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Thesis:

PROBABILISTIC SIZING OF FLOOD RESERVOIR USING UNIVARIATE AND MULTIVARIATE (COPULA) APPROACH IN BARCELLONA POZZO DI GOTTO, SICILY

Flash flood in torrente Longano, Barcellona Pozzo di Gotto in 22 November 2011.
(The photo is taken by Amadore Antonio)

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Contents

Abstract

In the present study are presented two methods of probabilistic sizing of flood reservoirs. These methods have been tested on two catchments which join in the Longano River near to Barcellona Pozzo di Gotto, which is located in the north part of Sicily, Italy. The one method is by using univariate approach and the other by using multivariate (copula (Sklar, 1959; Nelsen, 1999)) approach. In the univariate approach a GEV (Fisher–Tippett distribution) distribution is fitted to the rainfall depths and after the flood peaks and flow volumes are generated by a rainfall-runoff model, whereas in multivariate approach the generation of flood peaks and flow volumes has been obtained via copulas at 100 years return period, which describe and model the correlation between these two variables independently of the marginal laws involved. Also, in the latter approach the shape of the hydrographs has been generated on the basis of an average method. Then, in both cases have been used the modeling of flood propagation by a hyperbolic finite element model based on the DSV equations. (Aronica et. al, 1998) Finally, the results from both methods are compared.

Περίληψη

Στην παρούσα μεταπτυχιακή εργασία παρουσιάζονται δύο μέθοδοι πιθανολογικής διαστασιολόγησης των δεξαμενών πλημμυρών. Αυτές οι μέθοδοι έχουν δοκιμαστεί σε δύο λεκάνες απορροής που ενώνονται στον ποταμό Longano κοντά στην περιοχή Barcellona Pozzo di Gotto, η οποία βρίσκεται στο βόρειο τμήμα της Σικελίας, στην Ιταλία. Η μία μέθοδος είναι με τη χρήση μονομεταβλητής προσέγγισης, ενώ η άλλη με τη χρήση πολλαπλών μεταβλητών (copula). Στην μονοπαραγοντική προσέγγιση χρησιμοποιήθηκε η κατανομή τύπου GEV στα ύψη βροχής και μετά οι μέγιστες παροχές και όγκοι ροής δημιουργήθηκαν από τη χρήση ενός μοντέλου βροχής-απορροής, ενώ στη πολυπαραγοντική προσέγγιση οι μέγιστες παροχές και όγκοι ροής δημιουργήθηκαν μέσω συνδέσμων σε 100 χρόνια περιόδου επανεμφάνισης, που περιγράφουν και μοντελοποιούν τη συσχέτιση μεταξύ των δύο αυτών μεταβλητών ανεξάρτητα από τις εμπλεκόμενες οριακές. Επίσης, στην τελευταία αυτή προσέγγιση το σχήμα των υδρογραφημάτων έχει δημιουργηθεί επί τη βάσει της μεθόδου του μέσου όρου. Στη συνέχεια, και στις δύο μεθόδους έχει χρησιμοποιηθεί μοντελοποίηση της διάδοσης των πλημμυρών από ένα μοντέλο πεπερασμένων στοιχείων τύπου υπερβολής με βάση τις εξισώσεις DSV. Τέλος, τα αποτελέσματα των δύο μεθόδων συγκρίνονται.

Résumé

Dans ce rapport d'un stagiaire sont présentées deux méthodes de dimensionnement des réservoirs probabiliste des crues. Ces méthodes ont été testées sur deux bassins versants qui se rejoignent dans la rivière Longano près de Barcellona Pozzo di Gotto, qui est situé dans la partie nord de la Sicile, en Italie. La seule méthode est d'utiliser l'approche univariée et l'autre à l'aide de plusieurs variables (copule (Sklar, 1959; Nelsen, 1999)) approche. Dans l'approche univariée un GEV (Fisher-Tippett distribution) de distribution est monté sur les profondeurs des pluies et après les pointes de crue et des volumes de flux sont générés par un modèle pluie-débit, alors que dans l'approche multidimensionnelle de la génération des pointes de crue et des volumes de flux a été obtenu par copules à 100 période de retour ans, ce qui décrire et modéliser la corrélation entre ces deux variables indépendamment des lois marginales impliquées. Aussi, dans cette dernière approche de la forme des hydrogrammes a été générée sur la base d'une méthode de la moyenne. Puis, dans les deux cas ont été utilisés la modélisation de la propagation des crues par un modèle éléments finis hyperbolique basé sur les équations de DSV. (Aronica et al., 1998) Enfin, les résultats des deux méthodes sont comparées.

1. Introduction

Floods are the most frequent natural disaster worldwide. However not all floods are alike. Some floods develop slowly, whereas others such as flash floods can develop in just a few minutes and without visible signs of rain.(http://en.wikipedia.org/wiki/Flood)

Many studies show that the severity and frequency of floods have increased in recent years and underline the difficulty to separate the effects of natural climatic changes and human influences as land management practices, urbanization, etc. (Blöschl and Montanari, 2010; CRED, 2004; Di Baldassarre et al., 2010a; Kleinen and Petschel-Held, 2007). It is a fact that these events may have serious socio economic impacts in a community, causing victims, population displacement and damages in environment, ecology, landscape and services. During the last two decades economic and insured losses have drastically increased due to severe floods (Munich, 2006, 2008), and intangible impacts, for instance loss of life, memorabilia, health and cultural damages, stress or psychological trauma, have been considered even more serious than the loss of property (Green and Penning-Rowsell, 1986). Indeed, the damages of a flood event can be in a small range, impacting a neighborhood or community, or in a wide range, affecting entire river basins and multiple states.

Flash floods can destruct a dam or levee within a few minutes or hours of excessive rainfall. Moreover, flash floods often have a dangerous wall of roaring water carrying rocks, mud and other debris. Overland flooding which is the most common type of flooding event typically occurs when waterways such as rivers or streams overflow their banks as a result of rainwater or a possible levee breach and cause flooding in surrounding areas. It can also occur when rainfall or snowmelt exceeds the capacity of underground pipes, or the capacity of streets and drains designed to carry flood water away from urban areas.(http://www.ready.gov/floods)

While flood modeling is a fairly recent practice, attempts to understand and manage the mechanisms at work in floodplains have been made for at least six millennia. The recent development in computational flood modeling has enabled engineers to step away from the tried and tested "hold or break" approach and its tendency to promote overly engineered structures. Various computational flood models have been developed in recent years; either 1D models (flood levels measured in the channel) or 2D models (variable flood depth measured across the extent of the floodplain). HEC-RAS, the Hydraulic Engineering Centre model, is currently among the most popular if only because it is available free of charge. Other models such as TUFLOW combine 1D and 2D components to derive flood depth across the river channel and floodplain. To date the focus has primarily been on mapping tidal and fluvial flood events, but the 2007 flood events in the UK have shifted the emphasis there onto the impact of surface water flooding. (http://en.wikipedia.org/wiki/Flood)

Engineers involved in the design of hydraulic structures in river systems are often confronted with a lack of available data regarding the phenomenon under study, e.g. peak discharges at a specific point in a catchment. As rainfall data are often readily available, they usually serve as an indispensible source of information for the further analysis. A variety of point rainfall data products can be used in such design studies, thus, the historical time series, a synthetically generated time series, intensity-duration-frequency (IDF) relations and design storms.

Unfortunately, there is not a standardized agreed method for flood risk evaluation. Several hydrological models and methodologies to simulate hydraulic behavior of river systems make this task hard. Therefore, identification, assessment and mitigation of flood risk require a rich fund of knowledge regarding these methods and the area under study and should be based on a thorough uncertainty analysis. (Aronica et al., 2011)

Nowadays, every research activities should take into account the uncertainty related to the results, and this should become a common practice of the endusers. Merz and Kreibich (2008) emphasize the value of uncertainty considerations in flood risk analysis for decisions on flood mitigation and risk management.

In fact, flood defense systems are usually designed taking into account their capability to bear events corresponding to a settled exceedance probability (e.g. 100 years return period). In this case, more or less advanced deterministic approaches for modeling flood inundation are used. (Aronica et al., 2011) The usual procedure is to deal with a rainfall-runoff model and apply a flood frequency analysis to a given record of discharge data. However, only the flood discharges cannot give a reliable evaluation of hazard, depending also by the global characteristics of the flood event. In that way is not possible to take any account of the uncertainty and may lead to assessment of hazard.

Uncertainties came from the observed data and each processing step and accumulate in the final outputs. Major sources of uncertainty include the statistical analysis of extreme events from short time series, incorporating themselves measurement errors, the spatial extrapolation of data, the resolution and accuracy of DEM, the presence of defence structures, the process models (that are not perfect representation of reality) and parameterisation, scarce data for model validation, the flood damages estimation often based on data from limited numbers of events.

To model flood inundation a probabilistic approach seems to be more correct and suited to represent hazard (Di Baldassarre et al., 2010b). Many researchers have proposed more comprehensive design procedures (Apel et al., 2004, 2006; Bóchele et al., 2004). Risk oriented method is the most complete approach that can be used in the fields of flood design and flood risk management taking in account hydrological, hydraulic, economic, social and ecological aspects of the flood risk.(Aronica et al., 2011)

As mentioned in the previous paragraph, the frequency analysis is primarily based on the estimation of the PDF. The parametric approaches for estimating the PDF must assume that the data are drawn from a known parametric family of distributions. However, many studies on frequency analysis indicate that there is no universally accepted distribution for representing the hydrologic variables (Adamowski, 1985, 1996; Silverman, 1986; Yue et al., 1999; Smakhtin, 2001). It is evident that the parametric method, which depends on prior knowledge of the particular distribution function, has its limitations (Cunnane, 1985, Adamowski, 1989) and, as pointed out by Dooge (1986), "no amount of statistical refinement can overcome the disadvantage of not knowing the frequency distribution involved". To overcome some of the limitations of parametric method, nonparametric density function estimations have been explored in hydrologic frequency estimation (Lall, 1995). Nonparametric method does not require the assumption of any particular form of density function. As the choice of a distribution is bypassed altogether, the nonparametric method is uniform and its adoption would eliminate the need for any arbitrary imposition of uniformity in flood frequency analysis (Adamowski, 1989). Using the weighted moving averages of the records from a small neighborhood of the point of estimation, nonparametric function estimations have the advantage that they always reproduce the attributes represented by the sample (Lall, 1995; Sharma, 2000; Kim et al., 2003). Number of existing studies on nonparametric methods for hydrologic frequency analysis is not large. However, they show that nonparametric methods are accurate, uniform and particularly suitable for multimodal data. Adamowski (1985) proposed a nonparametric kernel estimation of flood frequencies. Lall et al. (1993) focused on the selection of the kernel function, representing the shape, and the bandwidth in nonparametric kernel estimation for flood frequencies. The techniques for selection of kernel function and band width are applied to three situations: Gaussian data, skewed data and mixed data. Adamowski (1989) performed a Monte Carlo simulation experiment to compare the nonparametric method with two parametric distributions, namely, log-Pearson type III and Weibull distributions. Results show that the nonparametric method gives more accurate results than parametric methods. Adamowski (1996) further proposed a nonparametric method for low-flow frequency analysis to find the conditions of drought. Kim et al. (2003, 2006) proposed a methodology for estimating bivariate drought return periods based on two different severity indices using nonparametric kernel estimator. A perusal of the statistical literature shows that nonparametric statistical estimation, using splines, kernel functions, nearest neighbor methods and orthonormal series methods (Efromovich, 1999; Bowman and Azzalini, 1997; Higgins 2004), is an active area of research, with major developments still unfolding (Silverman, 1986; Scott, 1992; Sharma et al., 1997).

In Italy the hydrogeological instability is partly natural, since the country has a high hydrogeological risk and partly is the poisoned fruit of decades in which the land has been systematically abused: an emergency less and less exceptional and happening more and more on a daily basis. 3,671 municipalities are within a high risk, 45% of the total. Only in the last 10 years there have been 12,993 problematic hydrogeological events, including six catastrophic landslides and floods with at least 50 billion euros of damage. (http://picturetank.com)

This work proposes a probabilistic sizing of flood reservoir using univariate and multivariate (copula) approach in Barcellona Pozzo di Gotto, Sicily. The Monte Carlo approach has been used to sample the hydrological input to a two-dimensional hydraulic model. Synthetic hydrographs have been obtained generating flood peak discharges and volumes through copula theory, and shapes through non-dimensional or normalized hydrographs. Therefore, in section 2 there is a thoroughly analysis concerning the methodology of the used approaches. In Section 3 presents in detail the study area located in Sicily that was subjected to several floods in the past and section 4 shows the comparison of the results. Finally, in section 5 there are the conclusions of the present study and recommendations and thoughts for future research.

An overall view before the methodology

In the present study the main scope is to size a reservoir. The flood control reservoirs are usually sized by using 100 year return time event. Despite the fact that it is possible to size a reservoir by using a univariate approach and, in this study it is impossible to accomplish the sizing only by the univariate approach because by the calculation there is only a 100 year peak discharge without the corresponding 100 year volume. To that end, a multivariate approach (bivariate copula) is proposed by which it is feasible to obtain couples of correlated peak discharges and flood volumes of a joint 100 year return period. Therefore, by comparing the results of the two methods, it is feasible to size the reservoir. This can happen because one can find the "optimum" event to be used to have the "best" behaviour of the flood reservoir

2. Methodology

This section describes in detail the procedure to use the univariate and multivariate approach in order to size the flood reservoir. In particular, the univariate and multivariate approach here developed have a modular structure consisting of different modules: flood frequency analysis to gain the hydrological input to the hydraulic model, transformation of flood discharge to inundated area through a two-dimensional hydraulic model.

The delineation of the catchments and the preparation of the files

At first, the delineation of the two catchments was done through the regional technical map of the planning department - Department of territory and environment. Concerning the delineation of the catchment some general techniques are used.

- The watershed line is detected by the closest section chosen.

- The watershed line encompasses the entire river system upstream of the section chosen, in practice, the line never intersects any branch of the hydrographic network.
- The watershed line passes through the points at higher levels of the river basin.
- The watershed line is always perpendicular to the contour lines.
- The boundary of the basin is facilitated when the outflows are derived from only sliding surface.
- Is extremely difficult, however, the delimitation of the basin when there is underground runoff.

Afterwards, further procedures are done in order to obtain the inputs for the rainfall-runoff model. Specifically, by the use of the ArcView, a software program produced by Esri.

Rainfall-Runoff model

In the present study the used rainfall-runoff model is based on the same model which is described in the paper of Candela et al., 2012.

Inputs of the rainfall-runoff model

1. Flow length: Calculates the distance or the weighted distance along a flow path.

2. Curve number: Is based on the hydrologic soil group and ground cover. It is an empirical parameter used in hydrology for predicting direct runoff or infiltration from rainfall excess.

3. Digital elevation model: Digital model or 3D representation of a terrain's surface which is created by terrain elevation data.

4. n= 0.432

5. a= 28.102 mm/h

   The **"n"** number is the exponent of the duration and the **"a"** number is a factor which multiples the duration. In other means, these two numbers are the parameters of the Intensity Duration (IDF) Curve.

6. Time step= 15_min

7. Grid size = 10_m

8. Roughness coefficient (Strickler) = 30 $m^{-1/3}/s$

9. Duration from 1h to 10h

10. Hyetograph of historic form (the hyetograph is taked by a rainfall analysis near to a station in Messina)

11. c=0.1(SCS-CN in dynamic form)

Outputs of the rainfall-runoff model

1. Couples of maximum discharges and volumes in different durations. (Q_{max}, V_{max}).

2. Hydrographs in accordance to different durations.

A more general description

According to the literature, the 300 pairs of Q_{max} and V_{max} are quite enough to sizing the reservoir. Therefore, in the univariate case 300 runs are done in order to obtain the pairs of Q_{max} and V_{max}. In the second case of the bivariate copula the methodology is quite different in order to obtain the 300 pairs with the joint return period of 100. To that end, in the multivariate case 1000 runs are done to obtain the pairs of Q_{max} and V_{max}. At first there is a fitting in the marginals for both Q_{max} and V_{max} and afterwards the copula selection. From these 1000 pairs, in the end, are generated and are used only 300 pairs of Q_{max} and V_{max} which are calculated by fixing joint probability equal to 0.99. In other words, only the couples with a joint return period of 100 year are chosen.

- **Univariate approach**

  - **Matlab software - Rainfall-Runoff univariate model**

In the univariate case, a model with the above input characteristics (section Raifall-Runoff model) is used in order to derive the 300 pairs of Q_{max} and V_{max}. By other means, in this approach a Generalized Extreme Value (GEV) distribution is fitted to maximum rainfall depths.

- **Multivariate approach**

  - **Matlab software – Rainfall-Runoff multivariate model**

In the multivariate case, a model with the above input characteristics and the incorporation of a Frank copula is used to derive the 1000 pairs of Q_{max} and V_{max}. Is well known in literature that in a storm duration, the average peak flows and rainfall volumes are variables of the same phenomenon and they are correlated, so they have to be analyzed jointly through multivariate models and, particularly, through copulas.

Eventually, concerning the copula approach one will end up with an ensemble of (u_i, v_i) -pairs (with i ranging from 1 to n, the ensemble size). By means of the inverse marginal CDFs, these pairs are easily transformed to real values. This ensemble could then be used to run simulations from which detailed information on the uncertainty of specific design parameters can be assessed. As an example, one could route an ensemble of 1000 pairs of peak discharge and volume through a dam model (approximately our case) and consider the water height in the reservoir. Using just one design event, only one single water height is obtained. However, using the ensemble, information on the range and likelihood of possible water heights for the given design joint return period is obtained, making it possible to incorporate the uncertainty in the design. (S. Vandenberghe et al., 2012)

❖ The Copula Function

Although, the word 'copula' is first employed in a mathematical or statistical sense by Abe Sklar (Sklar, 1959), many of the basic results on copula are traced to the early work of Wassily Hoeffding (Hoeffding, 1940) (Nelsen, 1999). The existing mathematical theories developed on copula for multivariate dependence analysis is quite significant. The only definitions and concepts related to copulas, used in the present study, are discussed in this section. The interested reader is referred to Joe (1997) and Nelsen (1999) for further details.

A copula is a joint distribution function of standard uniform random variables. A bivariate copula can be represented as:

$$C:[0,1]^2 \rightarrow [0,1] \qquad (1)$$

It has to fulfill the following conditions: the first one is: $C\{1,u) = C(u,1) = u$ and $C(u,0) = C(0,u) = 0$, and the second is the following: $C(u_1,u_2) + C(v_1,v_2) - C(u_1,v_2) - C(v_1,u_2) \geq 0$ if $u_1 \geq v_1, u_2 \geq v_2$ and $u_1,u_2,v_1,v_2 \in [0,1]$. The second condition ensures that the probability corresponding to any rectangle in the unit square is nonnegative. The Sklar' s theorem (1959) is the foundation of the concept of copula.

Sklar (1959) showed that every ndimensional distribution function F can be written as:

$$F(x_1,.....,x_n) = C(F_1(x_1),........,F_n(x_n)) \quad (2)$$

where F_1, …, F_n are marginal distribution functions. If F_1, …, F_n are continuous, then the copula function C is unique and has the following representation:

$$C(u_1,.....,u_n) = F(F_1^{-1}(u_1),.......,F_n^{-1}(u_n)), \quad 0 \le u_1,......,u_n \le 1 \quad (3)$$

where the quantile F_k^{-1} is defined by $F_k^{-1}(u_k) = \inf\{x \in \Re / F_k(x) \ge u_k\}$. Conversely, it can be proven that if C is a copula function and F_1, …, F_n are distribution functions, then the function F defined by equation (11) is a n-dimensional distribution function with margins F_1, …, F_n (Nelsen, 1999).

The Archimedean, elliptical, extreme value copulas are some widely applied classes of copula functions. In the present study, Ali-Mikhail-Haq, Cook-Johnson and Gumbel- Hougaard bivariate copulas (n=2) are considered for the analysis, which belong to the class of Archimedean copula. These copulas find a wide range of applications for the following reasons: the ease with which they can be constructed, the great variety of families of copulas which belong to Archimedean class and the many nice properties possessed by the members of Archimedean class (Nelsen, 1999). In general, a bivariate Archimedean copula can be defined as (Nelsen, 1999):

$$C_\theta(u_1,u_2) = \phi^{-1}\{\phi(u_1) + \phi(u_2)\} \quad (4)$$

where subscript s of copula C is the parameter hidden of the generating function ϕ. ϕ is a continuous function, called generator, strictly decreasing and convex from I = [0,1] to [0, ϕ(0)] . Nelsen (1999) provided some important single-parameter families of Archimedean copulas (Table 4.1, pages 116-118), along with their generators, the range of the parameter, and some special and limiting cases. (Subhankar K. et al., 2007)

In the present study Gumbel-Hougaard family has been chosen. It is a one parameter Archimedean copula with generation function $\phi(t) = (-\ln(t))^\theta$ and $t = u_1$ or u_2:

$$C(u_1,u_2) = \exp(-((-\ln u_1)^\theta + (-\ln u_2)^\theta)^{\frac{1}{\theta}}) \quad (5)$$

The parameter θ synthesizes the dependence strength among the dependent random variables. In general, for each bivariate

Archimedean copulas, value of θ can be obtained by considering mathematical relationship (Nelsen, 1999) between Kendall's coefficient of correlation (τ) and generating function ϕ(t), which is given by

$\tau = 1 + 4\int_0^1 [\frac{\phi(t)}{\phi^{'}(t)}]dt$, where $t = u_1$ or u_2. The Kendall's coefficient of correlation (τ) is a well known nonparametric measure of dependence or association in theories of copulas, defined in terms of concordance as follows (Nelsen, 1999): Let { (x_1, y_1), (x_2, y_2), ... , (x_n, y_n)} denote a random sample of n observations from a vector (X, Y) of continuous random variables. There are $\binom{n}{2}$ distinct pairs (x_i, y_i) and (x_j, y_j) of observations in the sample, and each pair is either concordant (agreeing) or discordant - let 'c' denote the number of concordant pairs and 'd' the number of discordant pairs. Then Kendall's tau for the sample defined as the following expression:

$$\tau = [\frac{(c-d)}{c+d}] = \frac{(c-d)}{\binom{n}{2}} \quad (6)$$

where (x_i, y_i) and (x_j, y_j) are concordant if $[(x_i - x_j)(y_i - y_j)] > 0$, and discordant if $[(x_i - x_j)(y_i - y_j)] < 0$. The Kendall's tau from the observation is determined as (i.e. estimate of τ):

$$\tau_n = \binom{n}{2}^{-1} \sum_{i<j} sign[(x_i - x_j)(y_i - y_j)] \quad (7)$$

where *sign is equal to* 1 if $[(x_i - x_j)(y_i - y_j)] > 0$, and *sign is equal to* -1 when $[(x_i - x_j)(y_i - y_j)] < 0$. (Subhankar K. et al., 2007)

In the present study peak flows and volumes are fitted on the basis of the historical data by using two statistical distributions. The results, for both left and right catchment is Generalized Extreme Value (GEV) distribution as best marginal distribution for average peak flows and volumes. Then they have been correlated to each other using the Gumbel-Hoogard family, as it was found as the best fitted copula for rainfall intensity and duration (De Michele & Salvadori, 2003, Zhang & Singh, 2007). According to the non-parametric method, the procedure to calculate the generating function and the resulting copula was described by Genest and Favre (2007) and was followed. The copulas obtained for the analyzed case study, using the procedure above mentioned, is characterized by a parameter θ of the Gumbel-Hoogard copula equal to 3.219 and 3.566 respectively for left and right catchment. In order to indicate the goodness of fit of different copulas, the empirical joint distribution and best fitted copula based joint distribution were compared and to confirm the goodness of the chosen copula to describe the data the parametric and nonparametric values of the function θ, as defined by Genest and Favre (2007) are shown later in this paper. The goodness of fit is done using the k-plot. The historic data came from two raingauge stations which are Torregrotta and S.Pier Niceto.

Once copula is known, Monte Carlo generation of pairs of discharge and volume (Q_{max}, V_{max}) can be easily carried out. Particularly, two independent random variables, r_1 and r_2, both uniform on [0, 1] can be generate; it is possible to set $u_1 = r_1$ and $u_2 = S_u^{-1}(r_2)$ with:

$$S_u(r_2) = \frac{\partial C(u_1, u_2)}{\partial u_1} = \Pr\{U_2 \le u_2 / U_1 = u_1\} \tag{8}$$

From the pair (u_1, u_2), a pair (Q_{max}, V_{max}) can be obtained by setting $Q_{\max} = F_{Q_{\max}}^{-1}(u_1)$ and $V_{\max} = F_{V_{\max}}^{-1}(u_2)$

Given peaks Q_{max} and volumes V_{max}, the determination of flood hydrographs requires the knowledge of the shape to assign them. In this study flood hydrograph shapes were generated coping with non-dimensional or normalised hydrographs and then by averaging the equal one. However, the hydrographs are normalised so that a unit peak flow and a unit flood volume could result. The resulting nondimensional hydrographs can be merged with generated peak-volume pairs using copula for obtaining generated hydrographs.

For each flood event of the available series, non-dimensional hydrograph representing direct runoff was evaluated as follows (Apel et al., 2004; Dyck and Peschke, 1995). At first, there is no baseflow because in this case there is only a single event, so the direct peak flow Q_{max} and the volume V_{max} were determined; and then, the direct runoff was normalised by the following equations:

$$Q_{norm} = \frac{Q}{Q_{\max}} \quad \text{and} \quad t_{norm} = t\frac{Q_{\max}}{V_{\max}} \tag{9}$$

where t_{norm} = normalised time; and Q_{norm} = normalised direct runoff. Hence the non-dimensional hydrograph has peak flow and volume equal to 1.

In order to find typical shapes of the hydrographs, an average analysis was applied. In general, the 100 normalised hydrographs, which were obtained by the multivariate rainfall runoff model, were separated in three groups in accordance to the similarity in duration, hence an anticipated shape for longer duration, a delayed one for short duration and a centered one for the middle duration. In the end, a single one shape of hydrograph is chosen by average the three ones. Afterwards, the shape of the hydrograph and the 300 values of Qmax and Vmax generated using copula for left and right catchment are used to obtain the final input hydrographs for the hydraulic model. The equations (9) are used as follows:

$$Q = Q_{norm} Q_{\max} \quad \text{and} \quad t = t_{norm} \frac{V_{\max}}{Q_{\max}} \tag{10}$$

Hydraulic model-flood propagation

For flood propagation the MLFP-2D (Multi Level Flood Propagation 2-D) model (Aronica et al., 1998) was used. It is a hyperbolic model based on DSV equations when convective inertial terms are neglected. The conservative mass and momentum equations for two-dimensional shallow-water flow can be written as follows:

$$\frac{\partial \mathrm{H}}{\partial t} + \frac{\partial p}{\partial x} + \frac{\partial q}{\partial y} = 0 \tag{11a}$$

$$\frac{\partial p}{\partial t} + gh\frac{\partial H}{\partial x} + ghJ_x = 0; \frac{\partial q}{\partial t} + gh\frac{\partial H}{\partial y} + ghJ_y = 0 \tag{11b}$$

where H(t, x, y) is the free surface elevation; p(t, x, y) and q(t, x, y) are x- and y-components of the unit discharge; h is water depth; g the gravitational acceleration; and Jx and Jy are hydraulic resistances in the x- and y-directions.

The hydraulic resistances are parameterised by the Manning-Strickler formulation and can be expressed as:

$$J_x = \frac{n^2 p\sqrt{p^2 + q^2}}{h^{\frac{10}{3}}}; \qquad J_y = \frac{n^2 q\sqrt{p^2 + q^2}}{h^{\frac{10}{3}}} \tag{12}$$

where n ($m^{-1/3}$/ s) is the Manning’ s roughness factor.
Eqs. (12) were solved by using a finite element technique with triangular elements. The free surface elevation is assumed to be

continuous and piece-wise linear inside each element, where the unit discharges in the x and y directions are assumed to be piece-wise constant. (Aronica et al., 2011)

3. Case study

The study catchments are near to Barcellona Pozzo di Gotto in Sicily, Italy. Barcellona Pozzo di Gotto is located in the north coast of Sicily and belongs to the Province of Messina.

The area of the catchments is in the south part of the Longano River catchment. The two studied catchments are connected in the main Longano River in the junction near to the village Santa Venera Grotta. (Picture 2)This area is characterized by vegetation and small hills. The area of the left catchment is 15.92 Km^2 and the water from the longest point from the upper part of the river to the junction cover a distance of 9.9 Km , whereas the right catchment is 7.73 Km^2 and the longest point is 6.1 Km.

Figure 1: The above figure presents the study area with both left and right catchments and the junction in which they are connected to the main Longano River. In the picture is obvious the geomorphology of the area. (Picture taken by Google earth)

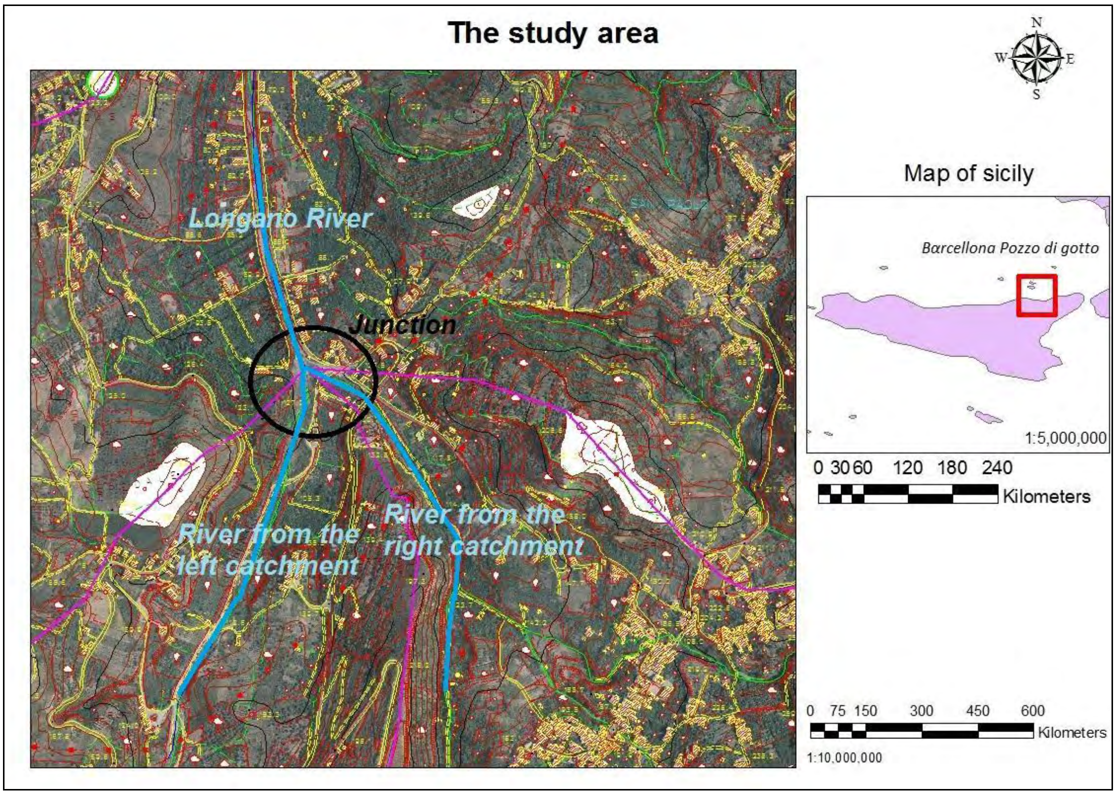


Figure 2: The study area is presented by a map made in GIS. The both catchments are well presented, as well as, the rivers and the junction in which are connected with the main Longano River. At the right part of the map there is also the map of Sicily in which the location of the studied area (Barcellona Pozzo di gotto) is visible.

There is a meteorological station in Barcellona Pozzo di Gotto. In this station, the observed data are collected in real time from the station Davis Vantage Pro 2. The data and the weather updated every minute. (http://www.barcellonameteo.altervista.org/index.php)

4. Results

Univariate approach

As mentioned in section 2, a univariate model of a 15-minute was used to derive the 300 pairs of Q_{max} and V_{max}. The chosen resolution of the input files of the r-r model were 10m, but were also done runs with resolutions of 20m and 100m. The results of the left and right catchment are presented in figure 3 and 4:

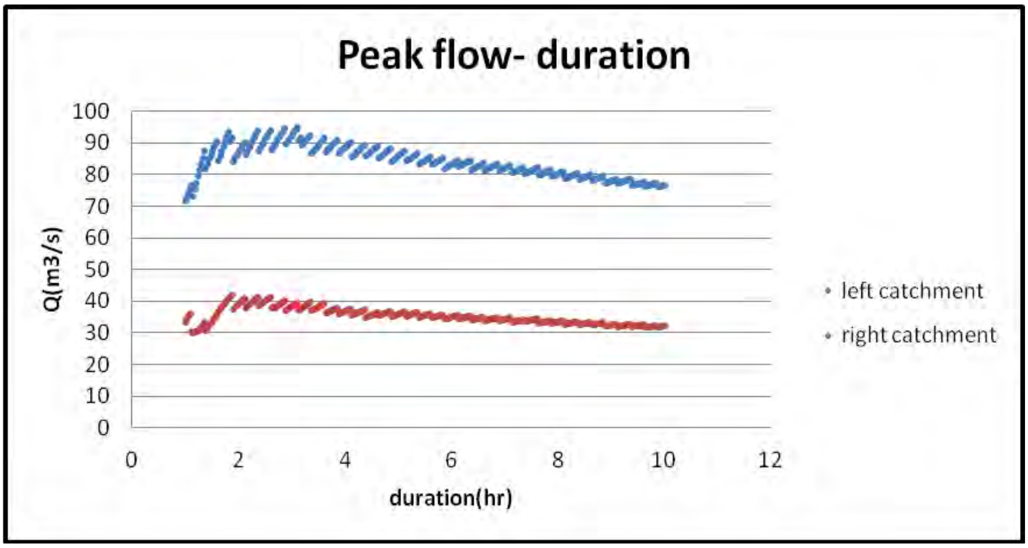


Figure 3: This graph presents the maximum values of peak flows of 300 hydrographs according to duration from 1h to 10h in the univariate approach, which were used as inputs in hydraulic model. The blue points present the left catchment and the red points the right catchment.

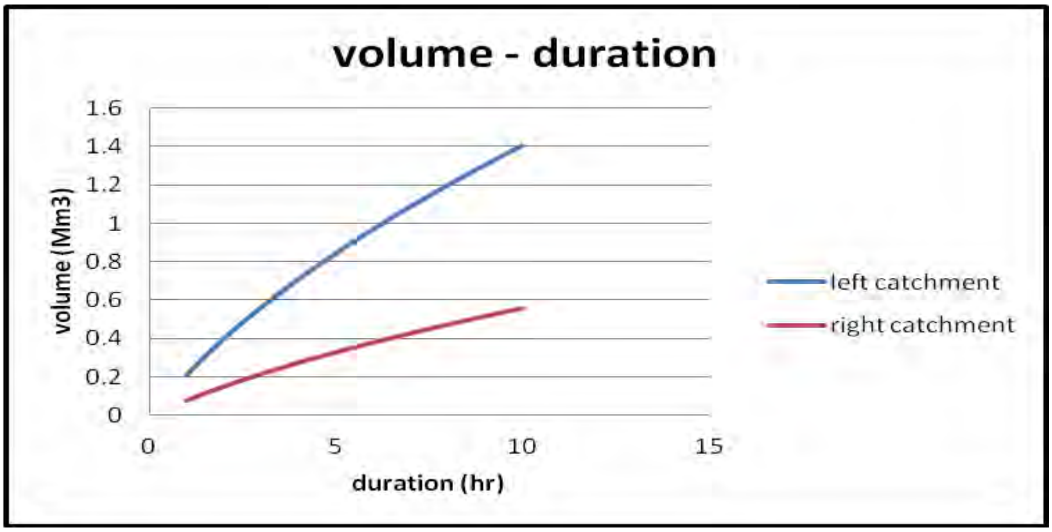


Figure 4: This graph presents the maximum volumes of 300 hydrographs in relation to duration from 1h to 10h in the univariate approach which were used as inputs in hydraulic model. In both catchments as the volume is bigger, the duration is larger. The blue curve presents the left catchment and the red curve the right catchment.

When someone size a flood reservoir must determine the optimal volume to control flood. Therefore, as a result from the univariate case, in both catchments the main observation is the optimal critical duration, which mainly is at about 3 hours. (Figure3) Despite the fact that the optimal duration was well known at the moment, it was impossible to size the reservoir for 100 year return period because as we mentioned before, the peak flows (Figure5) were come from the

100 year IDF curve but not the corresponding volumes. (Figure 6) There is the probability one of the 300 volumes to have 100 year return period but it is impossible to know which one. Therefore, the multivariate case was used. However, the results of the hydraulic model (Figure5&6) were also important for the comparison with the bivariate approach in order to accomplish the sizing of the reservoir.

An example of the hydraulic model was presented in figures 7 & 8 in order to show the area and the water depth. In the appendix there is also an example of an inundation map.

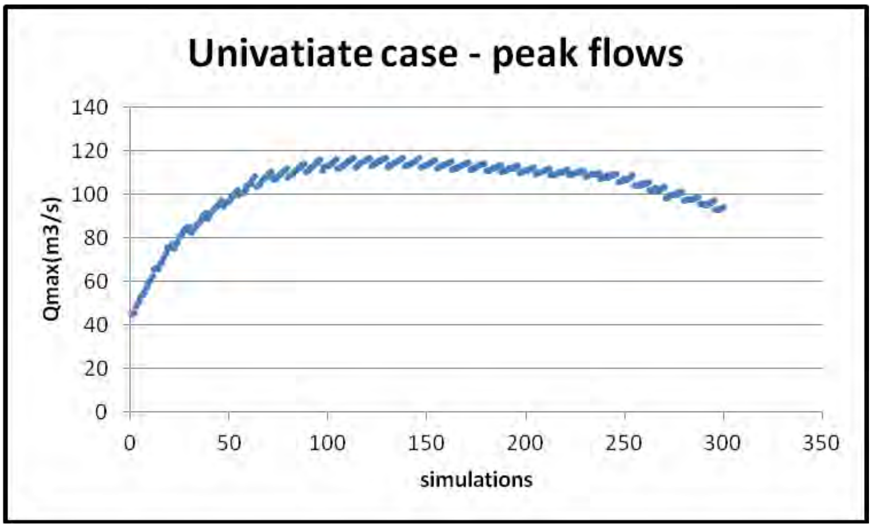


Figure 5: This graph presents the 300 max peak flows from the hydraulic model in the univariate approach of the 300 simulations. These peaks flows were come from 100 year IDF curve.

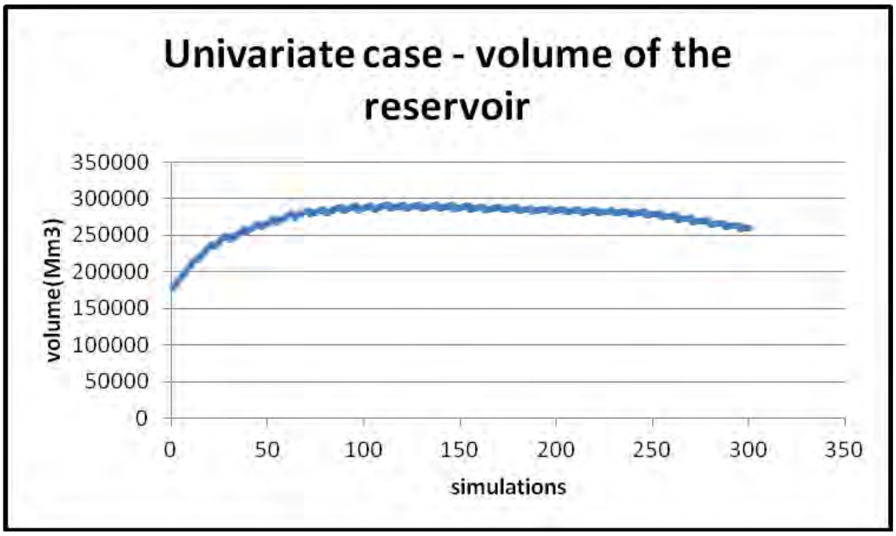


Figure 6: This graph presents the volume of the reservoirs of each simulation from the hydraulic model in the univariate approach of the 300 simulations.

- **Mesh model**

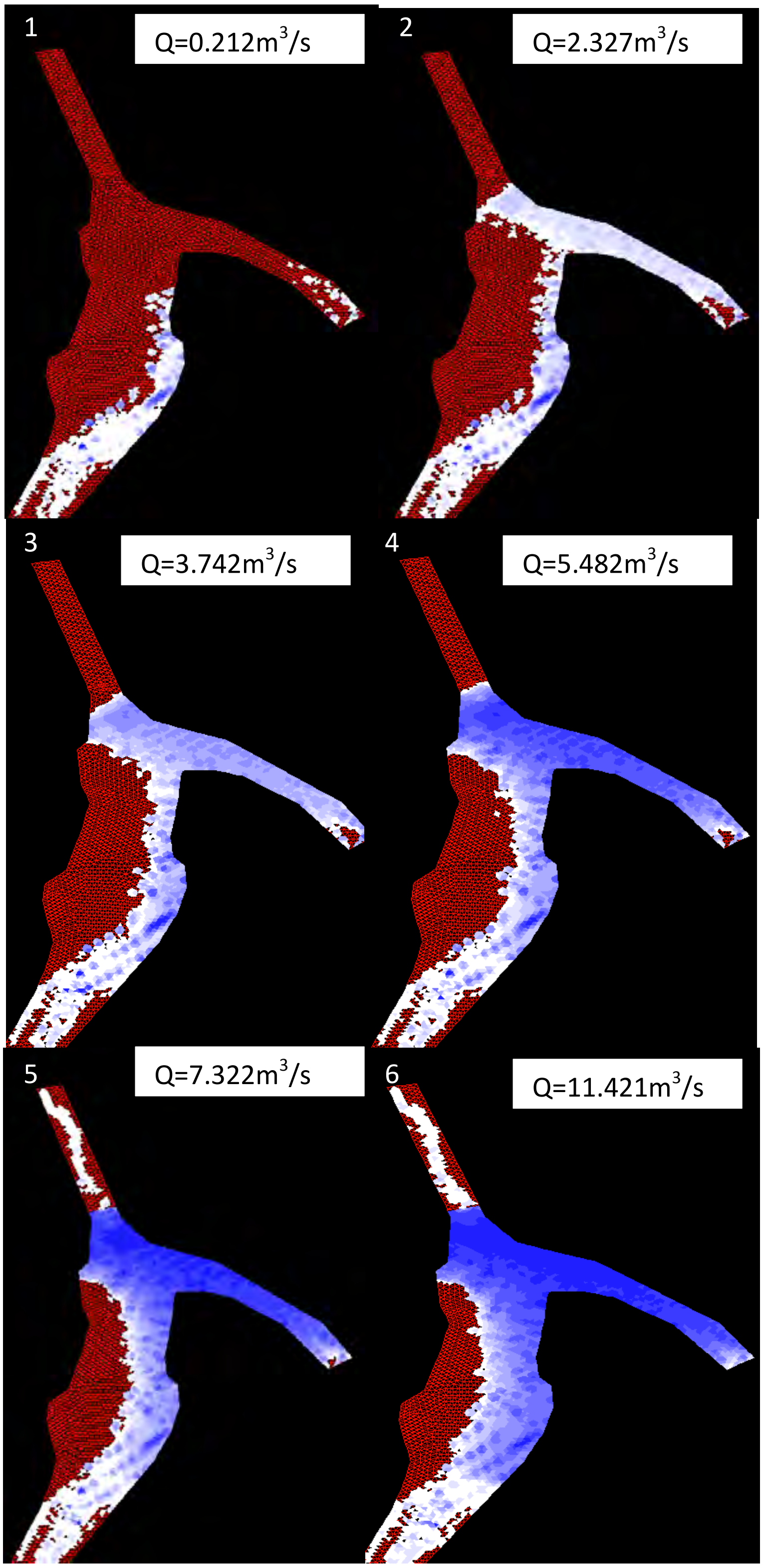


Figure 7: The above 6 figures were taken after a simulation of univariate case by the hydraulic model. As the flow continues increasing, the depth of the volume follows the same trend, hence it keeps increasing. The white color in the area of the model presents low height of water depth, whereas

the blue one presents the high values of the water depth. In this example, the flow of the both rivers reaches in the Longano River when is equal to $5.7m^3/s$ and the water depth equal to 0.6752m.

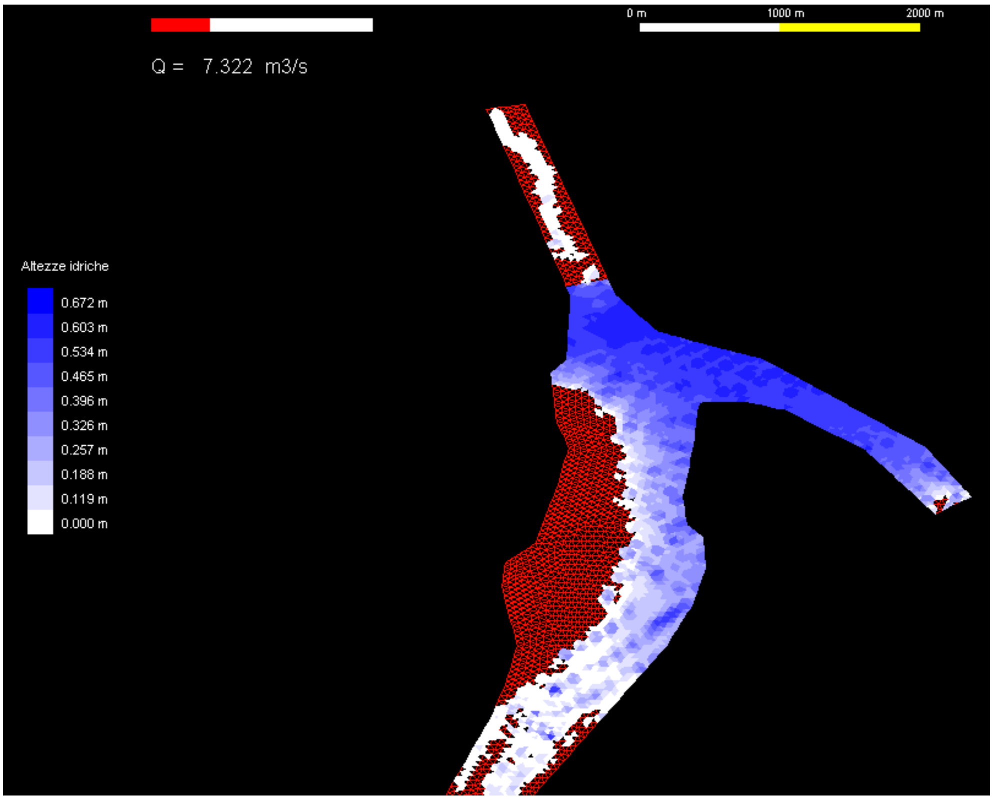


Figure 8: This graph gives a better representation of one simulation of the mesh model in the univariate approach.

Multivariate approach

Regarding the multivariate approach, a rainfall-runoff multivariate model was used to derive the 1000 pairs of Q_{max} and V_{max}. As mentioned in section 2 and also well known from the literature, more data are needed to fit a copula than to study a flood reservoir. The data that they were obtained by the multivariate rainfall-runoff model for both left and right catchment follow in figures 9 to 12. In this approach the duration is different in each peak flow and volume because the duration came from the copula and it was not fixed as in the univariate case. Moreover, the main observation was that the volume was more or less the same as in the univariate approach but the discharges were smaller in both cases. This happens because in this kind of approach the volume is the main target and not the discharge.

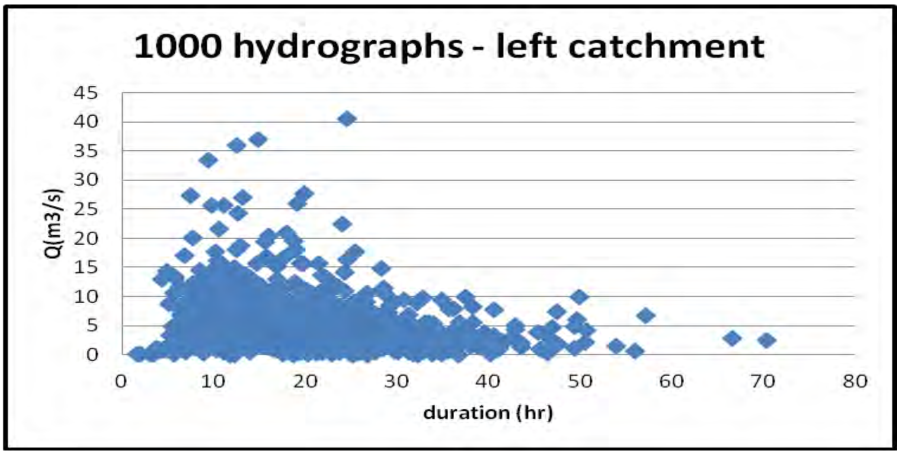


Figure 9: This graph presents the maximum values of peak flows of 1000 hydrographs according to random durations from 1h to 10h in the multivariate approach (left catchment).

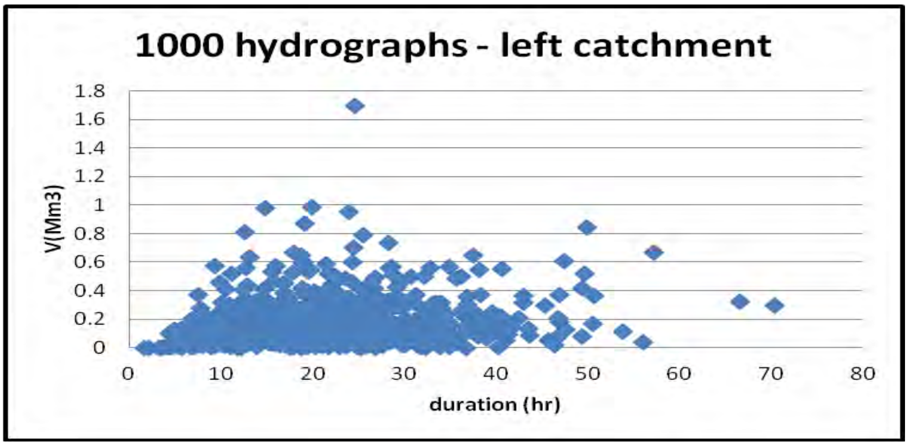


Figure 10: This graph presents the maximum values of volumes of 1000 hydrographs according to random durations from 1h to 10h in the multivariate approach (left catchment).

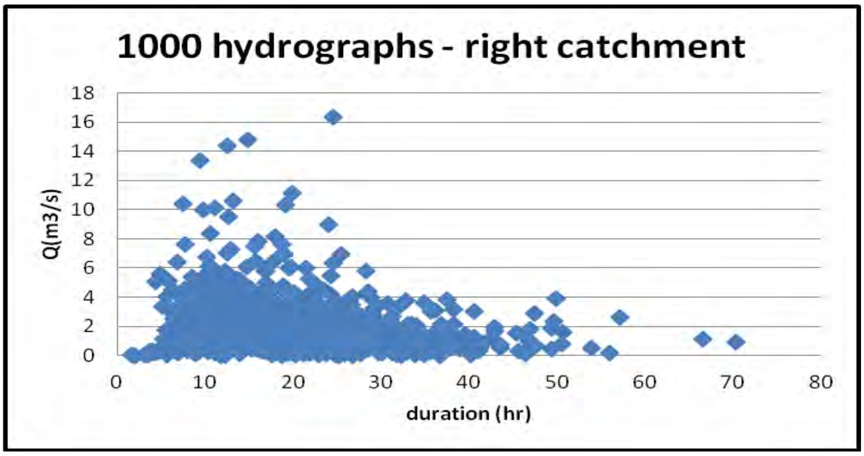


Figure 11: This graph presents the maximum values of peak flows of 1000 hydrographs according to random durations from 1h to 10h in the multivariate approach (right catchment).

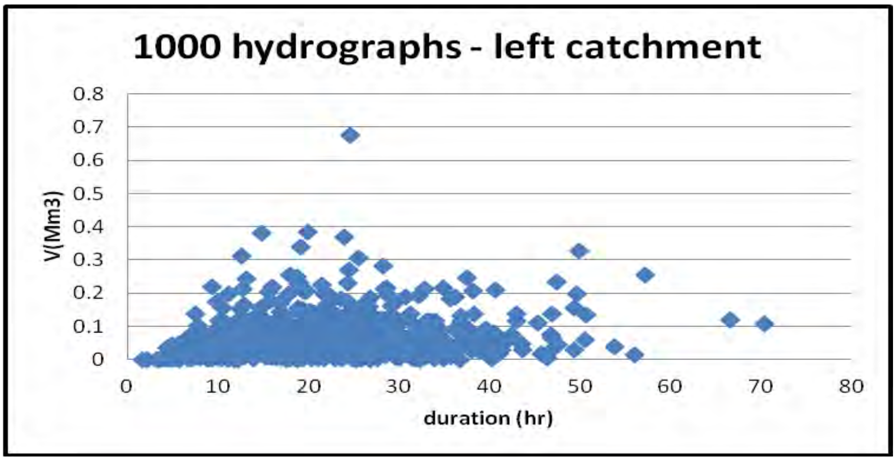
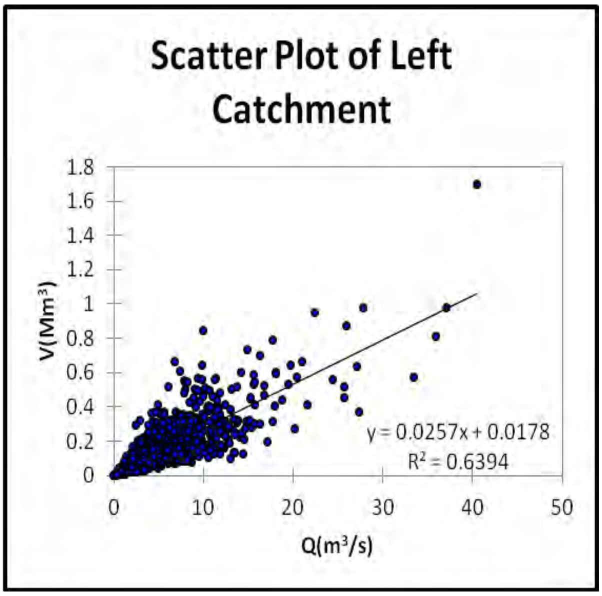


Figure 12: This graph presents the maximum values of volumes of 1000 hydrographs according to random durations from 1h to 10h in the multivariate approach (right catchment).

Further investigation of the correlation of the 1000 pairs was done. In the following figures 13 & 14, there are the scatter plot and the correlation map of Spearman's correlation rank test for both left and right catchment. Although in the scatter plot the R^2 wasn't approximately equal to 1, the correlation map of Spearman's test showed quite equal colors. According to the literature, in this kind of study, it is better to use Spearman's or Kendall's test.



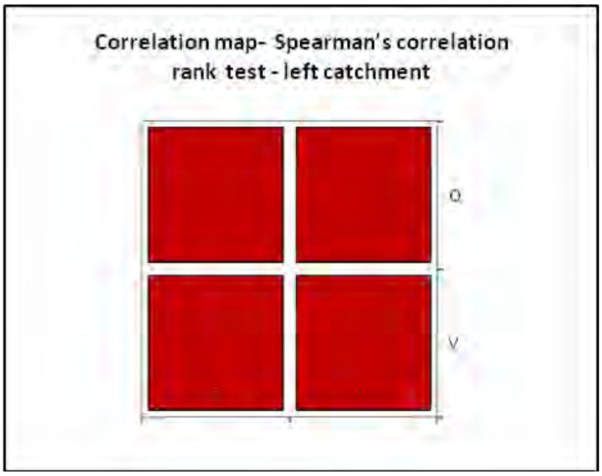


Figure 13: These two figures presents the correlation between the volumes and the peak flows after the 1000 runs of the left catchment. Despite the fact that the scatter plot didn't show high correlation, the Spearman's correlation map showed the opposite result.

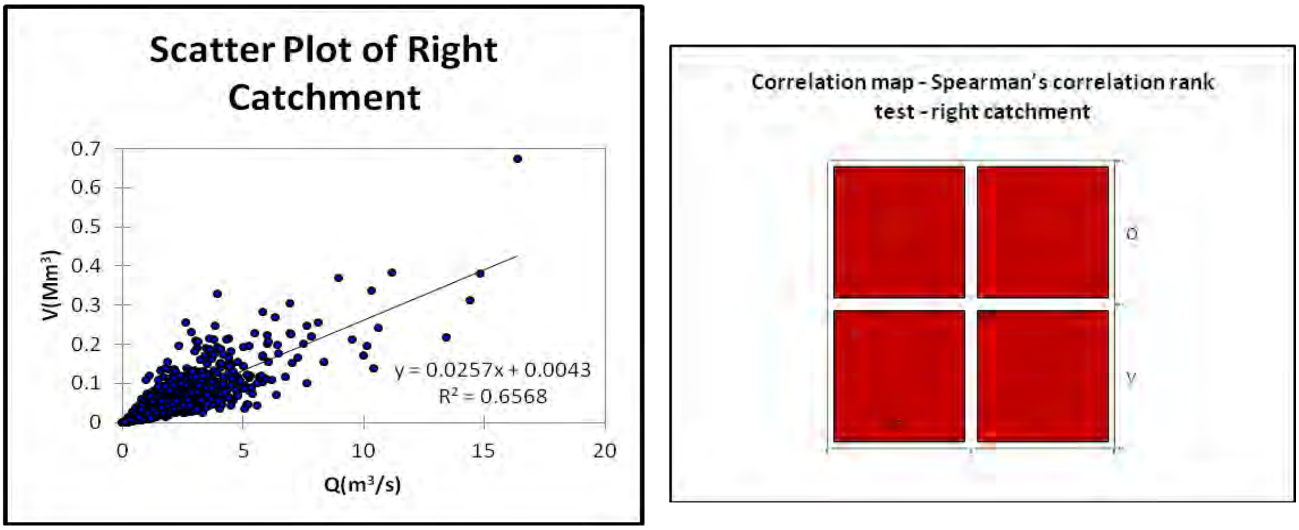


Figure 14: These two figures presents the correlation between the volumes and the peak flows after the 1000 runs of the right catchment. Despite the fact that the scatter plot didn't show high correlation, the Spearman's correlation map showed the opposite result.

The fitting of the marginals

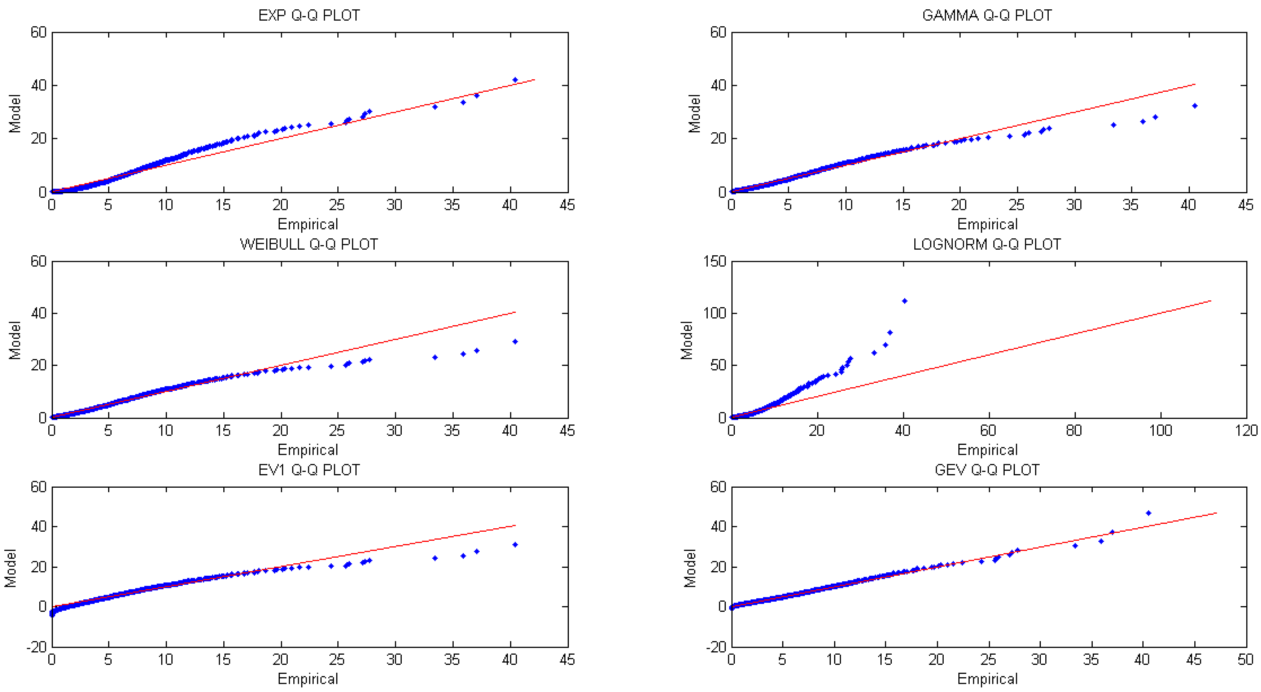


Figure 15: These graphs present the fitting of marginal (Q-Q plot) of the flow peaks of left catchment. The best distribution is GEV.

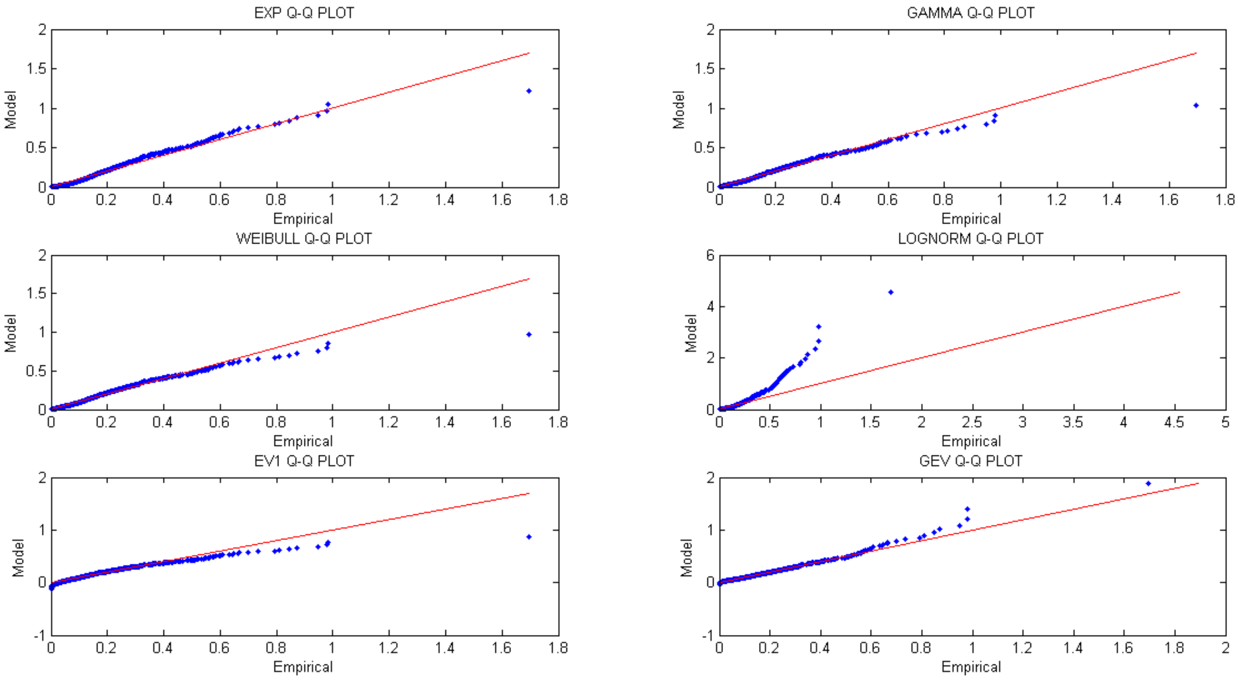


Figure 16: These graphs present the fitting of marginal (Q-Q plot) of the volumes of left catchment. The best distribution is GEV.

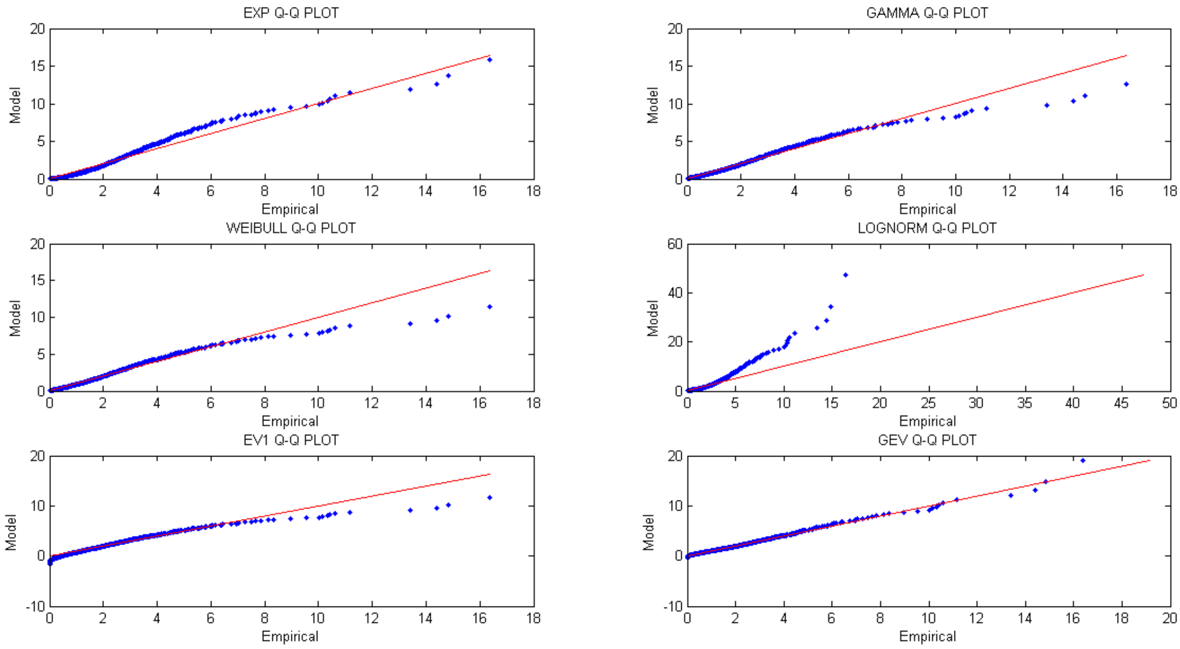


Figure 17: These graphs present the fitting of marginal (Q-Q plot) of the flow peaks of right catchment. The best distribution is GEV.

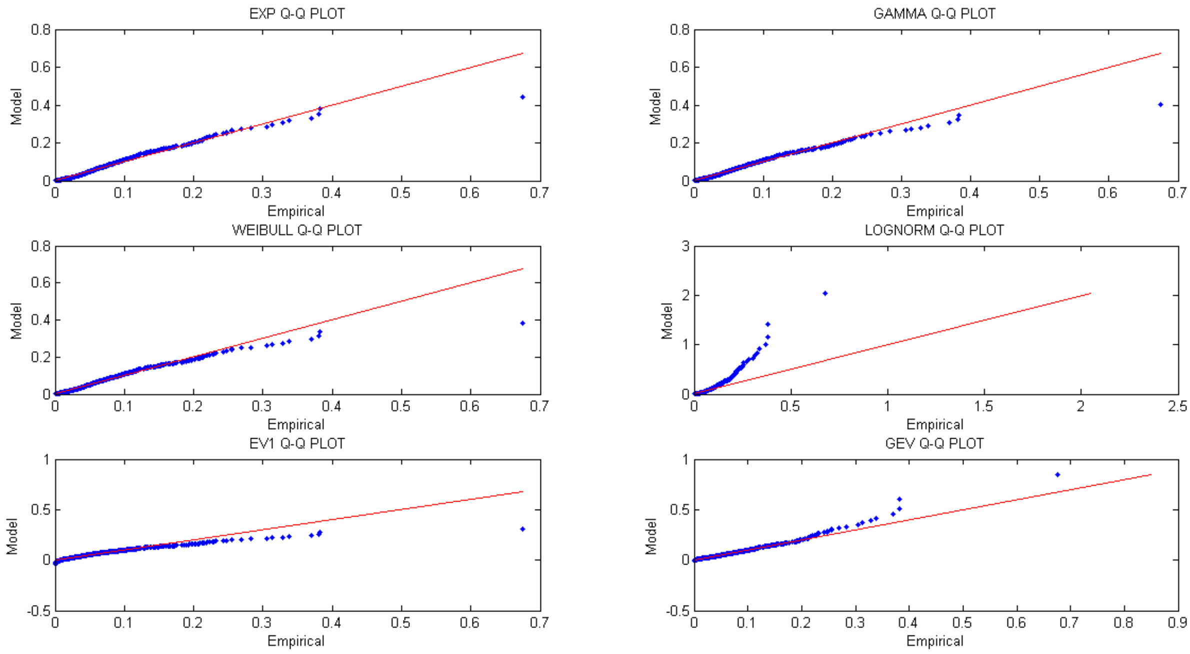


Figure 18: These graphs present the fitting of marginal (Q-Q plot) of the volumes of right catchment. The best distribution is GEV.

The fitting of the copulas

The discussion on the copula-based joint distributions in section 2 shows that, the Gumbel-Hougaard copula is found to be the best fitted copula for the joint distribution of peak flow-volume. (Figures 19 & 20)

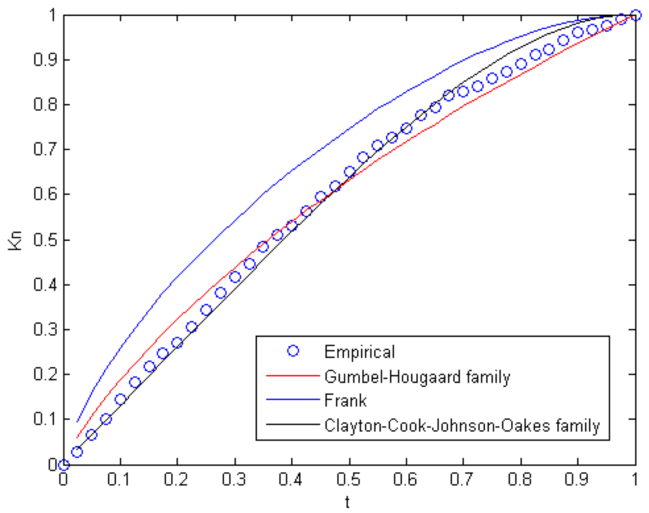


Figure 19: The graph was divided after the analysis and shows the fitting copula of the left catchment. It is obvious that the best one for this study is the Gumbel-Hougaard family.

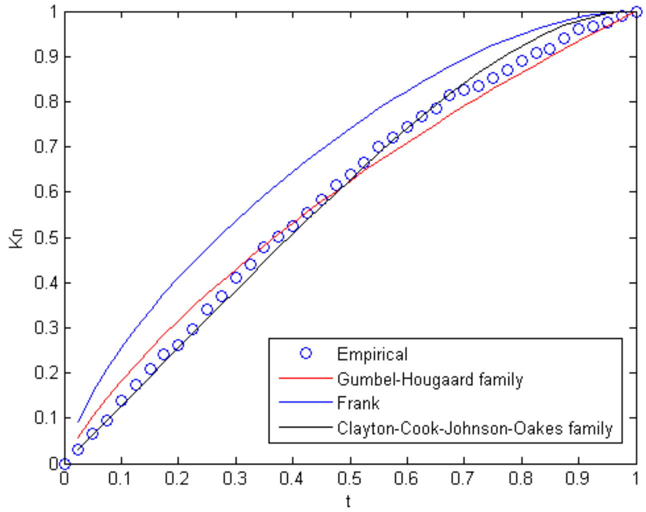


Figure 20: The graph was divided after the analysis and shows the fitting copula of the right catchment. It is obvious that the best one for this study is the Gumbel-Hougaard family.

In the present analysis, few particular Conditional Cumulative Distribution Functions (Conditional CDFs) and return periods are evaluated for the Gumbel-Hougaard model. For hydrologic design and planning purposes, given a flood event return period, it is possible to obtain various occurrence combinations of flood peaks, volumes and durations, and vice versa. It is also desirable in flood frequency analysis to obtain information concerning the occurrence probabilities of flood volumes under the condition that a given flood peak or duration occurs, and vice versa. (Subhankar et al., 2007)

In the present study it is decided to work with the values of the pairs (Qmax, Vmax) which were obtained by the 100 year recurrence interval. (Figures 21 & 22) The way how we obtained these 300 pairs of (Qmax, Vmax) was mentioned in section 2.

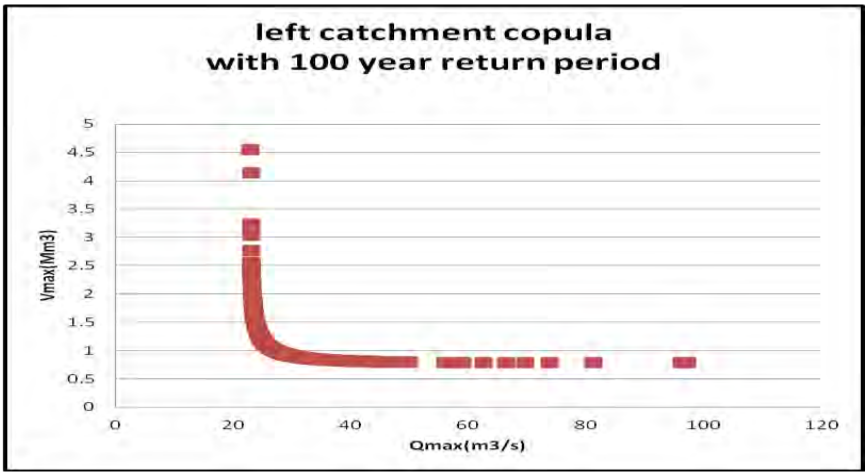


Figure 21: This graph presents the pairs that were obtained by copula in accordance to 100 return period. (Left catchment)

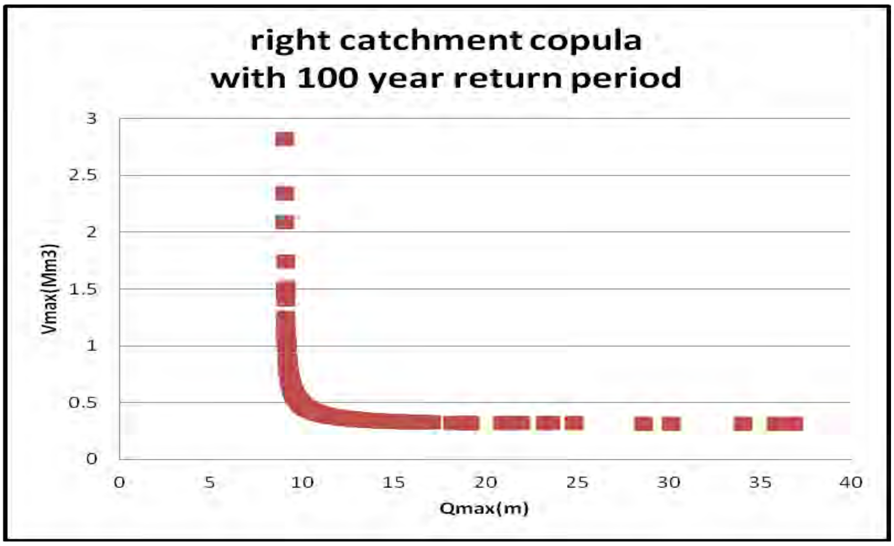


Figure 22: This graph presents the pairs that were obtained by copula in accordance to 100 return period. (Right catchment)

Once the copulas were obtained, hence the pairs of peaks Q_{max} and volumes V_{max}, the next step was to determine the shape of flood hydrographs to assign them. In this study flood hydrograph shapes were generated coping with non-dimensional or normalised hydrographs and then by averaging the equal one.

For each flood event of the time series, as mentioned before in the methodology, non-dimensional hydrograph representing direct runoff was evaluated as in the paper of Apel et al., 2004. Afterwards, in order to find typical shapes of the hydrographs, an average analysis was applied. In general, the 100 normalised hydrographs separated in three groups in accordance to the similarity in duration, hence an anticipated shape for longer duration, a delayed one for short duration and a centered one for the middle duration. Afterwards, a single one shape of hydrograph is chosen by the average of the three types. (Figures 23 & 24)

Finally, the shape of the hydrograph and the 300 values of Qmax and Vmax which were generated using copula for left and right catchment are used to obtain the final input hydrographs for the hydraulic model.

An example of the hydraulic model was presented in figure 27 in order to show the area and the water depth. In the appendix there is also an example of an inundation map.

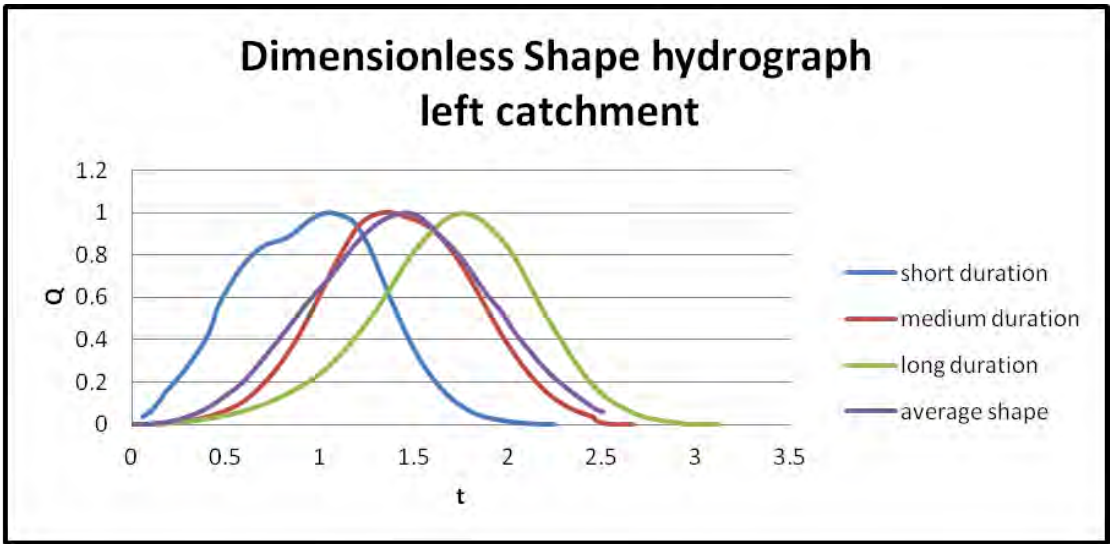


Figure 23: The three types of the 100 normalised hydrographs in accordance to the similarity of the duration. The blue color shows the short duration type, the red color shows the medium duration type and the green color shows the long duration type. The purple color indicates the average shape which was obtained by the three different shapes and was used in order to generate the 300 hydrographs for the hydraulic model. (Left catchment)

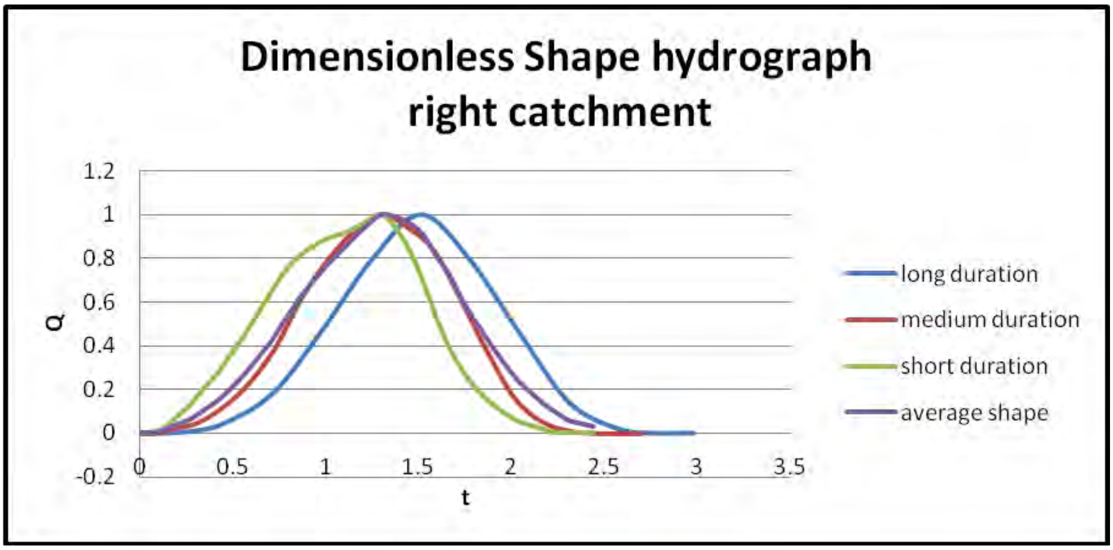


Figure 24: The three types of the 100 normalised hydrographs in accordance to the similarity of the duration. The blue color shows the long duration type, the red color shows the medium duration type and the green color shows the short duration type. The purple color indicates the average shape which was obtained by the three different types and was used in order to generate the 300 hydrographs for the hydraulic model. (Right catchment)

Mesh model

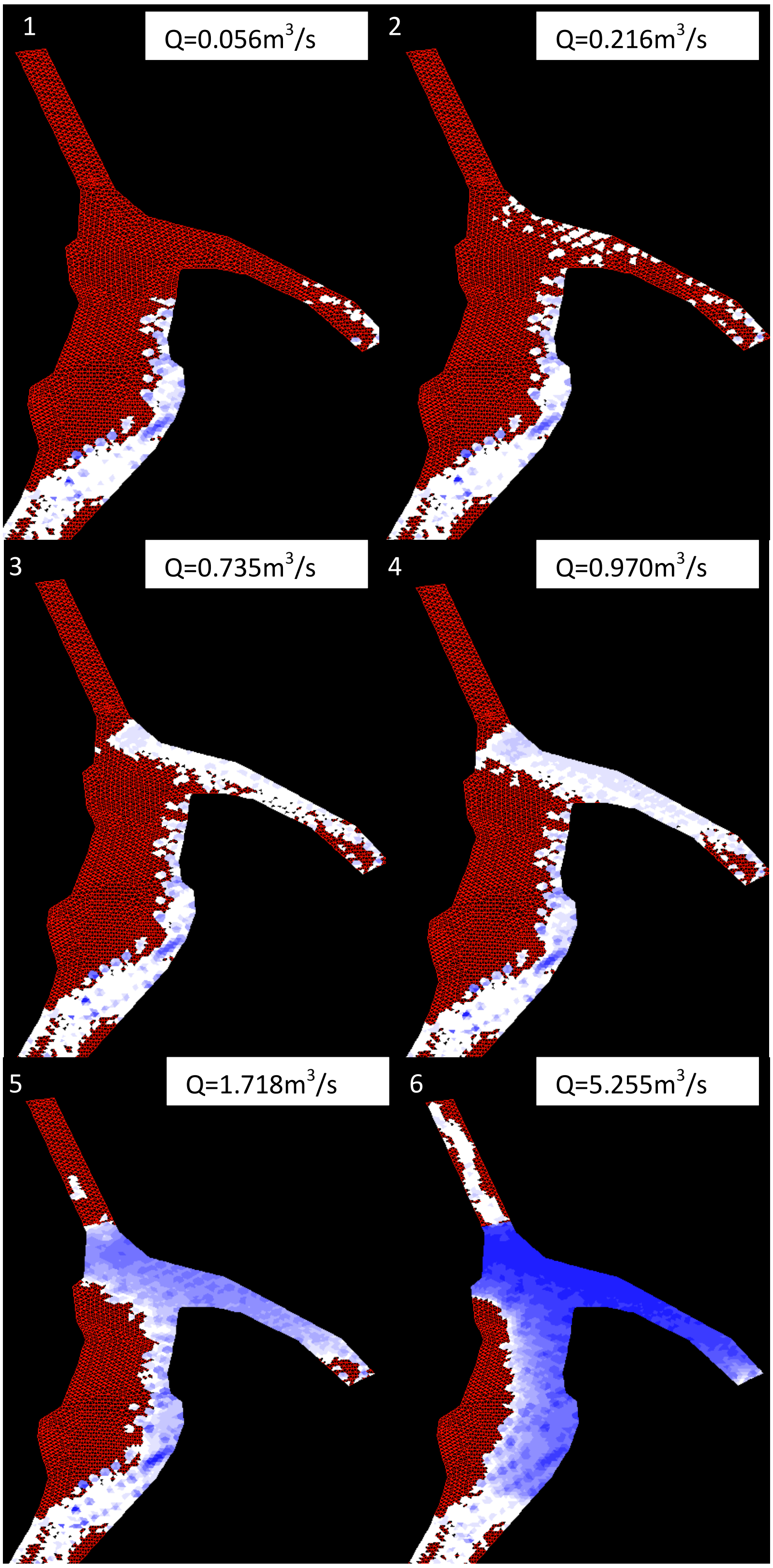


Figure 27: The above 6 figures were taken after a simulation of multivariate case by the hydraulic model. As the flow continues increasing, the depth of the volume follows the same trend, hence it keeps increasing.

The white color in the area of the model presents low height of water depth, whereas the blue color presents the high values of the water depth. In this precisely example the flow of the both rivers reaches in the Longano River when is equal to 1.718m^3/s and the water depth equal to 0.432m.

Sizing of the reservoir

In hydrological terms, sizing the reservoir means to find the "optimum" event to be used to have the "best" behavior of the flood reservoir. In the following figure 28, by comparing the input Qmax, the output Qmax and the volume one is able to size the reservoir. The main consequence was that the lower the efficiency, the higher the flood reduction.

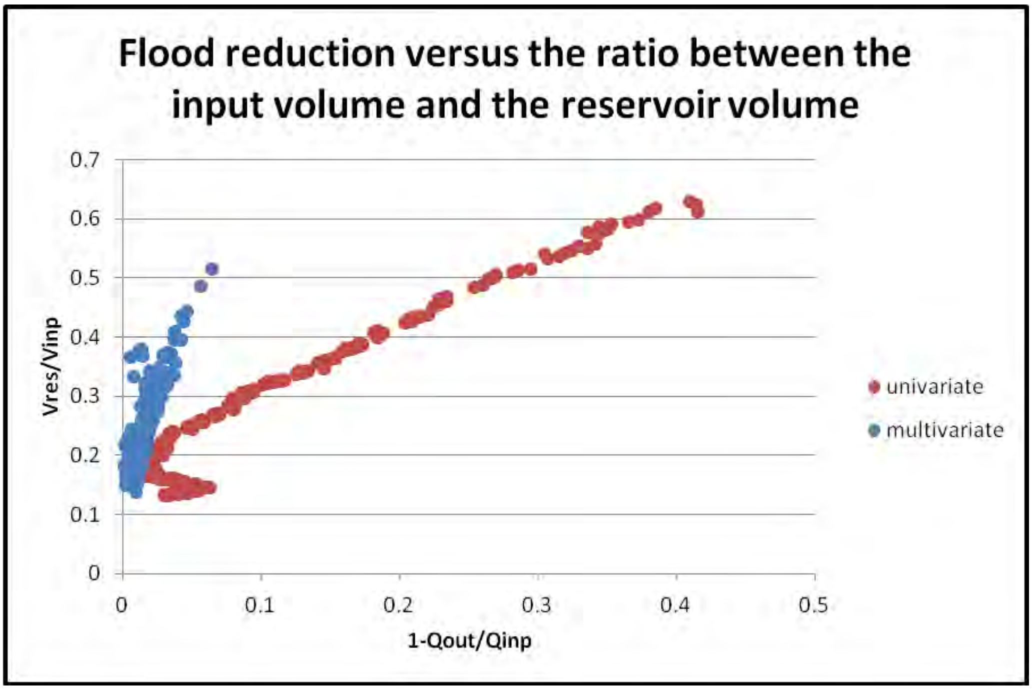


Figure 28: The above figure presents the flood reduction in accordance with the ratio between the input volume and the volume of the reservoir. The blue points are from the multivariate approach and the red points are from the univariate approach.

5. Conclusion and recommendation

Conclusion

In the present study were examined two methods of probabilistic sizing of flood reservoir in Barcellona Pozzo di Gotto, an area in the north part of Sicily. The first one was by using a univariate approach and the second was by using a multivariate approach – copula. In both cases the results were used as an input in a hydraulic model.

The input hydrographs were derived in the univariate case by a GEV distribution and in the multivariate case by a copula. In the multivariate approach, at first, the results showed for both left and right catchment that Generalized Extreme Value (GEV) distribution was the best marginal distribution for average peak flows and volumes. Subsequently, among 3 families of copulas that were fitted to the data of both catchments, the results showed that for both flow peaks and volumes only the Gumbel-Hougaard family yields an acceptable fit.

An interesting result was that the multivariate values were smaller than univariate, and there was also a correlation between Qmax and Vmax. The choice of the "critical" duration was easy to obtain by the univariate approach and it was at about 3 hours. Furthermore, in the multivariate approach it was surely that one had a ″real″ 100 year event, while in the univariate approach one had only the discharges at the 100 year return period by the rainfall – runoff model without the corresponding volumes.

Consequently, the difference between univariate and multivariate approach was that in the first one the actual problem in sizing of flood reservoir was to determine the optimal volume to control the flood, however the concept which we demonstrated from this study was that the definition of a critical value can lead to an overestimation, whereas in the multivariate approach overcame this.

To conclude with, the main result by the comparison of the both approaches in the latest figure showed that the lower the efficiency, the higher the flood reduction.

Further investigation –recommendation and more to study

Flood hazard assessment is the process of establishing the spatial extents of overall adverse effects caused by flooding of a particular area. It depends on a lot of parameters such as water depth, flow velocity, duration of flooding, product of water depth by flow velocity, rate of water rise, concentration of sediments or other transported materials, pollution load of water etc. (Abt et al., 1989; FEMA, 2002; Rosso, 2002; Kelman and Spence, 2004; Tingsanchali and Karim, 2005; Merz et al., 2007). In scientific literature several water depth-velocity hazard curves have been proposed (ACER Technical Memorandum, 1988; Penning-Rowsell and

Fordham, 1994; Staatscourant, 1998; Vrisou van Eck and Kok, 2001; Stephenson, 2002).

The former Italian Law 267/98 (1998) requires flood hazard and risk maps according to several probabilities: very low probability (return period in the range 20–50 years), medium probability (return period in the range 100–200 years), high probability (return period in the range 300–500 years). In particular, the Sicilian plan PAI (Regione Sicilia, 2004) set return period equal to 50, 100 and 300 years. Considering, the guidelines of the Sicilian plan refer to a four distinctive hazard classes (namely H1, H2, H3, H4) for these three different return times., new global hazard indexes are proposed. Those indexes are related to the depth of flooding (h) and flow velocity (w). Water depths of 0.3 and 1.5 m are limit related to the average stature of humans, values lower than 0.3 m give rise to low hazard level (H1) while values greater than 1.5 m can be considered giving rise to human instability and thus high hazard level (H3 or H4). Regarding velocity, the greater the magnitude, the grater is damage and risk for human life. Persons may be swept away from flow velocities above 0.5 m/s (Marco, 1994) but the product flow velocity and water depth is considered a better indicator. Another element plays an important role: the return time. In fact, higher hazard index could be associated with floods occurring frequently, while the hazard related to low probability (e.g. if the expected number of floods in 300 years is 1) may be tolerable.(Aronica et al. 2010)

In Barcellona Pozzo di Gotto, in 22 November of 2011 a flash flood event happened and the results were the death of some people and the destruction of a bridge. Therefore, the construction of a reservoir or dam in Longano river it will be a good way to avoid another event like that. Besides that, it will be also interesting to investigate the way of deal with a hazard assessing and mapping, depth of flooding and flow velocity, and indirectly the product of water depth by flow velocity.

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APPENDIX

An example of the inputs and outputs in the hydraulic model for both approaches

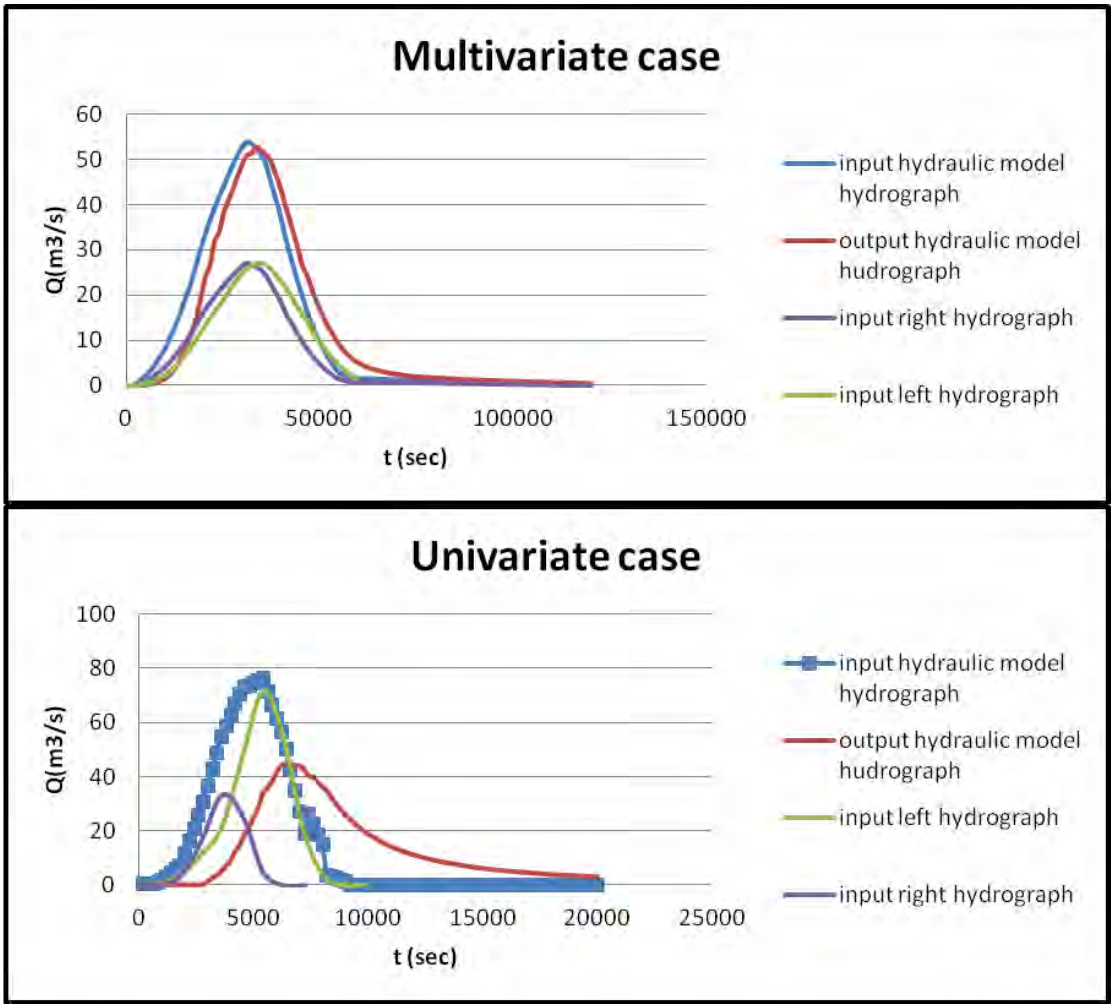

Figure 29: The graphs present the input hydrographs for the hydraulic model. In the multivariate case: The purple and the green curve present the input right and input left hydrograph respectively. As a sum of these two hydrographs and the input in the hydraulic model is the blue line and the output hydrograph of the hydraulic model is the red line. Correspondingly, in the univariate case the colors are the same. As a comment to the first simulation of both models, one can observe that there is a different lag time between the input and the output hydrograph.

<u>An example of an inundation map of both approaches</u>

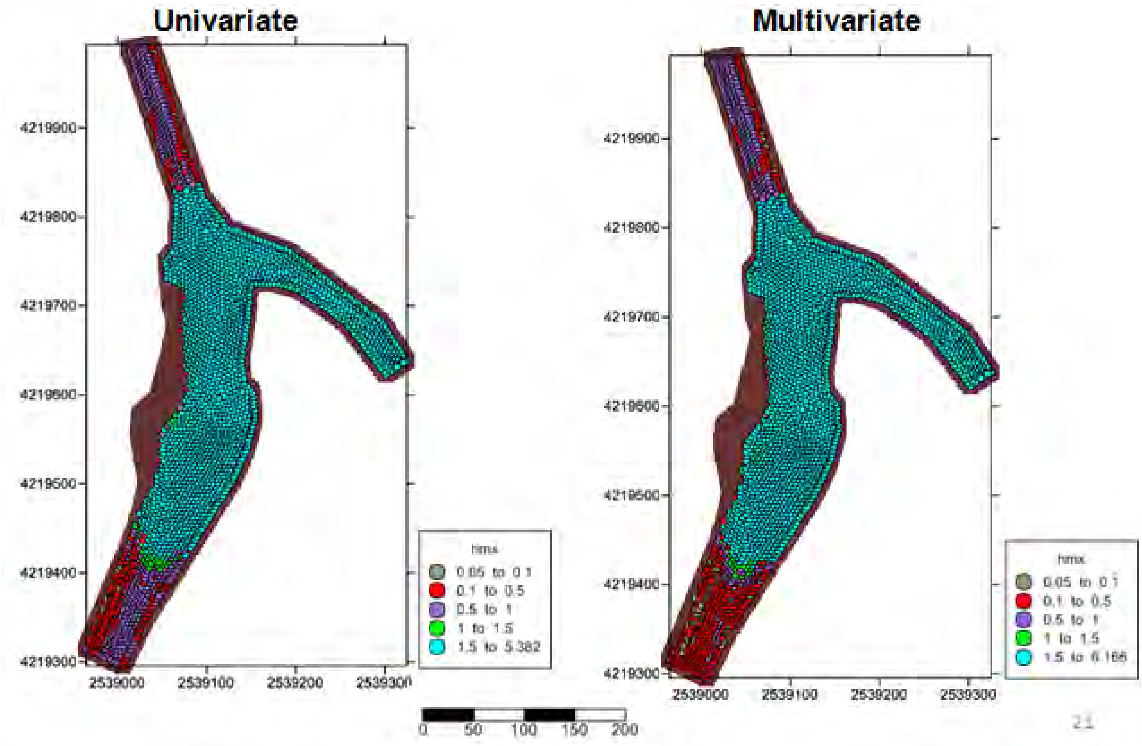

Figure 29: The above figures present a flood inundation map of the first simulation of the multivariate and univariate approach. The table which is in the right of the maps, there are the different intervals of the water depth. The difference is that in the multivariate approach the maximum water depth is bigger than in the univariate. The inundation area seems quite the same.

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