The Abdus Salam International Centre for Theoretical Physics

The CHyM hydrological model applied to the production of flood maps: a case study for the ALLIANZ Insurance Company

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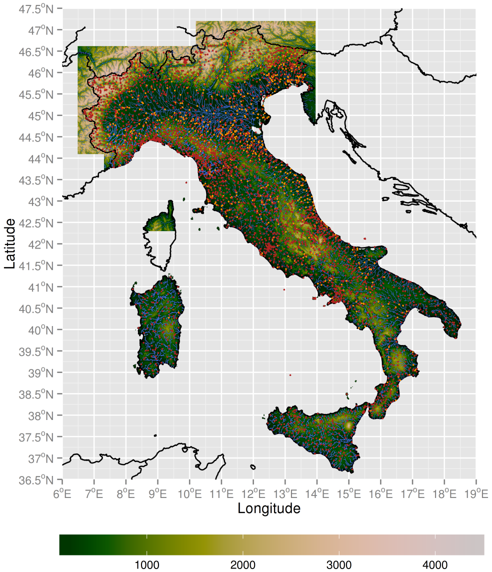
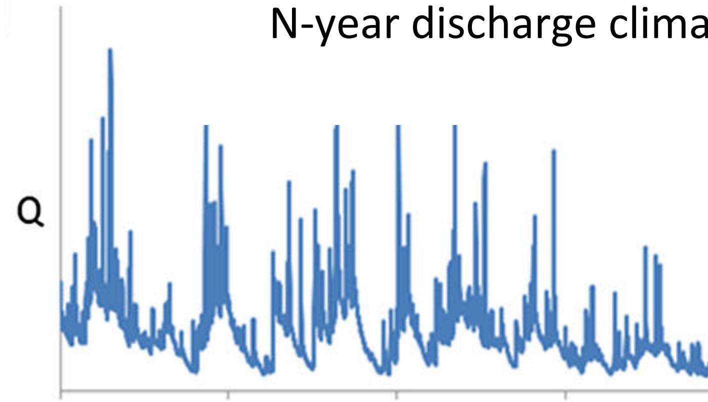
THE METHOD:
From the discharge climatology to the Flood hazard maps

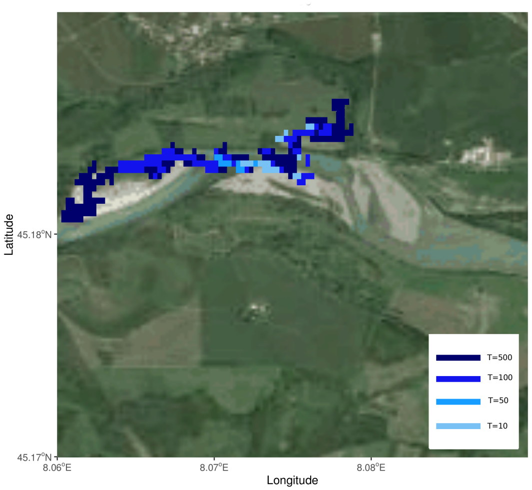


CHYM hydrological model or stations data

N-year discharge climatology



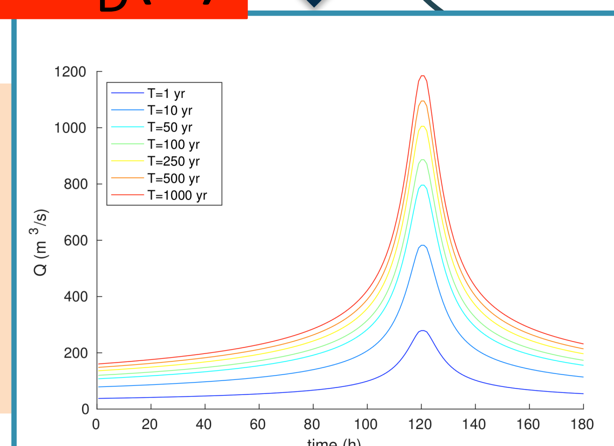
Flood hazard maps



$$Q_D = Q_D(T)$$

Statistical Flood Frequency analysis

Lisflood-ACC hydraulic model



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THE METHOD:
Statistical Flood Frequency analysis: why?

1. The maximum discharges of a river, can't be predictable and occur with remarkable variations in intensity, thus we need to define the feasible range of values that they can assume, through a statistical-probabilistic analysis on the base of OBSERVED (or MODELLED) DATA, so that the frequency of occurrence can be deduced.

   When speaking of flood events, the "frequency" is often expressed in terms of "RETURN PERIODS" = the probability that the event will be equalled or exceeded in any one year. This does not mean that a 100-year flood will happen regularly every 100 years, or only once in 100 years. Despite the connotations of the name "return period". In any given 100-year period, a 100-year event may occur once, twice, more or never.

Thus, the aim of the statistical analysis is the determination of the relationship:

$$Q_D = Q_D(T)$$

between discharges and return periods.

This is crucial in flood management (typical cases: definition of inundation maps and optimisation of flood plain management in view of risk mitigation) where the elements of interest are in the definition of hydrological risk are:

1. the peak discharge
2. the flood volume
3. the shape of the hydrograph (A **hydrograph** is a graph showing the rate of flow (discharge) versus time past a specific point in a river), that gives the information on when the peak would occur

Thus, the aim of the statistical analysis is the determination of the relationship:

$$Q_D = Q_D(T)$$

between discharges and return periods.

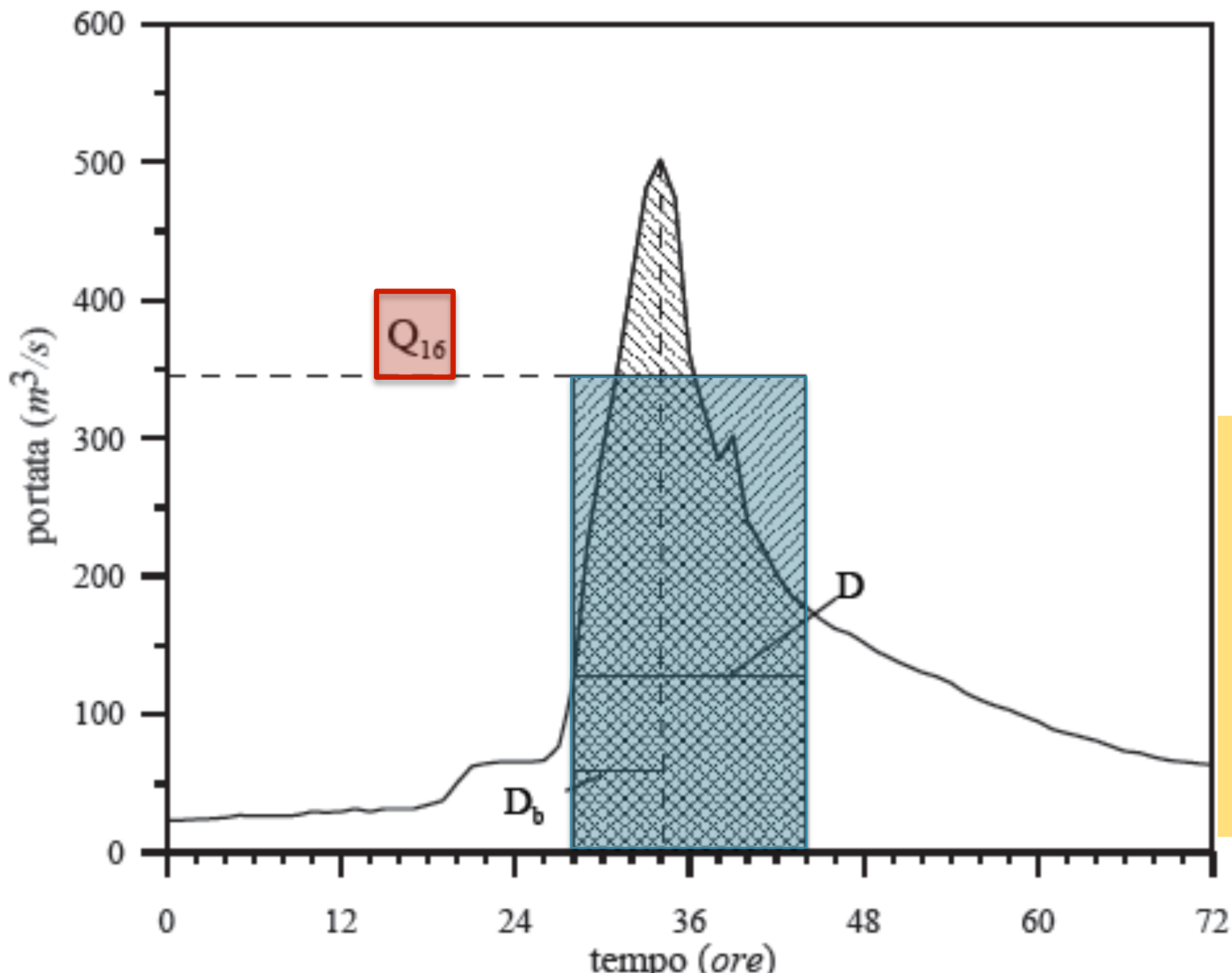
A possible solution is the formulation of a Synthetic Design Hydrograph (SDH)
(Maione et al., 2003)

The return period is the inverse of the probability that the event will be exceeded in any one year (or more accurately the inverse of the expected number of occurrences in a year). For example, a 10-year flood has a 1 / 10 = 0.1 or 10% chance of being exceeded in any one year and a 50-year flood has a 0.02 or 2% chance of being exceeded in any one year.

This does not mean that a 100-year flood will happen regularly every 100 years, or only once in 100 years. Despite the connotations of the name "return period". In any given 100-year period, a 100-year event may occur once, twice, more, or not at all, and each outcome has a probability that can be computed as below.

The construction of the SDH is based on the Flow Duration Frequency reduction curves (FDF) that can be obtained through the statistical analysis of historical hydrographs:

Data sampling of Q_D and r_D from an historical hydrographs (D=16):



$$Q_D = \max\left(\frac{1}{D}\int_{t-D}^{t} Q(\tau)d\tau\right)$$

$r_D = D_b/D$

the ratio between the time prior to the peak (in the time interval in which the maximum avarage discharge of given duration falls) and the duration D.

the annual maxima average discharges for each duration D are computed for each hydrograph and for all the durations ranging from 0 to D_f, representing the total duration of flood events for a given river site.

DON'T PANIC! Let's try to do an example..

	anno	0 ore	3 ore		12 ore		24 ore		36 ore	
		Q [m^3/s]	Q [m^3/s]	r	Q [m^3/s]	r	Q [m^3/s]	r	Q [m^3/s]	r
1	1956	104.9	104.2	0.167	92.8	0.292	73	0.229	60.7	0.167
2	1957	133.6	130.8	0.333	117.8	0.375	103.5	0.313	95	0.264
3	1958	206.3	181.5	0.167	119.4	0.208	94.3	0.521	88.3	0.861
4	1959	308.6	273.9	0.333	206.3	0.458	170.1	0.396	141.6	0.278
5	1960	450.5	430.2	0.333	318.9	0.417	247.5	0.896	235.6	0.708
6	1961	291.2	276.8	0.5	234.1	0.583	184.7	0.396	148.3	0.319
7	1962	189.4	186.6	0.667	180.6	0.208	161.1	0.021	158.8	0.097
8	1963	261.5	251.7	0.5	212	0.208	181.6	0.167	153.5	0.139
9	1964	211.7	210.2	0.333	180.3	0.333	145.4	0.313	126.4	0.333
10	1965	295.4	286.4	0.5	230.4	0.25	174.2	0.146	142.9	0.125
11	1966	501.8	477.9	0.5	353.1	0.417	259	0.25	207.7	0.194
12	1967	333.7	294.1	0.333	204.7	0.292	143	0.146	111.8	0.111
13	1968	368.9	335.1	0.167	259.4	0.208	204.6	0.271	170.4	0.375
14	1969	405.7	376.4	0.667	269.6	0.417	191.7	0.292	153	0.236
15	1970	194.5	184.2	0.333	165.2	0.583	144.9	0.438	140.3	0.264
16	1971	267.6	251.8	0.333	221.3	0.167	197.9	0.104	177.4	0.111
17	1972	278.4	273.2	0.667	250.3	0.542	202.1	0.396	164.8	0.292
18	1973	408.9	272.8	0.667	197.6	0.25	145.1	0.292	117.3	0.278
19	1974	340.4	319.7	0.333	279.2	0.458	216.6	0.375	178.7	0.292
20	1975	269.7	257.2	0.667	206.3	0.292	176.2	0.188	147.4	0.153

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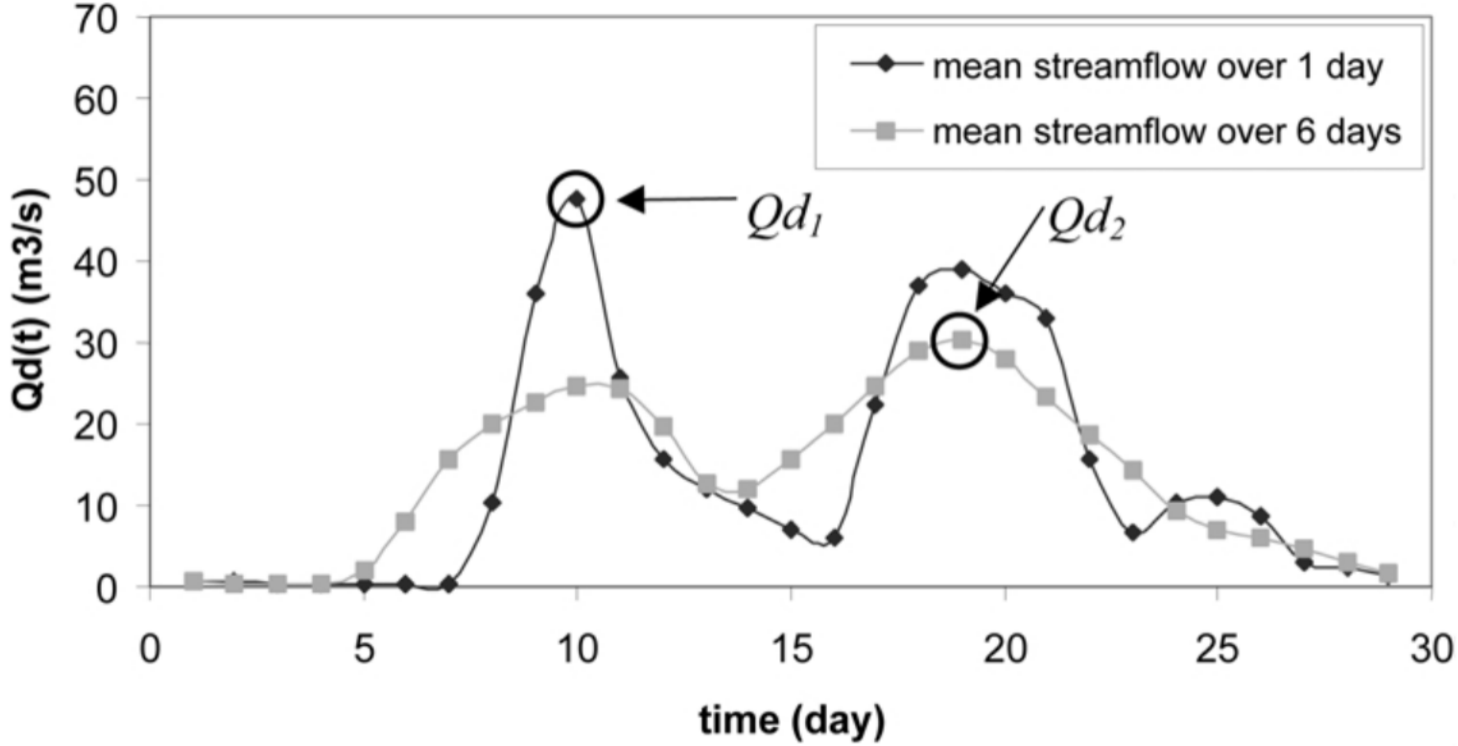

Fig. 1. Determination of the maximum mean streamflows *Qd*.

THE GOAL: $Q_D = Q_D(T)$

Following NERC (1975), let's consider this empirical relationship:

$$\varepsilon_D(T) = \frac{Q_D(T)}{Q_0(T)}$$

the maximum average discharges

the peak flood discharge

$$\varepsilon_D = \frac{\mu(Q_D)}{\mu(Q_0)}$$

Assumption (on the base of several studies in literature): the reduction ratio is independent of the return period T

Two possible approaches to identify the form of the reduction formula:

$$\varepsilon_D = \sqrt{\frac{\theta}{2D}\left[2 + e^{-\frac{4D}{\theta}} - \frac{3\theta}{4D}\left(1 - e^{-\frac{4D}{\theta}}\right)\right]}$$

(Bacchi et al., 1992)

$$\varepsilon_D = (1 + \beta D)^{-\gamma}$$

(NERC,1975)

Once estimated ε_D, the equation for the FDF curves becomes:
$Q_D(T) = Q_0(T)\, \varepsilon_D$, thus only the peak (maximum) flow discharge $Q_0(T)$ should be determined

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...	...	...	...	...	...	...	...	...	...	...
	media	313.29	280.12	0.40	209.95	0.33	161.44	0.30	138.15	0.25
	dev.st	197.39	165.98		117.72		84.38		81.17	
	c v	0.63	0.59		0.56		0.52		0.59	
	eps D	1.00	0.89		0.67		0.52		0.44	

Durate [h]	ε_D
0	1.00
3	0.89
12	0.67
24	0.52
36	0.44
48	0.39
72	0.32

$\varepsilon_3 = 280.12 / 313.29$

Durate [h]	r_D
0	
3	0.40
12	0.33
24	0.30
36	0.25
48	0.25
72	0.29

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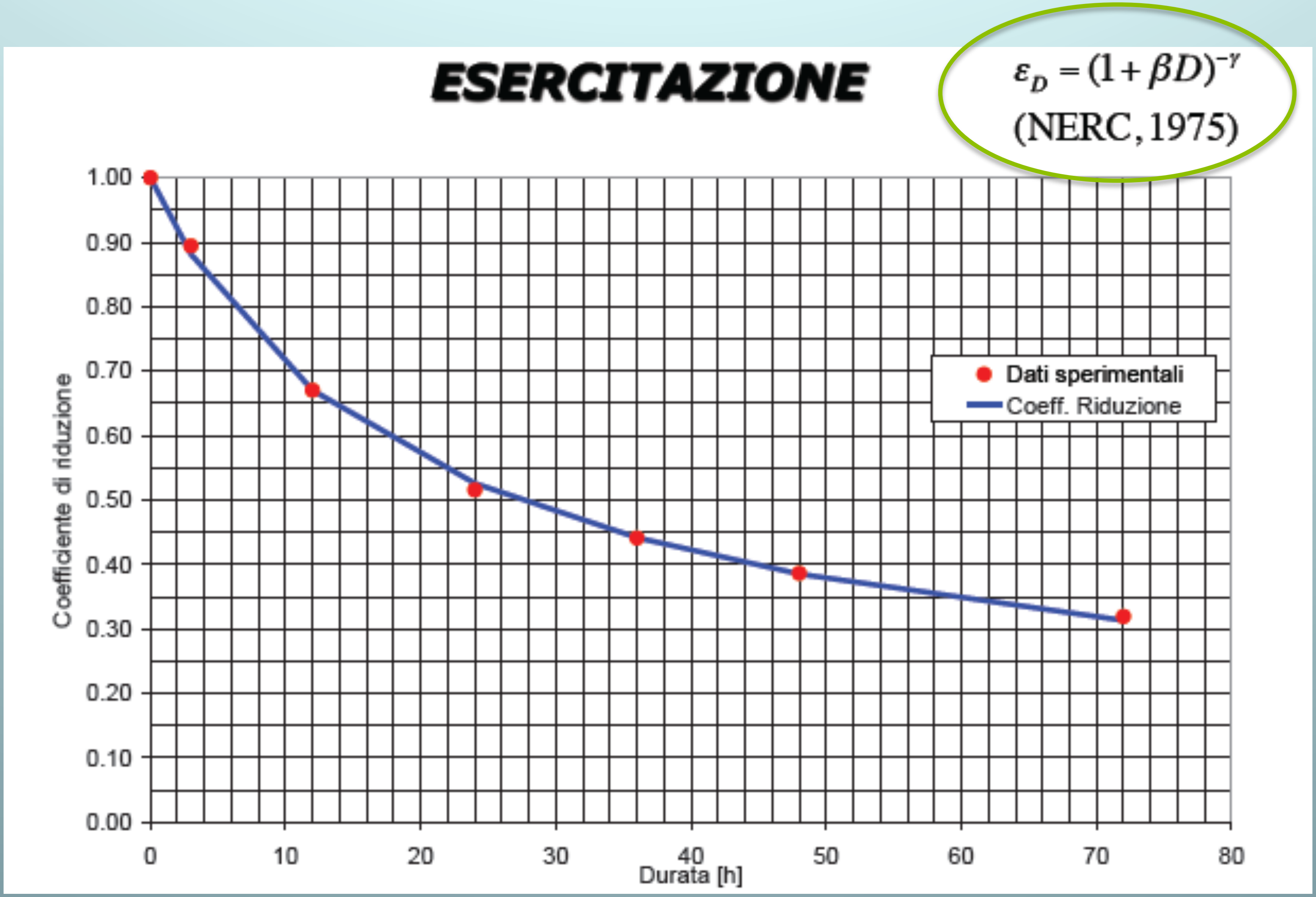
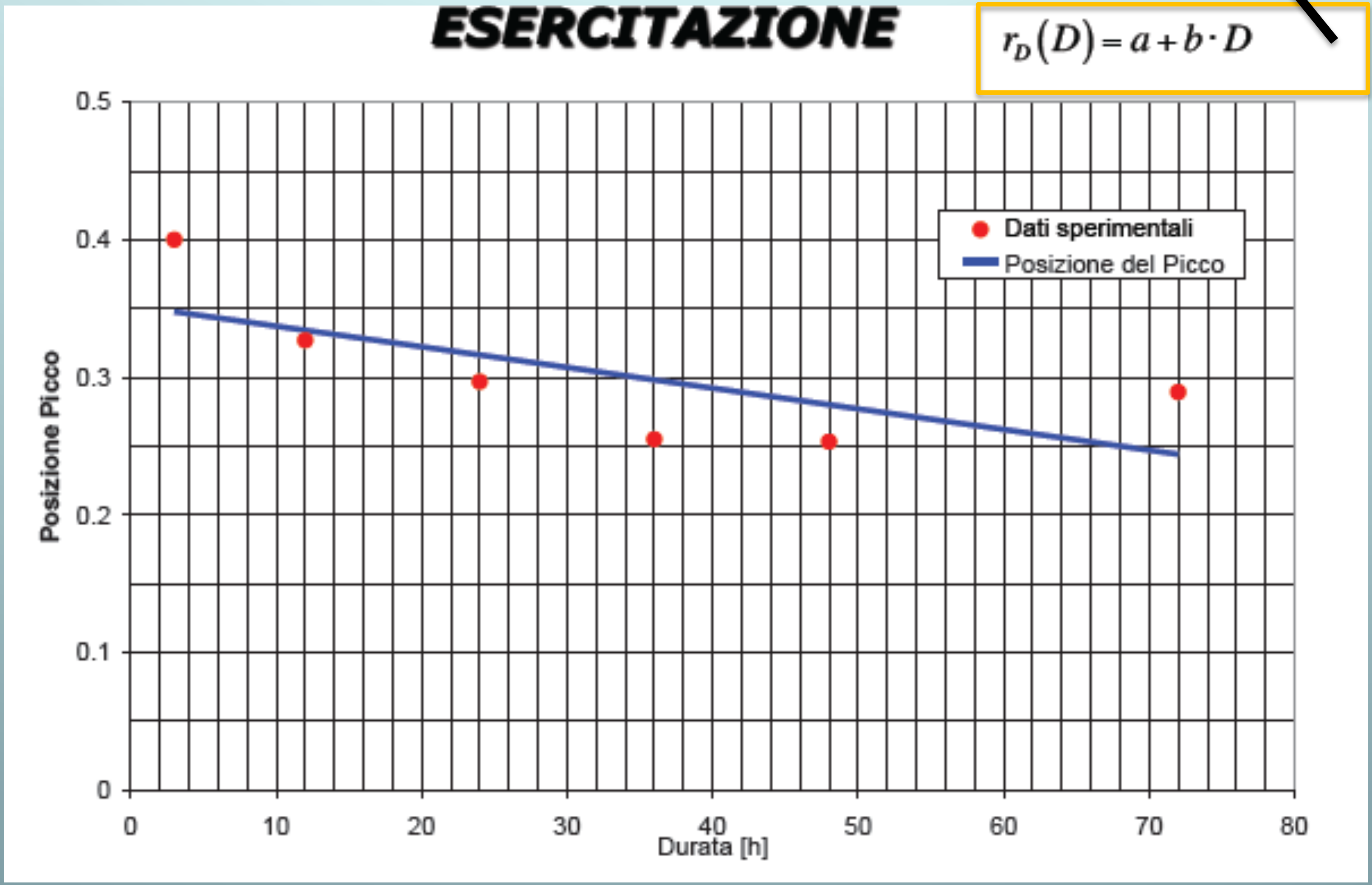


DON'T PANIC! Let's try

one of the possible functions that can well fit the values of r_D

$$r_D(D) = a + b \cdot D$$



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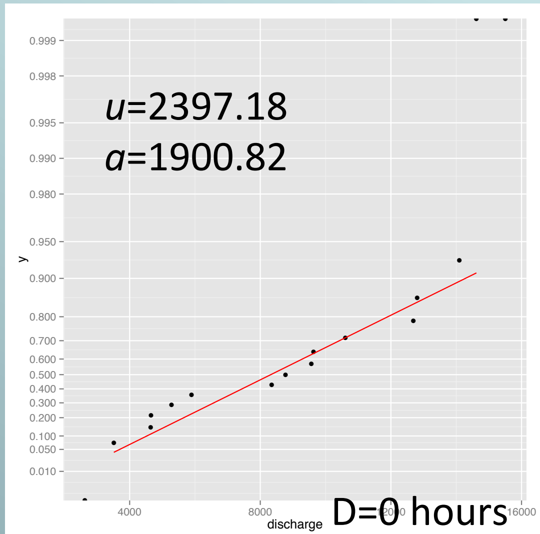
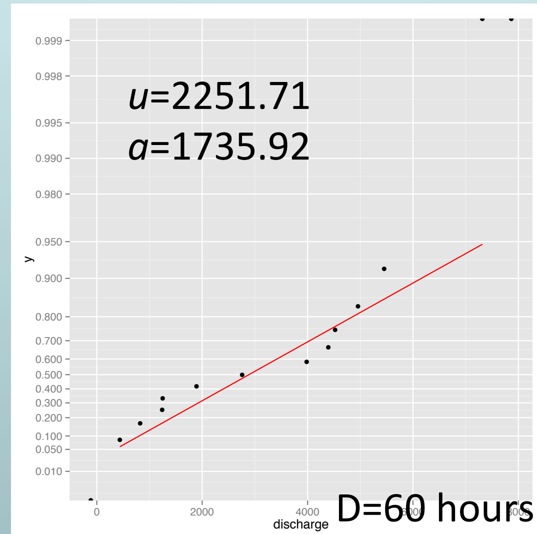
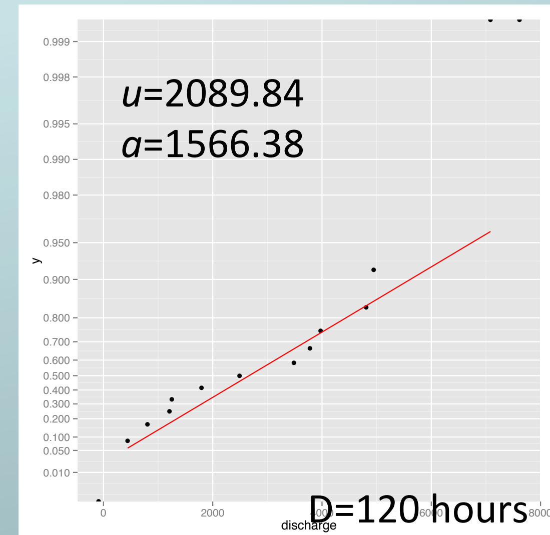
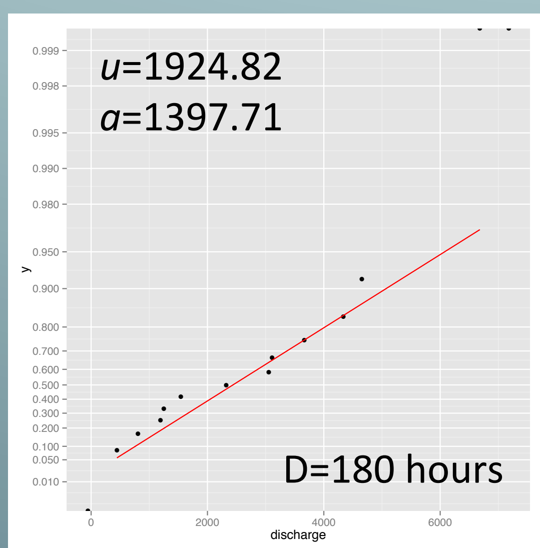
The Gumbel distribution is hypothesized as statistical distribution of the annual maxima of discharge (Beirlant et al., 2004) , so that the equation for the FDF curves is:

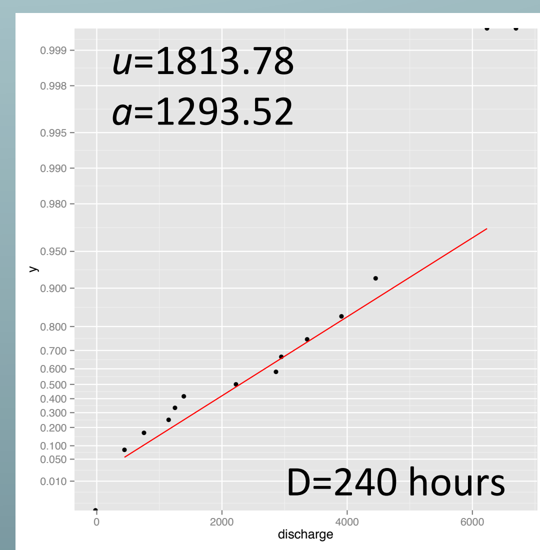
$$Q_D(T) = u - a \ln[-\ln(1-1/T)]$$











THE METHOD:
Statistical Flood Frequency analysis

The construction of the Synthetic Design Hydrographs (SDH) is performed imposing that the maximum average discharges for each duration coincides with the value obtained from the FDF curves, in a given duration D for each value of the return period T

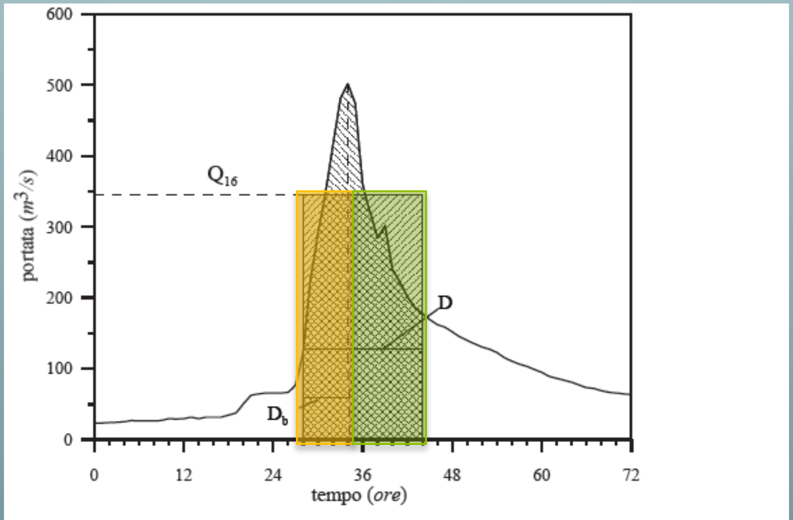
$r_D = D_b / D$

$$\int_{-r_D D}^{0} Q(\tau)\, d\tau = r_D Q_D D$$

the area BEFORE the peak

$$\int_{0}^{(1-r_D)D} Q(\tau)\, d\tau = (1 - r_D) Q_D D$$

the area AFTER the peak



The rising and the falling limbs of the SDH are obtained by differentiating both the equations with respect to the duration D as follows:

$$t = -r_D D$$

$$Q(t) = \frac{\left.\frac{d}{dD}\left(r_D D Q_D(T)\right)\right|_{D=D(t)}}{\left.\frac{d}{dD}\left(r_D D\right)\right|_{D=D(t)}}$$

$$t = (1 - r_D)D$$

$$Q(t) = \frac{\left.\frac{d}{dD}\left((1 - r_D) D Q_D(T)\right)\right|_{D=D(t)}}{\left.\frac{d}{dD}\left((1 - r_D) D\right)\right|_{D=D(t)}}$$

Before the peak: $t = -r_D D$

$$Q(t) = \frac{\left.\frac{d}{dD}\left(r_D D Q_D(T)\right)\right|_{D=D(t)}}{\left.\frac{d}{dD}\left(r_D D\right)\right|_{D=D(t)}}$$

$$\varepsilon_D = (1+\beta D)^{-\gamma}$$

$$r_D(D) = a + b \cdot D$$

$$Q_D(T) = \varepsilon_D(T) Q_0(T)$$

$$\frac{d}{dD}\left(r_D D Q_D(T)\right) = \frac{d}{dD}(r_D) \cdot D Q_D(T) + \frac{d}{dD}(D) \cdot r_D Q_D(T) + \frac{d}{dD}\left(Q_D(T)\right) \cdot r_D D =$$

$$= b \cdot D \varepsilon_D Q_0(T) + r_D \varepsilon_D Q_0(T) + \frac{d}{dD}\left(\varepsilon_D Q_0(T)\right) \cdot r_D D =$$

$$= Q_0(T)\left[b \cdot D \varepsilon_D + r_D \varepsilon_D - \gamma\beta\left(1+\beta D\right)^{-\gamma-1} \cdot r_D D\right]$$

the value obtained from the FDF curves

$$\frac{d}{dD}\left(r_D D\right) = \frac{d}{dD}(r_D) \cdot D + \frac{d}{dD}(D) \cdot r_D = bD + r_D$$

After the peak: $t = (1 - r_D)D$

$$Q(t) = \frac{\left.\frac{d}{dD}\left((1-r_D)DQ_D(T)\right)\right|_{D=D(t)}}{\left.\frac{d}{dD}\left((1-r_D)D\right)\right|_{D=D(t)}}$$

$$Q_D(T) = \varepsilon_D(T)Q_0(T)$$

$$\varepsilon_D = (1+\beta D)^{-\gamma}$$

$$r_D(D) = a + b \cdot D$$

$$\frac{d}{dD}\left((1-r_D)DQ_D(T)\right) = \frac{d}{dD}(1-r_D)\cdot DQ_D(T) +$$

$$+\frac{d}{dD}(D)\cdot(1-r_D)Q_D(T) + \frac{d}{dD}\left(Q_D(T)\right)\cdot(1-r_D)D =$$

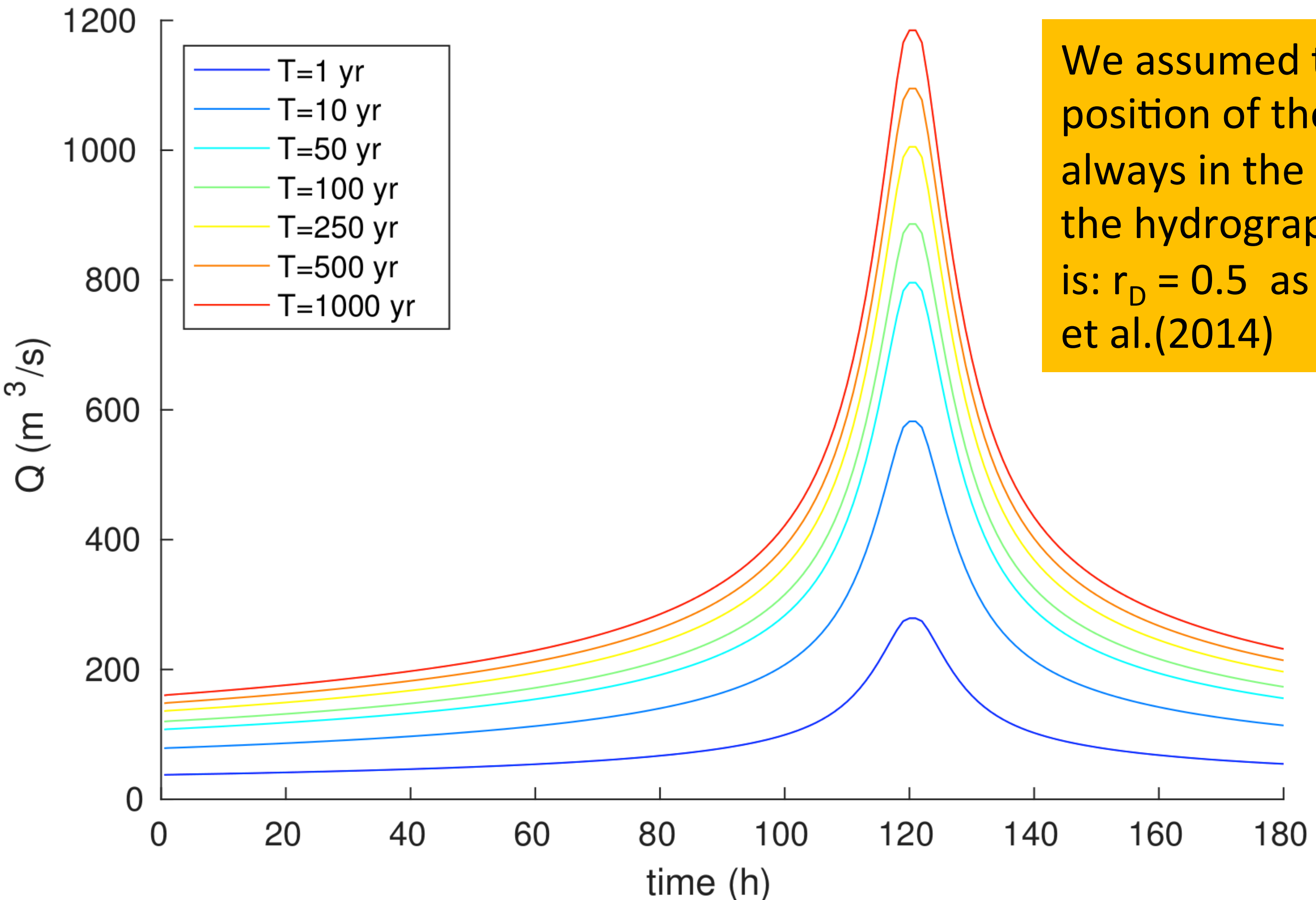
$$= -b\cdot D\varepsilon_D Q_0(T) + (1-r_D)\varepsilon_D Q_0(T) + \frac{d}{dD}\left(\varepsilon_D Q_0(T)\right)\cdot(1-r_D)D =$$

$$= Q_0(T)\left[-b\cdot D\varepsilon_D + (1-r_D)\varepsilon_D - \gamma\beta(1+\beta D)^{-\gamma-1}\cdot(1-r_D)D\right]$$

the value obtained from the FDF curves

$$\frac{d}{dD}\left((1-r_D)\cdot D\right) = \frac{d}{dD}(1-r_D)\cdot D + \frac{d}{dD}(D)\cdot(1-r_D) = -bD + (1-r_D)$$

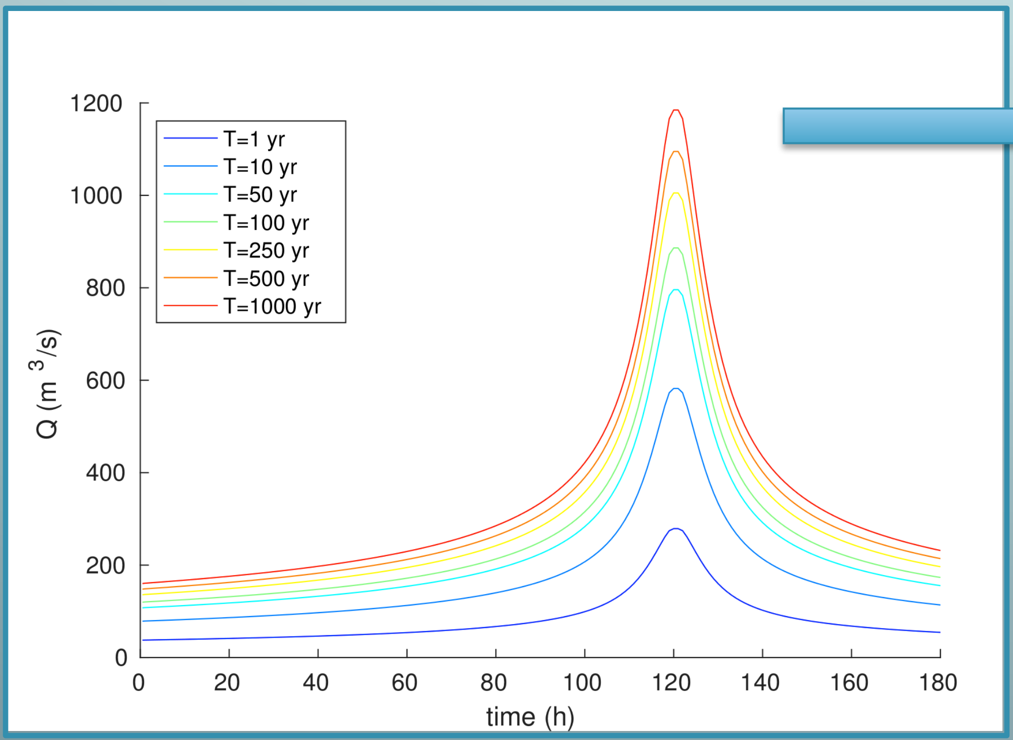
THE METHOD:
Synthetic Design Hydrographs

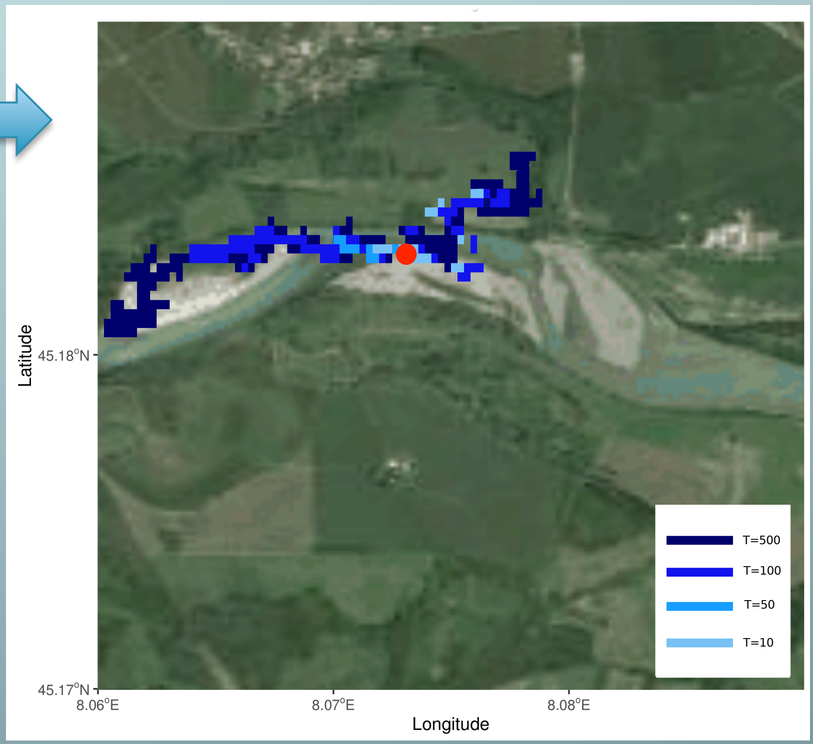


We assumed that the position of the peak is always in the centre of the hydrograph, that is: $r_D = 0.5$ as in Alfieri et al.(2014)

THE METHOD:
Lisflood-ACC hydraulic model

For each return period T, a SDH has been estimated and used as input data for the hydraulic model to predict the corrisponding maximum flood inundation extent and depth





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Thank you!