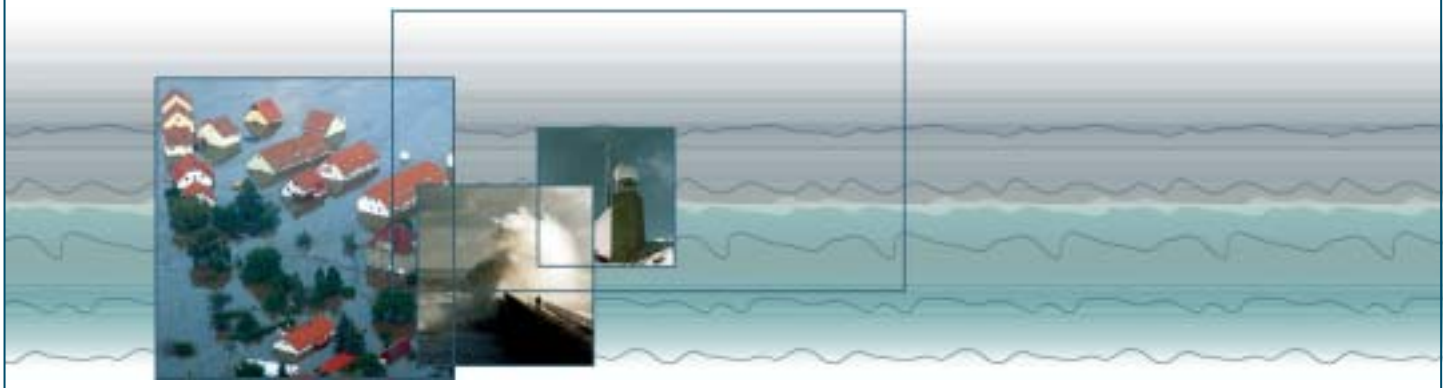
Integrated Flood Risk Analysis and Management Methodologies



Post Flash-flood Investigations

METHODOLOGICAL NOTE

Date **February 2006**

Report Number **T23-06-02**
Revision Number 1_0_P01

Deliverable Number: D23.2
Due date for deliverable: February 2006
Actual submission date: February 2006
Task Leader UniPad

FLOODsite is co-funded by the European Community
Sixth Framework Programme for European Research and Technological Development (2002-2006)
FLOODsite is an Integrated Project in the Global Change and Eco-systems Sub-Priority
Start date March 2004, duration 5 Years

Document Dissemination Level

PU	Public	PU
PP	Restricted to other programme participants (including the Commission Services)	
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CO	Confidential, only for members of the consortium (including the Commission Services)	

Co-ordinator: HR Wallingford, UK
Project Contract No: GOCE-CT-2004-505420
Project website: www.floodsite.net

DOCUMENT INFORMATION

Title	**Post Flash-flood Investigations - Methodological Note**
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Contributors	
Distribution	**Public**
Document Reference	**T23-06-02**

DOCUMENT HISTORY

Date	Revision	Prepared by	Organisation	Approved by	Notes
15/03/06	1_0_P05	E. Gaume	ENPC		
17/05/06	1_0_P01	J Bushell	HRW		Formatting; change of name from 'D23.2.doc'

ACKNOWLEDGEMENT

The work described in this publication was supported by the European Community's Sixth Framework Programme through the grant to the budget of the Integrated Project FLOODsite, Contract GOCE-CT-2004-505420.

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SUMMARY

Post event survey and investigation is one way to gain experience on natural hazards. The importance of the systematisation and standardisation of such investigations and re-analysis is progressively recognised in all the geophysical sciences as shown by the growing number of scientific papers and programs on the subject.

Large research efforts have been made on the analysis and modelling of the meteorological aspects of the flash-flood triggering storms (see for instance the proceedings of the European Geophysical Union Plinius conferences on Mediterranean storms) or on landslides and concentrated flows. In comparison, the analysis of the dynamics of the runoff processes during flash-floods is still at its infancy. The main limiting factor for the development of flash-flood studies has probably been the lack of accurate measured rainfall and discharge data.

Most of the existing reports on flash-floods are restricted to measured point rainfall intensities and some peak discharge estimates, generally for gauged river cross-sections. But recent works conducted in France (Delrieu et al., 2005; Gaume et al. 2004a; Gaume et al. 2003) have demonstrated that additional valuable data can be gathered after major flood events even on ungauged watersheds. These data, mainly peak discharge estimates based on flood marks and sometimes on films and partial time sequences of floods based on witnesses' interviews, can be used in combination with rainfall estimates to analyse the dynamics of the rainfall-runoff processes on the affected watersheds. This opens new perspectives: with the help of the Radar rainfall estimations it is possible to analyse the flash-floods wherever they occur and not only on well gauged watersheds – when by chance the gauges have not been damaged by the flood - and at the appropriate time and space scales.
This report aims at sharing the experience gained with the hope that it will help to increase the number of post flash-flood studies, which is a necessity since our common knowledge on flash-floods will only grow through the accumulation and inter-comparison of case-studies.
Note that this report is focussed on the analysis of hydrological processes, but other issues may also be considered during a post-flood investigation: the hydro-meteorological, geo-morphological as well as socio-economical aspects.

This report is a first attempt to formalize a post-flood field investigation procedure. The proposed approach will certainly be improved with the time and this will be the sign that a growing number of hydrologists are involved in post flood investigations, at least we hope so. Constructive critics are very welcome. The proposed method has been developed and tested at the national scale for the purpose of the Gard 2002 post flood investigation (Delrieu et al., 2005; Gaume et al., 2003b): 18 hydrologists from 8 different institutions were involved. The Floodsite project will give the opportunity to test it on further case studies with international teams.

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1. Post flash-flood investigations and field surveys what for?

The question formulated in that manner may appear a little surprising. In fact, flash-floods[1], rank as the most destructive process among weather-related hazards in many parts of the world. Not studying these extreme events because no measured data are directly to-hand, or if so, because they are not considered as sufficiently accurate, or even because it is time consuming, and limiting the hydrological analysis to moderate events on gauged watersheds, would be focussing on the trivial while skipping the essential.



Figure 1: Nîmes (France), 3rd of October 1988

The potential usefulness of flash-flood studies is not in question. It appears clearly as a necessity to increase the existing knowledge on such events to provide adapted methods of analysis and technical solutions for flood prevention and control. The question is rather how to proceed, what type of data should be collected for what type of analysis and to explore which particular questions.

The analysis of the past experiences, shows that two main types of post-flood investigations can be distinguished which differ by their objectives and context. The first type is generally commissioned by the local or national authorities after a major catastrophe. The main objective is to answer questions raised by the public opinion and the local stakeholders on the causes of the floods, the possible human impacts on the flood magnitude and frequency, but also on the management of the crisis, the efficiency of the flood mitigation measures and the solutions to recover from the flood and to limit the future risks (Huet, 2005). Typical examples are the investigations conducted after the major 1987 floods in Switzerland (Bundesamt fur wasserwirtschaft, 1991) or more recently in France (Huet et al., 2003; Lefrou et al. 2000) or in Algeria (Recouvreur, 2005). The purposes of such investigations are well defined and limited to the raised questions. Scientists are generally involved either to conduct studies on some specific questions or to take part to scientific support groups. Research activities may be conducted during such investigations, but it is then a by-product. The objective is mainly to draw the lessons of the event at the local scale and not to increase the overall scientific and technical knowledge.

[1] Sudden floods with high peak discharges, produced by severe thunderstorms that are generally of limited areal extent (International Association of Hydrological Sciences, 1974)

A second type of post-flood investigations is conducted by technical services like the U.S. Geological Survey[2] or the IRPI in Italy (Istituto di Ricerca per la Protezione Idrogeologica) for instance or by research institutions[3]. The aim is then to document (i.e. describe) the extreme events. Most of the past works have been limited to a description of the event through the available measured data (rain gauge or river gauge measurements) and some field observations as cross-section surveys and corresponding peak discharge estimates (Rico et al., 2001; Rey and Rouiller, 2001; House & Pearthree, 1995; Gutknecht, 1994; Hémain & Dourlens, 1989; Dacharry, 1988; Costa, 1987; Jarrett, 1987). Sometimes the description of the mass transfer processes, of their localisation and the estimation of the transferred volumes is provided (Alcoverro et al. 1999; Cariedo et al., 1998, Lajournade et al., 1998). A detailed rainfall-runoff analysis of the event is rarely done due to the lack of measured rainfall and discharge data.
The inventory of the extreme events and their peak discharge values is of course important to define the range of the possibilities, to built envelope curves and to study the regional patterns of the river flood extreme peak discharges (O'Connor and Costa, 2003; Perry, 2000, Parde, 1958), or to reduce the uncertainties in flood frequency analysis (Payrastre et al., 2005).
The recent developments of the measurement networks, especially the weather radar networks, open new perspectives for the analysis of flash-floods. The weather radar provides rainfall estimates at appropriate space and time resolutions. It seems therefore now possible to get deeper into the analysis of the rainfall-runoff dynamics of the watersheds (Delrieu et al., 2005; Sächsisches Landesamt für Umwelt und Geologie, 2004*;* Gaume & Bouvier, 2004b; Gaume et al., 2003; Gaume, 2001; Belmonte and Beltran, 2001; Ogden et al., 2000, Smith et al., 1996). This opens the possibility to work on important issues and to answer question as:

- What is the rainfall-runoff dynamics during a flash-flood, and what is the influence of the watershed characteristics, of the initial soil moisture or ground water recharge conditions on this dynamics?
- As a subsidiary question, what type of watershed characteristics (slopes, land use, geology, soil types…) should be considered in a regional flood frequency analysis?
- What are the dominant flood generating processes during a flash-flood?
- Is the answer to this question depending on the land-use and geo-morphological properties of the watershed?
- What part of the catastrophe can be attributed to anthropogenic factors (change in land use, deforestation, agricultural drainage, imperviousness, road network, river management)?
- Are the dominant processes the same during flash-flood events and medium flood events, and is it possible to extrapolate tendencies observed on medium flood events (flood frequency distributions, rainfall-runoff models)?
- What is the influence of "artificial" processes like blockages and their breaking ups, or of the solid load (i.e. mainly water flood versus hyper-concentrated or even debris flow) on the peak discharge and the shape of the rising limb of flood hydrographs?
- How do the existing flood forecasting models perform on such events?

Due to the time-space characteristic scale of flash-flooding, the majority of the upstream catchments affected by these floods are not gauged[4]. In addition, the peak discharges appear to be spatially highly

[2] Carter et al., 2002; Winston & Criss, 2002; Juracek et al., 2001; Bowers, 2001; USGS, 2001; Grigg et al., 1999, Slade & Persky, 1999.
[3] Marquet, 2000; Gilard & Mesnil, 1995; Cemagref, 1996; DDE du Gard, 1996 ; Cemagref, 1994 ; Hemain & Dourlens, 1989 ; Ville de Nîmes, 1989.
[4] It should be noted that the existence of a streamflow measuring station that remained undamaged during the flood does not mean that an accurate discharge value can be provided. As mentioned by Costa (1987), the short duration of rainfall able to produce floods in small watersheds and the danger associated with accompanying high-water velocities, sediment and debris, preclude the possibility of obtaining direct current-metre measurements for extremely large floods. As illustrated by several reports (Delrieu et al., 2005; Gaume et al., 2003b), the water levels reached during these events are often far greater than the range for which stage-

heterogeneous, even within small catchments: i.e. complementary data can also be useful on gauged watersheds. A detailed flash-flood study should not be limited to the few gauged river cross-sections if some exist. Flash-floods are by definition rare events. If an intensive research activity is to be set up on these hydrological events, it is necessary to develop specific methods to collect and analyse the existing information about the floods when and where they occur and not to limit the analysis to the few events affecting gauged watersheds.

This report, based on past experiences of post-flood studies, is a first attempt to propose some guidelines on how to identify, collect and analyse data available after a major flash-flood event. Three main types of data will be considered.

- Indicators of the peak discharge values: mainly cross-section surveys based on flood marks but also clues of flow velocities (video movies, witness observations, water super-elevations in river bends or in front of obstacles). The report presents and criticizes various indirect post-flood peak discharge estimation methods and puts the emphasis on the cross-validation procedures.
- Indicators of the time sequence of the flood: mainly eyewitness accounts where no stream gauge measurements are available. Accounts from eyewitnesses are occasionally cited in flash-flood studies, they have seldom been, to our knowledge, systematically collected and analysed. This report provides a methodology to collect and analyse eyewitness information and discusses the reliability of this source of information.
- Mass transfer processes (erosion and deposits on the slopes and in the river bed, hyper-concentrated, mud or debris flow) as the main focus of the post-flood investigation but also as an indication of the local flow energy and velocity.

Information on socio-economical aspects can also be collected like geo- and time- references of accidents, qualitative description of public behaviour, effectiveness of warning broadcasts, nature and extension of the damages caused to bridges, roads and buildings, but will not be discussed herein.

This report ends with some illustrations of the hydrological valuations of the collected data. This, we hope, will convince the readers that the conclusions that can be drawn from post-flood investigations are worth the time spent to collect and analyse the data. Our common knowledge on flash-floods will only grow through the multiplication of post-flood field surveys for two main reasons. The conclusions drawn on one single event, based on inaccurate and partial data may be questionable and will be consolidated on the basis of repeated post-flood analysis. Various case studies are needed to determine whether the hydrological behaviour described for one flash-flood is a general pattern for the considered region or type of watershed or is an outcome of spatial and temporal specific circumstances (i.e. rainfall pattern, wetness state of the soils, soil types, geology of the watersheds, etc...).

We hope that the guidelines presented herein will contribute to the systematization of post flash-flood field investigations.

discharge relations can be gauged. Consequently, peak discharge values can only be estimated on the basis of existing hydraulic know-how.

Check list for a post flash-flood survey

Phase 1:

Just after the flood

- Collect the data on the rainfall event (rain gauge measurements, radar images) to locate the affected areas
- If possible, first reconnaissance visit of the affected areas, pictures can be taken, but no survey work can generally be conducted during the crisis time.

Phase 2:

A few weeks after the flood

- The cross-section surveys can begin as well as some interviews of witnesses depending on the local atmosphere.

Phase 3:

A few months after the flood

- It is certainly the best period for the survey work especially for the interviews. The area is fully accessible and the stress has fallen again. The river beds may have been cleaned out, this is why the pictures taken in phase 1 or 2 are important.
- Collect additional data useful for the analysis (river gauge measurements, digital terrain model, soil, land-use, geological map, soil moisture measurements, satellite or pictures taken by plane, flood marks inventories…)
- Preparation of the rainfall-runoff simulations to support the interpretations.

Phase 4:

The year after the flood

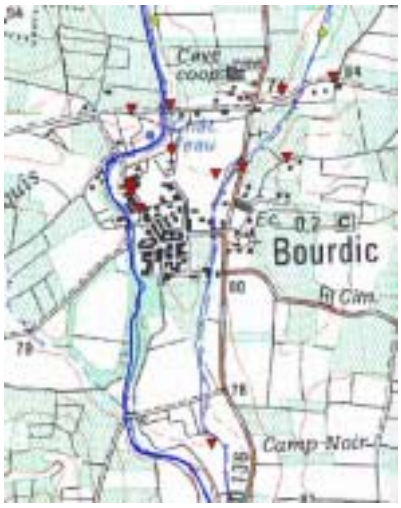
- Due to the inaccuracy of the available data, a post flood investigation has some similarities with police inquiries. It is a long lasting work, requiring cross-checking and possibly returns to the phase 3.

2. Preparation of the field investigation

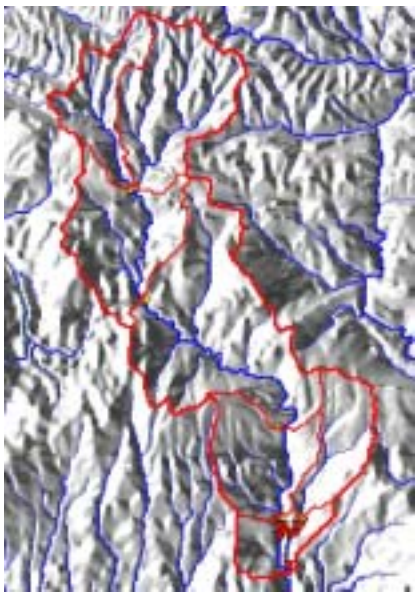
2.1 Analysis of the available data

2.1.1 Geographical data

We will here limit ourselves to the listing of the geographical maps or data bases, potentially useful for post-flood surveys and analysis. The Bourdic watershed, exposed to the 2002 heavy storms in the Gard region (France), is used for the illustrations.



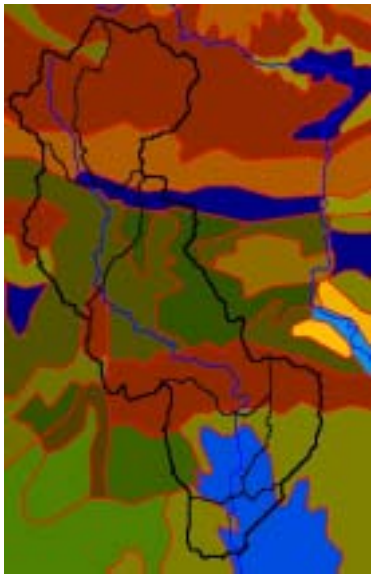
1. **Standard geographical map.** It is of course useful to prepare the field survey: identify the river valleys, the accesses to the river for the cross-section surveys, the towns and the possible flooded houses and buildings where interesting interviews could be collected. It is also the ideal background if the collected data - interview summary forms (red triangles) and cross-section survey forms (yellow dots) in the example – are put on a Geographical information system. A 1/100.000 scale is sufficient for the field study preparation and as a background image in a GIS. 1/25.000 maps may be useful on the field, especially to identify the accesses to the rivers. Note that in rural areas, the maps may be relatively old and not completely up to date. The figure shows here a bitmap scan of the Institut National Geographique (IGN) map. GIS layers may also exist (IGN TOPO database in France)



2. **Digital elevation model.** The digital elevation model of the studied region may be useful to extract automatically the limits and the topographical characteristics of the studied watersheds. Many commercial or free tools are available to achieve this task. It is also an input for distributed hydrological models. The figure shows the computed shadows due to the relief (the sun is supposed to be located in the north-west) and the extracted sub-watersheds corresponding to the seven surveyed river cross-sections. The Bourdic town lies in the downstream part of the watershed (south). The HYDROKIT software, developed for the Gard region flood forecasting service (France) by the company Strategis has been used here.



3. **Geological map.** The geology is one of the watershed characteristics which may influence its hydrological behaviour. The figure shows a bitmap scan of the geological map produced by the Bureau de Recherche Géologique et Minière (BRGM).



4. **Soil map.** Like the geology, the soils may have an influence on the runoff generation processes and their dynamics. Note that soil and subsoil are connected, and the spatial repartition of the soil types is correlated to that of the sub-soil material. The figure shows a GIS soil layer produced by the Institut National de la Recherche Agronomique (INRA).



5. **Land use map.** Again, the land use can explain variations in the rainfall-runoff responses of the watersheds. Note that the land use is generally closely linked to the soil types and geology. The figure shows the Corinne land cover land use GIS layer: delimitation of surface elements with homogeneous land use (red lines).

2.1.2 Rainfall measurements

The spatial density of 5-min or hourly rain gauge networks is necessarily limited due to the management costs of such networks. About 1000 gauges are in operation in France for instance which corresponds to a density of 1/500 km^2 (i.e. mean distance of about 22 kilometres between adjacent gauges). This is clearly too low to catch the spatial pattern of rainfall accumulations over time steps lower than the day. The typical diameter of flash-flood producing convective rainfall cells is about 10 kilometres. They are therefore very often located between the available gauges. Moreover, the spatial correlation structure of rainfall fields depends on the considered time step. Lebel et al. (1987) proposed the following empirical relationship: $d_0 = 25(\Delta t)^{0.3}$, relating the variogram range d_0 (km) and the rain accumulation time step Δt (hours) for the Gard region in France. The mean inter-distance between gauges (22 kilometres) is close to the variogram range (de-correlation distance) for a 1-hour time step (25 kilometres). This means that a linear spatial interpolation of the measured 1-hour rainfall rates is of no additional value.

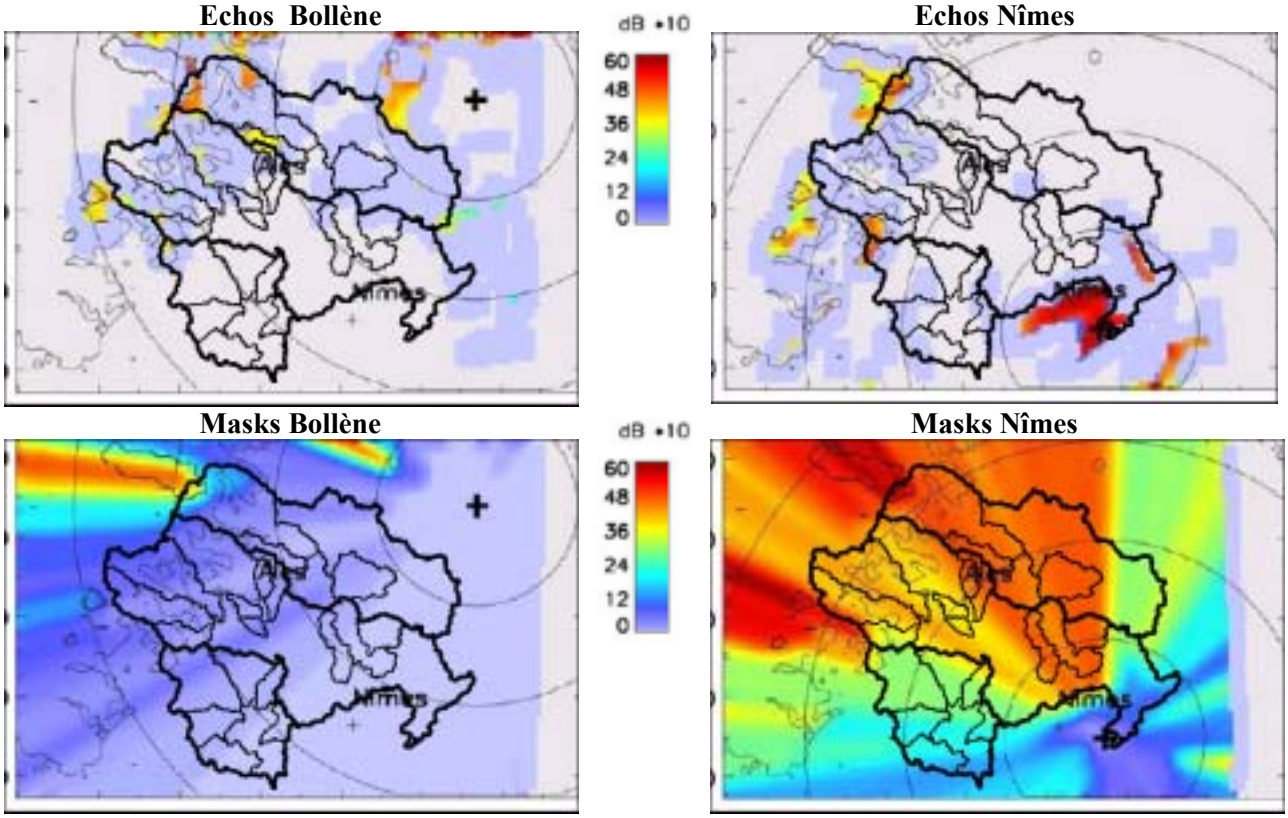


Figure 2: Map of the partial masks and echos around the radars of Bollène and Nîmes in decibels (Kirstetter, 2004). The circles around the radars have a radius of 25, 50, 75 et 100 kilometres.

This explains why the weather radar measurements are so important for flash-flood monitoring and studies. This indirect rainfall rate measurement technique is subject to various uncertainty or perturbation sources: electronic calibration defaults, partial or total masks due to the relief, ground echos, variation in the drop size distributions, inhomogeneous vertical reflectivity profiles, attenuation of the radar beam due to rainfall among others. The radar measurement technology is improving, and some of the uncertainty sources can be analyzed and corresponding correction factors proposed as illustrated in Figure 2 and Figure 3 for the weather radars located in Nîmes and Bollène (French Gard region) and the 8th and 9th of September 2002 storm event.

Nevertheless a real quantitative valuation of the radar reflectivity measurements can not be based on a mean relation between the measured reflectivity Z in mm^6 m^{-3} and the rainfall rate R in mm h^{-1} ($Z = 216R^{1.54}$ for the Nîmes and Bollène radars for instance). But it necessitates a calibration of this relation based on a radar –rain gauge comparison for each individual rainfall event.

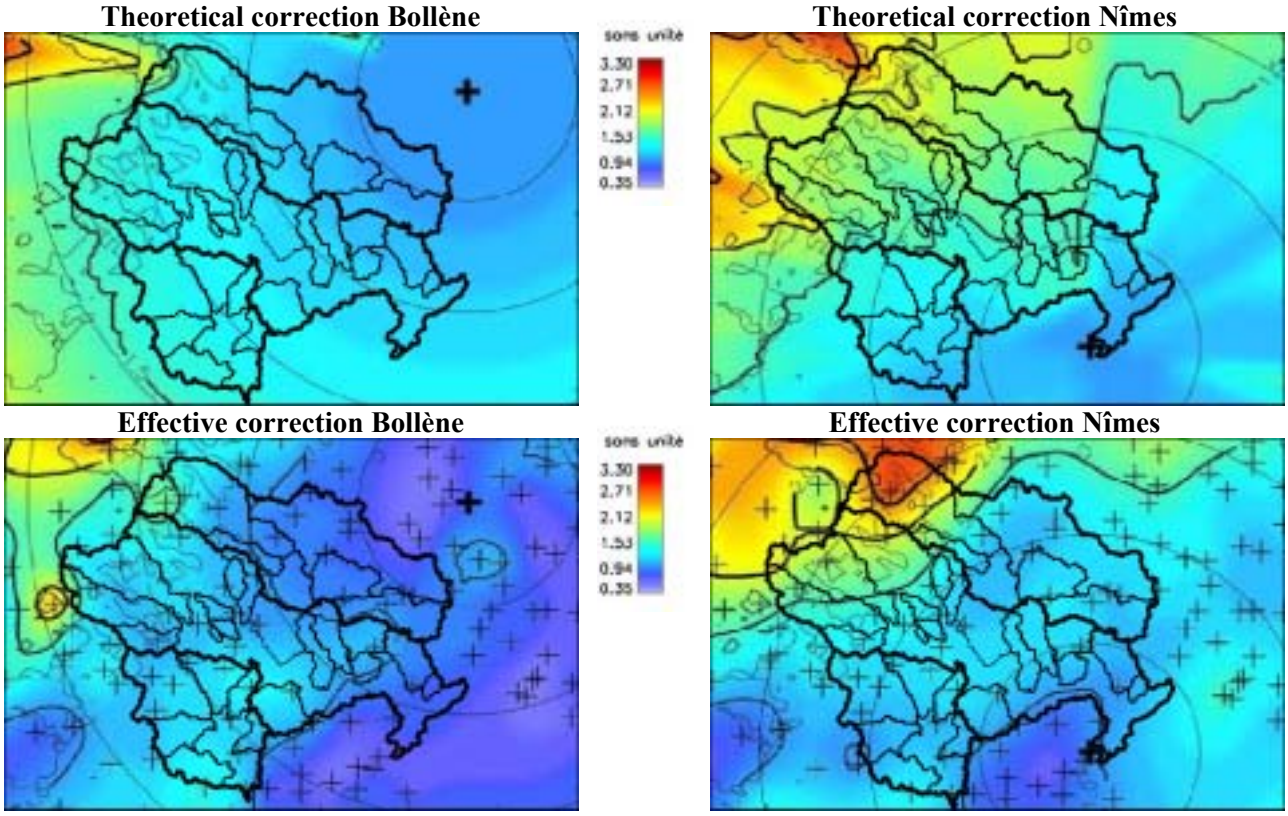


Figure 3: Spatial repartition of the radar calibration coefficients for the 8th and 9th of September 2002 rainfall amount computed by comparing the theoretical amounts measured by the radars and the interpolated measured amounts at the gauges (effective correction) and calibration coefficients determined theoretically on the basis of the identified masks (Kirstetter, 2004).

In a post-flood analysis perspective, it is preferable to use a robust calibration method: use a single Z-R relation for the whole rainfall event and applied uniformly in space after having corrected the reflectivity data file (Delrieu *et al.*, 2005). This is equivalent to calibrating the radar data on the mean rainfall amount of the considered event measured by the available rain gauges in the considered area. The more detailed available rainfall measurements (distribution of the rainfall amounts in space and time) can then be used to verify the accuracy of the radar rainfall estimates as illustrated in Figure 4. As shown on this figure, the 30-min rainfall rates estimated on the basis of the radar measurements are in a more than reasonable agreement with the measured ones in the central part of the region where the most severe storms occurred during the 8th and 9th of September 2002, while the radar seems to significantly over-estimate the rainfall rates in the downstream part of the watersheds, especially at the beginning of the rainfall event. There seems also to be temporal inaccuracies in the radar hyetographs for the upstream part of the watersheds.

The radar rainfall measurement technique is far from perfect. It is therefore important to conduct some validation tests as shown here, before a quantitative use of the estimated rainfall rates as input of rainfall-runoff models for instance.

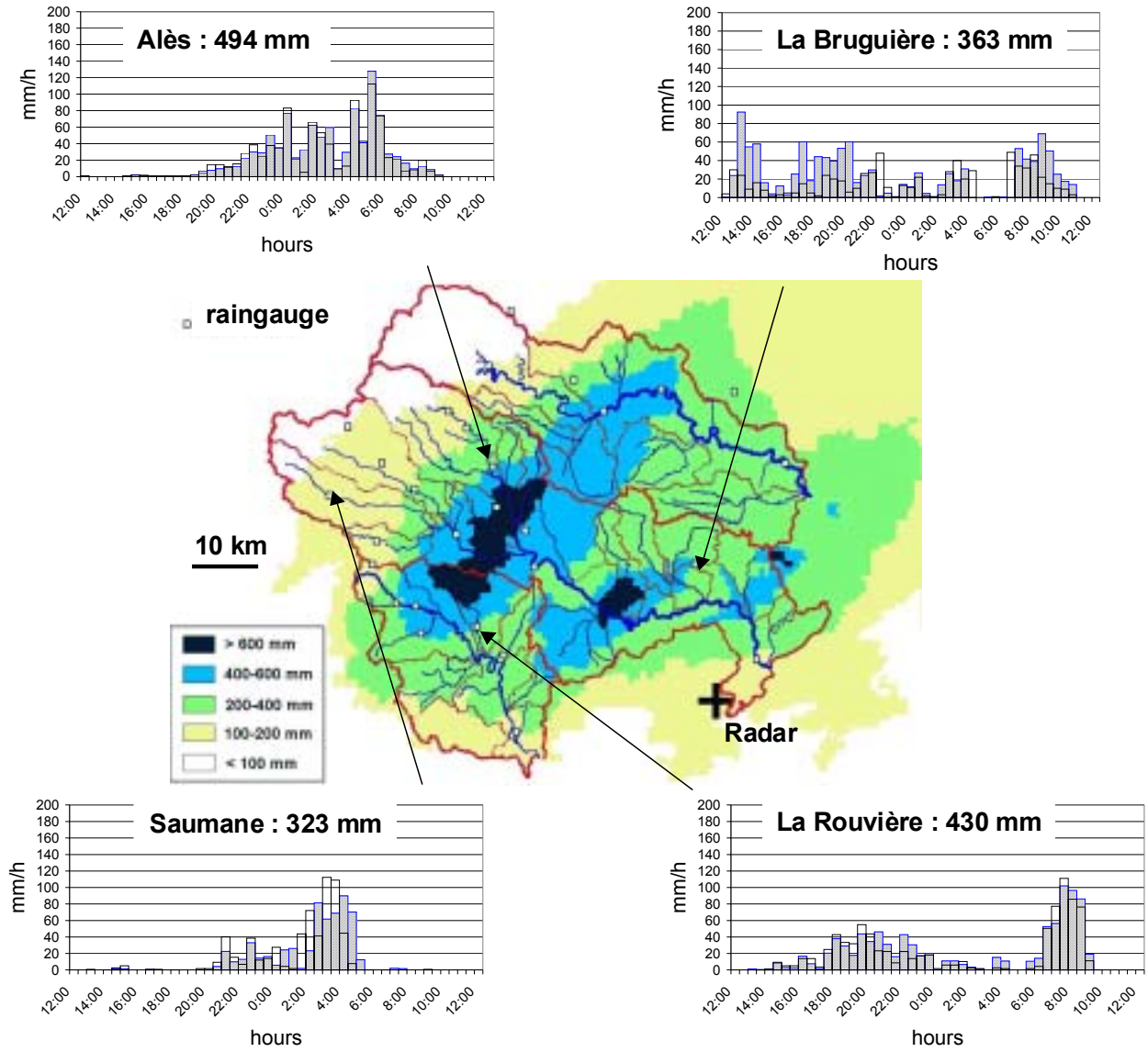


Figure 4: 5-min rain gauges location in the Gard region, calibrated radar rainfall amounts for the 8 and 9 September 2002, and comparison between radar (blue histogram) et rain gauge (white histogram) hyetographs

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2.1.3 River stage measurements

The existence of a streamflow measuring station that remained undamaged during the flood does not mean that an accurate discharge value is available. As mentioned by Costa (1987), the short duration of rainfall able to produce floods in small watersheds and the danger associated with accompanying high-water velocities, sediment and debris, preclude the possibility of obtaining direct current-metre measurements for extremely large floods. As illustrated in Figure 5, the water levels reached during these events are often far greater than the range for which stage-discharge relations (rating curves) can be gauged.

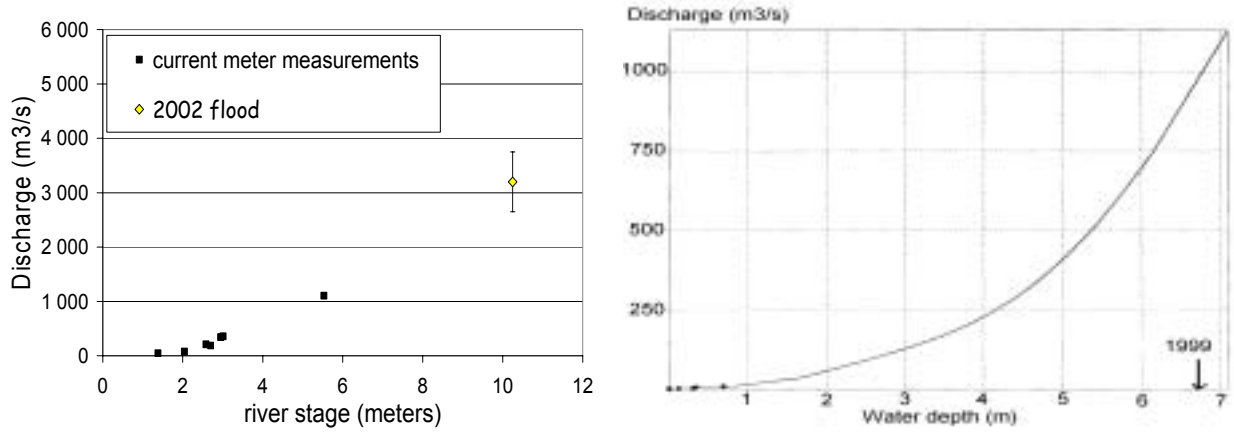


Figure 5: Current-metre measurements (black dots) and theoretical stage-discharge relation for two river gauging stations: (a) Anduze on the Gard river (Gard region) and (b) Luc on the Orbieu river (Aude region). (Source, Directions départementales de l'équipement du Gard et de l'Aude).

Depending on the main focus of the service operating the river gauging stations (water resources management, flood warning or forecasting) and on their means, the direct current-metre discharge measurements are more or less frequent and the range of measured discharges more or less expanded. Measuring the most important flood discharge values is for instance not a priority for services in charge of water resources monitoring. Likewise, until the last years, the French flood warning services did mainly produce flood alarms based on upstream measured river stages and no real forecasts. This explains the extremely low number of current-metre measurements conducted on their river gauging station networks as illustrated in Table 1 for the Gard region.

Station	Number of current-metre discharge measurements	Date o the last measurement	Max. measured discharge (m^3/s)	Estimated 2002 discharge (m^3/s)
Mialet	1	2000	125	850
Ners	0			7000
Saumane	3	2002	75	800
Saint Jean	0			1000
Remoulins	12	1988	1300	5500
Quissac	6	2001	240	900
Vic	5	2002	160	2500
Sommières	6	2002	430	3000
Anduze	7	2000	1107	3000

Table 1: Existing current-metre measurements for the river stage stations of the Gard region flood forecasting service (source, Direction départementale de l'équipement).

The river stage measurements themselves may also be dubious even if the gauging station has not been damaged during the flood. Figure 6 shows two examples of measurement errors of pressure gauges based on bubbling systems during the 2002 Gard region floods. In both cases, the pumps of the stations could not follow the gradient of evolution of the river water stages. This led to a linear evolution of the measured stages at the Anduze station on the Gard river, which reflects probably the response dynamics of the pumping system rather than the actual evolution of the river stages. As a consequence, the maximum stage could not be measured. Whereas the gauging station of Sommières on the Vidourle river did not react immediately as if it were stumped.

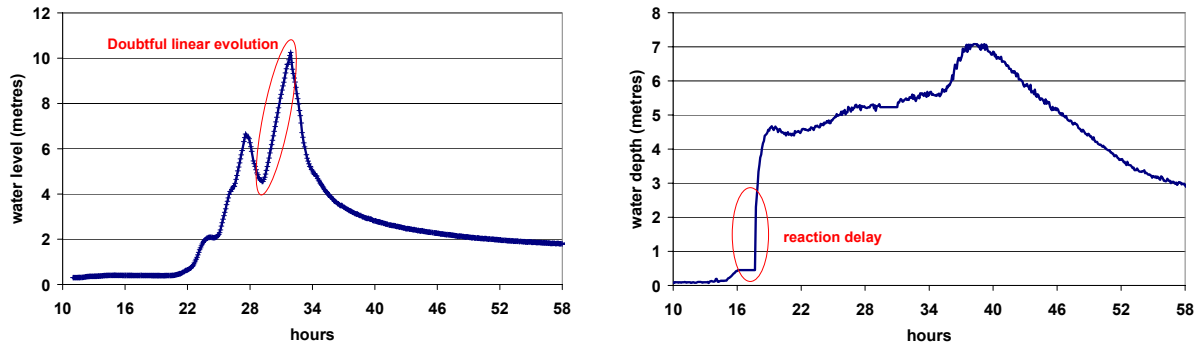


Figure 6: river stage measurements of two gauging stations in the Gard region during the 2002 floods (source, Direction départementale de l'équipement du Gard): (a) Anduze on the Gard river and (b) Sommières on the Vidourle river.

Last but not least, due to the highly transitional conditions, the stage-discharge relation may not be unique. It is well known that for a given river stage, the discharge is higher during the rising limb of a hydrograph than during a decreasing limb. This effect, called hysteresis, depends on the rating-curve, the shape of the cross-section and the gradient of evolution of the discharge with time. It can generally be neglected but may become important in highly non-stationary conditions and when the water flows in a large floodplain.

	La Rouvière	Conqueyrac	Sommières	Anduze	Remoulins
Watershed area (km^2)	91	83	620	544	1855
Rainfall amount (mm)	560	406	404	287	395
Runoff amount (mm)	452 (440)	253 (214)	262 (180)	120-180 (100-140)	230-260 (190-220)
Runoff deficit (mm)	110	150	150	100-170	130-170

Table 2: Water balance on some watersheds during the 8 and 9 September 2002 floods in the Gard region, runoff volume during the two days and during the 8th of September (within parenthesis).

This is illustrated in Table 1 showing the water balance on some watersheds during the 8th and 9th of September 2002. The estimated discharges in Sommières indicate that less than 70 % of the runoff total volume had passed during the 8th of September. This is not in accordance with proportions estimated on the upstream catchments: more than 80% in Conqueyrac and even more than 95% in La Rouvière downstream an impervious watershed. This difference can not be attributed to the transfer times on the watershed as indicated by the same ratios computed on the nearby Gard river: also more than 80% of the total runoff amount during the 8th of September in Anduze and Remoulins. As indicated by the runoff deficit values, the total runoff volume seems to have been relatively well estimated in Sommières, if compared to the upstream stations. But the runoff volume during the 8th of

September, including the rising limb of the hydrograph, has been under-estimated while it seems to have been over-estimated for the decreasing limb during the 9th of September. This is most probably the consequence of the hysteresis effect at the Sommières cross-section.

Like the rainfall data, the river stage measurements and the estimated discharges must be thoroughly analysed and criticized.

2.1.4 Other data of possible interest

Other existing data can also be useful as soil water content or groundwater level measurements if some exist which may help to estimate locally infiltrated water volumes, or at least confirm or infirm conclusions drawn on the infiltrated rainfall volumes.

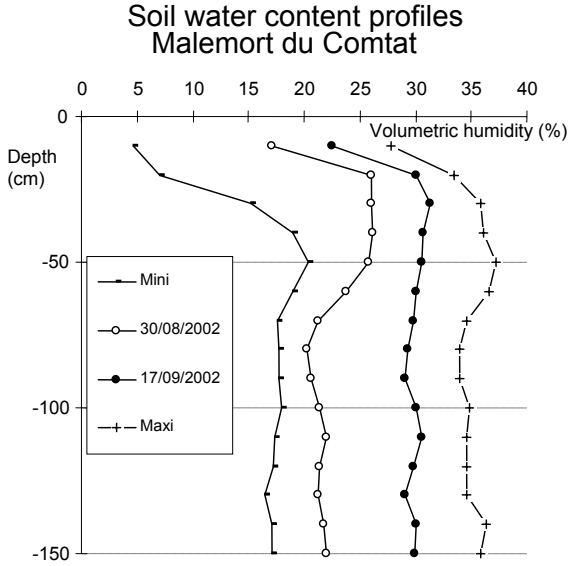


Figure 7: example of monthly measured soil water content profiles conducted to supervise irrigation in an agricultural region (source: Chambre départementale d'agriculture du Vaucluse).

According to Figure 7, about 100 millimetres have been stored in the first 1.5 meters of a soil profile locate in Malemort Comtat between the end of August 2002 and mid September. The area received 200 millimetres during the 8th and 9th of September 2002. This confirmed that despite the high rainfall intensities a large part of rainfall amounts of the first rainfall event of September 2002 did infiltrate, even in areas covered by vineyards which is the case of the Malemort du Combat plot.

2.2 Homogeneity of the collected data: field survey forms

Field investigation forms must be prepared before the field investigation to ensure that homogeneous data will be collected and put in the same format, especially if various people or teams contribute to the data collection. The forms make the use and compilation of the collected data easier. They serve also as checklist on the field. Two examples of forms established for the Gard 2002 post-flood investigations which involved about 20 researchers are presented hereafter: a river-cross-section form and a witness interview form. These, of course, are only suggestions.
Independently of the type of information collected, a form contains four major types of data which should be clearly identified in the form:

1. General information: date of the event, date of the survey, name of the persons involved in the survey, location (if possible GPS coordinates, point on a scanned map or at least description), description of the site and possibly the type of process for solid transfer.
2. The collected data (measured elevations, cross-sections, slopes, interview summary)
3. Pictures which are a necessary complement of the collected data.
4. And possibly, processed data as wetted cross-sections, mean water velocities, discharges, landslides, erosion or deposits volumes.

2.2.1 River cross-section survey report



This form has been developed to apply the so-called slope-conveyance discharge estimation method (see the next chapter of the report). Note that, the measured data (cross-section, high water marks, water surface slope) and the detail of the computations leading to the peak discharge estimate are clearly separated. A sensitivity analysis of this estimate to various sources of uncertainty is conducted and a range of possible discharge values is proposed. The detail of the computation is given, to be criticised and discussed. The empirical Manning-Strickler formula is used for the estimation. The main river bed and the right and left bank flows are considered separately for the roughness coefficient and hydraulic radius estimations. It is not directly the discharge which is estimated but the mean velocity which can possibly also be evaluated by other means and therefore criticized: analysis of video documents, erosive power of the flow.

Figure 8: example of a cross-section survey form

2.2.2 Witness interview account

River : The Lionnais, right hand tributary of the Alzon river

Witness : Mr et Mme Moury
Coordinates : X=736812, Y=1902451 (Lambert II étendu)
Established by : Olivier Payrastre (CEREVE)
Date : le 17 January 2003
Description of the site : House located on the left bank.

Location map



Photograph



Summary of the interview :

- The rainfall began on sunday the 8th of September during the day, and lasted without interruption until the monday in the morning.
- During the night, arround midnight, the house of M. Boilot, our neighbour, which is closer to the Lionnais, began to be flooded. He drove is cars in our garden, and at one o'clock M. Boilot left his house to take refuge in our house for the rest of the night. The water level seems to have risen progressively between midnight and one o'clock.
- The maximum water level - level of the mailbox of M. Boilot - occured on monday between 5h00 and 6h00.
- The water level dropped rapidly after 6h00. Within a quater of an hour there was no water left on the road in front of the house.



The maximum water level reached the mailbox of the house

Other observations :

- -

Information about past floods :

- The road in front of the house has been flooded twice : in 1958 and in 1995. But the maximum water level has never been so high than in 2002.

Figure 9: example of an intervew account form

2.3 Field survey equipment

Let us here close this chapter with a checklist of the useful equipments for a post-flood field survey

• **Digital camera:** illustration of the studied site, exact locations	
• **Tape recorder:** to record the interviews. The experience has shown that it is not absolutely necessary. It may even be a factor of stress for the witnesses.	
• **GPS receiver:** to locate easily the surveyed points and transfer the data into a GIS.	
• **Laser distance-metre:** to measure cross-sections of culverts, bridges, height of flood marks in buildings.	
• **Digital laser theodolite:** for cross-sections, flood marks, landslide surveys. The location of the general levelling reference points should be checked, to position, if possible, the surveyed data and to be able to combine them easily with other geographical data (DTM model, Lidar data …). Some examples of surveyed sections are shown on the next page (Figure 10).	

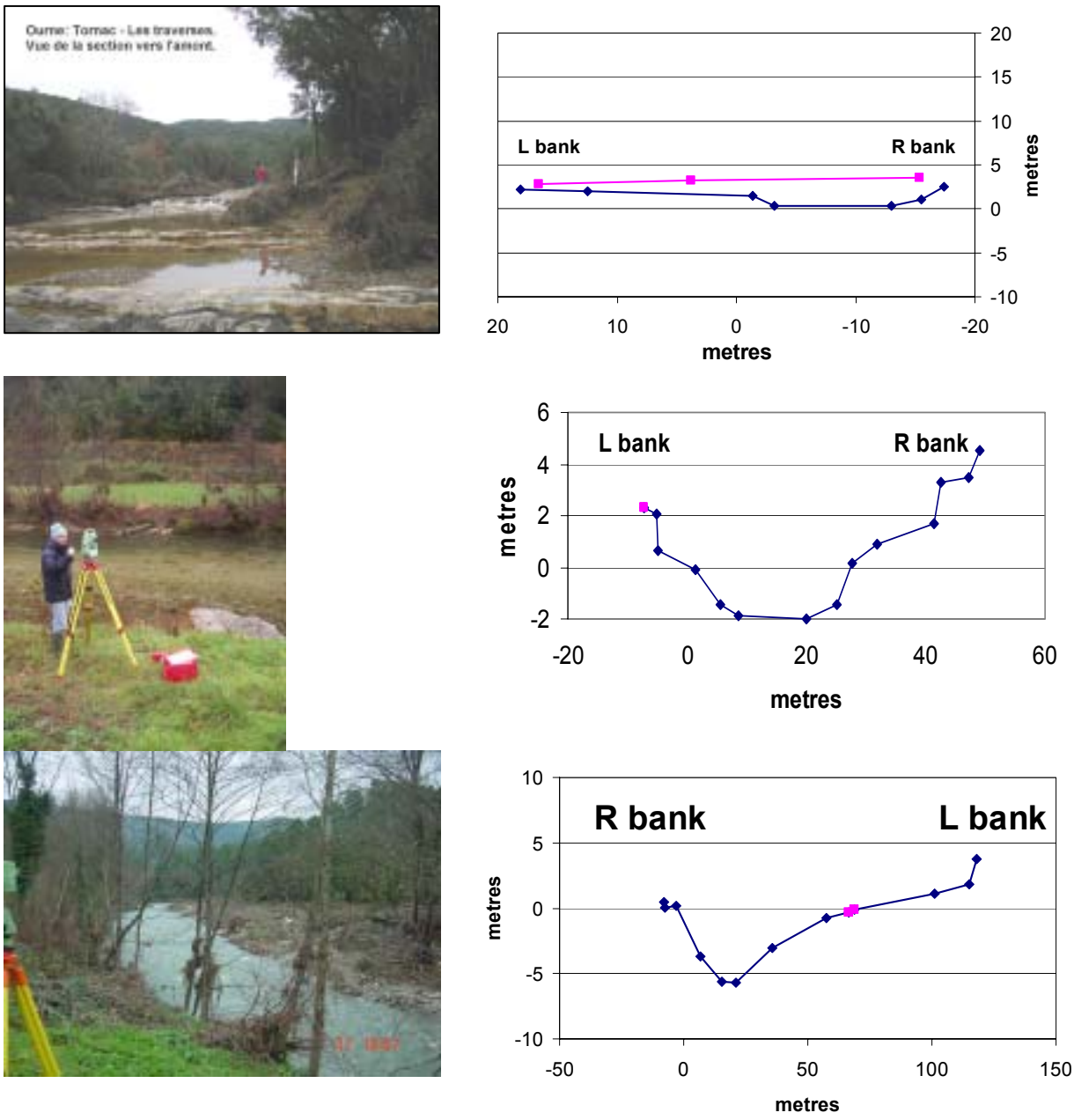


Figure 10: Examples of surveyed cross-sections (blue points) and flood marks (purple points), pictures of the river reaches and position of the digital laser theodolithe.

3. Indirect discharge estimation methods

3.1 About discharge estimates accuracy

As mentioned earlier, the majority of the upstream catchments where the most severe floods occur are often not gauged and the existence of a river stage measuring station brings no guaranty of obtaining accurate discharge values. Generally, no direct current-metre measurements are available and post flash-flood studies can only be based on peak discharge estimates.

Peak discharge estimation is a key issue of post-flood studies. The most important peak discharge values are gathered to establish flood catalogues (Rodier & Roche, 1984; Unesco, 1976; Pardé, 1958), to build envelope curves (O'Connor & Costa, 2003; Pery, 2000; Costa, 1987b), and serve as reference values for future studies. Moreover, estimations of runoff discharges and volumes are necessary for any further hydrological analysis. Erroneous values will lead to false conclusions.

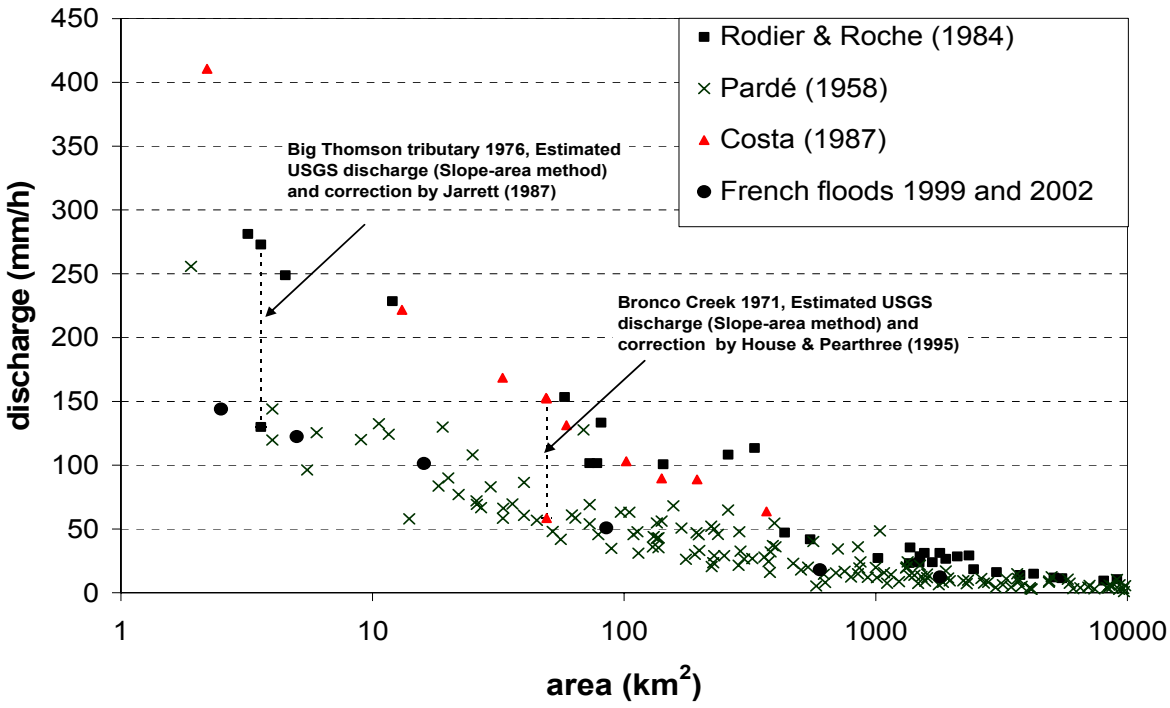


Figure 11: Maximum flood peak discharge values (mm/h) reported in various documents as function of the watershed areas.

Various discharge estimation methods, also sometimes abusively called "indirect discharge measurement methods", have been developed in the past, especially by the U.S. Geological survey (Webb & Jarrett, 2002; Benson & Dalrymple, 1967), to homogenise the procedures used and to share the experience gained by the hydrological community – experience summarized in empirical formulas to compute the Manning-Strickler roughness coefficient (see Chow, 1959). The hope was that the use of these formulas would reduce the necessarily large discharge estimation uncertainties when no current-metre measurements are available. "An estimate is rated as good if the calculated peak is believed to be within 10% of the true peak discharge; fair if the difference could be as much as 15%; and poor when the error could be 25% or greater" (Benson & Dalrymple cited by Costa, 1987). This is a very optimistic point of view, and this rating is probably better adapted to direct measurements of large discharge values. Jarrett (1987) diplomatically stated that: "most users of data assume that flood-

measurements accuracy (i.e. obtained through indirect methods) is within 25%, and many measurements have that accuracy or better. However, some of the flood measurements actually may be in error by as much as 100%."

Figure 11summarizes the maximum peak discharge values reported in various documents as function of the watershed areas. The clear separation of two clouds of points is striking. The catalogues differ by their dates but neither by the corresponding geographical area (Pardé's inventory covers the world), nor by the duration of observation (The catalogue of Costa & Rodier and Roche include very few events that occurred before 1950). Moreover, no real technical breakthrough has been achieved in the field of indirect discharge estimations. The most probable explanation of this discrepancy between the highest estimated discharge values during various periods, is that the same estimation methods were used but with different reference values, especially as far as the Manning-Strickler roughness coefficient or the mean flow velocities are concerned (Jarrett, 1987). Recent re-analysis works led to drastic revisions of the estimated discharges of some of the largest reported flash-floods in the United States (House & Pearthree, 1995; Jarrett, 1987). The initial estimated values were finally reduced by a factor of 2 or 3 (see Figure 11). We are far from the 25% error rate of Benson and Dalrymple (1967).

The main conclusion is that all in all, estimating peak discharges when no direct current-metre measurement is available is, above all, a question of sound engineering judgment and experience. Empirical relations must be used with caution, as guidelines, and their systematic use may have led in the past to systematic over-estimations of the largest flash-floods (Jarrett, 1987). A corollary to this conclusion is that large efforts must also be put on the critics of the estimated values during the field investigation.
Therefore we suggest herein estimating discharges for a minimum of two or three cross-sections for the same river reach to reduce uncertainties. The cross-sectional flow area may vary significantly between sections, and a discharge estimate made for one section may imply an unrealistic velocity value for another section and, consequently, be rejected. Uncertainties can also be reduced by testing the upstream-downstream coherence of the estimates and their coherence with the rainfall data. More accurate discharge or velocity estimates - critical depth estimates, super-elevation in bends, velocity estimated from films - are sometimes available to adjust the Manning roughness coefficients. Solid transport, erosion, deposition clues may also be used to validate the estimated velocity values.
However, the accuracy of the peak discharge estimates remains highly dependent on the experience of the expert. In the best case it is probably within 50%.

Various indirect discharge estimation methods are presented hereafter.

3.2 About high water marks

The first pitfall in peak discharge estimation lies in the identification of the mean high water levels in river cross-sections where no measurement is available. Much evidence of levels reached by the water can be found after an exceptional event. Vegetation fragments, silt or fuel marks on walls can be preserved over several decades (Figure 12). But, they are not necessary representative of the mean water level. They can result from projections due to the presence of an obstacle or be settled on vegetation temporarily bent by the flow. High water marks in still water areas, such as inside houses, are preferable. But, here again, there is a need for caution. Due to the short duration of flash-floods, the water level inside closed houses may never reach the maximum level of the water outside. However, it would be difficult for the margin of error on the high water levels estimated on the basis of marks left by the flood to be less than 10-20 centimetres. It can be concluded that it is actually impossible to identify water surface slopes much lower than 1%. This of course limits the accuracy of the discharge estimation methods based on the longitudinal water surface or energy line profile (see what follows).



Fragments on a wire fencing (note that the flood mark level is not horizontal)



Fragments in a tree



Silt marks on a wall outside a building



Silt marks on a wall inside a building



humidity marks on a wall (possible influence of the capillary rise)

Figure 12: Some examples of flood marks

3.3 Some possible peak discharge estimation methods

3.3.1 One-dimensional steady state hydraulic theory

Two-dimensional hydraulic models have been used in some recent studies for estimating peak discharges (Denlinger et al., 2001), but most of the hydraulic post-flood discharge estimations are based on one-dimensional models.

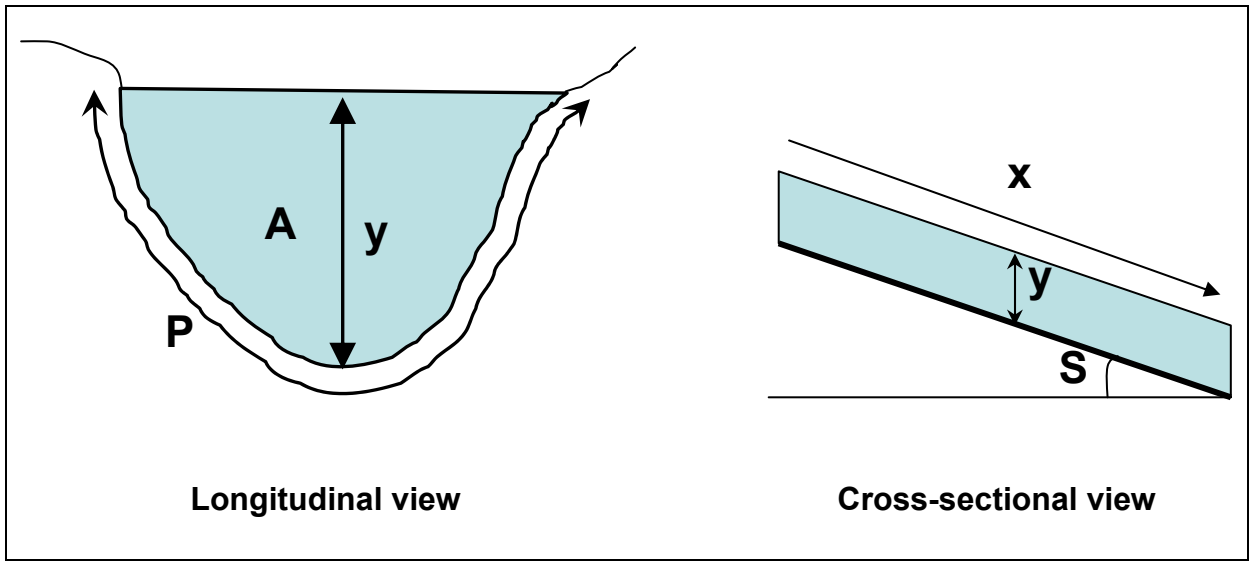


Figure 13: Cross-sectional and longitudinal views of a river channel

In steady state conditions, when the derivatives with time are equal to zero, the Barré de Saint Venant system of equations is reduced to the well-known Bernoulli equation:

$$\frac{d}{dx}\left(\frac{Q^2}{2gA^2}+y\right)=S-S_f=\frac{dHs}{dx}$$

equation 3-1

Where x is the longitudinal coordinate, Q is the discharge (m^3/s), A is the wetted cross-sectional area (m^2), y is the flow depth (m), g is the gravitational acceleration (m/s^2), S is the river bed longitudinal slope (m/m) and S_f is the friction slope (see Figure 13). The quantity Hs is called the specific flow head. We will call the quantity V=Q/A the mean flow velocity.

Empirical formulas have been proposed to relate the friction slope S_f to the characteristics of the flow and of the channel cross-section. The Manning-Strickler formula is the most popular one:

$$Q=KARh^{2/3}S_f^{1/2}$$

equation 3-2

Where Rh is the hydraulic radius (Rh=A/P, with P the wetted perimeter see Figure 13), and K known as the Manning-Strickler roughness coefficient depending on the river cross-section characteristics which generally takes its values between 0 and 100. The parameter n=1/K is also often used in the technical and scientific literature.

These two equations control the shape of the longitudinal water surface profile in river reaches. The simplifications done (one-dimensional flow hypothesis, synthesis of the friction effects into the empirical Manning-Strickler equation) have proven to lead to very satisfactory results in most of the situations.

Two particular values of the water depth y can be defined on the basis of these equations. The normal water depth y_n when $S_f = S$. y_n is solution of the following equation:

$$Q = KA(y)Rh(y)^{2/3}S^{1/2}$$

equation 3-3

In a uniform Channel, with constant cross-section shape and roughness, y_n corresponds to an equilibrium value. This value is observed in cross-sections located in a relatively straight and uniform reaches and far enough upstream and downstream from hydraulic singularities (bends, dams, bridges).

The second particular value is the critical water depth y_c. It is the value for which the derivative of the specific flow head Hs with y is equal to zero. This means that y_c is solution of the equation:

$$F(y) = \frac{Q^2}{gA(y)^3}\frac{dA(y)}{dy} = 1$$

equation 3-4

The left hand term of this equation is the well-known Froude number F(y). Note that y_c does not depend on the roughness coefficient which is one of the main sources of uncertainties in indirect discharge estimations. It is therefore appealing to try to find cross-sections in river reaches where the critical state may have been reached during the peak of the flood. However, the critical state is unstable (Chow, 1959). Apart from the specific case of a critical flow regime ($y_c = y_n$), the critical depth can only be observed in particular cross-sections: contraction in the channel cross-section, unsubmerged flow over a dam across the river bed.

3.3.2 Slope-conveyance method

It is a simple method which has been used, in combination with the rainfall-runoff checking method in the recent post-flood studies in France. The main idea is to select a river cross-section where the uniform flow conditions may have been reached during the peak of the flood. This means that the cross-section must be located in straight and uniform river reach, sufficiently far upstream and downstream hydraulic singularities. What sufficiently means depends on the local river bed slope. Typically a super-elevation of 1 metre, due to the presence of an obstacle in a river bed, will have an influence on the upstream water surface profile over about 100 metres if the river bed slope is 1% in sub-critical flow conditions. It will influence the water surface profile over about 1000 metres if the river bed slope is equal to 0.1%.

About the uniform state assumption and the use of the Manning-Strickler formula

When the Manning-Strickler equation is applied to compute the discharge, the friction slope being equal to the bed river slope (uniform flow assumption), the accuracy of the discharge estimate depends partly on the appropriate choice of the cross-section.
It is possible to refine the method by measuring the local water surface longitudinal slope using flood marks located downstream and upstream the considered cross-section. Due to the uncertainties in water surface elevation estimated through flood marks, this slope should also be considered as uncertain (see discussion in section 3.2 and Figure 14).

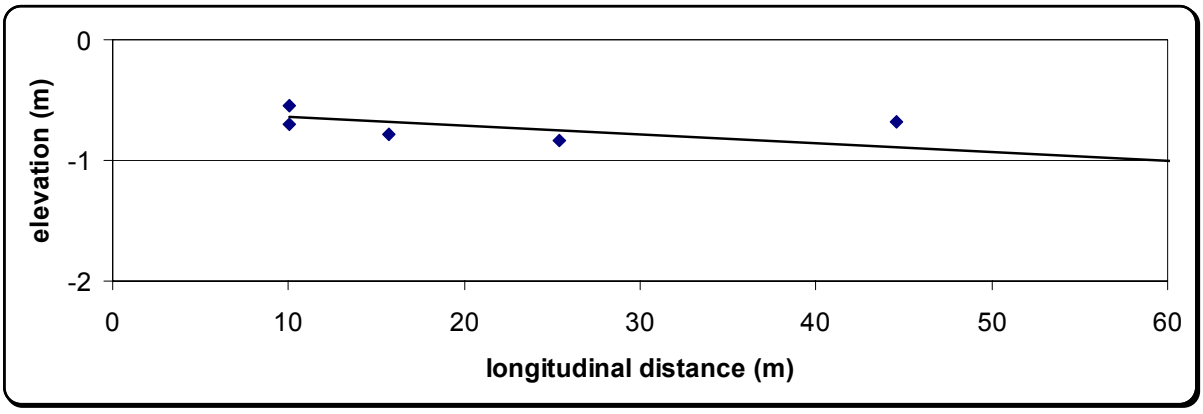


Figure 14: Example of a longitudinal water surface profile (continuous line) established on the basis of high water marks (points) in a river reach.

It can be nevertheless compared to the river bed slope. If a significant difference appears, it is possible to compute a second estimate of the discharge using the Manning-Strickler formula (equation 3-2) and a value S_f=dz/dx, z being the water surface elevation. S_f=S-dy/dx.

Recalling the Bernoulli equation 3-1, we can write:

$$S_f = S - \frac{dy}{dx} + F(y)\frac{dy}{dx}$$

equation 3-5

The proposed alternative computation is only valid if the Froude number F(y)<1 (subcritical flow). In the case of a supercritical flow (F(y)>1), the computation error will be increased if the energy slope is approximated by the water surface slope rather than by the river bed slope. Two cases can be identified:

- If F(y)<1 and dy/dx>0, then the hypothesis S_f=S will lead to overestimate the discharge and the hypothesis S_f=S-dy/dx will lead to an underestimation.
- If F(y)<1 and dy/dx<0, then the hypothesis S_f=S will lead to underestimate the discharge and the hypothesis S_f=S-dy/dx will lead to an overestimation.

The "true" discharge value lies between the two estimations. A further refinement consists in estimating the Froude number on the basis of a first guess taking S_f=S and then computing S_f with equation 3-5 and making the procedure converge.

In any case, one should keep in mind that the slope dz/dx can generally not be accurately estimated with flood marks, and the proposed refinements will not necessary reduce the discharge estimation error due to the other sources of uncertainties. The best solution consists in checking the estimated water surface slope and the bed river slope are close to one another. If it is not the case, we would advise to select another cross-section for the survey. Note that an error in the friction slope has a moderate impact on the discharge estimation since the square root of the slope is used in the Manning-Strickler formula (equation 3-2).

Handling composite cross-section shapes

In many cases the flow is not confined in the main river channel during extreme events, and the flow characteristics (roughness coefficients, mean velocities) may differ significantly between the main channel and the flood plain areas. Considering a mean roughness coefficient and velocity for such

cross-sections can lead to large errors. The first factor of error is linked to the computation of the hydraulic radius. The wetted perimeter increases much more rapidly than the wetted area when the flow extends over the banks. The hydraulic radius may drastically be reduced when overflow begins, leading to an absurd result if the Manning-Strickler formula is used to compute the discharge on the whole section: the resulting discharge in the section including overbank flow is lower than the main channel computed bankfull discharge.

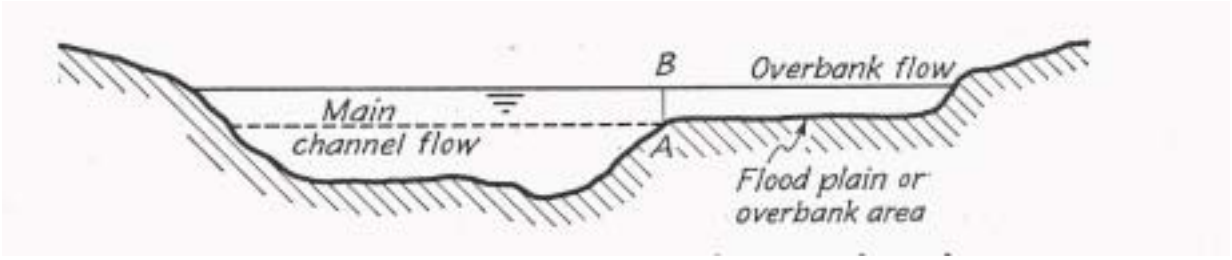


Figure 15: Cross-section of a natural stream channel (taken from Albertson & Simons, 1964)

In such a situation, the section has to be subdivided (Chow, 1959) into a main channel area and a right and left overbank flow area, and the discharge calculated separately for each of the sub-areas. What is then the status of the segment AB on Figure 15, and should it be included or not in the wetted perimeter of each sub-area? There is certainly a loss of energy along the frontier between the main channel and the flood plains due to the velocity gradients, but it is certainly lower than the friction losses. If the segment AB is included the loss of energy in the Manning-Strickler equation will certainly be overestimated, but it will be underestimated if not. From a practical point of view, the inclusion of the frontiers of the areas in the computation of wetted perimeters has generally a limited impact on the evaluated discharge values. Moreover, it must be considered that the Manning-Strickler formula is empirical and that the main source of uncertainty comes from the choice of the roughness coefficient values.

Choice of the roughness coefficient values

The choice of an appropriate roughness coefficient is the last but not least pitfall. Benson & Dalrymple (1967) proposed to use tabulated values and empirical equations like the ones proposed by Chow (1959). More recently, Jarrett (1990) argued that the tabulated roughness coefficient values had been determined in cases of moderate floods and low-gradient streams. As the velocity increases due to an increase of the discharge or the river bed slope, the turbulence increases resulting in increased energy loss. The Manning-Strickler equation may not completely account for these evolutions. The use of tabulated roughness values may therefore have led to a systematic overestimation of the peak discharges of flash-floods in steep streams in the United States according to Jarrett (1987). This is illustrated by some examples in the paper of Jarrett (see Table 3), where it is shown that the standard application of the slope-area method (similar to the slope-conveyance method, see next part) leads to results which are much higher than the results of other estimation method.

Location	Drainage area (km^2)	Slope % (m/m)	Estimated peak discharge (m^3/s)		
			Slope-area method	Critical-depth method	Rainfall-runoff method
Big Thomson River tributary, Colorado (1976)	3.5	7.7	246	133	153
Dark Gulch at Glen Comfort, Colorado	2.6	12.5	204	96	110

Table 3: Comparison of peak discharge estimates using various methods (Jarrett, 1987)

This led Jarrett to propose an empirical equation to predict the value of the Manning roughness number in steep channels:

$$n = 1/K = 3.17 R_H^{-0.16} S_f^{0.38}$$

Reformulation of Manning-Strickler equation leads to the following expression for the mean flow velocity V (m/s):

$$V = 3.17 R_H^{0.83} S^{0.12}$$

Of course this equation only partly explains the variability of the roughness coefficient values. The riparian vegetation and the presence of irregularities in the river bed are not explicitly taken into account. Additional corrections must be done if bank vegetation, irregular banks or obstructions exist. This relation had a certain success (Rico et al., 2001). In the case of the Aude river 1999 floods (Gaume et al., 2004a), its application would have systematically led to over-estimated peak velocities and discharges (see Table 4). In the two cases, chosen among others as examples and presented in the table, the estimates based on Jarrett's formula are not coherent with other estimates made on the same river or with observations – rainfall-runoff modelling, limited scour in the river beds bearing witness to moderate water velocities - and seem unrealistic: 6 m/s in a natural 30 metres wide channel, and almost 3 m/s in a 10 metres wide channel with a high level of vegetation! The observed high water levels are in these cases more likely the sign of considerable friction losses than of high velocities.

Cross-sectional area (m^2)	Hydraulic radius (m)	River bed slope (m/m)	Mean velocity (m/s) Jarrett	Mean velocity (m/s) estimated
26	1.53	0.02	2.84	1.8
195	4.53	0.007	6.12	3.5

Table 4: Examples of estimated peak mean velocities for two river Verdouble cross-sections (1999 floods, France) using Jarrett' formula and probable values coherent with other estimates

Finally, the use of these empirical formulas can give a false impression of accuracy. There is no miraculous solution. The use of the empirical Manning-Strickler formula the evaluation of a range of possible values for the roughness coefficient require a certain know-how which can not completely be replaced or summarised in formulas. Moreover, even the experts can wrongly evaluate a situation. It is therefore absolutely necessary to cross-compare various estimations done with different methods and/or in different sites, to limit the risks of wrong estimations.

3.3.3 Other methods based on the Manning-Strickler formula

Slope-area method

The slope-area method has been the most commonly used method to calculate discharges indirectly after flood events in the U.S.A. (Webb and Jarrett, 2002; Costa, 1987). Flow is generally non-uniform in natural channels, and because the use of the assumption that the friction slope is equal to the river bed slope (slope-conveyance method) may lead to large errors, a computation method involving two or more successive cross-sections has been developed (Dalrymple and Benson, 1968). This method is based on some simplifications of the steady state equations. If M, the channel conveyance is defined as:

$$M = KAR_h^{2/3}$$

equation 3-6

Then these developments led to the following result if two successive cross-sections 1 and 2 are considered:

$$Q = \sqrt{\frac{(z_1 - z_2)}{(L/M_1M_2) - (1-k)(1/A_1^2 - 1/A_2^2)/2g}}$$

equation 3-7

where z_1 and z_2 are the water surface elevations in the cross-sections (m), L is the length of the river reach between the two sections, and k is an energy-loss coefficient conventionally equal to 0.5 for expanding reaches and 0.0 for contracting reaches. But other values may be found in the literature (Webb and Jarrett, 2002).

Similar expansions have been proposed for three or more channel cross-sections as well as for the case of subdivided cross-sections.

The slope-area method is relatively sophisticated, includes parameters, and is not easy to use. As the slope-conveyance method, it is based on some assumptions. The flow profile should be continuous between the two selected cross-sections and not interrupted by a hydraulic jump for instance. This of course is difficult to check when the two sections are not close to one another.

Past works have shown that this method could lead to large errors (House and Pearthree, 1995; Jarrett, 1987). Complexity does clearly not guaranty accuracy. We would therefore advise to use the simpler slope-conveyance method with the previously exposed limits.

Hydraulic simulation method

The "ideal" discharge estimation method consists in simulating with a hydraulic model the flow profile reconstructed on the basis of the high water marks in the selected river reach. A trial and error approach helps to determine the discharge value which leads to the flow profile closest to the observed one. One-dimensional models are generally used (Naulet, 2002) but two-dimensional hydraulic models have also been used in some recent studies for estimating peak discharges (Denlinger et al., 2001).
It is nevertheless a time consuming approach. Apart from the model development and simulation time, a large amount of flood marks must be collected during the field survey. It can therefore be only applied on a limited number of cross-sections. Moreover, the backwater propagation distance depends on the river bed slope. If the slope is greater than 1%, it will be difficult to identify water surface profiles over short distances. If the slope is much lower, it will than be difficult to represent accurately the shape of the river bed and the hydraulic singularities which have an influence on the flow profile in the model. In any case, the accuracy of the method is limited by the accuracy of the water surface elevation estimated on the basis of the flood marks, the uncertainties concerning the values of roughness coefficients, the assumption of a steady state which is doubtful for flash-floods if the considered river reach is too long…
Finally, it is necessary to define a downstream boundary condition for the model in the sub-critical flow case and an upstream boundary in the super-critical flow case. The accuracy of the discharge estimation will also depend on the relevance of this condition.
To summarise, the hydraulic simulation method is certainly the best of the three previously presented methods if it is used with judgment. But it can not be systematically applied.

Culverts and Bridges

Culverts and bridges are particular cross-sections with a relatively simple shape when they have not been over-flooded: no vegetation having an effect on the roughness coefficient, no bank flow. In case of pressure flow under the bridge or culvert, the flow cross-section is even perfectly known. All the

uncertainty in the discharge estimation relies in the evaluation of the mean flow velocity under the bridge or more precisely upstream or downstream it.

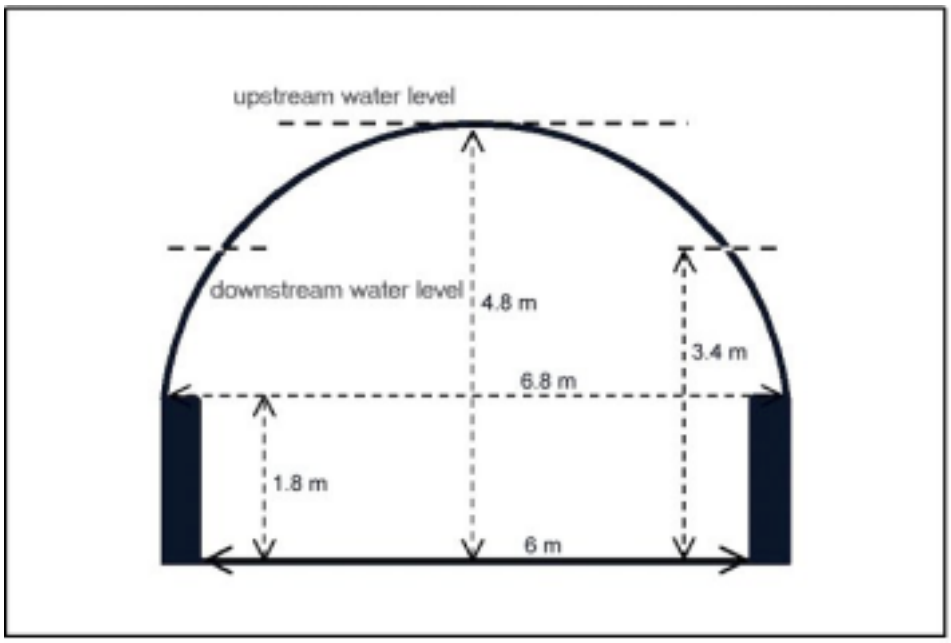


Figure 16: Example of a surveyed bridge cross-section after the 2002 floods in the Gard region (France)

Empirical hydraulic head loss relations have been proposed for flow through openings (Chow, 1959; Lencastre, 1999) which can be used to evaluate the mean flow velocities and discharge especially if the flood marks indicate a clear difference of the water level upstream and downstream the bridge and culvert (see Figure 16).

$$V_2 = C\sqrt{2g(y_1 - y_2) + V_1^2}$$

equation 8

In equation 8, V_1 and y_1 are the upstream mean water velocity and elevation respectively and V_2 and y_2 the downstream velocity and elevation, g is the gravitational acceleration, and C a head loss parameter which depends on the shape of the culvert (generally equal to 0.7 to 0.9).

These formulas are parametric, sensitive to uncertainties in the estimations of upstream and downstream water levels on the basis of the flood marks, and can only lead, as the application of the Manning formula, to ranges of possible values for the discharge.

As an example, the application of equation 8 on the example shown in Figure 16 leads to downstream velocity values comprised between 4.5 and 5.5 m/s depending on the chosen C and V_1 values. By the way, due to the large upstream-downstream water level difference, the result is not very sensitive to the value of the upstream mean velocity. This leads to a discharge value ranging from 90 to 110 m^3/s, for a watershed area of 8 km^2.

The experience has shown that mean water flow velocities during extreme floods under bridges and culverts generally range from 3 to 6 m/s. Large flow velocities, typically larger than 4 m/s, induce erosion processes downstream the bridge as illustrated on Figure 17. The presence or absence of such erosion evidences can be used to confirm or invalidate estimated flow velocities.



Figure 17: Downstream view of the Ners railway bridge on the Gard river after the 2002 flood. Erosion of the downstream edge of the left bank levee of the bridge, and erosion of the river bed material downstream the bridge that has created a few metres deep pools, signs of high flow velocities (estimated flow velocity: 5 m/s).

3.3.4 "Non-parametric methods"

Super-elevation in bends

As shown previously the evaluation of the roughness coefficient is one of the steps limiting the accuracy of peak discharge estimations. In some specific cases, the discharge depends on the water surface elevation, the shape of the cross-section but is independent on the channel roughness. It is particularly the case in cross-sections were the water depth is equal to the critical depth (see equation 3-4) or in river bends. The computation of the discharge on the basis of the critical depth equation is straightforward. The main difficulty relies in finding cross-sections in which the flow regime may have been critical during the peak of the flood: contraction in the channel cross-section, unsubmerged flow over a dam across the river bed. But even there, the flow regime is not necessarily critical. The author have never found during the several post flash-flood investigations they have conducted, cross-sections where the critical depth equation could obviously be applied. During the Aude river survey (1999, France), the critical depth method was only applied once out of over one hundred discharge estimates (case of a flow over a dam followed by a ten metre waterfall). It led to a severe underestimation of the flood peak discharge as shown by the estimations made in other cross-sections and rainfall-runoff simulations.

The estimation of discharges based on observed water super-elevation in bends appears to be a much more promising method. Some formulas have been proposed to evaluate the difference in water surface elevation observed between the inner and outer banks of a bend or curve. If this super-elevation is attributed to the centrifugal action only and assuming that the forward velocities are homogeneous in any cross-section of a bend, than it can be shown that:

$$\Delta h = \frac{V^2 b}{g r_c}$$

equation 3-9

Where Δh is the super-elevation (m), V is the mean forward velocity (m/s), b is the channel width (m) and r_c is the radius of curvature of the bend (m).

Flow velocity (m/s)	Bend super-elevation (m)
1	0,10
2	0,41
3	0,92
4	1,63
5	2,55
6	3,67
7	4,99
8	6,52
9	8,26

Table 5: Super-elevation in bends for various mean flow velocities, computed with equation 3-9 with $r_c/b=1$.

This formula is simple and is often used, especially in debris flow studies (Meunier, 1991). It is considered to lead to fair results. Its application is nevertheless not straightforward. As shown in Figure 18, the elevations are not homogeneous on the outer and inner banks of a bend. The estimated super-elevation depends on the considered cross-section. But the fact that the velocities are linked to the square root of the super-elevation limits the impact of the uncertainties in its estimation. Depending on the position in the bend, the super-elevation estimation lies between 10 and 5 on Figure 18, which means an estimated mean velocity value computed with equation 3-9 between 0.7 and 1 m/s to be compared to 0.8 m/s (actual value). In supercritical flow conditions, cross-waves appear in bends which will also influence the levels reached by the water on the inner and outer banks (Chow, 1959). These waves can increase the super-elevation by a factor of two (Lencastre, 1999). Of course, the last difficulty in applying the bend super-elevation method lies in the estimation of the radius of curvature of river bends which never are pure circular segments. It is finally important to repeat here that water surface elevation estimation based on flood marks have generally a level of uncertainty between 10 and 50 centimetres depending on the type of available flood marks. This limits the accuracy of the super-elevation estimation.

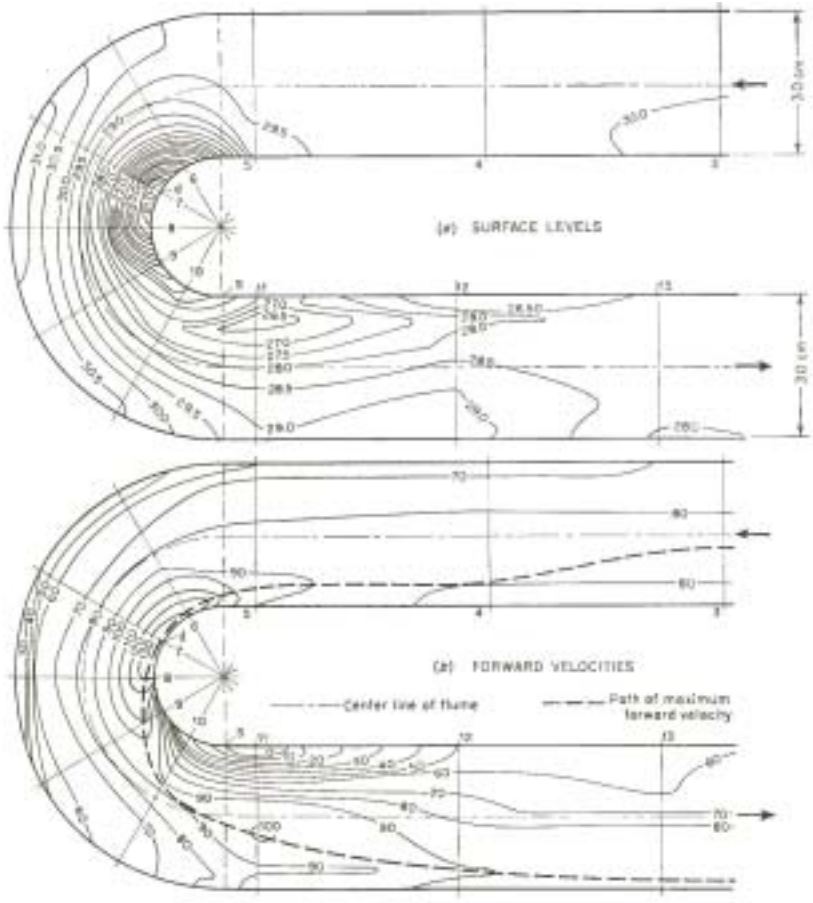


Figure 18: Contour lines of equal surface levels and forward velocities in flow around a 180° bend (After Shukry, cited by Chow, 1959). Surface levels measured in cm and velocities in cm/sec.

Nevertheless, compared to the other presented velocity estimation methods, the bend super-elevation method, despite the remaining cited sources of uncertainties and errors, is from far the most robust one. It can in particular help to delimitate the possible range of mean velocity values, especially when the flow velocities are high (i.e. greater than 3 m/s, see Table 5). The search for bends with evidence of super-elevation will be particularly interesting in river reaches and post-flood studies were other indirect estimation methods lead to high mean velocities, to validate (or invalidate) these values.

Super-elevation in front of obstacles

Another possible non-parametric estimation method is based on the super-elevation of the water surface in front of obstacles located in the flow. This super-elevation can hardly be obtained after the flood but may sometimes be detected on films or pictures taken by witnesses as in Figure 19. The simplified proposed approach is based on two hypothesises: (a) the specific hydraulic head is homogeneous in the vicinity of the obstacle and (b) the water velocity just in front of the obstacle is equal to zero. This means that

$$y_1 + \frac{V_1^2}{2g} = y_2 \Rightarrow V_1 = \sqrt{2g(y_2 - y_1)}$$

equation 3-10

With y_1 and V_1 the water depth and mean velocity in the area surrounding the obstacle, and y_2 the water depth in front of the obstacle.



Figure 19: Example of a super-elevation in front of an obstacle

equation 3-10 applied to the example presented in Figure 19 leads to a mean velocity value comprised between 2.5 and 2.8 m/s, which is in accordance with the estimations based and the slope conveyance method in nearby sections and what could be roughly estimated on a video taken by a witness. In any case the picture clearly shows that the velocity is significant: a velocity of 1 m/s would only have created a super-elevation of a few centimetres. It is also lower than 4 m/s which would have induced a super-elevation of 80 centimetres. It is clearly not the case on this picture.

Water surface velocity evaluation on films

Video cameras are relatively common family equipments, and films of floods are now frequently taken by eyewitnesses. Recent works have demonstrated the possibility to use image tracking methods to assess water surface velocities and hence discharges (Fourquet, 2003). Nevertheless, these works have been conducted on well surveyed cross-sections with a control on the viewpoint of the camera and on the distortion due to the perspective. Their application on films taken by eyewitnesses requires a preparation survey of the filmed river reach and the identification of the viewpoints of the camera. To our knowledge, no such work has been conducted for the moment. Nevertheless, films can be used at least for a qualitative assessment of the flow velocities: i.e. to assess the range of possible values.

3.3.5 Rainfall-runoff checking method

A coherency test of the estimated peak discharges and of the measured rainfall rates available on the watersheds upstream the considered river cross-sections is the first important step of an hydrological valuation of the collected data. It can at least reveal peak discharge over-estimations: except if an important dam breach occurred during the flood event, the comparison of peak discharges and rainfall rates should not lead to runoff rates significantly greater than 1. Two methods can be used to test this coherency: (i) application of the so-called "rational method" and comparison of the unit peak discharge (in mm/h) and the event maximum runoff rate over a time-step close to the estimated time of

concentration of the considered watershed, (ii) rainfall-runoff simulation with a constant runoff rate value taken equal to 1 in the model. These to methods are illustrated in Table 6 and Figure 20.

Table 6 presents some data produced by the U.S. Geological survey concerning flash-floods that occurred in the United States (taken from Costa, 1987b). In each case, the estimated discharges appear dubious when compared to the reported measured rainfall intensity. In the case of the Bronco creek, with the hypothesis of a mean flow velocity of 3 to 5 m/s (10 to 18 km/h) on the watershed and a maximum transfer distance on the watershed of 10 km, the time of concentration of the watershed is comprised between 30 minutes and 1 hour. This is probably the lower bound for the time of concentration considering the extreme mean velocity and distance values chosen. The estimated discharge would require a mean spatial rainfall intensity over the 49 km^2 of the watershed, almost twice as high as the maximum reported point rainfall intensity over 0.75 hours. This is of course not impossible, but highly unlikely, particularly if we consider that the reports on this flood do not mention that the relatively high rainfall rate measured at the rain gauge may have been significantly lower than the rainfall intensities over the watershed. This coherency test indicates a possible over-estimation of the peak discharge which is in accordance with the conclusions of House and Pearthree (1995) on this discharge estimation (see Figure 11).

Location	**date**	**Drainage area (km2)**	**Discharge (m^3/s)**	**Unit discharge (mm/h)**	**Measured rainfall intensity**
Humbolt river tributary near Rye Patch (Nevada)	31.05.1973	2.2	251	410	127 mm/1h
Meyers canyon near Mitchell (Oregon)	26.7.1965	32.9	1540	169	102 mm/2h
Bronco creek near Wikieup (Arizona)	18.01.1971	49.2	2080	152	89mm/0.75 h
Jimmy camp creek near Fontain (Colorado)	17.06.1965	141	3510	90	203 mm/ 6 h

Table 6: Some examples of maximum peak discharges estimated by the U.S. Geological Survey (Costa, 1987b). Comparison of the unit discharge and of the rainfall intensities.

Likewise, the same procedure leads to express some doubts concerning the three other peak discharge estimates. In each case, the estimated peak discharges require that the majority of the measured rainfall amount has fallen homogeneously in space over the watersheds during a reduced duration corresponding to the time of concentration of the watershed: 70 mm over 10 minutes for the Humbolt river tributary, 85 mm over 30 minutes for the Meyers canyon and 90 over 1 hour for the Jimmy camp creek. Such rainfall rates are really exceptional, especially for the conterminous United States (Costa, 1987) and particularly if they represent spatial mean rainfall rates rather than point rainfall intensities. Again, none of the reports on these floods indicate that the considered watersheds received such concentrated in time and space rainfall amounts.

A complete rainfall-runoff simulation can be conducted to validate the estimated discharges when rainfall measurements, sufficiently accurate in time and space, are available. Considering the density of rain gauge networks, this implies that validated radar data exist for flash-floods occurring on small watersheds. Two examples of inter-comparisons between simulated and estimated discharges are presented in Figure 20. The semi-distributed rainfall-runoff model used, CINECAR, is described in appendix I. Calibrated radar rainfall rates have been used in both cases as input of the CINECAR model. The mean areal 5-minute rainfall rates over each watershed are shown on the top of the figures.

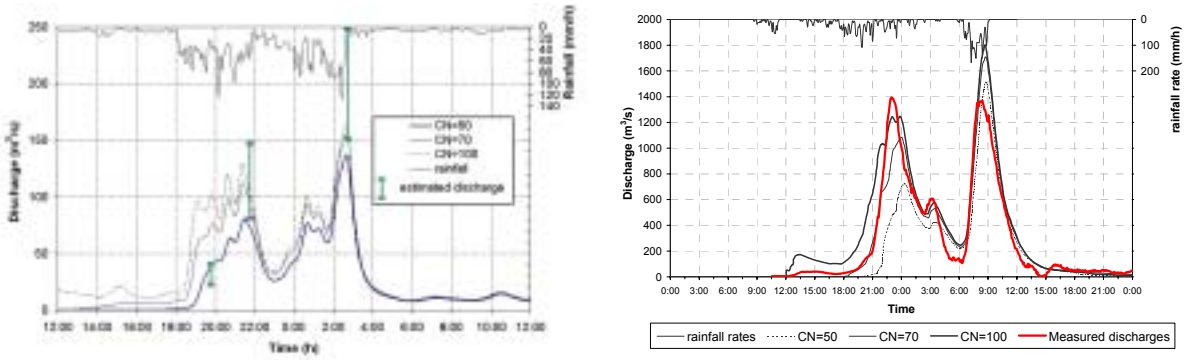


Figure 20: Comparison of estimated and computed discharges: (a) 10 km² Tournissan Watershed (Aude 1999 floods), uncertainty ranges for the estimated discharges (green bars), (b) 90 km² Crieulon Watershed (Gard 2002 flood), discharges estimated on the basis of water depth measurements in a flood control dam spillway (red curve).

In one case, the flood hydrograph could be reconstructed using water level measurements in a flood control dam spillway (Figure 20.b). In the other, the water levels and times given by witnesses were used to evaluate some discharge values (green bars in Figure 20.a). Horizontal and vertical bars indicate the estimated uncertainty ranges concerning the discharge values and the times indicated by the witnesses. CN, the "curve number" of the soil conservation service model, is the main parameter of the CINECAR model. It represents the runoff ability of the watershed. CN=100 means 100% runoff over the whole flood event: the watershed behaves like an impervious area.
The two examples show that the estimated hydrographs are relatively well reproduced by the model. This is a sign that the transfer function of the rainfall-runoff model has been well calibrated, but validates also indirectly the time sequence of the rainfall rates estimated on the basis of the radar data. The simulated and estimated peak discharges appear coherent. They indicate that the runoff coefficients were most probably close to 100 % during the paroxysm of the floods, which is not a surprise considering the rainfall amounts received by both watersheds: more than 400 millimetres over 24 hours.
Of course, this rainfall-runoff test does not certify the estimated discharge values, especially if the uncertainty of the estimated rainfall rates, but in both examples it did at least not reveal any obvious discharge over-estimation. The very high estimated discharge values (15 to 20 $m^3/s/km^2$) are coherent with the estimated rainfall rates.

3.3.6 Conclusions on peak discharge estimation methods

Estimating peak discharges without direct current-metre measurements is, above all, a question of sound engineering judgment. As shown herein, empirical relations must be used with caution, as guidelines. The various available estimation methods are necessarily inaccurate. To reduce the uncertainties and avoid significant errors (i.e. over-estimations), it is necessary to seek for various sources of information to enable a cross-checking: select more than one cross-section for each river reach with significantly different cross-sectional shapes and areas, test the upstream-downstream coherency of the estimated discharges on a watershed, valuate films and clues on photos or in bended reaches, solid transport and erosion indicators , test the rainfall-runoff coherency.

3.4 Witnesses and time sequence of the floods

Information concerning the chronology of a flood is essential for analysing the rainfall-runoff dynamics. An immediate discharge decrease once the rain has stopped indicates that the flood is mainly due to fast-responding processes (i.e. surface runoff, or sub-surface water transfer over a very small distance, with a short residence time in the soil). On the other hand, a clear attenuation of fluctuations of the rainfall intensity, and a slow reduction of the discharge following the storm event reveals the predominant contribution made by the soil and the ground water. Similarly, the response delay time to rainfall provides information concerning infiltration and water storage capacities at the watershed scale, and the hydrological response to intense showers can help to distinguish between purely hortonian and infiltration excess runoffs. While the former is triggered by rainfall intensities, the latter is mainly linked to rainfall amounts.

Accounts of eyewitnesses are the only alternative source of information about the time sequence of floods when no direct stage measurements exist. It is often the case for flash-flood studies either because the affected watersheds are ungauged or because the gauges were damaged by the flood.

Accounts of witnesses are sometimes mentioned in post flood survey reports. They have, to our knowledge, seldom been used in the analysis of the events.

3.4.1 Objectives

The main objective of the interviews is to collect data on the time sequence of the flood in areas where no direct river stage measurements are available: typically indications of water levels and the corresponding times (see Figure 9, Figure 21 and Table 7 for some examples). The sought-after information is specific and detailed. The second-hand accounts should therefore be disregarded because their level of accuracy can hardly be tested. Witnesses and inhabitants can also provide useful information on the rainfall (private rain gauge measurements), the runoff process (observation of surface runoff, origin of this runoff, groundwater levels observed in wells, inundation of cellars, soil saturation observed by farmers…) and the local flow characteristics (type of flow – i.e. water flood, hyper-concentrated or debris flow - approximated surface water flow velocities, blockages formed during the flood and their possible breaking up, existence flood waves, time and influence of the collapse of bridges or dykes …). Some films of the flood may have been taken which may help to estimate the surface water velocities and hence the discharges. Ten video records have for instance been collected after the Gard 2002 floods. Image analysis methods for computing automatically water velocities are under development (Fourquet, 2005). They could not be applied on these records an important complementary survey effort is necessary for each filmed cross-section to identify at minimum three fixed reference points for the velocities to be computed. Nevertheless, it was possible to verify that the ranges of velocity values used for the peak discharge estimates in each cross-section were “qualitatively” in accordance with what was observed on the films.

Finally, some elements of comparison with previous major floods can also be gathered. This is important to assess the return period of the flood. Very often, it appeared during the past post-flood investigations that the studied flood was not an isolated phenomenon: i.e. some comparable floods had been observed in the same area during the last fifty to hundred years, time span of the local memory. Such an assessment can motivate additional historical or paleoflood studies to evaluate more precisely the local flood hazard (Payrastre et al., 2005; Bariendos & Cœur, 2004; House et al., 2002).

3.4.2 When to proceed?

Ideally, the interviews should be conducted just after the event. Nevertheless, it is often not possible or even “decent”. People are fully occupied by recovery actions during the weeks after the flood and hydrological studies are clearly not considered, for the moment, as the first priority. This could change if the local authorities and populations were aware of the usefulness of post flood studies and of the importance of the data gathered through interviews. This may happen with the multiplication of post flood investigations and the communication on their results.

In past studies, it has not been possible to begin the interviews of the witnesses before the end of the major crisis, this means some months after the flood (Gaume et al. 2004a; Gaume & Bouvier

2004b). One survey has even been conducted more than one year after the event (Gaume et al. 2003). It is difficult to evaluate the quantity of information missed due to these delay, but it appeared that many of the eyewitnesses – some of whom had been in life-threatening situations – had a clear recollection of the event. A rapid intervention after a flood event is certainly preferable, but it is yet not always possible, and is not absolutely necessary.

3.4.3 Before beginning

As for the choice of the cross-sections for the peak discharge estimates, the areas with potentially flooded buildings can be firstly determined on maps. A journey round the identified areas is than necessary to identify in more detail the flooded zones on the basis of the flood marks. This first contact with the studied area is absolutely necessary. It is the time for understanding how the flooding may have occurred in the various sub-areas (direct river bank overflow, backwater effects, dike breach or overtopping), which will have consequences on the interpretation of the information given by the witnesses. It is also the moment for establishing the work plan: how many interviews will be conducted? Where? What are the strategic areas or buildings, generally the particularly exposed ones, where it is expected that the richest interviews will be done?

Before beginning the interviews it is also recommended to make a visit to the town hall for two main reasons. Everything is rapidly known in small towns and it is preferable that the local authorities are informed of the investigation. Secondly, the local administration can indicate possible witnesses, may have some documents on the flood (pictures, films) or on pervious major flood events and may also know older inhabitants who would be able to give some information about these previous floods.

3.4.4 Contact with the witnesses

People have not been informed of the investigation. The persons conducting the survey must rapidly create a confidence atmosphere with them. The best way is to expose in a few sentences the objectives of the interview: “good morning, we are hydrologists from the Ecole Nationale des Ponts et Chaussées, working for the Ministry of the environment on the flood which occurred. We are collecting data to analyse what happened and are, in particular, looking for information on the time sequence of the flood. Were you here during the event and could you grant us a few minutes to relate the events to us?” The persons conducting the survey should better be members of well-known services, like the U.S. Geological Survey or have an official mandate: all the investigations recently conducted in France were supported by the French ministry of the environment. This facilitates the first contact. They should also be hydrologists, and should have prepared the interviews by compiling the available measured data, and interpreting the clues (flood marks, damages) collected during the journey round the area, this, for two reasons. Firstly the interaction with the witness will be more efficient if the person conducting the interview has some a priori notions of what may have happen and knows the surrounding area. Secondly, the interview is a two ways exchange. The confidence atmosphere will be much easily established if the interviewer has also something to bring to the witness and can explain what is known about the flood (rainfall intensities, affected areas, existing river stage measuring stations…). The interviewer is often the first official direct contact of the witnesses and they are eager to get information and understand. This aspect must not be neglected and explains why such interviews generally take a certain time. The experience has shown that if the preceding rules are followed, very few people refuse to be interviewed.

3.4.5 Conducting the interview

The interviews of witnesses have some similarities with a police inquiry minus the stress. Something has happened, but the detailed time sequence of the flood at the considered location is not known. Neither is the amount of information held by the witness. Therefore, the interviews can only begin with open-ended questions: “tell us what happened”, “how fast did the water level rise?”, “when did the maximum occur?” The objectives of the interview are twofold: collect facts but also evaluate the level of accuracy of the witness’ account. Therefore, any type of information that can confirm the times given by the witnesses should be sought: intervention of the emergency services generally

reported in rough logs, electricity breakdowns, TV or radio programs, church bell, phone call… The question "how can you be so sure of the time" must be recurrent. Note that past experiences have shown that a surprisingly high number of witnesses look at their clocks during the event, probably because this is the only point of reference during these nightmarish events, that often occur at night.
The reliability of the information delivered by the witnesses can also be checked through cross-validations between accounts, comparison between available measured rainfall data and the information provided by witnesses concerning the rainfall dynamics or data on any other previously known aspect of the flood. It is therefore useful to question the witness about these known elements too. Finally, the same questions can also be asked more times in different ways at various moments of the interview to test if the answer does not differ. This is a classical interview technique (Grawitz, 1996).

The reliability and the accuracy of the collected accounts are, of course, extremely variable, but in most cases, the witnesses themselves are quite aware of where to draw the line. One of the most important results of the previous surveys is that it is generally possible to collect accurate information concerning the time sequencing of a flood event by interviewing witnesses. The time of the peak discharge could often be determined to within quarter of an hour and intermediate reference points determined probably with the same level of accuracy as will be illustrated in the coming example.

3.4.6 Example

The Tautavel site in the Verdouble catchment area gave the opportunity of comparing collected information through Interviews (Table 7) with data from a nearby river stage gauging station for the Aude region 1999 floods (Gaume et al., 2004a).

Witness 16 (garage located near the Tautavel river gauging station)	Around 11.00 p.m., the water entered the garage. The water level rose by around 60 cm within 30 minutes. I had just enough time to drive three cars to safety (300 metres away). The water level increased steadily. At midnight or 12.30 a.m., the water level in the garage was about 1 metre. The maximum level reached was 1.8 metres. At 5.00 a.m., there was still a metre of water in the garage. At 6.00 a.m., the water was gone. The water level rose and fell during the flood, I observed at least two peaks.
Witness 18 (farm located 1 km downstream from the Tautavel river gauging station)	We were evacuated at 12.30 a.m. Torrents were flowing down the slopes at that time. The maximum water level was reached. At 11.00 p.m., the hen house (1.8 metres below the courtyard level) began to flood. I went to save the hens. At 11.45 p.m., as I finished, the water was up to my knees. The water level then continued to rise rapidly. I had just enough time to change my clothes before the courtyard flooded.

Table 7: Summary of the accounts of two witnesses in Tautavel

As can be seen on Figure 21, they are fairly similar to one another. This figure also shows that the chronological data concerning the water rise are more accurate than those concerning the water withdrawal. This seems to be a common feature of interviews: the witness have a better recollection of the water rise than of its withdrawal.
As a conclusion, even if highly heterogeneous, the accuracy of the accounts may be quite acceptable, especially when compared with estimates of the rainfall amounts and peak discharges. The main difficulty lies in interpreting the collected information. It is too partial and too limited to enable any reconstruction of flood hydrographs. It has to be confronted with the results of a hydrological model to identify the main features of the rainfall-runoff processes on a watershed.

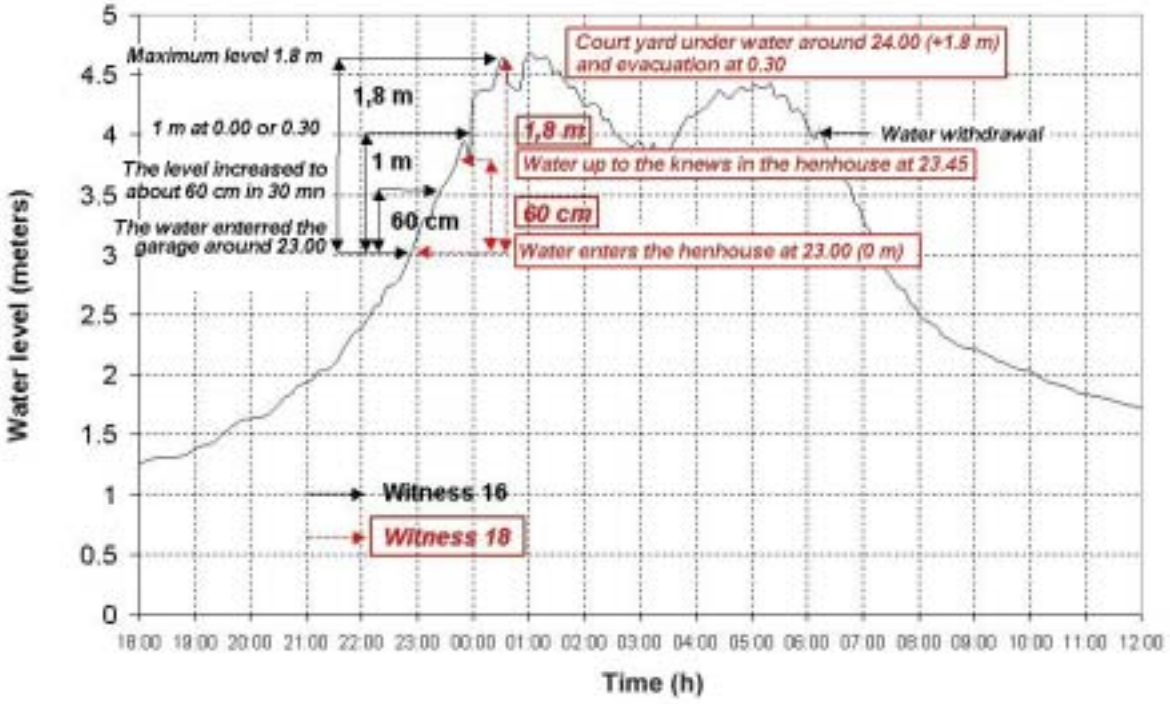


Figure 21: Comparison between measured water levels and the accounts of two eyewitnesses (1999 Verdouble river flood in Tautavel, France)

4. Solid transfer processes

4.1 As indicator of the stream flow characteristics



Figure 22: Galeizon tributary reach estimated discharge (45 to 75 m^3/s) for 3.2 km^2.

Various empirical relations have been proposed to related the characteristics of the flow, especially the shear stress and the characteristics (diameter) of the particles of the river bed which can be displaced by the flow (Meunier, 1991). One of these relations leads after some simplifications to equation 4-1 which relates the mean flow velocity V (m/s), the water depth y (m) and the maximum diameter of the displaced particles d (m).

$$V = 5.8\, y^{1/6} d^{1/3}$$

equation 4-1

In river reaches where clear evidences of solid transport exist, it is possible to use this type of relation to assess the possible range of the flow velocity. Two examples taken from the Gard region 2002 flood investigation serve here as illustration. Of course, this method can lead to under-estimations if the deposits on which the computation is based are not representative of the flood peak, but settled down during the decreasing limb of the flood. In both studied cases (Figure 22 and Figure 23) the river bed material, relatively clean, did obviously move during the flood. The particle diameters are relatively homogeneous, sign that no bigger particles were transported during the flood.

In the first example (tributary of the Galeizon), the water depth during the peak of the flood was approximately equal to 2.5 metres according to the flood marks. An approximate diameter of the particles of 10 centimetres leads to a water velocity of 3.2 m/s in this cross-section, a diameter of 20 centimetres leads to 4 m/s velocity. The estimated velocity was between 2.5 and 4.5 m/s on the basis of the slope-conveyance method. Both estimated ranges are coherent and they are also in accordance with the rainfall-runoff simulations.

In the second example (Auzon), the water depth was equal to 2 to 2.5 meters, and the bed material diameters between 5 to 10 centimetres. These values lead to a mean velocity comprised between 2.4

and 3.2 m/s while the slope-conveyance method led to a mean flow velocity from 1.6 to 2.7 m/s depending on the value of the Manning-Strickler coefficient (15 to 25). This time the solid transfer clues plead for the choice of the upper bound of the estimated velocity and corresponding discharge values, which again is in the range of the possibilities according to the rainfall-runoff simulations.



Figure 23: Auzon river reach estimated discharge (650 to 950 m^3/s) for 63 km^2.

4.2 As the main focus of the post-flood survey

Heavy rainfall events or floods are triggering factors for erosion or solid transfer processes. Flash-floods go therefore often with various mass transfer processes especially in mountainous areas, but it is not always the case. For instance, during the major 1999 floods in the Aude region in France, only one significant landslide occurred and induced two casualties, by the way not in the area which received the most important rainfall amounts. During the 2002 Gard floods in France, no major landslides, mud or debris flows were reported.

Solid transfer processes mainly occur in steep watersheds and mountainous areas. Due to their frequency, their destructive effects and the risk for human lives, they are a major concern in Switzerland, Austria, Italy, the French Alps and the Pyrenees. Depending on the local geological, soil and slope characteristics and on the event, a large variety of processes can be induced: from landslides to mudflows, from hyper-concentrated flows to debris flows, from the release of a new landslide to the reactivation of a previous well known area or the mobilization of river bed or bank material, or even the displacement of the river bed (see Meunier 1991 for a review).

Many previous research and inventory works have been conducted on these processes and they would be worth a separate report. We can for instance refer to the recent Interreg project Dis-Alp for some proposals of information collection methods (Marchi et al., 2005). These methods, mainly aiming at describing the processes (triggering factors like rainfall amounts and intensities, temperatures, localization of the landslides, mudflows, debris flows, geology and soil type, sources of the transferred materials, estimation of the transferred volumes, flow energy evaluation for debris flows…) and complementary to the methods proposed in this document will not been described in this document. Nevertheless, it must be clear for the reader that a post-flood investigation can not only be limited to a purely hydrological analysis in many areas prone to solid transfer processes.

5. Storage of the collected data and analysis

5.1 Data storage

The best tool to store the data (summarized in forms) collected during a post flash-flood investigation is a Geographical Information System. If a GPS has been used during the field study and the coordinates of the surveyed locations are known, the integration of the data into a GIS is straightforward.

We repeat here what has already been said while presenting examples of forms: the rough data (cross-sections, flood mark elevations, summary of interviews) should be stored and accessible as well as the processed ones (estimated discharges, times given by the witnesses).

Figure 24 shows the spatial repartition of the 97 surveyed cross-sections and 143 interviews collected after the 2002 floods in the Gard Region. Each spot on the GIS is linked to the file of the form and it can be opened directly by clicking on the spot.

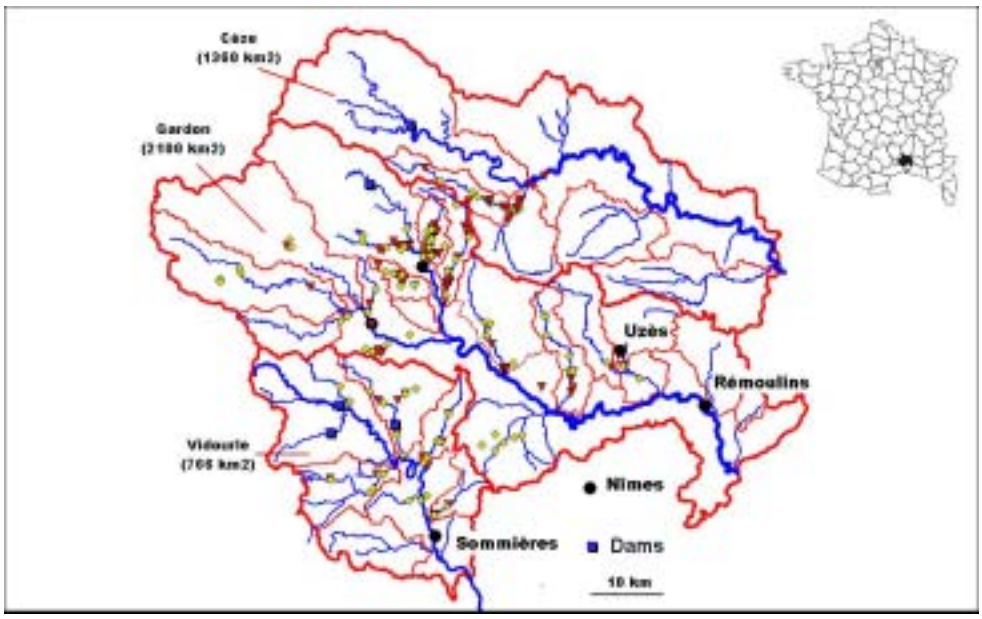


Figure 24: The three main watersheds of the Gard region and location of the surveyed river cross-sections (yellow diamonds) and collected interviews (red triangles) after the 2002 floods.

5.2 Examples of data valuation and analysis

The data analysis illustrations will be mainly based on two flash-flood examples well know by the authors of this report: the 2002 floods in the Gard region and the 1999 floods in the Aude region. These are the floods on which the post-flood investigation methodology presented herein has been developed and tested. To our knowledge, among the published works on flash-floods, these two case studies led to the most detailed rainfall-runoff interpretations. The majority of the data and analysis presented hereafter have already been published (Delrieu et al., 2005, Gaume et al., 2004a, Gaume et al., 2004b, Gaume et al., 2003b).

5.2.1 Spatial and temporal runoff repartition

The estimated peak discharge values can be used to identify the relative contributions of the various sub-areas of watersheds affected by flash-floods. A first example is given in Figure 25 for a 300 km^2 watershed affected by the 1999 storm events in the Aude region in France. In this case, the mapped

peak discharges show clearly a high spatial heterogeneity of the runoff contributions. The estimated peak discharge is about 50 times higher downstream Tuchan than upstream Padern (9 versus 0.2 $m^3/s/km^2$) for similar watershed areas and while the two locations are separated by less than 10 kilometres. This peak discharge repartition is coherent with the observed rainfall amount repartition. The western part of the watershed received about 200 millimetres of rainfall within 24 hours while the north-eastern received more than 450 millimetres. A desk-top application of the "rational method" with estimated time of concentration of the watersheds indicates that the runoff rate has probably remained lower than 10% upstream Padern and must have been close to 100% around Tuchan to explain the estimated discharges. The observed high spatial heterogeneity of the peak discharges can be therefore attributed to the spatial rainfall pattern but also to the rainfall-runoff reaction of the watersheds. We will be able to go further into the analysis of this last point in the next section.



Figure 25: Estimated specific peak discharges on the Verdouble watershed (300 km^2) after the 1999 floods in the Aude region.

A second example of a peak discharge map is given in Figure 26 for the three main river systems of the Gard region affected by the 2002 storm event. The spatial pattern appears here much more homogeneous. Almost all the region received more than 200 millimetres of rainfall within 48 hours and nearly all the watersheds of the surveyed tributaries of the Vidourle and Gard main streams produced more than 5 $m^3/s/km^2$. This spatial extension of the flash-floods explains the extraordinary estimated discharges of the Vidourle (about 3000 m^3/s for 650 km^2) and the Gard river (about 6500 m^3/s for 1800 km^2). Again the spatial pattern of the estimated peak discharges appears relatively well correlated with the pattern of the rainfall amounts.

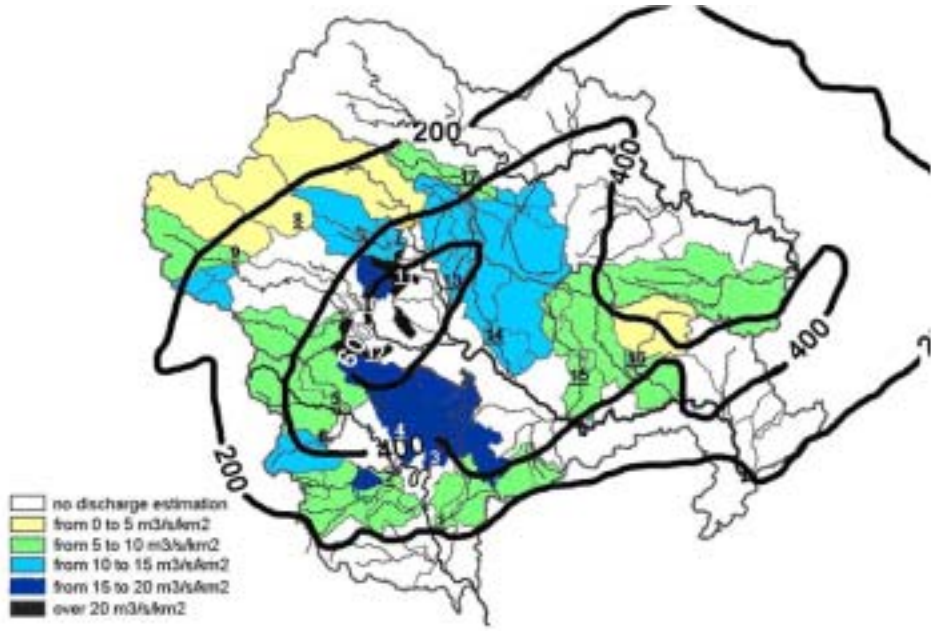


Figure 26: Specific discharges estimated after the 2002 floods in the Gard region and contour lines of the rainfall amounts received on the 8th and 9th of September 2002.

Finally, the time table of the contributions of the various tributaries of the Gard and Vidourle rivers could be built on the basis of the witnesses' data (see Table 8).

n°	Stream name and location	Watershed area (km^2)	Peak discharge (m^3s^{-1})	Peak specific discharge ($m^3\ s^{-1}\ km^{-2}$)	Time of first Peak (UTC)	Time of second peak (UTC)
1	Aigalade, Pondre	42	250	6	1500, 08/09	1100, 09/09
2	Brestalou, Brouzet-les-Quissac	88	500	5.5	2100, 08/09	0800, 09/09
3	Courme, Montmirat	38	650	17	1700, 08/09	1000, 09/09
4	Crieulon, la Rouvière	95	1400	15	2300, 08/09	0800, 09/09
5	Vidourle, Conqueyrac	83	600	7	0000, 09/09	0600, 09/09
6	Rieumassel, Ceyrac	45	500	11	2100, 08/09	0900, 09/09
7	Galeizon	85	1200	14	0600, 09/09	
8	Gardon de Mialet, Saint Etienne	84	400	5	0400, 09/09	
9	Gardon de Saint Jean, Saumane	67	500	7.5	0600, 09/09	
10	Amous, Générargues	21	400	19	0400, 09/09	
11	Alzon, Saint Jean du Pin	15	450	30	0500, 09/09	
12	Ourne, Tornac	11	300	27	2300, 08/09	0600, 09/09
13	Avène, Saint Hilaire de Brethmas	57	600	10.5	0300, 09/09	0500, 09/09
14	Droude, Brignon	99	1200	12	2200, 08/09	0900, 09/09
15	Bourdic, Bourdic	39	500	13	2200, 08/09	1000, 09/09
16	Alzon, Uzès	76	250	3	2200, 08/09	1000, 09/09
17	Auzonnet, Les Mages	46	450	10	0600, 09/08	

Table 8: Time of flood peaks indicated by eyewitnesses. The numbers correspond to the ones appearing in Figure 26.

It confirms the relatively unfavourable progress of the rainfall event especially at its end, in the morning of the 9th of September. A mesoscale convective system producing intense rainfall rates swept across the area from the North-West to the South-East, and this is exactly the orientation of the three main streams. It induced the succession in time of the floods and peak discharges of the

tributaries from upstream to downstream (see Table 8). The real effect of this rainfall configuration on the Gard river flood will be studied hereafter.

5.2.2 Rainfall-runoff dynamics

The 1999 floods: estimation of runoff deficits and influence of the land use

Let us come back to the 1999 floods. It was possible on some streams, owing to the number of witnesses and the level of detail of their accounts, to identify various water levels referenced in time: typically the time of the river bank overflow, of a bridge overflow, the time of the various flood peaks. These points of reference are represented by green bars in Figure 27. The vertical span represents the uncertainty in the discharge estimates – relatively large – and the horizontal span, the uncertainty in time (+/- 15 minutes with regard to the time indicated by the witnesses).

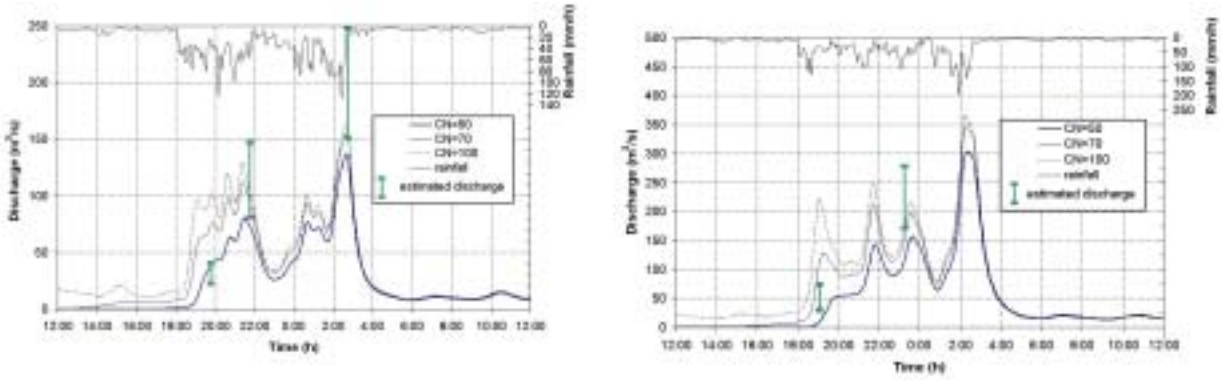


Figure 27: Comparison between estimated and simulated discharges for two upstream watersheds in the Aude region after the 1999 floods: (a) Tournissan (10 km^2), (b) Verdoul (18 km^2)

The comparison of the points of reference and of simulated flood hydrographs reveals two main characteristics of the Aude river tributary flash-floods. Firstly, it reveals the reactivity of the watersheds to the rainfall intensity fluctuations. According to the recorded radar rainfall data, the Tournissan watershed has been affected by two rainfall bursts which produced two flood peaks separated by a clear decrease of the discharge observed by the witnesses. Without surprise, the flash-floods volumes are mainly produced by fast-responding rainfall-runoff processes: typically surface runoff. The second characteristic is the delay of the response of the watersheds at the initial stage of the flood. According to the simulation results, this delay is not due to the transfer time of the flood flows on the watershed, but reveals that despite the high rainfall intensities, a large proportion of the initial rainfall volumes did not produce runoff and were stored on the watersheds, probably infiltrated in the soils and sub-soils. The calibration of the "curve number" of the rainfall-runoff model used (see appendix I), leads to a runoff deficit of about 250 millimetres (CN=50) for the Tournissan watershed. This value is relatively high but in accordance with values estimated for other flash-floods (Belmonte & Beltran, 2001).

Moreover, the analysis of the river gauge measurements available on the Aude main stream for the 1999 flood event leads also to the same range of runoff deficit values. Figure 28 shows the reconstructed flood hydrographs of the 1999 Aude river flood upstream and downstream the area which received the larger rainfall amounts: more than 300 millimetres on average on the sub-watershed located between the two gauging stations. The flood volume corresponding to the difference between the two hydrographs related to the area of the intermediate sub-watershed represents 90 millimetres. So, at least 200 millimetres of rainfall did not contribute to the flood and were stored on the intermediate watershed.

On these watersheds, high intensity rainfall rates seem not to be sufficient to trigger a flash-flood, but a rainfall accumulation (i.e. a certain level of saturation) is necessary. Despite the relatively high rainfall intensities (more than 50 mm/h), the 1999 flash-floods producing processes seem not to be of the hortonian type.

Finally, the comparison of the Tournissan and Verdoul watersheds, having the same geology, gives some insight into the possible impact of the land use type (Figure 27). 50% of the Tournissan watershed area is covered by vineyards while the Verdoul watershed is essentially covered by forest and scrub. According to the gathered data and the simulated hydrographs, the reaction delay has been observed on both watersheds. The runoff deficit may even have been lower on the Verdoul watershed. But according to the level of accuracy of the data it is not possible to clearly demonstrate it. The main conclusion is that the land use may have an influence on the rainfall-runoff dynamics of a watershed, but it could not be revealed on the basis of the collected data.
For the studied watersheds in the Aude region, the type of land use seems to have a limited influence on their rainfall-runoff dynamics during flash-floods. Of course, this conclusion should not be generalized to any watershed. The effect of land-use on floods may, in particular, depend on the soil types and the geology.

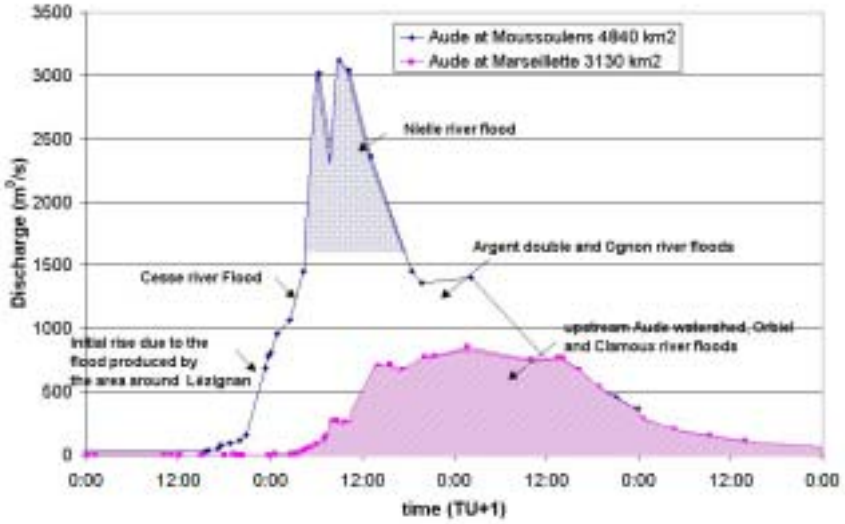


Figure 28:1999 flood hydrographs estimated on the Aude river main stream on the basis of the measured data of two river gauging stations upstream and downstream the part of the watershed affected by more than 250 mm of rainfall

The 2002 floods: estimation of runoff deficits and influence of the geology

Estimated discharges were also compared to simulated hydrographs for some upstream tributaries to analyse the rainfall-runoff dynamics. This revealed features significantly different than what had been observed on the Aude 1999 flood. Figure 29 shows such a comparison for two tributaries where a complete flood hydrograph could be relatively accurately reconstructed using existing water level measurements in flood control dam spillways.

The hydrological model appears well suited to the Crieulon stream estimated flood hydrograph. As in the previously presented example, the watershed reacts with a slight delay to the rainfall bursts. But the calibrated curve number (70) indicates a moderate water retention capacity of the soils and subsoils of the watershed (about 100 millimetres), relatively low if compared to the Aude case and to other studies of recent flash-floods (Belmonte & Beltran, 2001, Cosandey, 1993). On the other hand, the rainfall-runoff model is not able to reproduce the flood hydrograph of the Vidourle watershed. Its runoff coefficient never seems to exceed 50%: the watershed still has some rainfall water retention capacities even during the peak of the flood. Moreover, unlike what is observed on the Crieulon, a relatively high discharge, not simulated by the rainfall-runoff model, remains after the rain event has ceased indicating that a part of the temporary stored rainwater is rapidly returned to the stream after the event. About one third of the flood volume is released during the few days after the flood event in

the case of the Vidourle river. Taking into account this late released volume, the retention capacity of the Vidourle watershed appears also to be about 100 millimetres.

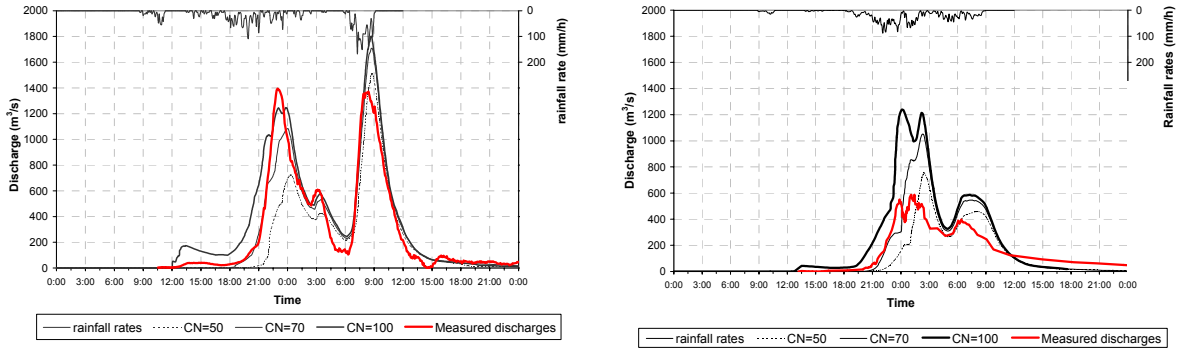


Figure 29: Comparison between estimated and simulated discharges for two upstream watersheds in the Gard region after the 2000 floods: (a)Crieulon (90 km²), (b)Vidourle (80 km²)

The geology and the corresponding soil types can be put forward to explain those clear differences in the rainfall-runoff reactions of both watersheds. The Vidourle watershed is mainly composed karstified limestone which explains its large retention capacities during the flood but also the rapid release of one part of the water stored in the karst after the flood. As for the Crieulon catchment, it is mainly composed of marls.

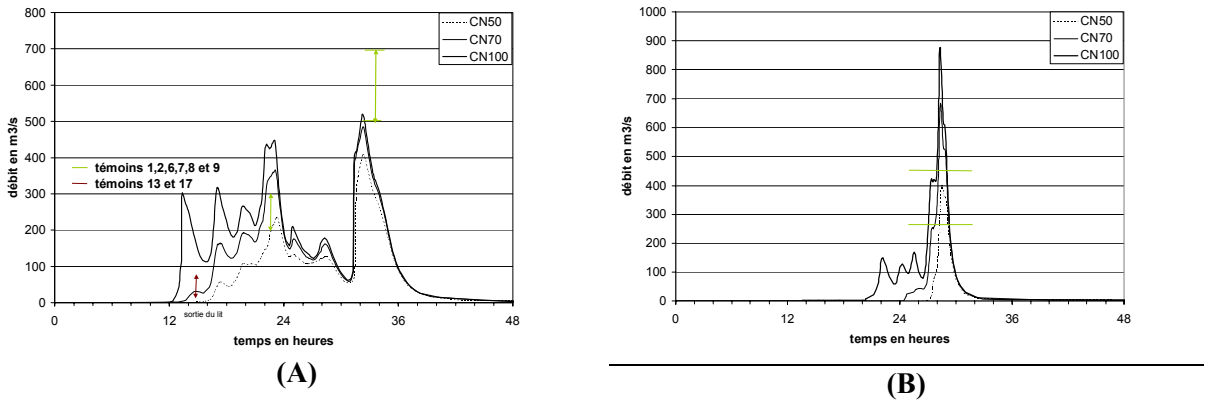


Figure 30: Comparison between estimated and simulated discharges for two upstream watersheds in the Gard region after the 2000 floods: (a) Bourdic (39 km²), (b) upper Gardon (32 km²)

Figure 30 shows two further examples of tributaries for which only a limited number of discharge values could be estimated. The reaction of the Bourdic, covered by marls and non-karstified limestone appears very similar to the reaction of the Crieulon. It reacts with a certain delay, the first rainfall burst does not induce a first flood wave, but leads only to the overflow of the river over its banks. The best suited "Curve number" value appears to be 70 (i.e. runoff deficit of 100 millimetres). Conversely, the estimated peak discharge of the upper Gardon river implies a relatively higher retention capacity of the watershed. All the other discharge estimations realised on the upper Gardon river lead to the same conclusion. Moreover, as in the case of the karstic Vidourle watershed, the river gauge measurements available on the upper Gardon river indicate the persistence of a significant flow during a few days after the 2002 flood. This part of the Gardon watershed was considered before the 2002 floods as the area most prone to flash-floods. It is mountainous, with steep slopes and narrow valleys. The subsoil is composed of schist stone and is covered by thin soils. The postflood investigation revealed that from a hydrological point of view, it reacts in a way very similar to karstic areas: high infiltration and retention capacities during the storm event and rapid release of one part of the stored water volumes of

the flood. Field investigations and infiltration tests have revealed that the upper schist layer is highly fractured and altered and have confirmed the very high retention volume and permeability of this layer (Ayral, 2005).

5.2.3 Time sequence of the flood

In the morning of the 9th of September 2002, a mesoscale convective system producing intense rainfall rates swept across the Gard region from the North-West to the South-East, which is exactly the orientation of the three main streams. At a first sight, this could be seen as a very unfavourable progress of the rainfall event. A detailed analysis of the data gathered on the Gard river and its tributaries, especially the times of flood peaks indicated by witnesses, does not confirm this first impression and reveals the very important attenuation role played by the large flood plain which occupies the intermediate Gard waterway. This analysis is summarized in Figure 31.

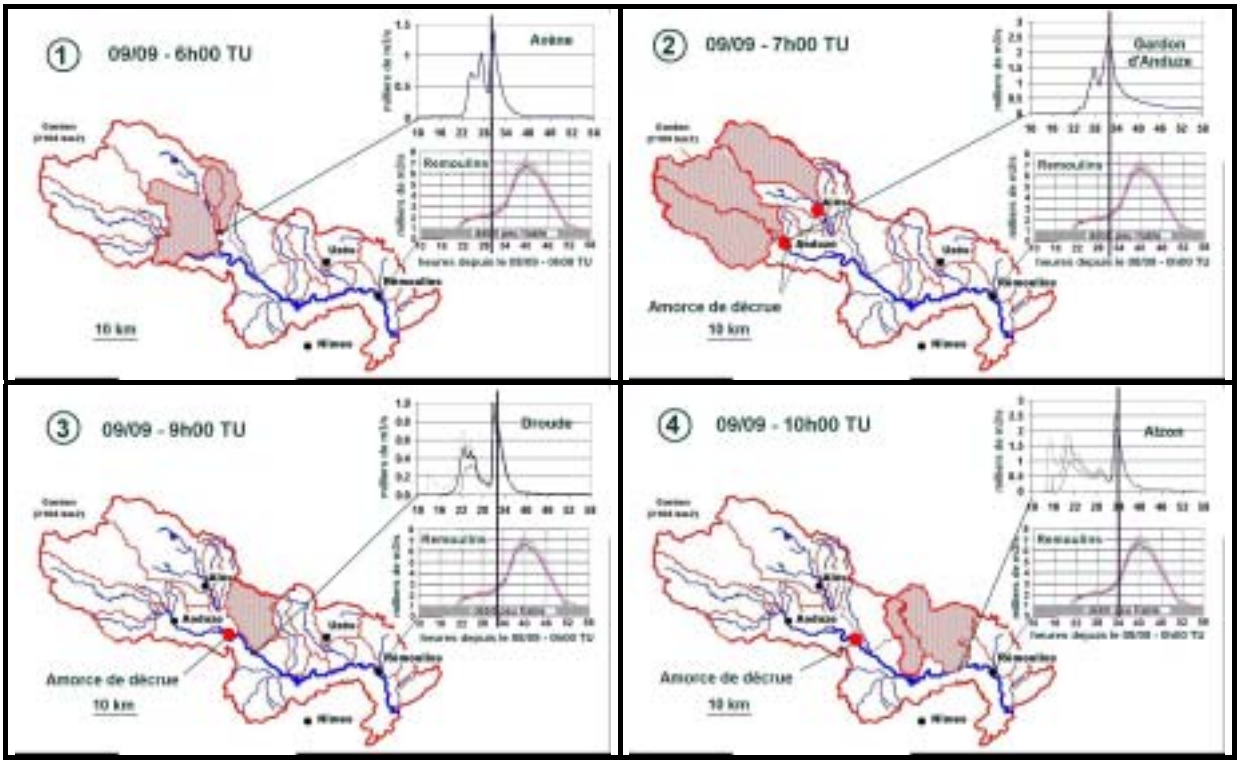


Figure 31: Time sequence of the 2002 Gard river flood and of the contributions of the sub-watersheds. The beginning of the decreasing limbs of the flood hydrographs are indicated in red for the tributaries and with a red point for the main stream. Simulated hydrographs of some tributaries and measured downstream hydrograph in Remoulins.

The rainfall event which occurred in the morning of the 9th of September 2002 induced the succession in time of the floods and peak discharges of the tributaries from upstream to downstream as shown in Figure 31. But the available river gauge measurements on the Gard river main stream indicate that in the upstream flood wave (indicated by the red points on the figure) progressed very slowly in the two kilometres wide flood plain comprised between the confluence of the Anduze and Alès Gardons downstream both towns and the Gard gorges located in the North of the Nîmes city. The flood wave celerity is estimated to about 1m/s in the flood plain and more than 3 m/s upstream and downstream these flood plain. Due to this slowing down, the resulting Gard downstream flood wave in Remoulins was composed of the succession in time of the contributions of the various tributaries rather than of the superposition of these contributions. The Gard river flood lasted 18 hours in Remoulins. Without the slowing down effect in the flood plain, the same flood volume would have passed in Remoulins within a reduced period of time, and the peak discharge would have been significantly higher …

5.2.4 About the return period of floods

The return period of extreme floods is an important question for stakeholders. It is a question not easy to answer since the discharges estimated during these extreme floods, freaks of nature (NCDC, 2005), are generally far higher than discharges observed on gauging stations in the same region during the last few decades. As illustrated in Figure 32, the result of standard statistical extrapolation procedures based on the calibration of a flood peak distribution model on available series of 10 to 30 years of measured peak discharges, depends highly on the available series (sensitivity to sampling fluctuation) and on the distribution model used. Clearly, the method is not suited to assess the return period of the 1999 flood peak discharge on the Clamoux river. The estimated values with the tested distributions – more than 100 million years - are obviously absurd.

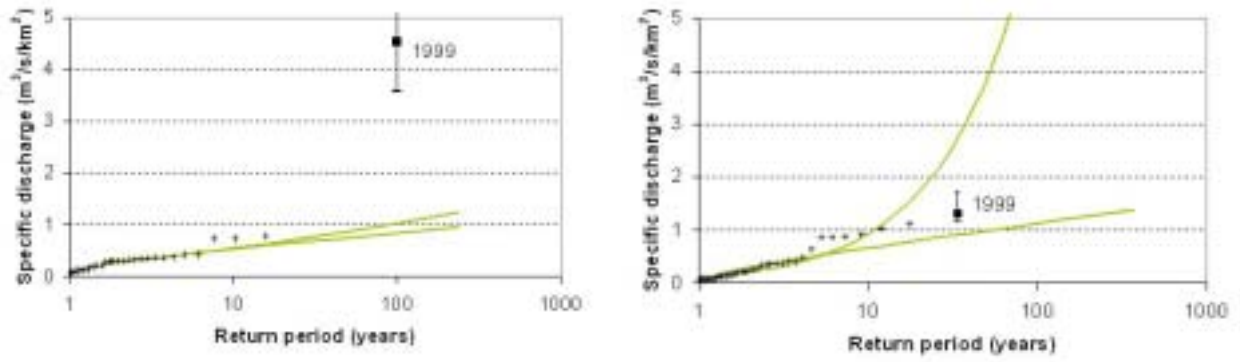


Figure 32:Flood peak distributions of two small gauged watersheds located in the Aude region (France), adjusted extreme value types 1 and 2 distributions and proposed position of the 1999 flood peak: a) Clamoux (42 km^2) and b) Orbiel (73 km^2)

What is then the return period of the 1999 flood on the Clamoux river? The common sense would advise to check if some comparable events occurred in the recent past to evaluate how rare it is. And in fact, it is striking how often older witnesses recalled previous flood events when they were questioned on this issue during post flood studies. Those freaks of nature may not be rare. To test this hypothesis, an inventory work of the extreme floods of four small watersheds of the Aude region based on archives has been conducted (Payrastre *et al.*, 2005). The archive documents (generally plans of cross-sections with indications of water levels) made it possible to evaluate the peak discharges of the major floods of these watersheds over a period of one or two centuries.

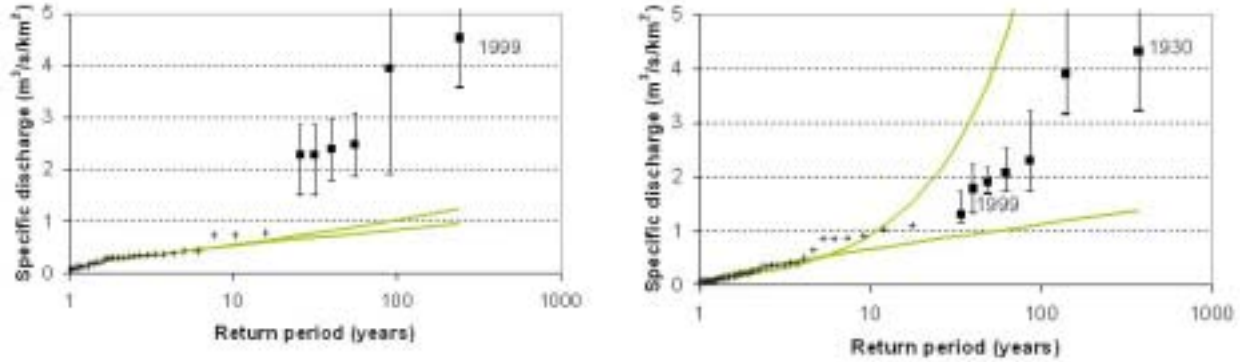


Figure 33: Same as Figure 32 including the reconstructed historical floods over the two past centuries: a) Clamoux (42 km^2) and b) Orbiel (73 km^2)

The obtained results for the Clamoux and Orbiel rivers are presented in Figure 33. In comparison with Figure 32, the 1999 flood appears less isolated for the Clamoux river: one very comparable flood has

been observed in the recent past and four other floods have reached half of the 1999 discharge. Moreover, the 1999 flood peak discharge was significantly lower on the Orbiel river, but other historical floods of the Orbiel compete with the 1999 Clamoux river flood. Taking into account the historical floods, both flood peak distributions appear very similar.
As a conclusion, common collective memory on natural disasters is relatively short (a few decades), especially in rural areas and small watersheds prone to flash-floods where the number of stakes at risk is limited. Information on past floods generally exists, either in archives, known by some inhabitants or marked somewhere on the wall of a house or the pillar of a bridge. But it is tedious to collect it and it is often disregarded in technical studies because it is considered as too inaccurate. Another important issue of post-flood investigations is the preservation of the information on the extreme floods, absolutely necessary to reduce the uncertainties of studies dealing with flood risk and flood prevention as shown here. And to avoid that our descendents in a few decades discard the data we are collecting now, it is necessary to store not only some estimated peak discharges, but also and mainly the rough data used to estimate these discharge values.

6. Conclusions

We hope that these few examples have convinced the readers of the usefulness of post flash-flood surveys. It is a tedious, difficult and apparently arid task. We have shown here that recent surveys have revealed important and sometimes unexpected aspects of flash-floods: importance of the geology and soil types, counter-intuitive behaviour of some areas, limited impact of the land use type, limited contribution of hortonian runoff processes. These observations are also stimulations for a better understanding of the underlying flood generating processes.

A post-flood survey procedure has been suggested herein as well as some methods. It is a first proposal which certainly will be amended. But beyond the procedure and methods, it is important to keep in mind the general philosophy: the data collected are necessarily inaccurate, no method is perfect and the very first concern must be to verify, crosscheck, verify and crosscheck again. It is the only way to limit the risks of errors on peak discharge estimates for instance as illustrate herein.

Finally, we hope that this document has also convinced the readers that accounts of eyewitnesses are also an important source of data, especially concerning the time sequence of floods, which is an essential information to study the rainfall-runoff dynamics.

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8. Appendix I: short presentation of the CINECAR rainfall-runoff model

The CINECAR rainfall-runoff model has been specifically developed to guide the interpretations of data collected during post flash-flood investigations. It is a simple semi-distributed model. Its main characteristics are as follows: 1) the flood flows are assumed to be essentially composed of surface runoff water, and other sources are set aside, 2) The SCS (``soil conservation service'') model is used to calculate the evolution of the mean runoff coefficient on each sub-watershed during the storm event (Equation 8-1), and 3) the kinematic wave model is used to route the flood flows through a watershed.

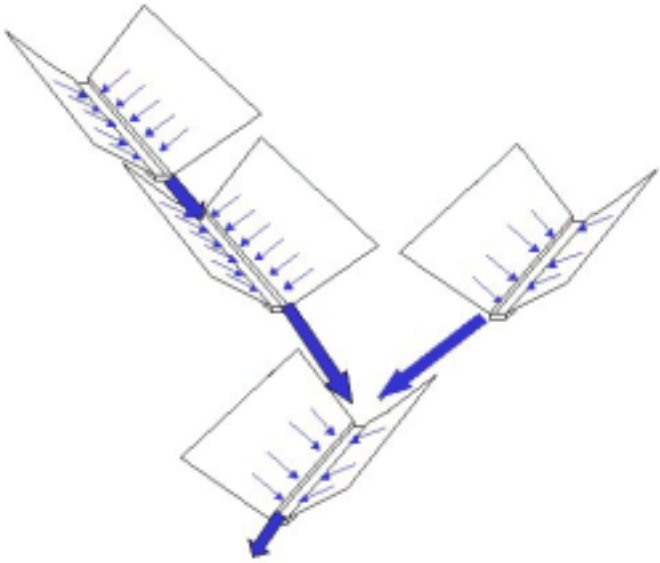


Figure 34: Representation of a watershed in the CINECAR model

In the model, a watershed is represented as a cascade of river reaches having a rectangular cross-section, connected to two rectangular slopes (see Figure 34). The number of reaches depends on the morphological complexity of the watershed rather than on its area. Considering the Froude numbers and the distance scales used, the kinematic wave model appears to be an accurate approximation of the Saint-Venant shallow water equations governing one-dimensional unsteady free surface flows (Borah *et al.*, 1980). This is the reason it was selected. An analytical method is used to solve the kinematic wave differential equations (Daluz-Vieira, 1983) to avoid the disturbing effect of numerical diffusion on the model results. The flow routing model is not, strictly speaking, calibrated: the width and Manning coefficients of each river reach correspond to those estimated during the field study. The Manning coefficient of the slopes was systematically set as being equal to 0.1 when the area was larger than one square kilometre - with the slope incorporating small streams - and equal to 0.2 in other cases. The river reach width had to be adjusted in the case of large valleys with overbank flow. In most cases, the modelled time to peak corresponds fairly accurately with the information provided by witnesses.

The SCS model was selected from among others because of its simplicity to estimate the rainfall contributing to the runoff. The evolution of the runoff coefficient value during the storm event depends on a single coefficient.

$$V_t = \frac{(P_t - 0.2S)^2}{(P_t + 0.8S)}$$

Equation 8-1

In which V_t is the total runoff volume from the beginning of the storm event in millimetres (V_t =0 if P_t < 0.2S), P_t is the total rainfall amount in millimetres, and S is the retention capacity of the watershed, also in millimetres, given by: S=25.4(1000/CN-10) in which CN is the so-called ``curve number'', and has a value in the range of 0 (constant runoff coefficient equal to 0%) to 100 (constant runoff coefficient equal to 100%).
The instantaneous runoff coefficient C_t at time t only depends on the total rainfall amount from the beginning of the event and not on the intensity of the rainfall:

$$C_t = 2\frac{(P_t - 0.2S)}{(P_t + 0.8S)} - \left(\frac{(P_t - 0.2S)}{(P_t + 0.8S)}\right)^2$$

Equation 8-2

The calibrated values of the curve number give two types of indirect information concerning the rainfall-runoff process through the watershed. On the one hand, it provides some idea of the catchment area's mean runoff coefficient at various moments during the storm event. On the other, if the match between the model results and the collected data is acceptable, then the CN value provides an indication of the watershed's water retention capacity.