PROGRAM GEO – Piena ver.3

1. Theoretical basis

1.1 Average annual runoff coefficient of a drainage basin.

Average annual runoff coefficient (Cd) is defined as the ratio between the annual direct runoff in a channel, with refering to the outlet, and the total depth of rainfall recorded in the same period inside the drainage basin. Runof coefficient can be calculated for a single storm too and in this case it is defined as the ratio of the peak of the direct runoff to the average intensity of the rainfall.
Average annual runoff coefficient (Cd) can be assessed by Kennessey simplified method, mostly applicable to small drainage basin. It is bases on the assessment of 3 partial indexes, referred respectively to the average slope of the drainage basin (Ca), to its ground cover (Cb) and to the permeability of the shallow rocks or soils, which are, joined the climate data, the main factors determining the direct runoff.

- Average slope.

An higher average acclivity generally involves an higher direct runoff, reducing the ponding of the rainfall and, consequently, infiltration and evaporation.

- Gound cover.

A dense plant cover involves lower value of the runoff coefficient, both because of the water volume losses by transpiration of the plants and because plants obstuct the direct runoff, slow down it and then facilitate infiltration.

- Permeability.

An higher permeability facilitates infiltration of the rainfall, decreasing consequently the direct runoff.

- Climate data (precipitation and temperature) .

More than the annual values of precipitation and temperature, the annual runoff coefficient is influenced by their distribution during the year.

They can occur two extreme cases.

1. Maximum values of precipitation correspond to maximum values of temperature: in this case it occurs high evapotranspiration and strong reduction of the runoff coefficient.
2. Maximum values of precipitation correspond to minimum values of temperature: in this case it occurs low evapotranspiration and high direct runoff.

An assessment of the influence of the climate factors upon the Cd value can be gotten through the Aridity Index, defined as:

$$Ia = [\ P / (T+10) + 12 \times p / t] / 2$$

where : P = average annual precipitation;
T = average annual temperature;
p and t = precipitation and temperature of the driest month of the year.

Value of Ia gets higher to the increasing both the ratio between annual total precipitation to average annuale temperature and the ratio between precipitation of the rainest month the corresponding monthly temperature. So it has to be generally expected, with same temperature, an higher direct runoff related to an increasing of the rainfall depth and, with same precipitation, an increasing of Cd related to a decreasing of temperature.
Kennessey method defines three ranges of Ia values, each corresponding to a different set of partial runoff coefficients.

Coefficient	Value	Ia < 25	25 ≤ Ia ≤ 40	Ia > 40
Ca-slope	> 35%	0.22	0.26	0.30
	10 - 35	0.12	0.16	0.20
	3.5 - 10	0.01	0.03	0.05
	< 3.5	0.00	0.01	0.03
Cp-permeability	Very low	0.21	0.26	0.30
	Low	0.17	0.21	0.25
	Medium	0.12	0.16	0.20
	High	0.06	0.08	0.10
	Very high	0.03	0.04	0.05
Cv-ground cover	None	0.26	0.28	0.30
	Pasture	0.17	0.21	0.25
	Cultivated land	0.07	0.11	0.15
	Wood	0.03	0.04	0.05

The procedure to be followed to get an assessment of the average annual runoff coefficient by Kennessey is the following:

- The Aridity Index is calculated;

- For each factors (slope, ground cover and permeability) distribution of the basin area inside the single classes in the previous table is evaluated.

E.g., for the ground-cover factor: total area of the basin = 16 Kmq, Ia<25;
4 kmq are wood (25% of the total);
6 kmq are cultivated (37.5% of the total);
3 kmq are pasture (18.75% of the total);
3 kmq are without ground cover (18.75% of the total).

- Percentual areas are multiplied by the corresponding partial coefficients.

For the ground cover:
0.03 (coefficient per wood) x 0.25 = 0.0075;
0.07 (coefficient for cultivated land) x 0.375 = 0.0263;
0.17 (coefficient for pasture) x 0.1875 = 0.0319;
0.27 (coefficient for no ground cover) x 0.1875 = 0.0506.

- Results for each single factor are summed, getting the partial coefficients.

For the ground cover:
Cv=0.0075+0.0263+0.0319+0.0506=0.116

- The three partial runoff coefficients Cv,Ca e Cp are summed, obtaining Cd, average annual runoff coefficient of the drainage basin.

As to the precision of this method, comparing the calculated values of Cd to ones assessed by direct measurements of the runoff, it has been evaluated error does not generally exceed 10%.
Finally the Kennesey method allows to assess the hydrological balance of just a portion of the drainage basin and this fact is useful to the assess of effective infiltration. Infact, limiting the assessment of the balance inside the areas where, for the landforms and permeability, infiltration is more probable, one could get more attendible values of the water volume which infiltrates in the underground.

1.2 Hydrologic balance of an hydrographic basin.

Definitions.

Hydrologic balance is the assessment of the water volumes which og in and out from an hydrographic basin in a specific time interval (generally one year).
Synthetically it can be expressed in the following form:

$$P = D + ET \pm DR;$$

with P = total precipitation in the considered time interval (mm);
D = total outflow (direct runoff and infiltration) (mm);
ET= actual evapotranspiration (mm);
DR= variation of the water storage (mm).

If the parameters P, D and ET are averaged over a long time interval (e.g. 30 years) DR tends to get null, because over a long period positive and negative fluctuations are compensated. In this case the balance is named Average Annual Hydrologic Balance.

Precipitation.

Once defined the time interval to be used to average the parameters of the hydrologic balance, one move forward evaluating the average inflow in the period itself. Parameter P of the balance is generally expressed in the form of rainfall depth (mm) and it can get by an map of the average annual isohyets or, more easily, by the Thiessen method, through interpolation of the recorded data in the measuring stations, by removing the measurement spots too far from the examined area and/or in different climate condition.

Actual evapotranspiration.

It can be directly assess through the Turc formula or indirectly by the evaluation of the potential evapotranspiration (formulas by Thornthwaite or Serra).

- Actual evapotranspiration after Turc.

It is the water volume actually lost through evapotranspiration. The formula is the following:

$$ET = P / \sqrt{(0.9 + P^2 / L^2)};$$

where P(mm) = average annual precipitation;
$L = 300 + 25 \times T + 0.05 \times T^2$;
T(C°) = average annual temperature.

This relation gives satisfaction results for all the climates, though it has to be used with caution in case of small basins, where it tends generally to give overestimated values.

- Potentila evapotranspiration (EP).

It is the maximum water volume that could be lost through evapotranspiration. It could not coincide with ET, when an enough water storage is not available inside the basin. A very popular formula is the Thornthwaite one, which requests as input data of the average monthly temperature only.
The Thornthwaite formula has the following form:

$$EP = K \times 16 \times (10 \times T / ic)^a;$$

with EP(mm) = average monthly evapotranspiration (mm);
T(C°) = average monthly temperature;
ic = monthly index of heat given by:

$$ic = (T / 5)^{1.514};$$

with $T \geq 0$ (C°) (if T<0 put T=0);

$$a = \frac{675 \times ic^3}{10^9} - \frac{771 \times ic^2}{10^7} + \frac{1792 \times ic}{10^5} + 0.49239;$$

K = corrective coefficient to take into account of the sun exposure.

Average annual evapotranspiration is given by the sum of the 12 monthly values. This relation gives results in good agree with the direct measurements.
Another popular formula is the Serra one, which requests however, for the assessment of the monthly values of EP, the relative humidity too.
The Serra formula for the assessment of the annual potential evapotranspiration is the following:

$$EP(mm) = 270 \times e^{0.0644 \times T};$$

the one for the monthly evapotranspiration:

$$EP(mm) = 22.5 \times [(1 - Um) / 0.25] \times [1 - (\Delta T / 2) / 1000) \times e^{0.0644 \times T};$$

where Um (mm) = average monthly relative humidity;
T (C°) = average monthly temperature;
ΔT (C°) = difference between the extreme temperatures of the month.

- Outflow (direct runoff and infiltration).

It is the water volume going out of the basin flowing on the ground or underground. The direct runoff can be assess either through direct measurements of the channel discharge at the outlet or through the product of the rainfall inflow by the runoff coefficient calculated by the Kennessey method.

$$Qs(mm) = P \times Cd;$$

Infiltration is consequently calculated by difference among the other parameters of the balance.

$$Ie(mm) = P - ET - QS.$$

It could happen that Ie be negative. This occurs, when ET has an extremely high value, e.g. when the Turc formula is applied to small basins.

Scheme of the hydrologic balance after Thornthwaite.

After having calculated the monthly EP values by the Thornthwaite method, one can build a scheme of the monthly variations of the water volumes going in and out from the basin that includes the following data:

row n.1	Monthly precipitation;
row n.2	Monthly potential evapotranspiration;
row n.3	P-EP difference;
row n.4	Water held in the shallow layer of the soil (Rs), normally ranging from 50 to 400 mm (it decreases with decreasing of permeability of the shallow layer and increasing with the density of the ground cover);
row n.5	Actual evapotranspiration, corresponding to potential one just in case it is P≥EP or P<EP, but Rs≥EP-P; on the contrary it will be ET<EP;
row n.6	Variation of the water volume held in the shallow layer of soil, positive when P>EP, negative when P<EP;
row n.7	Water surplus, i.e. water volume that flows on the ground or infiltrate; it occurs when P>EP and when Rs reaches its maximum value.
row n.8	Water deficit; it occurs when ET<EP, it is given by the difference between the two parameters (EP-ET).

One can notice that increasing of the Rs value takes to an higher ET value. Indicative Rs values can be gotten by the following table:

Ground condition	Rs (mm)
Sandy soil with scarce ground cover	50
Sandy-clayey soil with pasturer or shrub vegetation	100
Sandy-clayey soil with cultivated or wood land	200
Clayey-sandy soil with pasturer or shrub vegetation	250
Clayey-sandy soil with cultivated or wood land	300
Clayey-sandy soil with old-growth forest	400

Evaporation of a water body.

An approximate assessment of the water volume lost by a water body due to evaporation during one month can be executed through the Conti formula (1924). It has the following expression:

$$E(mm/mese) = \frac{760kcV}{p}$$

where:

kc = coefficient which changes as a function of the month of calculation;

V(mm Hg) = average monthly saturated water vapor pressure;

P(mm Hg) =average monthly barometric pressure.

The kc parameter can be directly gotten by the following table:

Je	Fe	Ma	Ap	May	Jn	Jl	Ag	Se	Ot	No	De
4,4	4,5	5,3	6,0	7,5	6,4	6,3	5,9	5,9	5,8	4,7	3,8

Saturated water vapor pressure is function of the average monthly temperature and can be extrapolated from the following table:

Temperature°C	V (mm Hg)
0	4,58
10	9,21
15	12,80
20	17,50
25	23,80
30	31,80
50	92,50

1.3 Assessment of the IDF (Intensity-Duration-Frequency) curves.

Starting from the pluviometric data given by a measurement station, one can perform the processing to get the curves describing the depth of rainfall (h) as a function of its duration (t).
The equation which links these two variables has usually the following form:

$$h\ (mm) = a\ t^n;$$

where a = variable function of the return period;
n = constant for a given value of t;

and is named Intensity-Duration-Frequency (IDF) curve.
This equation allows to calculate, e.g., the depth of rainfall (h) corresponding to a 30-minute precipitation (t) with a 10-year return period.
An valide estimation of the IDF curves needs a set of recorded data covering an interval 30-35 years long at least. The lesser is the recording interval, the lesser is the reliability of the curves.
To assess the curves corresponding to precipitation with duration higher than one hour, one can proceed as follow:

- for every duration of reference, the recorded data of precipitation are sorted and numerated, regularized by the Gumbel method (see below), in in decreasing way, putting then the maximum recorded values, for each time interval, on the first row of the table, the minimum ones on the last; consequently, e.g., if the recording interval is 30-year long, the first row has to be marked with the number 30, the last one with the number 1;
- using data of each row and processing a regression calculation, the values of the parameters a and n, corresponding to every year, are calculated; the number associated to each row indicates the return period of the rainfall; in the case, e.g., of a recording interval 30-year long, 30 IDF curves are given (i.e. 30 couples of a and n values);

Same procedure has to be adopted in case of duration shorter than 1 hour, when these data are available.
The curves calculated, one can notice that, while *n* stays more or less constant, *a* tends to assume different values as a function of the return period, increasing with it.
Through statistical procedures, one can assess of the parameter *a* in case of return period higher than the maximum number of available annual records.
A widespread statistical method is the Gumbel one. The procedure to be followed is descripted below:

- After the calculation of the IDF curves correspondig to the N years for which the recording are available, tha a values are sorted in increasing way, marking with the number 1 the maximum value and with the number N the minimum one.
- The N ratios:

$$P_i = i / (N + 1);$$

are to be calculated, with i ranging from 1 and N. These ratios indicate the probability that the corresponding *a* value is not exceeded. The calculated P_i values allow to define the scale of the return periods:

$$T_i = 1 / (1 - P_i).$$

- The N couples of values (T_i, a_i) are put in a semilogaritmic chart (the X axis – which shows the return periods – has to be drawn in logaritmic scale), interpolating the points: the chart allows to get the a value corresponding to each return period.

So to get, e.g., the depth of rainfall for a 1.3-long precipitation, with a 50-year return period, one proceeds as follow:

1. by the (Return period–*a* parameter) chart , the *a* value corresponding to a 50-year return period is obtained;
2. the n parameter is calculated averaging the n values given by the single IDF curves;
3. the assessed a and n values are put in the equation $h = a \times t^n$; imposing t = 1.3 hour.

Obviously the extrapolation of the parameters of the IDF curve should not go too far the recording interval.

1.4 Assessment of the effective rainfall.

Effective rainfall is defined as the fraction of the total precipitation of a specific storm, not lost by infiltration, held by plant cover or by evaporation, flowing on the ground surface. The ratio between depth of the effective and total rainfall is named runoff coefficient.
Effective rainfall mainly depends on three factors.

- *Saturation degree of the soil ground before the rainfall event*: the higher is the saturation degree, linked to previous storms, the lower is the capacity of the soil ground to infiltrate more water and consequently the higher is the water volume increasing the shallow outflow.
- *Permeability of the ground*: an higher ground permeability facilitates infiltration of the rainfall, involving a consequently decreasing of the shallow outflow.
- *Land use*: land use strongly influences the shallow outflow; a dense plant cover decreases it, urbanisation increases it, turning the ground impervious.

Curve Number method by Soil Conservation Service.

A method for the assessment of effective rainfall, which has a worldwide diffusion, is proposed by the Soil Conservation Service (1972).
This method, named Curve Number method, is based on the following formula:

$$P_e = (P - I_a)^2 / (P - I_a + S);$$

where: P_e = depth of effective rainfall (mm);
P = depth of total rainfall (mm);
I_a = initial infiltration (mm);
S = potential maximum retention (mm).

Parameter I_a rappresents the water volume initially adsorbed by the soil ground or by the plant cover: till the moment when $P > I_a$ there is no shallow outflow. Parameter S denotes the water volume held by the ground and by the plant cover, and, consequently, subtracted to the direct runoff, when P> Ia: while Ia has a constant value, S increases during the storm up to reaches a maximum value.
The CN method correlates the parameter S to a variable CN, which is function of the soil permeability, of the land use and of the degree of saturation before the selected rainfall. As to the last variable, the SCS procedure requests as input datum the total rainfall depth during the five days previous to the storm taken in account, defining three classes of moisture:

AMC	Dormant season	Growing season
I	< 13 mm	< 36 mm
II	13 - 28 mm	36 - 53
III	> 28 mm	> 53 mm

The terms 'dormant season' and 'growing season' are referred to the plant cover; that is one has to consider the period of the year, referred to the growing stage of the plant cover, in which the rainfall event occurs.
On the base of the selected antecedent moisture class (AMC), the corresponding CN values are defined, respectively CN_I, CN_{II} e CN_{III}.
Falling in the AMC II, one can get the CN_{II} values of the drainage basin by the following table.

LAND USE			PERMEABILITY			
Type	Soil treatment	Drainage	A	B	C	D
Fallow	Straight row	------	77	86	91	94
Row crops	“	Poor	72	81	88	91
	“	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	“	Good	65	75	82	86
	Terraced	Poor	66	74	80	82
	“	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	“	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	“	Good	61	73	81	84
	Terraced	Poor	61	72	79	82
	“	Good	59	70	78	81
Close-seeded or rotation meadow	Straight row	Poor	66	77	85	89
	“	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	“	Good	55	69	78	83
	Terraced	Poor	63	73	80	83
	“	Good	51	67	76	80
Pasture	Straight row	Poor	68	79	86	89
	“	Fair	49	69	79	84
	“	Good	39	61	74	80
	Contoured	Poor	47	67	81	88

	“	Fair	25	59	75	83
	“	Good	6	35	70	79
Meadow	-------	Good	30	58	71	78
Wood	------	Poor	45	66	77	83
	------	Fair	36	60	73	79
	------	Good	25	55	70	77
Farmsteads	------	------	59	74	82	86
Commercial and business districts	------	------	89	92	94	95
Industrial districts	------	------	81	88	91	93
Residential districts	65% impervious	------	77	85	90	92
“	38% impervious	------	61	75	83	87
“	30% impervious	------	57	72	81	86
“	25% impervious	------	54	70	80	85
“	20% impervious	------	51	68	79	84
Paved parking	-------	------	98	98	98	98
Streets and roads	Paved	------	98	98	98	98
“	Gravel	------	76	85	89	91
“	Dirt	------	72	82	87	89

The soil classes A, B, C e D are expression of the degree of permeability of the ground, on the base of the following table:

Soil class	Permeability
A	High
B	Medium
C	Low
D	Very low

On the hypothesis that the selected rainfall event falls in the AMC I or III, to calculate corresponding values of CN_I e CN_{III} has to be used the following correlations:

$$CN_I = CN_{II} / (2.3 - 0.013 \times CN_{II});$$
$$CN_{III} = CN_{II} / (0.43 + 0.0057 \times CN_{II}).$$

Once assessed the CN parameter , on the base of the selected AMC, variable S has to be calculated with the expression:

$$S \text{ (mm)} = 254 \times [(100 / CN) - 1];$$

The parameter I_a likewise can be correlated to S through the following formula:

$$I_a = c \times S;$$

where *c* is a corrective factor ranging from 0.1 to 0.2, but usually sets equal to 0.2. At this point, the total depth of precipitation known, one can assess the effective rainfall.

Rasulo and Gisonni (1997).

It is a simplified method, which allows to assess the runoff coefficient of a drainage basin as a function of the return period of the storm.
The expression is the following:

$$c_a = c_{ap}(1 - A_{imp}) + c_{ai} A_{imp}$$

where:
c_a = runoff coefficient;
c_{ap} = runoff coefficient of the impervious area;
c_{ai} =runoff coefficient of the pervious area;
A_{imp} =ratio between the impervious area and the total one.

Both c_{ap} and c_{ai} are tabulated by the Authors as a function of the return period of the selected rainfall event.

Return period(years)	c_{ap}	c_{ai}
<2	0-0.15	0.60-0.75
2-10	0.10-0.25	0.65-0.80
>10	0.15-0.30	0.70-0.90

Green and Ampt (1911).

The potential infiltration ratio (f) is the maximum water volume which can be infiltrated into the ground, if such a volume is available. The actual infiltration water volume may be lower if the surface runoff is not sufficient. Anyway it cannot be higher.
The potential infiltration ratio depends on the ground permeability and on the initial saturation ratio. The higher is the permeability, the higher will be the infiltration. The higher is the saturation ratio, the lower will be the infiltration.
Green & Ampt's method is commonly used to estimate the potential infiltration ratio. This procedure involves that the saturation front moves itself downward as a function of the time, dividing distinctly the saturated ground volume, with a water contents equal to the soil porosity (η), by the deeper one, not yet reached by the saturation front, having a water contents equal to the initial one (θ).
At time t, after beginning of the infiltration process, the cumulative infiltration F, that is the water volume which is infiltrated till that moment, can be express by the following formula:

$$F(t)(mm) = K\,t + \Delta\theta\,(h0 + \psi)\,\ln[1 + F(t) / \Delta\theta\,(h0 + \psi)]$$

where:
K(m/h) = vertical permeability of the ground, usually sets equal to the 50% of the horizontal one;
t(h) = calculation time;
ψ(mm) = capillary rise;
h0(mm) = hydraulic depth, in respect to the bottom of the lowered area.
$\Delta\theta$ = η-θ;
As the parameter F appears in both the members of the equation, the solution has to be found through an iterative process, imposing a first value inside the second member, solving the equation and then substituting the new calculated value in the second member. Calculation has to be repeated until the difference between two consecutive values of F will be lower than a prefix limit (for example 0.001).

The value of the capillary rise may be chosen, selecting it by the following table:

Soil type	ψ(m)
Gravel	0.05-0.30
Coarse sand	0.30-0.80
Medium sand	0.12-2.40
Fine sand	0.30-3.50
Silt	1.5-12
Clay	>10

Known the cumulative infiltration, the potential infiltration ratio can be calculated by the following expression:

$$f(t)(mm/h) = K\,[F(t) + \Delta\theta\,(h0 + \psi)] \,/\, F(t)$$

In precautionary way, it admits that the infiltration occurs only at the bottom of the lowered area.
To assess the effective rainfall, one proceeds dividing the rainfall duration in time intervals in which the rainfall intensity can be considered constant. For each interval, the calculated value of f(t) has to be compared to the rainfall intensity i(t). One can occur two cases

1) $f(t)>i(t)$:in this case the rainfall depth completely infiltrates in the soil ground and the direct runoff is null.
2) $f(t)\leq i(t)$: in this case a water surplus is generated which flow on the surface.

The depths of the infiltration and of the direct runoff are obtained summing the contributions for each time step in which the rainfall event has been divided.

1.5 Processing the design rainfall.

The calculation of the maximum peak discharge in a water course and the processing of the corresponding hydrograph has to be preceded by the assessment of the design rainfall, that is the heavier rainfall for a specific return period.
This assessment requests three steps:

- calculation of the depth of the selected rainfall;
- estimation of the isohyetal area factor;
- building of the hydrograph.

Assessment of the depth of the selected rainfall

After determining the return period of the storm and its duration, the rainfall depth can be assess through the IDF curve of the pluviometric station of reference (see paragraph 1.3):

$$h = at^n$$

In case of two or more stations inside or close to the watershed, h can be determined through the following procedure:

- the values of h for each station are determined;
- by the isohyetal method the areas of influence of each station are calculated;
- the average value h_{av} is calculated by doing the weighted mean, as a function of the area of influence of each station, of the single values of h.

Isohyetal area factor

It is a multiplication factor, ranging from 0 to 1, to take in account the trend to the decreasing of the rainfall depth as a function of the increasing of the area interessed by the rainfall event.

The rainfall depth, measured in a pluviometric station, is punctual datum and it must to be consequently corrected as a function of the area in which the rainfall event is distributed.
In case of small watersheds (up to 100 kmq) the DEWC formula (1981) can be used. It is based on the following expression:

$$R = 1 - at^{b}$$

where:
a = $0.0394A^{0.354}$
b = $0.40 – 0.0208\ln(4.6-\ln A)$ for $A \leq 20$ kmq
b = $0.40 – 0.00382\ln(4.6-\ln A)^2$ for $A>20$ kmq
A = watershed area in kmq

Another available method is the one proposed by Desbordes et Alii (1982), based on the following formula:

$$R = (100A)^{-0.05}$$

where A is the watershed area in kmq.
In case of very small drainage basin, the isohyetal area factor is often set equal to 1.
Once calculated R, the rainfall depth has to be asses through the following formula:

$$h_r = hR\,.$$

Building the hydrograph

The curve displaying the trend of the rainfall intensity as a function of time is named hydrograph. Several procedures ara available to build it.

Constant intensity

It is based on the hypothesis that the rainfall intensity keeps constant during the whole duration of the storm.

Practically it has to be set:

$$i(mm/h) = \frac{h_r}{t_p}$$

where:
i = rainfall intensity;
h_r = corrected rainfall depth;
t_p = rainfall duration.

It is a widespread method, especially used for very small watersheds.

Triangular hyetograph (Chicago method)(1953)

It is based on the hypothesis that the rainfall intensity gradually increases up to reach a peak, beyond which it decreases till the end of the storm.
The increasing part of the curve is given by

$$i(t) = ant_1^{n-1}$$

where:
a = a factor of the IDF curve;
n = n factor of the IDF curve;
t_1 = $(rt_p - t)/r$ with t ranging from 0 to rt_p
t_p = rainfall duration;
r = peak position, ranging from 0 to 1 and usually sets= 0.5.

The decreasing part fo the curve is likewise given by:

$$i(t) = ant_2^{n-1}$$

where:
$t_2 = (t-rt_p)/r$ with t ranging from 0 to t_p.

By a practical point of view, the temporal step and the positionof the peak are set and then the two previous expressions are applied, ranging them in the interval 0-t_p, with the discrete step of calculation selected.
This method , with respect the previous, gives a more realistic development of the rainfall intensity and can be used in case of several-hundred-square-kilometer watersheds and several-hour rainfall.
Using calculation steps too small (<0.5 h) might involve to an excessive accentuation of the central peak.

Sifalda method (1973)

Hyetograph is divided in three parts. In the central portion, which involves a time interval from 0.14t_p to 0.70t_p , with t_p equal to the rainfall duration, the rainfall intensity is given by:

$$i_c = \frac{h_c}{0.25 t_p}$$

where h_c is the rainfall depth obtained by the IDF curve, imposing t=0.25 t_p.
In the first tract of the curve, from t=0 to t=0.14t_p, one assumes that the intensity increases in a linear way from a minim value of 0.065i_c to a maximum of 0.435i_c. In the last part of the curve, from t=0.70t_p to t=t_p, one hypotizes that the intensity decreases, still in a linear way, from a maximum value of 0.435i_c to a minimum of 0.087i_c.

1.6 Hypsometric curve

The analysis of the morphological framework of the drainage basin can be sum up through the hypsometric curve. The curve is drawn on the base of the altitude and of the corresponding cumulated areas, dividing the watershed in interval of altitude, from the minimm value to the maximu, and calculating the area laying in each interval.
They have to calculate then the ratio between the areas of each interval (a) and the total area of the watershed (A), and the ratio between the differences of altitude of each interval with respect to the plane of reference (h) and the total difference of altitude of the drainage basin (H).
The function is of the sort:

$$y = f(x) \quad \text{where: } y=h/H \text{ and } x=a/A.$$

The integral of the hypsometric curve is given by the area underlying the curve with respect to the X axis.
By the hypsometric curve one can calculate the average altitude of the watershed, through the expression:

$$Hm = (1 / A) \times \sum a_i \times h_i.$$

with A = total area of the watershed;
a_i = area of the watershed inside the i-th altitude interval;
h_i = average altitude of the the i-th altitude interval.
The analysis of the curve allows to assess the stage of evolution reached by the watershed.

Basin stage: youth	the hypsometric curve displays an upward convexity with an average value of the integral higher than 60%
Basin stage: mature:	the hypsometric curve displays an inflection point with an average value of the integral close to 50%
Basin stage: old	the hypsometric curve displays an upward concavity with an average value of the integral lower than 30%

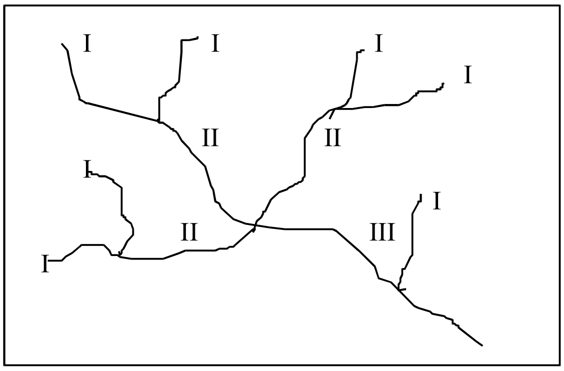
As the relative distribution of the areas and altitudes is subject to the shape of the horizontal projection of the watershed, the curve is significant only in case of a regular and sub-rectangular basin shape, condition hard to occur. So the analysis has to be limited to the central part of the hypsometric curve, that is to the one ranges from 15 to 85% of the global area, because it is the interval which allows a, appropriate assessment of the evolution degree reached.

1.7 Analysis of stream networks.

Stream networks can be classified as a function of the channels included among the several junction points. A order number is imposed to each of these, depending on its position inside the stream network, as suggested by STRAHLER.

Stream networks hierarchy by STRAHLER.

Channels of the stream network are identified by an order number increasing as a function of the lower-order streams converging in it. All the channels without upstream confluences are designated order 1. The junction of two channels of order 1 results in a channel of order 2, the junction of two channels of order 2 results in a channel of order 3 and so on. Consequently a channel of order N is given by two channel of order N-1.



Gerarchizzazione sec.Strahler.

Morphometric parameters.

The following parameters are defined:

$Rb=N_u / N_{u+1}$	Bifurcation ratio
$Rbd=N_{ud}/N_{u+1}$	Direct bifurcation ratio
Ib=Rb-Rbd	Bifurcation index
Su=(Rb/2)-1	Conservativity ratio
Ga	Hierarchical anomaly number
Dga=Ga/A	Hierarchical anomaly density
$Iga= Ga/N_1$	Hierarchical anomaly index

where:

N_u = sum of the number of u-order channels;
N_{ud}=sum of the number of u-order channels which directly confluence in the u+1-order ones;
N_{u+1}= sum of the number of u+1-order channels;
A = total area of the watershed;
N_1= number of 1-order channels.

Bifurcation ratio (Rb) gives indications about the structure of the stream network. The Rb value to be taken as representative of the watershed is that given by the mean (arithmetic or ponderate) of the partial Rb values, with reference to the single couples of order u and u+1 . Rb is normally ranging from 3 to 5, with a theoretical minimum value equal to 2 (two streams of order u for each stream of order u+1). Generally the higher is Rb the lesser is the hierarchical degree of the drainage basin. Values higher than 5 are extremely rare, suggesting a strong tectonic influence on the network development. Watersheds with the same value of Rb can be distincted on the base of the direct bifurcation ratio (Rbd). For the same watershed, different values of Rb and Rbd involves the presence of anomalous confluences, that is u-order confluences in channels of u+2 or higher order.
More significative is consequently the bifurcation index (subtraction Rb-Rbd), which normally assumes values ranging from 0.2 to 4. Anomalous

values can be found where the network develpment is strongly controlled by lithologic and tectonic factors.
The maximum hierarchical degree is given when the bifurcation index assumes a value equal to 0 (Rb=Rbd), that is when all the u-order channels confluence in u+1-order streams. Values close to 0 are typical of drainage basin in a old or mature stage. Ib high values are characteristic of young stage. A limit case is given when Ib=0 and Rb=Rbd=2, that is when the stream network has the maximum hierarchical degree and the maximum conservativity (watershed in old stage).
A conservative network is the one with the minimum number of channels to result in the higher order of the drainage basin.
The network conservativity is expressed by the conservativity ratio (Su), which assumes 0 as minimum value (maximum conservativity).
One can generally tell that the hierachical degree, expressed by Ib, and the conservativity level, expressed by S0, decrease over time (both Ib and Su tend to 0) as a function of the network evolution. This is true when a strong tectonic or lithologic control over the drainage basin is missing and if does not occur event which can be break the normal network evolution (e.g. sudden variation of the base level).
Another parameter which allows to define the hierachical organization of a drainage basin is the hierarchical anomaly number, defined as the minimum number of 1-order channels to get a perfectly hierarchized network. This variable is expressed by the sum of the number of anomalous i-order channels confluencing in streams of order (r), with i ≤ r-2, inside a drainage basin of order (s):

$$Ga = \sum_{\text{da } i=1 \text{ a } s-2} \sum_{\text{da } r=i+2 \text{ a } s} N_{i,r} \text{ x } f_{i,r};$$

with $N_{i,r}$ = number of i-order channels confluencing in streams of order r;
$f_{i,r} = 2^{r-2} - 2^{i-1}$.

Generally a higher value of this parameter involves a lower hierarchical degree of the stream network. This variable can be used to get the hierarchical anomaly density and index, parameters which allows to compare the evolution stages of drainage basins in different climate

conditions or with different areas, which one cannot do through the parameters Ib and Su.
Better indications about the evolution stage of a watershed can be obtained comparing the hierarchical degree of the stream network with the hypsometric curve; if, e.g., a watershed would display a low hierarchical degree and a hypsometric curve indicating an old stage, this might involve a recent variation of the base level, which influences the stream network but not the relief as a whole.

Other morphometric parameters.

- Circularity coefficient (Kc):

$$Kc = Pb / (2 \times \sqrt{\pi \times A});$$

with Pb=watershed perimeter;
A= watershed area;

- Circularity ratio (Kr):

$$Kr = A / (0.0796 \times Pb^2);$$

both the parameters give an indication how much a watershed deviates from circular shape. Values of Kc and Kr far from 1 are typical of elongated-shape watersheds and vice versa in case of Kc and Kr close to 1. A circular watershed, for the same other factors, has lower concentration time and more sudden and more pronounced floods, with a hyetograph characterized by a tight and sharp shape.

- Drainage density (Dr):

$$Dr = \sum l / A;$$

with $\sum l$ = sum of the lengths of all the channels, without regard about their order.

- Drainage frequency (Fr):

$$Fr = N / Ab;$$

N = number of channels inside the watershed (sum of the channels of all the different orders).

This two parameters mark the devolepment stage of a stream network. Low values are typical of a low-degree-evolution watershed or of drainage basins developed over erosion-proof lithologies or high-permeability ground with a dense plant cover. Average Dr values ranging from 2 to 4, the Fd ones from 6 to 12.

- Length ratio (Rl): it is given through the ratio between the average length of the i-order channels and the average length of the (i-1)-order channels.
- Areal ratio (Ra): it is given through the ratio between the average area of the i-order channels and the average area of the (i-1)-order channels.

Morphometric-parameter charts.

- N.order – N.channels per order.

In case of well-developed network the points of the number of channels as a function of order number has to lay on a straight line, in a semi logarithmic chart. If not it means anomalous distributions of the channels are present.

- N. order – average length of the channels.
- N. order – average area of the sub-basins inside the main watershed.
- N. order – average slope of the sub-basins inside the main watershed.

Lengths, areas and slopes has to follow a linear law (in semi-logarithmic scale). If not, it means the watershed displays heterogeneity inside its surface due to, e.g., different ground permeability, lithologic varability, tectonic controll.

1.8 Assessment of the maximum flood discharge.

Leaving aside empirical formulas, which give very approximately assessments and needed to be locally calibrated, the more widespread methods are the rational and the probabilistic ones.

Rational methods.

They are based on the assessment of the time of concentration (τ_c).
Time of concentration is the time interval necessary because the direct runoff reaches the watershed outlet, starting from farthest point of the drainage basin. This quantity is constant for each watershed, because it is exclusively a function of the land shape, ground lithology and plant cover.
Some widespread calculation methods are listed below:

- Giandotti:

$$\tau_c(h) = \frac{4\sqrt{S_b} + 1{,}5L_p}{0{,}8\sqrt{H_m}}$$

- Pezzoli:

$$\tau_c(h) = \frac{0{,}055L_p}{\sqrt{0{,}01P_m}}$$

- F.A.O.:

$$\tau_c(h) = \frac{{L_p}^{1.15}}{15{h_{\max}}^{0.38}}$$

- Kirpich:

$$\tau_c(h) = 0{,}003245\left(\frac{1000\mathrm{L}_\mathrm{p}\sqrt{1000L_p}}{\sqrt{h_{\max}}}\right)^{0{,}77}$$

- Ventura:

$$\tau_c(h) = 0{,}1272\left(\frac{S_b}{0{,}01S_a}\right)^{\frac{1}{2}}$$

- Pasini modified:

$$\tau_c(h) = \frac{0{,}0864\sqrt[3]{\mathrm{S_b}L_p}}{\sqrt{0{,}01S_a}}$$

- Ongaro:

$$\tau_c(h) = 4{,}32\sqrt[3]{\mathrm{S_b}L_p}$$

where :

S_b (kmq) = watershed area;
L_p (km) = length of the main channel;
P_m (%) = average slope of the drainage basin;
H_m (m) = average altitude of the watershed with respect to the outlet;
h_{max} (m) = maximum altitude of the watershed with respect to the outlet;
L (ft) = length of the main channel extended up to the watershed;
CN = S.C.S. Curve Number;
S_a(%) = average slope of the main channel.

All this expressions are usable in case of drainage basin of small and medium extension. The Giandotti formula generally gives, in case of very small watersheds (lesser than 100 kmq), overestimated values. The ongaro formula has to be used in case of watershed inside alluvial plains with a surface lesser than 1 kmq.

Once calculated τ_c , one can assess the maximum-peak-flood discharge. First, a design rainfall has to be chosen, for an established return period, corresponding to a duration equal to time of concentration. This quantity may be calculated by the processing method seen in the paragraph 1.5. Then the calculated rainfal depth (h) has to be inserted in one the several rational formulas available.

- Rational formula.

It has the following expression:

$$Q_{\max}(mc/s) = 0.278\frac{k_f c_a hA}{\tau_c}$$

where:

Q_{max} (mc/s) = maximum-peak-flood discharge for a given return period;
c_a = runoff coefficient, ranging from 0 to 1 (see paragraph 1.4);
A (kmq) = watershed area;
h (mm) = corrected depth of rainfall referred to a duration equal to τ_c for a given return period;
k_f = frequency factor as a function of the selected return period;
τ_c (h) = time of concentration.

Quantity c_a can be assess through one of the methods seen in the paragraph 1.4. As an alternative, it can be calculated in a approximately way through simplified expression, as the Schaake et Alii (1967) one:

$$c_a = 0.14 + 0.65A_{imp} + 0.05i_c$$

where:

A_{imp} = ratio between the impervious area of the watershed and the total one;
i_c = average slope, in %, of the main channel.

or derived by the following table (Chow et al., 1988)

Land use	c_a
Asphalt	0.657
Concrete, roofs	0.657
Crops(0-2%)	0.375
Crops(i=2-7%)	0.395
Crops(i>7%)	0.401
Pasture(i=0-2%)	0.349
Pasture(i=2-7%)	0.381
Pasture(i>7%)	0.395
Wood(i=0-2%)	0.316
Wood(i=2-7%)	0.368
Wood(i>7%)	0.381

In case of very small watershed (few kmq), mainly impervious, , the runoff coefficient may be imposed equal to 1.
Frequency factors can be obtained by the following table:

Return period	k_f
10	1.23
20	1.33
30	1.38
50	1.42
100	1.47
200	1.50
500	1.52

This quantity allows to take in account the variability of the runoff coefficient as a function of the rainfall depth. Parameter ca, infact, depends on several factors, as infiltration and evapotranspiration, which are in turn function of the rainfall volume and rainfall intensity. The higher is the rainfall depth, the lower is the water volume, in proportion, retained by the watershed, that is the higher is the direct runoff. Since to higher rainfall depth correspond longer return periods, ca has to be imposed as a function of the return period of the rainfall event too.
The tabulated data can be interpolated in case of interim return periods.

- Giandotti formula

It has the following expression:

$$Q_{\max}(mc/s) = 0.278\frac{ChA}{\tau_c}$$

where C for watershed with area lesser than 300 kmq has to be imposed equal to 1.25. As an alternative, Visentini (1938) suggested to assess C through the expression:

$$C = 6.19A^{-0.319}$$

where A is the area of the dranage basin in kmq.

Experimental data show, however, this expression tends to overestimate the peak-flood discharge in case of small watersheds (few tens kmq), because originally calibrated over drainage basin with an extension higher than 500 kmq.

- Merlo formula.

It has the following expression:

$$Q_{\max}(mc/s) = C_m hA$$

where:

C_m = 0.0363 + 0.0295 x ln(T_r);
T_r (years) = return period.

This method has been calibrated over small watersheds.

Probabilistic methods.

Probabilistic methods get deal of the problem of the flood forecasts, by assuming that they are merely random phenomena, which occur unrelated among them. Probabilistic methods can be of local or regional kind.

Local probabilistic method (Gumbel)

Gumbel method is the most common to analyse local sets of data.
The procedure is explained below

- having N values of annual-peak discharges (Q), such data are sorted in increasing order, setting the number 1 to the maximum value and the number N to the minimum one.
- The N ratios:

$$P_i = i / (N + 1);$$

are calculated, with i ranging from 1 to N. These ratios display the probability that the corresponding Q value not be reached or exceeded. The resulting P_i values allow to define the return period scale:

$$T_i = 1 / (1 - P_i).$$

- The N couples of values (T_i, Q_i) are riported in a semilogarithmic chart (X axis, showing the return periods, has to be built in logarithmic scale), interpolating among the points a straigth line: the resulting chart allows to get a Q value for each selected return period.

Obviously extrapolation has not to be extended too far the recording period.

Regional probabilistic method (T.C.E.V.)

The limit of applicability of the Gumbel method is the availability of a set of data for the examined watershed. In case of unequipped basin this methodology is not applicable. The regional probabilistic methods overtake this condition through the individuation of zones, at regional scale, in which the probability distribution function F(x) may be considered homogenous.

The quantity F(x) is named *regional growth curve* and allows to identify, inside an homogenous zone, the variability of the flood discharges as a function of the return period, applying a scale factor.
The T.C.E.V. (Two Component Extreme Value) method has the following expression:

$$F(x) = \exp[-\lambda_1 \exp(-\eta x) - \lambda_* \lambda_1^{1/\theta_*} \exp(-\eta x / \theta_*)]$$

The quantities λ_1, λ_*, θ_*, η are the regionalized parameters of the regional growth curve, invariant inside of the homogenous zone to which they are referred. By a practical point of view, the method consists of applying the Gumbel method to two sets of data, the first one referred to the ordinary floods, the second one to the exceptional floods. Precisely because the data relative to the exceptional floods are extremely rare, the procedure has to be extended over a regional base, involving more measurement stations.
The calculation steps to assess the maximum flood discharge of a watershed of area A_b, corresponding to a specific return period T_0 is the following:

- once the homogeneous zone inside which the watershed is lying is identified, the regional growth curve is built; the curve is obtained ranging x, with a regular step, inside a specific interval, e.g. 0-10, and then calculating the corresponding values of F(x); along the x axis is usually reported the return period instead of F(x), given by:

$$T = \frac{1}{1 - F(x)}$$

- established T_0 , the corresponding value of x is searched along the x axis;
- the maximum-flood discharge is given by the expression:

$$Q = xq^*$$

where q* is the index discharge.
The quantity q* can be directly obtained, applying a local empirical correlation, where the index discharge is linked to the morphological and climate condition of the examined watershed, or indirectly, correcting by a

scale factor, given by the ratio between the areas of the drainage basins, the average maximum-flood discharge of an equipped watershed inside the same homogeneous zone. In the last case the procedure to be followed is the below one:

- inside the homogeneous zone, in which the watershed is laying, an equipped water course, for which data corresponding to an enough long time interval are available (10 years at least), is identified;
- an average annual maximum-flood discharge Q_m, is calculated, averaging the available data;
- the area A_r of the equipped watershed, upstream with respect to measurement spot, the is calculated;
- the index discharge is finally given by:

$$q^* = Q_m \frac{A_b}{A_r}$$

Synthetic hydrograph by Nash.

Aside the value of the maximum flood discharge, it can be necessary to assess the discharge as a function of the time at a selected point of the channel (hydrograph). The curve is called synthetic hydrograph in case it is built through empirical procedure.
Nash's method, which allows to draw a syntetich hydrograph, starting from the data of the effective rainfall as function of time, is based on the following formula:

$$Q\,(m \times \Delta t) = \frac{S_b}{[k \times \Gamma(n)]} \times \sum_{da\ i=1\ a\ m} (e^{-i \times \Delta t/k} \times (i \times \Delta t/k)^{n-1} \times h_{m-i+1} \times \Delta t);$$

where:

Q (m x Δt) = flood discharge a the moment m x Δt, with m ranging from 1 to N, where

N=maximum number of time interval taken in account;

Δt (h) = time step (generally equal to 1 h);

m = number of current time step;
Γ(n) = gamma function;
S_b (kmq) = watershed area;
h_{m-i+1} (mm) = direct runoff during the interval (m-i+1);
k,n = characteristic coefficients of the watershed, normally ranging from 1 to 10;

This method requests the parameters *k* and *n* to be know. These can be calculated if there is the availability of previous measured flood events and of the corresponding hyetographs, referred to the same outlet.
As an alternative the two parameters *k* and *n* can be assess through correlations with geometric or morphometric quantities of the watershed.
Several correlations are available, among which Rosso(1984), Nash (1960) e Mc Sparran (1968).

- Rosso (1984)

The author correlates k and n to some morphometric parameters of the watershed through the following expressions:

$$n = 3.29 \times (Rb / Ra)^{0.78} \times Rl^{0.07};$$
$$k = 0.70 \times [Ra / (Rb \times Rl)]^{0.48} \times (L / v);$$

where: Rb = bifurcation ratio;
Ra = area ratio;
Rl = length ratio;
L (m)= length of the main channel;
v(m/s) = average velocity of the runoff across the stream network.

While Rb, Ra, Rl and L are easily inferred by the morphometric analysis of the watershed, the parameter v is hard to be evaluated and, approximately, can be set equal to the measured value in other watershed with similar area and altitude.

- Nash (1960)

Set:

$$m_1 = nk$$
$$m_2 = \frac{nk^2}{m_1^{\ 2}}$$

n and k can be calculated, assessing the quantities m_1 and m_2 through the expressions:

$$m_1 = 27.6A^{0.3}{i_b}^{-0.3}$$
$$m_2 = 0.41L^{-0.1}$$

where:

A = basin area expressed in square miles;

L = length of the main channel extrapolated up to the watershed in miles (1 mile= 1.609 km);

i_b = average slope of the drainage basin expressed in parts per 10000.

- Mc Sparran (1968)

Mc Sparran has suggested the following expressions:

$$n = 4.1\frac{t_p}{k_1}$$
$$k = \frac{t_p}{n-1}$$

where t_p and k_1 have the following form:

$$t_p = 5.52A^{0.208}i^{-0.447}$$
$$k_1 = 3.34A^{0.297}i^{-0.354}$$

where:
A = basin area expressed in square miles;
i = average slope of the drainage basin expressed in parts per 1000.

Synthetic hydrograph by S.C.S. method.

It is based on the expression by the SCS method used to calculate the effective rainfall:

$$P_e = (P - I_a)^2 / (P - I_a + S);$$

A synthetic unit hydrograph (hydrograph referred to a depth of rainfall equal to 1 cm) of triangular shape is used. The peak is reached at time Tp since the single pulse of rainfall began. Tp is given by:

$$T_p = t_r/2 + t_p$$

where:
t_r = duration of the single pulse of effective rainfall;
t_p 0.6 t_c, with t_c = time of concentration calculated by the SCS formula:

$$\tau_c\,(\text{min}) = \frac{100L^{0.8}\left[\left(\frac{1000}{CN}\right) - 9\right]^{0.7}}{1900{S_a}^{0.5}}$$

with :
L (ft) = length of the main channel extrapolated up to the watershed;
CN = Curve Number;
S_a(%) = average slope of the drainage basin.

The peak-flood discharge, corresponding to the single pulse of rainfall, is given by:

$$q_p = 2.08\ A/T_p$$

where A is the watershed area in kmq.
The total duration of the unit hydrograph is equal to:

$$T_f = 2.67\ T_p$$

Then the hydrograph relative to the selected rainfall event is obtained, applying the discrete deconvolution equation:

$$Q_n = \sum_{m=1}^{n \leq M} P_m U_{n-m+1}$$

with:

Q_n	=discharge at the i-th calculation step;
P_m	=m-th effective rainfall pulse (M=total number of pulses);
U_{n-m+1}	=unit hydrograph corresponding to a single effective rainfall pulse.

Synthetic hydrograph by rational method.

The effective rainfall is given by

$$P_e = c_a\ P;$$

where c_a is the runoff coefficient amd P the rainfall depth.
A synthetic unit hydrograph of triangular shape is used. The peak is reached at time Tp since the single pulse of rainfall began. Tp is given by:

$$T_p = t_r/2 + t_p$$

where:

t_r	= duration of the single pulse of effective rainfal;
t_p	0.6 t_c, with t_c = time of concentration calculated by the SCS formula:

$$\tau_c\,(\min) = \frac{100 L^{0.8}\left[\left(\frac{1000}{CN}\right) - 9\right]^{0.7}}{1900 S_a^{\ 0.5}}$$

with :

L (ft) = length of the main channel extrapolated up to the watershed;
CN = Curve Number;
S_a(%) = average slope of the drainage basin.

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with:

Q_n =discharge at the i-th calculation step;

P_m =m-th effective rainfall pulse (M=total number of pulses);

U_{n-m+1} =unit hydrograph corresponding to a single effective rainfall pulse.

1.9 Soil erosion of a drainage basin.

Assessment of the soil erosion in a drainage basin

Assessment of the soil erosion inside a watershed can be afforded by several methodologies, which differs both for the meaning of the results and for the applicability conditions.

- Gavrilovic method.

This method requests the knowledge of geometrical data of the selected drainage basin and of some parameters linked to the soil erodibility (as a function of the plant cover, of the lithology and of the morphological conditions) of the part of it subject to erosion.
The expression, on which the method is based, is the following:

$$W(mc/anno) = Fh\pi\sqrt{\frac{t°}{10}+0.1}\sqrt{[m_1+m_2m_3]^3}$$

where:
F = watershed area in kmq;
h = average annual precipitation in mm
t° = average annula temperature in °C;
m_1 = land use factor;
m_2 = shallow lithology factor;
m_3 = slope factor.

The m_1, m_2 e m_3 factors are given by:

$$m_1 = \frac{0.2A + 0.5B + 0.8C + 1.0D}{F}$$

with:

A = area covered by wood or orchade in kmq;
B = area covered by meadow and pasture in kmq;
C = area covered by crops in kmq;
D = bare rocks in kmq.

$$m_2 = \frac{1.6J + 0.8K + 0.3L + 1.6M}{F}$$

where:
J = soils with scarce erosion resistance in kmq;
K = rock with moderate erosion resistance in kmq;
L = hard rock, erosion resistant in kmq;
M = Fault spreading in km x 0.1 kmq in kmq.

$$m_3 = \theta + \sqrt{I}$$

with θ, function of the V/F ratio, gives an indication about the morphological instability inside the drainage basin. V is given by the expression:

$$V(kmq) = 0.2N + 4.2P + 4.9Q + 2.25R + 0.75S + 2U$$

where:
N = area generically subject to landslides in kmq;
P = zones with landslides involving granular or semi/pseudo cohesive soils in kmq;
Q = zones with pseudo-gully landforms due to tectonization of cohesive rocks in kmq;
R = zones with numerous rockslides in kmq.
S = zones with widespread rockslides;
U = area involved by avalanches (km x0.1km) in kmq.

Once V is assessed, quantity θ is given by the following table:

V/F	θ
0	0
0.5	0.2
2	0.4

4	0.6
6	0.8
7	0.9
7.5	0.95

Where, for intermediate values, one can proceed by interpolation.
Factor I is relative to the effect of the basin slope and is given by:

$$I = \frac{\sum_{m=1}^{6} I_m i_m}{F}$$

where:
I_1 = area in kmq with average slope ranging from 0 to 10% ; i_1=0.05
I_2 = area in kmq with average slope ranging from 10 to 20% ; i_2=0.15
I_3 = area in kmq with average slope ranging from 20 to 40% ; i_3=0.30
I_4 = area in kmq with average slope ranging from 40 to 60% ; i_4=0.50
I_5 = area in kmq with average slope ranging from 60 to 80% ; i_5=0.70
I_6 = area in kmq with average slope ranging from >80% ; i_6=2.00

Quantity B represents the volume of sediment available to be moved. The effective volume of eroded sediment which will cross down, during a selected time interval, the basin outlet is given by:

$$Q_s(mc/anno) = \frac{4\sqrt{PH}}{L+10}W$$

valid for small watersheds, where:
P = basin perimeter in km;
H = average altitude with respect the basin outlet in km;
L = length on the main channel in km.

As this method takes in account all the four main factors, which conditions the erosion degree in a drainage basin (shallow lithology, plant cover, average slope and climate data), through parametrs easily obtnaible, it can be considered an user-friendly method with a good reability degree.

This method has been calibrated over numerous watersheds in whole Europa, involving very different climate, morphological and lithologic conditions. It can be used to realize an erodibility map, dividing the basin area in an appropriate number of sub-basins.

- Climates methods.

Solid discharge in corresponding to the cross-section of the basin outlet is given by the relationships by Langbein & Schumm and Fournier.

Langbein & Schumm.

$$S(mc/kmq) = \frac{1.631(0.03937P)^{2,3}}{1 + 0.0007(0.03937P)^{3,3}}$$

where:

S (mc/kmq) =volume of the annual solid transport trasporto solido per kmq;
P (mm) = annual effective precipitation.

Fournier.

$$Log_{10}D_s(t/kmq) = 2.65 Log_{10}\left(\frac{p^2}{P}\right) + 0.461 Log_{10}\left(\frac{H^2}{S_b}\right) - 1.56$$

with:

D_s (t/kmq) = weight of the annual solid transport trasporto solido per kmq;
p (mm) = precipitation of the rainiest month;
P (mm) = total annual precipitation;
H (m) = average altitude of the watershed with respect to the basin outlet;
S_b (kmq) = watershed area.

These methods give significative assessment in case of large drainage basin, where the effect of morphology, shallow lithology and plant cover tends to get null.

1.10 Check of a channel cross-section.

Uniform flow

Outflowing discharge across a selected-channel cross-section is given by:

$$Q\ (mc/s) = A \times v_m;$$

where:

A (mq) = area the cross-section;
v_m (m/s) = average velocity of the stream.

Assuming an uniform-flow condition, that is when energy line, water surface and channel bottom are all parallel, criterion generally valid in a water course with a sligth slope of the channel bottom, the average velocity of the stream can be expressed by the Manning-Strickler expression:

$$v_m\ (m/s) = K_s \times R_h^{2/3} \times (i/100)^{1/2};$$

where:

K_s ($m^{1/3}s^{-1}$) = Strickler coefficient;
R_h(m) = hydraulic radius = A / wetted perimeter;
i (%) = upstream slope of the channel.

In case of circular pipe not under pressure, the previous formula may be simplified as follow:

$$v_m\ (m/s) = K_s \times (D/4)^{2/3} \times (i/100)^{1/2};$$

where D is the pipe diameter.
Using the Chézy-Tadini formula, the expression of the average velocity assumes the following form:

$$v_m\ (m/s) = \chi \times (R_h \times i/100)^{1/2};$$

where the χ parameter is given by:

$$\chi = \frac{100}{1 + \frac{m}{\sqrt{R_h}}}$$

with m = roughness coefficient by Kutter.
Once the stream velocity has been computed and the cross-section area has been calculated, one can assess the maximum water discharge, to be compared with the reference-flood discharge.
The K_s (Strickler) and m(Kutter) coefficients can be chosen by the following table:

Channel type	m (m1/2)	K_s ($m^{1/3}s^{-1}$)
OPEN CHANNEL (Rh ≈1)		
Lined with:		
asphalt	0,33-0,76	57-75
masonry	0,39-0,76	57-72
concrete	0,29-0,76	57-77
random stones	1,00-4,00	20-50
stones	2,33-5,67	15-30
Excavated or dredged:		
Earth, straight and uniform	0,67-2,33	30-60
Earth, winding and uniform	1,00-4,00	20-50
Not maintained or rock cuts	1,00-4,00	20-50
MINOR STREAMS (Rh ≈ 2) (top width at flood stage <30 m)		
straight	1,39-4,89	20-45
winding	3,62-6,99	15-25
mountain streams with few boulders	2,19-4,89	20-35
mountain streams with large boulders	3,63-6,99	15-25
MAJOR STREAMS (Rh ≈ 4) (top width at flood stage ≥ 30 m)		
regular section	1,53-3,29	30-45
irregular section	3,29-5,94	20-30
FLOOD PLAINS		
pasture	1,50-4,00	20-40

Cultivated areas	1,00-4,00	20-50
Brush	2,33-4,00	20-30

K_s factor can be directly assessed by the following relationship, valid in particular for mountain streams:

$$K_s\ (m^{1/3}s^{-1}) = 26\ /\ d_{90}^{\ 1/6};$$

d_{90} (m) = 90% effective diameter of grains.

Gradually varied flow

In this case one suppose the energy line has a different slope with respect to the channel bottom. With a constant water discharge, that is with no significative immission or loss along the selected channel tract, the procedure to be follow is explained below.

1) The water discharge to be checked is established.
2) The check channel cross-section and the one to be checked are defined, laying at a distance ΔX between them. The check channel cross-section is the one in which the water depth is known or where a critical-depth condition is present. A critical-depth condition occurs when a specific water discharge passes with the minimum energy with respect to the channel bottom (e.g. in the case of a sudden variation of the bottom slope). In this case the water depth can be assess through the formula:

$$\alpha_c \frac{Q^2 b}{gA^3} = 1$$

where:
Q(mc/s) = water discharge;
b(m) = top width of the channel;
g(m/s²) = gravity acceleration = 9.81;
A(mq) = water area;

α_c = Coriolis coefficient.

The Coriolis coefficient has to be calculated by the following expression:

$$\alpha_c = \frac{A_{tot} \sum_{i=1}^{n} \frac{C_i^3}{A_i^2}}{C_{tot}}$$

with:

n = number of coordinates of the cross-section profile;
A_i = water area between the point (i) and the point (i+1) of the section;
C_i = flow-resistance factor between the point (i) and the point (i+1) of the section, given by: $C_i = K_{si} A_i R_{hi}^{2/3}$, where K_{si} is the roughness coefficiente by Gaukler-Strickler and R_{hi} is the hydraulic radius at the point (i);
A_{tot} = total water area;
C_{tot} = total flow-resistance factor, given by the sum of the single flow-resistance factors.

If the flow is supercritical (Froude number>1), the check section has to be the upstream one. Vice versa, if the flow is subcritical (Froude number<1), the check section has to be the downstream one.

3) The flow velocity is calculated by the following formula:

$$v_c = \frac{Q}{A_{tot}}$$

4) The depth of the energy line at the check section is assessed by the expression below:

$$E_c = h + z + \alpha \frac{v^2}{2g}$$

where:

h = water depth with respect to the channel bottom;
z = altitude of the channel bottom.

5) The slope of the energy line is calculated at the check section through the ratio:

$$J_c = \frac{Q^2}{C_{tot}^{\ 2}}$$

6) A first value of the water depth (h_v) for the section to be checked is assumed. One can assume the same value of the check section.
7) The Coriolis coefficient of the section to be checked is calculated, using the same procedure seen for the check section.
8) The slope of energy line of the section to be checked is assessed by the following formula:

$$J_v = \frac{Q^2}{C_{tot}^{\ 2}}$$

where C_{tot} is referred to the section to be checked.
9) The depth of the energy line of the section to be checked is calculated by the formula below:

$$E_v = E_c + \frac{1}{2}(J_v + J_c)\Delta x$$

10) The depth of the energy line for the estabilished value h_v is given by:

$$E'_v = h_v + z_v + \frac{Q^2}{2gA_v^{\ 2}}$$

where:
z_v = altitude of the channel bottom of the section to be checked;
A_v = water area of the section to be checked corresponding to the water depth h_v.
11) The difference between E_v‘and E_v is calculated. If this one is lesser than a few mmm, the check is to be considered satisfied and h_v is the final water

depth. Vice versa a correction Δy, to be applied to h_v has to be calculated. Δy is given by:

$$\Delta y = \frac{E_v' - \left[E_c + \frac{1}{2}\left(J_c + J_v\right)\Delta x + k \left| \alpha_v \frac{v_v^2}{2g} - \alpha_c \frac{v_c^2}{2g} \right| \right]}{1 - \alpha_v \frac{Q^2 b_v}{g A_v^3} \pm k \alpha_v \frac{Q^2 b_v}{g A_v^3}}$$

with:

k = coefficient which measures the energy loss due to the expantion or contraction of the flow (e.g. because a variation of the section area) , ranging from 0.1 to 0.3 for the contracting flow and from 0.3 to 0.5 for expanding flow; to the highest value correspond the sharpest section changes;

b_v = top width of the section to be checked..

12) A new corrected value of h_v is given by the sum of the previous value of h_v and Δy, then repeating the calculation sequence by the point 7.

Warning: the coordinates of the two section profiles have to be referred to the same reference plane.