A method for estimating flash flood peak discharge in a poorly gauged basin: Case study for the 13–14 January 1994 flood, Giofiros basin, Crete, Greece

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SUMMARY

A method for estimating flash flood peak discharge, hydrograph, and volume in poorly gauged basins, where the hydrological characteristics of the flood are partially known, due to stage gauge failure, is presented. An empirical index is used to generate missing hourly rainfall data and hydrologic and hydraulic models perform the basin delineation, flood simulation, and flood inundation. The peak discharge, hydrograph, and volume, derived from the analysis of measured hydrographs in a number of non-flood causing rainfall events with operating stage gauge, were used for calibration and verification of the simulated stage-discharge hydrographs. An empirical equation was developed in order to provide the peak discharge as a function of the total precipitation, its standard deviation, and storm duration. The peak discharge for a flash flood case based on the empirical equation was in close agreement with the results from a number of consolidated methods. These methods involved hydrological and hydraulic modeling and peak flow estimates based on Manning’s equation and post flash flood measurements of the maximum water level observed at the control cross-section, for the 13–14 January 1994 flash flood in the Giofiros basin on the island of Crete, Greece. This method can be applied to other poorly gauged basins for floods with a stage higher than that defined by the rating curve.

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Introduction

Flash floods are those floods that follow the causative storms in a short period of time, with water levels in the drainage network reaching a crest within minutes to a few hours resulting in a very limited opportunity for warnings to be prepared and issued (Borga et al., 2007; Collier, 2007). The severity of flash flood generating storms is poorly captured by using conventional rain-gauge networks (Norbiato et al., 2007). Poor knowledge of this phenomenon comes from the fact that, in most flash-flood cases, the rain accumulations and the discharges are unknown over most of the watersheds of concern (Creutin and Borga, 2003). The number of rainfall stations in poorly gauged basins is not sufficient to fully describe the spatiotemporal hydrometeorologic characteristics of the storm events. In addition during the flash flooding process, the stream gauges usually fail. This renders each methodology almost unique and coupled with some special catchment characteristics difficult to generalize and apply at a different location.

Various methods have been developed in order to study extreme floods in ungauged or poorly gauged basins. An ungauged basin is one with inadequate records of hydrological observations to enable computation of hydrological variables of interest at the appropriate spatial and temporal scales, and to the accuracy acceptable for practical applications. There are a number of methods that can be applied to study extreme floods on ungauged watersheds including the so-called “indirect” peak discharge estimates, rainfall–runoff modeling through hydrological models and empirical methods.

Gaume (2006), refers to the peak discharge as a key issue of post flood studies and further hydrological analysis and makes an extensive reference to a number of indirect estimation methods, discussing the estimates accuracy. The methods presented, vary among those based on the Manning–Strickler formula like slope-area and slope-conveyance methods, non-parametric methods like critical depth and “super-elevation” in bends and in front of obstacles, water surface velocity evaluation on films and rainfall–runoff checking through the “rational-method” or rainfall–runoff simulation. Gaume et al. (2009), enumerates the various methods of peak discharge estimation used for 550 documented flash flood events across seven European hydrometeorological regions. This categorization includes Manning–Strickler formula estimation, extrapolation of calibrated stage–discharge relation, hydraulic 1D and 2D simulation, reconstruction from reservoir operation and direct current–meter measurement, with the first two being more popular. Webb and Jarrett (2002), provide a particularly concise review of the mathematic and hydrologic assumptions that underlie the commonly used slope–area, step-backwater, and critical-depth.

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methods. Bathurst (1990), tested three discharge gauging techniques in mountain rivers and concluded that flood flow can be estimated, with limited accuracy, by the slope–area and critical-depth methods. Costa (1987), combined the maximum rainfall–runoff floods in basins from 0.39 to 370 km² in the conterminous USA where the slope–area indirect method was used to estimate the peak discharge. The slope–area method computes discharge based on an adaptation of a uniform-flow equation (the Bernoulli energy equation) using channel-geometry characteristics, water-surface profiles, and Manning roughness coefficients. The uniform-flow assumption and Manning’s equation were used by Gaume et al. (2004) for the estimation of the peak discharge of several cross-sections during the 12 and 13 November 1999 flash flood of the River Aude in France. Wohl (1995), used step-backwater method in order to convert peak stage estimates from scour lines to discharge estimates in ungaged mountain channels. Hydrological models of varying complexity approaches are applied to provide detailed estimates of flow processes for ungaged sites (Sangati et al., 2009; Bonnifait et al., 2009; Bloshch et al., 2008; Reed et al., 2007; Bohorquez and Darby, 2008).

However, empirical methods have also been developed and applied at ungaged sites. Empirical methods can be of great value in view of constraints of application effort when compared to alternative traditional methods. Wharton (1992) and Wharton et al. (1989) applied an alternative indirect peak discharge technique at ungaged sites, first developed by the US Geological Survey. The channel-geometry method is based on the assumption that channel cross-section size and geometry at specific location reflects the drainage process during a storm. The method can be applied when estimating relations have been defined for a region and channel width or cross-sectional area, are the only available information. Specific procedures have to be followed for accurate and consistent measurements during field survey. Nasri et al. (2004), presented a geomorphological model methodology to predict shape and volume of several cross-sections during the 12 and 13 November 1999 flash flood of the River Aude in France. Wohl (1995), used step-backwater method in order to convert peak stage estimates from scour lines to discharge estimates in ungaged mountain channels. Hydrological models of varying complexity approaches are applied to provide detailed estimates of flow processes for ungaged sites (Sangati et al., 2009; Bonnifait et al., 2009; Bloshch et al., 2008; Reed et al., 2007; Bohorquez and Darby, 2008).

Guidance for flash floods surveys is provided by Marchi et al. (2009), describing the investigation process of a flash flood in Western Slovenia. Peak discharges and associated uncertainty were estimated for 22 cross-sections using the slope–conveyance method that proved to be practically useful and simple. Pruess et al. (1998), presented the methodology and implications of 15 field reconnaissance surveys for estimation peak discharges of extreme flash floods along mountain channels of southwestern Colorado, using step-backwater flow modeling. Thirty selected major floods in the US were analyzed by Costa and Jarrett (2008) aiming to the most possible accurate estimate of the peak discharges using the best available practices. The dominant method used was the slope–area method for 21 of the events. For the rest of the events, direct current-meter measurements, culvert measurement, rating-curve extension, and interpolation and rating-curve extension was used, as well as combination of the above. The study makes an extensive report to the errors and uncertainty embedded to the peak flow estimation.

The estimation of peak discharges without direct current-meter measurements is, above all, a question of sound engineering judgment (Gaume et al., 2004; Gaume, 2006). Empirical relations must be used with caution and estimates should be made at a minimum of two or three cross-sections for the same river reach to reduce uncertainties (Gaume et al., 2004; Gaume, 2006). Errors in the estimation of peak discharges can be controlled by careful hydraulic and laboratory analysis (NRC, 1999). The estimation accuracy of peak discharge based on slope–area method in small steep watersheds in the US was evaluated by Jarrett (1987) by examining the effect of various parameters such as the roughness coefficient, the geometry of the cross-section, and the unsteady nature of large discharges, leading to overestimation of peak discharge by up to 100% or over. Dottori et al. (2009), presented a dynamic rating curve approach for indirect discharge estimation, based on simultaneous stage measurements at two adjacent cross-section. The authors compared various approaches described in the literature, examined the significant uncertainty produced in the extrapolation beyond the range of measurements of the rating curve, and presented the improvement in the discharge estimation using their proposed approach. In order to avoid significant errors and reduce the uncertainties, it is necessary to use various sources of information to enable cross-checking using different approaches (Gaume and Borgia, 2008).

The purpose of the paper was the comparison between a number of consolidated methods for estimating peak discharge under a stage gauge failure, or, in general, when limited information are available. The Giofiros basin (158 km²) located in the central-north
part of the island of Crete (Fig. 1) was used as case study and, in particular, the flash flood occurred on 13th January 1994 was analyzed.

Methodology

**Reconstruction of precipitation realizations of variable spatiotemporal distribution**

The precipitation reconstruction approach combines the Thiessen polygons method, the extrapolation of the hourly rainfall intensity variations to the daily rain-gauges and the generation of gridded realizations using the assumption of multiplicative space-time separability as presented by Woods and Sivapalan (1999). In the case of the Giofiros basin the hourly rainfall data was available from the station with the higher elevation that recorded at least four times higher rainfall than the other stations in the lower elevations in the valley with daily rainfall data. Therefore the hourly distribution of the rainfall in the stations with lower rainfall will not significantly alter the hydrograph. The transformation is done with the introduction of index \( A \) for each of the daily recording stations. Index \( A \) correlates the daily precipitation of each station with the daily precipitation of the gauge with hourly data. Index \( A \) is introduced as:

\[
A_{j}^{n_{k}} = \frac{P_{j}^{n_{k}}}{P_{j}^{0}}
\]

where \( P_{j}^{n_{k}} \) is the total daily precipitation at gauge \( x_{k} \) (\( k = 1\ldots n \) for \( n \) daily precipitation gauges) on day \( j \) and \( P_{j}^{0} \) is the total daily precipitation of gauge \( x_{0} \) on day \( j \). Index \( A \) reflects the correlation of the total amount of precipitation between gauges \( x_{0} \) and \( x_{k} \), within a period of 1 day. The hourly estimations for gauge \( x_{k} \) is produced by multiplying the index \( A_{j}^{n_{k}} \) for each daily recording gauge \( x_{k} \) with the hourly precipitation of gauge \( x_{0} \). At a second step, the method by Woods and Sivapalan (1999) can be applied by splitting the spatial and temporal variability into two separate terms. Different realizations of stationary hourly rainfall of different intensity and timing reproduce the spatiotemporal variability. The average hourly or distributed precipitation, over the basin is then estimated by the Thiessen method (Thiessen, 1911) or by other interpolation methods such as inverse distance weighted – IDW (Shepard, 1968), Kriging (Matheron, 1965). The proposed methodology takes into account the spatial distribution of precipitation due to topography (e.g., orographic precipitation) or locality of the meteorological phenomena. There are limitations of the size of the basin that can be applied since the spatial structure of convective rainstorms depends on the local meteorology and topography, but generally the distance of decorrelation between measured rainfall intensities lies between 10 and 20 km at an hourly time step (Lebel et al., 1987).

**Empirical peak discharge estimation**

In order to fill in missing data for runoff, an attempt was made to correlate the total event runoff volume with precipitation characteristics and peak discharges. The suggested method does not use directly soil properties and infiltration processes during runoff but rather compares the total runoff volume \( V_{T} (m^3) \) occurring during a single event with the total precipitation \( P_{T} (mm) \) that causes it, the standard deviation of the precipitation time series \( \sigma_{P} \) in mm) and the rainfall event duration \( D \) (h). When the total runoff volume \( V_{T} (m^3) \) for a sufficient number of gauged floods are plotted against the product of the square of total precipitation \( P_{T} \) and the standard deviation of precipitation \( \sigma_{P} \) their relation can be represented via a linear regression equation:

\[
V_{T} = \alpha P_{T}^{2} \sigma_{P}
\]
where \( a \) is a coefficient for the basin. Eq. (2) expresses that the average flow rate during the rainstorm is proportional to the product of the total precipitation, its intensity and its standard deviation while the coefficient of proportionality \( a \) is related to the characteristics of the basin. The nonlinearity of the peak discharge \( Q_{\text{peak}} \) with the total volume is expressed as:

\[
Q_{\text{peak}} = \beta V^\gamma
\]

where \( D \) is the event duration in seconds, and \( \beta \) and \( \gamma \) are coefficients related to the characteristics of the basin. Due to limited data availability a number of non-flood causing rainfall events with an operating stage gauge meter are used to evaluate the coefficients in Eq. (3). Peak discharges of selected rainfall-runoff events, cover a wide range of discharge conditions for optimal method validation. Combining Eqs. (2) and (3) results in the elimination of \( V^\gamma \):

\[
Q_{\text{peak}} = \frac{1}{D} \theta (P^2 \sigma_p)^\gamma
\]

where \( \theta = \beta \gamma \). Given the coefficients related to the characteristics of the basin and the precipitation time series, the peak discharge of a flood can be determined via Eq. (4).

**Flood peak flow estimation from cross-sectional geometry and parameters**

In the absence of field flow data, the evaluation of the results can be based on the estimation of the observed peak flow from a specific cross-section if the geometric and hydraulic characteristics of the cross-section along with the water level are available. The cross-section can be divided in two parts, the first of which is considered as the main channel and the second is the floodplain. Using Eq. (5) (Streeter and Benjamin, 1988) the total flow between two cross-sections can be estimated as:

\[
Q = (K_1 + K_2) \sqrt{S}
\]

where \( Q \) is the total flow rate in \( m^3/s \), \( K_i = \frac{A_i R_i^{n_i}}{2} \) is the energy slope along flow, \( A_i \) is the area of cross-section \( i \) for water level \( j \), \( R_i \) is the hydraulic radius, and \( n_i \) is the roughness coefficient (\( i = 1 \) for the main channel and \( i = 2 \) for the floodplain).

**Hydrological and hydraulic modeling**

The US Army Corps of Engineers Models, developed at the Hydrologic Engineering Center (HEC) in Davis, California, were applied for flood simulation and mapping. The hydrological analysis was divided in four main steps according to the HEC modules: (a) the extraction of basin’s characteristics (HEC-GeoHMS), (b) the distributed rainfall-runoff simulation (HEC-HMS), (c) the simulation of the flood wave (HEC-RAS) and (d) the mapping and representation of flood extend (HEC-GeoRAS). In HEC-HMS model, the deficit and constant was used as the loss method, the ModClark was selected as the transform method and the recession method for sub-basin baseflow (HEC, 2001). For the precipitation representation, the generated hourly precipitation data were interpolated to hourly gridded precipitation datasets over the study area using the IDW method.

**Deficit and constant loss method**

Deficit and constant is a quasi-continuous model of precipitation losses (HEC, 2000). The concept of the deficit and constant rate loss model is that the maximum potential rate of precipitation loss, \( f_c \) (in mm/h), is constant throughout an event and the initial loss can “recover” after a prolonged period of no rainfall. An initial deficit, \( I_0 \) (in mm), represents interception and depression storage. Interception storage is a consequence of absorption of precipitation by surface cover, including plants in the watershed. Depression storage is a consequence of depressions in the watershed topography; water is stored in these and eventually infiltrates or evaporates. This loss occurs prior to the onset of runoff. Until the accumulated precipitation on the pervious area exceeds the “recovered” initial loss volume, no runoff occurs. Thus, if \( p_i \) is the storm mean areal precipitation depth (mm/h) during a time interval \( t \) to \( t + \Delta t \), the excess, \( p_e \) (in mm/h), during the interval is given by:

\[
p_e = \begin{cases} 
0 & \text{if } \sum p_i < I_d \\
p_i - f_c & \text{if } \sum p_i > I_d \text{ and } p_i > f_c \\
0 & \text{if } \sum p_i > I_d \text{ and } p_i < f_c 
\end{cases}
\]

**ModClark transformation and recession methods**

The modified Clark (ModClark) model in HEC-HMS is a distributed parameter model in which spatial variability of characteristics and processes are considered explicitly (Kull and Feldman, 1998; Peters and Easton, 1996). This model accounts explicitly for variations in travel time to the watershed outlet from all locations of a watershed. The ModClark algorithm is a version of the Clark unit hydrograph transformation modified to accommodate spatially distributed precipitation (Clark, 1945). Runoff computations with the ModClark model explicitly account for translation and storage. Storage is accounted for within the same linear reservoir model incorporated in the Clark model. Translation is accounted for within a grid-based travel-time model \( t_c = t_s (d_{cell}/d_{max}) \) (HEC, 2000), where \( t_c \) is the time of concentration for the subwatershed and is a function of basin’s length and slope, \( d_{cell} \) is the travel distance from the cell to the outlet, and \( d_{max} \) is the travel distance from the cell furthest from the outlet. The method requires an input coefficient for storage, \( R \), where \( R \) accounts for both translation and attenuation of excess precipitation as it moves over the basin toward the outlet. Storage coefficient \( R \) is estimated as the discharge at the inflection point on the recession limb of the hydrograph divided by the slope at the inflection point. The translation hydrograph is routed using the equation:

\[
Q(t) = \frac{[\Delta t/(R + 0.5\Delta t)]I(t) + [1 - (\Delta t/(R + 0.5\Delta t))]Q(t - 1)]}{1 - \Delta t/R}
\]

where \( Q(t) \) is the outflow from storage at time \( t \), \( \Delta t \) is the time increment, \( R \) is the storage coefficient, \( I(t) \) is the average inflow to storage at time \( t \) and \( Q(t - 1) \) is the outflow from storage at previous time \( t - 1 \).

The exponential recession baseflow method, used in HEC-HMS (Chow et al., 1988), approximates the watershed drainage behavior when channel flow recedes exponentially after an event (Linsley et al., 1982). This recession model defines the relationship \( Q_b \) baseflow at anytime \( t \), to an initial value as:

\[
Q_b = Q_0 k^t
\]

where \( Q_0 \) is the initial baseflow and \( k \) is an exponential decay constant defined as the ratio of the baseflow at time \( t \) to the baseflow 1 day earlier. In HEC-HMS the parameters of this model include the initial flow, the recession ratio, and the threshold flow. The parameters can be easily estimated if gauged flow data are available.

**Calibration of the model**

Various indices of evaluation were used in order to calibrate the model. They are divided in two categories depending on time scale issues. The first group of indices that evaluate the model is in the scale of entire rainfall events.
On 13th January 1994 and around 15:00, the intensity of the light precipitation that had saturated Giofiros soils during the previous days started to increase. Reaching a maximum 5 h accumulated precipitation of 123 mm at 21:00 this extreme rainfall eventually stopped at 24:00 lasting almost 9 h. By that time the resulting flash flood had catastrophic impacts on Giofiros basin. Many houses located near the coast were flooded leaving 49 people homeless. In terms of precipitation severity, the 100-years recurrence interval of 5 h accumulated precipitation of a nearby meteorological station (Heraklio EMY in Fig. 1) is 98 mm according to Gumbel distribution (Ganoulis, 2003). The maximum 5 h accumulated storm rainfall of 123 mm that was observed at the south part of the basin (Ag. Varvara station in Fig. 1), far exceed the 100-years recurrence interval of 98 mm of the nearby, lowland, Heraklio EMY station. The single most adverse effect of the flood was the damage caused to the city’s wastewater treatment plan which was still under construction at the time of the flood. Many of the concrete tanks were rendered unserviceable or were completely destroyed by the force of the oncoming flood wave (Ganoulis, 2003). The exact flood characteristics cannot be known since the only stage gauge was destroyed at 20:00 during the flash flood.

During a recent flash flood data compilation effort in the frame of the EC funded research project HYDRATE (CN: 037024), an inventory – first step towards an atlas of extreme flash floods in Europe was developed containing over 550 documented events (Gaume et al., 2009). The 1994 Giofiros flash flood event was one among the 22 Greek floods that enriched this flood inventory. In terms of flash flood magnitude and under a pan-European and international perspective (Gaume et al., 2009), the 1994 Giofiros storm had a moderate unit peak discharge of 1.85 m$^3$/s/km$^2$ (300 m$^3$/s/158 km$^2$). Due to lack of high resolution precipitation data both spatially and temporally it is possible that the unit peak discharge could be much higher, i.e., (a) assuming that the majority of flood was produced in the upper 1/5th of the basin (186/5 = 37.2 km$^2$) due to orographic effect the unit peak discharge could be 8.1 m$^3$/s/km$^2$, (b) simulations with the peak hourly precipitation occurring in 15 min instead of an hour reveal an increase in velocity from 3.62 m/s to 4.08 m/s, in water depth from 5.1 m to 5.4 m and in peak discharge from 300 m$^3$/s to 375 m$^3$/s, that brings the unit peak discharge as high as 9.67 m$^3$/s/km$^2$.

Since 1994, several studies have been conducted on Giofiros basin, related to that flash flood. Most of them focused on the estimation of the hydrological parameters of the basin, so that appropriate flood prevention measures could be taken, and the rest aimed at simulating the 1994 flood. Due to lack of flood recording data these studies could not verify the simulation results but rather empirically estimated the total flood volume. It should be noted that flood researchers have come up with peak discharge estimates that vary significantly from 300 m$^3$/s up to 600 m$^3$/s (Ganoulis, 2003; Barboudakis, 1998).

**Field data**

Flash-flood monitoring requires rainfall estimates at small spatial scales (1 km or finer) and short-time scales (15–30 min, and even less in rugged mountainous areas and urban areas) (Borga et al., 2008; McCain and Shroba, 1979). Floods research at hydrological basin scale requires high resolution data, particularly for using the most robust hydrologic models that have been recently developed. Existing data usually does not include all the data required for newer models. From the three rain-gauges of Giofiros basin, one has hourly records and two stations providing daily precipitation measurements (Fig. 1). The stations that provided the daily precipitation records and the station with the hourly measurements are located along the south–north direction, which is the main orientation axis of Giofiros basin. Along this line,
topography changes significantly from sea level to 570 m. Ag. Varvara hourly precipitation station is located near the mountain headwaters at 570 m above sea level (a.s.l.) (see Fig. 1) and recorded 182.8 mm of total precipitation during the 14th of January 1994, while Profitis Ilias (380 m a.s.l.) and Finikia (40 m a.s.l.) precipitation stations recorded 43.9 mm and 37.7 mm, respectively. Using the previously described methodology for generating distributed hourly precipitation data-sets, total average areal precipitation for the 1994 flood event was $P_{\text{total}} = 75.7$ mm. A water flow level gauge is located in the north part of the basin, but about 6 km upstream from the outlet on the Mediterranean Sea (Fig. 1). At the same location, the geometric characteristics of the certain cross-section site of Giofiros stream were available, as well as the flood extent of the 1994 flood event. The only water flow level gauge with hourly records was destroyed during the flood of 1994.

Results and discussion

Synoptic meteorological analysis

The meteorological analysis is focused on the synoptic prevailing background conditions for heavy, localized rainfall over Crete. ERA-40 re-analysis data (ERA-40 is a re-analysis of meteorological observations from September 1957 to August 2002 produced by

Fig. 2. (a–b) Mean sea level pressure analysis and convective precipitation based on the ECWMF 40 years re-analysis daily dataset and Meteosat satellite images for the 13 January 1994 flash flood event.
the European Center for Medium-Range Weather Forecasts (ECMWF) in collaboration with many institutions – Uppala et al., 2005) supported the synoptic analysis of the conditions prevailing to heavy local rainfall, leading to flash flooding. Moreover, high resolution visible channel METEOSAT 4 snapshots used during daylight and infrared channel images used during night for cloudy features (storm) generation and tracking (Ottenbacher et al., 1997). The satellite images of 30 min temporal resolution were provided from the European Organisation for the Exploitation of Meteorological Satellites (EUMETSAT).

On January, 1994, a frontal depression in south-east Mediterranean moved eastward and north crossing the island of Crete. On the surface analysis, a low pressure area was already present at 12:00 UTC 13 January (Fig. 2a) all over the Eastern Mediterranean area. The lower pressure area of 1010 hPa appeared at 12:00 UTC 13 January over Egypt moving in a northerly direction. This low pressure area deepens to 1007 hPa centered just south of Crete at 00:00 UTC 14 January, producing high precipitation over central Crete and the flash flood of Giofiros basin. High convective precipitation rates up to 45 mm per 6 h (00:00 UTC 14/1/1994) were observed during 13 and 14 January 1994 (Fig. 2b).

The average areal daily precipitation over the island of Crete was calculated in order to examine the correspondence with the ERA-40 convective precipitation dataset. The average areal daily precipitation estimated based on 53 daily temporal resolution rain-gauges through IDW interpolation method (Fig. 3) and then
compared with the spatial aggregated convective precipitation over Crete. The average areal daily precipitation of the 14th of January 1994 based on gauged data was 61.5 mm while ERA40 estimated 12.3 mm resulting in an underestimation of total precipitation. For the 13th of January measured precipitation was 20 mm while ERA40 results to 1.5 mm and for the 15th of January measured and ERA40 precipitation was 9 mm and 3.3 mm, respectively. The comparison of ERA-40 re-analysis daily convective precipitation (Uppala et al., 2005), results in underestimation of measured precipitation due to lack of incorporating orographic effects on the storm.

The studied flood produced by a heavy, orographic affected, thunderstorm based on the fact that the precipitation in Ag. Varvara station deep in the mountain at the furthest south point of the basin was almost five times larger (182.8 mm in total with peak intensity of 37 mm/h) than the ones recorded in the other two stations (Fig. 4). In fact, a station close to the sea east of the Mediterranean Sea and east of the outlet of Giofiros basin (Heraklio EMY station) recorded less than 28.4 mm of rain with the peak rainfall intensity of 7.8 mm/h (Fig. 5). The warm air masses with high dew points with a southwest–northeast flow (analysis at 500 hPa) and high positive relative vorticity were primary blocked by the Psiloritis Mountains’ steep topography (Fig. 4). The air mass was forced to higher elevations, thus temperatures cooled and produced high rates of precipitation of up to 37 mm/h at Ag. Varvara station.

Hydrological analysis

A digital terrain model DTM of the island of Crete in 1:10 000 scale (Fig. 1) was used with the HEC-GeoHMS extension of Arc-View GIS in order to delineate the Giofiros watershed. The
same DTM was also used for the delineation of the stream network and the sub-basins. Subsequently, HEC-GeoHMS output was imported in HEC-HMS. Regarding the rest of the input for HEC-HMS, basin parameters were estimated through the calibration of gauged rainfall–runoff events and based on hydrologic and physiographic characteristics of the basin from land use, soil type, basin slope, and geological maps of the study area, provided by the Water Resources Department of the Prefecture of Crete. Hourly gridded precipitation of 1 km² spatial resolution was generated according to the previously described methodology. Rainfall–runoff simulation of several events were conducted for the area (158 km²) upstream from the Finikia flow stage gauge (Fig. 1). The extent of the hydraulic simulation of the 1994 flood event were downstream of the flow stage gauge to the outlet of the basin (a distance of about 6 km), where the most significant impacts of the flood were observed.

**Sensitivity analysis**

For a successful simulation of the 1994 flood, model calibration was based on the simulation of eight rainfall–runoff events of various intensities, prior and posterior to the 1994 flood. These events were derived from two time periods (from 1981 to 1995) with total areal rainfall up to 74.4 mm and were the only clearly gauged and consistent for simulation events. Before the calibration of the rainfall–produced floods and the simulation of the flood, one of these floods (11–12 January 1995) was used to assess the sensitivity analysis of the model, which was based on all the parameters of the model. The results of sensitivity analysis are presented in Table 1a and b, based on the total runoff volume, the peak flow, the

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### Table 1

Sensitivity analysis results on the 11–12 January 1995 rainfall–runoff event with total areal rainfall of 16.0 mm.

#### (a)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Initial deficit, steady values: constant rate 5.7 mm/h, impervious 5%, Clark time of concentration 3.45 h, Clark storage coefficient 3.7 h, recession constant = 0.99, baseflow threshold ratio = 0.14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial deficit (mm)</td>
<td>5 6 7 8 9 10 11 12 13 14</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>28.5 28.5 28.5 28.5 28.5 28.5 28.5 28.5 28.5 28.5</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>68.2 63.8 58.3 52.6 46.3 38.3 28.5 18.2 6.9</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
<tr>
<td>Variable = constant rate, steady values: initial deficit = 10.3 mm, impervious 5%, Clark time of concentration 3.45 h, Clark storage coefficient 3.7 h, recession constant = 0.99, baseflow threshold ratio = 0.14</td>
<td></td>
</tr>
<tr>
<td>Constant rate (mm/h)</td>
<td>1.5 2 2.5 3 3.5 4 4.5 5 5.5 6</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>40.1 36.7 32.9 28.3 23.8 19.0 14.4 10.3 6.5 3.0</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>20.6 12.9 5.3 2.7 2.7 0.9 1.8 2.9 0.9 42.1</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
<tr>
<td>Variable = impervious, steady values: initial deficit = 10.3 mm, constant rate 5.7 mm/h, Clark time of concentration 3.45 h, Clark storage coefficient 3.7 h, recession constant = 0.99, baseflow threshold ratio = 0.14</td>
<td></td>
</tr>
<tr>
<td>Impervious (%)</td>
<td>1 2 3 4 5 6 7 8 9 10</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>74.5 71.0 66.9 62.0 55.8 48.3 39.4 30.6 22.6 14.1</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>20.6 12.9 5.3 2.7 2.7 0.9 1.8 2.9 0.9 42.1</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
</tbody>
</table>

#### (b)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Clark time of concentration, steady values: initial deficit = 10.3 mm, constant rate 5.7 mm/h, impervious 5%, Clark storage coefficient 3.7 h, recession constant = 0.99, baseflow threshold ratio = 0.14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clark storage coefficient (h)</td>
<td>2.5 3 3.5 4 4.5 5 5.5 6 6.5 7</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>6.6 12.2 15.9 17.9 19.2 19.4 19.1 18.4 17.4 16.3</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>1.8 6.1 0.9 2.7 2.7 0.9 1.8 2.9 0.9 4.9</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
<tr>
<td>Variable = Clark storage coefficient, steady values: initial deficit = 10.3 mm, constant rate 5.7 mm/h, impervious 5%, Clark time of concentration 3.45 h, recession constant = 0.99, baseflow threshold ratio = 0.14</td>
<td></td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>24.6 22.7 20.3 17.5 14.6 11.7 8.8 5.6 2.1 0.7</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>20.6 12.9 5.3 2.9 2.9 0.9 1.8 2.9 0.9 4.9</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
<tr>
<td>Variable = recession constant, steady values: initial deficit = 10.3 mm, constant rate 5.7 mm/h, impervious 5%, Clark time of concentration 3.45 h, Clark storage coefficient 3.7 h, baseflow threshold ratio = 0.14</td>
<td></td>
</tr>
<tr>
<td>Recession constant</td>
<td>0.01 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>7.0 9.6 11.7 12.9 14.3 15.0 16.1 17.0 17.9 19.0</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>–6.9 –4.9 –3.8 –2.9 –1.9 –0.9 0.0 0.0 0.9 1.8</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
<tr>
<td>Variable = baseflow threshold ratio, steady values: initial deficit = 10.3 mm, constant rate 5.7 mm/h, impervious 5%, Clark time of concentration 3.45 h, Clark storage coefficient 3.7 h, recession constant = 0.99</td>
<td></td>
</tr>
<tr>
<td>Baseflow threshold ratio</td>
<td>0.01 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9</td>
</tr>
<tr>
<td>Volume error (%)</td>
<td>19.0 19.0 19.0 19.0 20.0 22.8 26.6 30.8 35.2 39.6</td>
</tr>
<tr>
<td>Peak flow error (%)</td>
<td>1.8 1.8 1.8 1.8 1.8 1.8 1.8 1.8 1.8 1.8</td>
</tr>
<tr>
<td>Phase error (h)</td>
<td>0 0 0 0 0 0 0 0 0 0</td>
</tr>
</tbody>
</table>

---

**Fig. 5.** Hourly Precipitation intensity of Ag. Varvara rain-gauge (total precipitation of 182.8 mm at 570 m a.s.l.) and Heraklio rain-gauge (total precipitation of 28.4 mm at 40 m a.s.l.).
Model calibration results for the simulation of the eight rainstorms.

Calibration procedure and parameter estimation

Following the sensitivity analysis, rainfall–runoff simulation was carried out for the eight precipitation events calculating the optimum hydrological model parameters using the indices given in Eqs. (9) to (16). The estimated and observed hydrographs with the corresponding hourly precipitation data of three most representative rainstorms (in terms of peak discharge amplitude) are presented in Fig. 6. Model calibration was based on the minimization of the error of the water volume (VE) from Eq. (11), the peak flow error (PFE) from Eq. (9), the phase error (PE) from Eq. (10) and the best performance of the time step based evaluation criteria from Eqs. (12) to (16). The calibration procedure resulted to the estimation of the characteristic parameters for the Giofiros basin representing the physical properties of the watershed soils and land use and the antecedent-moisture condition. The initial deficit of the storms varied from 3.9 to 37 mm (Table 2) and is used as a hydrograph, and the time phase of the peak discharge. Constant rate (CR) parameter, Clark storage coefficient (R), impervious percent of the basin, and initial deficit were found to be the parameters that could mostly affect the simulation results. The initial deficit varies depending to the antecedent soil condition of each event. A range from 5 mm to 14 mm has been tested for this rainfall event of 16 mm of total average precipitation. Sensitivity analysis of this parameter resulted to a peak flow error from 28.5% to –24.1% and a volume error from 40.1% to 3.0%. Timing of the peak flow is not affected by initial deficit (Table 1a, first rows). The CR parameter depends on the physical properties of the watershed soils, the land use and the storm precipitation characteristics (e.g., intensity) and represents the infiltration rate. Different values of CR were tested during sensitivity analysis varying from 1.5 to 6 mm/h, taking into account Giofiros basin soil properties and land use. Results show a peak flow error from 68.2% to –6.9%, a volume error from 74.5% to 14.1% and a phase error of 1 h for CR values between 1.0 and 2.5 mm/h (Table 1a, middle rows). Impervious parameter represents the percent impervious area of the basin and is defined based on the land use and soil properties of the basin. Giofiros basin can be characterized by 5% impervious based on watershed properties. Moreover, values ranging from 1% to 10% tested in order to examine the effect of the parameter, resulting to a peak flow error from –54.3% to 32.1% and a volume error from –36.3% to 46.4%, with no effect on phase error (Table 1a, lower rows). The R parameter accounts for both translation and attenuation of excess precipitation as it moves over the basin toward the outlet and depends on basin attributes and precipitation characteristics. Values ranging from 2.5 to 7 h tested resulting to a peak flow error from 20.6% to –47.9% and a volume error from 24.6% to –0.7% (Table 1b, middle rows).

### Table 2

<table>
<thead>
<tr>
<th>Event</th>
<th>Date</th>
<th>Volume (m³)</th>
<th>Average rainfall intensity (mm/h)</th>
<th>Pᵢ (mm)</th>
<th>Iᵢ (mm)</th>
<th>Peak flow (m³/s)</th>
<th>Peak flow error (%)</th>
<th>Phase error (h)</th>
<th>Nash Sutcliff</th>
<th>Volume error (%)</th>
<th>RMSE (%)</th>
<th>AAPE (%)</th>
<th>r</th>
<th>EV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30–31 January 1993</td>
<td>33,812</td>
<td>0.83</td>
<td>2.5</td>
<td>3.9</td>
<td>2.3</td>
<td>0</td>
<td>0</td>
<td>0.90</td>
<td>–1.9</td>
<td>0.19</td>
<td>22.0</td>
<td>0.99</td>
<td>90.5</td>
</tr>
<tr>
<td>2</td>
<td>11–12 January 1995</td>
<td>180,579</td>
<td>2.67</td>
<td>16.0</td>
<td>10.3</td>
<td>10.8</td>
<td>1.8</td>
<td>0</td>
<td>0.90</td>
<td>13.4</td>
<td>0.96</td>
<td>16.5</td>
<td>0.99</td>
<td>91.7</td>
</tr>
<tr>
<td>3</td>
<td>13–14 January 1995</td>
<td>151,577</td>
<td>0.32</td>
<td>10.4</td>
<td>11.0</td>
<td>10.0</td>
<td>–4.2</td>
<td>0</td>
<td>0.69</td>
<td>18.5</td>
<td>1.10</td>
<td>17.4</td>
<td>0.99</td>
<td>82.2</td>
</tr>
<tr>
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<td>9.1</td>
<td>9.5</td>
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<td>0</td>
<td>0.88</td>
<td>13.1</td>
<td>0.90</td>
<td>28.4</td>
<td>1.00</td>
<td>90.5</td>
</tr>
<tr>
<td>5</td>
<td>14–15 January 1995</td>
<td>408,760</td>
<td>2.91</td>
<td>32.0</td>
<td>8.0</td>
<td>20.1</td>
<td>–2.0</td>
<td>0</td>
<td>0.88</td>
<td>13.0</td>
<td>1.60</td>
<td>16.6</td>
<td>1.00</td>
<td>93.0</td>
</tr>
<tr>
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<td>7–8 February 1994</td>
<td>384,502</td>
<td>1.06</td>
<td>21.8</td>
<td>5.1</td>
<td>20.3</td>
<td>0</td>
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<td>26.9</td>
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<tr>
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<td>1,394,986</td>
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<td>53.1</td>
<td>5.0</td>
<td>40.5</td>
<td>1.7</td>
<td>0</td>
<td>0.74</td>
<td>5.4</td>
<td>3.51</td>
<td>27.0</td>
<td>1.00</td>
<td>88.5</td>
</tr>
<tr>
<td>8</td>
<td>22–24 February 1981</td>
<td>3,333,347</td>
<td>1.58</td>
<td>74.4</td>
<td>37.0</td>
<td>61.0</td>
<td>4.1</td>
<td>0</td>
<td>0.70</td>
<td>9.2</td>
<td>3.93</td>
<td>15.3</td>
<td>0.98</td>
<td>92.9</td>
</tr>
<tr>
<td>Average values</td>
<td></td>
<td></td>
<td></td>
<td>0.2</td>
<td>0</td>
<td>0.78</td>
<td>3.91</td>
<td>0</td>
<td>0.70</td>
<td>9.2</td>
<td>3.93</td>
<td>15.3</td>
<td>0.98</td>
<td>92.9</td>
</tr>
</tbody>
</table>

* Storms corresponding to flow hydrographs of Fig. 6.

* P is average rainfall over the basin.
fitting parameter (during the present study \( I_d \) was poorly linked to its physical meaning). The maximum deficit estimated at 91 mm and the impervious area to 5% of the basin based on land-use, geology, and soil maps. The parameter \( CR \) varied from 2.5 to 5.5 mm/h according to the average precipitation intensity (API) (high API results to high \( CR \) for all the storms). Fig. 7a contains the values of \( CR \) for the calibration events plotted against the API. The time of concentration for Giofiros basin was estimated from the Kirpich formula \( t_c = \frac{0.0078}{(L/0.77)^{0.385}} = 3.45 \) h, where \( L \) is the basin’s travel length and \( S \) is the slope along main channel. \( R \) varied from 2.5 to 5.5 h according to API (high precipitation rates results to faster basin response and eventually to lower \( R \) values).

Fig. 7b contains the values of \( R \) plotted against API, for the calibration events. Base flow for each event was estimated and recession ration and threshold flow calibrated at 0.99 and 0.14, respectively. The calibration for the eight events resulted in (Table 2) an average \( R^2 \) of 0.99 and a zero phase error (PE) for all the events. The average volume error was 7.9% and the average value of the peak flow error was approximately 0.18%, while the average Nash and Sutcliffe (1970) criterion was 0.78. Overall, the calibration of the model on the eight events was successful, showing good efficiency.

Flash flood peak flow estimation through hydrologic and hydraulic simulation

Time series of 1 km\(^2\) spatial resolution hourly precipitation fields were generated from the three rainfall stations of Giofiros basin according to the method that was previously described. The effect of initial deficit on the flood peak discharge estimate was tested and proved to slightly affect the result. The \( I_d \) was set at 8 mm, used as a fitting parameter to the measured hydrograph until the stage gauge failure (Fig. 8). The \( CR \) based on Fig. 7a was estimated at 7.1 mm/h and \( R \) at 1.5 h based on Fig. 7b. The simulated flow hydrograph with the data from the damaged river gauge is presented in Fig. 8. The estimated total water volume was 5.20 Mm\(^3\). The peak flow value was 296 m\(^3\)/s. The maximum precipitation intensity as well as the center of mass of precipitation time series was observed at 20:00 of 13 January 1994, while the peak discharge based on the simulation was observed at 24:00 (Fig. 8). According to the rain-gauge measurements and the meteorological analysis, high precipitation rates were observed over the
south part of the basin far from the outlet. Moderate precipitation was recorded at the center and north part of the watershed. Post event field campaigns confirm the timing of the flood peak through witness interviewing. An attempt to examine the propagation of uncertainty of the CR and R parameters on the simulated peak flow made by adding a percent of variation to these parameters, from –10% up to 10%. In addition, different precipitation realizations in terms of temporal (±1 h offset) and intensity distribution (±40% peak precipitation intensity) between the stations with daily and the one with hourly precipitation as described in Section “Reconstruction of precipitation realizations of variable spatiotemporal distribution”, constructed to examine the uncertainty of precipitation records. A set of 343 different parameter combinations and precipitation realizations, used in the hydrological model, resulting to an ensemble of output hydrographs. The effect of the CR and R parameters and precipitation component variation on the hydrographs and peak flow is presented in Fig. 8 within the uncertainty area (boundaries). The uncertainty boundaries are defined by the ensemble area formulated by the 343 output hydrographs. Peak flow shows a linear rate of decrease as Clark storage coefficient (R) increases (shorter response time of the basin). The rate of decrease is expressed by the equation peak flow = –43.79R + 361.91 taking into account a CR = 7.1 mm/h and values of R varying from 1.35 to 1.65. Moreover, peak flow shows a linear rate of decrease as CR increases (higher infiltration rates). The rate of decrease given by the equation peak flow = –26.28CR + 504.03 for R = 1.5 and values of CR varying from 7.1 to 8.9 mm/h.

HEC-RAS was used for hydraulic modeling in order to validate the flood extent at a control cross-section (cross-section with measured flood high-water marks) and to simulate the flood wave. The area of simulation is presented in Fig. 1 as the outlet sub-basin. The flood hydrograph resulted from rainfall–runoff modeling used as input for the hydraulic model and the geometric setup of the model was based on information retrieved by cross-sections and the DTM of the study area. A three dimensional flood wave simulation is presented in Fig. 9 at a 3-h time step. The simulated flood extent corresponds to the measured flood extent in the control cross-section as presented in Fig. 10. The observed value of the flood peak flow level was 5.1 m and the simulated value was 5.08 m at 24:00 January 13. The maximum value of average flow velocity at the control cross-section and the corresponding Froude number were estimated at 3.62 m/s and 0.62, respectively for January 13th at 24:00. Moreover, the propagation of uncertainty through the hydraulic model was examined. The previously presented ensemble of 343 hydrographs imported to the hydraulic model providing 343 realizations of the maximum flood extent for the control cross-section. The maximum flow level varied between 4.96 m and 5.19 m, the maximum average flow velocity varied from 3.47 m/s to 3.77 m/s and the corresponding Froude number varied from 0.61 to 0.64.

**Flash flood peak flow estimation from post flood measurements**

Because there were no observed field flow data, the evaluation of the results was based on an indirect method for the estimation of the real peak flow, from a specific cross-section called Finikia (Fig. 10). This cross-section was at the location where the water flow level gauge was located, during the flood. Eq. (5) was applied using S (energy slope) equal to 0.003 m/m as calculated from the stream's topography. Giofios is a natural stream channel with fairly regular sections with some grass weeds and little brush in it. Flood plains, during winter time, are covered by short grass and medium to dense bushes. The literature (Arcement and Schneider, 1989; Chow et al., 1988) suggests n1 value of 0.03 for the main channel (the main channel is a weedy – stony earth channel) and n2 values for floodplains from 0.04 (short grass with minor obstructions and moderate rises and dips) to 0.08 (medium to dense brush with appreciable obstructions and very irregular shape of floodplain). Flow rates were available from the existing rating curves for up to 2.8 m water level. Furthermore the total flow was estimated for selected values of water level, starting from 3.0 m up to 5.1 m (Fig. 11). Roughness coefficient is the most important parameter of Eq. (5). A sensitivity analysis of flow output for different values of n1 ranging from 0.03 to 0.04 and for n2, ranging from 0.04 to 0.08 was performed. Table 3 contains the parameters of Eq. (5) for various flow levels up to 5.1 m that was the maximum flood level of the control cross-section. The results of this analysis showed that Q varied from 220 m³/s for n1 = 0.04 and n2 = 0.08 to 350 m³/s for n1 = 0.03 and n2 = 0.04 (Table 3). Peak flow for 1994 flood was estimated using Eq. (5) at

![Fig. 10. Channel-geometry characteristics of the Finikia River gauge control cross-section of the 1994 flood and measured flood extend.](image-url)

![Fig. 11. Water level stage-flow curve at the cross-section of Finikia (grey triangles corresponds to the direct field measurements).](image-url)
292 m³/s for \( n_1 = 0.03 \) and \( n_2 = 0.06 \), compares well with the simulation result based on hydrological model.

**Flash flood peak flow estimation based on an empirical method**

A number of non-flood causing rainfall events when the water-stage gauge was operating were used for the estimation of the parameter \( \delta \) of Eq. (4). The hydrologic characteristics of the rainfall–runoff events are included in Table 4. When the total runoff volume \( V_T \) (m³) for a sufficient number of gauged events are plotted against the product of the square of total precipitation \( P \) and the standard deviation of precipitation \( \sigma_P \) (Fig. 12), a linear relation is evident described by Eq. (2) with \( a = 215 \) as a coefficient for Giofiros basin. The extrapolation of the regression line meets the values of the 1994 flash flood event. Moreover, when the total runoff volume \( V_T \) for the same events, are plotted against the product of the peak discharge \( Q_{\text{peak}} \) and duration of precipitation time series \( D \) of each event (Fig. 13) provides the power regression Eq. (3) with \( b = 0.0118 \) and \( c = 1.3702 \) as coefficients for Giofiros basin. The elimination of \( V_T \) from Eqs. (2) and (3) results in \( \delta = 18.5 \) and consequently to Eq. (17), so given the coefficients and the precipitation time series allows estimating the peak discharge of an event for Giofiros watershed.

\[
Q_{\text{peak}} = 18.5 \frac{D}{(P^2 \sigma_P)^{1.3702}}
\]  

Based on Eq. (17) the peak flow of the gauged and the flash flood events was estimated. Table 4 contains the estimated peak flow. Peak flow for the 1994 flash flood event based on the empirical

---

**Table 4**

Variation of flow for different flow level and \( n_1 \) and \( n_2 \) values.

<table>
<thead>
<tr>
<th>( n_1 )</th>
<th>( n_2 )</th>
<th>( A_1 ) (m²)</th>
<th>( A_2 ) (m²)</th>
<th>( P_1 ) (m)</th>
<th>( P_2 ) (m)</th>
<th>( R_1 ) (m²/m)</th>
<th>( R_2 ) (m²/m)</th>
<th>( S ) (m/m)</th>
<th>( Q_{\text{total}} ) (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.03</td>
<td>20.53</td>
<td>0</td>
<td>11.9</td>
<td>0</td>
<td>15.9</td>
<td>0.53</td>
<td>38.57</td>
<td>0.85</td>
</tr>
<tr>
<td>3.2</td>
<td>0.03</td>
<td>22.23</td>
<td>0</td>
<td>12.31</td>
<td>0</td>
<td>17.17</td>
<td>0.53</td>
<td>38.57</td>
<td>0.85</td>
</tr>
<tr>
<td>3.4</td>
<td>0.03</td>
<td>25.14</td>
<td>0</td>
<td>13.18</td>
<td>0</td>
<td>18.84</td>
<td>0.53</td>
<td>38.57</td>
<td>0.85</td>
</tr>
<tr>
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<td>28.18</td>
<td>0</td>
<td>14.05</td>
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<td>20.61</td>
<td>0.53</td>
<td>38.57</td>
<td>0.85</td>
</tr>
<tr>
<td>3.8</td>
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<td>31.3</td>
<td>0</td>
<td>14.92</td>
<td>0</td>
<td>22.39</td>
<td>0.53</td>
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<td>0.85</td>
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<td>24.17</td>
<td>0.53</td>
<td>38.57</td>
<td>0.85</td>
</tr>
</tbody>
</table>

---

**Fig. 12.** Total runoff volume \( V_T \) (m³) as a function of precipitation squared times its standard deviation.

**Fig. 13.** Total runoff volume \( V_T \) (m³) as a function of the product of the peak discharge and the duration of precipitation for each event.
Empirical equation rationale

Based on hydrological characteristics of the 1994 flash flood event \( (P_T, \sigma_P, D, V_T) \) we examine the rationale of the empirical Eq. (4). The duration of the event was \( D = 19 \) h, the total areal precipitation over the basin was \( P_T = 75.5 \) mm, the standard deviation of the precipitation time series was \( \sigma_P = 4.36 \) mm and the total volume based on rainfall–runoff simulation was estimated \( V_T = 5.20 \) Mm\(^3\). Assuming a set of the same total precipitation and duration but of different standard deviation varying from 1 mm up to 6 mm increasing by 0.5 mm, the expected peak flow was estimated to be 287 m\(^3\)/s. Fig. 14a illustrates the expected peak flow versus a precipitation event with the same total precipitation and duration of the 1994 flash flood event but different temporal distribution of rainfall expressed by the standard deviation of the precipitation time series. Precipitation distributions with higher standard deviations results in higher peak discharges. By keeping the total precipitation and increasing the standard deviation by 20%, which corresponds four times higher precipitation intensity at the 2-h peak of the storm, the peak flow was estimated via Fig. 14a at 365 m\(^3\)/s which agrees with the value obtained with the other two methods. With the duration varying from 13 h up to 46 h in increments of 3 h, but keeping the total precipitation and standard deviation the same, the peak flow was estimated. Fig. 14b shows that shorter duration events correspond to higher peak discharges.

Conclusions

During many flash flood events there may be a water-stage gauge failure or the flood stage far exceeds the maximum value of the gauge's rating curve that makes the estimation of the peak discharge difficult. A method for estimating flash flood peak discharge, hydrograph, and volume is presented that uses the measured rainfall data and river hydrographs in a number of non-flood causing rainstorms with an operating water-stage gauge for calibration and verification of the simulated stage-discharge hydrographs. An empirical equation also was developed in order to provide the peak discharge as a function of the total precipitation, its standard deviation and storm duration while the value of the coefficients depend on the characteristics of the basin. The peak discharge for a flash flood case based on the empirical equation, is in agreement with the one calculated via the verified hydrological model and the one derived based on Manning’s equation and post flood measurements of the control cross-section (e.g., all values were close to 290 m\(^3\)/s for the 1994 flood on the Giofiros River). These methods can be applied to other poorly gauged basins facing common stage gauge failures or poorly defined rating curves during flash floods caused by severe convective rainstorms. The coefficients that represent the characteristic of the basin have to be developed for the individual basin.

A severe storm that produces a flash flood can be approached with a variety of methods. Among others, meteorological analysis, hydrological modeling, hydraulic modeling and analysis, post event campaigns for data retrieving (flood marks, peak flow timing through interviews) can be used to provide additional information for reliable peak discharge estimations. This multilateral approach,
References


