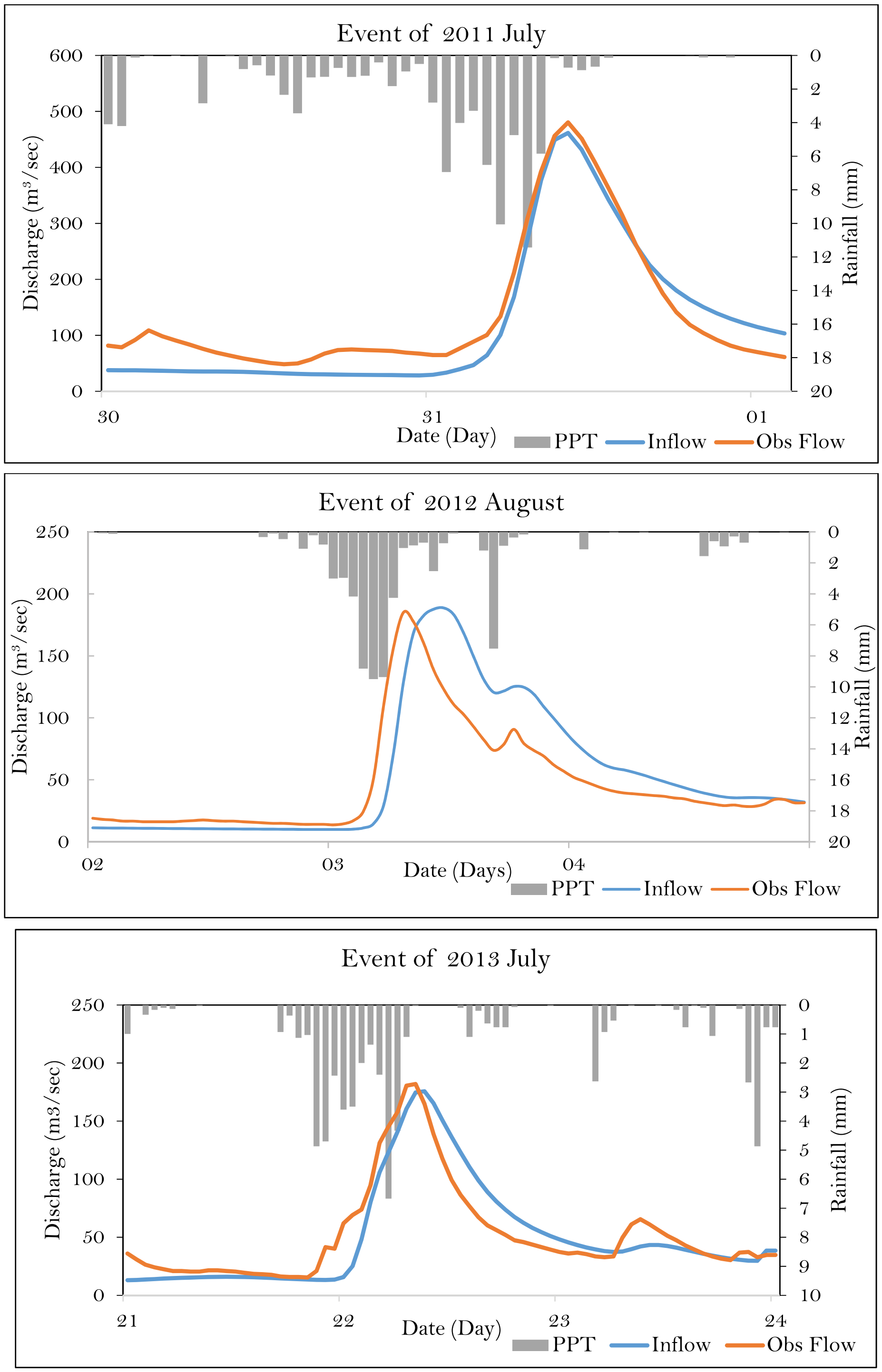

Figure 6: Validation of the model with the event of Jul 2011, Aug 2012, and Jul 2013

Model parameters deficit and constant, recession, simple canopy, and simple surface were calibrated. The calibrated model parameters for each sub-basin are presented in Table 4. For the model parameters, highly sensitive parameters were ratio to peak and recession constant they influenced for base flow.

Table 4: Calibrated model parameters for HEC-HMS model of Bagmati River Basin

Parameters	Deficit and Constant			Recession			Simple Canopy		Simple Surface	
	Constant Rate	Initial Deficit	Maximum Deficit	Initial Discharge	Ratio to Peak	Recession Constant	Initial Storage	Max Storage	Initial Storage	Max Storage
Sub basins	mm/hr	mm	mm	m^3/sec			%	mm	%	mm
W1150	0.346	30.00	150.00	4.090	0.364	0.233	5.188	9.287	5.00	9.170
W1040	0.235	50.34	248.64	4.599	0.650	0.348	5.000	10.294	5.00	5.387
W1240	0.707	50.00	350.00	4.743	0.347	0.367	5.126	10.250	5.00	12.874
W1290	0.691	49.00	250.00	5.850	0.453	0.378	4.999	5.210	5.00	13.233
W1300	0.624	40.00	300.00	4.820	0.348	0.529	5.651	5.699	5.00	8.591
W1340	0.653	50.00	250.00	4.080	0.567	0.237	7.387	5.875	5.00	5.943
W1350	0.356	45.00	185.00	2.809	0.233	0.344	4.107	10.107	5.00	12.310
W1390	0.797	40.00	220.00	5.078	0.656	0.237	4.651	5.634	5.00	10.118
W1400	0.394	56.00	250.00	4.800	0.632	0.235	4.360	19.888	5.00	15.770
W710	0.651	45.00	200.00	4.800	0.584	0.346	5.000	4.445	5.00	14.771
W720	0.679	50.00	350.00	5.080	0.348	0.454	4.900	7.950	5.00	12.090
W900	0.893	45.00	195.00	5.853	0.636	0.310	5.105	15.825	5.00	15.825
W930	0.663	60.00	210.00	5.825	0.651	0.364	3.049	9.521	5.00	9.800

3.3 Application of the Model for Estimating Extreme Events

We have chosen the date of 23 July 2002 as the storm event to simulate extreme events because of the maximum flood recorded on that day from 1993 to 2013. The event value of rainfall, simulated discharge from the model, and estimated discharge using frequency analysis are present in Table 5. The maximum daily discharge of this flood event at the Khokana was 814 m^3/s. The flood was generated by 5 – days of continuous rainfall starting from 19th through 23rd July 2002. The total cumulative rainfall amount for the 5 – day's period is 276.4 mm, with a maximum of 175.6 mm on 23rd Jul 2002. The rainfall of earlier days would have generated favorable moisture conditions and then an increase in the base flow for that flood.

The event model was then applied for estimating design floods as well as PMF at the outlet of the basin. Floods of different return periods were estimated using the deterministic approach, that is, by feeding the calibrated event model with rainfall of different return periods and compared with estimates from the probabilistic approach (i.e., flood frequency analysis) (see Table 5). It indicates the estimated design floods using the determinist approach are compared with those estimated using the probabilistic approach. As estimates from the probabilistic approach are slightly higher, they are recommended as design floods for the dry port. The floods of 50, 100, and 500 year return periods are estimated as 957.5 m^3/s, 1,080.4 m^3/s, and 1,364.3 m^3/s, respectively. The estimated PMF is 3629.70 m^3/s and the PMF hydrograph is shown (see Fig. 7).

4. Conclusions

In this study, HEC – HMS model was used for input daily rainfall and compare with daily observed discharge from 1992 to 2013 in calibration at the gauging station of Khokana (530.05). And for validation, hourly rainfall was compared with hourly observed real-time discharge for the event of 2011, 2012, and 2013. For the study, the spatially averaged precipitation in each sub-basin is derived using the Thiessen polygon method in ArcGIS. In this research work, the Bagmati Basin was divided into thirteen sub-basins, and unit hydrograph was developed for each sub-basin wherein the discharge of the outlet i.e. Khokana (550.05) was only taken. The UH for each sub-basin was developed by using Snyder equations. And the Snyder equations use the physical parameters of the basins. The dimension of UH was fixed so that the area of UH belongs to unity. After the UH development, the flood discharge at the basin outlet was then estimated by combining the sub-basins, using flood routing procedures. For calibration of the model parameters, Trial and error is performed to compare the precipitation and observed discharge. Based on this, by calibrating Snyder's equations, the coefficients required for the development of synthetic unit hydrographs, Lag time Co-efficient (Ct), Peak discharge coefficient (Cp), unit hydrograph widths at 50% and 75% of the peak and base time were determined. The lag time coefficient (Ct) for the watersheds ranges from 0.244 – 1.016. The peak discharge coefficients (Cp) of the unit hydrographs of the watershed range from 0.439 – 0.410, these values are recommended to construct UH of the Bagmati basin. All the HEC HMS model parameters are calibrated with the entering of calibrated Snyder UH. The results obtained are satisfactory and acceptable. During the model parameter calibration, the ratio to peak and recession constant module for the contribution of the base flow was highly sensitive. The applicability of the model is also ensured by extreme rainfall of different return periods. The simulated discharge was compared with the calculated discharge of different return periods, which gives a reasonable response. And again the calculated PMP of the basin was processed in the model and determine the PMF for the basin. The one-day PMP for the basin is 612.91 mm using Hersfield's method. The peak outflow at the outlet of the study area is 3629.70 m^3/sec. which is 1.94 times higher than the storm of 10,000 year return period.

Table 5: Simulated discharge and predicted discharge

Return period	Input Rainfall	Simulated discharge from HEC – HMS	Discharge from frequency analysis using observed data	Difference in Discharge
Year	mm	m^3/sec	m^3/sec	
5	113.1	542.6	534.8	-1.4%
10	134.05	670.9	666.8	-0.6%
25	160.4	832.5	833.7	0.1%
50	180.0	938.3	957.5	2.0%
100	199.5	1072.3	1080.4	0.7%
500	244.4	1349.3	1364.3	1.1%
1000	263.7	1468.6	1486.4	1.2%
10000	327.9	1865.4	1891.7	1.4%
PMP	612.9	3629.70 (PMF)		

As specified in the flood control and management manual by WECS, bridges, cross-drainage structures, river training structures, and other hydraulic structures are designed for 100 years to return periods' flood (WECS, 2019). However important projects like dry port may require flood greater than 100 years return period. Construction of dry port has been proposed in Chovar which lies little upstream of the outlet of our study basin and design of such project requires probable maximum flood. In this study, PMF has been estimated for the probable maximum precipitation for the basin which can be used in designing the river training structure upstream of Chovar and similar applications can be done for other river basins with similar projects. Furthermore, the satisfactory performance of the model proves its applicability in flood forecasting. The extreme rainfall events can be efficiently simulated to obtain the flood. Moreover, simulating the model for hourly rainfall can be used to fix the warning and danger level for the river. Hence this model can be applied in flood forecasting and early warning.

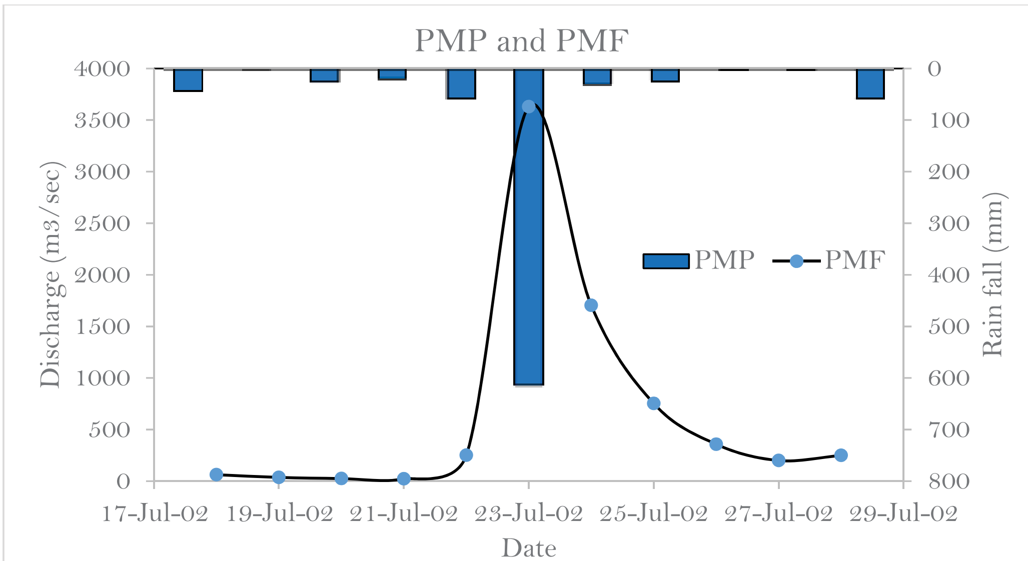

Figure 7: Estimated Probable maximum flood at the basin outlet

Conflict of Interests

Not declared by authors.

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