University of Madeira
Faculty of Exact Sciences and Engineering
Curricular Unit: Hydrology, Water Resources and Environment
Course: Bachelor/1st Cycle of Bologna - Civil Engineering

HYDROLOGY, WATER RESOURCES AND ENVIRONMENT

AUTHORS:

Prof.º Dr.º Sérgio António Neves Lousada
Eng.º Rafael Freitas Camacho
Eng.º Adhonys Alexander Rincon Rodrigues

Technical Specifications

Title	Hydrology, Water Resources and Environment: Theorical Classes
Authors	Sérgio António Neves Lousada Rafael Freitas Camacho Adhonys Alexander Rincon Rodrigues
Editor	University of Madeira
Edition	1st
Year of edition	2019
Volume	I
ISBN	978-989-8805-53-9
Support	Electronic
Format	PDF

Index

Figure Index

Table Index

List of Symbols and Abbreviations

SYMBOLS

!	Factorial
$\bar{P}_S$	Average extension of a superficial flow
C_f	Adjustment coefficient as a function of the return period
C_{mass}	Coefficient of massiveness
C_o	Orographic coefficient
D_h	Stream frequency
D_m	Average slope of a watercourse
D_r	Drainage density
$\bar{H}$	Average basin height
I_d	Slope index
K_C	Compactness coefficient
K_L	Elongation ratio
K_f	Form factor
L_D	Directrix length
L_b	Basin length
L_e	Equivalent length
L_t	Total length of watercourses
$\bar{P}$	Average precipitation
R_h	Hydraulic radius
R_b	Bifurcation ratio
R_i	Radius of Influence
$\bar{Z}$	Average altitude of the basin
Z_{eq}	Equivalent watercourse height
i_{10-85}	Watercourse Slope 10-85
i_q	Equivalent slope of a watercourse
i_{relevo}	Relief index
l_e	Equivalent width
n_r	Porosity
t_c	Time of concentration
$\bar{u}$	Average speed
Δh	Charge losses
h	Height
A	Area
C	Coefficient of the rational formula that depends on the type and the occupation of the soil of the basin
D	Useful rain
E	Field capacity
F	Infiltration capacity
Fr	Froude Number
H	Uniform height
I	Slope; Precipitation intensity
K	Coefficient of permeability
L	Distance
M	Ratio between the waterproof area of a basin and its total area

N	Number of streams
Q	Flow rate
Re	Reynolds number
S	Storage; Degree of saturation; Sinuosity
T	Transmissivity
U	Characteristic speed
V	Volume
Z	Elevation or height
c	Hydraulic resistance
g	Gravitational acceleration
i	Slope
m	Thickness of the aquifer; Mass
q	Specific flow rate
r	Distance until pumping hole
s	Lowering
t	Time
v	Velocity
x	Coordinate; variable
y	Height; coordinate; variable
z	Ghyben-Herzberg ratio
η	Yield
θ	Volumetric content
ρ	Density

ABBREVIATIONS

AMC	Antecedent Moisture Condition
ANPC	*Autoridade Nacional de Proteção Civil*
CN	Curve Number
USA	Unite States of America
GIS	Geographic Information System
HU	*Hidrograma Unitário*
HUT	*Hidrograma Unitário Triangular*
IDF	Intensity-Duration-Frequency
NA	*Nível da água*
PMP	*Precipitação Máxima Provável*
RAM	*Região Autónoma da Madeira*
SCS	Soil Conservation Service
IS	International System
SIG	*Sistema de Informação Geográfica*
SNIRH	*Sistema Nacional de Informação de Recursos Hídricos*
US	United States
USBR	United States Bureau of Reclamation
USD	United States Dollar

CHAPTER 1 - HYDROLOGY, WATER RESOURCES AND ENVIRONMENT

1.1 Introduction

Hydrology is the study of the water, according to a “lato” sense, from Greek *Hýdro* (water) + *Lógos* (study, knowledge). It is the science that describes multiple aspects related to water, as for example: study frequency, distribution and circulation of water on earth, its physic-chemicals characteristics and its interactions with the environment, including its relationship with all living beings. One of the main objectives of hydrology is to serve as a base to manage all water resources in its multiple branches:

- Selection of sources for water supply;
- Design and dimensioning of art works;
- Hydroelectrical and irrigation exploitation;
- Dams construction and spillways dimensioning;
- Study of groundwater characteristics;
- Study of flow rate variations, prediction of maximum floods;
- Study of the variation of level in flood areas;
- Control of erosion and fluvial regularization;
- Recreation and leisure.

Hydrology must be boarded as a multidisciplinary subject (Figure 1) and can be separated in two equally important parts: Superficial Hydrology and Underground Hydrology or Hydrogeology.

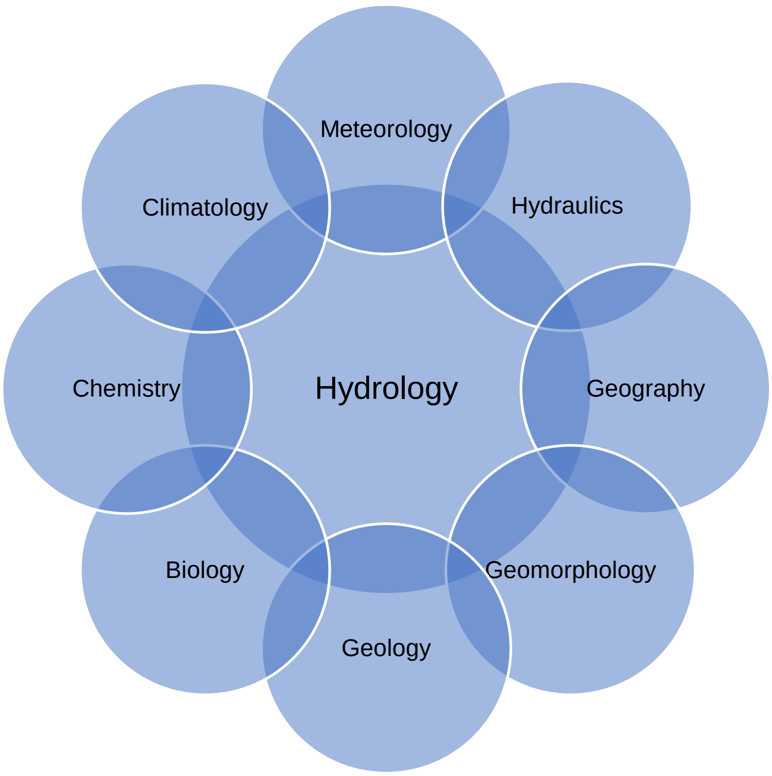

Figure 1 – Hydrology: Multidisciplinary subject.

The importance of hydrology extends mainly to:

- Importance of water to live (consumption, food production, energy production, movement, etc);
- Importance and necessity of a quality environment and healthy ecosystems, which depend from water;
- But water can also be a great risk (for example: Floods and droughts, health risks, etc).

In this way, hydrologists are those scientists that ensure and holds necessary knowledge to have a good management of this vital resource.

1.2 Water: availability, consumptions and utility

The hydrosphere (all water composition of the earth, including lakes, rivers, see, oceans, groundwaters, air moisture, etc.) makes the earth a unique planet. The planet is located at a position with a certain distance for the sun, which allows water to exit in three states, solid, liquid and gas.

As Ricardo Arnst said: “Seen from afar the earth is pure water, but it is not pure water, it is rare and increasingly expensive”. In fact, the available water for human consumption represent less than 1% of the water resources of the planet and more than 1 200 million has no access to safe, clean and potable water. The population growth will increase water problems with an exacerbate demand due to such growth and industrialization.

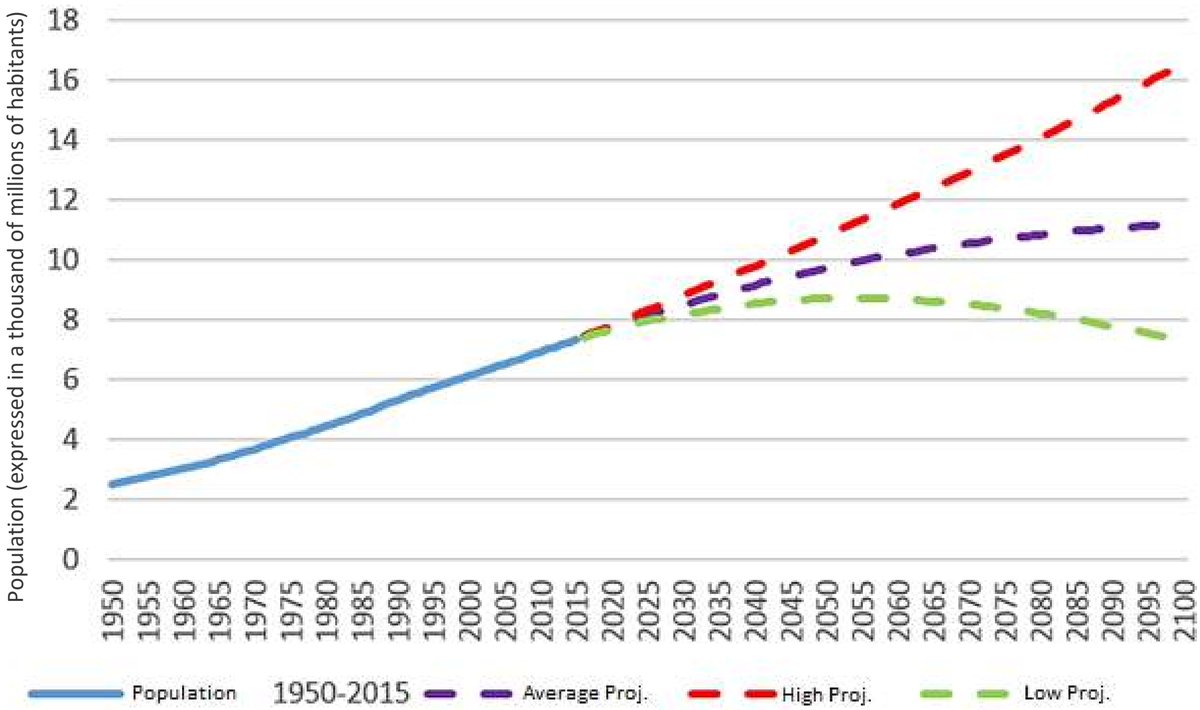

Figure 2 – World population 1950-2015 and scenery of projection for 2100 (UN/SA Population Division, 2015).

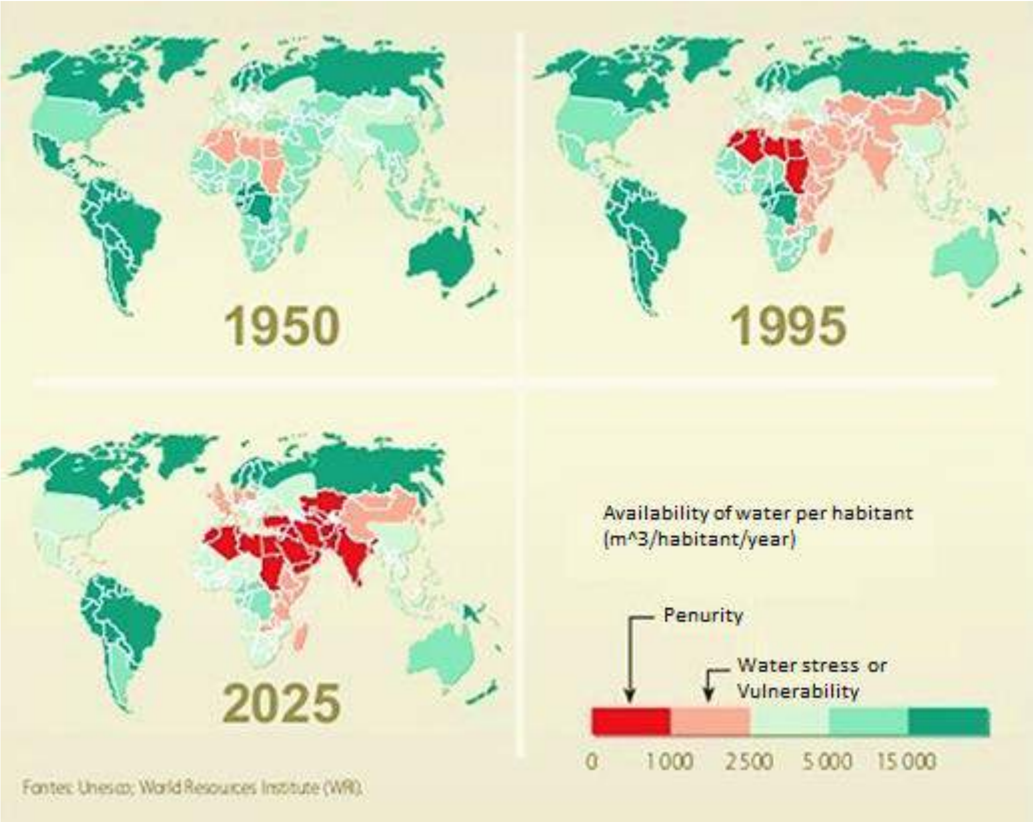

Figure 3 – World: availability of water per habitant, 1950, 1995 and 2025 (Armand Colin, 2006).

1.2.1 Consumption and water utilization

The world consumption in the last 100 years has abruptly increase (Table 1), either due to the vertiginous increase of population or due to its utilization (Table 2), which is every time in a higher scale, associated to the progress of society.

Table 1 – Annual consumption in cubic miles (adapted from Blatt, 1997).

Region	1900	1950	2000	Increase (nº of times)
Africa	10	13	80	8
North America	19	69	191	10
South America	3.6	14	52	14
Asia	99	206	800	8
Australia/Oceania	0.5	2.4	11	22
Europe	9	23	162	18
Total	141.1	327.4	1292	9

Table 2 – Utilization of water (adapted from Cunningham & Saigo, 1995).

Domestic Sector	Liters	Agriculture and food processing	Liters	Industry	Liters
Bath	150-200	**1 egg**	150	**1 newspaper**	1000
Shower	20/min	**1 corn cob**	300	**1 car**	380000
Laundry	75-100	**1 bread**	600	**500 g of steel**	110
Food confection	30	**500 g of cow meat**	3000-9500	**500 g of synthetic rubber**	1100
Garten irrigation	40/min	**1 glass of milk**	380		
Flush of toilet	10	**500 g of rice**	2100	**500 g of aluminium**	3800

According to Possas (2011), the uses of water can be separated in two categories:

- Consumptives – refers to those types that retire water from its natural source decreasing its spatial and temporal availability (agriculture, industry, domestic and municipal, livestock and fish farming);
- Non-consumptives – refers to those uses that return to the sources most of the used water but including some changes in its standard temporal availability (ecologic/environmental, navigation, production of energy, playground and tourism).

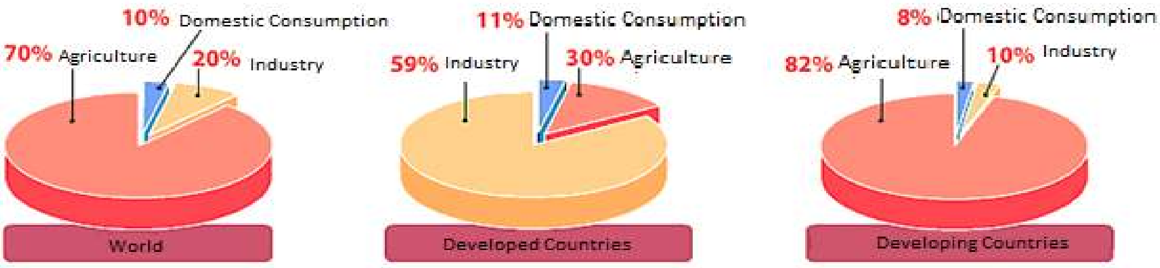

Figure 4 – Sector consumptions (adapted from Water for People, Water for Life, UNESCO, 2003).

1.3 Time of Residence

Time of Residence (sometimes treated as time of removal) correspond to the average time in which each molecule of water remains in a certain reservoir of the hydrologic cycle and varies directly with the amount of water that it is present in the system. (Montgomery & Reichard, 2007).

According to Montgomery & Reichard (2007), the basic definition for time of residence is also a mathematical equation, which is represented through:

$$\frac{System\ capacity\ of\ water\ retention}{Rate\ of\ flow\ of\ water\ in\ the\ system}$$

The form of the generic variables of such equation is:

$$\tau = \frac{V}{q}$$

where τ is used as a variable of time of residence, V is the system capacity and q is the rate of flow for such system.

The time of residence begins when a particle of water enters into the system and finish when such particle leaves the system. If a great amount of water enters into the system, it will be necessary a greater time to leave such system (a mayor time of residence), when supposing that the input and output of water remains constant. For the same logic, for a minor amount of water, then a lower time of residence (Montgomery & Reichard, 2007).

Inputs and outputs of flow have also an effect in the time of residence of a system. If the input and output are increased, the time of residence will be lower and vice versa, supposing that the water concentration and the size of the system remains constant, and by assuming conditions of a stationary state (refers to a system in which a determined parameter is maintained constant along time) (Montgomery & Reichard, 2007).

If the size of the system is change, the time of residence is also affected. As greater the system, major the time of residence and vice versa, when rates of input and output flows are assumed constant. Such conditions average a stationary state (Montgomery & Reichard, 2007). In Table 3 shows the average time of residence of water, according to the reservoir.

Table 3 – Average time of residence in different reservoirs (adapted from Voskresensky, 1974).

Reservoir	Average time of residence
Biosphere	Hours to days
Atmosphere	8 days
Rivers	16 days
Soil moisture	1 year
Lakes and swamps	5 to 17 years
Groundwater	1400 years
Oceans	2500 years
Polar caps and glaciers	9700 years

CHAPTER 2 - HYDROLOGIC CYCLE

2.1 Introduction

The **Hydrologic cycle** is a closed circuit of all processes of water movement between ground and atmosphere(IST, 2018).

Along such cycle, water **evaporates** from oceans and earth surfaces, then enters in the atmospheric circulation as vapour to lately return to earth surface as liquid or solid **precipitation**, as soon as reach a terrain surface, start **"flowing"** over it, then **infiltrate** the soil allowing **aquifers reload,** concentrate as a canalized flow in the fluvial web that guides water back to the oceans, where the cycle starts again (Figure 5) (IST, 2018).

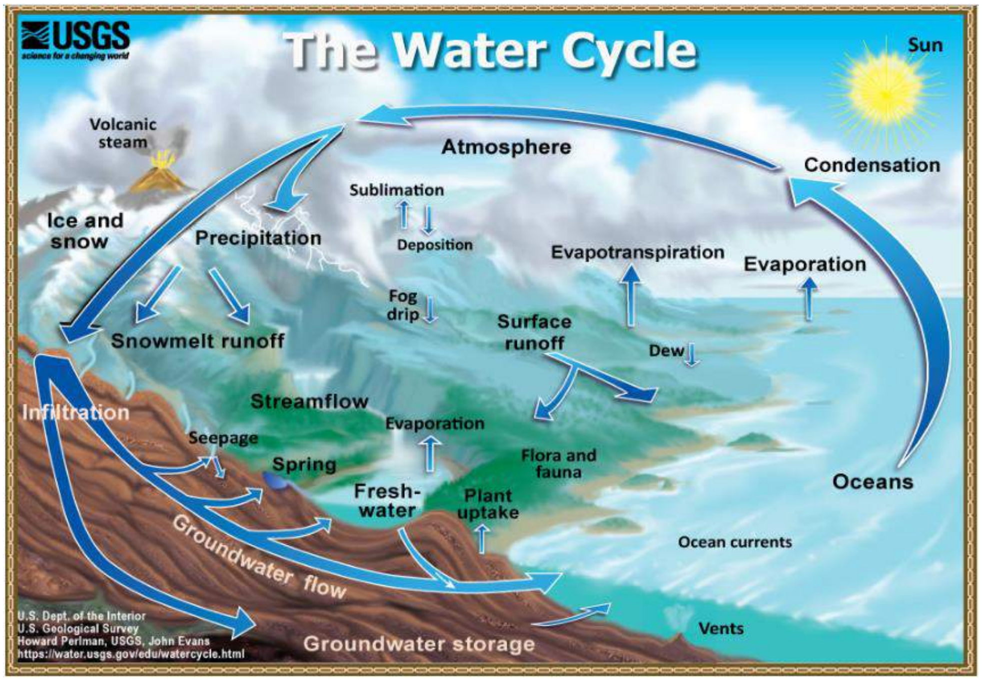

Figure 5 – Hydrologic cycle (source: http://ga.water.usgs.gov/edu/watercycle.html).

Such permanent and uninterrupted regime of water movement in the hydrologic cycle is maintained by **solar energy** and by **gravitational energy**. The total amount of water in earth is constant and approximately $1400x10^{15}$ m^3.

It stands out the main processes involved in the hydrologic cycle (IST, 2018):

- Transfer water from earth surface to atmosphere by **evaporation** of oceans, lakes, rivers, soil, by **sublimation** of ice and by **transpirations** of animals and plants;
- **Partial Condensation** of water steam from atmosphere (in clouds and mist);
- Transportation of water steam by atmospheric circulation;
- Transfer of water from atmosphere to earth surface (liquid or solid);
- Infiltration and aquifers reload;
- Retention in lakes, glaciers and vegetation;
- Flows in continents surfaces oriented to oceans.

2.2 Global hydrological balance

Due to the fact, that the total amount of water available in Earth is finite and indestructible, makes sense to assume the hydrologic cycle as a closed system or circuit. A more generic version of hydric balance could explain the different components of such cycle and provide an idea of technics to solve problem in hydrologic complex regions (Porto & Filho, 2005).

Every year is moved an approximately total of 624 000 km^3 of water, promoted by the hydrologic cycle (Figure 6).

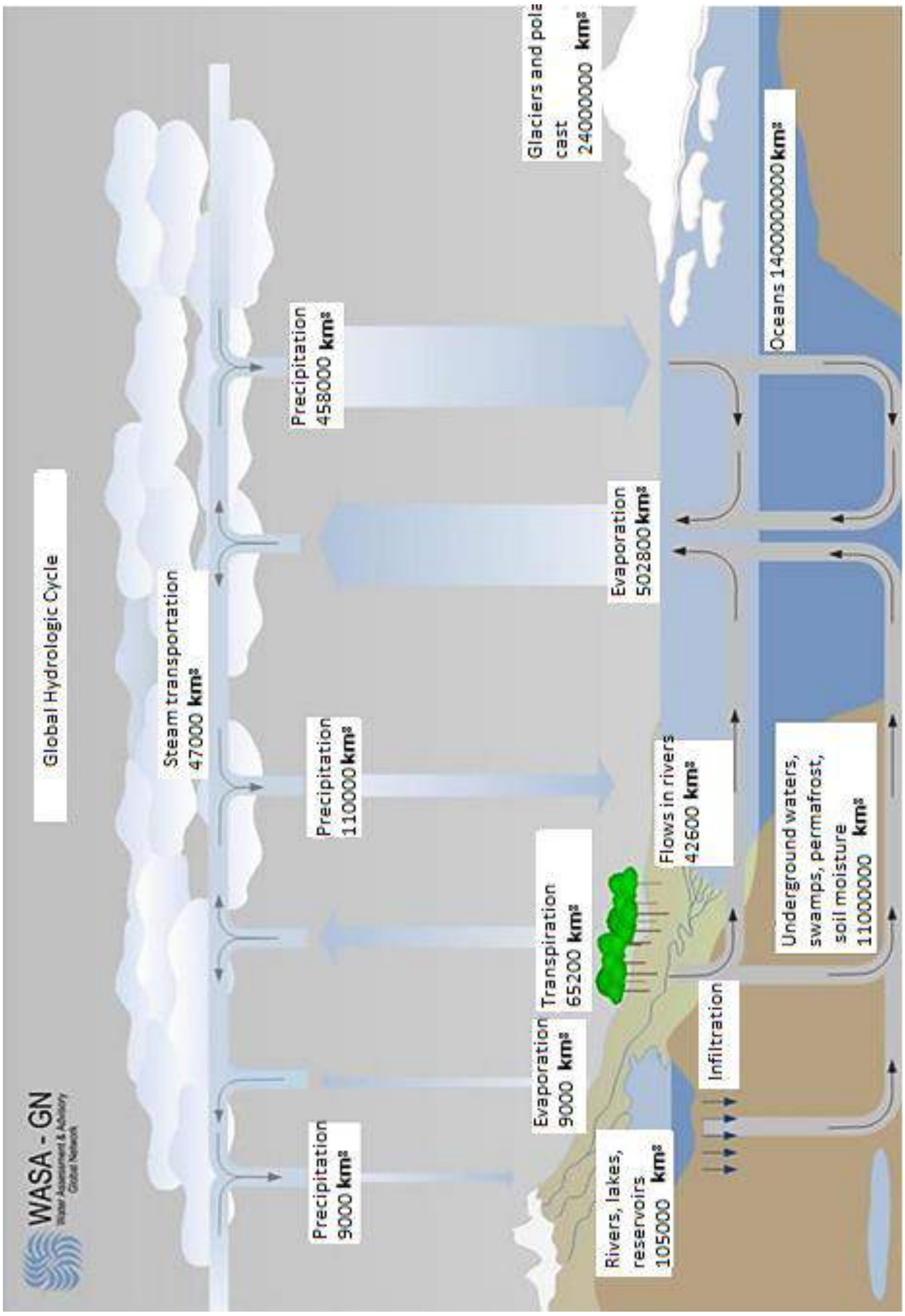

Figure 6 – Annual global hydrologic cycle (adapted from WASA - GN).

The referred hydrologic magnitudes in different intervals of time are frequently expressed in heights of water (H) uniformly distributed over the horizontal projection of areas (A) of the volumes (V) of such referred magnitudes. The fact that most of the hydrologic magnitudes are expressed in terms of the corresponding height of water gives advantages to offer a more perceptive physical evaluation, facilitating the comprehension of the transmitted information (Figure 7) (IST, 2018).

$$H \sim \frac{V}{A}$$

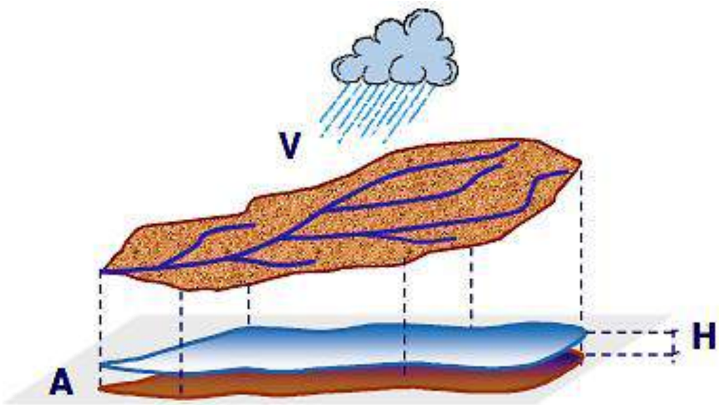

Figure 7 – Hydrologic magnitudes (adapted from IST, 2018).

Note (IST, 2018):

- mm, is equal to the height of water in a square meter;
- 1 mm, is the same as 1 mm of height in 1 m^2 of area;
- mm = l/m^2.

CHAPTER 3 - HYDROGRAPHIC BASIN

3.1 Introduction

According to Rodrigues et al. (2011), a **Hydrographic Basin** of a course of water, it is the natural caption of precipitated water, which its flow converge into a unique output section - Section of reference (Figure 8).

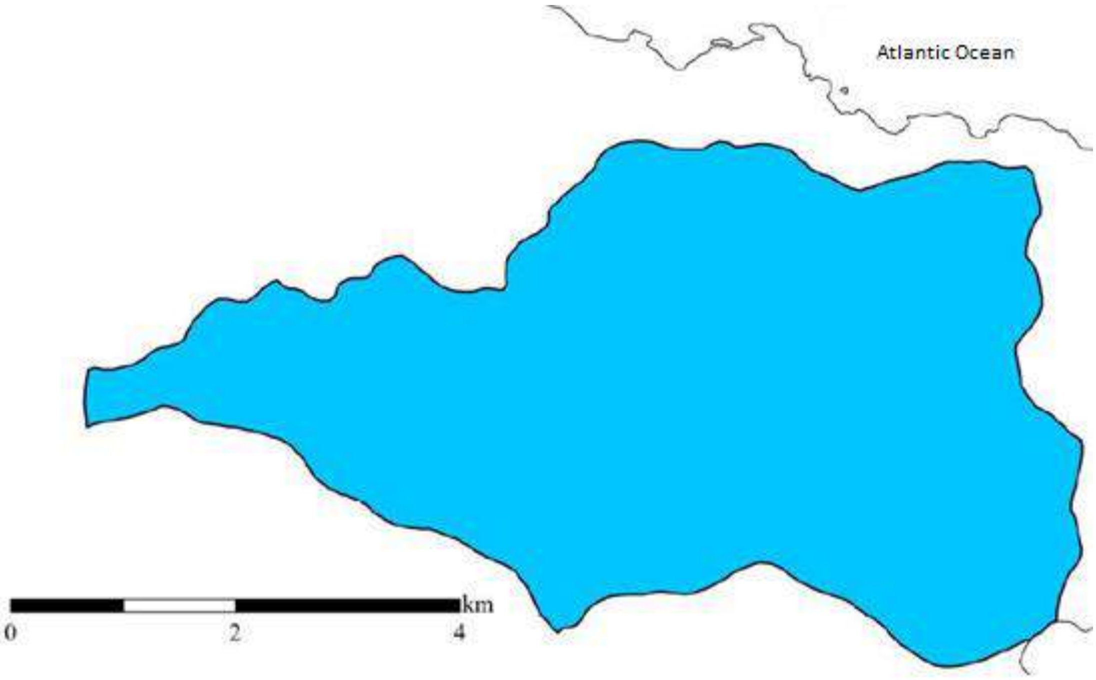

Figure 8 – Machico Hydrographic basin (adapted from Barreto, 2013).

A hydrographic basin is always referred to any section of a course of water. When a section is not indicated in a research, it is assumed the totality of the basin, in relation to the river mouth or confluence with another, but more significative course of water. As for example: hydrographic basins of Zêzere river, Mondego rive or Machico.

The rainwater or precipitation that falls on the hillsides tents to totally infiltrate the soil until superficial saturation. The rate of infiltration decreases and if the precipitation does not stop, then the superficial flow increase (oriented to the hydrographic web). Then, water is transported until the output section. In the section of reference, the resultant hydrograph includes not only the superficial flow, but also underground contribution which is delayed in time relatively to the precipitation occurrence (Rodrigues et al., 2011).

3.2 Delimitation of the hydrographic basin

Based in Rodrigues et al. (2011), in natural waterproof terrains or artificial waterproofed terrains, the limits of the hydrographic basins coincide with ridge lines (“water separation lines” or ridgeling lines). In permeable soils, the existence of underground flows turns such delimitation less linear. In situations in which the existence of volcanic conditions or karst formations, the topographic contour line-superficial ridge lines can significantly differ with the underground line of water separation (Figure 9).

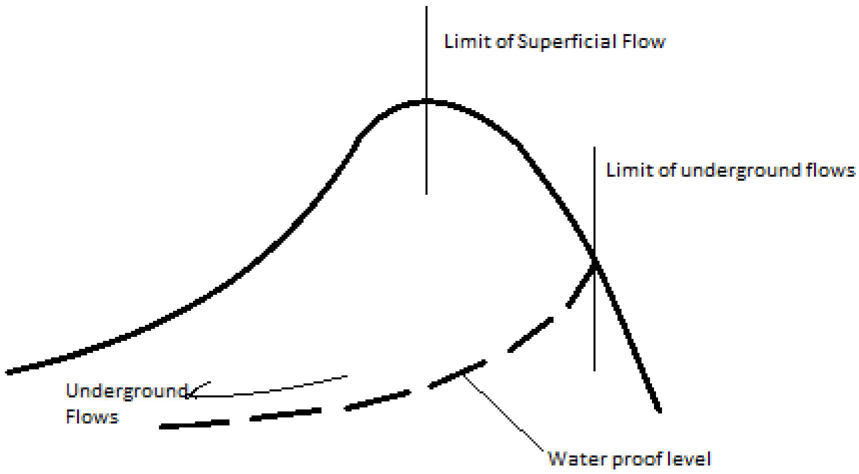

Figure 9 – Limits of the superficial and underground flows (adapted from Rodrigues et al., 2011).

In basin of small dimension, the contribution for flows of near basins as a result of the non-coincidence between the superficial and groundwater separation lines can be perceptually significative. In great or big basins, the importance of the contributions or losses of resultant flows is exactly the opposite, usually small and not so significative.

In practical terms and in order to facilitate, the delimitation of the hydrographic basins is done only by the topography of terrains. In such process must be respected a few rules:

- An adequate scale. For example, for a basin of 1000 km^2 of area, a scale of 1:25000 will be enough, but for smaller basins, a scale of 1:10000; and for majors, a scale of 1:50000;
- The outline line (or divisor line) must perpendicularly cross contour lines;
- In the pass of one contour line to another, if altitude increase, then the outline line crosses the contour line by it convex part, but if altitude decrease, the contour lines are crossed by its concave part;
- The divisor line cannot cross water courses (except in the position of the section basins reference).

3.3 Physiographic characteristics of a hydrographic basin

Such characteristics are considered as those elements that can be taken from maps, aerial photos or satellite images. It is only relevant to characterize the basin in geometric terms, related to the drainage system and relief. It is equally important the conditioning aspects of the basin behaviour, as for example, its geologic constitution, type of soil and predominant vegetation. Then, such characterization allows to find affinities between different hydrographic basins and consequently, collect and classified some data and hydrologic parameters according to regions (Rodrigues et al., 2011).

3.3.1 Geometric Characteristics

The main geometric characteristic consider is **area of drainage** (A) (Km^2 or hectares). This one must result from the horizontal projection of the basins, once its contours are defined. The characterization of the **form** of the hydrographic basins is related with its major or minor tendency to concentrate a superficial flow as a reaction to intense precipitations. The planimetric form of the basins can have a great influence in the regime of the course of water, mainly in the flow rate of floods. (Rodrigues et al., 2011).

An elongated basin or longitudinal basin is constituted by a unique principal line of water of great length, but with a small width and receiving tributary lines of insignificant importance (Figure 10).

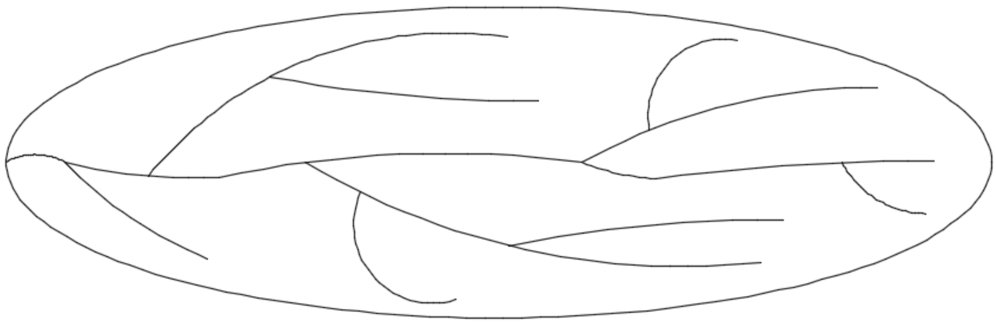

Figure 10 – Elongated or longitudinal basin (source: author).

A rounded basin is formed by various water lines of sensitively similar importance that concentrically converge, which leads to origin a new final water line, relatively short (Figure 11).

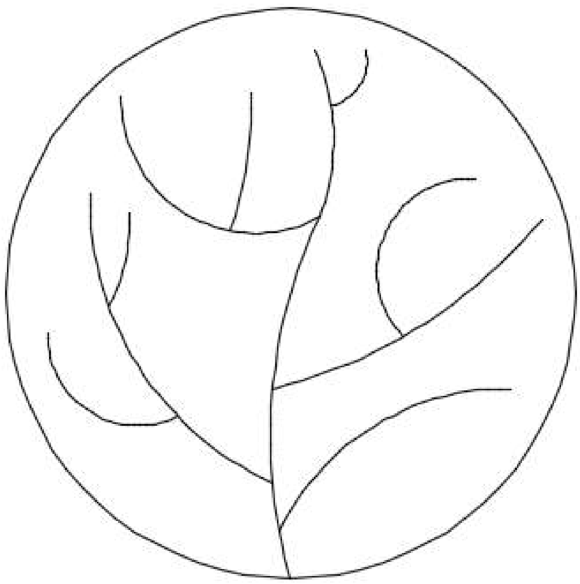

Figure 11 – Rounded basin (source: author).

A radial basin or branched basin has various partial elongated basins that meet at a final water line (Figure 12).

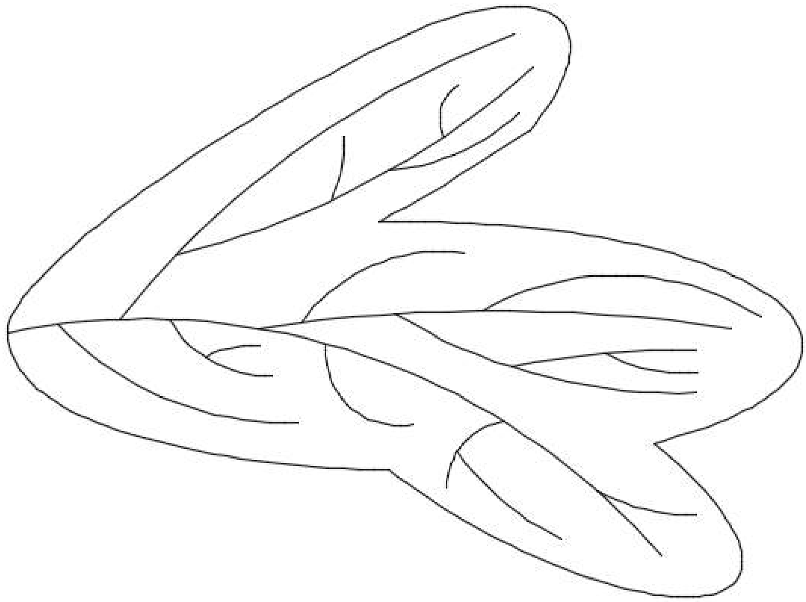

Figure 12 – Radian basin or branched basin (source: author).

Verifying the equality of all other conditions, the flow rate of flood of elongated basins will be lower than the rounded ones.

In rounded basins, the concentration of water occurs faster, which lead to a maximal flow rate of a major flood, but of shorter duration.

In a rounded basin, the occurrence of floods is more accentuated due to its form (favour a major concentration for the flow), which it is opposite for an elongated basin.

Based in Costa & Lança (2011) and Rodrigues et al. (2011), in sense to quantify the form of the basins, it possible to be supported by some quantitative index:

1. **Compactness coefficient (or Gravelius coefficient), $\boldsymbol{K_c}$** – ratio between the perimeter of a basin (km), P, and the perimeter of a basin with equal area (km^2), A, but with rounded shape. The area of the perimeter of a circular basin will be respectively, $A = \pi r^2$ and $P = 2\pi r$, so:

$$K_c = P/2\sqrt{\pi \times A}$$

Such coefficient is dimensionless, and its value does not depend of the basins size. IT is minimally equal to a unit, which correspond to a circular basin. Therefore, equally to other factors, the tendency for great floods is more accented in basins with a K_c proximal to a unit value. For values of K_c equals or lower to 1.13 represent generally rounded basins.

2. **Elongation ratio, $\boldsymbol{K_L}$** – considering a rectangle equivalent to the basin of study, this factor represents the ratio between length, L_e, and width, l_e, of the same equivalent rectangle; a basin is consider elongated for values higher than 2.

$$K_L = L_e/l_e$$

With:

$$L_e = \frac{K_c\sqrt{A}}{1,128} \cdot \left| 1 + \sqrt{1 - \left(\frac{1,128}{K_c}\right)^2} \right|$$

$$l_e = \frac{K_c\sqrt{A}}{1,128} \cdot \left| 1 - \sqrt{1 - \left(\frac{1,128}{K_c}\right)^2} \right|$$

3. **Form factor, K_f** – represent a ratio between average width l and the length of a basin L_b. The average width of a basin is defined by a ratio between length and area of a basin, A. So, K_f will be:

$$K_f = l/L_b = A/L_b^2$$

A basin with a low Form factor prone less to floods than other one of same size but higher Form factor, because for an elongated basin there is a lower possibility to have a flood, due to the difficulty of intense rain along the basin at same time. On the other hand, the contribution of each effluents does not occur at the same time, reducing critical flow rate of flow. K_f will maximum achieve 1 (um), which correspond to a square basin.

3.3.2 Characteristic of Drainage System

Flow Record or Consistency

Courses of water can be classified in three categories according to its regime of flow: **perennial, intermittent and ephemeral** (Rodrigues et al., 2011).

Perennial course of water flow along whole year. In conditions of drought, such flow is maintained by underground reservoirs, which keeps continuously supplying such course of water, even at severe conditions (Figure 13) (Rodrigues et al., 2011).

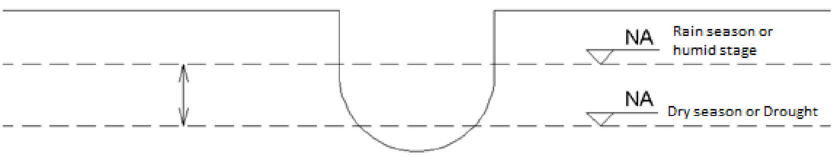

Figure 13 – Perennial course of water (source: author).

An **intermittent** course of water has flows only at humid season (but dry at drought season). Usually, during the humid stage, the soils water level rises higher than the riverbed, leading to a flow, but during the drought stage, the soils water level falls under the riverbed, so the flows ends. In such type of water courses, it is possible to have rare flows in or out of season due to an unusual rain. (Figure 14) (Rodrigues et al., 2011).

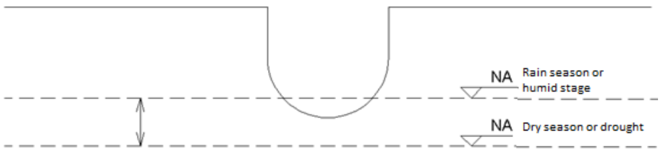

Figure 14 – Intermittent course of water (source: author).

An **ephemeral course** of water are those types in which only exist a superficial flow as a reaction of a precipitation event. Its flows periods are quite short and happen only as soon as the precipitation occurs or after it. There are not groundwater contributions in such kind of flows, because the soils water level never reaches the rivers bed (Figure 15) (Rodrigues et al., 2011).

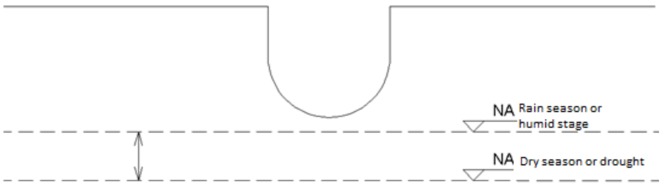

Figure 15 – Ephemeral course of water (source: author).

Classification of water courses

The need of a quick recognition and mapping of water courses of a hydrographic web, in some region or country had led to an establishment of classifications related to its grade of branching or bifurcation. The most used are (Rodrigues et al., 2011).

1. Strahler's stream order

Course of water are ordered according the grade of branching or bifurcation of a hydrographic basin. According to Stralher classification (Figure 16), it must be considered:

- Courses of water without any tributary are 1º order;
- When two courses of water of the same order or grade converge, then the order or grade increase one unit. In an opposite case, the mayor grade or order is assumed.

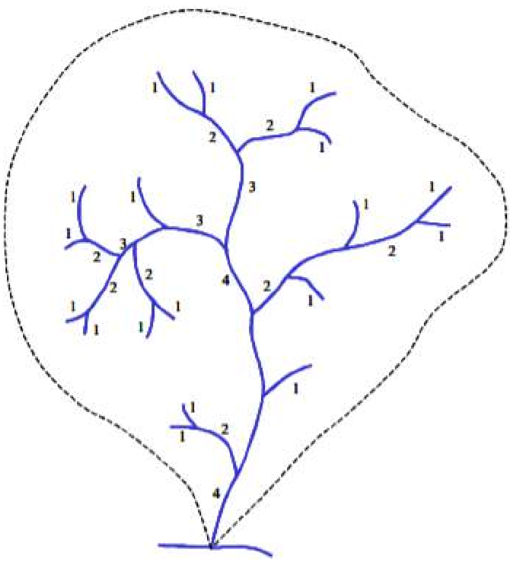

Figure 16 – Application of Strahler's Method (adapted from Rennó & Soares, 2003).

The bifurcation ratio (R_b) is defined as a ratio between the number of channels of a certain order or grade (N_i) and the number of channels of an following superior order or grade (N_{i+1}), which generally vary between 2 and 4.

$$R_b = N_i/N_{i+1}$$

So, for a given basin of order i, $i-1$ values of R_b can be determined according to next table:

Table 4 – Bifurcation ratio - example.

Order, i	N_i	R_b
1	18	2.6
2	7	3.5
3	2	2.0
4	1	-
-	$\overline{R_b}$ =	2.6

The average values of each i basin R_b's, represent an average bifurcation ratio for the basin.

$$\overline{R_b} = \sqrt[i-1]{\prod_{i=1}^{i-1} \frac{N_i}{N_{i+1}}} = \sqrt[i-1]{N_1}$$

2. Shreve's stream order

Shreve's stream order is similar to Strahler's, but it differs in an aspect:

- All magnitudes are added every time two lines converges. For example, when two lines of second order converge, the resultant line has an order of 4. In this way, some magnitudes or orders may not exist. Figure 17 shows how this method is applied to a drainage system of a basin.

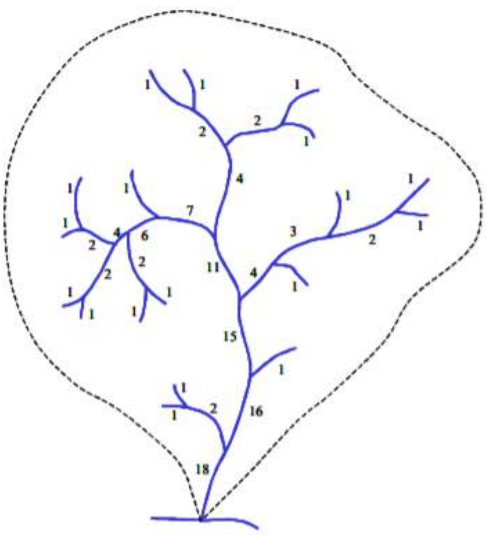

Figure 17 – Application of Shreve's Method (adapted from Rennó & Soares, 2003).

Time of concentration

Time of concentration of a basin, t_c, is the necessary time to have a fully contribution for a superficial flow at output section; it can be also defined as the necessary time for a drop of water to reach the output section from the furthest hydraulic point of a basin. It is considered as a constant characteristic of the basin independently of the precipitation characteristics (Mata-Lima, et al., 2007).

For floods study, the interval of intense precipitation to consider must be at least equal to the time of concentration, such parameter is called “Critical interval” or “Critical duration”. Such requisite is made in order to ensure that all contributing area of a hydrographic basin is consider at the reference section. However, in order to avoid oversized designs must be considered time of concentrations higher than 5 minutes, which correspond to the minimal possible interval obtainable from udograms of 24 hours (Those one used for IDF curves formulation) (Lencastre & Franco, 2006; Mano, 2008).

Different expression used to define time of concentration at a certain section (selected point of a basin) are presented as follow.

1. Témez

Témez’s formula was tried in American and Spanish basins, obtaining result close to reality. This and the formula of Giandotti are the most used in Portugal. Such formula is:

$$t_c = (L/i^{0.25})^{0.76}$$

where:
t_c - time of concentration (h);
L - main stream length (km);
i - main stream’s average slope.

However, if it is necessary the time of concentration for a certain section, then it is rewrite as follows:

$$t_c = 0.3 \times (L/i^{0.25})^{0.76}$$

(Pelaez, 1978) refers that such formula can be used in urban basins, if those zones are dispersed in the hydrographic basin and not concentred, so it is necessary to adjust time of concentration because the urbanization favours the superficial flow. To consider such factor, Témez proposed an adjustment of the time of concentration for urban basins, t_c', with the following expression:

$$t_c' = \frac{t_c}{1 + 3 \cdot \sqrt{\mu \cdot (2 - \mu)}}$$

where t_c (in hours) is the time of concentration calculated for natural basins and μ (dimensionless) is a parameter related to the grade of waterproofing of the basin. Being μ given by the ration between the waterproof area and the total area, so:

$$\mu = \frac{A_{waterproof}}{A_{total}}$$

In Table 5, As an example, it shows the value of the parameter, μ, and the respective urbanization level considered.

Table 5 – Urbanization level according to a parameter μ (adapted from Pelaez, 1978).

Urbanization level	μ
Small	μ < 0.05
Moderate	0.05 < μ < 0.15
Big	0.15 < μ < 0.30
Highly Developed	μ > 0.30

Limitations:

- This expression is valid for basins of area lower than 3000 km^2.

2. Ven Te Chow

$$t_c = 0.8773 \times \left(L/\sqrt{i}\right)^{0.64}$$

where:
t_c - time of concentration (h);
L - length of the largest waterline of the basin (km);
i - main stream's average slope.

Limitations:

- This expression is only valid for basins of are between 1,1 and 19 km^2.

3. Giandotti

$$t_c = \frac{\left(4 \times \sqrt{A}\right) + (1.5 \times L)}{0.8 \times \sqrt{\bar{H}}}$$

where:
t_c - time of concentration (h);
A - area of the basin to a given point (km^2);
L - length of the largest waterline of the basin to a given point (km);
$\bar{H}$ - average height of the basin, measured from the height of the studied section (m).

Limitations:

- No limitations detected.

4. Average time of concentration

The average time of concentration is the average value between every time of concentration obtained with the addressed formulas:

$$\overline{t_c} = \frac{\sum t_{c,i}}{n}$$

Drainage density

According to Rodrigues et al., (2011), to model a drainage web or system of a hydrographic basin, it is necessary to quantify its extension. Such evaluation is done by an index relating the total length, L_t, of water courses (no matter its type, perennial, intermittent or ephemeral) with the corresponding area of drainage, A. Such index is called **drainage density**, $\boldsymbol{D_r}$:

$$D_r = L_t/A$$

This index supplies a good indication of the natural drainage efficiency of a basin. Generically, its values vary between 0.5 km/km^2, for poorly drained basins ,3.5 km/km^2 or more for well drained basins (this one usually belongs to basins with a tendency to flood, because favours water superficial movement in detriment of infiltration).

Stream frequency

This index relates the total number of stream segments of a basin, N, with the corresponding area, A (Santos et al., 2012). It is called **Stream frequency**, $\boldsymbol{D_h}$:

$$D_h = N/A$$

Average extension of a superficial flow

The **average extension of a superficial flow**, $\boldsymbol{\overline{P_s}}$, at some basin, express the average distance in Km, which rainwater must travel until reaching the closest water course (Rodrigues et al., 2011). Its value is approximately a quarter of an inverse of drainage density:

$$\overline{P_s} = A/4L_t \approx 1/4D_r$$

Sinuosity

It is a ratio between the main stream's length, L, and the length of the directrix L_D. **Sinuosity** is a characteristic that controls the velocity of the river (Costa & Lança, 2011).

$$S = L/L_D$$

When equals to a unit, averages that the river is rectilinear.

3.3.3 Relief features

Relief correspond to variations that verifies terrain surface. In this aspect the main features of a basin are declivity of a basin, average height and slope of the main river (Costa & Lança, 2011).

Temperature, precipitation and evaporation assume variable values in function of the altitude of a basin. The velocity of superficial flow is determined by the surface's slope, which shows the importance and need of relief features due to its significative influence in hydrologic factors (Costa & Lança, 2011).

Hypsometry

A hypsometry characterization of a hydrographic basin consists in quantify areas per altitude class in order to stablish the distributions of the respective altimetric frequencies (Rodrigues et al., 2011).

The most common way to do it, it is graphically by representing a function $A = f(Z)$, which express the area of a basin above an altitude Z, expressed in area units or percentage of total area (Rodrigues et al., 2011) – **Hypsometric curve of a basin** (Figure 18).

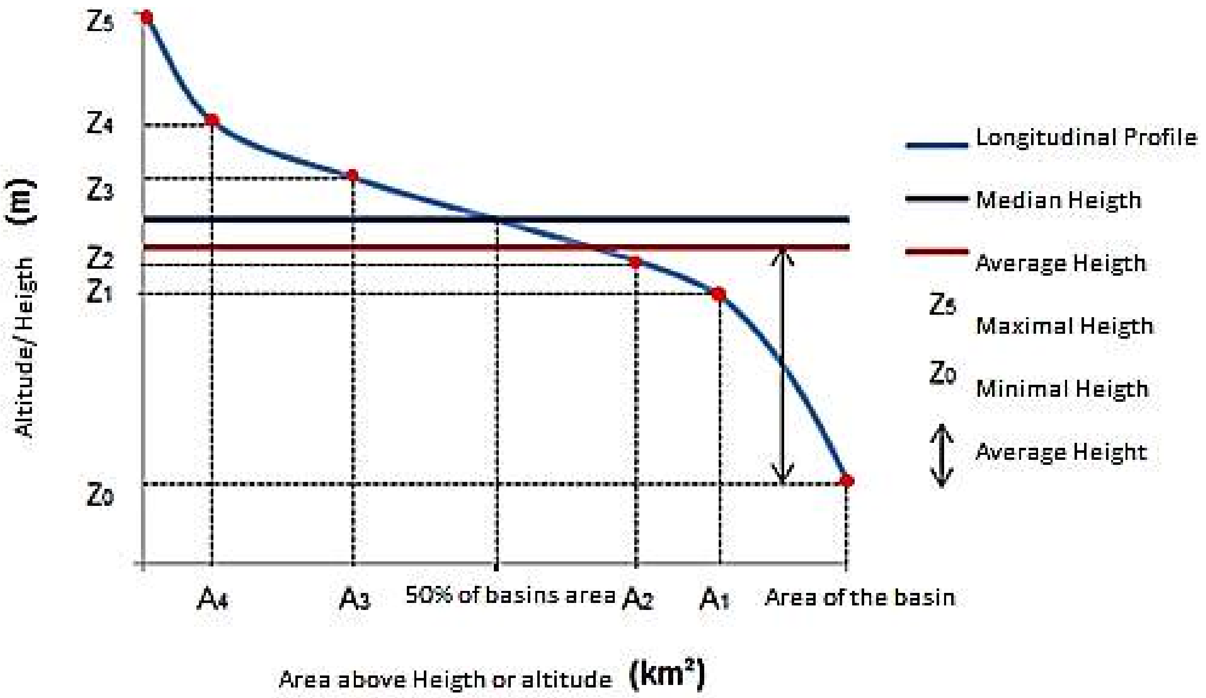

Figure 18 – Hypsometric curve (adapted from Oliveira R. P., 2009).

Nowadays, with help of digital cartography and associated tools associated to Geographic Information Systems (GIS), the study of hypsometry in terrains became much easier due to the existence of hypsometric cartography (Rodrigues et al., 2011). As an example, in Figure 19, obtained from a topographic map of hydrographic basins of "Ribeira dos Socorridos" and "Ribeira do Vigário".

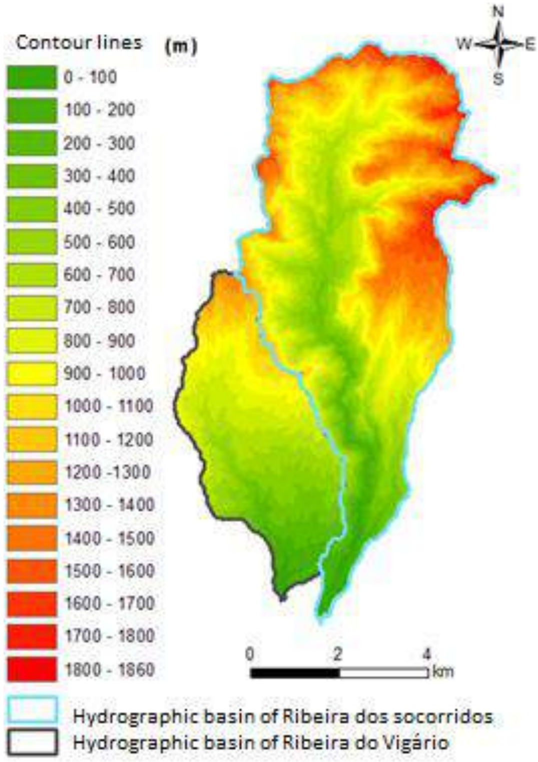

Figure 19 – Hypsometric map of hydrographic basins (adapted from Marques, 2014).

Altitude and average height

According to Rodrigues et al. (2011), the average height $\bar{Z}$ (in meters) is a summation of the products of average height between two consecutives contour lines $\bar{Z_i}$ and the value of the corresponding area, A_i, and then divided by the total area of the basin, A:

$$\bar{Z} = \Sigma \bar{Z_i} \times A_i / A$$

In the same way is defined average height also as $\bar{H}$, considering in that case the original height reference (the height at the section of reference of a basin) Z_{min}. So:

$$\bar{H} = \Sigma \overline{H_i} \times A_i / A = \bar{Z} - Z_{min}$$

Orographic (Massiveness) coefficient

Such coefficient represents a ratio between average height, $\bar{H}$, (in meters) and the total area of the basin, A in square kilometres (Marcuzzo et al., 2012).

$$C_{mass} = \bar{H} / A$$

The orographic coefficient represents a ratio between the square average height $\bar{H}^2$ (in meters) and the total area of the basin, A in square kilometres (Marcuzzo et al., 2012):

$$C_o = \bar{H}^2 / A$$

Average slope of the hillsides of a basin

The magnitude of the floods peaks, the tendency of mayor or minor infiltration and the susceptibility of soil erosion depends of the velocity of a superficial flow in a basin (Costa & Lança, 2011).

A method to determinate the slope of a basin is "Squares associated to a vector". Such method consists in determinate the percentile distribution of terrains slopes by a statistical sampling of slopes, that are normal to contour lines in several points of the basin. (Costa & Lança, 2011).

Such points are localized in a topographic map (of a basin) by using a transparent grid placed on the map. (Costa & Lança, 2011).

For a more precise process to determinate the average slope of a basin, it is represented in the next example:

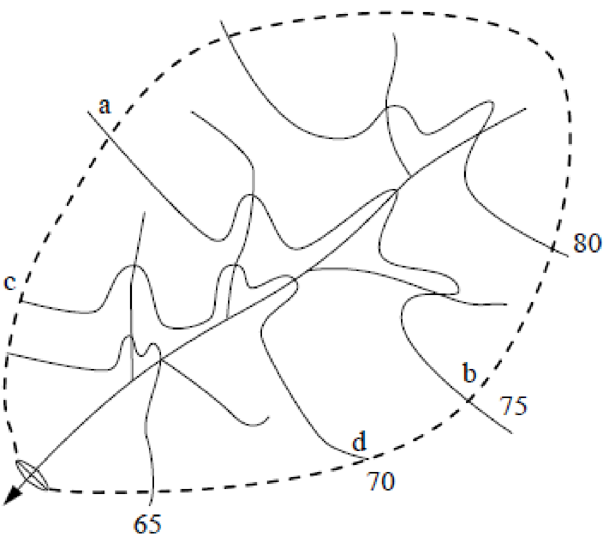

Figure 20 – Average slope of a basin (Costa & Lança, 2011).

where:
a_1 - area of the band $a\ b\ c\ d$;
c_1 - length of the contour line of height 75;
e_1 - Average width of the band $a\ b\ c\ d\ (a_1/c_1)$;
i_1 - Average slope of the band $a\ b\ c\ d$;
I - Average slope of the basin;
D - equidistance between contour lines;
A - Total area of the hydrographic basin;
L - Total length of the contour lines.

$$i_1 = D/e_1 = D \times c_1/a_1$$

Considering weighted average of slopes in relation to areas:

$$I = \frac{D \times c_1}{a_1} - \frac{a_1}{A} + \frac{D \times c_2}{a_2} - \frac{a_2}{A} + \cdots + \frac{D \times c_n}{a_n} - \frac{a_n}{A} \rightarrow I = \frac{D}{A}(c_1 + c_1 + \cdots + c_1) = \frac{D \times L}{A}$$

So, average slope of a hydrographic basin is equal to the product of natural equidistance between contour lines and the total length of it, divided by the total area of the basin (Costa & Lança, 2011). According to the average slopes of the sidehills, relief can be classified according with the following table:

Table 6 – Relief classification (adapted from Costa & Lança, 2011).

Relief	Slope
Plain	0% - 2%
Slightly ondulated	2% - 5%
Ondulated	5% - 10%
Highly ondulated	10% - 20%
Montains	20% - 50%
Very montainous	50% - 100%
Precipitous	> 100%

Longitudinal profile of a River

A longitudinal profile of a water course relates in each point, the height of the riverbed or bottom with the distance of the point until the river mouth. In a graphical trace of the profile is usual to mark most relevant hydraulic works (dams, weirs, confluences, etc.). Figure 21 shows a longitudinal profile of a hypothetic river (Rodrigues et al., 2011).

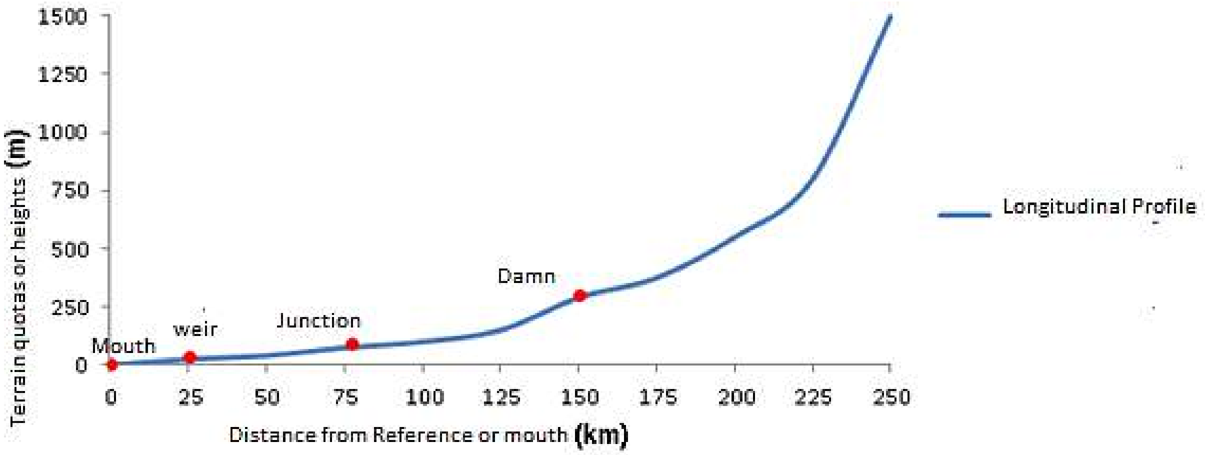

Figure 21 – Longitudinal profile of a river (adapted from Rodrigues et al., 2011).

It can also be obtained from topographic maps with enough contour lines to have a good recognition of the terrain. Topographic charts with contour lines of equidistance of 10 meters can provide a good profile (Costa & Lança, 2011).

According to Costa & Lança (2011), the velocity of flow of a river basically depends in the inclination of the riverbed or bottom. As mayor is the slope of the thalweg, mayor will be the speed of the water.

Slope or inclination between two points of a riverbed or bottom is the ratio between the deepness and the reduced horizon length, which averages “the tangent of the inclination angle”, as shows Figure 22:

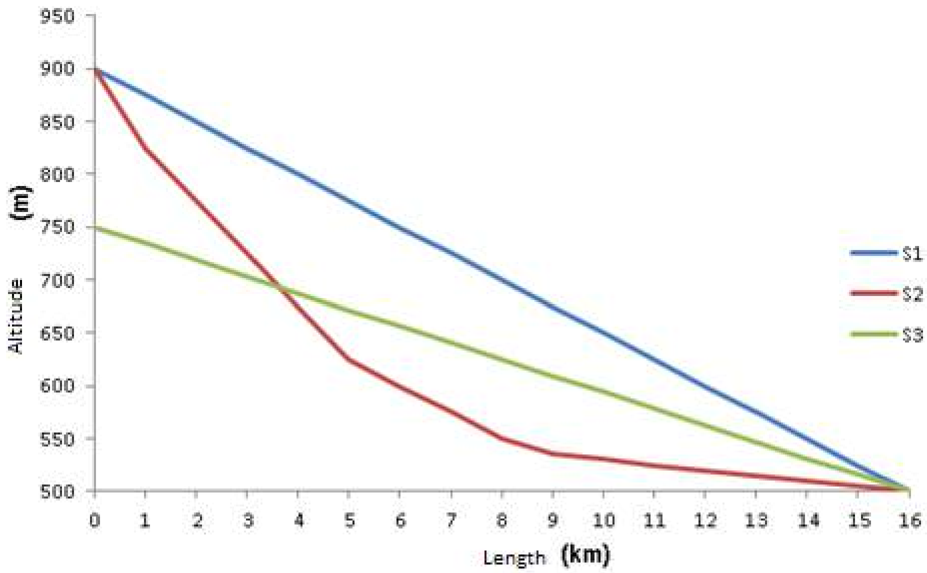

Figure 22 – Longitudinal profile of a river (adapted from Costa & Lança, 2011).

where:
S1 - Joint the fount or source with the mouth, giving maximal slope (always unreal- theory purpose only);
S2 - Average slope. The area of a triangle formed by coordinate axis and the corresponding line of average inclination, it is equal to the area defined by the coordinate axis and the longitudinal area of the profile;
S3 - Constant equivalent slope. Obtained through the harmonic weighted average of the square root of various slopes:

$$S_3 = \left(\frac{\sum L_i}{\sum {L_i}/{S_i}} \right)$$

Average slope of a course of water

This value is obtained by relating a difference of altitudes ΔZ (or height in meters) between the highest and the lowest points of the main course of water and its total length L in kilometres (Rodrigues et al., 2011).

$$D_m = \frac{\Delta Z}{1000 \times L}$$

Equivalent Slope of a water course

This value shows the slope of a line that when overlapped in a longitudinal profile gives equal areas above and below (Oliveira R. P., 2009).

$$i_q = \frac{Z_{eq} - Z_{min}}{L}$$

where:

$$Z_{eq} = \frac{1}{L}\sum_{i=0}^{n-1}(Z_i + Z_{i+1}) \cdot (X_{i+1} - X_i) - Z_{min}$$

Slope10-85 of a course of water

This parameter eliminates the parts of minor and major slope- A part located at finals 15% and initial 10% of the total length of a course of course (Oliveira R. P., 2009).

$$i_{10-85} = \frac{Z_{85} - Z_{10}}{0{,}75 \times L}$$

Relief index

Tis index is expressed through a ratio between the difference of altitudes in meters ΔZ (Between the highest and the lowest point of a basin) and its length L_b in kilometres (Castro & Carvalho, 2009).

$$i_{relevo} = \frac{\Delta z}{1000 \times L_b}$$

As higher is the value of a relief index, faster water will reach the mouth or final section, because this "index of Inclination" influence significantly the flows speed.

Slope index of a basin

The determination of average slope of a basin is nowadays facilitated by resources as GIS and digital cartography. GIS tools allows to obtain a digital model of the terrain (DMT) from topographic elements and the hydrographic web, to determinate a value of slope associated to each web forming DMT. Then, an average slope of a basin can be determined statistically from those values (Rodrigues et al., 2011).

In absent of resources for a such procedure, bibliography provides some slope index for a basin since n equivalent rectangle is previously determined (Rectangle of area and perimeter coincide with area and perimeter of the basin) (Rodrigues et al., 2011). The length L_e and width l_e of the equivalent rectangle of a basin of area A and perimeter P , is obtained from the following equation system:

$$\begin{cases} L_e \times l_e = A \\ 2 \times (L_e + l_e) = P \end{cases}$$

The resolution of such equation system in functions of L_e and l_e allows to obtain:

$$\begin{cases} L_e = \dfrac{P + \sqrt{P^2 - 16 \times A}}{4} \\ l_e = \dfrac{P - \sqrt{P^2 - 16 \times A}}{4} \end{cases}$$

Figure 23 shows an equivalent rectangle of the hydrographic basin of Machico.

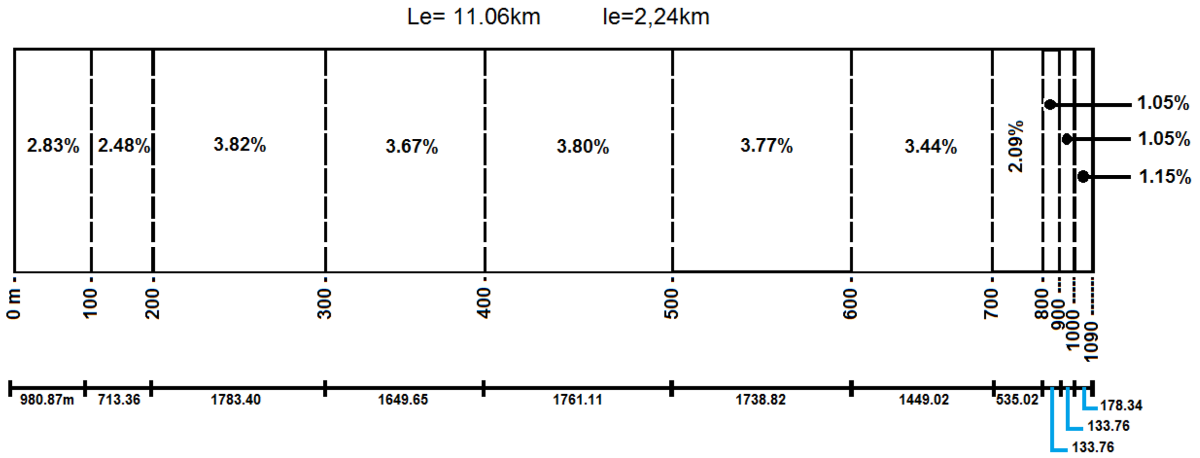

Figure 23 – Equivalent rectangle of the hydrographic basin of Machico (source: author).

Parallel to shortest edge of the rectangle can be found traced contour lines. The distance x_i separates two consecutives contour lines and it is directly proportional to the area a_i between them, then:

$$x_i = \frac{a_i}{l_e}$$

Known as equivalent rectangle of a basin, the slope index I_d can be obtained by:

$$I_d = \sqrt{\frac{\sum \Delta Z_i \times \frac{A_i}{A}}{1000 \times L_e}}$$

where ΔZ_i and A_i represent the difference od altitude and area between two consecutives contour lines.

The value ΔZ_i is constant and equal to the equidistance. As an exception, the first value will correspond to a difference between the lowest height and the first contour line, and the last value that represents a difference between a point of maximal height and the last contour line of the basin (Rodrigues et al., 2011).

Drainage pattern

Drainage patterns explains the arrangement of course of water, which are influenced by nature and rock layers configuration, geo-morphology of the region and slope differences (Figure 24). Such pattern is:

a) **Angular drainage** - Similar to rings of similar form which emerge from a body section of a tree;
b) **Dendritic drainage** - Designated with such name, due to its similarity to a tree (of Greek, dendros - tree). It is developed in rock of uniform resistance;
c) **Parallel drainage** - the courses of water flows almost parallel. It is also denominated equine or ponytails and it is in areas with precipitous hillsides or places with structural forms that creates irregular spaces;

d) **Radial Drainage** - water courses organized as a bike wheel connecting to a central point (final point). It is typical in old volcanoes;
e) **Rectangular Drainage** - it is a modification of the last type, because of failure systems or joints systems influence;
f) **Trellis drainage** - characterized by principal rivers which flows parallel to each other's, and by secondary rivers (also parallel between each other) draining perpendicularly to the main ones. It is typical in structures with failures.

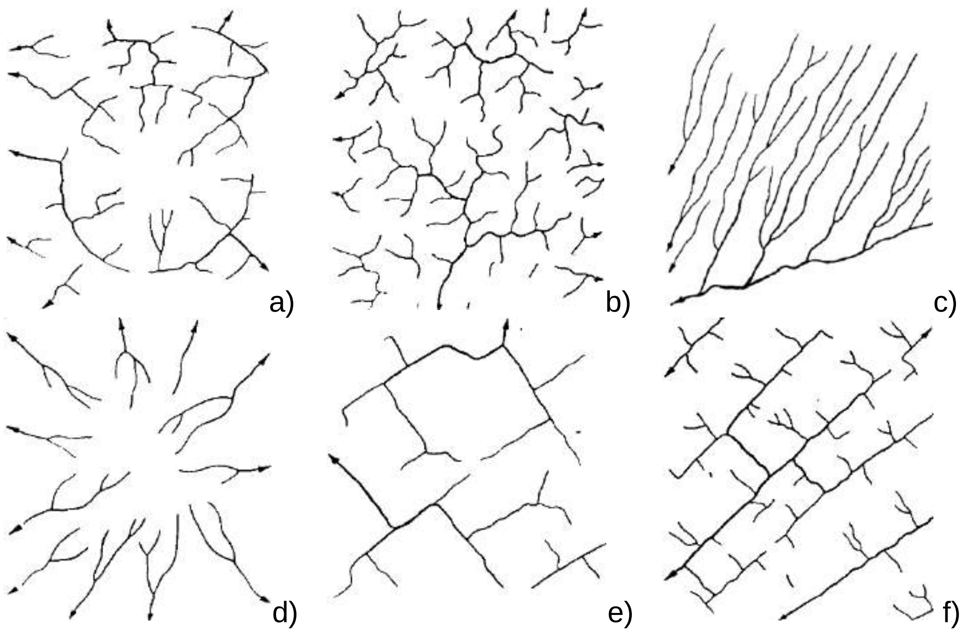

Figure 24 – Drainage patterns: a) Ring, b) dendritic, c) parallel, d) radial, e) rectangular, f) trellis (adapted from Catique, 2010).

3.3.4 Geology, soils and topsoil

The geologic characteristics of a basin have a strong influence in the drainage system, type of soil and in the distribution of water movement in it (Rodrigues et al., 2011).

The flows regime of a basin tent to be stable or constant as higher is the permeability of the soil and geological formations (because aquifers reservoirs are favoured). In the other hand, it will be more irregular when permeability is lower, in which hygrographs are characterized by accented peaks as an answer to precipitation (Rodrigues et al., 2011).

The most influents soils characteristics for water movement in a basin are infiltration capacity (generally higher with granulometry) and retention capacity (generally higher as granulometry decrease) (Rodrigues et al., 2011).

As an example, the permeability of rocks and absorption capacity of soils have a mayor or miner influence in infiltration of water and increase of groundwater reservoirs, reducing the available volume of superficial flows. The waterproofing of great areas by urbanization (buildings, highways, etc) leads to a reduction of superficial retention and infiltration, which increase floods possibility (Rodrigues et al., 2011).

Considering topsoil (vegetation layer) and soil uses of a basin is also important for a hydrologic behaviour analysis of hydrographic basins due to its great influence in flow and infiltration (Rodrigues et al., 2011).

The topsoil intercept part of the precipitated water, slow down superficial flows allowing extra-time for infiltration, protect soils from water impact and erosion, so contributing for (Rodrigues et al., 2011):

- Reduction of erosion of soil and stabilization in areas of abrupt slopes;
- Reduction of maximal flow rate of fullness or flood;
- Increase of hydric reserves;
- It also contributes for water caption from mist by precipitating it (a process called hide precipitation).

As an example, next figures show a soil graph (Figure 25) and a geologic graph (Figure 26) adapted from the hydrographic basin of Machico. The chart of soils uses of Madeira island is showed in Figure 27.

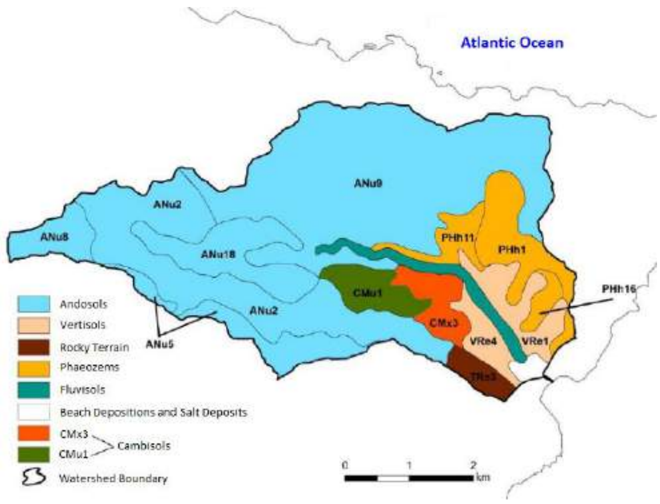

Figure 25 – Soil Charts (adapted from "Carta de solos da ilha da Madeira", 1992).

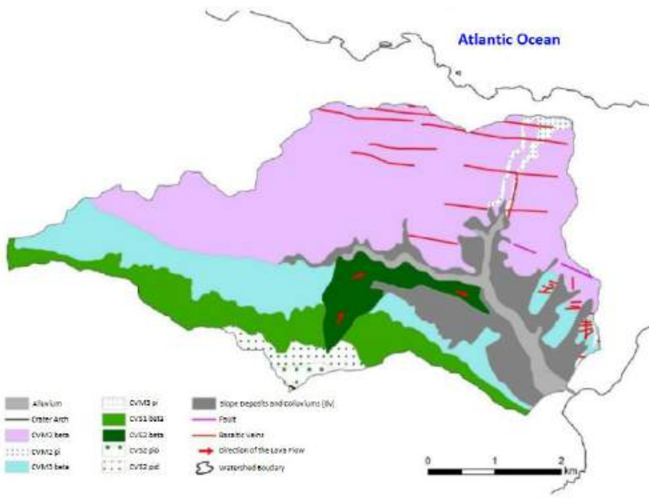

Figure 26 – Geologic Chart (Silveira, Madeira, Ramalho, Fonseca, & Prada, 2010b).

Figure 27 – Chart of soil uses of Madeira island (source: https://ifcn.madeira.gov.pt/).

CHAPTER 4 - HYDROLOGIC BALANCE OF A BASIN

4.1 General equation of hydrologic balance

The input flow rate (*in*) minus output flow rate (*out*) is equal to the variation of storage in time. This equation can be applied to system of all dimensions:

$$Q_i - Q_o = \frac{\Delta S}{\Delta t}$$

Hydrologic balance of a hydrographic basin in a certain interval of time, in which the calculation of "gains and losses" of water promoted in that interval by hydrologic processes and eventual human action, is fundamental to equate the binomial **"Needs/Availabilities"** (IST, 2018).

During a certain interval of time, the system **"hydrographic basin"** is feed by **precipitation** and eventually by water **dropped** by human action. Leading to a **flow** which allows **evapotranspiration** and eventually also allowing **extraction** of water by humans. Along the considered interval of time is possible to find alterations of the **amount of stored water** in the hydrographic web, because is retained at surface as soil moisture or in underground reserves (IST, 2018).

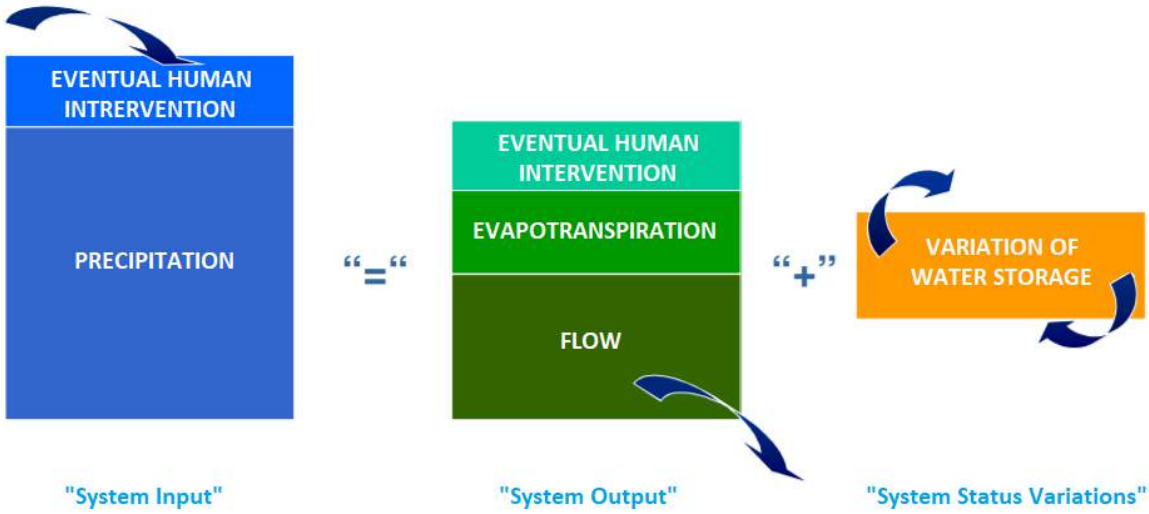

Figure 28 – Hydrologic Balance (IST, 2018).

Such system can be expressed as an equation:

$$P = H + E + \Delta S_P + \Delta S + \Delta S_U + E_X - R$$

where:
P - Precipitation in the basin;
H - Flow at downstream section of a basin;
E - Evapotranspiration of the basin;
ΔS_P - Variation of intercepted and stored amount of water at rivers bed;
ΔS - Variation of the amount of soil moisture (water at not saturated zone);

ΔS_U - Variation of the amount of water at underground reserves;
E_X - Amount of extracted water from the basin by human action;
R - Amount of water dropped in the basin by human action.

If the interval of time evaluated for the hydrologic balance were big enough to **ignore variation of different types of storages in** contrast of other terms, then (Quintela A. C., 1996):

$$P = H + E + E_X - R$$

If at such circumstances **evaluated water quantities due to human action were zero**, then (Quintela A. C., 1996):

$$P = H + E$$

$\boldsymbol{P = H + E + \Delta SP + \Delta S + \Delta S_U + EX - R}$

- General Equation of Hydrologic Balance

$\boldsymbol{P = H + E + EX - R}$

- When the time frame is large enough for variations of the various types of storage to be approximately zero ↔ **one hydrological year** (in Portugal it starts on October 1st and ends on September 30th of the following year)

$\boldsymbol{P = H + E}$

- If the quantities of water introduced by human action are approximately zero

$\boldsymbol{D = P - H = E}$

- The difference between precipitation and flow, the flow deficit, is equal to the loss of water from the basin by evapotranspiration

Figure 29 – Synthesis of hydrologic balance.

4.2 Hydrologic Year

The equation of hydrologic balance in the form of:

$$P - H - E = 0$$

It can be applied at time intervals of one year (**hydrologic year**), provided that the previous hypothesis is fulfilled, that at the beginning of each of these intervals the water storage in the basin is practically constant (and there is no transfers between basins) (Quintela A. C., 1996).

A convention in Portugal was stablished its initial point at **October 1st** and ends at **September 30th.** Because at the end of summer, most of the water reserves are proximal to its minimum limits and remain similar from year to year. So, accomplishing a condition: $\Delta S_P \approx \Delta S \approx \Delta S_u \approx 0$ (Quintela A. C., 1996).

According to Quintela A. C. (1996), the adoption of a hydrologic year allows:

- **To model an equation of hydrologic balance in order to co-relate annual values of precipitation and flows** (Since it is possible to evaluate evapotranspiration values);
- **To obtain series of values of annual flows statistically independent** (considering the flow rate that flows in a river, between October 1st and September 30th of each year, depending almost exclusively from precipitation along such year and with no-relationship with the last year precipitation).

In South-African countries of Portuguese Language:

- Cape Verde - July 1st until June 30th;
- Guinea-Bissau - May 1st until April 30th;
- São Tomé and Príncipe - September 1st until August 30th;
- Angola, Mozambique - October 1st until September 30th.

CHAPTER 5 - PRECIPITATION

5.1 Introduction

By precipitation we average all meteoric water which, coming from the water vapour of the atmosphere, reaches the surface of the globe. Meteoric water is the constituent of rain, drizzle, downpour, snow and hail. Generally, only meteoric precipitation is considered under the previous forms, neglecting the deposition of water on the surface due to fog, dew or frost (occult precipitation). Due to its importance in generating runoff, rainfall is the most important type of precipitation in hydrology (Rodrigues et al., 2011).

The amount of precipitation in a region is critical for the determination, inter alia, of crop irrigation needs or domestic and industrial supply. The precipitation intensity is important for the determination of flood peaks and determinant in erosion studies (Rodrigues et al., 2011).

The main characteristics of precipitation are its total, duration and the way it is distributed in space and time. The amount of precipitation has averaging only when associated with a duration. For example, values of 100 mm may represent little for a month of the wet season, but it is enough if it occurs in a day and an exceptionality if verified in an hour (Rodrigues et al., 2011).

The occurrence of precipitation is a purely random phenomenon that does not allow predictions with great anticipation. For this reason, the treatment of precipitation data passes, in most cases, by the application of statistical inference techniques in order to estimate the magnitude of rainfall events as a function of a given probability of occurrence (Rodrigues et al., 2011).

For precipitation, a thermal imbalance at the level of clouds caused by condensation of the water vapour must occur whenever the temperature drops below the point of saturation of the mass of air. However, condensation alone does not lead to an increase in the water droplets to the point of its detachment and fall, by the action of gravity. It is necessary that simultaneously occur the successive fusion of the micro droplets, which are thus increasing in size - **direct coalescence process** (Rodrigues et al., 2011).

The horizontal convergence of the water vapour in the direction of the atmospheric layers under the clouds is fundamental for the duration of the rainfall to have a fixed duration. In this way, the liquid water is accumulated next to the cloud for later replacement of the losses, as it is precipitating. If the horizontal convergence mechanism decreases or eventually changes in direction (divergence), the precipitation decreases or ceases, and in case of divergence, there is a cloud dissipation phenomenon (Rodrigues et al., 2011). Figure 30 schematizes the precipitation mechanism described above

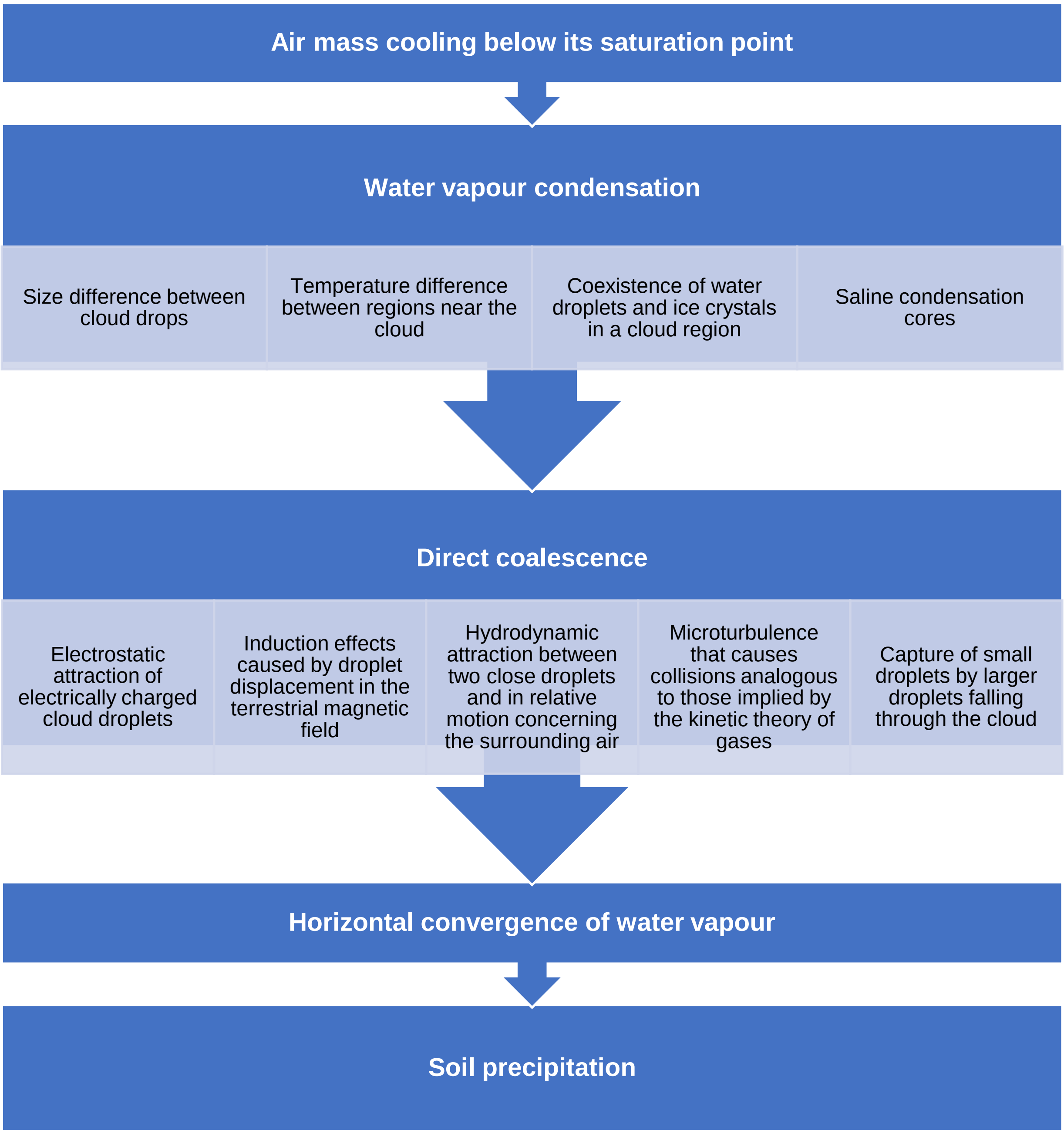

Figure 30 – Sequence of process for precipitation mechanism (adapted from Rodrigues, 1986).

5.2 Atmosphere

The atmosphere is the complex system of gases (nitrogen, oxygen, argon, carbon dioxide, ozone, water vapour, etc.) and solid and liquid particles of various types in suspension (aerosols), which surrounds the planet and is maintained over it by the action of gravity (Naghettini, 2012).

From the meteorological point of view, the most important phenomena are those that occur in the lower atmosphere, about 15 km thick. Due to the reduced thickness, the terrain relief greatly influences the distribution of precipitations, temperatures etc. (Naghettini, 2012).

The atmosphere is a reservoir with a modest volume of water when compared to the remaining reservoirs: only 25 mm on average. The water vapour therein constitutes the humidity of the air and, although it only corresponds to 2% of the air volume, plays a very important role in the atmosphere because it is the source of all hydrometeors; 90% of the atmospheric water vapour is in the first 5 km of altitude, the limit in which man inhabits (Naghettini, 2012).

As regards condensation of water vapour, it occurs only by lowering the temperature (by rising in the atmosphere) or by increasing the water vapour content (by increasing the amount of water in the gaseous form); however, the presence of "condensation cores" (dust, smoke, sea salt, ions, etc.) is necessary (Naghettini, 2012).

5.2.1 Classification of precipitations

Depending on the mechanism that conditions the elevation of humid air to colder layers of the atmosphere, precipitations are classified as **orographic, convective, and frontal or cyclonic** (Rodrigues et al., 2011).

1. Orographic

As its own name induces, orography has a preponderant action in its genesis. They occur when, impelled by the wind, a mass of air encounters a mountainous chain that forces it to ascend by sliding on the slopes until it cools below the saturation point forming the clouds and later, giving rise to precipitation (Figure 31) (Rodrigues et al., 2011).

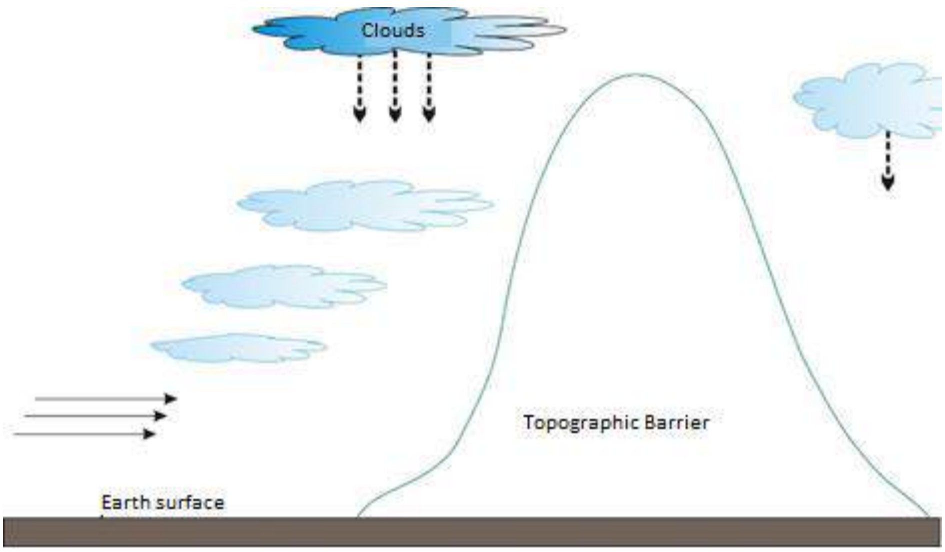

Figure 31 – Orographic process of precipitation (Estúdio Conejo, 2014).

Precipitations of orographic origin translate into rain of low intensity although they can prevail for long periods of time (Rodrigues et al., 2011)

The windward-facing slopes tend to register high precipitation values when compared to the leeward slopes because most of the moisture is discharged during ascending.

This fact leads to the formation of semi-arid zones in the leeward area (the so-called pluviometric shade), because when they reach these areas air masses are already depleted of moisture (Rodrigues et al., 2011).

2. Convective

Convective precipitations are those that are caused by the direct heating of a mass of air on the earth's surface (Figure 32). A sudden increase of less dense air that will reach its condensation temperature with the consequent cloud formation and, often, precipitation is recorded (Rodrigues et al., 2011).

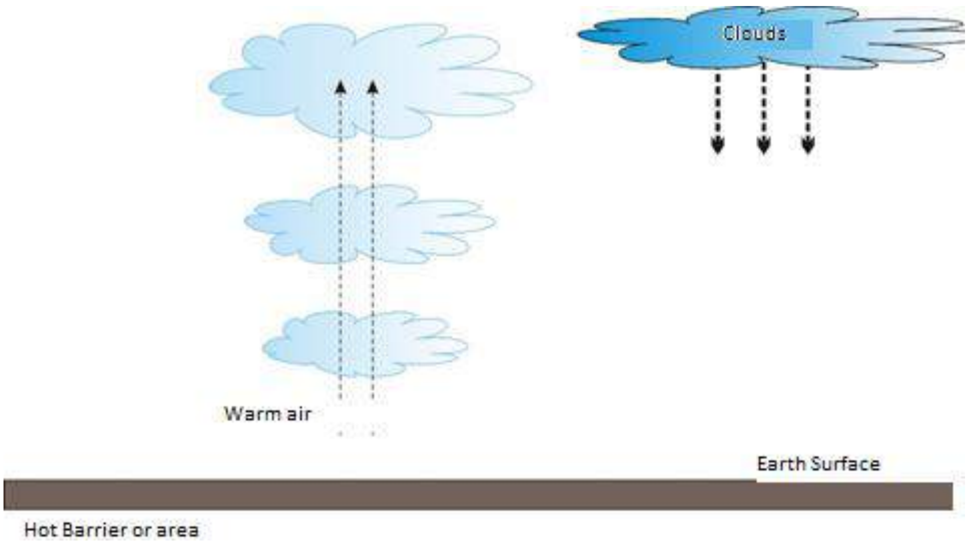

Figure 32 – Convective process of precipitation (Estúdio Conejo, 2014).

Convective showers are characteristic of the tropical regions, also occurring in our conditions during the summer. They are usually heavy rains of reduced duration, very localized and usually accompanied by thunderstorms. Its occurrence leads to flooding in small river basins (Rodrigues et al., 2011).

3. Frontal or Cyclonic

The rains of cyclonic or frontal origin are of great duration, with average intensities, but affecting large areas. They are sometimes accompanied by strong cyclonic winds. Its long duration eventually leads to flood formation in large basins (Rodrigues et al., 2011).

In the Portuguese territory cyclonic precipitation is conditioned by the winter depression that tends to form in the Azores, as opposed to the high pressure center - the Azores anticyclone - characteristic of the summer period (Rodrigues et al., 2011).

These precipitations are associated with the passage of cyclonic disturbances, and the rise of air may be caused by a barometric depression or by contact between two air masses, one hot and one cold (Figure 33) (Rodrigues et al., 2011).

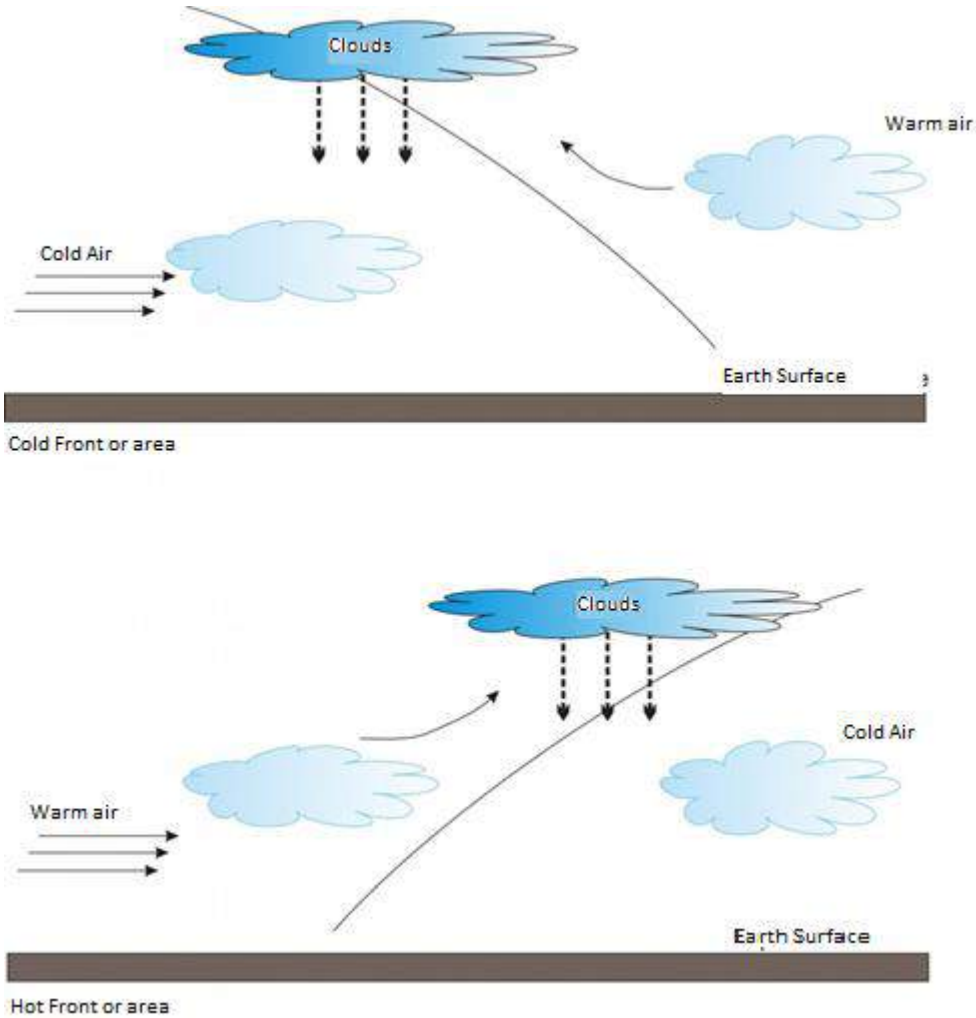

Figure 33 – Frontal or Cyclonic process of precipitation (Estúdio Conejo, 2014).

5.3 Precipitation Measurement

The amount of precipitation is the height of water accumulated on a horizontal surface relative to a certain time interval. It is expressed in millimetres or meters (Rodrigues et al., 2011).

The intensity of precipitation is the amount of precipitation referred to the unit of time. It is expressed in mm/h or l/s/ha. (1 mm/h = 2.78 l/s/ha) (Rodrigues et al., 2011).

5.3.1 Quantification of Hidden Precipitation

Despite the apparent importance of occult precipitation in the island's hydrological processes, its quantification becomes difficult not only because of the lack of standardized automatic instrumentation, but also because of the lack of knowledge about the various mechanisms and conditions of fog interception in natural environments (Gonzalez, 2000). Thus**, the amount of occult precipitation** is given by comparing precipitation values measured under vegetation and in the "open" area.

During episodes of rainfall, occult precipitation or simultaneous occurrence of both, water does not reach the soil of a forest homogeneously as in an uncovered area (Bruijnzeel, 2000). According to Crockford and Richardson (2000), water is distributed heterogeneously by three fractions (Figure 34):

- Vegetative interception (I), water that is retained by vegetation and is evaporated during or after the occurrence of precipitation;
- Draining of Trunks (SF), water that is sent to the ground through the trunks or branches in contact with the ground;
- Through fall (TF), water that may or may not have been in contact with the vegetation and which falls to the ground through its various components (branches, leaves, etc.).

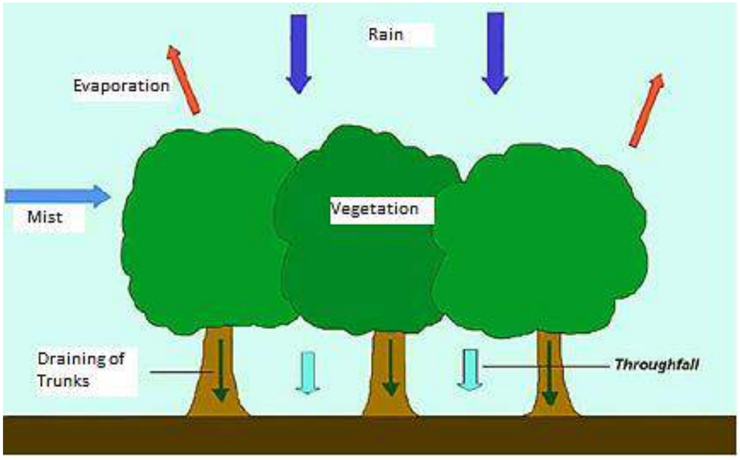

Figure 34 – Branching of precipitation (Figueira C. et al., 2006).

Such Branching is generally expressed by:

$$I = P_{gross} - TF - SF$$

where:
I - Vegetative interception (mm);
P_{gross} - Gross precipitation measured in a “Open sky” (mm);
TF - Throughfall (mm);
SF - Drainage of Trunks (mm).

The summation of TF and SF leads to “Liquid precipitation” (P_{liq}), logo:

$$I = P_{gross} - P_{liq}$$

where:
P_{liq} - Liquid precipitation (mm);

Under the canopy of a forest, there is always interception that is influenced by the type of vegetation cover (such as the storage capacity of the crown and its variation with the seasons and species, leaf surface index, leaf angle and cover, storage capacity in a Epiphytic and shrubby level of different aerial parts of plants) and climatic factors such as the amount, intensity and duration of rainfall, wind speed and direction during the rain episode, and air temperature and humidity (Crockford and

Richardson, 2000). Because of this, an open space station usually receives a greater amount of precipitation (gross precipitation) than a station under vegetation (net precipitation), with plant intercept having a positive value. However, when the net precipitation value is higher than the gross precipitation value (negative vegetation interception), the additional water is considered to be from the fog intercepted by the canopy (Holder, 2003).

However, occult precipitation is not exactly the same as the difference between net and gross precipitation when the first exceeds the second one mentioned, since evaporation and storage by the forest cover during the interception process is not considered in the equation, due to the great difficulty in its quantification. Thus, the value of occult precipitation is underestimated (Holder, 2003), because it is considered that only the water of the fog was contributed in the days in which the values of net precipitation surpass those of the gross precipitation. In this way, the volume of occult precipitation that may be present on days when the vegetation intercept positive value is ignored, as well as the volume that compensated for the rainfall interception value on the days when the vegetation interception was negative. In this way, what Bruinjzeel (2000) calls liquid occult precipitation (here called occult precipitation) is quantified.

Vegetal interception is by nature positive since the amount of rainwater measured in the "open" is always greater than the amount under the canopy ($I = P_{gross} - P_{liquid}$)

↓

When the volume of water collected under the vegetation is higher than that recorded in the open area, there must be another source of water: occult precipitation

↓

Thus, the value of the daily average input of the occult precipitation is given by the formula:

$$P_{occult} = |\ I \text{ on days when it is negative}|$$

Figure 35 – Synthesis about occult precipitation.

5.3.2 Measuring devices and associated errors

Measurement of precipitation is done using devices called udometers (or pluviometer) and udographers (or pluviographers), located at different points located on the ground (udometrics's stations) (ISEL, 2015).

Udometers measures the amount of precipitation that has occurred at a certain time (for example 1 day), expressed in height, while udographers measure and record on a graph or udogram (Figure 36) the precipitation in a continuous way, allowing to know the intensity of precipitation at any time (ISEL, 2015).

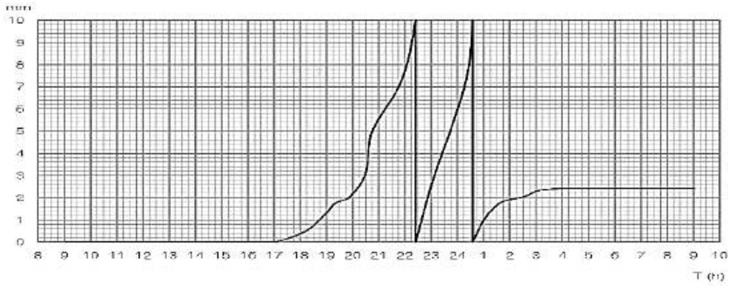

Figure 36 – Example of an udogram (Source: https://fenix.isa.ulisboa.pt/qubEdu/conteudos-publicos/ficheiros?oid=3972844804408).

In an udographer of floatation, the collected rain is conducted into a reservoir containing a light, hollow float. As the water level rises, the vertical movement of the float is transmitted (by appropriate mechanisms) to a pen (scroll) that moves over a graph. The scale value is obtained by conveniently matching the dimensions of the receiving funnel, float and reservoir mouth or input. In this type of instrument, we can also adapt a mechanism that automatically executes fast emptying of the reservoir when it becomes full, i.e., a siphon system is used. After the deposit is empty, the pen (trim) returns to the zero of the graph. In winter, when there is a possibility of frost, a heating device is installed inside the udograph (Agostinho, 2009).

Figure 37 – Udograph (source: SNIRH).

Despite the standard installation rules, the precipitation measured in an apparatus may differ from the precipitation that reaches the ground by the following circumstances:

- Device defects;
- Evaporation;
- Effect of wind on rainfall trajectories.

5.4 Hydrological Series. Homogeneity and consistency

Hydrological Series are the data resulting from the observation of **hydrological magnitudes** (as for example: precipitation, flow, etc.). The same magnitude may correspond to different hydrological series depending on the time interval or other characteristic that defines them (IST, 2018).

For example, for the records of an udometric station, it is possible to have series of values of the maximum daily precipitation, or of the annual average precipitation. For flow in a given section of a river, it is possible to have series of values of the average daily flow, the annual maximum instantaneous flow rate, etc. In order to be used in hydrological studies, the hydrological series must have **homogeneity** and **consistency** (IST, 2018).

A hydrological series is said to be **homogeneous** when, during a period of observation, there are **no changes in the factors** that condition the phenomenon expressed by this magnitude (IST, 2018).

The deforestation of a river basin or the creation of a reservoir can cause a loss of homogeneity of a series of values of the liquid flow and of the solid flow in its watercourse, because they constitute a **change in the factors** that the phenomenon depends on (IST, 2018).

A hydrological series is said to be **consistent**, if during its observation period there is no change in the **systematic error** of measurement of the magnitude (IST, 2018).

The change in the location of an installation of a udometer or the change in the flow section where the flow is measured in a water course can cause a lack of consistency in the data obtained by altering the systematic error of the reading (IST, 2018). The verification of the data quality of a certain series can be carried out following the steps of the following scheme:

1º Closing data gaps (if any)

↓

2º Test sample **randomness**

↓

3º Verification of **consistency** and **homogeneity** of data series:

- **Accidental Errors:**
  - Measuring/Reading error
  - Recorder stop
- **Systematic Error:**
  - Change of station's location
  - Poor beaker graduation regarding the water input section of the pluviometer
  - Vegetation growth around the station
  - Climate changes

Figure 38 – Verification of the data quality.

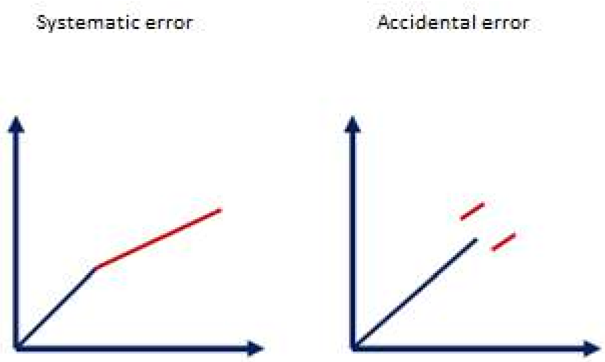

Figure 39 – Types of Errors (IST, 2018)

5.4.1 Test of double accumulated values

The breach of homogeneity or consistency of annual series can be detected in many cases by a **test of double accumulated values** (ISEL, 2015).

In order to verify the consistency of a series of annual precipitation at a certain udometer station, it is marked in a Cartesians system axis, the cumulative annual values of precipitation at such station VS. the summation of the cumulative (or arithmetic average) of annual precipitation of a group of neighbouring stations (ISEL, 2015)**.**

If the series of values of the annual precipitation at the station concerned is consistent, points will be sensitively aligned along a line (ISEL, 2015)**.**

If the series at the post under study is inconsistent, it is generally obtained two straight segments, with a breach at the point corresponding to the year in which a significant change was found in the operating conditions of the post (ISEL, 2015).

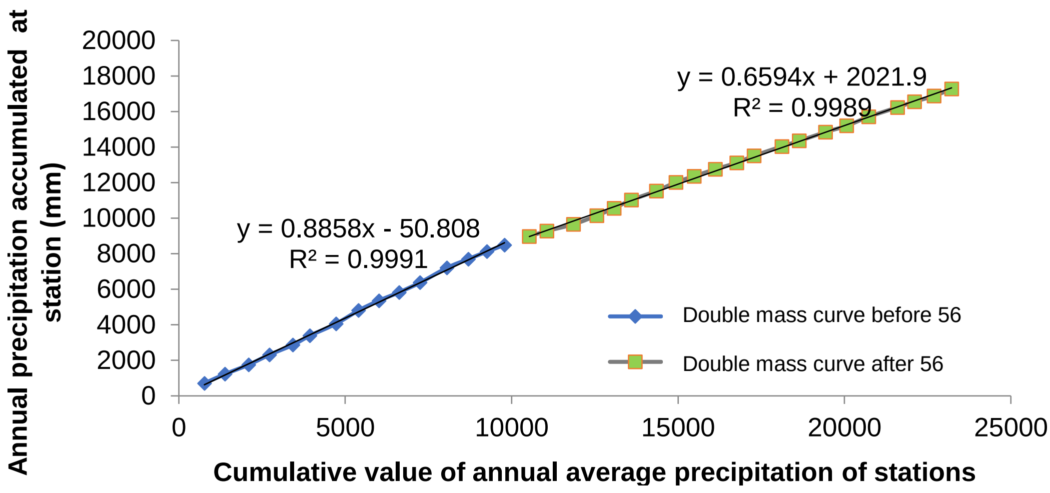

Figure 40 – Test of double accumulated values (source: author).

If a test reveals inconsistency of the data and if its explanation is found, the precipitation values corresponding to the time interval in which the deviation is verified can be adjusted in relation to the intervals considered correct (ISEL, 2015).

The adjustment is made from the proportionality of the angular coefficients of the straight-line segments of the double-accumulated values:

$$P = \frac{b_0}{b} \times P_0$$

where:
P - Adjusted precipitation;
P_0 - Measured precipitation;
b_0 - angular coefficient in an interval of time taken as reference for the adjustment;
b - angular coefficient corresponding to the observations to adjust.

The angular coefficients of the line segments can be determined based in the plotted graph or by the adjustment of least squares lines to the pairs of values (ISEL, 2015).

$$y = ax + b, \qquad with \qquad a = \frac{y_2 - y_1}{x_2 - x_1}$$

where:
a - line's slope;
b - ordered in origin.

The records of the group of stations used to test the double accumulated values should be individually analysed, eliminating from the group the stations with inconsistent records (ISEL, 2015).

When testing for doubly accumulated values, changes in the alignment of the points can occur only due to the very randomness of hydrological phenomena. Some authors ignore changes that do not persist for more than five years, as they may be due to fortuity (ISEL, 2015).

5.4.2 Bug Filling / Regional Weighting Method

Often precipitation records in a udometric station have faults during one or more days, and it is of interest to obtain the corresponding estimates to make it possible to calculate the monthly and annual totals (ISEL, 2015).

Most common faults or failures in observations are:
- Wrong completion of observed data;
- Value estimated by the observer because he is not at the sampling site;
- Machine failure or damage, mechanical problems in the graphic recorder.

The regional weighting method is a simplified method for filling a station's faults by weighting data from that station with data from at least three neighbouring stations that do not show fault at the same time. The stations under analysis must belong to similar climatological regions and must have a minimum of 10 years of data records (ISEL, 2015).

Let X be the station with faults and A, B and C the neighbouring stations, used for the weighting (ISEL, 2015). The missing precipitation P_X can be determined at station X by the following equation:

$$P_X = \frac{1}{3}\left(\frac{\overline{P_X}}{\overline{P_A}} \times P_A + \frac{\overline{P_X}}{\overline{P_B}} \times P_B + \frac{\overline{P_X}}{\overline{P_C}} \times P_C\right)$$

where:
$\overline{P_X}, \overline{P_A}, \overline{P_B}\ and\ \overline{P_C}$ - annual average precipitation in stations X, A, B and C;
$P_A, P_B\ and\ P_C$ - verified precipitations in stations A, B and C, at the moment of fault P_X verified in station X.

The presentation of monthly or annual values of the precipitation that required the completion of failures should always be accompanied by the indication of the existence of this filling or correction (ISEL, 2015).

5.5 Spatial distribution of precipitation/precipitation over an area

Knowledge of occasional rainfall alone has little interest. It is usually interesting to know the weighted precipitation over a given area (erg., a river basin or irrigation perimeter) (ISEL, 2015).

There are three main methods to obtain the weighted precipitation in a zone, from precipitation records in hill or udometers stations: The **Arithmetic Average** method, the **Thiessen** method and the **Isolines** method (Rodrigues et al., 2011).

5.5.1 Arithmetic Average Method

It is the simplest method to obtain the average precipitation value. It makes use of precipitation heights recorded in several rain gauges or udometers. This method produces good results if the stations / stations are uniformly distributed over the basin and the height measured in the different stations does not vary much from the average (Figure 41) (Rodrigues et al., 2011).

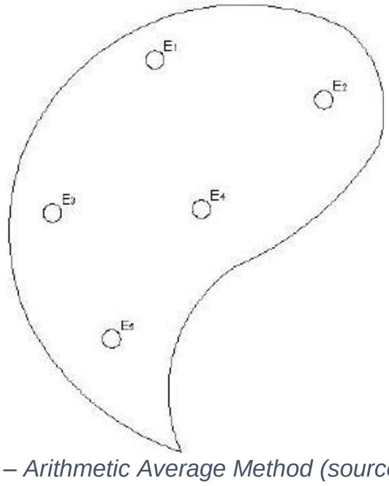

Figure 41 – Arithmetic Average Method (source: author).

$$\bar{P} = \frac{\sum_{i=1}^{n} P_i}{n}$$

where:
P_i - Precipitation at station i;
n - number of stations.

5.5.2 Thiessen Method

This method assumes that at any point in the basin precipitation is equal to the measurement at the nearest station. In this way, the height register in a certain station is applied in other points, if they are up to the middle distance of the other station (in any direction). The relative weights for each station are determined by the respective areas, calculated by the application of the Thiessen polygon method, where the boundaries of the polygons are formed by the mediatrix of the lines joining two adjacent stations (Figure 42) (Rodrigues et al., 2011)). The average or average rainfall for a basin, $\overline{\mathrm{P}}$, is calculated by the following expression:

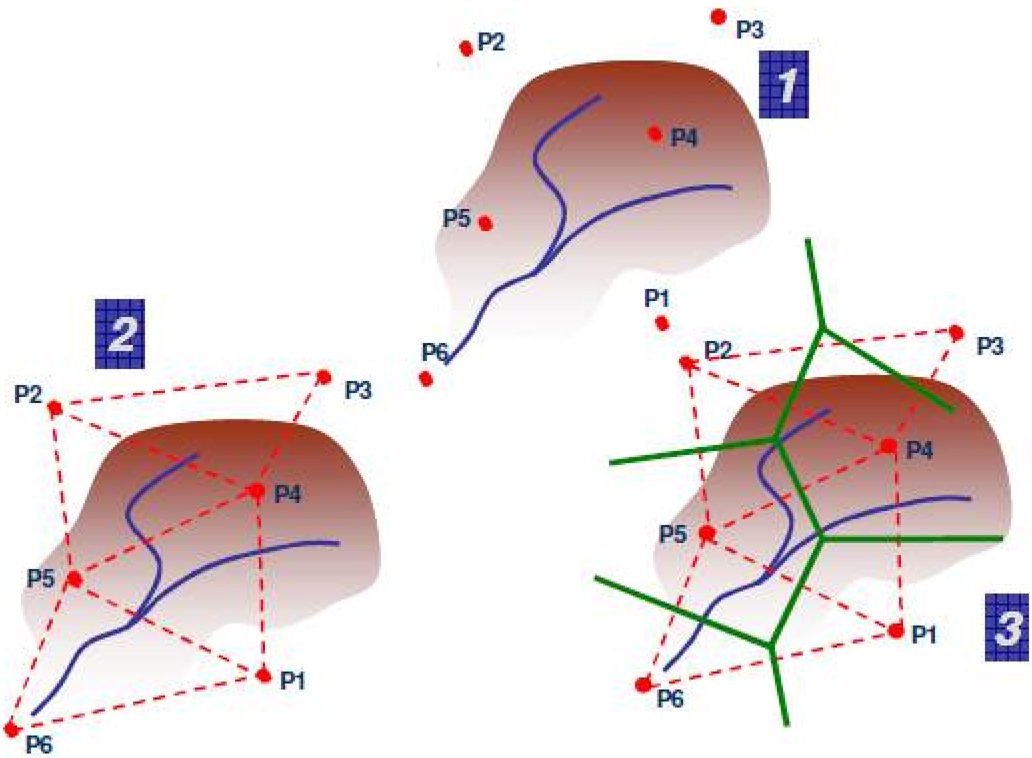

Figure 42 – Thiessen method (IST, 2018).

$$\bar{P} = \frac{\sum_{i=1}^{n} P_i \cdot A_i}{A_b}$$

where:
P_i - Precipitation at station i;
A_i - Polygons area associated to station i;
A_b - Basin area.

The Thiessen method is usually more accurate than the arithmetic average method and can be used with stations outside the basin area under study. However, it is not flexible either to data loss at a given point in a certain period or to each time a job change is made (which involves constructing a new polygon trace). Also, the method does not directly consider the orographic influences in the rains or variations of spatial distributions of intensity of a rainfall (Rodrigues et al., 2011).

5.5.3 Isolines Method

Some of the difficulties presented in the previous section can be overcome by the construction of **isolines** - precipitation isolines, which represents the geometric places of the points of equal precipitation on the ground for a certain period of time (minutes, hours, days, months or years) - using the observed rain heights at stations and values interpolated between adjacent stations. A graph of isolines provides a clear and synthetic view of the **spatial distribution of precipitation** (Rodrigues et al., 2011).

According to ISEL (2015), to trace isolines, it is necessary to follow next procedure:

- the zone stations under study are marked on a plant of the zone under study, and the precipitation verified in each one of them;
- by interpolation between the points of known precipitation, determine the points of equal precipitation, corresponding to a given isoline:
    - consider in (Figure 43) two points (A & B), distance L, which pluviometric heights are respectively, h_A and h_B. The position at point C, located in segment AC with a distance x from point A and where pluviometric height is h_C, is given by:

$$\frac{x}{h_A - h_C} = \frac{L}{h_A - h_B} \rightarrow x = L \times \frac{h_A - h_C}{h_A - h_B}$$

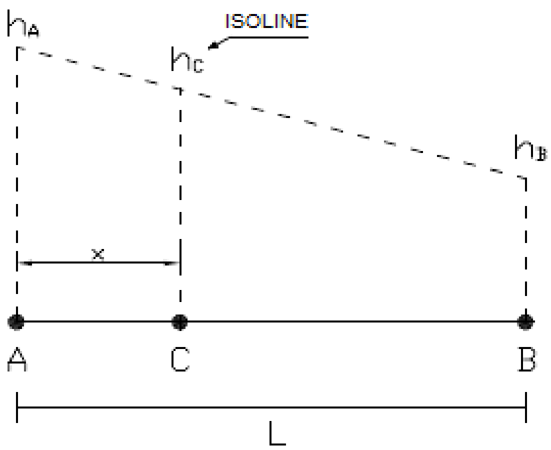

Figure 43 – Interpolation of precipitation values (ISEL, 2015).

- Point of equal precipitation are joined (Figure 44);
- Bisectrix of the angle formed by such lines are taken;
- In considered points, perpendicular lines are traced;

- Finally, the isolines are drawn, tangent the perpendicular lines at the referred points.

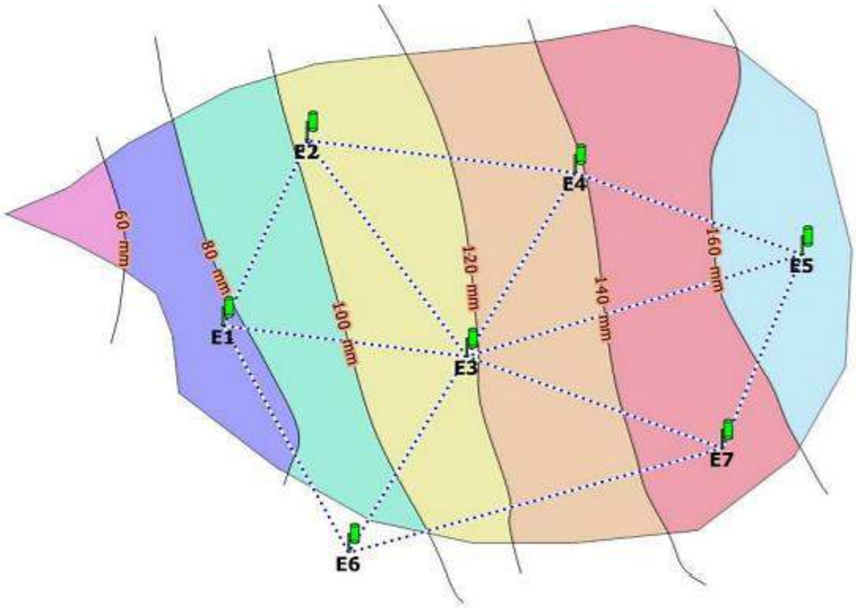

Figure 44 – Isolines Tracing (source: http://ing.unne.edu.ar/pub/hidrologia/hidro-tp2.pdf).

When there is a dense network of measurement stations, the iso-map can be constructed using computer programs to automate control. Once the isotope map is completed, the area, A_i ,between each pair of isolines within the basin is measured and multiplied by the average, P_i, of the precipitation heights represented by the isolines bordering such area. Thus, the average precipitation can be calculated by the previous formula (Rodrigues et al., 2011).

The isolines method is very flexible and the knowledge of the storm model can influence the path of the isolines, but for a relatively high density of stations is necessary for the correct construction of the maps for a complex storm (Rodrigues et al., 2011). The isolines method provides more correct results than the Thiessen method, but is much harder, since it requires tracing isolines for each analysed case, unlike the method of Thiessen, where the areas of influence are always the same (ISEL, 2015).

5.6 Temporal distribution of precipitation

5.6.1 Annual precipitation

The values of hourly, daily, monthly or annual precipitation (**series of rainfall values**) observed in a hill or udometer station or calculated over a zone over several years, form a random set of useless information, if it is not properly analysed with statistical treatment (ISEL, 2015).

The annual precipitation measured at a post or station (or calculated on a given area) is a random variable whose statistical distribution is approximately symmetric and can be translated by normal law, being completely characterized by the estimate of the average and the estimate of the standard deviation (ISEL, 2015).

In the analysis of the annual rainfall series, it will be necessary to start by testing its quality including the reconstitution of the data of the series and the verification of its consistency. Ideally, each station should be contrasted with neighbouring stations held stable in terms of average (Rodrigues et al., 2011).

For a quick identification of the stations with a stable average, it is possible to resort the graphical representation of the accumulated annual averages (Figure 45). The analysis of the graph obtained gives indication for the minimum number of years necessary for the characterization study (about 15 years, in this case) (Rodrigues et al., 2011).

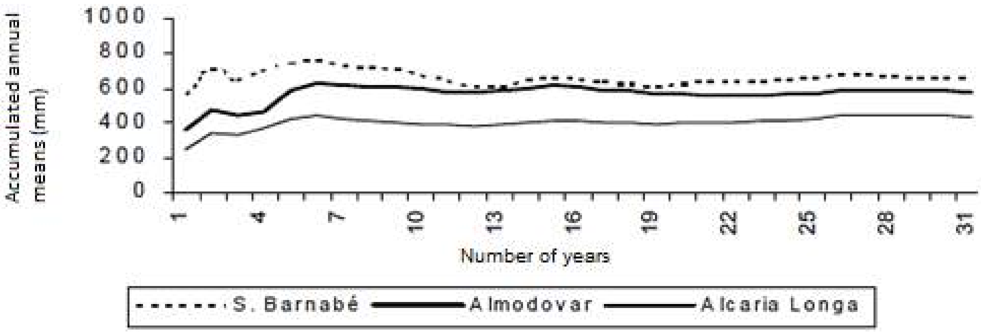

Figure 45 – Accumulated annual averages (Rodrigues et al., 2011).

The actual characterization of the annual series is summarized in the determination of the first four statistical moments (average, standard deviation, asymmetry coefficient and flatness coefficient or kurtosis) and in the identification of the probability density function that best adjusts to the values observed in each station as well as the coefficient of variation of the sample. The selection of the theoretical function can, in the first analysis, be obtained through the histogram determination and the evaluation of the adjustment to the theoretical function, and it can be determined through statistical tests where the chi-square is more powerful (Rodrigues et al., 2011).

In this phase of the characterization it is possible to determine the precipitation associated to a given return period, T, provided that the value corresponding to the probability of 1 / T is determined in the adjusted statistical function (Rodrigues et al., 2011). Under these conditions, the value of T will be:

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

where $F(X)$ shows the probability of non-exceedance, which averages correspond to the probability of a certain value of precipitation not to be exceeded $F(X) = P(X \leq x)$, and $G(X)$ is the probability of exceedance, such that: $G(X) = 1 - F(X)$.

Even before the adjustment of the series data to a theoretical probability distribution (normal law in the annual series), the value of $F(X)$ can be obtained empirically, as a positional probability, by the application of the Weibull expression (Rodrigues et al., 2011):

$$F(X) = \frac{m}{N+1}$$

where m correspond to the position of each value of the series classified or grouped in an ascendant order and N is the total number of elements of such series.

5.6.2 Monthly Precipitation

According to Rodrigues et al. (2011), the summary characterization of monthly precipitation is made appealing to:

- **Chronological Diagrams of average values of each month** (Figure 46);

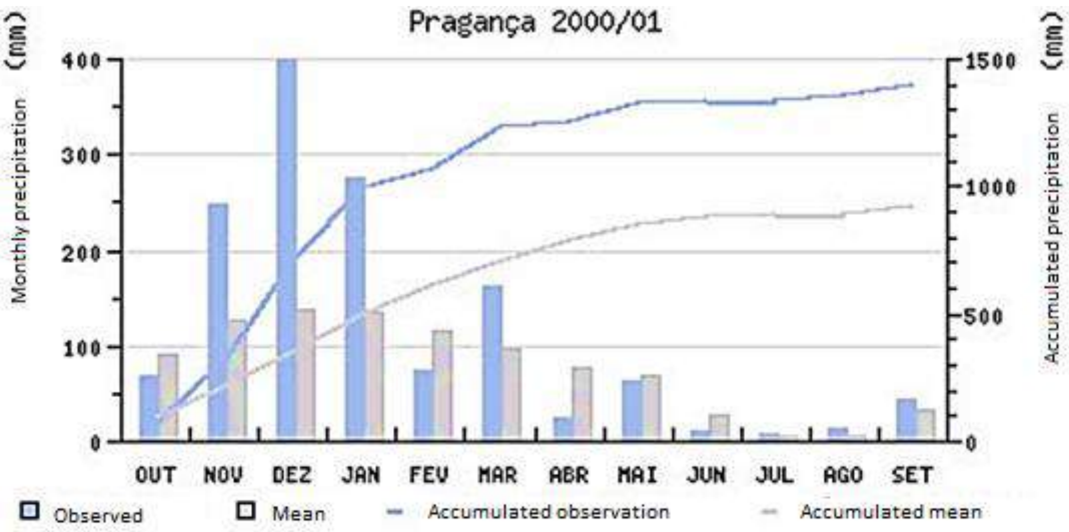

Figure 46 – Monthly and accumulated precipitation registered in a station of Bragança (Loures's Municipality) at the hydrologic year of 2000/2001 (Sources: SNIRH).

The graph (diagram) of the average precipitation values in each calendar month, over a period of study, is called as the monthly precipitation diagram in the average year. The diagram of values corresponding to the year of **average characteristics** is obtained as follows:

- A matrix is created with 12 rows and n columns, with rows corresponding to the 12 months of the year and columns at n years of the study period;
- In each n-column, the 12 values of monthly precipitation are ordered in descendent order, independently of it month of occurrence;
- The average values of each matrix line are calculated (according to the horizontal);
- Then, the average values calculated are attributed to the several months of the year by the same order of magnitude of the values of the average year, thus creating a fictitious year, which is called year of average characteristics.

The year of characteristic averages values is more representative than the average year due to the variability of monthly precipitation in a station or a region, because in the average year, the extreme values lose visibility with the average calculation:

- **Classified diagrams of relative frequencies (probabilities)** - It is the representation of the frequency in which the values are exceeded in each month of the sample;
- **Graphical Representation of maximal and minimal values** - Graphical representation of maximum and minimums registered in the period for each month of the series;
- **Determination of the coefficient of Monthly precipitation variation** - is a measurement of monthly variability of precipitation within the year and can be defined as:

$$Cv_{monthly} = \frac{\sqrt{\frac{\sum_{i=1}^{12}(P_i - \bar{P})^2}{12}}}{\bar{P}}$$

where P_i is the value of precipitation in each month and $\bar{P}$ is the average monthly precipitation of the analysed year. Such coefficient allows to notice that the regularity of the average year is higher than the regularity of the average of the years of the respective period.

5.7 Intense precipitation

5.7.1 Introduction

Intense precipitation must be understood as high intensity rainfall, lasting from days to few minutes (10 to 5 minutes), whose calculation is fundamental to the design of hydraulic works (drainage systems, flood protection dams, dam unloaders, etc.), such that directly condition the value of the maximum flow of a flood (peak flow rate or maximal flow(critical)). The study of intense precipitation is also fundamental for the analysis of soil susceptibility to erosion (Rodrigues et al., 2011).

The analysis of the maximum values of precipitation associated to a duration (considering the analysis of historical series as the maximum hourly precipitation considering a series of at least 15 years) evidences remarked differences in extreme values of precipitation (Rodrigues et al., 2011).

According to Rodrigues et al. (2011), Intense rainfalls are characterized by three parameters:

- **Duration** - The analysis of the precipitation according to its duration is fundamental for the design of the hydraulic works, where the determination of the flood flows is required. The period to consider can range from a few minutes (rainwater collectors) to a few hours (works in rivers with small hydrographic basins) or even a few days (river works with large hydrographic basins));

- **Intensity** - It was already referred that intensity is the quotient between height of a rainfall and the duration of such event;
- **Frequency** - Represents the probability of occurrence of a known rainfall duration and intensity, usually expressed in terms of return period, (T).

5.7.2 Depth-duration-frequency curves (DDF curves)

Introduction

In this subchapter we will use a concrete example to explain the process of creating a depth-duration-frequency curve based on the work of Camacho (2015).

The data refer to the daily precipitation (per season) for the municipality of Funchal, for a series of 30 years ending on 12/31/2014, provided by the Madeira delegation of the IPMA (Portuguese Institute for the Sea and the Atmosphere) and the same data, but for a series of 17 years ending on 12/31/2014, obtained through the SNIRH (National Water Resources Information System) website, such data were organized and treated, following the process described in the following paragraphs. As the most unfavourable case was obtained for the SNIRH data, all the data / results obtained are related to the same.

Initial Methodology

The first stage consists of identifying which stations can be used in each of the basins, their operating time and the coordinates of the stations to insert them into ArcGIS; (using AutoCAD) and apply the Thiessen method formula to obtain the average daily rainfall for each basin (in this case, the procedure described above was the most appropriate). At the end, the following table is constructed:

Table 7 – Daily and annual maximum precipitation.

Hydrologic Year	Maximum precipitation (mm)	
	Daily	Annual
1998	113.0	1190.1
1999	84.4	1172.9
2000	77.9	1150.3
2001	155.0	1628.9
2002	84.9	1398.8
2003	65.0	1195.2
2004	129.9	876.1
2005	92.6	1498.0
2006	96.8	1447.0
2007	60.5	795.5
2008	143.3	1099.8
2009	108.6	1554.7
2010	177.1	2491.7
2011	144.3	930.9
2012	187.3	1201.4
2013	60.6	864.5
2014	56.2	1027.9

The next step consists in obtain the maximum precipitation values at 1,2,3,4 and 5 days for each basin, in conformity with the next table:

Table 8 – Maximum precipitation at 1, 2, 3, 4 and 5 days.

Hydrologic year	Maximum precipitation (mm)				
	1 day	2 days	3 days	4 days	5 days
1998	113.0	167.7	209.8	220.7	256.9
1999	84.4	105.9	125.1	133.5	139.4
2000	77.9	107.5	132.5	160.5	187.9
2001	155.0	265.6	297.5	318.2	373.6
2002	84.9	101.8	131.1	138.4	143.9
2003	65.0	98.4	104.9	137.5	153.1
2004	129.9	142.6	166.3	166.4	166.5
2005	92.6	147.7	165.3	194.0	227.2
2006	96.8	169.4	208.2	216.7	221.3
2007	60.5	112.4	135.0	179.8	213.4
2008	143.3	193.7	225.0	243.1	252.9
2009	108.6	130.7	140.2	184.4	231.4
2010	177.1	229.1	261.8	322.3	337.1
2011	144.3	182.6	228.5	266.9	272.8
2012	187.3	210.7	263.8	290.3	291.5
2013	60.6	104.8	156.4	185.7	230.7
2014	56.2	102.5	111.6	112.8	121.9

Obtaining DDF

In this subchapter is exposed the process of adjustment of statistical laws to hydrological variable samples and estimation of the values of these variables as a function of probability of exceedance, which consists of the following steps:

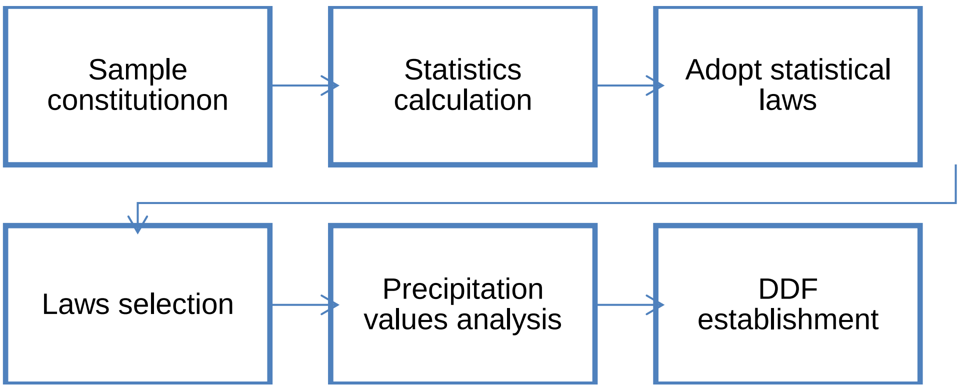

Figure 47 – Stages for a probabilistic analysis.

The first step is to adopt an adequate/reliable sampling technique, using the data provided by IPMA or SNIRH.

In the second phase, from the sample (maximum precipitation at 1, 2, 3, 4 and 5 days, along the hydrological years), certain statistical parameters such as:

- Average:

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

- Variance/Standard deviation:

$$s' = \frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}$$

- Coefficient of variation:

$$c_v = \frac{\sigma^2}{\bar{x}}$$

- Coefficient of asymmetry:

$$c_a = \frac{n \cdot \sum_{i=1}^{n}(x_i - \bar{x})^3}{(n-1)\cdot(n-2)\cdot s'^3}$$

In a third phase, laws are expected to be adequate to represent the distribution of sample values, in this case extreme laws such as: Log-normal or Galton's Law, Gumbel's, Pearson's III and Normal Law.

The fourth step will be to select the law with the best fit, either by visual adjustment or by applying other techniques, such as non-parametric tests. In this step we can individualize three tasks:

i. Graphical representation of the theoretical laws, by arbitrating successive probabilities of non-exceedance (F) and calculating the values of the random variable (precipitation) corresponding to these probabilities, according to the different postulated laws.

Table 9 – Statistical Laws.

Law	Formula	Statistical parameters from
Normal	$\hat{X} = \bar{X} + K \cdot s'$	Maximum annual precipitation
Galton	$\hat{X} = \bar{X} + K \cdot s'$	Base algorithm and (Maximum annual precipitation)
Gumbel	$\hat{X} = \bar{X} + K \cdot s'$	Maximum annual precipitation
Pearson III	$\hat{X} = \bar{X} + K \cdot s'$	Maximum annual precipitation
*K - factor of probability depends on the postulated law		

Table 10 – Factor of probability.

Law	K	Additional Parameters
Normal	$K_N = Z = w - \frac{2{,}515517 + 0.802853w + 0{,}010328w^2}{1 + 1{,}432788w + 0{,}189269w^2 + 0{,}001308w^3}$	$w = \sqrt{\ln(T^2)}$
Gumbel	$K_G = -\frac{\sqrt{6}}{\pi}\left\{0{,}577216 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\}$	-
Pearson III	$K_P = Z + (Z^2-1)k + \frac{1}{3}(Z^3 - 6Z)k^2 - (Z^2-1)k^3 + 4Zk^4 + \frac{1}{3}k^5$	$k = \frac{c_s}{6}$

ii. Representation of the points of the sample on a logarithmic scale, matching each point to its empirical probability.

$$F = i/(N+1)$$

where:
i - order/position of the sample;
N - total number of samples.

iii. Selection of the law leading to the best visual adjustment.

The penultimate step consists in estimate values of the hydrological variable for different durations (in this case, from 1 to 5 days) and for the pretended probabilities of non-exceedance, which averages, for the desired return periods.

Table 11 – Maximum precipitation for a certain duration, return period and probabilistic law.

Duration (hours)	Law of best adjustment	Annual maximum precipitation (mm)		
		10 Years	100 Years	1000 Years
24	Galton	165.72	248.60	334.42
48	Gumbel	218.06	311.71	403.67
72	Gumbel	257.46	365.98	472.53
96	Gumbel	288.68	407.32	523.81
120	Galton	321.92	447.94	570.31

The last step consists in define a relation for the considered return period, better known as Udometric possibility line (DDF), based on the pairs of values (duration, precipitation) and for the law with the best adjustment.

Table 12 – Precipitation in function of duration.

T = 100 years	
Duration of precipitation (hours)	Maximum precipitation (mm)
24	248.60
48	311.71
72	365.98
96	407.32
120	447.94

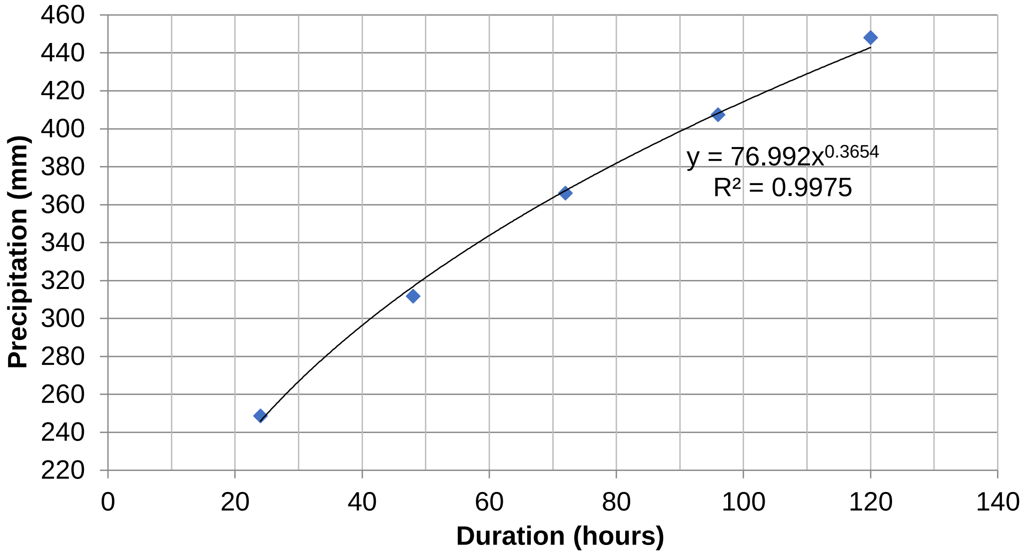

Figure 48 – DDF curve for T=100 years.

It is designated as udometric possibility line, DDF, the graphical representations of type: $P = a.t^n$ (which represent such relation). The parameters "a" and "n" of the equation are determined by the least square method. The value of "a" increase as the return period does, while the parameter "n" may decrease or increase depending of the location.

In this type of graphical evaluation, the precipitation increases because of an increase of the time for its accumulation. The udometric possibility line, DDF, established for the pretended return period, based on precipitations with durations superior than a day, can be extrapolated for shorter durations of the day, up to a given limit: in general, it can be extrapolated up to 6 hours.

The duration of intense precipitation to be considered in the flood analysis in a section of a hydrologic web, shall match the time of concentration of the hydrographic basin - critical duration - in order to have the highest intensity of precipitation involved in that analysis, ensuring the contribution of all the hydrographic basin area for the flow at such section - critical precipitation - and consequently, give rise to the highest flood peak flow rate, for the considered return period. For precipitation durations greater than critical, the flood peak flow rate decreases due to the decrease in the average precipitation intensity.

After analysing the DDF and assigning the formula that best fits the graph, it is possible to calculate the precipitation value for the duration required for the analysis and, consequently, the intensity. In turn, the previous values are used in the calculation of the flow, using formulas for this purpose and that require those same values.

CHAPTER 6 - SUPERFICIAL FLOW

6.1 General Concepts

The **flow**, R, of a hydrographic basin, it is defined as the amount of water that cross a certain section of a course of water in a determined interval of time (year, month, days, etc.). It can be expressed in volume (m^3, hm^3, km^3) or in water height uniformly distributed over the hydrographic basin area(mm) (Rodrigues et al., 2011).

The **flow rate**, Q, of a watercourse, expresses the relation between the volume of water, ΔV, passing through a section of the watercourse and resulting from the contribution of the entire upstream catchment in a certain time, Δt (Rodrigues et al., 2011).

$$Q = \frac{\Delta V}{\Delta T}$$

Then, the flow rate indicates the volume of water that passes per unit of time, has the dimensions L^3T^{-1}, and it is generally expressed in m^3s^{-1} or ls^{-1} ($1\,mm = 1\,lm^{-2} = 1dm^3m^{-2}$) (Rodrigues et al., 2011).

It is defined as **specific flow rate**, q, the ratio between flow rate at section, Q, and the area of the region of contribution, A (Rodrigues et al., 2011).

$$q = \frac{Q}{A}$$

In this way, a flow rate per unit of surface, allows to compare two flow rate or flows rates of two different areas, such comparison is made independently of its dimensions, which it is expressed in $m^3s^{-1}km^{-2}$, $m^3s^{-1}ha^{-1}$ or $ls^{-1}ha^{-1}$. The dimensions of specific flow rate are only LT^{-1}, which it is similar to precipitation intensity which is usually expressed in mmh^{-1}. However, the use of the indicated units is usually maintained to highlight the ratio between flow rates and their respective areas of origin (Rodrigues et al., 2011).

For a given time, the **average flow rate** of a watercourse is defined as the average flow volume passing through in an interval of time (Rodrigues et al., 2011). So, it can be defined:

- Average daily flow rate (on a given day);
- Average monthly flow rate (in a given month);
- Average annual flow rate or annual module (in a given year);
- Multi-year average flow or module (over a period of several years): this value is normally used as the design flow.

When referring to the flow volume relative to a longer period of quantity than the unit of time, then, **the integral or accumulated flow rate**, **integral or accumulated flow** are designations (used indifferently) relative to a given period of time; or only flow rate or flow rate referred to a certain period of time, as for example: monthly or annual flow rate (Rodrigues et al., 2011).

Mathematically, this notion corresponds to an integration, in order of time, of the law of variation $Q(t)$ of the flow with time, within the stipulated limits,

$$R(t_0, t_1) = \int_{t_0}^{t_1} Q(t)\, dt$$

The dimension of such quantity is the same as for volume, L^3 (m^3, hm^3, etc.). It can also express a specific value (water height), when divided by the area of the region of contribution, having "length" as a dimension, L (mm) (Rodrigues et al., 2011).

6.2 Process of Flow

When precipitation begins, some of the water can be trapped by vegetation and other obstacles that prevent it from reaching the soil, returning to the atmosphere in the form of vapour. This phenomenon is called **interception** (ISEL, 2015).

If the precipitation is prolonged in time, the water reaches the soil and is initially retained in the depressions of the terrain, initiating **"infiltration"** (ISEL, 2015).

Precipitation over a given area is divided into several parcels, which its proportion varies during the duration of the event. At first, water can be intercepted by vegetation or by obstacles that prevent it from reaching the ground. If precipitation proceeds, the water reaches the earth's surface from which it evaporates, infiltrates or remains retained in depressions. During this initial period, the increase in flow in the watercourse is produced solely by the small fraction of precipitated water directly in the hydrographic network (Quintela, 1992).

From the moment that precipitation falls, it exceeds the capacities related to the previously described processes, the volume of excess water, in obedience to the laws of gravity, flows on the surface terrain to the nearest water line, giving rise to the **superficial flow**. The water lines of smaller sections (grooves, ravines, creaks, brooks) associate then, with others of successively major section (rivers), which will eventually communicate (except for rare exceptions (endorheic basins)) with the sea (Lencastre, 1992).

Surface retention refers to a portion of water that does not infiltrate or give rise to surface runoff, which is intercepted water, water stored in soil depressions, and that portion which passes to the vapour state during the occurrence of precipitation (Rodrigues et al., 2011).

Surface detention refers to surface flow or runoff water in transit on the ground and represents a rapidly variable water storage in time (Rodrigues et al., 2011).
The process of flow formation is illustrated in Figure 49 which shows the cross-sectional profile of a watercourse (Rodrigues et al., 2011).

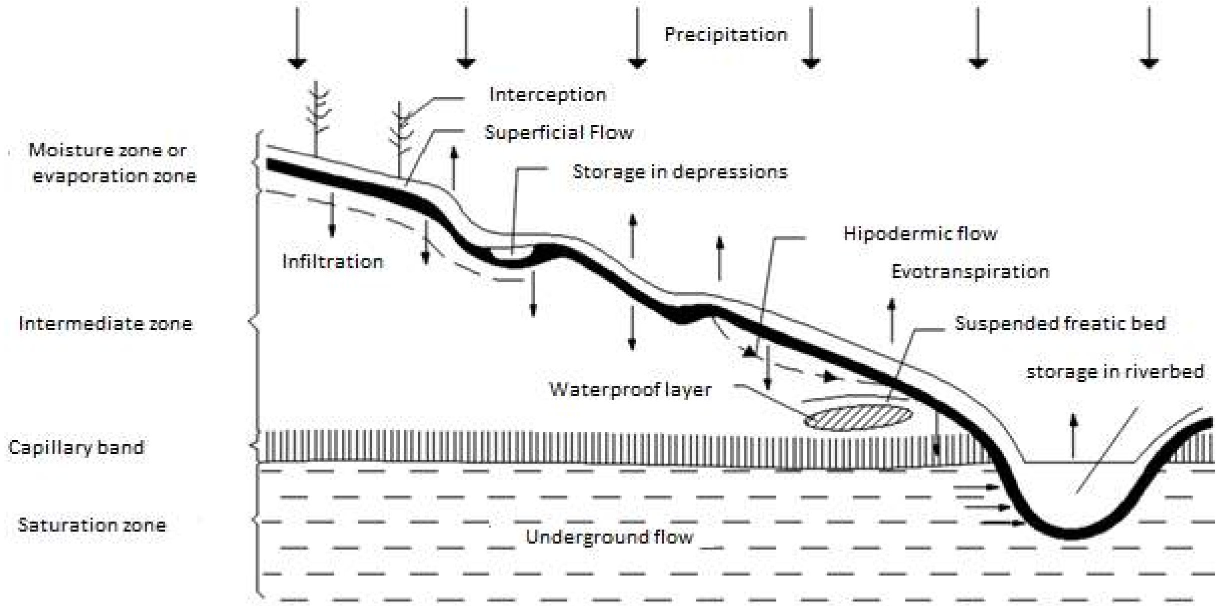

Figure 49 – Process of flow (adapted from Quintela, 1992).

In the lower part of the figure, represent the water table that constitutes a zone of saturation, in which the pores of the soil are completely filled by water subject to hydrostatic pressure. The water in this area is called groundwater, underground reservoir of water or groundwater. Above this saturation zone, three other zones are distinguished: moisture (water) zone or soil water zone or evaporation zone; intermediate zone; capillary fringe. In which water is retained by forces of molecular attraction that counteract the action of gravity, and where part of the voids is filled by air (Rodrigues et al., 2011).

The soil water zone extends from soil surface to a depth at which water can be returned to the atmosphere by plant transpiration or evaporation, depending its depth on the root depth. Hereby, it is also called the **evaporation zone** (Rodrigues et al., 2011).

In the capillary fringe, which lies immediately above the saturation zone, the water remains due to capillarity, the pores in the base being completely filled by water, whose content within the fringe decreases with altitude. The thickness of this zone varies as a function of soil texture, from values less than 0.02 m, for sandy soils, to values of about 2.50 m for thinner soils (clay and silt) (Rodrigues et al., 2011).

Between the capillary fringe and the soil water zone, exists the intermediate zone, whose thickness can vary from zero to multiple meters. The amount of water retained in this zone is at least equal to the retention capacity by molecular attraction forces (field capacity) and may be higher when the zone is crossed by moving water (Rodrigues et al., 2011).

Sometimes it is possible to find impermeable lenses above the “water table” or water table and serving as support to suspended water tables (Rodrigues et al., 2011).

When by an effect of evapotranspiration, the moisture zone (soil water zone) has water deficiency in relation to the field capacity, all the infiltrated water is retained in that area. As the water content increases, the infiltration capacity (amount of water that can infiltrate per unit of time and area) reduces, thus increasing the amount of water that flows as “Surface flow”, which will cause an increase in the flow rate in the courses of water (Rodrigues et al., 2011).

When the water content in the soil water zone or moisture zone reaches the field capacity, the infiltrated water passes into the saturation zone, enriching the groundwater reserves, which will feed the water courses with a time lag. On the other hand, part of the infiltrated water may have movement with a horizontal component, coming back to surface due to a greater permeability in the horizontal direction (Rodrigues et al., 2011).

6.3 Flow components

According to Rodrigues et al. (2011), and checking the process of flow already mentioned, the flow that cross a section of a course of water is composed according to its origin as:

- **Superficial Flow**, that reaches the hydrographic network running on the terrain surfaces, without infiltrating. It is also called **direct flow**, and results from the useful precipitation, that is, it results from the fraction of the precipitation, which, after the processes of evaporation, infiltration and superficial retention in the basin, arrives at the hydrographic network. It constitutes the most significant component of runoff during periods of intense precipitation, but as soon as it ceases the importance of this component begins to decline until it finishes;
- **Hypodermic or sub-superficial flow**, that comes from the infiltrated water that comes back to the surface, without having reached the saturation zone. It is also referred to as an **intermediate flow** resulting from the fraction of the precipitation that infiltrates, but which is shallow in the ground, due to the existence of deeper impermeable substrates, reaches the water courses with only a slight delay in relation to the superficial or direct flows and ends shortly after the cessation of the superficial runoff;
- **Underground flow**, that comes from the infiltrated water that hit the saturation zone. It is also referred to, as a **base flow**, resulting from the portion of the precipitation that was subjected to deep infiltration processes, and represents the contribution to the superficial runoff of the groundwater reserves accumulated in the geological formations where the water course passes. This component is of little importance during periods of intense precipitation but represents the entire flow as soon as the other components are exhausted.;
- **Resulting flow from precipitation in the basin,** which varies in importance according to its density and slightly with the continuation of total precipitation, since the rise of levels in water lines corresponds to an increase in of area occupied by the water surface.

As a summary of what has been mentioned, we present in Figure 50 a scheme referring to the distribution of precipitation water of constant intensity and occurring after a long dry period. In Abscissas is the time and, in order, the quantities of water forwarded in the unit of time to the various destinations (Rodrigues et al., 2011).

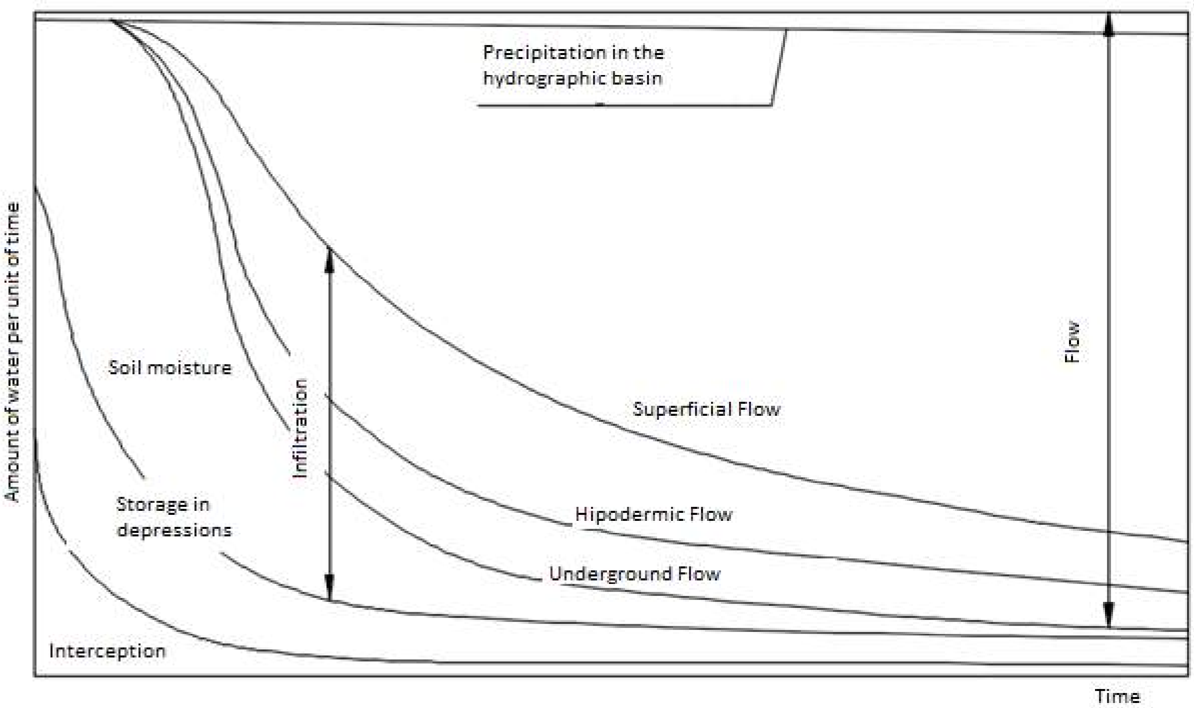

Figure 50 – Destination of precipitated water (adapted from Quintela, 1992).

In the initial period of precipitation, the increase in flow of the river comes only from precipitated water directly over the hydrographic network (Rodrigues et al., 2011).

The intensity of interception, very strong in the initial period, decreases rapidly until it reaches a constant value, corresponding to the substitution of the portion of the intercepted water that is being removed by evaporation (Rodrigues et al., 2011).

The intensity with which the precipitated water fills the storage in the depressions of the soil decreases rapidly, becoming constant and equal to the evapotranspiration that occurs during the rainy. The intensity of the infiltration is decreasing progressively as the moisture content of the soil increases. The infiltrated water is retained as soil moisture or will participate in the drains, hypodermic and underground (Rodrigues et al., 2011).

The dotted area represents the flow that passed in the considered section of the river, as a consequence of the precipitation (one part, after the precipitation is finished), which is composed by water directly precipitated in the hydrographic network and from the superficial flows, hypodermic and underground (Rodrigues et al., 2011).

6.4 Factors of Flows

Such factors that influence the flow at some section of a course of water can be classified in two groups: **Climatic and physiographic** (Rodrigues et al., 2011).

6.4.1 Climatic Factors

Related to precipitation: form, intensity, duration and distribution, in time and space, of the precipitation (Rodrigues et al., 2011).

Precipitation in the liquid form can give immediate origin to a flow in the water course, while precipitation in the form of snow can produce it with great lag in time (Rodrigues et al., 2011).

As the intensity of the precipitation exceeds or not the infiltration capacity (after satisfied the interception capacity), there will be or not superficial runoff (Rodrigues et al., 2011).

The increase in rainfall duration has the effect of gradually decreasing the infiltration capacity (by adding the water content in the soil) and consequently increasing the flow (Rodrigues et al., 2011).

The distribution of precipitation in time (time of occurrence and interval between precipitation phenomena) conditions the soil water content when precipitation starts and conditions the water availability for evaporation and transpiration (Rodrigues et al., 2011).

Conditions of evapotranspiration: evapotranspiration, responsible of the loss of water for the flow, is conditioned by temperature, solar radiation, wind, air humidity, atmospheric pressure, nature of the evaporating surface, water content in the soil, species and distribution of vegetation (Rodrigues et al., 2011).

6.4.2 Physiographic factors

Geometrical Characteristics: The area and shape of the basin have great influence on the formation of floods and, therefore, in the specific values (per unit area) of the flow of full tip or flood flow, and a small influence on the value of the annual flow, expressed in uniform water height over the basin (Rodrigues et al., 2011).

Drainage system characteristics: The drainage density influences the form of floods and annual runoff, because in it depends on the superficial route on the terrain, so then, also the greater or lesser opportunity for infiltration and evapotranspiration (Rodrigues et al., 2011).

Relief characteristics: The relief influences the infiltration and, therefore, the runoff, the water content in the soil and with this, also the evapotranspiration and the feeding of the underground reserves. On the other hand, the orientation of the basin influences the exposure to wind and solar radiation, conditioning the evapotranspiration (Rodrigues et al., 2011).

Physical characteristics: soil, vegetation and geology (Rodrigues et al., 2011).

From the soil type, depends the infiltration capacity, which is function of the dimension and distribution of pores and its stability (Rodrigues et al., 2011).

The vegetation has the effect of intercepting part of the precipitated water, delaying the runoff, giving it more time to infiltrate, and protect the soil from water erosion. The roots make the soil permeable to water infiltration (Rodrigues et al., 2011).

The geological conditions influence the soil structure, the possibility of infiltration of water in the soil and the constitution of the underground reserves that feed the watercourses in the periods without precipitation (Rodrigues et al., 2011).

It is of interest to examine in more detail the influence that the use of the soil exerts in the hydrological cycle of a hydrographic basin, translated by the occupation by forest, cultivation or urbanization (Rodrigues et al., 2011).

According to Rodrigues et al. (2011), the principal effect of forest is the deviation of the trajectory of the precipitated water, then:

- In a soil under forest vegetation, water infiltration is greater than for another form of occupation;
- Compared to smaller vegetation, the forest offers a larger area for interception;
- When soils are deep, the forest has a thicker evaporation zone in which water can be stored and returned to the atmosphere by transpiration;
- In areas with abundant and well distributed rainfall, annual total evapotranspiration is greater in forests and within these forests is higher in permanent leaf forests than in deciduous forests. In areas where precipitation is scarce and thin soils, both forests and other crops take the water content in the soil up to the wilting coefficient and therefore, there is no significant difference in annual total evapotranspiration.

Due to all this, forests play an important role as regulators of river flow, on one hand, reduces the tips of flood and on the other hand, contributing to the recharge of the aquifers that will maintain the flow in the rivers in the seasons without precipitation (Rodrigues et al., 2011).

As water is scarce in many regions, there have been attempts to increase the yield of water from the watersheds through deforestation, because when cutting a forest, it reduces the interception and evapotranspiration which, consequently, results in an increase in the humidity of the soil and runoff. However, forest clearing or deforestation from fires has drawbacks and can cause serious problems. It has been proven that the increase in runoff caused by deforestation decreases exponentially with time. On the other hand, the cutting of the forest will allow a faster washing of the nutrients of the soil, by increasing the speed of the surface runoff. Another drawback is the substantial increase in sediment transport, which impoverishes the soil by erosion of the slopes but parallelly creates problems of sedimentation at downstream, causing flooding. Another disadvantage of deforestation concerns the reduction of infiltration, and consequently the natural recharge of groundwater reserves (Rodrigues et al., 2011).

As for the influence of soil cultivation, the replacement of trees and shrubs by smaller plants and shorter vegetation period, generally reduce evapotranspiration and increased runoff. The reduction of vegetation and the creation of a bare soil during part of the year, increase the irregularity of the river flow. The uncovered soil, when subjected to heavy rainfall is more prone to erosion and occurs with a higher peak flow rate rates or flood flows (Rodrigues et al., 2011).

As for the influence of urbanization, the waterproofing that implicates vast areas gives rise to the reduction of surface retention and infiltration. The most important effect on the liquid flows in a quantitative aspect is the increase of the flood flows (tips flow) or maximal flow and decrease of the underground reserves (Rodrigues et al., 2011).

6.5 Measurement of superficial flow

Unlike all other components of the hydrological cycle, which can only be quantified by sampling, the flow can be entirely measured (Rodrigues et al., 2011).

There are several methods for measuring flow rates or flow rates, but the most used in natural water courses is the so-called **"section-speed" method**. Another is the **"structural" method**, which results from the possibility of using certain hydraulic structures, usually dischargers, but sometimes also gates. Other methods exist that rely on techniques with restricted use, such as the "dilution" method, the "ultrasonic" method, the "electromagnetic" method or the "moving boat" (Rodrigues et al., 2011).

6.5.1 "Section-speed" method

By Rodrigues et al. (2011), the flow rate measurement Q by this method is based on the measurement of the surface S, a cross section of the watercourse, and the average velocity U through that section, the flow rate value being given by,

$$Q = U \times S$$

Usually, the section is divided into parts, and the respective flow rate Q_i is determined for each of them. Then the total flow, Q, is obtained by adding up the values of each part,

$$Q = \sum_{i=1}^{n} Q_i$$

In most current measurements, surveys are performed on several verticals in the cross-section, along with measuring the distances of those verticals to a reference point located at one of the margins, so as to obtain a cross section profile (Figure 51), and velocities are measured at points of these same verticals, using windlasses (Rodrigues et al., 2011).

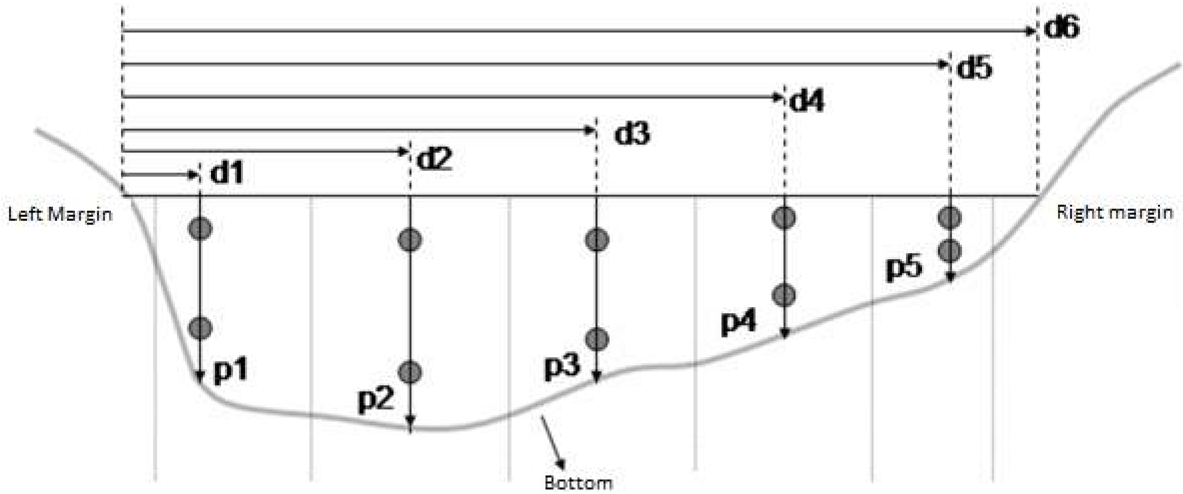

Figure 51 – Profile of a transversal section of a course of water by sampling (UTFPR, 2005).

The windlasses (Figure 52) are instruments provides by an helix connected to an axis, which counts rotation, that are parallel measured in time, allowing to determinate the angular velocity of an helix (Rodrigues et al., 2011).

Figure 52 – Windlass of helix and rotation counter (source: http://www.hidrometria.com.br).

The speed acquired by the propeller within the flow tends towards a one-to-one relationship with the speed of the same flow. The ratio between the water velocity and the number of revolutions of the windlass is determined in previous laboratory calibration tests by moving the windlass at a certain speed in standing water (Rodrigues et al., 2011). The calibration equation is called the characteristic curve of the windlass and is of the type:

$$U = a + b \times n$$

where U is the speed of water, n the number of rotations of the windlass in an interval of time, and a and b are two characteristics constants of each device.

The section flow rate can then be determined arithmetically. Thus, at each vertical the average V_i is determined from the velocities measured at different depths, and then the flow rate of the section is estimated (Rodrigues et al., 2011).

$$Q = \sum_{i=0}^{n-1} \left(\frac{\overline{U}_i + \overline{U}_{i+1}}{2}\right)\left(\frac{p_i + p_{i+1}}{2}\right)(d_{i+1} - d_i)$$

where p_i and d_i represents, respectively, the depth at vertical and distance from origin of reference.

The determination of an average speed, $\overline{U}_i$, in each vertical, of depth p_i, can be simplified by using only two meassurements for its determination in previously determined depths, such that (Rodrigues et al., 2011):

$$\overline{U}_i = U_{0.6p_i} \vee \overline{U}_i = \frac{1}{2}\left(U_{0.2p_i} + U_{0.8p_i}\right)$$

where $U_{0.2p_i}$, $U_{0.6p_i}$ and $U_{0.8p_i}$ represents, respectively, average velocities at 0.2, 0.6 and 0.8 of vertical depths of order i.

6.5.2 Structural Method

Fixed hydraulic structures can be used to measure river flows. These structures, which can be dischargers, channels or gates, are more frequent in the upper and middle sections of the water courses than in the lower ones. In the latter, the width required for such structures makes construction prohibitive, and there may also be problems of flooding at upstream, due to the fact, that in these river sections the longitudinal slopes are reduced. However, in the upper sections of watercourses, difficulties may also arise in relation to their coarse sediment transport capacities, which are generally high (Lencastre, 1984).

The use of a hydraulic structure in flow measurement is based on the principle, that a ratio between the flow rate and the water level at upstream of the structure or between flow rate and simultaneous levels at up- and down- stream can be determined either theoretically or experimentally (Rodrigues et al., 2011).

Among the hydraulic structures for measuring flow rates, dischargers are the most used (Rodrigues et al., 2011). They consist of structures designed to be overpassed or by water, and can be:

- **Thin Threshold** (Figure 53), when part of the threshold is in contact with water, which is the thickness of the threshold of the discharger with ignorable dimensions in relation with the height of the discharger laminate. Such type is only used as flow rate measure devices;

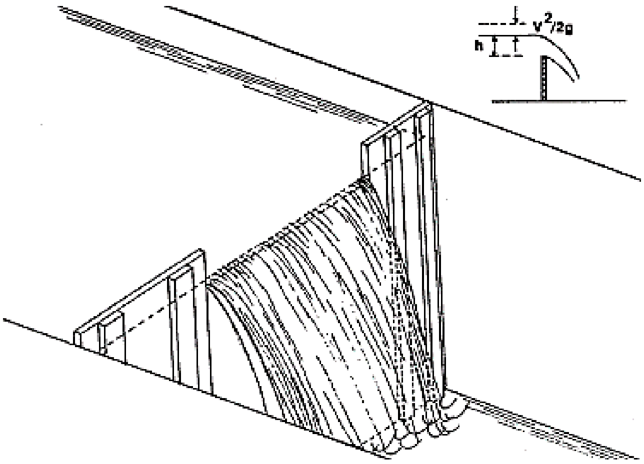

Figure 53 – Discharger of thin threshold (Rodrigues et al., 2011).

- **Thick threshold** (Figure 54), in other cases. Normally, such dischargers became part of hydraulics structures with other uses (damn, etc), but that can be also used as measure of flow rate devices.

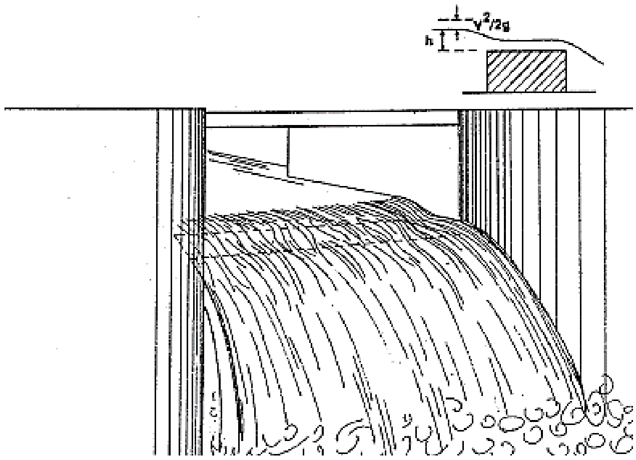

Figure 54 – Discharger of thick threshold (Rodrigues et al., 2011).

Within these two types of unloaders, there are several models characterized by the geometry of its crest: triangular, rectangular, trapezoidal, circular, etc. (Rodrigues et al., 2011).

In most cases, the flow passing through the discharger is obtained by an expression that relates it to the hydraulic load $Q = f(h)$ which is fixed for a given discharger geometry (Rodrigues et al., 2011). In general:

$$Q = \mu \times L \times \sqrt{2 \times g} \times h^{3/2}$$

Where Q is the flow rate that pass through the discharger, μ is the coefficient of discharge (varies depending of the discharge type, usually between 0,35 and 0,45), L is the length, g is the acceleration of gravity and h is the hydraulic charge – which is the difference between the energy lines (away from the zone next to the discharger , where energy line coincide with a free surface), at upstream and after the discharger (Rodrigues et al., 2011).

The application of such an expression implies that the downstream level of the discharger does not rise above a certain level in order to prevent its "drowning" (Rodrigues et al., 2011).

6.5.3 Flow curve

The **flow curve** constitutes the bi-unambiguous ratio between the flow rate in a given section and the corresponding water height (level). The existence of a ratio between these two quantities is a fundamental requirement for the determination of the flow rate drained in a section, through the existence of a continuous level register in the same section (Rodrigues et al., 2011).

The flow curve is obtained from the set of pairs of values resulting from the measurement of the flow rate and the observation of the water height. The water height is called hydrometric height and is determined by reading on a hydrometric scale, placed in the measuring section (Rodrigues et al., 2011).

In unloaders with regular geometric shapes, the flow curve can be accurately expressed by a theoretical analytical expression. This is no longer the case in the irregular sections of natural water courses, where it is possible to use graphic or analytical processes to adjust a curve to the results of as many joint measurements of flow rates and hydrometric heights - Figure 55 (Rodrigues et al., 2011).

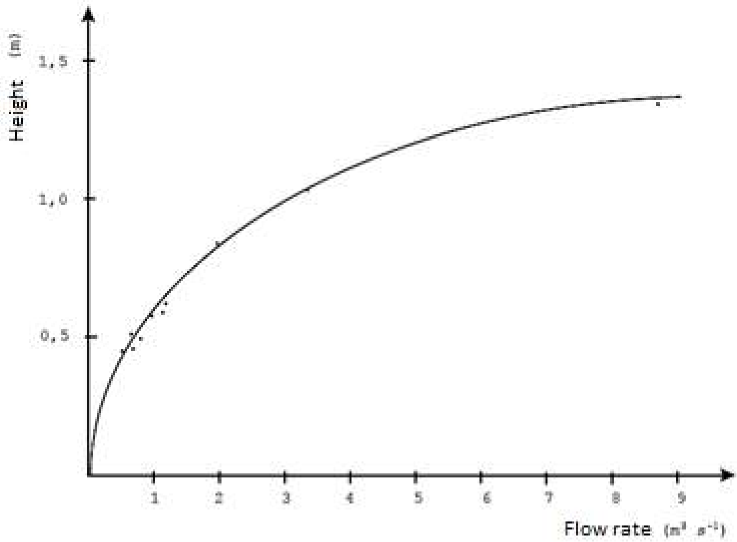

Figure 55 – Flow curve (Rodrigues et al., 2011).

In flood periods, the ratio between the hydrometric heights and the flow rates can be far away from the bi-univocity conditions on which the establishment of the flow rate curve is based. That is, for a given hydrometric height the flow rate is higher during the ascent phase and lower during the descent-stress phenomenon (Figure 56). This is due to the fact that during the ascent phase the downstream level is lower, which facilitates the flow, and during the descent is greater, which hinders the flow. However, when the distance between the ascending and descending branches is not significant, the average can be taken as a biunivocal flow curve (Rodrigues et al., 2011).

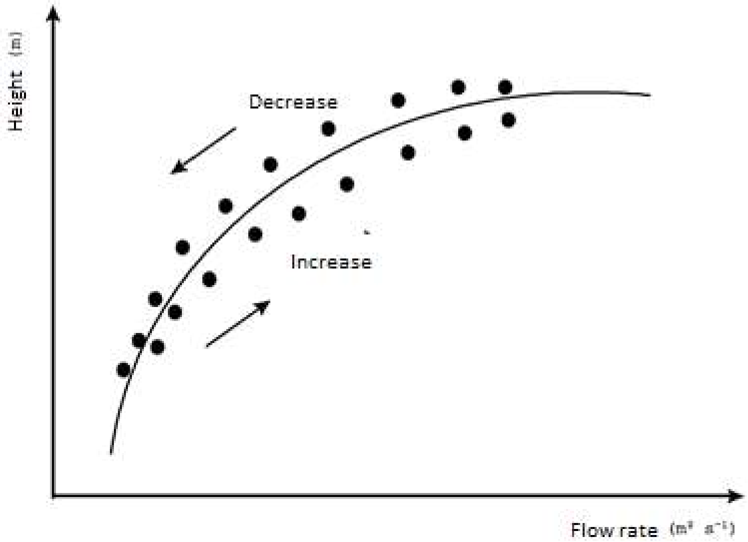

Figure 56 – Stress of a flow curve (Rodrigues et al., 2011).

Analytically, flow curves can be represented by different types of expressions, being more used,

$$Q = a \times (h + h_0)^b$$

where Q is the flow rate, h the hydrometric height, h_0 the height of zero in the hydrometric scale in relation to the level of water which correspond to a flow rate of zero, which in general is the lowest height of the section, also known as riverbed (h_0 is positive is the scale is above the level of flow rate zero, and vice versa - Generally, the zero of the scale and the lower height of the section do not coincide, and there is a zero of the scale buried in the bed, sometimes suspended in the margin, respectively, by sedimentation or erosion phenomena), a and b are characteristic parameters of the section, to be empirically determined (Rodrigues et al., 2011). By applying logarithm,

$$\log Q = \log a + b \times \log(h + h_0)$$

that in a graph with logarithmic coordinates translates by a line. From this last expression and from the set of pairs of values (Q_i, h_i), the values of a and b can be calculated by averages of a regression analysis by the least squares method, provided that they know h_0. Therefore, several values of h_0 (and the corresponding values of a and b) are arbitrated , then, choosing the set of values of h_0, a and b for which resulted in the best graphic adjustment to the pairs of values (Q_i, h_i) or the highest correlation coefficient (Rodrigues et al., 2011).

The fact that most natural water courses are constantly evolving, undergoing erosion and/or sedimentation processes, makes it essential to permanently update the flow curves, through the periodic execution of new joint measurements of heights and flow rates (Rodrigues et al., 2011).

6.5.4 Registration of Hydrometric levels

The values of the hydrometric height can be obtained, discontinuously, by visual observation of a hydrometric scale, also called **limnometric scale or limnometer** (Figure 57) or continuously, by averages of a recording device called **limnograph**. They consist of a water level measurement mechanism in the section and a continuous registration mechanism of the same levels (Rodrigues et al., 2011).

Figure 57 – Limnometric scale or Limnometer (source: www.grupoconstruserv.eng.br).

As for the respective level measurement mechanism, limnographs can be of various types: float limnographs, pneumatic limnographs, bubbles limnographs; and as for the type of registration, Limnigraphs can be: graph limnographs; digital registration limnographs (Rodrigues et al., 2011).

Nowadays, all hydrometric stations (Figure 58) are being equipped with limnographs. However, in the stations where this does not exist, the interval between readings of the hydrometric scale should be fixed in order to avoid appreciable error in the evaluation of daily runoff, which should be lower in the rainy season, and particularly during floods, due to the greater variation in the level of water that is then verified. Lately, the classic level measuring equipment begins to be replaced by the pressure probes connected to data acquisition systems, which allows the immediate use of the registers in digital format, and also enable the easy integration of transmission systems of values in almost real time (Rodrigues et al., 2011).

Figure 58 – Hydrometric station (source: https://snirh.apambiente.pt).

6.5.5 Hydrometric network

A **Hydrometric station** is designated as a section of a water course where a periodic registration of levels is made, and where a flow curve is defined to convert the respective values into flow rates. Hydrometric stations can be **limnometric**, which are only provided with a hydrometric scale for periodic reading of levels, and **limnographical,** which are provided with a limnograph for continuous level recording. The set of hydrometric stations in a region or country constitutes its **hydrometric network** (Rodrigues et al., 2011).

According to Rodrigues et al. (2011), the general purposes of the observations made in a hydrometric network are:

- **Obtaining data for planning** (planning and design of hydraulic works and modelling of a watershed). For this purpose, the existence of historical successions of hydrometric observations, that is, of records of measurements carried out over a certain period, is essential. A succession of hydrometric data, to be good, needs to have at least 20 years of observations, or even more, when dealing with very irregular regime basins. It is, therefore, clear the necessity of installing a basic hydrometric network, even when there is no immediate need to carry out hydrological studies;
- **Obtaining operational data** (real-time management of a river system). These data are intended to allow decision-making in very short periods of time, particularly in alarm or emergency situations, so it is so important to speed up its transmission as the quality of its measurement. In order to obtain these data, it is associated the development of modern telemetry systems, which comprise, in addition to the hydrometric stations, an automatic communication system of the information obtained from them, via radio or telephone, for a central command of the system, where decisions are made concerning the opening or closing of floodgates, the launching of flood warnings, etc.

Also, at Rodrigues et al. (2011), Hydrometric stations can be classified into:

- **Main or basic**, permanent stations that operate on a continuous basis and are intended to provide the basic elements for the statistical study of the flow;
- **Secondary**, its operation is limited to a certain number of years, and are intended to provide additional data that may be extrapolable beyond its operating period;
- **Special or tertiary**, are designed to obtain elements for specific studies and are not part of the Hydrometric network.

6.6 Spatial distribution of the flow

To represent the spatial distribution of the flow, we can draw up letters of isolines of the flow-geometric places of the points which, per unit area in plant, contribute to the hydrographic network with equal amount of water. This amount of water can reach the hydrographic network by several routes: surface flow, hypodermic or underground (Rodrigues et al., 2011).

The Flow Isoline charts refer more often to the annual flow (in a given hydrological year) and to the average annual flow (average annual flow in the range of several hydrological years). Figure 59 shows the Isoline chart of the average annual flow (Rodrigues et al., 2011).

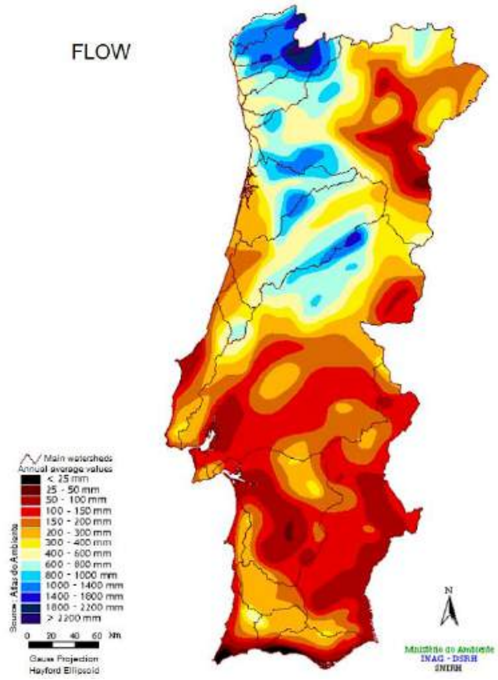

Figure 59 – Chart of annual average flow in Portugal (mainland) (adapted from Rodrigues et al., 2011).

6.7 Temporal distribution of the flow

The hydrometric observations obtained by readings isolated from the hydrometric scale, which provide the value of the flow rate drained at different times (by reading the flow curve), give rise to successions of discrete values (Rodrigues et al., 2011).

The hydrometric observations obtained by continuous level readings, which allow the knowledge of the instantaneous flow evolution, give rise to several types of successions, which can vary from the registers themselves to the discrete successions (Rodrigues et al., 2011).

According to Rodrigues et al. (2011), the forms of presentation of the hydrometric observations can thus be generically the following:

i) **Chronological series**, are formed by values presented according to their order of occurrence, involving:
   a. **Chronological series of instantaneous flows,** consisting of instantaneous flow values, resulting from the direct transformation of

the limnograhs, by use of the flow rate curve. Its graphical representation is the Hydrograph (Figure 60);

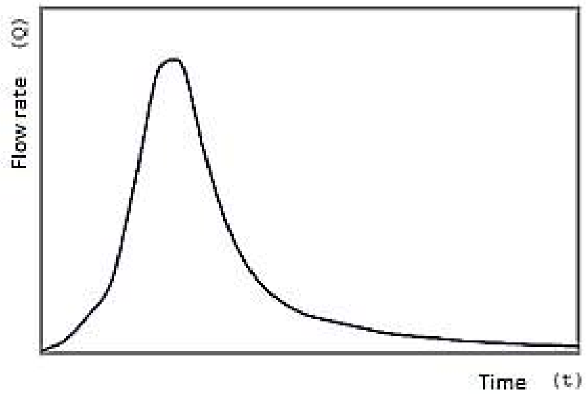

Figure 60 – Chronological curve if instantaneous flow rates or hydrograph (Rodrigues et al., 2011).

b. **Chronological series of average flow rates,** consisting of the average values of equal or successive periods-hours, days, weeks, months or hydrological years (Figure 61);

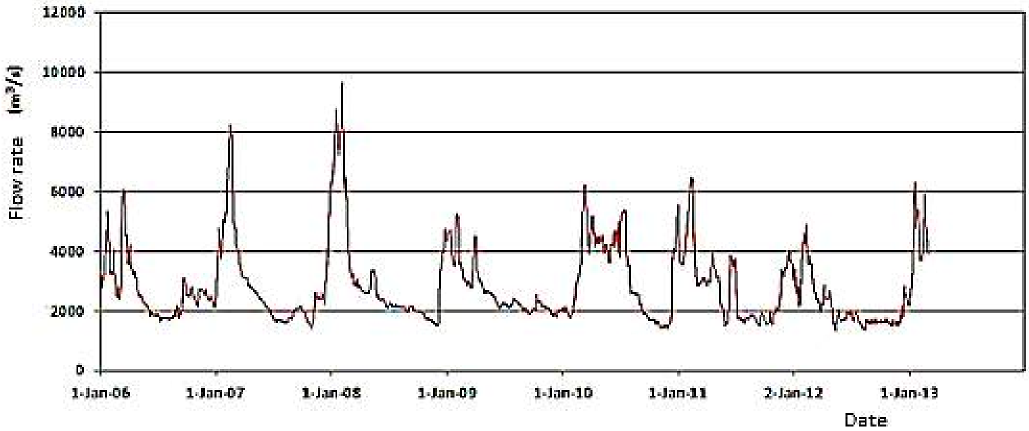

Figure 61 – Average daily flow rates at station E291 - Caia (source: http://www.coba.pt).

The integration of a chronological succession of flow rates, $Q(t)$, gives the value of the volume or integral flow rate, $R(t)$, drained in a given period, $\Delta(t)$, corresponding to the succession. Dividing this value by the interval duration, gives the average flow rate in such interval, which is a fictitious flow rate because it does not necessarily occur.

ii) **Accumulated series**, are formed by the values of the drained volumes or integral flow rate in chronological order, thus constituting the integral successions of the chronological succession. It gives to the considered section at every moment, the total volume of water that has passed in the section since the considered origin of the time, so, becoming a great tool in the study of affluents in places, where it is intend to build storage reservoirs (Figure 62);

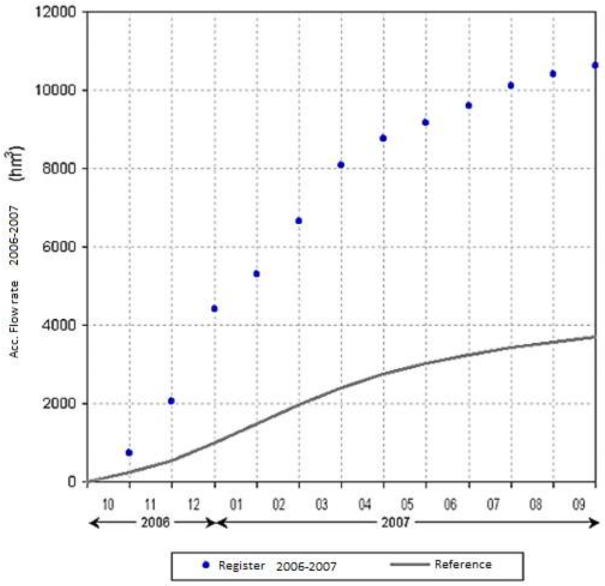

Figure 62 – Example of a curve of accumulated flows (source: Confederação Hidrográfica do Norte).

iii) **Classified series**, the values of the flow rates, Q, are grouped in order of magnitude. The corresponding graphic representation, having as ordinate value ,the flow rates and as abscissa, the number of days in which are equated or exceeded, has the designation of **the duration curve of average daily flows** and is of great importance in the studies of hydraulic evaluations (Figure 63);

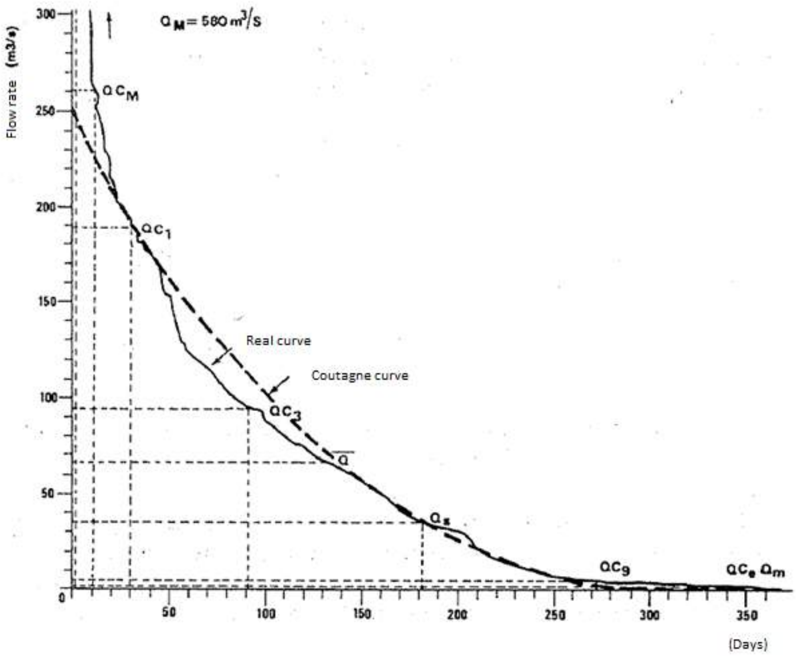

Figure 63 – Duration curve of daily average flow rates of Mondego river in Coimbra (Rodrigues et al., 2011).

The widespread employment of annual curves of flow duration led to the fixation of a terminology of its own for some of its points, which is considered to define the characteristic flow rate of a water course:

- Maximum flow rate (Q_M), maximun registered flow rate or flow rate, or maximum predicted flow rate, with a certain return period;
- Maximum characteristic flow rate (Q_{CM}), flow rate equated or exceeded only in 10 days of the year;
- Median or semi-permanent characteristic flow rate (Q_S), flow equalled or exceeded at 6 months of the year, with great interest in the study of water-based evaluations;
- Typical flow rates of 1, 3 or 9 months ($Q_{C_1}, Q_{C_3}, Q_{C_9}$), equated or exceeded flow rates, respectively, at 1, 3 or 9 months of the year;
- Minimum or dry flow characteristic (Q_{C_e}), flow rate equated or exceeded in 355 days per year;
- Average or modular flow rate ($\bar{Q}$), is equivalent to the average of the classified flow rates;
- Minimum flow rate (Q_m), minimum recorded flow rate, or predictable minimum flow rate, with a given return period.

Given the importance of the flow duration curve, some authors attempted to adapt to the observed curve, a mathematical expression of a few parameters, that represents it, with enough approximation. For example, the expression proposed by Coutagne is presented, where the curve of the graded flow is translated in terms of a parabola of a degree n:

$$Q = Q_m + (\bar{Q} - Q_m) \times (n+1) \times \left(\frac{T-t}{T}\right)^n$$

where Q represents The average daily flow equaled or exceeded during t days in the course of T days; $\bar{Q}$ is the modular flow rate of the period; Q_m is the minimum flow rate of the period; n is a characteristic parameter of the water course, to which Coutagne proposed to call "the coefficient of irregularity". The value of such coefficient is usually determined considering $Q = Q_S$ (where $t = 182.5$ and $T = 365$ days; Q_S is the rate equalled or exceeded in 6 months of the year, erg. the flow duration of $365/2 = 182.5$ days in a year. Then, $T = 365$ days and $t = 182.5$ days), which allows to transform the previous equation into:

$$\frac{Q_S - Q_m}{\bar{Q} - Q_m} = \frac{n+1}{2^n}$$

to solve in relation to n by an iterative calculus.

6.8 Estimation of flow in the absence of measurements

In the absence of hydrometric measurements in a given section of a water course, the surface flow values in the same section are estimated by indirect processes, which are indicated below (Rodrigues et al., 2011).

6.8.1 Annual Values

According to Rodrigues et al. (2011), the estimation of the annual flow values, can only be used the precipitation registers, or it can be used simultaneously with records of precipitation and temperature:

- **From precipitation measurements in a basin defined by a concerned section.** It is usually used a statistical regression/precipitation calculated for another section, in the same basin or in another neighbour and which is applicable to the concerned section. In general, it is admitted that this regression is translated by the equation:

$$R = a + b \times P$$

where R and P are the annual values, respectively, of the flow in the section and the precipitation in the basin defined by it, in the same units of water height, usually in mm; A and B are the regression parameters.

- **From precipitation and temperature measurements**. It is possible to establish a formula that relates the flow deficit, *D,* with the annual precipitation *P* and average annual temperature *T*. The formula of this most widespread type is the TURC formula, deduced from observations in 254 river basins located on 4 continents and subjected to various climates, with the following equation applicable to annual values,

$$D = \frac{P}{\sqrt{0.9 + \frac{P^2}{L^2}}}, valid\ for \frac{P^2}{L^2} > 0.1$$

where D and P are expressed in mm; $L = f(T)$ is called evaporating power of the atmosphere, which constitute the superior limit of the flow deficit values and is given by,

$$L = 300 + 25 \times T + 0.05 \times T^3$$

where T is expressed in °C.

The flow deficit, D, translates the difference between the precipitation on the basin, P, and the flow in the final section of the water course, R, and can be considered equal to the actual evapotranspiration of the basin, E , as it results from the simplified equation (if the beginning of the hydrological year is chosen in such a way that water reserves are constant and if the quantities of water introduced by man are null, it is possible to write the hydrological balance equation in the simplified form,, $P - R = E$) of the hydrological balance,

$$P - R = D = E$$

For $P^2/L^2 \leq 0.1$, consider $D = P$ in the last equation, so $R = 0$.

6.8.2 Duration values lower than annual

Here, in addition to the risks inherent to such method, it should be considered that the flows relating to those periods may be dependent on the immediately preceding periods, and the more strongly as longer is the considered interval of time (Rodrigues et al., 2011). In the case, that it is necessary to estimate values in referred times, and in the absence of any other type of information, it can be tried to overcome the problem by using chronological succession, accumulated or classified, determined in the same year in sections that define basins with similar physiographic and climatic characteristics and adjusting the values of these successions with the proportion of annual flows in both sections, according to the following equations:

$$Q_2(t) = \frac{A_2 \times R_2(year)}{A_1 \times R_1(year)} \times Q_1(t) \vee R_2(\Delta t) = \frac{R_2(year)}{R_1(year)} \times R_1(\Delta t)$$

where $Q(t)$ represents the value of flow rate; R a value of flow, measured in water heights; A is the area of the basin; 1 and 2 represents respectively the section in comparison and the section of study. $R_2(year)$ is determined indirectly by the mentioned processes. Nowadays is common to use hydrological models to obtain series of flow from precipitation and other components of the hydrologic cycle of easiest determination or calculus (Rodrigues et al., 2011).

6.9 Study of a Hydrograph

6.9.1 Components

According to Rodrigues et al. (2011) When analysing a hydrograph in a section of a water course, the following components of the flow can be considered (figure 64), which run through this section:

- Base flow or underground flow;
- Direct flow or surface runoff or surface flow;
- Intermediate flow or hypodermic flow;
- Flow resulting from precipitation over the hydrographic network.

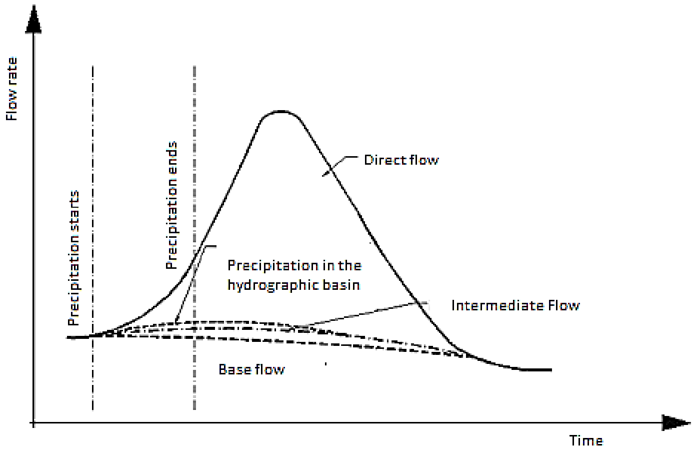

Figure 64 – Components of a hydrograph (adapted from Rodrigues et al., 2011).

Whether the base flow or the intermediate flow, can be both expressed as exponential of,

$$Q_t = Q_0 \times e^{-a \times t}$$

where, Q_t is the flow rate in the section in the instant t; Q_t the flow rate at the beginning of the considered period; e is the natural logarithmic base number; a is the considered coefficient characteristic of local formations. In case of a base flow, the referred equation expresses the flow curve of underground reserves (Rodrigues et al., 2011).

In the analysis of hydrographs, it is often considered only the direct flow and the basic flow, due to the reduced relative importance of the other components (Rodrigues et al., 2011).

6.9.2 Separation of the hydrograph components

The exact separation of all components previously considered in the runoff it is very difficult to perform. However, some more or less empirical techniques have been developed to solve the problem. One of the most simplistic, but widely used, is to unite the point of the beginning of the rise of the hydrograph with point N where it is thought that the direct flow ends (Figure 65) (Rodrigues et al., 2011).

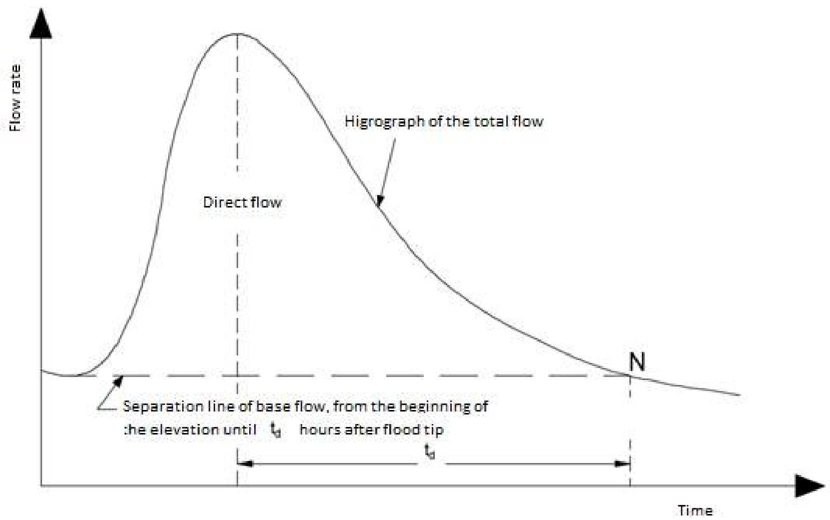

Figure 65 – Separation of the components of a hydrograph (adapted from Rodrigues et al., 2011).

The position of this last point can be determined either subjectively, based on the previous experience of the analyst, or using empirical formulas, as the type of the following formula, due to Linsley (1982),

$$t_d = 20 \times A^{0.2}$$

where t_d is the duration of direct flow after the flood tip, in hours; A is the area of the basin, in km^2 (Rodrigues et al., 2011).

6.9.3 Form of the hydrograph

According to Rodrigues et al. (2011) hydrograph, registered after an isolated rainfall occurring in its basin, usually has the form of an asymmetric bell, where the following four distinct parts can be considered (Figure 66):

- The **growth curve**, corresponding to the increase of flow motivated by the increment of the flow, and that occurs during the **time of growth** or time for the tip, t_p;
- the **tip of the hydrograph**, which is its maximum value;
- The **decreasing curve**, corresponding to the progressive decrease of direct flow, and that occurs during the **decreasing time**, t_d. The sum of growth and decreasing times corresponds to the **baseline time** of the Hydrograph, t_b;
- The depletion curve, already mentioned, corresponding to the exponential decrease of the base flow, after the contributions of the remaining components of the surface runoff or flow have ceased.

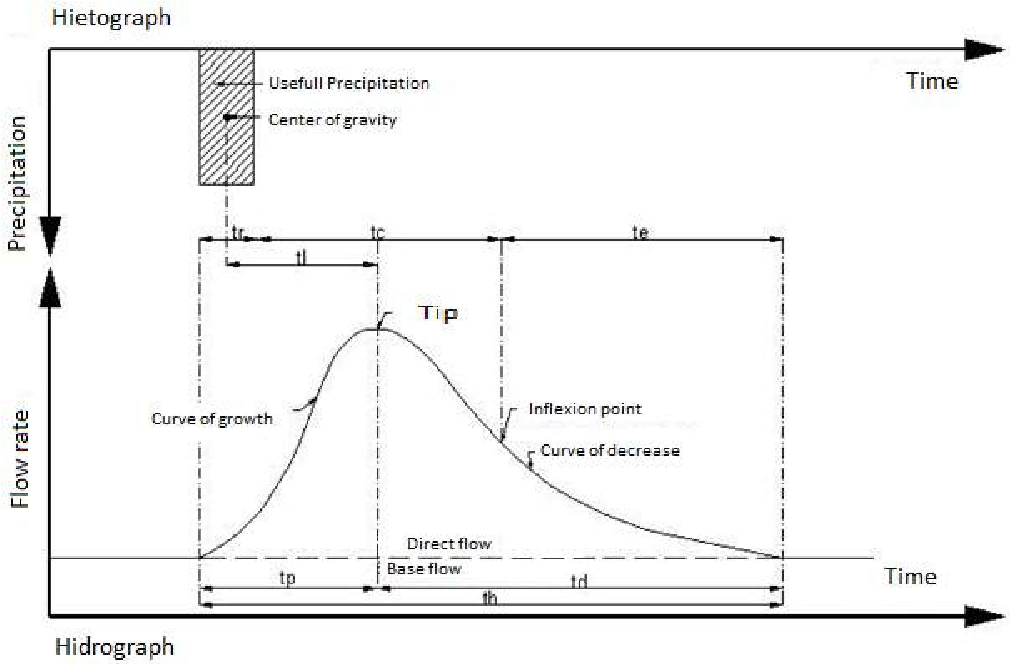

Figure 66 – Characteristics of a typical hydrograph (adapted from Rodrigues et al., 2011).

It is called **time of response of a basin**, t_l, the time interval defined by the instants corresponding to the center of gravity of the useful precipitation and the tip of the hydrograph. It represents the lag between the chronological flow rate curve at the section and the chronological flow rate curve corresponding to the precipitation, supposedly uniformly split throughout the basin, at the moment it occurs (Rodrigues et al., 2011).

It is called **precipitation time**, t_r, the time during which a useful fraction of the rainfall occurs, giving origin to a direct flow of the hydrograph (Rodrigues et al., 2011).

The time of concentration of a basin, t_c, is the time required for a entire area to contribute to the runoff or flow in the output section; It can also be defined as the time required for a drop of water dropped at the hydraulically furthest point of the basin to reach the exit section. In a hydrograph resulting from a useful precipitation that uniformly covers the entire basin, it corresponds to the time interval between the cessation of precipitation and the occurrence of an inflection point in the decreasing curve. It is considered as a constant characteristic of the basin, regardless of the characteristics of the rains (Rodrigues et al., 2011).

For each statistical frequency, it is called critical rainfall of a basin to a uniform rain that is likely to cause the highest value of the peak flow rate or flood tip. By defining the time of concentration, it is understood to have to be the duration of the critical rainfall equal to or greater than the time of concentration of the basin, i.e., $t_{r_{crit}} \geq t_c$ (the notion of critical rainfall only has averaging in small basins, due to the improbability of the occurrence of uniform rains lasting equal to the respective time of concentration in large river basins) (Rodrigues et al., 2011).
In a hydrograph there is still to consider the **emptying or draining time** of the hydrographic network, t_e, between the occurrence of the inflection point in the decreasing curve and the cessation of direct flow; corresponds to the passage in the section, of a volume of water stored in the network during the rain. The respective value depends on either fixed factors (geometric characteristics of the network channels), or variable factors (characteristics of the rains) (Rodrigues et al., 2011).

The base time of the hydrograph corresponds to the passage of the direct flow in the section. For a useful precipitation uniformly distributed over the entire basin, direct flow begins to occur immediately after the beginning of such precipitation; in the same way, only ends when, after the final contribution at the most distant point reaches the output section, going through the entire volume of water stored in the basin during the precipitation. The base time of the hydrograph, which was already referred as the sum of the growth and decreasing times, can also be considered as the sum of the precipitation times, the time of concentration, and the draining time of the basin, i.e. (Rodrigues et al., 2011):

$$t_b = t_p + t_d = t_r + t_c + t_e$$

6.9.4 Factors that affects the form of the hydrograph

The factors influencing the distribution of the superficial runoff or flow, i.e., the shape of the hydrograph. It should be noted here, that the factors related to precipitation (form, intensity, duration and distribution), predominantly influence the definition of the growth curve of the hydrograph and the physiographic factors of the basin (area, shape, drainage density, relief, soils and geology) influence the decreasing curve.

Regarding precipitation, it is important to define the following parameters:

- Precipitation intensity (i) - the quotient between the precipitation height and the time interval considered;
- Infiltration rate - rate at which water enters the soil;
- Infiltration Capacity (f) - maximum infiltration rate (when at the surface of the soil there is available water for the infiltration process - provided such

precipitation water, it can be affirmed that the infiltration rate equals the infiltration capacity, if such capacity is less than the intensity of precipitation; otherwise, an infiltration rate equal to the intensity of the precipitation will occur);

- Field Capacity (e) - is the moisture content (ratio between the volume of water of a soil and its total volume) of a natural soil that has been saturated and left to drain freely, i.e. the residual amount of water that a soil manages to retain against the action of gravity (e) (water linked to grains by adhesion, cohesion or capillarity forces);
- Porosity (nr) - ratio between the volume of the pores filled by fluids and the total volume of the sample.

Based in Rodrigues et al. (2011) are not schematized the effects of some of the above-mentioned factors, which begin to describe:

a) i ≤ f and e ≤ nr - Hydrograph of a perennial water course composed solely of the base flow. The occurrence of a rainfall whose intensity is inferior to the soil infiltration capacity (i ≤ f - absence of direct flow) and in a situation where its field capacity is not satisfied (e ≤ nr - absence of intermediate flow), causes only an imperceptible rise in flow, due solely to the precipitation on the water course itself.
b) i ≤ f and e > nr - Hydrograph of a water course resulting from the occurrence of an intermediate flow. If the precipitation intensity is lower than the soil infiltration capacity (I ≤ f - there is no direct flow), but if its field capacity is satisfied (e > nr), the infiltrated water will originate intermediate flow and increase the contribution of base flow.
c) i > f and e ≤ n_r - Hydrograph of a water course resulting from the occurrence of a direct flow. If the field capacity of the soils of the basin is not satisfied (e ≤ nr - absence of intermediate flow), but if the rainfall intensity exceeds the soil infiltration capacity (I > f), the increase of the flow in the river is due solely to the surface flow.
d) i > f and e > n_r - Hydrograph resulting from a situation in which the field capacity of the soils of the basin is satisfied (e > nr), either the intensity of precipitation exceeds the infiltration capacity (I > f); in this situation all the components considered in the constitution of the superficial flow occur, that is, the increase of the river flow is due to the direct, intermediate and basic flow.
e) and f) - Hydrographs resulting from the occurrence of equal rainfall but with different spatial distribution in the same basin.
g) and h) - Hydrographs resulting from the same rainfall in identical area basins, but in a different way.

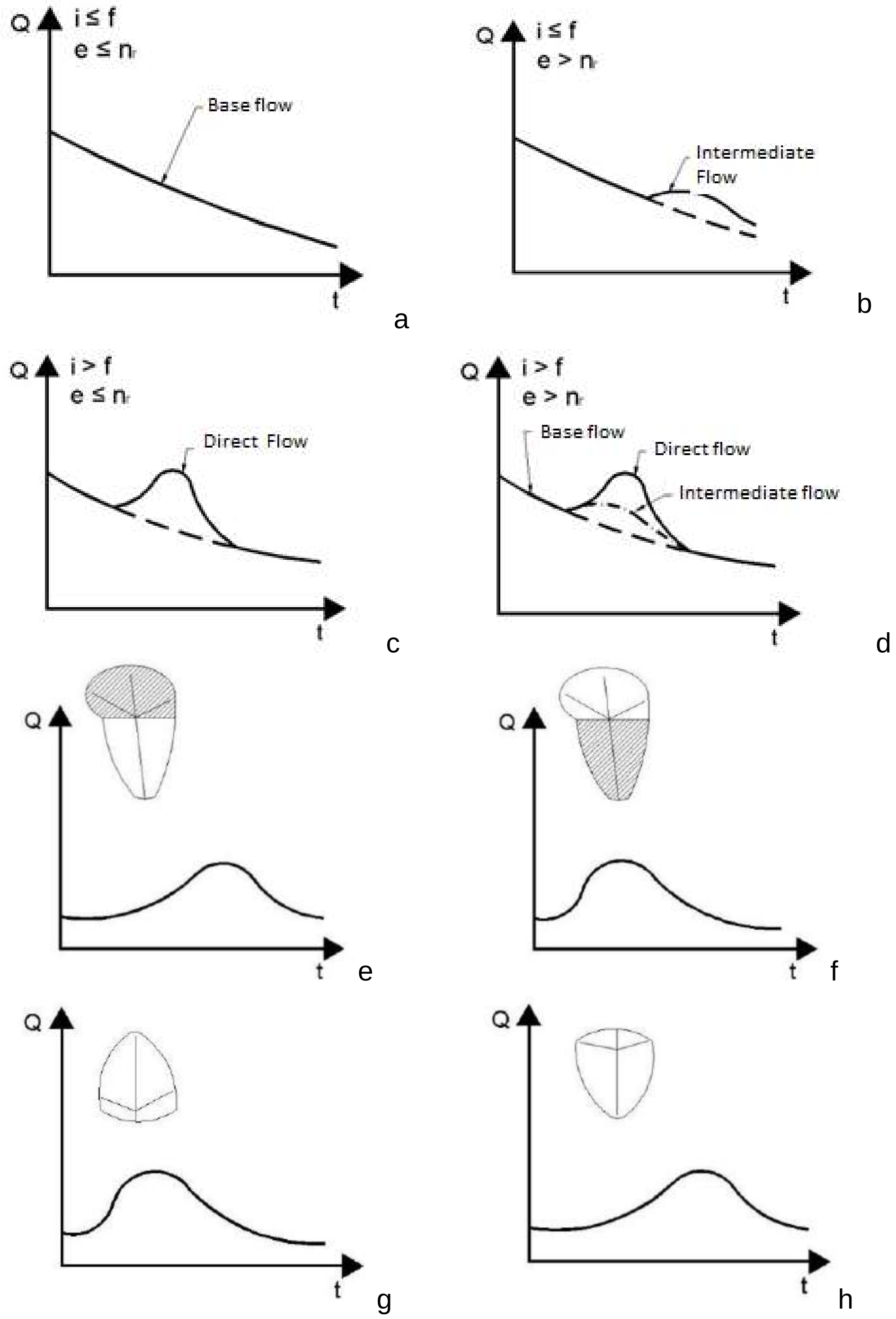

Figure 67 – Effects of the characteristics of precipitation and the basin in the form of the hydrograph (adapted from Rodrigues et al., 2011).

6.9.5 Unitary hydrograph

The effect that the amount and intensity of rain causes on a hydrograph is studied through the unit Hydrograph method. Leroy S. Sherman in 1932 presented the following proposition: "If two rains occur on a hydrographic basin under identical conditions, before the rains, the direct flow hydrographs of the two rains may be supposed equal"(Costa & Lança, 2011).

Unitary Hydrograph is the result of a surface flow (unit) corresponding to 1 cm of water height over the entire basin (Costa & Lança, 2011).

The **Unitary hydrograph** is ruled by three foundations:

1º Principle – Time of constant base

Rains of equal durations originate equal durations of superficial flows (Figure 68) (Costa & Lança, 2011).

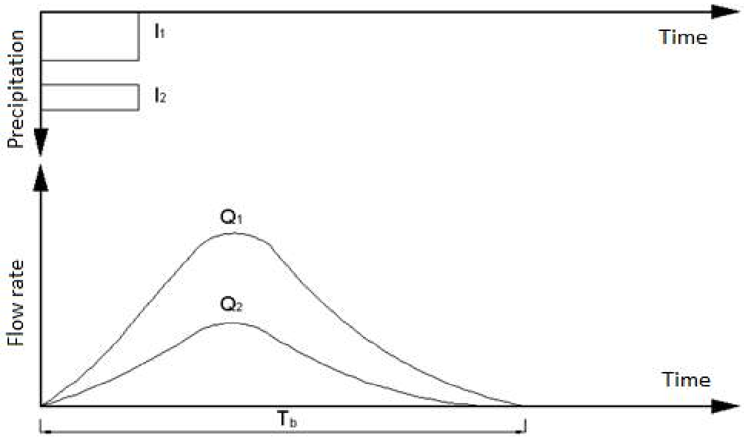

Figure 68 – Time with constant base (adapted from Costa & Lança, 2011).

The figure shows that in a basin, the length of the superficial flow is the same for uniformly distributed rainfall and of equal duration, whatever the draining volume is (Costa & Lança, 2011).

2º Principle - Proportional flow rates or affinity principle:

A unitary height h_1, creates a direct flow V_1, another unitary height h_2 creates a flow V_2. By the hydrographs, it is deduced that exists an affinity between V_1 and V_2 in relation to time and any other ordinates as for example, points A_1 and A_2 referents to time T (Figure 69) (Costa & Lança, 2011).

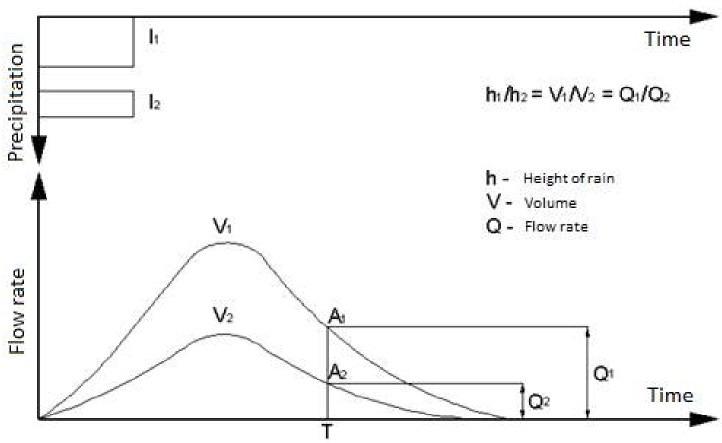

Figure 69 – Proportionality of flow rates or rates of flow (adapted from Costa & Lança, 2011).

Thus, knowing the unit hydrograph for a given duration D , the unitary rain can determine the hydrograph for another rain of different intensity, but with the same duration (Costa & Lança, 2011).

3º Principle – Principle of additivity or interdependence of simultaneous flow rates:

The direct flow time of a given rain does not depend on the direct flow caused by a previous rain (Figure 70) (Costa & Lança, 2011).

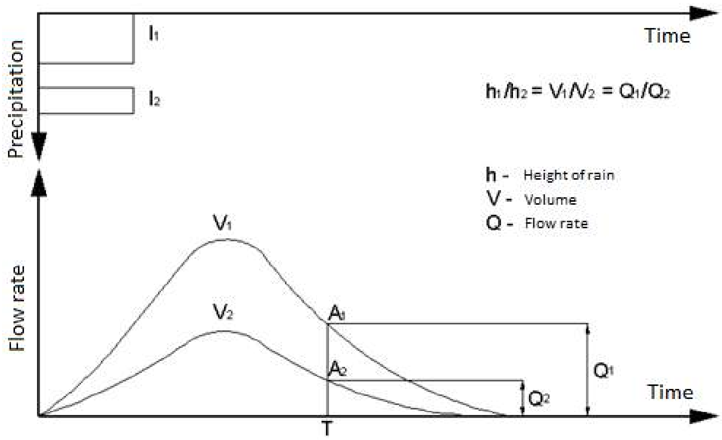

Figure 70 – Principle of additivity (adapted from Costa & Lança, 2011).

The total hydrograph is obtained by summing the ordinated of the partial hydrographs that correspond to each of the rains (Costa & Lança, 2011).

6.9.6 Unitary rain and Unitary hydrograph

If we consider D as a usefull rain (which is supposed to be uniform in time and space) falling over a basin whose time of concentration is t_c which the base time is t_b:

$$t_b = D + t_c + t_e$$

According to principles 1 and 2, the hydrographs that come from a uniform rainfall, with the same duration, will have the same basic time and the flow rates will be proportional to the rainfall intensities and corresponding to the respective flows (Costa & Lança, 2011).

Experience shows that if the duration t_p of the rain is sufficiently inferior to t_c , it is possible to apply these principles to cases of non-uniform, but "similar" rains that are with the same distribution in time and space (Costa & Lança, 2011).

It is common to use in practice $D = t_c/5$. Rains with time D are called unitary rains (Costa & Lança, 2011).

The flow in a unit hydrograph corresponds to the volume generated by a 10 mm thick water slide evenly distributed over the entire basin to a unit rain lasting D (Costa & Lança, 2011).

In the hydrograph of superficial flow, the area under the curve represents the total volume drained (Costa & Lança, 2011).

$$V_e = \int Q \cdot \partial t$$

As the rain is considered uniformly distributed over the area A of the hydrographic Basin, the height of the water slide will be:

$$h = \left(\frac{V_e}{A}\right) = \frac{1}{A} \cdot \int_0^t Q \cdot \partial t$$

In practice ∂t is attributed to the value in which the flow variation can be linear, so:

$$h = \frac{1}{A} \cdot \sum_0^t Q \cdot \Delta t$$

ergo:

$$h = \frac{Hydrograph\ area}{Basin\ area}$$

where:
Δt - time elapsed between two flow observations which must be constant in the Hydrograph;
Q - flow rate measured in period Δt;
h - average height of water blade or laminate.

If we divide all the ordered *Q* from the observed Hydrograph, by the average height *h* we find the HU (Costa & Lança, 2011).

$$\sum \left(\frac{Q}{h} \cdot \Delta t\right) \cdot \frac{1}{A} = 1$$

6.9.7 Unitary triangular hydrograph - HUT

The method was conceived by the SCS (Soil Conservation Service) in the USA in 1957 and can be applied in basins with areas up to 500 km^2. For this method, flows are obtained for a known rainfall or determined by statistical processes (udometric curves). Its usefulness is enormous in the design of hydraulic structures in regions of scarce or no hydrological information (Costa & Lança, 2011).

The used parameter for the to build a HUT are next:

$$q_p = \frac{(2.08 \cdot A)}{t_p}$$

where:
q_p - specific flow rate in m^{3}s^{-1}cm^{-1};
A - area of the hydrographic basin in km^2;
t_p - time of ascension or increase in hours.

$$t_c = 0.39 \cdot \left(\frac{L^2}{S}\right)^{0.385}$$

where:
t_c - time of concentration in hours;
L - *length of the river* in km;
S - *constant equivalent slope or inclination in percentage*.

$$D = \frac{t_c}{5}$$

where D is the duration of the unitary rain in hours.

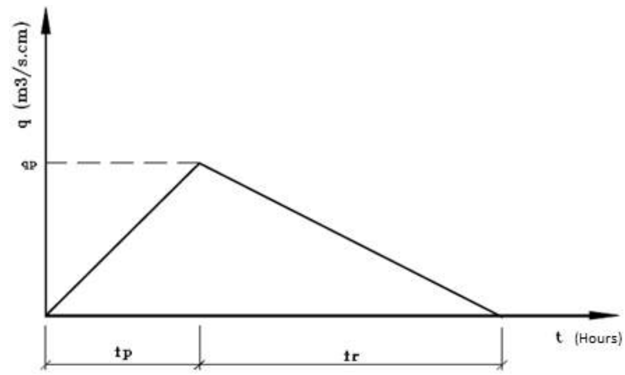

Figure 71 – Triangular unitary hydrograph (Costa & Lança, 2011).

$$t_p = \frac{D}{2} + 0.6 \cdot t_c$$

where t_p is the time os ascension in hours.

$$t_r = 1.67 \cdot t_p$$

where t_r is the decreasing time in hours,

Known $q(t_p)$, t_p and t_r, the rest ordinates are calculated $q(t_i)$, by stablishing simple proportions between triangles. For t_i, exact or aproximates values are stablish in the unitary time $t_i = n \cdot \Delta t$ e $\Delta t = D$ (Costa & Lança, 2011).

The HUT, in the part referring to the useful rainfall portion (effective rainfall), is based on a parameter that considers the type of soil, its use and the capacity of surface runoff.

This parameter is called CN (number curve or flow number) and is understood between the values from "0" to "100".

The value "0" relates to a basin that does not generate any flow (infinite hydraulic conductivity basin). The value "100" relates to a waterproofed basin whose precipitation is drained in full (Costa & Lança, 2011).

The CN flow numbers are tabulated for various rainfall numbers and values, obtained through the analysis of many basins with soils of different types, uses and conditions of moisture background (Costa & Lança, 2011). Soil is classified into 4 hydrological groups:

- **Type A** - Low flood potential. Very permeable land with little silt and clay. The lowest values of the CN are within this type;
- **Type B** - Infiltration capacity above average after complete wetting. Sandy soils less deep than those of type A;
- **Type C** - Below average infiltration capacity after pre-saturation. Contains appreciable percentage of clay;
- **Type D** - Highest potential for flood. Very clayey, almost impermeable. The highest CN values are within this range.

It is possible to relate the hydrological group of the soil with its granulometry. For this, the following triangular abacus of textural classification is used, and its modified version for determination of the hydrological group (Figure 72 and Figure 73) (Costa & Lança, 2011).

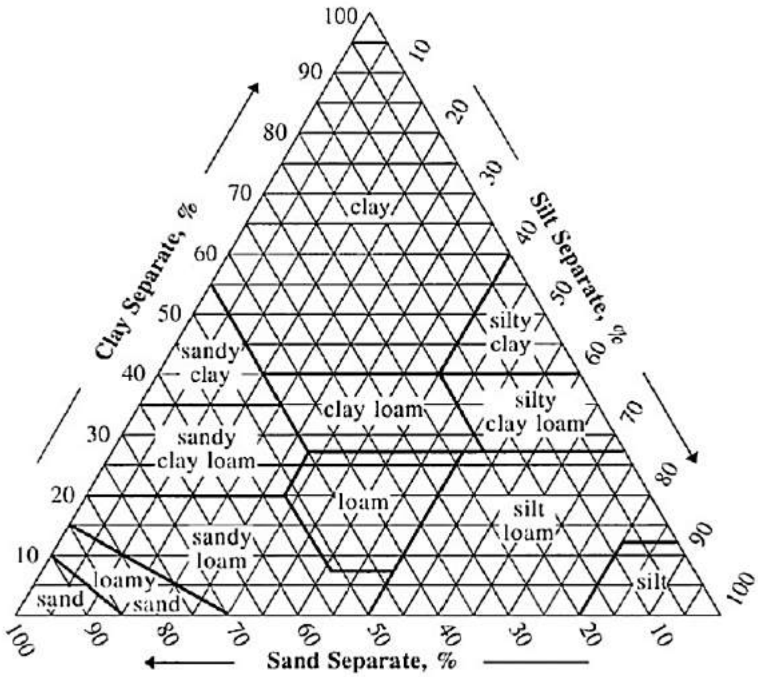

Figure 72 – Triangular abacus of textural classification (www.nrcs.usda.gov).

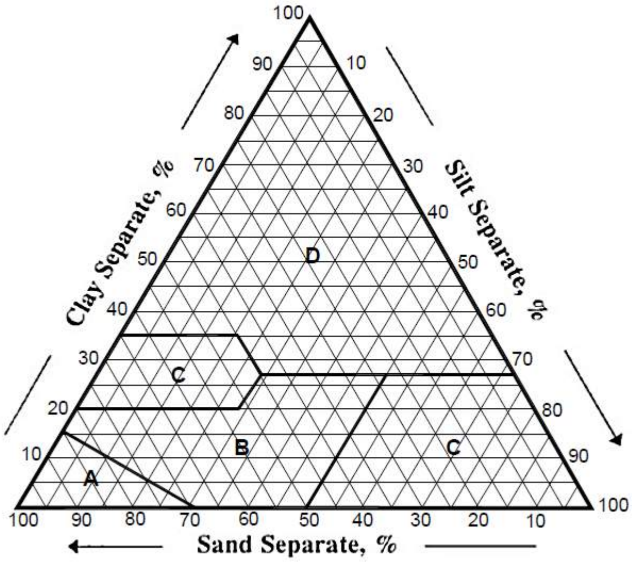

Figure 73 – Triangular abacus of textural classification per hydrologic group (adapted from Costa & Lança, 2011).

The values of effective precipitation P_e are obtained through the next formula:

$$P_e = \frac{(P - 5080/CN + 50.8)^2}{P + 20320/CN - 203.2}$$

where:
P_e - effective precipitation in mm;
P - precipitation in mm;
CN - curve number obtained in tables after a deep classification *in loco* (Table 13 and Table 14).

Table 13 – Values of the number of flow (CN) for rural regions (Costa & Lança, 2011).

Use or top layer of a soil	**Superficial Conditions**	**Type of soil**			
		A	**B**	**C**	**D**
Tilled soil		77	86	91	94
Arable crop	According to major slope	64	76	84	88
	According to contour lines	62	74	82	85
	According to contour lines in terraces	60	71	79	82
Crop rotation	According to major slope	62	75	83	87
	According to contour lines	60	72	81	84
	According to contour lines in terraces	57	70	78	82
Pasture	Poor	68	79	86	89
	Normal	49	69	79	84
	Good	39	61	74	80
	Poor - According to major slope	47	67	81	88
	Poor - According to contour lines	25	59	75	83
	Good - According to contour lines	6	35	70	79
Permanent meadow	Normal	30	58	71	78
Rural social areas	Normal	59	74	82	86
Roads	Permeable pavement	72	82	87	89
	Waterproofed pavement	74	84	90	92
Forest	Widely open or of low transpiration	56	75	66	91
	Open or of low transpiration	46	68	78	84
	Normal	36	60	70	76
	Dense or of high transpiration	26	52	62	69
	Highly dense or of high transpiration	15	44	54	61
Waterproof surface		100	100	100	100

Table 14 – Curve Number (CN) for urban ad sub-urban regions (Costa & Lança, 2011).

<table>
<tr><th rowspan="2">Use or top layer of a soil</th><th rowspan="2" colspan="2">Conditions at surface</th><th colspan="4">Type of soil</th></tr>
<tr><th>A</th><th>B</th><th>C</th><th>D</th></tr>
<tr><td rowspan="2">Cultivated zones</td><td colspan="2">Non-measures of soil conservation</td><td>72</td><td>81</td><td>88</td><td>91</td></tr>
<tr><td colspan="2">With measures of soil conservation</td><td>62</td><td>71</td><td>78</td><td>81</td></tr>
<tr><td rowspan="2">Pasture or abandoned</td><td colspan="2">Bad conditions</td><td>68</td><td>79</td><td>86</td><td>89</td></tr>
<tr><td colspan="2">Good conditions</td><td>39</td><td>61</td><td>74</td><td>80</td></tr>
<tr><td colspan="3">Meadow in good conditions</td><td>30</td><td>58</td><td>71</td><td>78</td></tr>
<tr><td rowspan="2">Woods or forest areas</td><td colspan="2">Poor coverage</td><td>45</td><td>66</td><td>77</td><td>83</td></tr>
<tr><td colspan="2">Good coverage</td><td>25</td><td>55</td><td>70</td><td>77</td></tr>
<tr><td rowspan="2">Lawn, parks, golf areas, cemeteries etc.</td><td colspan="2">Good conditions, with grass covering more than 75% of the area</td><td>39</td><td>61</td><td>74</td><td>80</td></tr>
<tr><td colspan="2">Reasonable conditions, with grass covering more than 50% to 75% of the covered area</td><td>49</td><td>69</td><td>79</td><td>84</td></tr>
<tr><td>Offices and commercial areas</td><td colspan="2">Approximately 85% of the permeable area</td><td>89</td><td>92</td><td>94</td><td>95</td></tr>
<tr><td>Industrial zone</td><td colspan="2">Approximately 72% of the waterproof area</td><td>81</td><td>88</td><td>91</td><td>93</td></tr>
<tr><td rowspan="6">Residential areas</td><td>Average areas</td><td>Waterproof average area</td><td colspan="4"></td></tr>
<tr><td><500 m^2</td><td>65%</td><td>77</td><td>85</td><td>90</td><td>92</td></tr>
<tr><td>1000 m^2</td><td>38%</td><td>61</td><td>75</td><td>83</td><td>87</td></tr>
<tr><td>1300 m^2</td><td>30%</td><td>57</td><td>72</td><td>81</td><td>86</td></tr>
<tr><td>2000 m^2</td><td>25%</td><td>54</td><td>70</td><td>80</td><td>85</td></tr>
<tr><td>4000 m^2</td><td>20%</td><td>51</td><td>68</td><td>79</td><td>84</td></tr>
<tr><td colspan="3">Parking lots, roofs, viaducts, etc.</td><td>98</td><td>98</td><td>98</td><td>98</td></tr>
<tr><td rowspan="3">Roads and deposits</td><td colspan="2">Asphalted with rainwater drainages</td><td>98</td><td>98</td><td>98</td><td>98</td></tr>
<tr><td colspan="2">Grave</td><td>76</td><td>85</td><td>89</td><td>91</td></tr>
<tr><td colspan="2">Soil</td><td>72</td><td>82</td><td>87</td><td>89</td></tr>
</table>

The CN values obtained in the preceding tables should be corrected considering the previous conditions of soil water content (Costa & Lança, 2011). This correction considers three background conditions of humidity:

- **AMC I** - dry soils below the withering. They should not be considered in full-flow studies;
- **AMC II** - humidity corresponds to field capacity. Moist soil gives rise to medium flows;
- **AMC III** - very drenched soil, almost saturated (conditions of dust), originated by persistent rains for at least five days before. Situation favourable to the formation of the largest floods.

The SCS recommends that the CN values should be corrected according to the conditions prior to soil moisture. Thus, it was elaborated in a framework to obtain the antecedent conditions of humidity, depending on the total precipitation in the previous five days (Costa & Lança, 2011).

Table 15 – Antecedent moisture conditions for total precipitation over the previous five days (Costa & Lança, 2011).

<table>
<tr><th colspan="2">Total precipitation over the previous five days (mm)</th><th rowspan="2">Antecedent moisture conditions</th></tr>
<tr><th>Inactive period</th><th>Growing or active period</th></tr>
<tr><td>< 13</td><td>< 36</td><td>AMC I</td></tr>
<tr><td>13 a 28</td><td>36 a 53</td><td>AMC II</td></tr>
<tr><td>> 28</td><td>> 53</td><td>AMC III</td></tr>
</table>

The SCS recommends correcting the CN for AMC I and AMC III in function of values of CN for AMC II (Costa & Lança, 2011).

Table 16 – Correction of the values of (CN) for AMC I and AMC III in function of the values of (CN) for AMC II (Costa & Lança, 2011).

CN for AMC II	Corrected Value of CN	
	AMC I	AMC II
100	100.00	100.00
95	88.86	97.76
90	79.08	95.39
85	70.41	92.87
80	62.68	90.19
75	55.75	87.34
70	49.49	84.29
65	43.82	81.03
60	38.65	77.53
55	33.92	73.76
50	29.58	69.69
45	25.57	65.30
40	21.87	60.53
35	18.44	55.32
30	15.25	49.64
25	12.28	43.39
20	9.50	36.51
15	6.90	28.87
10	4.46	20.35
5	2.16	10.80

Some definitions to remember:

- **Withering point -** soil water content below which the plants no longer recover turgidity;
- **Field capacity -** water content existing in the soil and that resists the effects of gravity (drainage).

CHAPTER 7 - HYDROLOGICAL BALANCE

7.1 Definition

The hydrological balance translates into the continuity equation, that is, the difference between the inputs (affluents) and the outputs (effluents) of water in a given space and for a certain period of time, is equivalent to the variation of the volume reported in that interval of time (Rodrigues et al., 2011).

$$Inputs - Outputs = Variation\ of\ storage$$

Or:

$$\int_t^{t+\Delta t} q_a(t)\, dt - \int_t^{t+\Delta t} q_e(t)\, dt = S(t+\Delta t) - S(t)$$

where $q_a(t)$, $q_e(t)$ and $S(t)$ represent, respectively, the laws of variation with the time of inputs, outputs and water storage within the considered space (Rodrigues et al., 2011).

On an annual scale (hydrological year) and taking into account the ratio between precipitation and flow; the balances equation in a hydrographic basin is summarized as the quantification of three variables: precipitation, P, real evapotranspiration, Et_r and flow, R (Rodrigues et al., 2011).

By knowing two of these variables, it is always possible to determine the value of the third by the equation:

$$R = P - Et_r$$

When transits from the annual scale of analysis to the monthly scale, the variation component of underground storage becomes significant, even in medium terms. In these conditions, it is not possible to linearly relate the flow with precipitation as the state of the underground reserves of one month does not remain constant over the years. In this time scale it is also necessary to consider, the portion of water retained as soil moisture, a fraction of which will constitute the water usable by plants (Rodrigues et al., 2011).

The balance equation that relates monthly, the value of precipitation with the corresponding value of flow, is:

$$R = P - Et_r - \Delta S - \Delta H$$

where ΔS and ΔH represents, respectively, the monthly variations of underground storages and the state of soil moisture (Rodrigues et al., 2011).

The balance equation that relates monthly, the value of precipitation with the corresponding flow value, is at the base of the sequential balance models that relate the precipitation with the flow. Among these, stands the "Thornthwaite and Mather" resolution model, due to its simplicity and greater dissemination (Rodrigues et al., 2011).

7.2 Hydrologic year

The equation of the hydrological balance can be applied to intervals of time equal to 1 year (**hydrological year**), provided that the previous hypothesis is fulfilled, that is, that at the beginning of each of these time intervals the water storage in the basin is practically constant (and there is not leaks between basins) (Quintela A. C., 1996).

In Portugal, it was agreed that the hydrological year starts on October 1 and ends on September 30, because at the end of the summer period, water reserves in the soil are close to their minimum limit and are similar from year to year; And in African countries of Portuguese expression: Cape Verde - 1 July to 30 June, Guinea - 1 May to 30 April, Sao Tome and Principe - 1 September to 31 August and Angola and Mozambique - October 1 to September 30 (Quintela A. C., 1996).

According to Quintela A. C. (1996) the adoption of the model of a hydrologic year allows:

- **Write the equation of the hydrological balance so that it can correlate annual values of precipitation and flow** (if it is possible to evaluate the value of evapotranspiration);
- **Obtain series of values of annual flow statistically independent from each other** (considering that the flow rate of a river, between 1 October and 30 September each year, depends almost exclusively on the precipitation during this year and has little influence of the precipitation of the previous year).

7.3 Sequential model of balance of Thornthwaite

7.3.1 Introduction

The sequential balance model of Thornthwaite is based on the macroscopic description of the terrestrial phase of the hydrological cycle and allows to generate monthly flow values based on the precipitation and evapotranspiration values. The involving variables are, the precipitation, P, the potential evapotranspiration, Et_p, a real evapotranspiration Et_r and the storage of water in the soil (Rodrigues et al., 2011). Such variables relate between each other in such way as:

$$P - (Et_r + \Delta S) = R + \Delta S_S + R_b + \Delta S_{SSO}$$

where:
P - precipitation;
Et_r - real or effective evapotranspiration;
R - superficial flow;
R_b - underground flow;

$\Delta S_S, \Delta S, \Delta S_{SSO}$ - variation of storage at surface, in the soil and in the sub-soil.

The variables must be expressed in the same volume units (or equivalent water height - mm) and referenced at the same time interval mm (Rodrigues et al., 2011).

7.3.2 Methodology of balance

At the beginning of each monthly simulation, it is quantified the nature and magnitude of the difference between the value of the occurring precipitation and the potential value of the affected evapotranspiration (of a reduction factor, K_C), depending of the vegetal species present in the analysis. In ours conditions, it is usual to attribute the average value of 0.7 for K_C (Rodrigues et al., 2011).

$$P - \left(Et_p \times K_C\right) = Dif, with\, K_C = 0.7$$

According Rodrigues et al. (2011), and according to the nature of the Dif (positive or negative value) is necessary to verify two different situations, based in the magnitudes of Dif:

- **The total filling of the soil reserves, $\boldsymbol{S}$ -** water surplus situation, *SH*, whenever the precipitation equals or exceeds the actual evapotranspiration the *SH* value will be:

$$SH = P - \left(Et_p + \Delta S\right);\ \Delta S \geq 0$$

- **The depletion of soil surface reserves** - water deficit situation, *DH*, when the precipitation value is lower than the potential evapotranspiration value. The value of *DH* is:

$$DH = Et_p - Et_r = \left(Et_p + \Delta S\right) - P;\ \Delta S < 0$$

Given:

$$Et_r = P - \Delta S; \Delta S < 0$$

If the superficial reserves are already fulfilled - fact that, in our climate, usually occurs in winter - the surplus (SH) will be separated by losses in depth, contributing to the underground reserves, and by the contribution to the superficial flow. The underground reserves will be exhausted with a delay of one month, constituting the base flow, according to a geometric progression of 1/2 ratio, usually called the **aquifer discharge coefficient ($\boldsymbol{\alpha}$)** (Rodrigues et al., 2011).

The α coefficient is a characteristic of the hydrogeological formations present in the area, which is therefore capable of calibration according to the characteristics of each site (Rodrigues et al., 2011).

When the soil reserves are exhausted - fact that in our climate occurs roughly in the summer months - the flow is fed solely by the depletion of the underground reserves, and the originated water deficit (DH) will constitute an amount of supplementary water that could have been used by plants and soil if it is artificially supplied by irrigation (Rodrigues et al., 2011).

In the intermediate situations between saturation and depletion of soil moisture reserves, the flow is feed only by underground reserves, and the differences between the value of precipitation and evapotranspiration will increase or decrease the value of the superficial reserves depending on the sign of these differences is positive or negative - the fact is in general verified in the autumn and spring for our climate (Rodrigues et al., 2011).

The exposed methodology admits, as a simplification hypothesis, that the variation of water storage in the soil varies linearly with the difference in precipitation over evapotranspiration (positive in the humid and negative period in the dry period) within the limits of usable capacity. Lencastre,1984, however, it considers to be more realistic that during the dry period, and due to the increase of water retention forces in the soil as a result of its drying, the decrease in storage is done according to the following exponential equation:

$$S = n_u \cdot e^{L/n_u}$$

where S is the storage of water that relays in the soil of usable capacity n_u, when it is under a potential loss of water, L (Rodrigues et al., 2011).

The value of L, in each interval of a dry period, is given by:

$$L(i) = \sum_{j=1}^{i} [P(j) - Et(j)]\, ; L < 0$$

where i is the order number of the interval in cause, since the beginning of the dry period, and j of any interval of the same period (Rodrigues et al., 2011).

To simulate the flows through the Thornthwaite model are required as input data:

- Area of the hydrographic basin at upstream of the section under study (km^2);
- Monthly values of weighted precipitation (mm);
- Monthly values of weighted evapotranspiration (mm);
- Soil moisture storage limit (mm) - this value is calibrated for each case depending on the soil capacity to store water. Thornthwaite, in the base design of the model, admitted a generic soil with storage capacity equal to 100 mm.

The result of the balance is presented in mm, including a monthly value of each used variable, respectively:

- Potential of evapotranspiration (Et_p);
- Precipitation (P);
- Superficial reserves of water in the soil (S);

- Real evapotranspiration (Et_r);
- Hydric deficit (DH);
- Hydric surplus (SH) - for Thornthwaite, “water surplus”;
- The flow (R).

A possible organization in terms of a spreadsheet, it is the one proposed in table 17, considering that the values of each column are determined in accordance with the assumptions of the model of Thornthwaite (Rodrigues et al., 2011).

Table 17 – Hydrologic balance of Thornthwaite Mather (adapted from Rodrigues et al., 2011).

Year	Month	Et_p (mm)	P (mm)	Dif (mm)	S (mm)	ΔS (mm)	Et_r (mm)	DH (mm)	SH (mm)	Q (mm)	Q (m^3/s)
1	Oct … Sep										
2	Oct … Sep										
… n	… …										

CHAPTER 8 - STUDY OF FLOODS

8.1 General considerations

The flood concept is not perfectly standardized. In some cases, an exceptional occurrence is associated to such term, flooding the adjoining ground of a given waterline. From a hydrological point of view, the term is associated with the occurrence of direct surface flow which may, under certain circumstances, correspond to the first definition (Rodrigues et al., 2011).

The purpose of this chapter is to determine the flood hydrograph and its flood tip, data necessary for the design of the hydraulic works bodies for flowing liquids, the flood bed of a water line or the evaluation of flood effects. Floods are natural phenomena, due to the random nature of the components of the hydrological cycle, it is important to minimize all possible harmful effects and to take advantage of the available energy (Rodrigues et al., 2011).

Rodrigues et al. (2011), points out that in flood studies it is basically intended to meet one or both of the following objectives:

- **pre-determination of floods** - determination of the flood tip and / or flood hydrographs that will occur for preestablished conditions (for a given return period depending on the lifetime of the work), applied in the design of dam dischargers, dams flood protection, etc.;
- **flood forecasting (real-time)** - determination of flow rates that may occur in the near future, at the earliest possible time, for real-time operational purposes.

8.2 Factors influencing floods

According to Rodrigues et al. (2011), the factors that contribute to the alteration of flood conditions are: **physiographic** - area, shape, relief, vegetation cover, geological nature and soils of the hydrographic basin, drainage density and relief of the hydrographic network; **climatic conditions -** temperature, air humidity and soil moisture, **and temporal and spatial distribution of precipitation**.

8.3 Pre-determination of flood tips

8.3.1 Generalities

In the design of bodies for the discharge of hydraulic works, works of art in roads or collectors of rainwater, it is necessary to determine the flow rates of the flood tip. Sometimes it is still important to know the flood hydrograph (e.g. evaluation of the variation of level in a reservoir). These values are necessarily associated with a given return period (Rodrigues et al., 2011).

There is a great diversity of methods of pre-determination of floods, presented by different authors, from simple expressions, deduced empirically, to complex models of hydrograph definition (Rodrigues et al., 2011).

8.3.2 Empiric formulas

The simplest empirical expressions for the determination of the flood tip or peak flow rate, consider this flow as a function (only) of the area of the river basin. The return period associated with these flows rates is not quantified, but is considered as low (Rodrigues et al., 2011).

These formulas should be applied in the case of river basins with similar characteristics to the basins for which they were deduced and in the absence of better information (Rodrigues et al., 2011).

Formula of Iskowski:

This formula was deduced from the values of flows measured in 289 rivers of Central Europe with very varied river basins. The peak flow rate (m^3/s) is determined according to the area (km^2), the average annual rainfall (m), the category of soils, vegetation cover and relief of the catchment area or basin (Rodrigues et al., 2011).

$$Q_P = K \cdot m \cdot P \cdot A$$

where:
K - depends in the category of the soil, vegetation cover and relief; varies between 0,017 and 0,8 (Table 18);
m - varies, based on the area of the hydrographic basin, between 10 to 1 (Table 19);
A - area of the basin in m^2.

Table 18 – Coefficient K (adapted from Lencastre & Franco, 1992).

Basin Orography	Values of K			
	Category I	Category II	Category III	Category IV
Low and marshy zone	0.017	0.030	-	-
Slightly undulating zone	0.025	0.040	-	-
Zone in part flat and partly with hills	0.030	0.055	0.100	-
Zone with hills not too steep	0.035	0.070	0.125	-
High/low slopes zone	0.060	0.160	0.360	0.600
High hills/medium slope zone	0.070	0.185	0.460	0.700
High/very steep hills zone	0.080	0.210	0.600	0.800

Note:
Category I - Very permeable land with great vegetation or completely cultivated;
Category II - Hill or mountain terrain with normal vegetation, and those of slightly wavy, but not very permeable plain;
Category III - Waterproof terrain with normal vegetation on a steep or mountainous hill;
Category IV - Waterproof terrain with little or no vegetation.

Table 19 – Coefficient m (adapted from Lencastre & Franco, 1992).

A (km^2)	1	10	40	70	100	200	300	400	500	600	700	800	900	1000
m	10.0	9.0	8.23	7.60	7.40	6.87	6.55	6.22	5.90	5.60	5.35	5.12	4.90	4.70

Rational Formula

This is an equation that it has been widely used in our country in small basins. The peak flow rate is determined according to the catchment area, the average precipitation intensity, for a given return period and duration of the rainfall equal to the basin time of concentration, and a flow coefficient depending on the nature of the soils and of the vegetation cover, however, such coefficient is considered constant for any intensity of rainfall and for any conditions previous to a situation, in a hydrographic study (Rodrigues et al., 2011).

$$Q_P = C \cdot I \cdot A$$

where:
C - dimensionless flow coefficient obtained from hydrology tables;
I - average intensity referring to the maximum rainfall interval, for a given return time with duration equal to the basin time of concentration. Generally, in mm/h it turns into m/s;
A - basin area in m^2.

Comparison of the "Rational Formula" with the "Iskowski Formula" allows the introduction of the critical rainfall concept for a given return period that gives rise to the peak flow rate of a flood. The characterization of the relief is indirectly considered in the precipitation intensity, which varies with the zone of location of the basin, and in the coefficient of flow. This formula relates the peak flow rate of a flood with the precipitation that gives rise to it, admitting that the return period of such flow is equal to the return period of such precipitation. Naturally, this would only be true if the basin conditions before the referred rainfall were the same at each occurrence, implying the same direct flow behaviour in the basin reference section (Rodrigues et al., 2011).

A different interpretation of the "Rational Formula" allows identifying the determination of the volume of precipitated water per unit of time. This volume does not contribute at all to the flow rate in the reference section, being applied the coefficient of flow, relation between the direct flow and the useful precipitation that gave rise to it (Rodrigues et al., 2011).

The flow coefficient can only be well applied, if it has been determined experimentally in basins with similar behaviour, from the point of view of a flow. This equation has been widely used in Portugal with satisfactory results for basins with less than 25 km^2. It is applied in determining the design flow rate of rainwater collectors. For a given region, the flow coefficient, the intensity-duration-frequency (IDF) curves and the basin time of concentration are chosen, the peak flow rate is directly proportional to the area (Rodrigues et al., 2011).

With the "Rational Formula", the end character of the flood peak flow rate is included by the precipitation intensity corresponding to the maximum precipitation value for a given return period and a given duration. The duration of the rainfall is, however, considered equal to the time of concentration, erg. the total duration of the rainfall is equal to the duration of the useful rainfall which in turn is equal to the time of concentration (Rodrigues et al., 2011).

Martino's Formula

Surface flows in urbanized areas is subject to changes in topography, caused by anthropogenic interventions (Costa & Lança, 2011).

One of the first and still most widely used methods for dimensioning rainwater collectors is the rational method, whose accuracy depends on the value established for factor *C* (flow coefficient), obtained in tables. The value arbitrated depends on the greater or lesser experience of the designer (Costa & Lança, 2011).

The rational method is restricted when it comes to urbanized areas of flat or slightly undulating relief (inclination of slopes less than 5%), conducive to intersections and storages within the basin (Costa & Lança, 2011).

The "Martino Formula" is based on the rational method, but considers the storage in the basin (Costa & Lança, 2011):

$$Q_P = \Psi \cdot C \cdot I \cdot A$$

where:
Ψ - coefficient of delay or storage, dimensionless, less than 1, obtained in tables;
C - coefficient of flow, dimensionless, less than 1, obtained in hydrologic tables;
I rain intensity with time equal to the time of concentration. It is obtained by the rain equation, of type $I = a \cdot t^b$, with I in mm/h, transformed in m/s;
A - basin area in m^2.

Mockus's Formula

This method allows to calculate the maximum flow rate of a hydrograph, assuming the same principles of the HUT of the SCS (Costa & Lança, 2011).

Its application follows the following formulation for an effective rainfall time, so, a critical time is established (Costa & Lança, 2011):

$$T_{cr} = 2 \cdot \sqrt{T_c}$$

where T_c is the time of concentration in hours.

The duration of the rain is:

$$t = T_{cr} + \frac{I_\alpha}{I(T_{cr})}$$

where:
I_α - initial losses, obtained by $I_\alpha = (5080/CN) - 50.8$;
$I(T_{cr})$ - the intensity of the rain corresponding to the critical time T_{cr} is obtained by the rainfall equation of type $I = a \cdot T_{cr}^{-b}$, (I in mm/h; T_{cr} in minutes).

By having time t, it is calculated the respective height P using the rainfall equation $P = a \cdot t^c$ (P in mm).

With the values P, it is calculated the effective rain P_e through the formula of SCS:

$$P_e = \frac{\left(P - \frac{5080}{CN} + 50.8\right)^2}{P + \frac{20320}{CN} - 203.2} \cdot 0.1\ (P\ in\ mm;\ P_e\ in\ cm)$$

Finally, the maximum flow rate or flow rate is calculated by the formula:

$$Q_p = \frac{2.08 \cdot A \cdot P_e}{T_c^{0.5} + 0.6 \cdot T_c}$$

where:
P_e - effective rain in cm;
T_c - time of concentration in hours;
A - basin area in km^2.

Giandotti's Formula

The structure is similar to the rational formula, but the coefficient of flow is obtained from the area of the basin under study (Costa & Lança, 2011).

$$Q_P = \frac{\lambda \cdot A \cdot h}{T_c}$$

where:
h - maximum precipitation in mm corresponding to the time of concentration and a given time of return;
A - basin area in km^2.

Time of concentration according Giandotti:

$$T_c = \frac{4 \cdot \sqrt{A} + 1.5 \cdot L}{0.8 \cdot \sqrt{\bar{H}}}$$

where:
T_c - time of concentration in hours;
A - basin area in km^2;
L - length of the main river in km;
$\bar{H}$ - average height of the basin in m.

The parameter λ is found detailed in Table 20.

Table 20 – Parameter λ (adapted from Costa & Lança, 2011).

A (area of the basin in km^2)	Value of λ
≤ 300	0.346
300-500	0.277
500-1000	0.197
1000-8000	0.100
8000-20000	0.076
20000-70000	0.055

Giandotti's formula was advocated in the Regulation of Small Earth Dams, published in 1973 (Costa & Lança, 2011).

Témez Formula

The structure is similar to the previously presented, except for the time of concentration, according to Témez is:

$$T_c = 0.3 \cdot \left(\frac{L}{i^{0.25}}\right)^{0.76}$$

where:
T_c - time of concentration in hours;
L - length of the main river in km;
i - slope %.

Kirpich's Formula

The structure is similar to the previously presented except for the time of concentration, so, according to Kirpich is:

$$T_c = 0.39 \cdot \left(\frac{L^2}{S}\right)^{0.385}$$

where:
T_c - time of concentration in hours;
L - Length of the main river in km;
S - equivalent slope constant of the river in %. The average slope of the river may also be used without loss of accuracy.

Formula of Loureiro

In Portugal F. Loureiro developed studies for the North and South of the country, where zones were delimited and flood rates of flood tips were correlated, measured and analysed by the distribution of Gumbel with the area of the basin, through the expression:

$$Q_P = C \cdot A^Z$$

where:
C - regional parameter with return period;
Z - regional parameter;
A - basin of the area in km^2.

8.4 Usual methods for design

The methods of calculating maximum full flow rates went through several phases. Initially the calculation was based on experience and practical rules were deduced. Then, stablished theories based on measurements and finally the rational formula (Costa & Lança, 2011).

The indiscriminate use of the rational formula gave rise to great errors, almost all evidencing over-dimensioning (Costa & Lança, 2011).

Ven Te Chow classified the current methods for the dimensioning of flow rates sections as follows:

Judgment Method

Design or dimensioning depends on the judgment experience and the general information obtained through the people living in the places (Costa & Lança, 2011).

Classification and Diagnostic Method

A classification of the basins is made taking into account the local conditions, topographic, type of soil and its use, slopes, intensity of rains, etc. (Costa & Lança, 2011).

In some areas this classification is made through tables elaborated for the specific conditions of the regions. The type and size of the flow rate sections depend on the criteria and experience of the engineer (Costa & Lança, 2011).

Empirical Rules Method

A practical rule is established to replace the judgment. It was widely used at the beginning of this century (Costa & Lança, 2011).

Formula Method

A formula is deduced to find the maximum flow rate (Costa & Lança, 2011).

Ven Te Chow related the best-known formulas, in number of 120, from very simple to more complex (Costa & Lança, 2011).

It is a method that was very much in vogue and it should be noted that almost all countries presented "their" formula, the method can still be used for evaluation or comparison with other methods (Costa & Lança, 2011).

The imprecision of this method lies in the difficulty in establishing adequate coefficients for the basins under study (Costa & Lança, 2011).

Tables and Abacuses Method

For the application of the empirical formulas tables and abacus are elaborated that facilitate the calculations, although today, with computer science, it is no longer as difficult as before to use complicated formulas (Costa & Lança, 2011).

It should be noted, however, that the use of tables and abacuses has the advantage of being able to quickly assess the phenomenon in its global aspect (Costa & Lança, 2011).

Rational Method

Very widespread, it is based on the rational formula already described (Costa & Lança, 2011).

Direct Observations Method

This method requires detailed studies of the hydrographic basin and the river channel, as well as regular and accurate meteorological observations, which will give rise to hydrological and hydraulic studies (Costa & Lança, 2011).

Correlation Analysis Method

The statistical analysis of field hydrological measurements is performed. Formulas or abacuses may be obtained for practical applications. Many regular observations are required (Costa & Lança, 2011).

In large basins it is the most advisable method, usually in large basins, special places (gorges or canyons, important points, river mouth, etc.) which have stations to measure flow rate , sediment and meteorological stations (Costa & Lança, 2011).

Unit Hydrograph Method

The unitary hydrograph theory is used. It is a method that is applied in the study of small river basins of which there is no flow rate data. In this case, through careful measurements of the flow rate and rainfall, a hydrograph overlay is obtained from which the HU originates and can then be used for any rainfall (Costa & Lança, 2011).

8.5 Statistical Methods

The hydrological studies are approached according to two schools: physics and statistics (Costa & Lança, 2011).

Deterministic models explain the behaviour of hydrological phenomena according to the laws of physics. These models were discussed in the previous chapter, with special relevance to the unit hydrograph (Costa & Lança, 2011).

Stochastic models explain the behaviour of hydraulic phenomena through statistical methods - the random component overlaps with the physical component (Costa & Lança, 2011).

The construction of an aqueduct or a section of road, due to insufficient drainage, does not lead to loss of human life and its dimensioning is governed by considerations different from those that govern, for example the dam unloader. When there is no possible losses of life, there is a risk to be taken, otherwise the works will become very expensive (Costa & Lança, 2011).

There is a correspondence between the magnitude of the flood and its frequency, and such correspondence can be used to execute a more economical construction work (Costa & Lança, 2011).

If exists series of instantaneous flows rates values of a flood tip in the reference section of the river basin, a statistical study of this series will be made so as to be able to infer or predict about the peak flow rate of flood tip for different periods of return (Costa & Lança, 2011).

In case of holding data of the reference section of a basin with similar characteristics to the hydrographic basin under study, it is possible, through a correlation with physical characteristics, to determine the flow rate or flow rate of flood tips in the basin under study (Costa & Lança, 2011).

The statistical analysis of peak flow rates values recorded in previous floods in the reference section, provided that enough numbers allow to check the adjustment to a given distribution. The statistical law most applied in Portugal to the study of floods has been the distribution of Gumbel, 2 parameters (Costa & Lança, 2011).

Although in A. G. Henriques,1983, the author has presented a comparative study of the adjustment of different distributions from extremes to series of flood flow rates, measured in different regions of the country and concluded that any distribution of two parameters (Gumbel, Log-normal and Gamma) and the distribution with three parameters of Log-Pearson are not adequate (Costa & Lança, 2011).

Of the other distribution laws with three parameters analysed, the Pearson distribution is the favourite to the generalized asymptotic distribution of "extremes", which in turn is preferable to the Log-Normal distribution, when it is a series of annual maximum instantaneous flow rates (Costa & Lança, 2011).

If the series of available instantaneous maximum flow rates values is small and there is a series of maximum precipitation values in the river basin, with a duration equal to or greater than the time of concentration, a larger dimension may be attempted to extend the series of peaks flow rates from the maximum rainfall series, through a regression model (Costa & Lança, 2011).

In the absence of data of the section under study, the correlation of the values determined in different reference sections relative to other river basins with physical characteristics of the basins (for different periods of return) can be attempted. The **regionalization of the values** is done in this way (Costa & Lança, 2011).

The already presented Loureiro formula was based on the regionalization of values (Costa & Lança, 2011).

The recurrence period T, also called recurrence time or return period is the average interval of years in which a given phenomenon occurs, with the same magnitude or greater (Costa & Lança, 2011).

If P is the probability of occurrence or being overpassed, then:

$$T = \frac{1}{P}$$

As the theoretical probability is unknown, then is made an estimate from the observed frequency (Costa & Lança, 2011).

If n is the number of observed years of a given event (for example, a maximum flow rate), then, there is a series of annual values (Costa & Lança, 2011). By ordering these values in a descending order, the frequency with which a given value of m order is matched or exceeded in n years is:

$$F = \frac{m}{n+1}\ (criteria\ of\ Kimbal)$$

When n is very large, the value of F approaches P . For periods of recurrence smaller than the number of years of observations, the value F can give a good approximation of the real value of P, but for large periods of recurrence, the distribution of the frequencies has to be fitted to a theoretical probabilistic law (Costa & Lança, 2011).

Ven Te Chow has shown that most of the frequency functions in hydrological analysis can be written in the form:

$$X = \bar{X} + K \cdot S_x^{0.5}$$

where:
$\bar{X}$ - average;
K - factor of frequency;
S_x - standard deviation.

An application of this theory refers to the calculation of maximum intensity rains. When it is intends to find the values of extreme intensities, the maximum annual series is chosen, that is, for a given duration, the maximum observed rainfall intensity in each hydrological year is chosen (Costa & Lança, 2011).

Gumbel's formula shows that the probability P of na extreme value of the series being lower than, is:

$$P = (e^{-e})^{-y}$$

being y a reduced variable to:

$$Y = \left(X - X_f\right) \cdot \frac{S_n}{S_x}$$

where X_f is the tendency of the extreme values:

$$X_f = \bar{X} - S_x \cdot \frac{\bar{Y}_n}{S_n}$$

where:
$\bar{X}$ - average of the variable X;
$\bar{Y}_n$ and S_n - average and standard deviation of the reduced variable;
S_x - standard deviation of the variable x.

The values of Y (reduced variable) can be found tabulated (Table 21) in function of the return period (Costa & Lança, 2011).

Table 21 – Reduced variable (adapted from Costa & Lança, 2011).

Reduced variable Y	Return variable (years)
0.000	1.58
0.367	2.00
0.579	2.33
1.500	5.00
2.250	10.00
2.290	20
3.395	30
3.902	50
4.600	100
5.926	200
5.808	300
6.214	500
6.907	1000

The values of $\bar{Y}_n$ and S_n are found in tables (Table 22) in function of the number of years n (Costa & Lança, 2011).

Table 22 – Average ($\bar{Y}_n$) and standard deviation (S_n) (adapted from Costa & Lança, 2011).

n (nº of years)	$\overline{Y_n}$	S_n
20	0.52	1.06
30	0.54	1.11
40	0.54	1.14
50	0.55	1.16
60	0.55	1.17
70	0.55	1.19
80	0.56	1.19
90	0.56	1.20
100	0.56	1.21
150	0.56	1.23
200	0.57	1.24
∞	0.57	1.28

8.6 Maximum possible flood or peak flow rate

The flow rate corresponding to the maximum probable flood is usually much higher than the values recorded, corresponding to a very low probability of occurrence, that is, to a very high return period (Rodrigues et al., 2011).

This value can be determined by the statistical regression between the **probable maximum precipitation** (PMP) and the corresponding flow rate or by the application of the unit hydrograph (Rodrigues et al., 2011).

PMP represents the upper limit of the precipitation value associated with a return period, which averages, it corresponds to the highest estimated value of precipitation, for a duration, physically possible for a given region at a given time of the year (Rodrigues et al., 2011).

The determination of the PMP is outside the scope of the discipline, however, it is indicated in the Table 23, the values of the highest rainfall in the world, from which it was possible to adjust the following equation:

$$P = 39 \cdot D^{0.5}$$

where P is the precipitation expressed in cm, and D is the duration in hours (Rodrigues et al., 2011).

Table 23 – Major values of globally registered precipitations (extracted from Brandão, 1995).

Duration	Height (cm)	Local	Date
1 min	3,8	Barot, Guadeloupe	26/11/70
8 min	12,6	Fussen, Bavaria	25/05/20
15 min	19,8	Plumb Point, Jamaica	12/05/16
42 min	30,5	Holt, Mo	22/06/47
2 h 10 min	48,3	Rockpot, WV	18/07/889
2 h 45 min	55,9	D'Hanis, TX	31/05/35
4 h 30 min	78,2	Smethport, PA	18/07/42
9 h	108,7	Belouve, Reunion	28/02/64
12 h	134	Belouve, Reunion	28-29/02/64
18 h 30 min	168,9	Belouve, Reunion	28-29/02/64
24 h	187	Cilaos, Reunion	15-16/03/52
2 days	250	Cilaos, Reunion	15-16/03/52
3 days	324	Cilaos, Reunion	15-16/03/52
4 days	372,1	Cherrapunji, India	12-15/09/74
5 days	385,4	Cilaos, Reunion	13-18/03/52
6 days	405,5	Cilaos, Reunion	13-19/03/52
7 days	411	Cilaos, Reunion	12-19/03/52
15 days	479,8	Cherrapunji, India	24-30/06/31
31 days	930	Cherrapunji, India	07/861
3 months	1637	Cherrapunji, India	05-07/861
6 months	2245	Cherrapunji, India	04-09/861
1 year	2646	Cherrapunji, India	08/860; 07/861
2 years	4077	Cherrapunji, India	1860-1861

8.7 Damping of floods

If the purpose of the flood analysis is to design a safety discharger of a dam or only to realize the effect of a reservoir in the flood control that promotes the damping of flood waves, it will be necessary to know not only the flow rate of a tip of the affluent flood, but also the volume of its flood wave (Portela, 2005).

Flood-dampening can be understood as the capacity of a reservoir to propagate downstream to a flow rate below the maximum flow rate flowing through it, this capacity is only possible if part of the volume affluent to the reservoir is stored there and discharged downstream in a more gradually than it does, according to a time lag (Portela, 2005).

In the case of a dam provided with a surface unloader with a free discharge, which it is not controlled by gates, the damping of flood waves leads to affluent and effluent hydrographs of the type showed in the following figure (Portela, 2005).

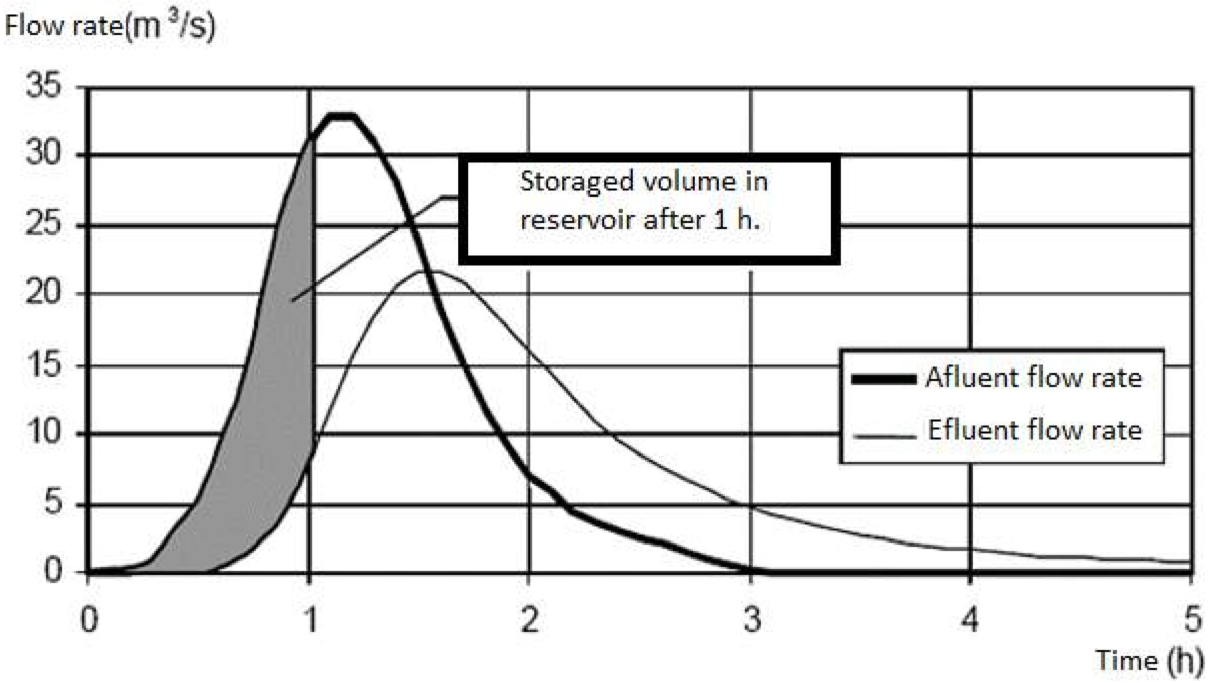

Figure 74 – Hydrographs of affluent flood and effluent of a reservoir destinated to damping flood waves, provided with a flood spillway with free discharge (Portela, 2005).

It is observed that the dimensioning of the flood dump of a dam in the case of no flood damping, it is performed for the maximum flow that is admitted to the reservoir. In both design conditions, as well as for floods with lower flow rates, the affluent hydrographs and the corresponding effluents always coincide (Portela, 2005).

The **linear reservoir model** can be used to investigate this flood damping in a reservoir, that it is a model that considers the effects of storage and its attenuation. However, it is considered by several hydrologists as a conceptual model that is based on the concept of conservation of mass. The mass balance equation of the basin can be written by:

$$\frac{dS}{dt} = I(t) - Q(t)$$

where S the volume stored in the drainage basin at a given time (m^3/s); I is the useful precipitation falling on the basin at a given moment or the affluent flow rate (upstream) at a given instant (m^3/s); Q is the direct flow left downstream of the basin at a given time or the effluent flow (downstream) at a given instant (m^3/s). The difference between the upstream flow rate and the downstream flow rate, both time functions, is equal to the change in the storage time in the stretch.

The storage is related to the flow that comes out and enters through a simple function according to the simplest cases, or in the more complex models. The attraction of these methods is its relative simplicity compared to physically based methods, requiring very little data and allowing very fast simulations. They present as main limitations the facts of ignoring the downstream effects, and of not allowing to represent any effect of transitory flows due to specificities along the stretch. In the linear reservoir model the flow rate is proportional to the storage:

$$S = K \cdot Q$$

Resulting from the combination of two expressions following the next differential equation:

$$I - Q = K \cdot \frac{dQ}{dt}$$

By solving the differential equation by an analytical method, it is obtained:

$$Q(t) = Q \cdot e^{\frac{(K-\Delta t)}{K}} \vee Q(t) = \frac{1 - e^{-\frac{1}{K}}}{\Delta t}$$

The next figure shows the model of a linear reservoir:

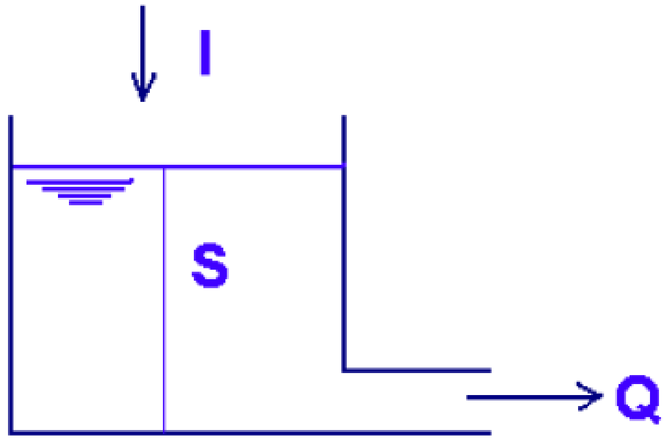

Figure 75 – Characterization of a linear reservoir (Porto, Filho, & Marcellini, 1999).

The coefficient of proportionality of a reservoir K is obtained experimentally, representing the time lag of the basin, which is the difference between the center of mass of the hydrographs.

8.8 Propagation of the hydrographs of floods in water lines

8.8.1 Introduction

Similar to other models applied in hydrological modelling, also flood propagation models - in a canal stretch or in a reservoir, encompassing, in the latter case, the so-called flood damping models in reservoirs, previously presented – can be classified in aggregate models and in distributed models according to time, providing flow rates or corresponding free surface coordinates in a single cross-section coincident with the downstream end of the stretch or in successive cross-sections of that stretch (Figure 76). Aggregate propagation models are sometimes referred to as hydrological and distributed, as hydraulics (Portela, 2005).

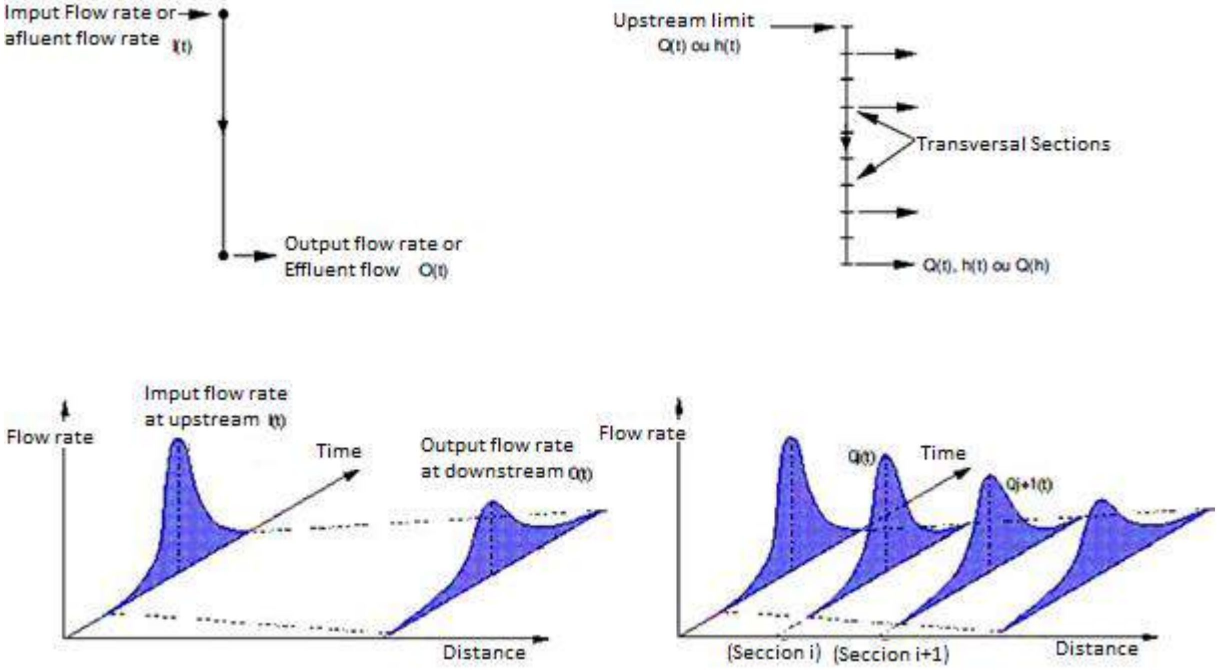

Figure 76 – Schematic representation of a model of propagation aggregated and distributed (adapted from Portela, 2005).

8.8.2 Aggregate models. Previous considerations

The aggregate models use the equation of continuity expressed in the form of a storage equation (Yevjevich, 1975) as a function of the variable values over time, t, of the flow rate entered in the upstream section of the stream or affluent flow rate , I, of the flow rate coming at downstream strecht or effluent flow rate, O, and of storage, S (Portela, 2005). Such equation was already presented for flood damping, so reproduce as follows:

$$\frac{dS}{dt} = I - O \leftrightarrow dS = I \cdot dt - O \cdot dt$$

As mentioned above, the use of the above equation must be supplemented by an additional relation, a function of the stored volume, that matches the quantities in the presence of S, I and O, which, in the general case, can be described by an arbitrary function of I , of O and their derivatives in order to time (Portela, 2005).

The form of the function f, of the stored volume, depends on the nature of the system to analyze, allowing to differentiate between aggregated models. For example, a function f of the following type:

$$S = f(O)$$

expresses a one-to-one ratio between S and O, making each value of O a single value of S and reciprocally. This relationship is appropriate, for example, to the damping of flood waves in reservoirs provided that the free surface can be considered horizontal at any instant, as in the case of reservoirs with a width and depth well in excess of the length in the direction of propagation of the flow. Under these conditions, at each boundary of the free surface corresponds a single effluent flow, being the stored volume only a function of an elevation or height, on which also the effluent flow depends (Portela, 2005).

The main advantage of aggregated models in relation to disaggregated models is their simplicity. However, they are not always applicable or sufficiently precise, as for example in the case of application to flood waves, characterized by hydrographs with a marked increase in flow over time, propagating in channels with moderate to low slope (Portela, 2005).

According to Portela (2005) the aggregate models can be categorized as they consider at each instant that:

- the free surface is horizontal (the case of the finite difference method applied to damping of flood waves in reservoirs);
- because of propagation of the flood wave, the free surface is not horizontal, presenting a certain slope (Muskingum method);
- the system along which propagation takes place consists of successive linear reservoirs, connected by straight channel sections, characterized by a unit-type response function (impulse), with the relation between the tributaries and the effluent defined using a convolution integral.

8.8.3 Muskingum's Method

The Muskingum method is, among the aggregated models, the one with the most widespread application to the propagation of flood waves in channels. The method uses the equation of continuity expressed in the form of the equation previously presented and considers that the storage in the channel stretch results from the sum of two stores, one prismatic and the other said in wedge, shown in Figure 77 (Portela, 2005).

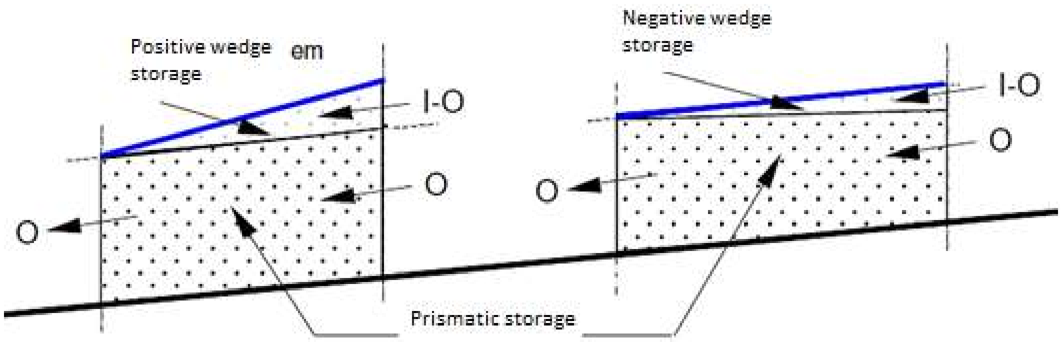

Figure 77 – Method of Muskingum. Prismatic storages or in wedge (adapted from Portela, 2005).

The prismatic storage is what would correspond to the configuration of the free surface in steady state and the storage in wedge to the volume stored between the previous configuration and the configuration of the free surface during the occurrence of the flood. In the phase of increase of the free surface dimensions by an increase of the flow rate of the flood, the wedge storage is positive, adding to the prismatic storage, and in the phase of decrease of the free surface dimensions after the passage of the negative peak flow rate, subtracting from prismatic storage (Portela, 2005).

In each channel section to which the method is applied, the prismatic storage is given by the product of the effluent flow rate of the stretch by the course time in the stretch, K. The wedge storage is given by a weighted difference between the affluent flow rates in the section at upstream of the stretch and effluent at downstream, difference also multiplied by the course time in the stretch in order to obtain a volume, that is:

$$S = K \cdot O + K \cdot X \cdot (I - O) = K[X \cdot I + (1 - X) \cdot O]$$

where X is the factor of weighting (Portela, 2005).

If the storage in the channel segment is essentially controlled by conditions occurring downstream depending intrinsically from the effluent flow rate, then, $X = 0$ and then $S = K \cdot O$, a relation that defines a linear reservoir. If $X = 0.5$, the effluent and effluent flows rates have the same "weight" and the propagation of the flood wave occurs without attenuation, that is, the wave essentially undergoes a translation when propagating in the channel stretch, according to Figure 78 (Portela, 2005).

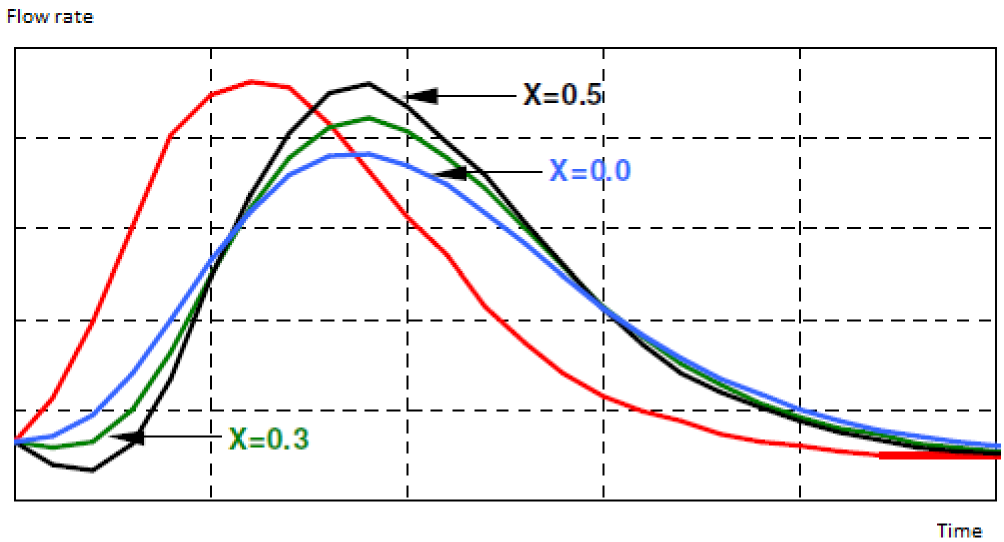

Figure 78 – Muskingum method. Effects of the parameter X in the damping of the flood waves propagating in a given section of a canal (Portela, 2005).

The discretization of the storage equation by finite differences between two successive instants of calculation identified by indices 1 and 2 followed by the application of the storage equation in the channel stretch at these instants, leads to:

$$\begin{cases} \dfrac{I_1 - I_2}{2} - \dfrac{O_1 - O_2}{2} = \dfrac{S_2 - S_1}{\Delta t} \\ S_1 = K[X \cdot I_1 + (1 - X) \cdot O_1] \\ S_2 = K[X \cdot I_2 + (1 - X) \cdot O_2] \end{cases}$$

By introducing the storages defined by the last two equations of the previous system in the continuity equation and manipulating the results obtained, it successively become:

$$I_1\Delta t + I_2\Delta t - O_1\Delta t - O_2\Delta t = 2KXI_2 + 2K(1-X)O_2 - 2KXI_1 - 2K(1-X)O_1 \rightarrow$$

$$\rightarrow I_1\Delta t + 2KXI_1 + I_2\Delta t - 2KXI_2 - O_1\Delta t + 2K(1-X)O_1 = 2K(1-X)O_2 + O_2\Delta t \rightarrow$$

$$\rightarrow I_1\left(\frac{\Delta t}{K} + 2X\right) + I_2\left(\frac{\Delta t}{K} - 2X\right) + O_1\left[2(1-X) - \frac{\Delta t}{K}\right] = O_2\left[2(1-X) + \frac{\Delta t}{K}\right] \rightarrow$$

$$\rightarrow I_1\frac{\Delta t/K + 2X}{2(1-X) + \Delta t/K} + I_2\frac{\Delta t/K - 2X}{2(1-X) + \Delta t/K} + O_1\frac{2(1-X) - \Delta t/K}{2(1-X) + \Delta t/K} = O_2$$

That is:

$$I_1\underbrace{\frac{\Delta t/K + 2X}{2(1-X) + \Delta t/K}}_{C_1} + I_2\underbrace{\frac{\Delta t/K - 2X}{2(1-X) + \Delta t/K}}_{C_0} + O_1\underbrace{\frac{2(1-X) - \Delta t/K}{2(1-X) + \Delta t/K}}_{C_2} = O_2$$

which leads to the following system of equations defining the method of Muskingum:

$$\begin{cases} O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \\ C_0 = \dfrac{\Delta t/K - 2X}{2(1-X) + \Delta t/K} \\ C_1 = \dfrac{\Delta t/K + 2X}{2(1-X) + \Delta t/K} \\ C_2 = \dfrac{2(1-X) - \Delta t/K}{2(1-X) + \Delta t/K} \end{cases}$$

In which verifies that:

$$C_0 + C_1 + C_2 = 1$$

The parameter K can be understood as the travel time of a flood wave along the channel stretch, considering the translation of such wave (Portela, 2005).

The parameter X appears as a weighting factor that introduces the effect of the damping of the wave during propagation. Such damping translates into reduction of the effluent hydrograph peak flow rate at the downstream end of the section according to the peak flow of the tributary hydrograph at the upstream with consequent increase of the base time of that hydrograph in relation to the base time of this last hydrograph (Portela, 2005).

If $K = \Delta t$ and $X = 0.5$ then $C_0 = C_2 = 0$ and $C_1 = 1$, that is, $O_2 = I_1$ and the flood wave only suffers a translation as it propagates, therefore without any damping. If $X = 0$ then $S = K \cdot O$, equation that translates the linear reservoir model (Portela, 2005).

As mentioned, $0 \leq X \leq 0.5$ and more frenquently $0.1 \leq X \leq 0.3$, does not require great precision because the results are relatively insensitive to such parameter. The time increment is usually between $K/3 \leq \Delta t \leq K$ (Portela, 2005).

So, it is possible to write:

$$\frac{S_2 - S_1}{\Delta t} = \frac{K\{[X \cdot I_2 + (1 - X) \cdot O_2] - [X \cdot I_1 + (1 - X) \cdot O_1]\}}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}$$

Obtaining:

$$K = \frac{0.5\Delta t[(I_2 + I_1) - (O_2 + O_1)]}{X(I_2 - I_1) + (1 - X)(O_2 - O_1)}$$

ratio that can be applied to assign values to the parameters K and X since, for this purpose, it is available the effluent flood hydrograph corresponding to the affluent flood hydrograph (Portela, 2005).

For this purpose, successive values of X are arbitrated. For each of these values and by recourse to a system of coordinated axes, the values provided by the numerator of the preceding equation (yy axis) are represented according to the corresponding values of the denominator of the same equation (xx axis). Normally the graph thus obtained displays a loop. The value of X to be adopted should be what leads to the best overlap of the sides of the loop, approaching the loop to a segment of the line as much as possible, illustrated in Figure 79 (Portela, 2005).

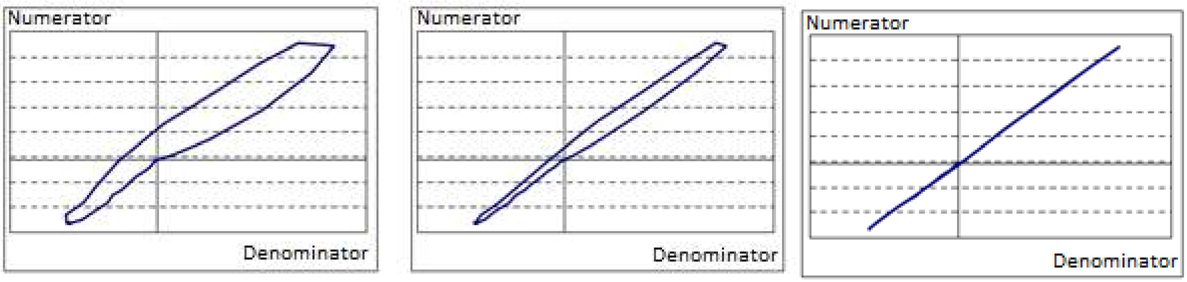

Figure 79 – Muskingum Method. Search of the value of parameter X in the availability of hydrographs corresponding to the affluent direct flow in the upstream section of the channel and effluent section in the downstream portion of such a stretch (Portela 2005)

Once the value of X is identified, the corresponding K value is given by the slope of the previously obtained line segment, as is also apparent from the expression $C_0 + C_1 + C_2 = 1$. It is noted that, representing K the travel time in the channel section, its value may also be approximated by the peak flow rate propagation time between the extreme upstream and downstream sections, if such time is possible to be estimated (Portela, 2005).

8.9 Flood characterization

8.9.1 Introduction

Floods are at Earth-scale, a natural danger that the largest fraction of the population affects. It is a danger that reaches the area of the territory located in the vicinity of the hydrographic network, the coastline, or of levees and dams. According to the World Meteorological Organization, flood-induced disasters have been increasing as a consequence of urban expansion in alluvial plains (Miranda & Baptista, 2006).

According to FEMA, full can be defined as "a general and temporary condition of full or partial flooding of an area exceeding 20 acres (about 8092 m^2) of habitually dry terrain, or more than one property, as result from the overflow of inland waters or from the sea, or by the fast and unusual accumulation of surface water of any origin, mud or collapse of land along the coast, of a surface of water, as a consequence of erosion or destruction by the waves or by a current which intensity is higher than the cyclical levels (...)" (Miranda & Baptista, 2006).

Several flood-generating phenomena can be identified: progressive river floods, storms (Storm Surges), fast flooding (Flash Floods), including mud and lait torrents, and the collapse of levees or dams. According to FEMA, each American housing has a 26% probability of being hit by a flood in the next 30 years, while the probability of fire is 9%. The value of the average annual losses of the USA because of floods was 2.4 billion USD in the decade 1996/2005 (Miranda & Baptista, 2006).

8.9.2 Origin of floods

Floods are associated with some extreme natural events occurring in a given hydrographic basin, whether natural (rural) or urban.

The main cause for floods occurring in the natural environment are the intense rains of short duration or when combined with snow melt exceeding the natural capacity of the water lines. Other causes for the occurrence of floods are:

- Overload of water levels due to natural or man-caused obstructions in the path of the flood (bridges, gate-controlled unloaders, etc...);
- Rupture of dams;
- Slope slides;
- Debris flows (sludge);
- Inadequate urbanisation (in flood beds or when they cause too much water lines or its overlapping);
- Snow fast defrost;
- Massiveness deforestation of the hydrographic basin.

8.9.3 Types of floods

Progressive river floods

As being particularly important to Portugal, due to its frequency, the irregularity of precipitation (interannual and seasonal) leads to a variation of great amplitude of the flow rate of rivers , with significant differences between the rivers in South and Northwest: The rivers of the South have specific flows annually 6 to 7 times lower than those of the Northwest, greater irregularity, and the flow rate in the rainy years exceeds 100 to 240 times that in drier years. The southern rivers are almost all temporary and have flood tips or flow peaks that reach 200 to 300 times the average annual flow rate (Miranda & Baptista, 2006).

Upon the occurrence of a flood, the affluent flow to the drainage network can lead to a situation, where the channel is no longer able to contain the volume of circulating water, it exceeds the protection zones and invades the surrounding area (flood bed). About 80% of Portugal (mainland) has a reduced permeability of substrate (granites, shales and clayey formations), apart from the springs fed by the karst limestone from the center of the country. These conditions contribute to increase the risks of flooding (Miranda & Baptista, 2006).

The progressive floods are conditioned by a system of dams that can eventually promote cushioning, but also, under certain conditions, can contribute to the increase of the peak of flow rate(flood peak flow rate rate) (flood of 1979) (Miranda & Baptista, 2006).

Storm surges

The storm surges are generated by the combined action of a weather storm and the tide. They can generate very high levels of destruction in lower coastal regions, particularly those that are protected by dikes, and the destruction is important both in the "run-in" and "run-out" phases.

A European example is the storm surge of 1953 which severely hit England and Holland, causing more than 1800 deaths and ten thousand displaced people. One of the countries in the world most vulnerable to storm surges is Bangladesh, reached in 1970 and 1991, with records of deaths of 300000 and 140000 inhabitants, respectively (Miranda & Baptista, 2006).

Collapse of Dams

In the last decades one or two collapses of large dams has occurred in the world. On 9 October 1963, a landslide of 240 million cubic meters reached a reservoir of water in Vaiont originating a flood wave at the dam with 265 meters high, causing 3000 dead. On 11 August 1972, 68 communities were destroyed along the Macchu River in India, leading to the deaths of thousands of people and also affecting about 150000 people (Miranda & Baptista, 2006).

According to Almeida (2001), some paradigmatic accidents that occurred in Europe, in the second half of the 20th century, forced to reflect on the risk in the downstream valleys and to prevent the potential effects of dam ruptures. For example, the most important ones are cited:

- Malpasset Dam, in France, of concrete (arch) with 61 m high, broke in 1959, causing an induced flood, with 421 deaths along the 11 km of valley until the Mediterranean;
- The Vega de Tera Dam in Spain, with 34 meters high, had partially collapsed in 1959 and causing the death of 144 people in the valley downstream;
- Dam of Vaiont, in Italy, of concrete (arch) with 265 m of height, was overpassed or drowned in 1963, by water that it was initially stored in a reservoir (150 × 106 m^3) that collapsed due to an landslide of about 240 × 106 m^3 of rock from a hillside, and consequently causing the death of about 2 600 people in at downstream valley and areas.

These accidents have arisen a polemic in recent years, a polemic about dam safety. According to Almeida (2001) and according to the Portuguese Dam Safety Regulation (article 12º), the safety of a dam involves structural, operational and environmental aspects:

- Structural safety, corresponds to the capacity of a dam to meet the requirements of structural behaviour facing the actions and other influences, associated with construction, exploitation and exceptional occurrences;
- Hydraulic safety corresponds to the capacity of a dam to meet hydraulic behaviour requirements as safety, operating systems, waterproofing, filtration and drainage systems;
- Operational safety corresponds to the dam's capacity to meet behavioural requirements related to operation and functionality of the safety and operating equipment;
- Environmental security corresponds to the capacity of a dam to meet the behavioural requirements related to the limitation of harmful effects on the environment, especially on population and productive resources.

Almeida (2001) refers that the environmental security includes, among other aspects, a concern of the effects on population and productive resources located in the areas under the potential effect of the dam (including the reservoir), namely:

- The need to relocate habitants and production media at upstream (particularly in the construction phase), as a result of the flood or "drowning" caused by the creation of the reservoir;
- At the downstream valley, particularly at the design and operational stage, the need to predict all possible consequences of an accident, regardless of the probability of its occurrence.

Flash floods

A flash flood is the result of very intense precipitation over a period of a few hours. Rapid floods are deadly (in Portugal they took place in 1967, 1983 and 1997, for example). They affect small drainage basins and are caused by stationary convective depressions caused by the interaction between polar and tropical circulations, namely in the south of the country, in the regions of Lisbon, Alentejo and Algarve (Miranda & Baptista, 2006).

The flash flood that occurred in Lisbon at dawn on November 26, 1967, in the Loures area was generated by high precipitation during a short period: in the meteorological station of Monte Estoril, 159 mm were recorded between 10 o'clock on day 25 and 10 o'clock on day 26 (about 1/5 of the average annual precipitation), with 129 mm in only five hours (from 7:00 p.m. on day 25 to 0:00 p.m. on day 26) and 60 mm, between 21:00 and 22:00 p.m. About 700 people died, most of them habiting buildings located in floodplains (Miranda & Baptista, 2006).

Examples of major losses in Europe are those in October 3rd, 1988, in Nimes (France) where, over a period of a few hours, losses of around 1 billion USD were generated. In Switzerland, on September 24, 1993, in the city of Brig, a loss of more than 400 million USD was caused by the torrent of the Saltina River, amplified by torrential rains (Miranda & Baptista, 2006).

Sludges torrents

Poorly consolidated soils can easily slide under the action of intense precipitation. If its saturation is too high the mixture of water and mud can move at high speed with a very large destructive potential. An example of this phenomenon occurred in August 1987 in Switzerland, with the formation of numerous torrents of mud on the slopes of the Alps and a final loss of hundreds of millions of USD (Miranda & Baptista, 2006).

Lahar

The word *lahar* has Indonesian origin and describes a torrent of mud originated at a volcanic cone. When an eruption occurs, large amounts of ash accumulate at the base of the volcanic cone, which can be mobilized by intense precipitation. For example, a volcano covered by snow and ice erupts, the mass of water from the quick thaw can mix with volcanic ash and debris, becoming a process of high destructive power.

Some examples of *lahar* are those that occurred on May 24, 1926 in the Japanese volcano Tokachi-dake on Hokkaido Island, that generated the destruction of about 5080 fires and caused 144 deaths. One of the most famous occurrences took place in 1985 in the volcano Nevado del Ruiz in Colombia, that victimized 23000 people and destroyed 5100 houses (Miranda & Baptista, 2006).

Inundation

The flood concept is associated with the inundation concept. Although they are used synonymously, they are not: an inundation happens whenever there is submersion of an area that, usually, is emerged. An inundation is not necessarily caused by a flood. However, according to Ramos (2013a) and Strahler (1975) a flood originates, invariably, an inundation. Taking this distinction into account, the term full is equivalent to the term "fluvial inundation" (Rodrigues S. P., 2017).

Although the concept of flooding or inundation varies, it is largely consensual and was defined in Portuguese legislation by Decree-Law n.º 115/2010, of October 22, transposing into national judging order law, the Directive 2007/90/CE of the European Parliament and of the Council of the European Union – Directive, Inundations in the following form: "(...) understood as: (...) b) «Inundation» as the temporary drowning or coverage of a plot outside the normal waterbed by water, as a result of flood incited by natural phenomenon as precipitation, increasing the flow rate or flow rate of rivers, mountain torrents, streams and ephemeral water courses according to the pluvial floods or overelevation of the sea water levels in coastal zones;" (Rodrigues S. P., 2017).

Such definition of inundation surge in the context of evaluation and management of flooding risk and includes both types, pluvial and coastal (Rodrigues S. P., 2017).

8.10. Mitigation measures and effects arising from floods

An essential factor for alerting authorities, warning populations and preparing relief actions is the time taken to forecast a flood (whether flooded or not) and its implementation. Flood prevention is carried out through two components: the forecast, which allows the anticipation of mitigation actions; and the monitoring, which allows to detect and to know, in each moment, the degree of gravity of the situation. This last component is strongly hydrological (ANPC, 2018).

The time required for a flood to occur and its duration depend on the characteristics of the catchment area of the river in question. Small basins usually have conditions to form and rapidly propagate for a flood, sometimes in very few hours. On the contrary, in large basins, the peak of the flood wave, and the inherent floods, take longer to settle, allowing a timelier warning to the populations. They also take longer to disappear, which can take several days (ANPC, 2018).

According ANPC (2018) losses resulting from floods are often large and can lead to:

- **Direct effects:**
    - loss of human life, evacuation and displacement of people;
    - the isolation of settlements;
    - the damaging of public or private property;
    - submersion and/or damage to communication routes and other infrastructures and equipment;
    - the destruction of agricultural and livestock farms;

  - the interruption of the supply of basic goods or services (drinking water, electricity, telephone, fuel, etc.);
  - production losses;
  - the cost of Civil Protection actions, including the relocation and treatment of victims.
- **Indirect effects:**
  - production losses;
  - social-economics activity distortion, sometime for very prolonged period;
  - distortion of the environment.

8.10.1 Structural measures

Structural measures are engineering interventions that seek to reduce the risk of flooding (CPRM, 2004).

Types of works:
- Flood containment works;
- Detention Reservoirs;
- Lateral reservoirs;
- Containment dikes;
- Flood clearance works;
- Micro and Macro-drainage;
- Reversion of basins.

Cautions:
- Risks of breakups;
- Occupation of the larger bed.

Disadvantages:
- Execution of Works;
- Intervention in natural drainage;
- Intervention in the channel;
- Localized effects: displacement of inundations;
- High cost: concentrated investments;
- Hydrological risks;
- Alteration of the hydrological behaviour of the basin;
- Operation and maintenance;
- Environmental impacts.

In the urban floodplains, the artificial structures, as a principle to be applied, must be carefully dimensioned and constructed in order to fit sustainably into the natural surroundings of the green areas of urbanisation, as shown in Figure 80.

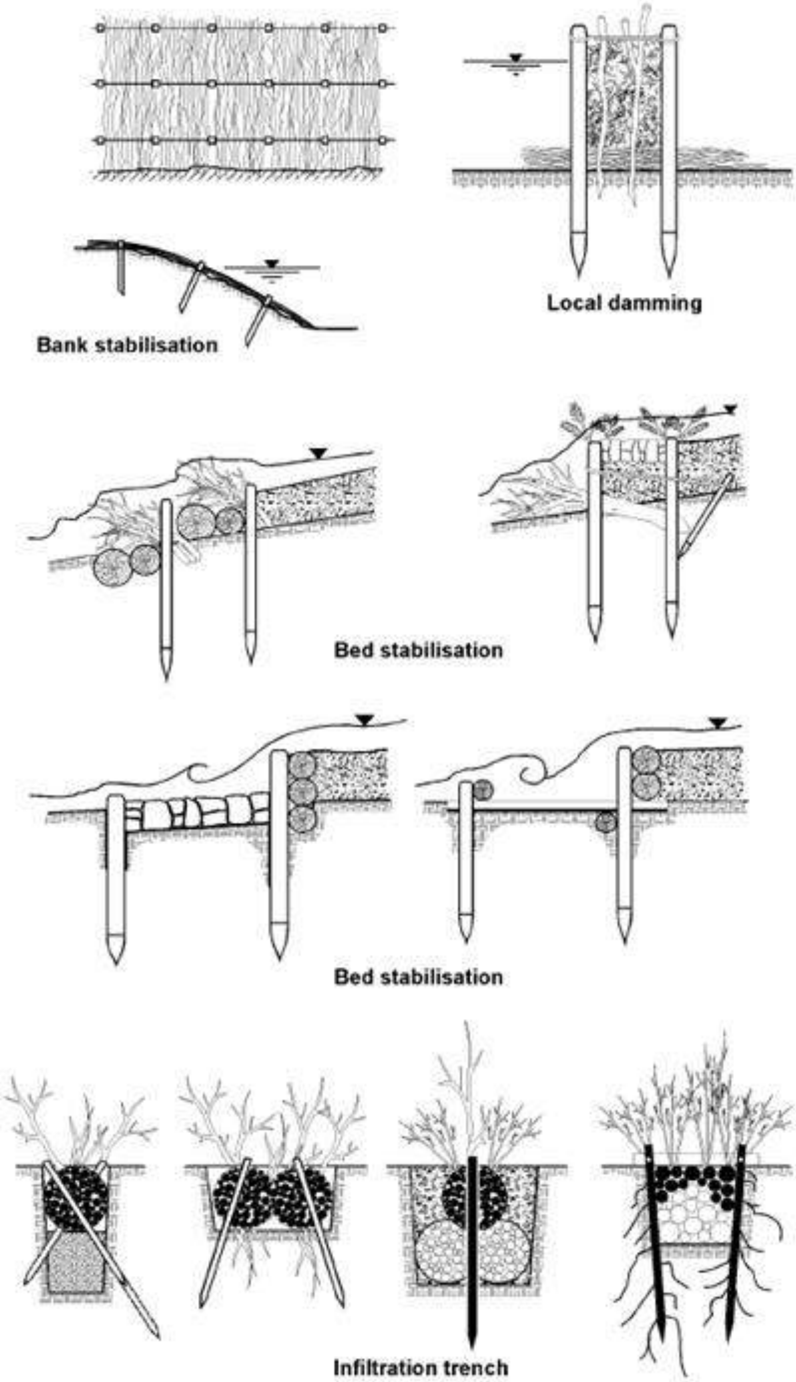

Figure 80 – Des-naturalization of urban river using wood and tar (Andjelkovic, 2001).

In some countries the buildings are suitable for the occurrence of floods, by constructing in them, elevated artificial or natural terrain, construction on piles or columns, accesses through staircases in the main sidewalk, use of sandbags to temporarily prevent flooding, among others (Figure 81).

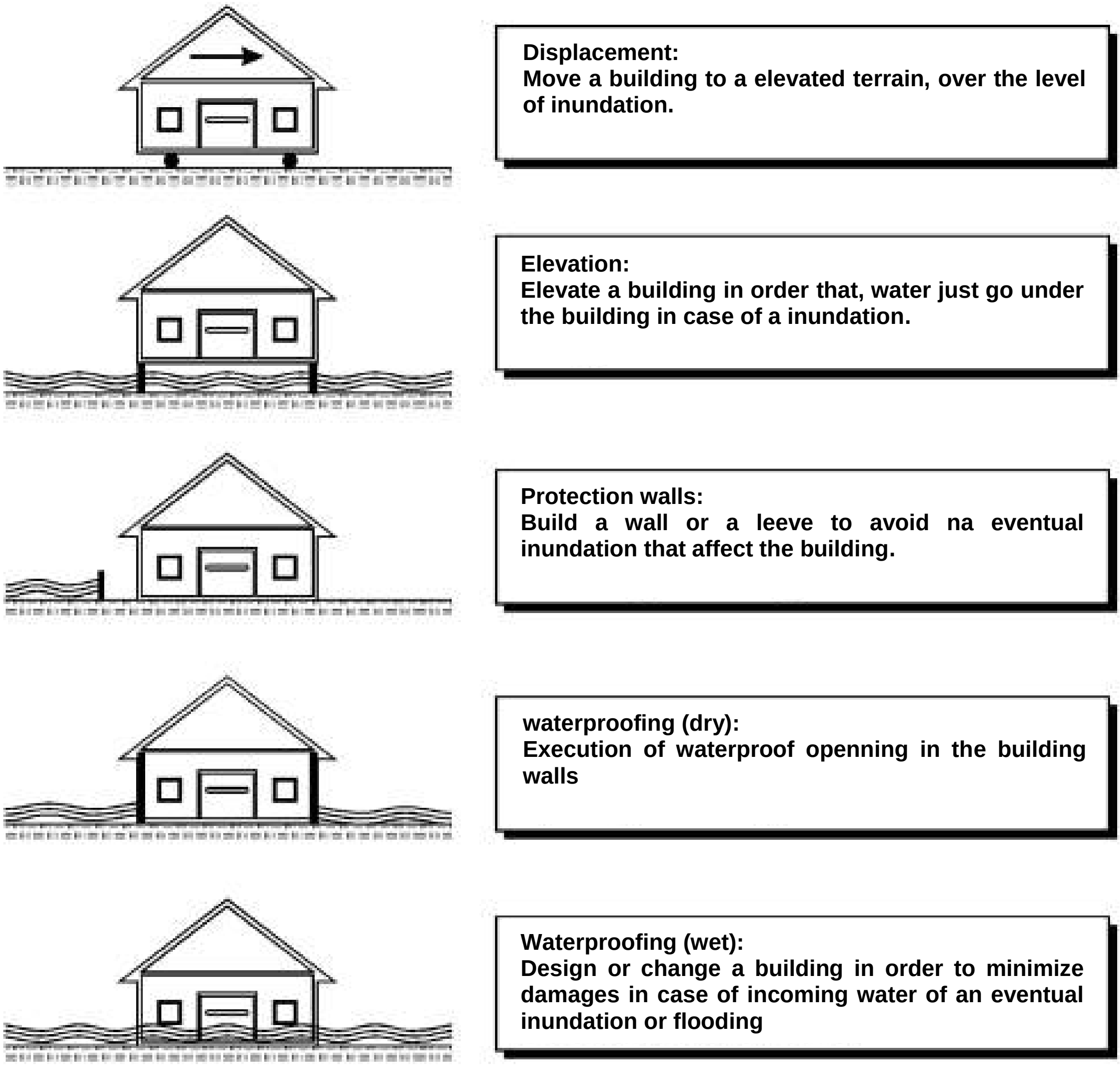

Figure 81 – Position of building facing a flooding (Andjelkovic, 2001).

8.11.2 Non-structural measures

Non-structural measures aim to reduce losses through better coexistence of the population with floods (CPRM, 2004).

Types:

- **Preventive:**
  - Regulation of land use;
  - Purchase of floodable areas;
  - Control of water and sewage networks;
  - Information and education programs;
  - Forecasting and alarm systems;
  - Flood insurance.

- **Corrective:**
    - Flood-proof constructions;
    - Relocation;
    - Land purchases;
    - Population displacement;
    - Gradual occupancy adjustment.

Benefits:
- Damage reduction;
- Reductions in standstills and chaos;
- Reduction of street maintenance costs;
- Greater opportunities for recreation;
- More green areas and urban spaces;
- Better protection of margins;
- Less sedimentation.

Advantages:
- Do not only involve works;
- Seek to adapt urban life to the natural phenomenon of floods;
- Seek to adopt preventive measures;
- Applied in a diffuse way over the basin and more specifically in the flood floodplain;
- It relies on socio-political aspects such as population education and public participation;
- It is cheaper, but not necessarily easier to apply.

Flood margins mapping

The mapping of vulnerable areas (Figure 82) is realized starting from the definition of inundation risks of different heights and the respective mapping. The regulation and mapping of riverside areas will define types of occupation that will be allowed in regions of greater or lesser risk to flooding and must be part of the city's urban master plan (CPRM, 2004).

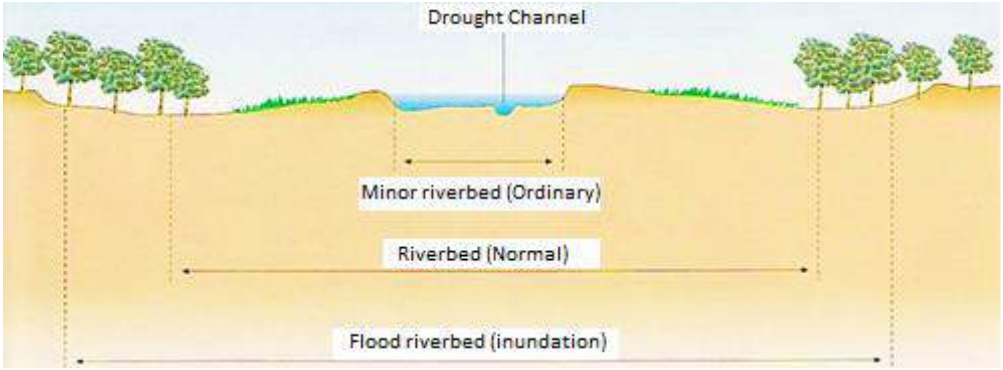

Figure 82 – Transversal profile of a course of water (Coque, 1987).

Areas occupied by the flow:

- Zone 1 - Minor riverbed:
    - Area of fast flow, major part of the flow rate or flow. It must be clear, free and empty;
    - Works: only water catchments and bridges;
- Zone 2 - Major bed:
    - Significant portion of the flow. It must have a high restriction;
    - Works: Parks and adequate buildings;
- Zone 3 - Only inundation:
    - Just steady waters;
    - Works: construction flood-proofed; limited only for essential services;
- Zone 4 - Secure areas:
    - Above of NA, T = 100 years;
    - Works: Non-restricted.

Mapping purposes:

- Prevent new projects, incompatible with risks;
- Prevent the acquisition by inadvertent persons;
- Reduce public spending with aid, emergencies, etc.;
- Reduce spending on future flood control works;
- Greater leisure opportunities, green areas etc.

8.11 National Vulnerability

The area vulnerable to floods is, at first, the "flood riverbed", since a flood can take place due to an excess of local precipitation, the integration of precipitation in the basin of retention associated to a river or a catastrophic phenomenon at upstream. The determination of the flooded area in a given scenario can be performed by methods described above, and it is possible to define objective criteria for a delimitation of vulnerable regions (Miranda & Baptista, 2006).

According to Miranda & Baptista (2006) the National Water Plan evaluated the vulnerabilities based on flood effects up to 2000/2001. This survey included the elements at risk (affected settlements, by isolation or flood, type of affected buildings, agricultural and farming areas, infrastructures and miscellaneous equipment). This plan points to the existence of critical situations in the Tejo basins (Santarém district), Douro (districts of Porto and Vila Real) and Vouga (Aveiro district). In the national basins, the following critical points have been identified, as:

Rio Minho - The areas most affected by floods, are available in the riverside area of the national margin of the main course, highlighting the localities of Valença, Vila Nova de Cerveira and Monção, which suffers most problems. It is to record the strong dependence to vulnerabilities, due to flooding when facing precipitation occurring in the Spanish part of the basin and the discharges of its dams.

Rio Lima - Ponte de Lima, Ponte da Barca and Arcos de Valdevez are the urban areas most affected by floods in this basin. The laminar flow of flow rates in the existing hydroelectric dams allows to mitigate the risks of flooding in the first two localities, but the effect of the terrain orography (Serra da Peneda) raises an increase of precipitation that translates into the formation of a high peak flow rate, not always likely to be stored in the reservoirs.

Rio Cávado - This basin is strongly influenced by the precipitation occurring in the Gerês region, which registers some of the highest values in the country during the winter period. Braga, Barcelos, Guimarães, Vieira do Minho, Terras do Bouro and Esposende are some of the municipalities with more affected urban centres.

Rio Ave - By the influence of the Ave and its tributary Vizela, some councils of the basin are affected by floods, usually of short duration, due to the relatively small dimension of the basin.

Rio Leça - The final section of this river, in the area of Maia, is the most vulnerable to flooding, usually with high peaks but of short duration.

Rio Douro - It is a river that originates, in some sections, large cyclical floods, with great impact on the socio-economic fabric of riverside populations. Localities such as Porto, Vila Nova de Gaia and Peso da Régua, on the Douro River, and Chaves and Amarante, in the Tâmega, are often overrated by fiery floods. The successive construction of dams in the basin, mainly in the Spanish territory, did not introduce significant changes in the flood regime, because its reservoirs have a reduced docking capacity, preventing them from exercising the necessary Damper effect.

Rio Vouga - The estuarine conditions of the final section of the Vouga River are susceptible to aggravate some water drainage problems, particularly in situations of high maritime agitation in which the flow from the river to the sea is difficulted. It deserves to highlight in this basin, the critical problems of some basins as the cases of the basins of the river Águeda (influenced by precipitation in the area of Caramulo), which affects the city of Águeda and the river Cáster, affecting Ovar.

Rio Mondego - The main problems in this basin arise in the agricultural fields of the Baixo Mondego and are usually not only to Mondego itself but also to its main tributaries (Dão, Alva and Arunca). The regularization made in the Aguieira Dam allows to mitigate the main problems of floods, through the lamination of flow.

Rio Lis - Without major flood problems at the level of human consequences, the most affected areas are located on agricultural land.

Rio Tejo - In the case of an international basin, the water storage capacity in Spain and the way water management is carried out also determines the frequency and intensity of floods in Portugal. However, it will be important to remember that the set of hydroelectric evaluations built in the Portuguese part of the basin are not enough to prevent the occurrence of floods.

The floods in the Tejo basin originate in the district of Santarém situations as cuts of several national and municipal roads, interruption of the railway circulation, flooding of agricultural fields and isolation of populations (Reguengo do Alviela, Caneiras, Valada, Valada do Ribatejo, Azinhaga and Palhota). The municipalities of Santarém, Cartaxo, Golegã, Almeirim and Alpiarça (Tejo river), Tomar (River Nabão) and Coruche (Rio Sorraia) are some of the most vulnerable. There are also sudden floods, as a consequence of intense precipitations of short duration, fundamentally in the highly waterproofed areas of great urban development. And in the case of Lisbon metropolitan area, on the right bank of the Tejo river, between the municipalities of Cascais and Azambuja.

Rio Sado - the Sado river Basin is located in an essentially flat area where floods are only expected in special cases. The dams implanted in the Sado river basin have mainly agricultural purposes but ensure the regularization of a significant part of the flow. In the municipality of Alcácer do Sal, however, some settlements are located at risk of isolation, when the storage capacity of the dams is not enough. Occurrence of sudden floods in the municipality of Setúbal is highly possible.

Rio Mira - without major flood problems at the level of human consequences, the most affected areas are in agricultural land.

Rio Guadiana - Vulnerable to the discharge of some hydroagricultural projects, both on the Portuguese and the Spanish side, it has in areas at downstream of the reservoirs of Caia (district of Portalegre), and above all, at downstream in the riverside areas of Mértola and Alcoutim (both downstream of the Chança, tributary of the Left bank) are the most vulnerable areas. This situation will be naturally modified with the entry into operation of the Alqueva Dam.

Ribeiras do Oeste, Alentejo e Algarve - The reduced extension of these basins favours fast flow rates, so, it is not expectable to have floods of great duration. However, areas such as Lourinhã, Alcobaça (Ribeiras do Oeste), Silves and Tavira (Ribeiras do Algarve) have evidenced in the past some vulnerabilities to floods.

CHAPTER 9 - EVAPORATION AND EVAPOTRANSPIRATION

9.1 Introduction

It is designated as **evaporation (E)** to the process, where water pass from liquid state to gaseous state at any temperature below the boiling point. The passage from solid state to gaseous state is called **sublimation**, however, in the hydrological balance the sublimation is computed globally with evaporation. The change of the solid or liquid state to the gaseous state occurs when the kinetic energy of the molecules that constitute the substance increases, demanding therefore according to a constant temperature, consumption of a certain amount of energy. This amount of energy per unit mass of the substance is called vaporization heat (Rodrigues et al., 2011).

The evaporation that would occur if water does not constitute a limiting factor and the vapour pressure of the evaporating surface would be of "saturation", then it is called the potential evaporation (E_p) (Rodrigues et al., 2011).

It is designated as transpiration **(T)**, the evaporation of water absorbed by plants and eliminated by them in the different biological processes (Lencastre, 1984), that is, the passage of water vapour from plants to the atmosphere. The passage of water absorbed by the plants into the atmosphere is mainly through the stomas (pores in the lower part of the leaves). The Stomas open with sunlight, allowing the diffusion of carbon dioxide into the leaves, the water contained in the cells passes into the intercellular spaces, where it is vaporized and, when the pores are open escapes to the atmosphere, being the exuded water replaced by the water that the roots will bring from the soil (Rodrigues et al., 2011).

The set of these two processes constitutes **evapotranspiration (Et)**, which therefore includes the transpiration of the plants and the evaporation of the surrounding environment (land surface, water from the ditch, rivers, lakes, etc.). Evapotranspiration constitutes, thus, all the "water loss that would occur in soil conditions perfectly stocked with water for the use of vegetation" (Thornthwaite, 1944) that is, translates the evaporation from a large cultivation surface, which covers the soil in totality, exerts a minimum resistance to the water flow, this being a non-limiting factor (Rodrigues et al., 2011).

According to Quintela, 1984, the names of evaporation and evapotranspiration are used to refer either to the processes of water transfer to the atmosphere or the respective quantities, which are expressed in height of water on its surface (mm) (Rodrigues et al., 2011).

It is called **potential evapotranspiration (Et_p)** to the amount of water that can pass into the atmosphere, directly and/or through the plants, if the moisture of the existing soil is always available in sufficient quantity, i.e. if there is no deficiency water supply to the mentioned process. The defined evapotranspiration depends on the type of vegetation cover and its degree of development. For potential evapotranspiration to be presented as a climatic magnitude, it must be referred to a surface. Penmman, cited by Quintela, 1984, suggests that the original definition should be modified to include the specification that the soil surface is completely covered by grass. In these conditions, potential evapotranspiration is independent of the type of culture and is referred to as **reference evapotranspiration (Et_o)**. The evapotranspiration referenced to a given culture is said to be **cultural evapotranspiration (Et_c)** and results from multiplying Et_o by the cultural coefficient, k_c, of culture (Rodrigues et al., 2011).

The **real evapotranspiration (Et_r)** corresponds to the amount of water truly lost by the soil, depending on the atmospheric conditions, the water content in the soil and the characteristics of the vegetation (Rodrigues et al., 2011).

Evapotranspiration covers a large part of the water removed from the basin, so it is important to consider it from a hydrological point of view. The estimates of evapotranspiration are indispensable for predicting water needs in irrigation projects (Rodrigues et al., 2011).

The increase in artificial lakes makes it increasingly significant the portion of the hydrological balance that results from the evaporation of the lakes. Thus, before the establishment of a new reservoir, it will have to be considered the increase of annual evaporation from the addition of new aquatic surfaces. Estimates of water losses by evaporation in reservoirs are necessary to define their capacity and conditions of exploitation, namely the satisfaction of the expected consumption (Rodrigues et al., 2011).

9.2 Involved factors

The factors that mostly condition the evaporation are of two types, climatic and physical. The conditions of evapotranspiration beyond the climatic, are particularly important those related to the characteristics of the vegetation and the type of soil present (Rodrigues et al., 2011).

9.2.1 Climatic factors

Evaporation occurs when some heated liquid molecules reach enough kinetic energy to overcome surface tension and then, it releases from the surface of the liquid. The energy comes from the solar radiation, the heat transported by the atmosphere or the arrival of hot water (urban sewage, cooling waters of the power plants or chemical processes, etc.). Evaporation is therefore conditioned by the solar radiation that depends on the latitude, season of the year, time of day and nebulosity. On the other hand, the vaporized molecules produce a vapour tension (pressure exerted by steam in a certain space). When the space is not able to hold more vapor, the it is saturated, and the pressure exerted by steam in these conditions is called the vapour saturation tension, becoming equal to the atmospheric pressure at the boiling point. The difference between the vapour saturation tension and the actual vapour voltage is called saturation deficit. Thus, evaporation is influenced by air and water temperature, atmospheric pressure and humidity (Rodrigues et al., 2011).

In the absence of wind, the water vapour concentrated in a layer of the atmosphere very close to the free surface, layer that is designated by evaporating layer, reaches the state of saturation. For evaporation to continue, it is necessary to remove the saturated air layer. Thus, appears a new conditioning element of evaporation, the wind (Rodrigues et al., 2011).

In addition, it is necessary for evaporation: energy, difference of vapour tension between the neighbouring layer of the water surface, the atmosphere and wind (Rodrigues et al., 2011).

Apart from to the mentioned characteristics, it is also appropriate to consider the variations of the heat stored by the water masses in a year. In the case of small lakes, especially in semi-arid regions, the very dry air that will replace the moistened layers by evaporation can cause sensitive increases in evaporation-effect of oasis- which in the case of large lakes is practically non-existent (Rodrigues et al., 2011).

9.2.2 Physical factors

The most conditioning factors of evaporation are the geometric characteristics of evaporating surfaces and surrounding regions and the existence of plants and substances contained in water. Equally to the other factors, evaporation is greater the greater the turbidity of water, as such conditions favour a greater absorption of solar radiation (Rodrigues et al., 2011).

9.2.3 Vegetal factors

The evapotranspiration depends on the albedo of the vegetation, because it is greater as lower its value. The albedo varies depending on the plant species, and within the same species, varies with the state of vegetative development. In general, crops have an albedo of 0.25 in green, but their value tends to lower as crops develop (Rodrigues et al., 2011).

In general, forests have more transpiration than arable crops, and there are also differences between the various arboreal species. Due to differences in the resistance of stomas to the diffusion of water vapor, there are also important differences in the potential evapotranspiration intensities between species with the same albedo and the same height when exposed to the same time state. On the other hand, in the same species, the opening of the Stomas functions as a regulator of evapotranspiration, reducing either in conditions of excessive potential evapotranspiration or in conditions of limited soil moisture (Rodrigues et al., 2011).

When the upper layer of the soil is dry, plants with shallow roots reduce transpiration, however the plants with very deep roots continue to transpire normally. This is a reason why trees have more transpiration than herbaceous plants (Rodrigues et al., 2011).

The density of plant roots can also be important in this aspect, as it is related to the possibility of capturing water to maintain evapotranspiration (Rodrigues et al., 2011).

The resinous, when intercepting more water than the hardwood, increase the evaporation. Moreover, it transpires more because it has a lower albedo, and the leaves have longer duration (Rodrigues et al., 2011).

9.2.4 Soil Factors

The soil influences evapotranspiration either by its albedo, or by its ability to retain and store water, which depends on its texture. Soils of sandy characteristics end up limiting the loss of water because, once dry to the surface, is more easily broken the continuity of water at the pore level and consequently reduced the water loss by evaporation since, the capillary ascension is eliminated (Rodrigues et al., 2011).

The major amount of water stored by clayey soils, ends by favours plant development and evapotranspiration, as well as direct evaporation loss from the soil (Rodrigues et al., 2011).

9.3 Evaporation measurement

The measurement of evaporation can be done through the observation of evaporation in **tins or tank evaporimeters** (tins evaporimeter or evaporimeter tank) or in **atmometers** (evaporimeter of Piche, Livingstone or of Balance) (Rodrigues et al., 2011).

9.3.1 Tin or Tank Evaporimeters

The tins are no more than reservoirs containing water exposed to atmospheric conditions (Figure 83). It can be installed both on the soil surface, as is common in the case of a class A American Tin, buried or still floating in the waters of the reservoirs or lakes. Table 24 presents the main characteristics of the most current and commonly used tins (Rodrigues et al., 2011).

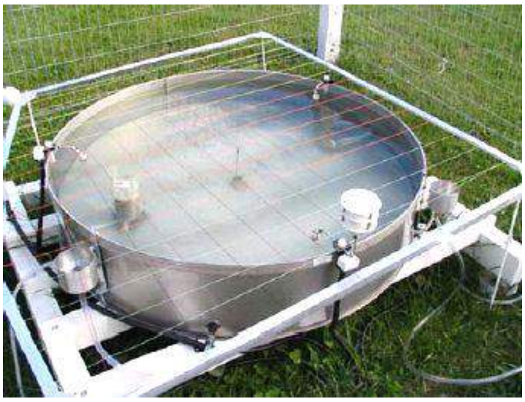

Figure 83 – Evaporimeter (http://www.iginstrumentos.com.ar).

Table 24 – Characteristics of tins (Rodrigues et al., 2011).

Model	Country	Form	Surface (m^2)	Depth (m)	Obs.	$C = E/E_t$
Class A	EUA	Circular	1.167	0.254	-	0.700
Colorado	EUA	Square	0.836	0.457	Buried	0.800
B.P.I.	EUA	Circular	2.627	0.610	Buried	0.920
GGI-300	Russia	Circular	0.300	0.600	Buried	0.820
Balsa 20	Russia	Circular	20.000	2.000	Buried	1.000

The evaporation measured in tins should be affected by a coefficient C, usually called the coefficient of tin or tank. This is a reduction factor in relation to the values measured in the tins, since, given a small water height, the tins receive large amounts of energy by radiation and conduction through the sides and the base, which increases evaporation. On the other hand, the area, which is quite small compared to a lake, increases the evaporation because it is easier to remove, by wind, the saturated air layer at the surface of the water mirror. The very edge of the tub exerts influence on the speed and turbulence of the wind providing a faster removal (Rodrigues et al., 2011).

This reduction coefficient can be defined as the ratio between evaporation in the lake, E, and evaporation in the tub (tins) or tank, E_t:

$$C = \frac{E}{E_t}$$

The determination of the C coefficient can be made through hydrological and energetic balances. Although these methods are difficult to apply, allows the determination of regional values of tins coefficients (Rodrigues et al., 2011).

In some countries, C, values appear published in regional letters for use in the study of small reservoirs. In areas where the coefficients of tin have not yet been determined, average coefficients are usually like those shown in Table 24. In the case of "class A tins" are expected monthly variations of C between 0.6 and 0.8 depending on the seasons. In Portugal, the following values are used for the class A tins: October to November-0.7; December to March-0.6; April and May-0.7 and June to September 0.8. The values measured in the tin can still be distorted by the deficient quantification of precipitation and because it is not avoided, that some animals (mainly birds) drink water from the tin. Sometimes to avoid animals, tins are placed on the metal or plastic nets, sealing the access. However, it should be present that, the placement of such artefacts, ends up conditioning, by the shadow they provoke, the amount of radiation and, therefore, the energy available for evaporation (Rodrigues et al., 2011).

9.3.2 Atmometers

Like the evaporating tins, the atmometers are evaporimeters used for the direct measurement of evaporation. There are several types of atmometers, of which stand out, the Piche, Livingstone and the scale type. Figure 84 illustrates the three types of mentioned atmometers.

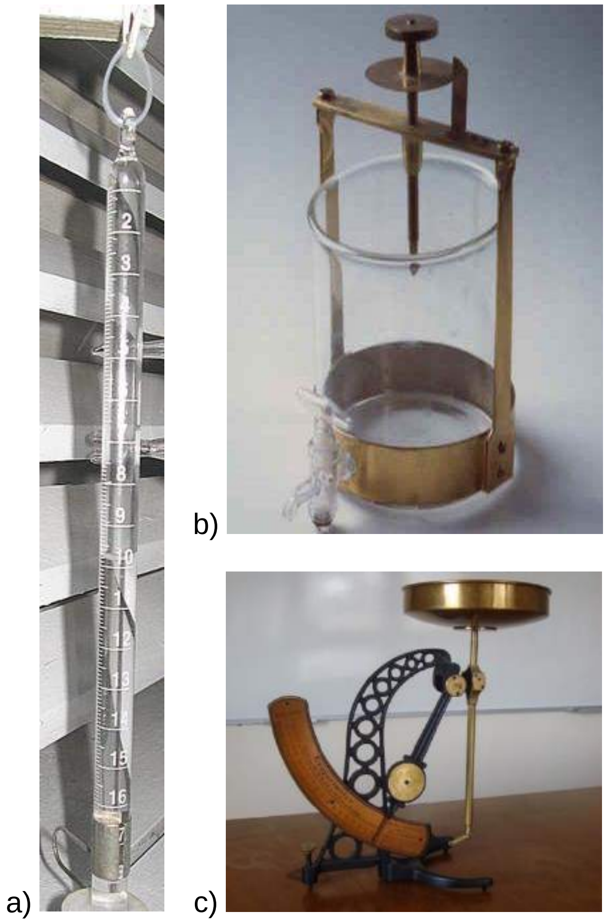

Figure 84 – Atmometers: a) Piche, b) Livingstone, c) Scale (www.google.com).

The Piche Evaporimeter is the most widely used in our country and consists of a porous paper disc powered by a water column. The decrease in the level of the water column allows to evaluate the evaporation as it translates a measure of the evaporating power of the air (Rodrigues et al., 2011).

The evaporation values obtained by the Piche evaporimeter, are compromised by defect when compared to those obtained from the TINA (Figure 85). The ratio between the measurements of evaporation in tins and in Piche evaporimeter, for annual values in mm and valid for the south of Portugal, can be translated by a linear regression as (Loureiro, 1987):

$$E_{tina} = 915.245 + 0.422E_{Piche}$$

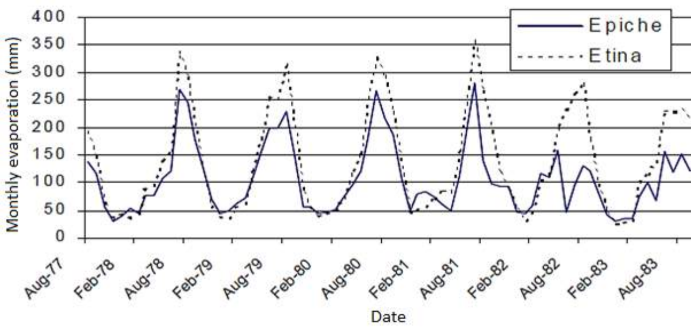

Figure 85 – E_{tin} vs E_{piche} in climatologic station of Divor (Rodrigues et al., 2011).

9.4 Calculus of evaporation through hydrological balance

In order to have a better quantification of evaporation in a given region, and even to control the results obtained by the tins, it is possible, where feasible, to make the balance between all the volumes of tributaries (affluents) and effluent waters or outputs of an existing lake or reservoir, as shows Figure 86 (Rodrigues et al., 2011).

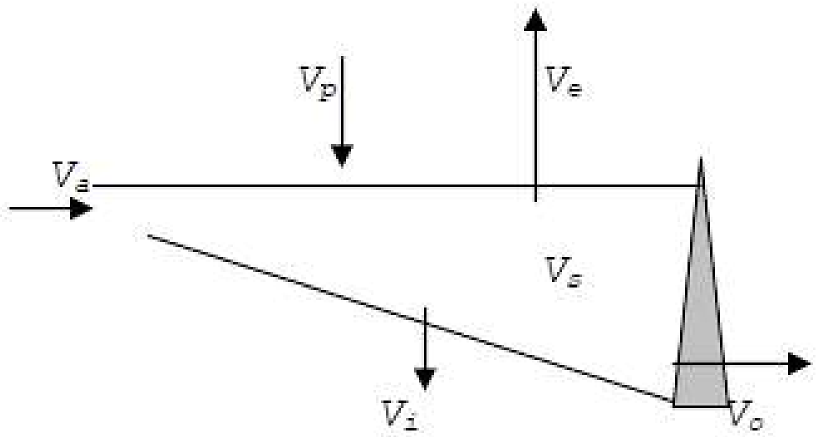

Figure 86 – Scheme of the involved variables in a reservoir balance (Rodrigues et al., 2011).

The balance of mass is expressed as:

$$V_e = \left(V_a + V_p\right) - (V_o + V_S + V_i)$$

as V_e is the evaporated volume, V_a the volume of affluent water coming into the reservoir, V_p correspond to the reported precipitation in the area of the water mirrow, V_o is the efluent volume of water from the reservoir, V_s is the variation of storaged volume of water (positive as increase, and negative as decrease) and V_i, is the infiltrated volume or lost by percolation. All these volumes are easily quantifiable except for V_i, which is generally estimated based on the hydraulic conductivity of the soils of the reservoir. The value of V_e obtained by the preceding equation can be expressed in terms of evaporation relative to the period under analysis, if it divides that volume, by the area, S, of the water mirrow (Rodrigues et al., 2011).

9.5 Measurement of evapotranspiration

Evapotranspiration can be measured directly using devices called evapotranspirometers or lysimeters. These devices allow have a mass balance at the level of an isolated soil block, in which the same culture of the surrounding area is made. The loss of water by evapotranspiration is measured by the difference between the amount of water that flows into the soil of the lysimeter through rain or irrigation, and the one that abandons it, by deep drainage or by superficial flow-drainage lysimeter, illustrated in Figure 87 (Rodrigues et al., 2011).

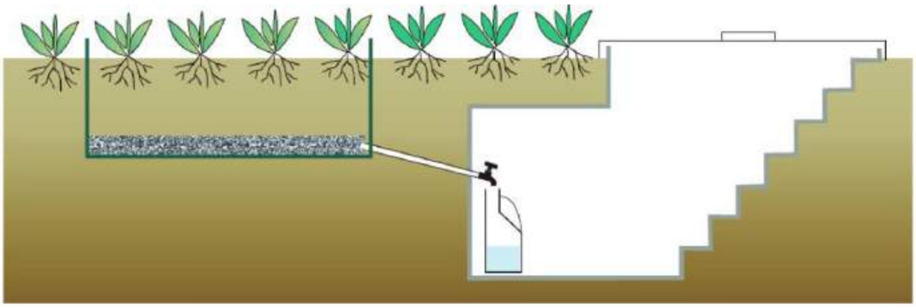

Figure 87 – Transversal cut of the supply system and drainage lysimeter (Varejão-Silva, 2006).

Lysimeters are expensive and difficult-to-maintain facilities so they are normally used only with experimental character for the validation of empirical formulas based on hydrometeorological parameters of easier determination, so, it is generally available (Rodrigues et al., 2011).

9.6 Empirical calculation of evapotranspiration

There are several methods for evaluating evapotranspiration (Figure 88), of which it is important to highlight the Thornthwaite type (Thornthwaite, 1944) or Turc, which will be presented in detail in the subsequent subchapters.

PENMAN	•Semi-empirical formula to evaluate evaporation (water surfaces)
PRIESTLY E TAYLOR	•Formula to evaluate evaporation (water surfaces)
THORNTHWAITE	•Formula to evaluate potential evapotranspiration
TURC	•Formula to evaluate potential evapotranspiration
BLANEY-CRIDDLE	•Formula to evaluate potential evapotranspiration

Figure 88 – Method to evaluate evapotranspiration (Hipólito & Vaz, 2011).

Quintela (1986) states, that the formulas of Thornthwaite and TURC, supplying values that are about 50% and 70% of the evaporation observed in class A tins, seem to lead estimations, respectively, quite by defect and slightly by excess of potential evapotranspiration in Portugal (Hipólito & Vaz, 2011).

9.6.1 Thornthwaite formula

According to Hipólito & Vaz (2011), Thornthwaite (1948) has stablished, for the weather of New Jersey, eastern coast of the United States, the following formula, which is essentially based on the average monthly temperature and which has been widely disseminated in regions where the average monthly temperature is positive:

$$ETP_m = \begin{cases} 16N_m\left(\dfrac{10\bar{T}_m}{I}\right)^a & , \quad \bar{T}_m > 0 \\ 0 & , \quad \bar{T}_m \leq 0 \end{cases}$$

where:
ETP_m - potential evapotranspiration in the month m (mm);
N_m - adjustment factor according to the number of days of a month and related to the astronomical and daily average astronomical insolation in the given month (-);

N_m rounded to two significant digits, it will be:

$$N_m = \frac{H_{0m}D_m}{360}$$

where:
D_m - number of days of the month m (d);
H_{0m} - daily average astronomical insolation at month m (h), defined in Table 25.

Table 25 – Daily average astronomical insolation (h) (Hipólito & Vaz, 2011).

Lat	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dez
60	6.5	8.8	11.5	14.3	16.9	18.3	17.6	15.2	12.4	9.6	7.1	5.6
55	7.6	9.4	11.6	13.9	15.9	17.0	16.5	14.6	12.4	10.1	8.0	7.0
50	8.4	9.8	11.6	13.6	15.2	16.1	15.6	14.2	12.3	10.4	8.7	7.9
45	9.0	10.2	11.7	13.3	14.7	15.4	15.0	13.8	12.3	10.6	9.3	8.6
40	9.5	10.5	11.7	13.1	14.2	14.8	14.5	13.5	12.2	10.9	9.7	9.2
35	9.9	10.7	11.8	12.9	13.8	14.3	14.1	13.3	12.2	11.1	10.1	9.7
30	10.3	11.0	11.8	12.7	13.5	13.9	13.7	13.0	12.1	11.2	10.5	10.1
25	10.6	11.2	11.9	12.6	13.2	13.5	13.4	12.8	12.1	11.4	10.8	10.5
20	10.9	11.3	11.9	12.5	13.0	13.2	13.1	12.7	12.1	11.5	11.0	10.8
15	11.2	11.5	11.9	12.3	12.7	12.9	12.8	12.5	12.1	11.6	11.3	11.1
10	11.5	11.7	11.9	12.2	12.5	12.6	12.5	12.3	12.0	11.8	11.5	11.4
5	11.7	11.8	12.0	12.1	12.2	12.3	12.3	12.2	12.0	11.9	11.8	11.7
0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
-5	12.3	12.2	12.0	11.9	11.8	11.7	11.7	11.8	12.0	12.1	12.2	12.3
-10	12.5	12.3	12.1	11.8	11.5	11.4	11.5	11.7	12.0	12.2	12.5	12.6
-15	12.8	12.5	12.1	11.7	11.3	11.1	11.2	11.5	11.9	12.4	12.7	12.9
-20	13.1	12.7	12.1	11.5	11.0	10.8	10.9	11.3	11.9	12.5	13.0	13.2
-25	13.4	12.8	12.1	11.4	10.8	10.5	10.6	11.2	11.9	12.6	13.2	13.5
-30	13.7	13.0	12.2	11.3	10.5	10.1	10.3	11.0	11.9	12.8	13.5	13.9
-35	14.1	13.3	12.2	11.1	10.2	9.7	9.9	10.7	11.8	12.9	13.9	14.3
-40	14.5	13.5	12.3	10.9	9.8	9.2	9.5	10.5	11.8	13.1	14.3	14.8
-45	15.0	13.8	12.3	10.7	9.3	8.6	9.0	10.2	11.7	13.4	14.7	15.4
-50	15.6	14.2	12.4	10.4	8.8	7.9	8.4	9.8	11.7	13.6	15.3	16.1
-55	16.4	14.6	12.4	10.1	8.1	7.0	7.5	9.4	11.6	13.9	16.0	17.0
-60	17.5	15.2	12.5	9.7	7.1	5.7	6.4	8.8	11.6	14.4	16.9	18.4

$\bar{T}_m$ - monthly average temperature at month m (°C);
I - annual thermal index.

The annual thermal index is calculated as follows:

$$I = \sum_{i=1}^{12} i_m$$

where:
$i_m = (\bar{T}_m/5)^{1.5}$ - monthly thermal index.
a - exponent according to the annual thermal index, given by:

$$a = 6{,}75 \times 10^{-7}\, I^3 - 7{,}71 \times 10^{-5}\, I^2 + 1{,}792 \times 10^{-2}\, I + 0{,}49239$$

9.6.2 Turc Formula

Hipólito & Vaz (2011) refers that Turc (1961) present the next formula usable in a period of ten or more days for weather conditions of West-Europe:

$$ETP = 0{,}013 \frac{T}{T+15}\left(\frac{I'_g}{0{,}042} + 50\right)$$

where:
ETP - potential evapotranspiration (mm/d);
T - average temperature in such period (°C);
I'_g - global average radiation incident on the surface (MJ/m^2/d).

The last formula must be multiplied by the next number when the relative moisture (U in percentage) were lower than 50%:

$$1 + \frac{50 - U}{70}$$

The average global radiation incident on the surface can be calculated as follows:

$$I'_g = I_0(a + b r_H)$$

where:
I_0 - solar radiation in the top of atmosphere (MJ/m^2/d), defined in Table 26;
$r_H = n/H_{0m}$ - ratio of insolation (-), where n represents daily average insolation;
a and b are the coeficcient of Ångstrom.

Table 26 – Daily average solar radiation at the top of the atmosphere (MJ/m²/d) (Hipólito & Vaz, 2011).

Lat	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dez
60	3.48	8.29	16.82	27.36	36.31	40.55	38.33	30.51	20.17	10.60	4.43	2.26
55	6.18	11.29	19.65	29.39	37.25	40.83	38.94	32.14	22.76	13.58	7.21	4.77
50	9.09	14.3	22.34	31.24	38.10	41.11	39.50	33.63	25.19	16.51	10.15	7.58
45	12.11	17.26	24.87	32.89	38.79	41.28	39.93	34.92	27.42	19.35	13.16	10.56
40	15.15	20.15	27.21	34.32	39.28	41.29	40.17	35.99	29.46	22.08	16.16	13.61
35	18.18	22.91	29.35	35.5	39.54	41.09	40.20	36.82	31.27	24.65	19.13	16.68
30	21.14	25.54	31.26	36.43	39.57	40.66	39.99	37.40	32.84	27.06	22.00	19.71
25	24.00	27.99	32.95	37.09	39.34	40.00	39.54	37.72	34.17	29.28	24.77	22.67
20	26.74	30.25	34.38	37.49	38.85	39.10	38.84	37.78	35.24	31.29	27.39	25.52
15	29.32	32.31	35.55	37.61	38.11	37.96	37.89	37.57	36.04	33.07	29.84	28.24
10	31.72	34.13	36.46	37.46	37.11	36.58	36.70	37.09	36.57	34.60	32.11	30.80
5	33.92	35.71	37.09	37.02	35.86	34.97	35.27	36.34	36.83	35.89	34.16	33.18
0	35.91	37.04	37.43	36.32	34.36	33.14	33.6	35.33	36.80	36.91	35.99	35.36
-5	37.66	38.10	37.50	35.35	32.64	31.10	31.72	34.07	36.50	37.66	37.58	37.31
-10	39.17	38.89	37.28	34.12	30.70	28.87	29.64	32.56	35.92	38.14	38.91	39.03
-15	40.41	39.40	36.78	32.63	28.56	26.47	27.37	30.83	35.06	38.33	39.98	40.51
-20	41.4	39.63	36.00	30.91	26.24	23.93	24.94	28.87	33.94	38.25	40.79	41.73
-25	42.12	39.57	34.95	28.97	23.76	21.25	22.36	26.72	32.57	37.88	41.33	42.69
-30	42.57	39.24	33.63	26.81	21.13	18.48	19.67	24.38	30.95	37.24	41.60	43.40
-35	42.76	38.63	32.07	24.47	18.40	15.64	16.88	21.89	29.09	36.33	41.60	43.86
-40	42.71	37.76	30.26	21.95	15.58	12.77	14.04	19.25	27.02	35.16	41.35	44.07
-45	42.42	36.64	28.22	19.29	12.72	9.91	11.19	16.50	24.74	33.75	40.86	44.07
-50	41.93	35.29	25.97	16.51	9.85	7.12	8.36	13.68	22.28	32.1	40.17	43.89
-55	41.29	33.74	23.53	13.63	7.04	4.48	5.64	10.81	19.66	30.25	39.31	43.59
-60	40.60	32.03	20.93	10.70	4.37	2.12	3.13	7.95	16.89	28.23	38.26	43.30

9.7 Concept of crops evapotranspiration

The changes induced in Et_o by a crop's coefficient, k_c, allows to include factors related with crops in order to calculate its water necessities. In practice, crops evapotranspiration Et_c, represents the loss of water due to evapotranspiration of a crop in excellent health conditions, in rapid development and, therefore, capable of producing maximum yields, such that:

$$Et_c = Et_o \cdot k_c$$

The value of k_c is a tabulated value, conditioned for each crop by aspects linked to planting or sowing dates, the rate of development of the crop and the duration of its growing period, depending on the climatic conditions and the frequency of precipitation or irrigation (Rodrigues et al., 2011).

Table 27 presents values for an intermediate stage of development for some crops.

Table 27 – K_c for the intermediate stage of some cultures or crops (Hipólito & Vaz, 2011).

Crops	$K_{c,mid}$
Celery	1,05
Lettuce	1,00
Cotton	1,15-1,20
Rice	1,20
Banana	1,20
Potato	1,15
Sweet potatoes	1,15
Aubergine	1,05
Beet	1,20
Sugar cane	1,25
Onion	1,00
Carrot	1,05
Citrus fruit	0,65
Spinach	1,00
Sunflower	1,00-1,15
Flax	1,10
Manioc	1,10
Corn	1,20
Soy	1,15
Tomato	1,15
Wheat	1,15

9.8 Calculus of real evapotranspiration

It has already been mentioned that real evapotranspiration, Et_r, corresponds to the amount of water truly lost by the soil, depending on its moisture content, atmospheric conditions and vegetation characteristics (Rodrigues et al., 2011).

The method of calculation of Et_r is **"the water balance method"**. Thus, whenever the amount of rainfall, P, in a given period of time, i, exceeds the potential evapotranspiration or reference given for the same period, the value of Et_r is the equal to value Et_o (or Et_p). This occurs under our conditions in the wet season (Rodrigues et al., 2011). Then:

$$Et_{r,i} = Et_{o,i} \quad se \;\; P_i > Et_{o,i}$$

In dry period (precipitation values below evapotranspiration in the period), the amount of water retained in the soil under usable conditions by the crops should be considered. Thus, the value of actual evapotranspiration should be determined by:

$$Et_{r,i} = P_i + |\Delta H_i| \quad se \;\; P_i < Et_{o,i}$$

where ΔH_i corresponds to the change in the usable reserve of the soil (mm) obtained by:

$$|\Delta H_i| = H_i - H_{i-1} \quad se \;\; P_i < Et_{o,i}$$

where, i, represents the interval of calculus of balance (Day, decade, month) and, H the usable reserve (Rodrigues et al., 2011).

CHAPTER 10 - WATER IN SOILS

10.1 Introduction

The transformation of precipitation into flows acquires its own characteristics at each location, due to the climate and the different geological formations that manifest themselves on the surface of the Earth's crust at such local. Effectively, the weathering of rocks, by disintegration, mechanical or chemical decomposition, transportation of the resulting particles by currents of air or water and sedimentation, shape the crust surface over time, defining the drainage network of basins, the nature of slopes and the soil types (Hipólito & Vaz, 2011).

The soils act in the terrestrial phase of the hydrological cycle as reservoirs of regulation, controlling the supply of aquifers and delaying the descent of water that penetrate them, and therefore, dampening the flow drained from its surface (Hipólito & Vaz, 2011).

The soil is formed by materials that present in three thermodynamic states or phases: solid, liquid and gaseous; thus, the soil can be defined as a heterogeneous, multiphase, particulate, disperse, porous and anisotropic physical system (Hipólito & Vaz, 2011).

10.2 Amount of water in the soil

The amount of water contained in a soil is generally assessed in relation to the solid phase or in relation to the whole soil. In Figure 89, where each of the phases of a soil column is considered to be agglutinated, the quantities allowing for the definition of these relative quantities are shown(Hipólito & Vaz, 2011).

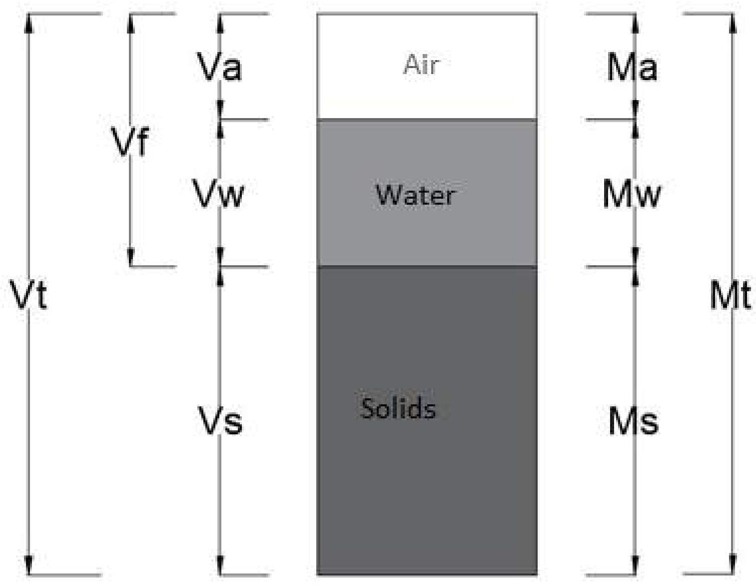

Figure 89 – Represent a column of soil with all agglutinated phases (adapted Hillel, 2004).

On the left side of the soil column states, the volume of air, V_a, the volume of water, V_w, the volume of solids, V_s, the volume occupied by fluids, V_f, and the total volume, V_t. On the right side of the soil column states, the air mass, M_a, practically negligible due to the small mass of the air, the mass of water, M_w, the mass of solids, M_s, and the total Mass, M_t (Hipólito & Vaz, 2011).

It is designated as **Volumetric amount of moisture (θ)** the ratio between the volume of water of a soil portion and its total volume:

$$\theta = \frac{V_w}{V_t}$$

It is designated as **mass amount of moisture (w)** the ratio between the water mass of a soil portion and the mass of the solid phase of such portion:

$$w = \frac{M_w}{M_s}$$

It is designated as **degree of saturation (S)** the ratio between the volume of water of a portion of soil and the volume in this portion is occupied by fluids:

$$S = \frac{V_w}{V_f}$$

It should be noted that, the **porosity of a soil (n)** is defined by

$$n = \frac{V_f}{V_t}$$

Thus:

$$\theta = n \cdot S$$

In other words, the moisture content is equal to the product of the porosity by the degree of saturation (Hipólito & Vaz, 2011).

Other relations of interest regarding the water content of a soil are:

$$n = 1 - \frac{\rho_d}{\rho_s};\ \theta = \frac{\rho_t - \rho_d}{\rho_w}; w = \theta \frac{\rho_w}{\rho_d}$$

where:
$\rho_d = M_s/V_t$ - apparent volume of dry soil (kg/m^3);
$\rho_s = M_s/V_s$ - density of soil solids (kg/m^3);
$\rho_t = (M_s + M_w)/V_t$ - apparent volume of the soil (kg/m^3);
$\rho_d = M_w/V_w$ - water density (kg/m^3).

It is called **field capacity (θ_{cc} or w_{cc})** the final value of the moisture content of a natural soil with uniform characteristics, which has been saturated and allowed to drain freely for two to three days, i.e. the moisture content corresponding to the quantity of residual water that a soil is capable to retain against the prolonged action of gravity (Hipólito & Vaz, 2011).

10.3 Soil water potentials

The concepts already presented, although necessary, are not enough to strictly characterize the water state of a soil. Water in a soil may be in equilibrium or in movement in a given direction and according to a defined rate, for which it is fundamental to know also its energy state (Rodrigues et al., 2011).

The kinetic energy, being proportional to the square of the velocity, assumes little importance considering the reduced velocities of water displacement in the soil. The potential energy, on the contrary, has significant importance since it depends on the position and the internal condition of the water (Rodrigues et al., 2011).

The total potential Ψ is a measure of its potential energy. In the measurement of energy between different states, it is usual to consider a standard state of null energy, in which, for water corresponds to the state of pure water, subjected to normal conditions of pressure and temperature when placed in a given location of a gravitational field. The total water potential also represents the necessary work to bring the water from the standard state to the considered state; such potential can be decompose into several components: Pressure component, Ψ_p, gravitational component, Ψ_g, and osmotic component, Ψ_O (Rodrigues et al., 2011).

$$\Psi = \Psi_p + \Psi_g + \Psi_O + \cdots$$

Underground flows hydraulics are based on Darcy's law and the continuity equation. The conjugation of these two laws allows to establish differential equations of the underground flow (Hipólito & Vaz, 2011).

10.4 Movement of water in soils

Water moves in the direction that allows it to occupy the state of the lowest total potential. That movement is governed by Darcy's law, which for vertical motion is expressed by:

$$\mathrm{q} = -k(\theta)\frac{\partial \Psi}{\partial Z}$$

where q represents the flow (cm/s), $k(\theta)$ the hydraulic condutivity (cm/s) and $\partial\Psi/\partial Z$ represents the gradient of total potential (Rodrigues et al., 2011).

The water flow represents the amount of water (cm^3) that passes through the unit of soil area (cm^2) per unit of time (s). The potential gradient translates the variation of the total potential of water along the Z direction. The conductivity is a characteristic coefficient of proportionality of each soil, which value is a function of humidity (major as more moistures the soil is capable to hold) being saturation maximal (Rodrigues et al., 2011).

10.5 Infiltration

10.5.1 Introduction

The entry of water into a soil by its surface is a phenomenon that is generally referred as **infiltration** (Hipólito & Vaz, 2011).

The specific flow rate of water (flow per unit area in a plain) that crosses the terrain surface of a soil is referred to as **infiltration intensity** and has the dimensions of a velocity. The volume of water that per unit area of a plain crosses the soil surface in a given period of time is called **accumulated infiltration** and has the dimensions of length (Hipólito & Vaz, 2011).

The infiltration of water into a permeable soil depends essentially on three factors: the availability of water on the soil surface, the hydraulic characteristics of the soil and the water content in the soil. Effectively, when there is no water on the surface of the soil, there can be no infiltration, and as greater the availability of water on the surface, as greater the infiltration; if the soil is impermeable to water, no infiltration will occur; if the soil is saturated and there is no drainage at the bottom, there will also be no infiltration (Hipólito & Vaz, 2011).

Infiltration capacity of a soil is the infiltration that occurs when at the surface of a soil is available, and maintained over time, a thin film of water. In this thin film, water is evidently at atmospheric pressure. To specify the infiltration intensity, it is said that this occurs at the soil capacity (intensity of infiltration at soil capacity). To specify the accumulated infiltration, it is said accumulated infiltration at soil capacity (Hipólito & Vaz, 2011).

10.5.2 Infiltration and Superficial flow

The infiltration rate is defined as the volume of water flow moving in the soil profile per unit area. This flow has units of velocity because, translates the velocity with which the water crosses a soil from the surface. The maximum infiltration rate defines the soil infiltration capacity in the study (Rodrigues et al., 2011).

When the process is controlled by the flow, the infiltration is determined by the rate of water application. However, if the application rate exceeds the infiltration capacity, such capacity determines the current rate of infiltration and the process is controlled by the profile (case of submersion) (Rodrigues et al., 2011).

Generically, it can be said that the infiltration capacity begins to be elevated at the beginning of a rainy (in particular if the soil is dry) and tends to decrease, approaching asymptotically of a value that corresponds to the final infiltration rate, which is also designated as permanent rate of infiltration, as illustrated in Figure 90 (Rodrigues et al., 2011).

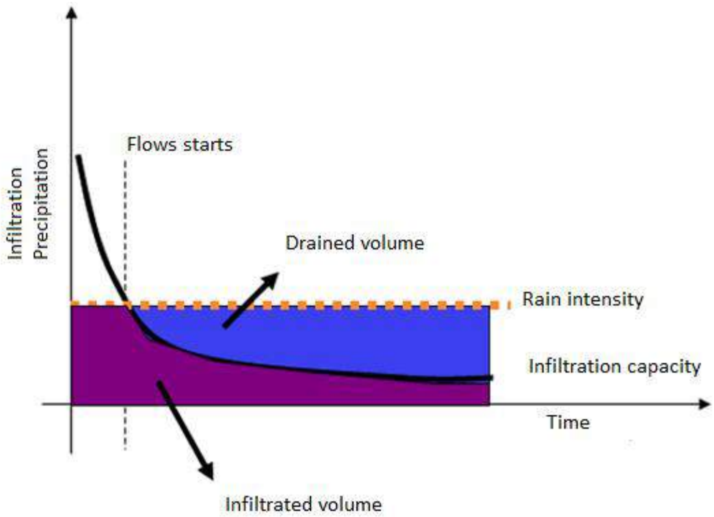

Figure 90 – Precipitation, superficial flow and infiltration during a constant rain (Andrade, 2014).

The decrease of infiltrability with time, mainly results from a decrease of the matric potential gradient. The higher the hydraulic conductivity of the soil is, higher is its infiltration capacity (Rodrigues et al., 2011).

If we observe a profile of a homogeneous soil during infiltration, under conditions of flooding (Figure 91), it is possible to see that even in a small depth, the soil is saturated and down this zone there is an apparently uniform humidity transmission zone. The mentioned is a dampening zone in which the moisture decreases in depth until the front of the moistening that constitutes the furthest area from the surface (Rodrigues et al., 2011).

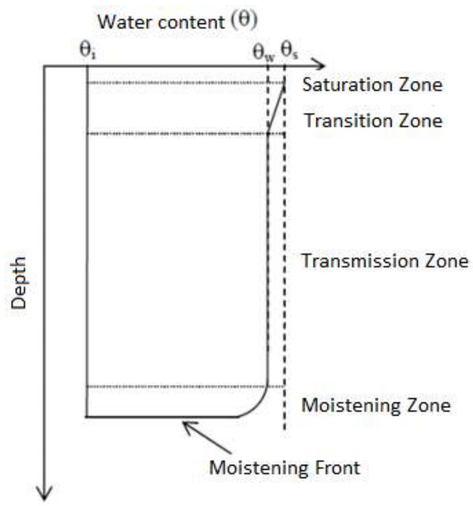

Figure 91 – Distribution of moisture in a soil profile (θ_i is the initial soil moisture and θ_S is the saturation moisture of a soil) (Cecílio, Martinez, Pruski, & Silva, 2013).

There are numerous models and empirical formulas for the calculation of infiltration, of which some of the most frequent are presented in the subsequent subchapter.

10.5.3 Models of infiltration

In Hipólito & Vaz (2011) the following models are highlighted:

Horton

The distribution of rainwater between the amount that is infiltrated and the surface flow, depends on:

- Soil infiltration capacity (f);
- Precipitation intensity (i).

When a rainy day occurs, admitting that it has not rained for some time, the infiltration capacity decreases from a maximum value at the beginning (initial infiltration intensity is the same as soil capacity), f_0, and tends towards to the limit, f, corresponding to the saturation of the soil (infiltration intensity at the end of a long time, i.e. the hydraulic conductivity of a saturated soil). Horton (1939) proposed the following law of variation for infiltration, as a function of time:

$$f = f_c + (f_0 - f_c)e^{-kt}$$

where:
k - temporal scale factor (-);
t - time counted from the beginning of the precipitation (h).

Philip

Philip's Model (1957) uses two parameters, S and K_S. The first one has been called sorbability, and the second is the hydraulic conductivity of a saturated soil. In the initial instant, as in the Green and Ampt model, the intensity of infiltration at soil capacity is infinite.

$$f = \frac{1}{2}St^{-\frac{1}{2}} + K_S$$

Kostiakov

The Kostiakov Model (1932) also uses two parameters, α and β. When comparing with Philip's model, it is acknowledged that the α value should be about 1/2, of about half of the sorbability. The Kostiakov model is widely used in Portugal, connected to an irrigation study, eventually, with the hydraulic conductivity of the saturated soil added to the infiltration intensity.

$$f = \beta t^{-\alpha}$$

CHAPTER 11 - GROUNDWATER

11.1 Introduction

Groundwater resources have always played an important role and should continue in such way, in the supply of populations as also in the source of water for agriculture and industry. Those resources almost always constituted the first origins of water, being on such way in many regions until recently, and keeping it still in others. Even in vast areas where groundwater is scarce, it can be fundamental in the absence of other economically mobilizable water resources, by enabling the supply of small urban or industrial centres, agricultural holdings and the irrigation of small farms (SNIRH, 2018).

11.2 Groundwater reservoirs

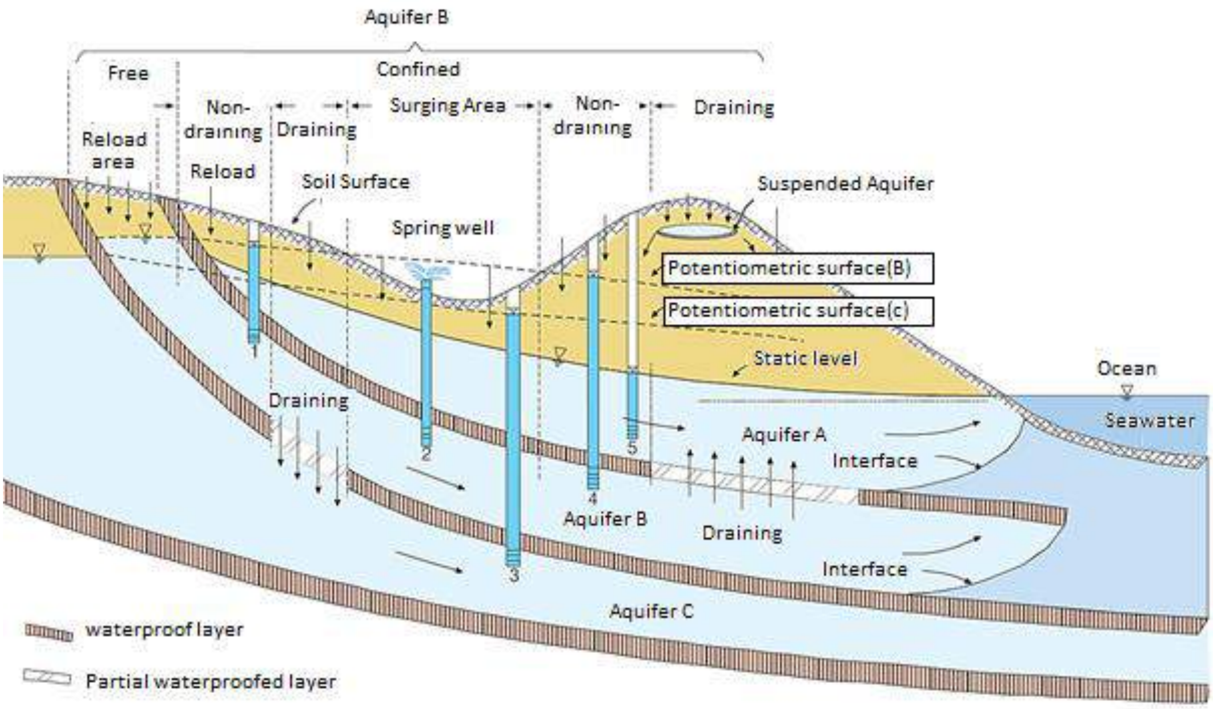

Figure 92 – Schematic representation of different types of aquifers (CPRM, 2008).

Groundwater is found in geological formations known as **aquifers** (Figure 92), that occupies empty spaces in rocks, namely the intergranular pores, which are canaliculi that interconnect each other and the fractures.

An **aquifer** is a geological formation that stores and enables the circulation of water, and where it is possible to extract it, in enough quantities in order to enable its exploitation by man. For its exploitation, aquifers should not only store water, but also permits its circulation. They are erroneously designated by expressions such as water tables, groundwater and aquifer. The enough quantity varies from region to region: in semi-arid climates, it is considered economically feasible a capture (borehole, well or gallery) that provides flows considered derisive in another, such as 0.5 l/s.

If the geological formations are not aquifers (aqua + Fero = Bring water), i.e., geological formations whose physical conditions prevent storage, and soon the circulation of water, are named as **Aquifuges**. As it is the case of the non-fractured magmatic rocks-granites, gneisses, basalts and very compact limestones.

Aquitard (*tardare* = delay) is a geological formation that stores water but transmits it slowly, producing small amounts of water. They present medium/low permeabilities - cases of clayey sands and altered volcanic rocks. They are important for the recharge of the proximal aquifers.

Aquiclude (*claudere* = close) is a geological formation that can store water but does not transmit it - water does not circulate - cases of clays and Tufts (consolidated pyroclasts). They function as a confining or impermeable layer.

11.2.1 Classification of aquifers

Aquifers can be classified in three ways:

- due to its geological structure: porous, fissured and karst;
- due to its geographic location: coastal and internal, basic and suspended;
- due to the pressure to which it is subjected: free or phreatic, confined or captive.

Geological structure

Porous or sedimentary aquifer: formed by consolidated sedimentary rocks, unconsolidated sediments or sandy soils. The water circulation is made in the pores between the grains of sand, silts and clay. They occur in large areas and have a large volume of water, which are considered the most important aquifers. These types of aquifers occur in sites of accumulation of sandy sediments such as large sedimentary basins. They have homogeneous porosity that causes water to flow in any direction defined by the difference in hydrostatic pressure (Winck, 2015).

Fractured or cleft aquifer: formed by igneous, metamorphic or crystalline rocks, hard and massiveness. The water is found in this type of aquifer, in fractures and failures formed due to the tectonic movement. The amount of water is related to the number of fractures and the flow is given through the existing connections and communications (Winck, 2015).

Karst Aquifer: Formed in limestone or carbonate rocks. Water circulation occurs in fractures and other discontinuities resulting from the dissolution of the carbonate in water. These discontinuities can have large openings and form underground rivers that reach large dimensions. These are aquifers that present high heterogeneity and hardness. The rocks that make up this type of aquifer are the limestone, Dolomites and marbles (Winck, 2015).

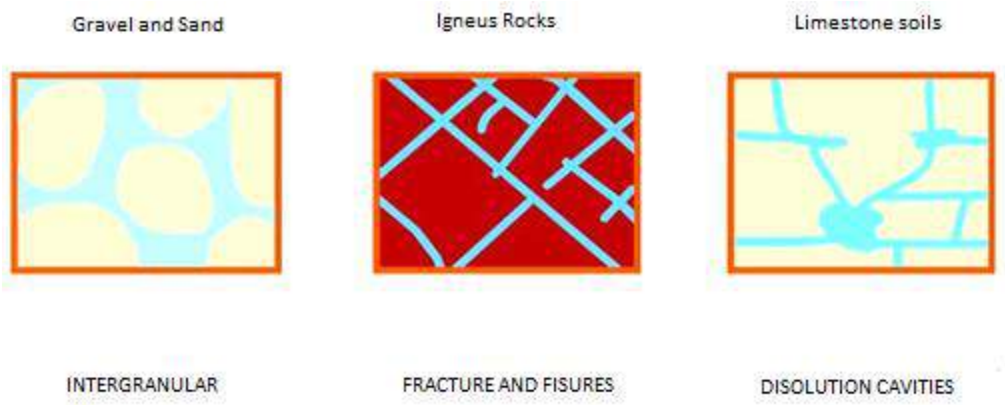

Figure 93 – Circulation of water in porous media, fractured and karsic (Ciência Viva, 2006).

In most cases, an aquifer system is simultaneously composed from more than one type. Volcanic regions are usually formed by alternation of cleft and pyroclastic and may also contain lava tubes. Granites can have a very altered upper area where circulation is made through pores and a lower zone of healthy rock where the circulation is made by fractures. Limestone can be karst and fissures circulating water through cracks in the rock, in pipelines or underground rivers.

Geographic position and pressure

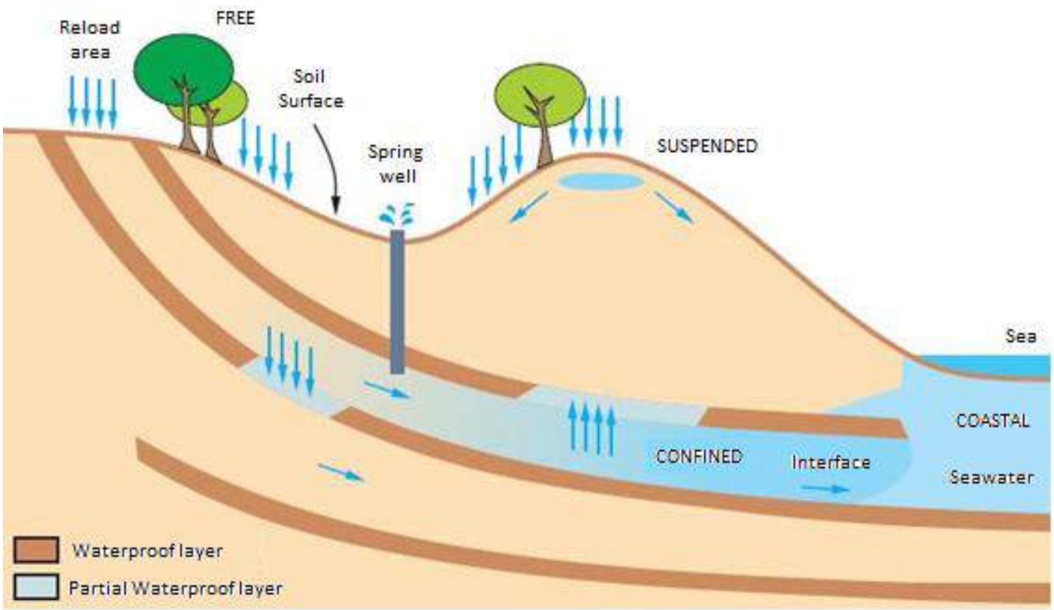

Figure 94 – Types of aquifers: Free, suspended, confined and coastal (Carneiro, 2007).

A **free aquifer** is found with a permeable superior extract, being inferiorly limited by a permeable or semi-permeable rock. Such are generally in small depths, almost always limited by its own surface or by the water accumulation limit. This type of aquifer is the easiest for extraction of water resources and is often called a phreatic aquifer.

Suspended aquifers are a case of a free aquifer, formed by a waterproof bottom base and a permeable or semi-permeable superior base, without the ability to transmit, accumulate or receive more water.

The **confined aquifers** are those surrounded by impermeable layers and kept under an internal pressure superior to atmospheric pressure. When drilled, such wells usually flow water at reasonable speed due to this higher pressure.

Despite the heterogeneities and discontinuities that characterize the volcanic media, there is in Madeira at a certain depth, a level of regional saturation or **base aquifer** (Figure 95) with distinct characteristics depending on the volcanic complex (Prada, et al., 2005).

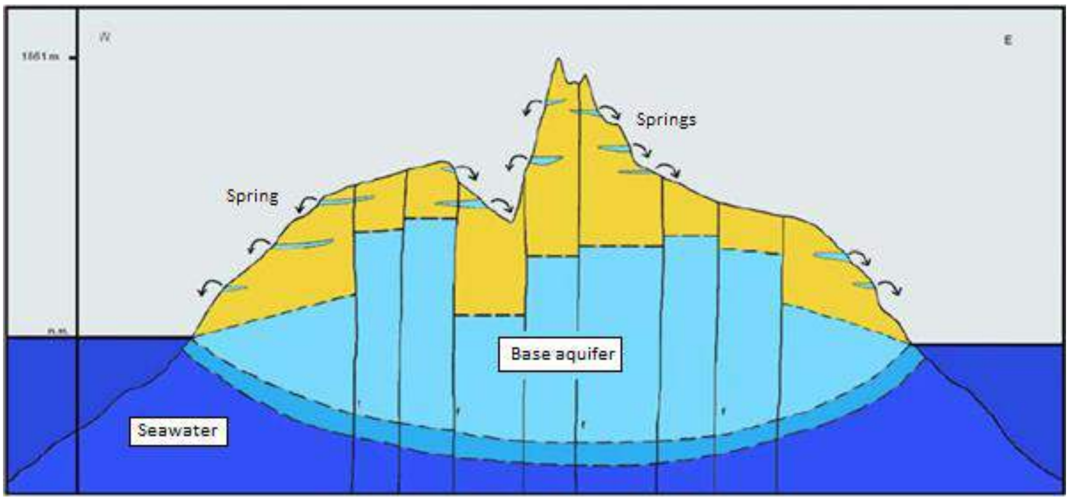

Figure 95 – Base Aquifers (Prada, et al., 2005).

11.2.2 Fundamental hydrodynamic parameters of aquifers

Based in Hipólito and Vaz (2011), then:

Table 28 – hydrodynamic parameters.

Type	Designation
Capacitive function	Porosity (n) and effective porosity (n_e) Coefficient of storage (S)
Transmissive function	Hydraulic conductivity (K) Transmissivity (T)
Piezometry and gradient	Piezometric level (h) Hydraulics coefficient (i)
Flow rate and speed	Flow rate or flow rate (Q) Effective speed of circulation (v_e)

Porosity

Property of soils and rocks of possessing pores or cavities. In the saturated zone the groundwater fills all the voids, so the **porosity (n)** is a direct measure of the water contained by a unit of volume. It is a dimensionless parameter and is usually expressed as a percentage (%):

$$n = \left(\frac{V_v}{V_t}\right) \times 100$$

where:
V_v - volume of voids (cm^3 or m^3);
V_t - total volume occupied by the soil or rock (cm^3 or m^3).

It is usually distinguished between primary porosity and secondary porosity:

- **Primary porosity**: characteristics intrinsic to rock formation(genesis);
- **Secondary porosity**: originated by subsequent processes to its formation, such as fracturing, cavities by dissolution, etc. Secondary factors can act in both directions, increasing or decreasing the volume of voids (clogging).

The porosity depends on the size, shape, arrangement and homogeneity of the grains:

- **Size**: as smaller as granulometry (grain size), although the pore volume is smaller, the sum of all pores gives to a rock a higher porosity.
- **Shape**: the shape of a particle can vary from very angular to very rounded. The porosity is higher in the case of particles being very angular, because it is not able to compress as well as those in a more spherical form.

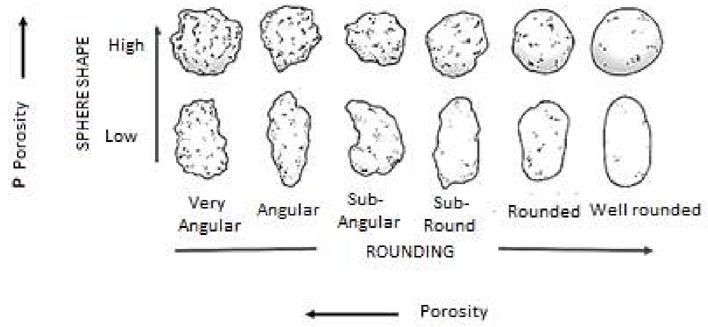

Figure 96 – Form of the particles (Sansone, 2014)

- **Arrangement**: for the same dimension and shape, the various porosity with the geometry of the arrangement of the grains.
- **Homogeneity**: if the grains are of different sizes, the porosity tends to be smaller than in a case of uniform grains, since the smaller grains occupy the empty spaces between the largest.

The **effective porosity (n_e)** or **specific yield** corresponds to the free water, which is not linked to the grains of sand by adhesion, cohesion or capillarity forces. It is the quotient between the volume of voids available for a flow, or volume of drained water (V_e) and the total volume (V_t):

$$n_e = \left(\frac{V_e}{V_t}\right) \times 100$$

Since the entire water contained in the volume of voids is not available for disposal, the porous medium retains water against the gravitational attraction.

The **specific retention** ($\boldsymbol{n_r}$) correspond to water retained in soil against gravity forces, which correspond to the field capacity. It is the quotient between retained volume of water (V_r) and the total volume (V_t):

$$n_r = \left(\frac{V_r}{V_t}\right) \times 100$$

then:

$$n = n_r + n_e$$

Coefficient of storage (S)

Corresponds to the volume of water ceded by an aquifer column of unitary area when the piezometric level descends to a unit (dimensionless): $S = m^3/(m^2.m)$. In the case of free aquifers, the storage coefficient (S) corresponds to the effective porosity(n_e), since the water extraction corresponds, in reality, to a pore emptying ($S = n_e$ and varies between 0.01 and 0.3). In confined aquifers, the extracted water corresponds to the deformation of an aquifer, that is, water and pores (in such cases, S, varies between 10^{-3} and 10^{-6}).

Permeability or Coefficient of permeability. Hydraulic conductivity

Permeability or Coefficient of permeability ($\boldsymbol{K}$) is not an intrinsic characteristic of the medium, because it depends on the fluid that crosses the formation ("Normal" water, hot water, oil, gas, air, etc.), which is the reason that it is more correct to designated as conductivity, when applied to "normal" waters. **Hydraulic conductivity** is the major or minor easiness with which an aquifer allows to be cross by water, and is expressed in m/d, cm/s or m/s. It is measured in the field through flow tests and in laboratory with assistance of permeameters and empirical formulas.

The hydraulic connection between voids is fundamental, so that a high porosity can correspond to a high permeability. It is very common to relate the hydraulic conductivity with porosity, which is not always correct: a porous rock can have a high hydraulic conductivity if its pores are large and well interconnected (e.g. sand and gravel) or have an almost null conductivity, if its pores are too small and strongly retainers of water (e.g. clays).

Table 29 – Porosity vs Permeability.

Type of rock	Porosity (%)	Permeability (m/d)
Gravel	30	> 1000
Sand	35	10 a 5
Clay	45	< 0,001

Transmissivity

Corresponds to the capacity of an aquifer to transmit water throughout its saturated thickness, expressed in m^2/day:

$$T = K \times b$$

where:
K - hydraulic conductivity (m/d);
b - saturated thickness of the aquifer (m).

Transmissivity is the parameter that conditions at most, the flow of groundwater, and therefore, it is the most used hydrodynamic parameter in the evaluation of groundwater resources. It is determined *in situ* through flow rate tests in extraction works.

11.3 Darcy's Law

Darcy's law is one of the fundamental equations for the study of underground flows hydraulics. It establishes that water flow through a homogeneous and isotropic saturated porous medium is proportional to the gradient of hydraulic potential or hydraulic gradient, Figure 97 (Hipólito & Vaz, 2011).

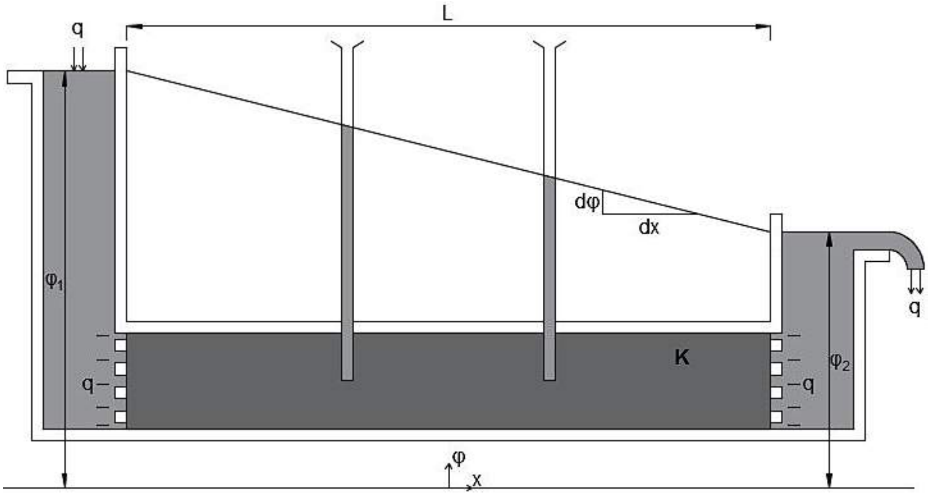

Figure 97 – Laboratory experience to show Darcy's law (adapted form Hipólito & Vaz, 2011).

Such law is expressed by:

$$v = K\frac{\varphi_1 - \varphi_2}{L}$$

where:
v - flow velocity (m/day);
φ - hydraulic potential or charge (m) and $\varphi_1 - \varphi_2$ is the loss of charge (m);
L - distance between points were charges φ_1 and φ_2 were measured (m);
K - proportionality constant (m/day).

As the velocity value is very low, the kinematic load component can be ignored, then, φ is equal to the Piezometric dimension. The value of $(\varphi_1 - \varphi_2)/L$ is the **hydraulic gradient** $i = grad(\varphi)$ and is dimensionless. Then:

$$v = -Ki$$

It can be appreciated, that the proportionality constant has the dimensions of a velocity (m/day). In fact, this proportionality constant is **permeability (*K*).** When in the experimental device is introduced thicker sand, with greater permeability, a higher flow rate is recorded. The negative sign in the previous equation expresses that the flow is in the opposite direction to the positive direction of the hydraulic gradient (Hipólito & Vaz, 2011).

v represents an **apparent speed of filtration.** The effective flow velocity is greater than v, because part of the flow section is occupied by solid particles, with only the pores available for the flow. Given the irregularity of dimensions and distribution of pores, it is only possible to define an average **effective flow velocity**, through:

$$v_e = \frac{v}{n_e}$$

Being n_e the **effective porosity of the medium**, that is, the portion of pores used by underground flow. For gravel, sand, silts and clay, the value of effective porosity is approximately equal to the specific yield, S_y (is the ratio between the volume of water drained by gravity in an initially saturated soil and the total volume of such soil) (Hipólito & Vaz, 2011).

It's important to know the validity limits of Darcy's law. As it is known from fluid mechanics (Quintela, 1981), in the laminar flow, the average flow velocity is directly proportional to the hydraulic gradient, as in Darcy's law. Therefore, it is assumed that Darcy's law is valid for laminar flows in porous medium (Hipólito & Vaz, 2011).

From Darcy's law, the **specific flow rate** (flow per unit of aquifer width) can be expressed by the formula:

$$q = -KHi$$

In which the specific flow rate q is expressed in m^2/day. The hydraulic gradient depends on the orientation. Darcy's law can be written in a generalized way:

$$v_x = -K_x i_x$$

where:
v_x - flows velocity in the direction of x (m/day);
K_x - permeability in the direction of x;
$i_x = \frac{\partial \varphi}{\partial x}$ - hydraulic gradient in the direction of x.

Analogous expressions can be written for v_y and v_z. It should be noted that in the case of an isotropic aquifer it will be $K_x = K_y = K_z = K$ (Hipólito & Vaz, 2011).

11.3.1 Stratified Deposits

It is important to consider a situation of a **stratified deposit**, in other words, a deposit composed of parallel layers, each homogenous and isotropic, but with differences of permeability between the various layers. The stratification can be verified in a direction perpendicular to the flow or in a direction parallel to the flow (Hipólito & Vaz, 2011).

Stratification in a direction perpendicular to the flow

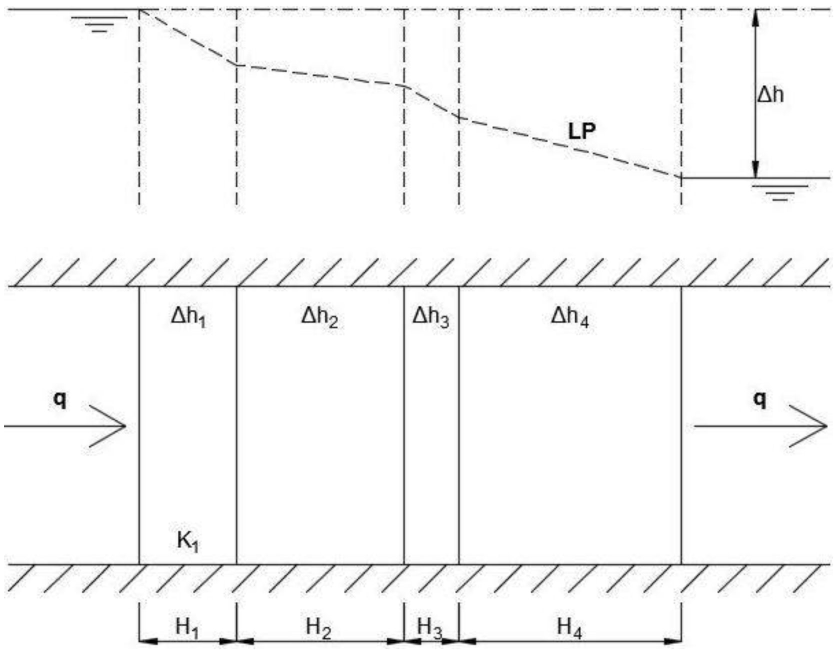

Figure 98 – Example of a flow in a stratified aquifer with a perpendicular direction to such flow (source: author).

In this case, the flow and velocity are equal in all crossed layers, according to the scheme of Figure 98 (Hipólito & Vaz, 2011). For each of the layers j can be written as:

$$v = -K_j i_j = -K_j \Delta h_j / H_j \Rightarrow \Delta h_j = -v H_j / K_j$$

The loss of total flow load when crossing various layers, Δh, is calculated as:

$$\Delta h = \sum_j \Delta h_j = -v \sum_j (H_j / K_j)$$

For the whole aquifer, with thickness H and in which the flow is processed with the loss of total load Δh, it is possible to write Darcy's law as:

$$v = -K_{eq} i = -K_{eq} \frac{\Delta h}{H} = -K_{eq} \sum_j \Delta h_j \Big/ \sum_j H_j$$

where K_{eq} is the equivalent permeability of the aquifers:

$$K_{eq} = K_v = \frac{H}{\sum_j \frac{H_j}{K_j}} = \frac{H}{\sum_j c_j}$$

Being c_j the **hydraulic resistance** of layer j (days). It can then be treated a stratified aquifer in a perpendicular direction to the flow as an isotropic homogenous aquifer with permeability $K_{eq} = K_v$ (Hipólito & Vaz, 2011).

Since the hydraulic resistance of the entire aquifer in the vertical direction is equal to H/K_v, it can also be concluded that for a flow through a stratified aquifer, the hydraulic resistance equal to the sum of the hydraulic resistances of crossed layers (Hipólito & Vaz, 2011).

Stratification parallel to flow

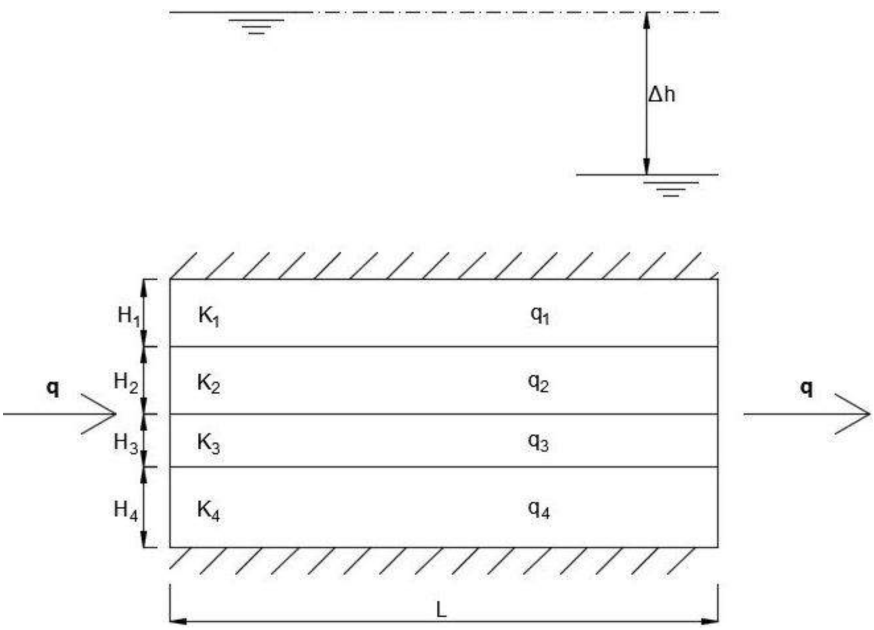

Figure 99 – Example of a flow in a stratified aquifer with layers parallel to the flow direction (source: author).

In this case, the gradient of the underground flow is the same in all layers, according to the scheme of Figure 99 (Hipólito & Vaz, 2011). For each j layers, it can be written:

$$q_j = -K_j H_j i = -K_j H_j \frac{\Delta h}{L}$$

where q_j is the specific flow rate of each layer j. The total flow rate q that cross all layers is given by:

$$q = \sum_j q_j = -\sum_j (K_j H_j) \frac{\Delta h}{L}$$

For all the aquifer complexion with total thickness H and total flow rate q, it is processed with the same total load or charge loss Δh, and the Darcy's law can be written as:

$$q = -K_{eq} \sum_{j} H_j \frac{\Delta h}{L}$$

where K_{eq} is the equivalent permeability of the whole aquifer:

$$K_{eq} = K_h = \frac{1}{H} \sum_{j} \left(K_j H_j\right)$$

A stratified aquifer can be treated in the direction parallel to the flow as an isotropic homogeneous aquifer with permeability $K_{eq} = K_h$. Since the **transmissivity** of the entire aquifer in the horizontal direction is equal to $K_h H$, it may also be concluded that the transmissivity of a stratified aquifer in the flow direction is equal to the sum of the transmissivities of the layers (Hipólito & Vaz, 2011).

In a stratified aquifer, exists marked differences in flow of more permeable (aquifers) and less permeable (aquitard) layers. The horizontal flow, in the direction parallel to the stratification, it is practiced in almost all the aquifers, being negligible the flow that passes through aquitards (Hipólito & Vaz, 2011).

On the other hand, a vertical flow is a very small fraction of the horizontal flow. Generally, the vertical flow in the aquifers can be neglected in relation to the horizontal flow. Aquitards contribute almost in all the vertical hydraulic resistance, while in general, the vertical hydraulic resistance in aquifers can be neglected. Thus, there is almost no loss of charge in aquifers with vertical direction. This is an advantage of installing piezometers to record piezometric levels because the depth of the filter is not critical (Hipólito & Vaz, 2011).

11.4 Groundwater catchment

Groundwater abstraction averages any device which allows the extraction of water contained in an aquifer system, whether by gravity, pumping or any other lifting system:

- Water holes;
- Galleries;
- Springs;
- Wells.

According to ARM (2018), the Madeira water supply and management system comprises a series of systems and infrastructures for capturing, producing, treating, transporting, discharging and hydroelectric exploitation. In the region, the main infrastructures are:

- 4 water catchment galleries;
- 23 water catchment holes;
- 18 other sources of water (surface springs/catchment);
- water treatment plants;
- 23 chlorination stations;
- 2 hydroelectric power plants (hydro);
- 24 lifting stations;
- 1 storage pond;
- 50 storage reservoirs;
- 235 km of water mains network;
- 125 km of distribution network.

11.4.1 Water holes

Figure 100 – Water hole (view from surface) (ARM, 2018).

The capture of deep groundwater is made through holes, not reaching depths greater than 300 m. The creation of a hole must be preceded by an adequate hydrogeological characterization of the zone, this to ensure the success of works and avoid wasting financial resources.

After drilling, a piping must be introduced to protect the hole walls. Up to a depth of not less than 3 m, the space between a pipeline and terrain must be filled with cement mortar with waterproofing additive or crushed clay. In the production zone, the hole jacketed tubing must be perforated.

This type of uptake must be protected at surface by a waterproof area. A concrete box should be constructed for the installation of lifting equipment and accessories (Figure 101).

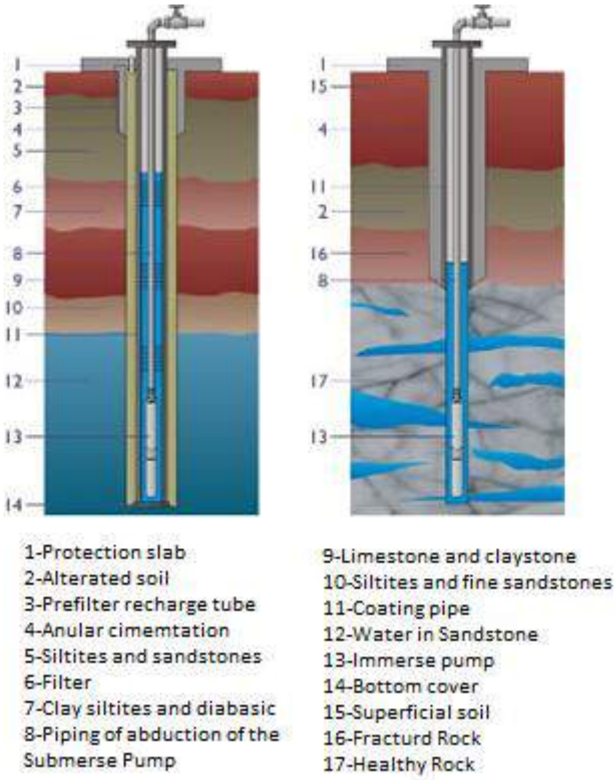

Figure 101 – Scheme of a typical hole (http://seapocos.blogspot.com).

11.4.2 Galleries

Figure 102 – Catchment gallery of Fajã da Ama, S. Vicente, Madeira (ARM, 2018).

It is a large diameter sub-horizontal perforation (1,5x2m) with a much greater depth than the diameter (lengths from 500m to 3000m, until the saturation level is intercepted). The water penetrates throughout the work creating an approximately parallel and horizontal flow. The captured water circulates by gravity.

11.4.3 Springs

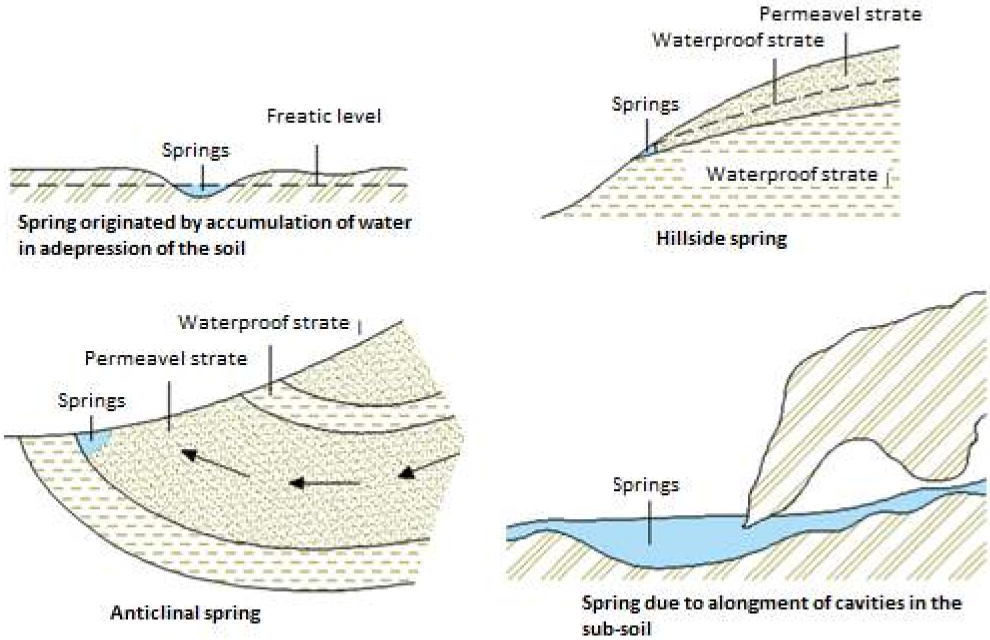

Figure 103 – Types of springs (adapted from Sousa, 2001).

When the surface of the aquifer intersects the surface of a terrain, the water springs naturally to such surface and it is usually designated by **spring**, which is the source of water and it is not an uptake. About catchment or capture, in Madeira, there is a system of canals, the ***Levadas***, which collect and transport water from the Springs (Sousa, 2001).

Also, the capture of water from a spring can be carried out by constructing a chamber in the outcrop zone. A characteristic of this type of uptake is the variability of the flow rates throughout the year. The beginning of the time of lower productivity of the sources coincides, in general, with the periods of higher consumption. For this reason, it can only guarantee the consumption of small agglomerates. However, even when large flow rates are needed it can be used as a source and be supplemented with another uptake (Sousa, 2001).

Being the water of good quality and having the outcrop, a height higher than the agglomerate, which is, the adduction can be made by gravity, then it will be a good idea to use the full capacity of the source (Sousa, 2001).

According to Sousa (2001) the collection or catchment works should be designed to allow:

- The conservation of the physical conditions of water, temperature and gas content;
- Flow rate regulation between origin and consumption;
- Sedimentation of sand and fines, avoiding entering the distribution duct;
- Waterproofing for external waters.

The amount of work involved in the construction of this type of uptake relates to the reinforcement of the area of intake with materials that allow the formation of a filter (gravel, coarse sand) and with the execution of a reservoir of two compartments, being the first for sedimentation and the second for the area of intake. A Chamber of manoeuvres shall also be provided for the installation of pipes and valves (Sousa, 2001).

11.4.4 Wells

The groundwater catchment using wells (Figure 104 and Figure 105) is characterized by provoking a flow that is radially processed inside the porous medium containing the aquifer (or groundwater table) (Júnior, 2015).

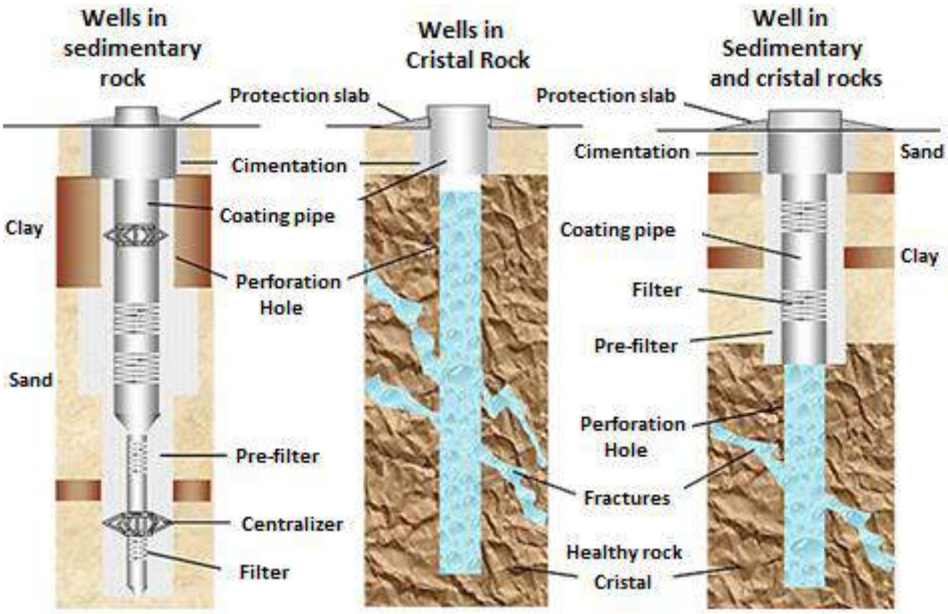

Figure 104 – Example of common wells (http://www.soluaguas.com.br).

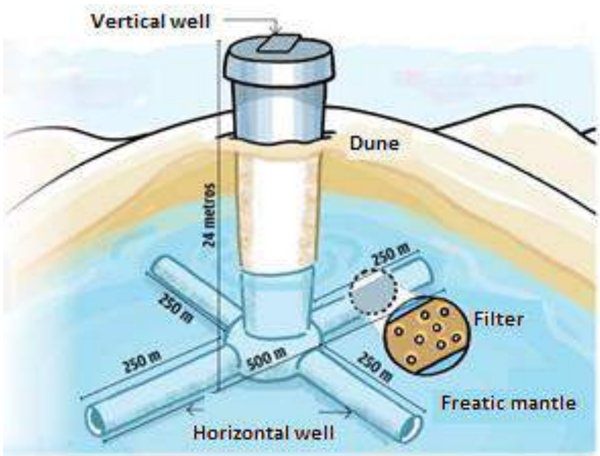

Figure 105 – Example of a radial well (https://www.opovo.com.br).

The catchment made using wells can be performed:

a) with the use of the groundwater aquifer, which is the first to be found when digging and which, as already seen, contains the water inside the porous medium subjected to atmospheric pressure;
b) with the use of the artesian aquifer, where the water pressure is higher than the atmospheric because it is confined between impermeable layers.

According to the aquifer used as a source of supply, the well is called phreatic or Artesian.

In order to illustrate, in Figure 106, a water well is presented, suffering pumping at constant flow rate (Q). In this figure, it is noted that in vicinity of the well, the water level of groundwater is lowered. The first amount of water removed by pumping is from the storage in the aquifer around the well. As the pumping proceeds, a larger amount of water coming from increasingly distant regions is removed, producing depressions in the water level of the aquifer that constitute what is called the cone of depression (Júnior, 2015).

The depth of a catchment well varies according to the aquifer situation in relation to the soil surface (Júnior, 2015).

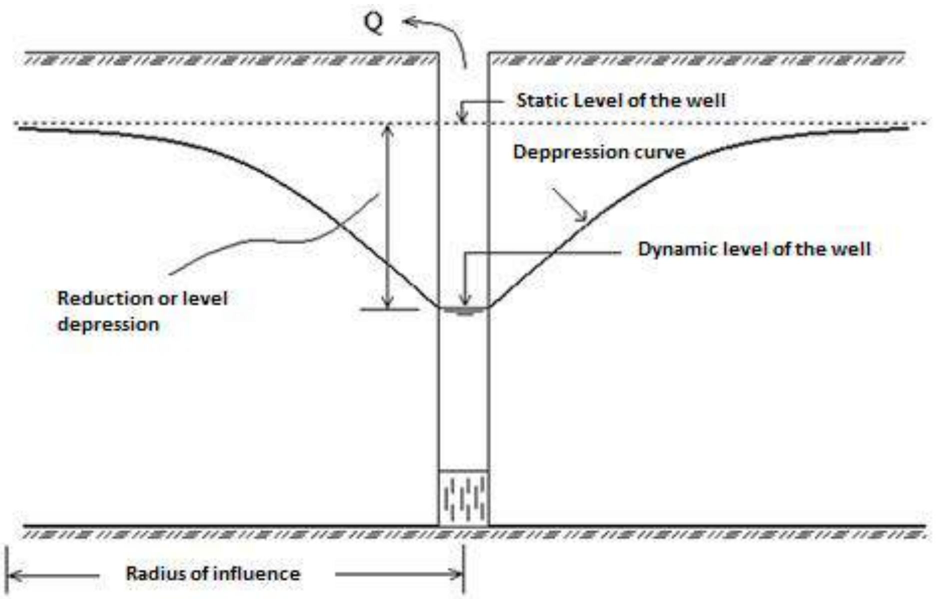

Figure 106 – Reduction and curve of depression due to pumping in a phreatic well (Júnior, 2015).

Terminology

According Júnior (2015), In wells, hydraulic terminology is used below, with its definitions:

a) **Static level of the well** is the level of water balance in the well, when it is not under the pumping action, nor under the influence of previous pumping, nor under the influence of the pumping action that is processed (or sued) in its vicinity:
   a. The groundwater wells, the static level corresponds to the level of the water table;

   b. In artesian wells, the static level is always located above the level of the water table and even above the level of the ground when the well is gushing;

b) **Dynamic level of the well** is the water level in the well when it is being pumped, or suffering the action of a previous pumping or pumping in its vicinity:
   a. In any well (phreatic or artesian), the dynamic level is below the static level, as lower as higher the pumping;
   b. The dynamic level of greatest importance is the one corresponding to the flow rate of design (flow to be supplied by the well). Its determination constitutes one of the important aspects to be considered in the hydraulics of wells;

c) **Equilibrium Regime** is the one in which the dynamic level is stationary after a certain pumping time, because it becomes the flow rate of the well equal to the flow rate of the pump;

d) **Non-balanced Regime** starts with pumping, proceeding with the lowering of the dynamic level until the equilibrium regime is reached. After the pumping has ceased, a new non-balanced regime restarts, which lasts until the total recovery of the well, when the static level is again reached;

e) **Recovery time** is the elapsed time, since the pumping is ceased, until the instant when the dynamic level, which goes on always rising, reaches the position of the static level;

f) **Static level depth** is the distance measured from the surface of the ground to the static level of the well:
   a. By the previous definition, in the case of gushing well, the depth of the static level will be negative;

g) **Depth of the dynamic level** is the distance that is measured from the level of the ground to the dynamic level of the well;

h) **Depression, lowering or reduction of level** is the difference in the elevation or height between the static level and the dynamic level of the well;

i) **Surface of depression**, in the groundwater wells, it is the surface that results from the depression of the sheet or water table level as a result of pumping. Its shape approximates to a lateral surface of an inverted cone trunk, whose lower base is the pit section at the dynamic level position:
   a. In artesian wells, the surface of depression is imaginary and constitutes the geometric place of the piezometric points that suffer depression due to pumping;
   b. The surface of depression is a function of the pumping flow rate;

j) **Depression curve** is the curve that is obtained from the intersection of the depression surface with a vertical plane passing through the axis of the well. The two branches of such depression curve are generally asymmetric, which is more pronounced in the vertical plane parallel to the displacement of groundwater, especially in water tables:

   a. It is possible to trace the well's depression curve, provided that other opened wells are aligned with it and that in all cases is determined the dynamic level of equilibrium, when pumping the evaluated well;

k) **Zone of influence** is the area covered by the depression surface of a well. It is as bigger as the pumping flow:
   a. Any other well that is opened in this zone of influence will be with its depressed level, and due to the pumping of the first, depression would be as greater as shorter the distance between the wells.

Pumping in Freatic and artesian wells

As has been showed, according to the aquifer which promotes water pumping, the well can be called phreatic or artesian. The pumping produces the water level depressions of the aquifer (or the piezometric surface, in case of artesian), constituting the so-called "cone of depression". The radius of this cone, called "**radius of influence"**, is a function of the pumping flow, and varies with the pumping time. The radius of influence, as well as the level of depression, grows with the pumping time, at decreasing rates, until the capacity of reloading of the aquifer is balanced with the pumping flow rate (Júnior, 2015).

Equilibrium Regime

According to what has already been mentioned, the cone of depression stops growing when a situation of equilibrium is established: the pumping flow rate equals the reloading capacity (Júnior, 2015).

"Thiem", studying the variations of the cone of depression within the equilibrium regime, established the expressions that correlate these variations with the pumping flow rate of the phreatic and artesian wells. Thiem formulas, showed below, assume that the granulometry of the aquifer is invariant, as well as its thickness, and that the well reaches the lower limit of the aquifer (in which case, it is called the **complete well**). Thiem's formulas also admit that water in the aquifer shifts on a laminar basis according to radial lines that have as a center, the axis of the well (Júnior, 2015).

Equilibrium regime. Freatic well

Figure 107 represents a complete phreatic well during pumping under constant flow rate. The figure contains the necessary elements to obtain the Thiem equation (Júnior, 2015).

In the illustration, the regime is in equilibrium: the reduction (s) is invariant in time. And around of the well, the aquifer is lowered in a funnel form (depression cone) (Júnior, 2015).

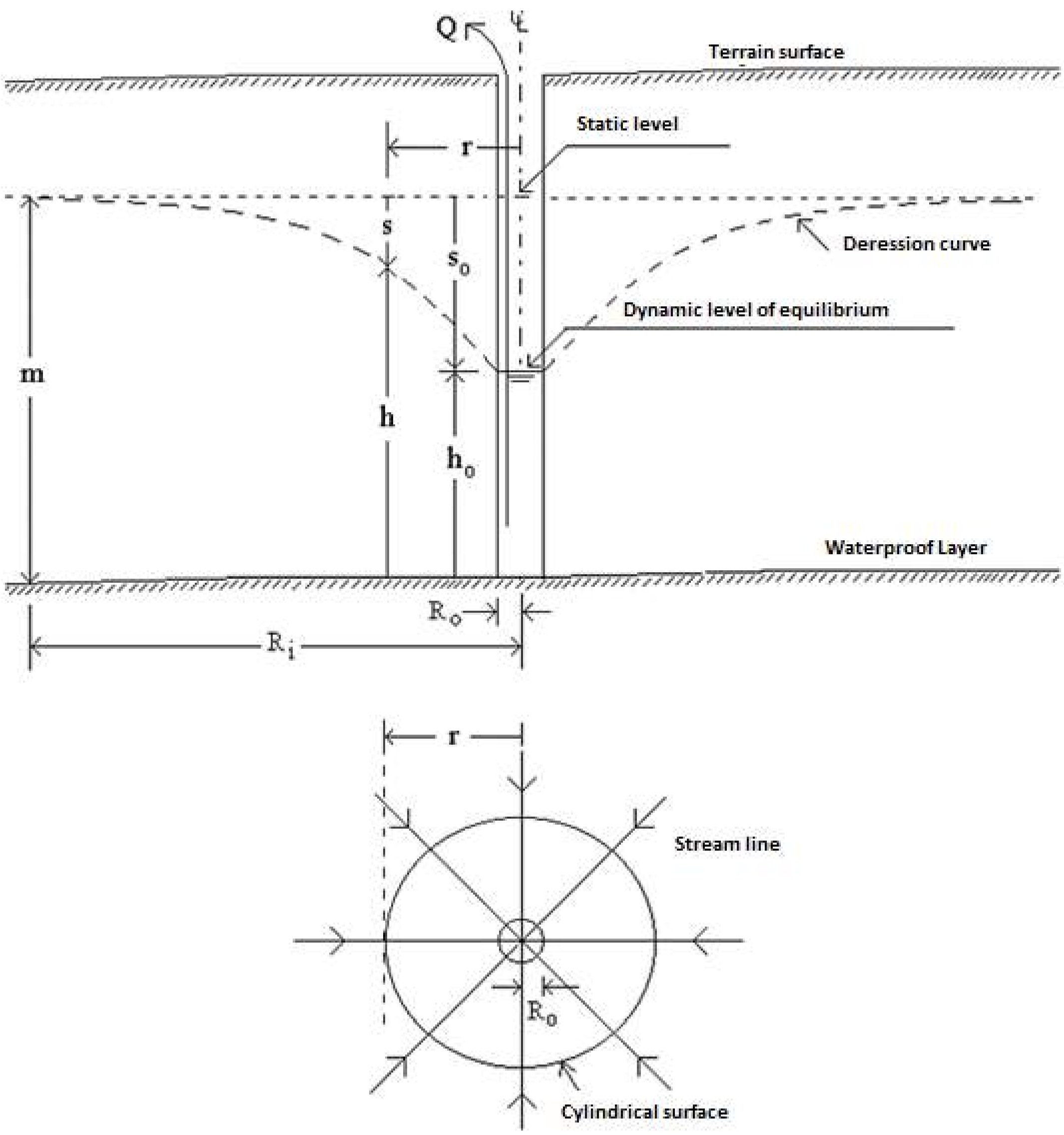

Figure 107 – Artesian well under pumping with constant flow rate and piezometer of observation(Júnior, 2015).

Obtain the curve that express the reduction of the sheet or phreatic level within the zone of influence of the pumping, can be made based on the Darcy's equation. For this, it is considered an imaginary cylindrical surface located at the generic distance (r) of the well's axis, through which flows pumped water from the aquifer. For this surface, it is possible to write:

$$Q = V \cdot A = K \cdot i \cdot A, where \begin{cases} i = dh/dr \\ A = 2\pi \cdot r \cdot h \end{cases}$$

$$Q = 2K\pi r \cdot h \frac{dh}{dr} \rightarrow Q \frac{dr}{r} = 2K\pi \cdot hdh$$

The preceding equation is the differential equation of the depression surface (or the depression cone). It can be integrated between any two boundaries, such as (R_0, h_0) and (r, h):

$$Q\int_{R_0}^{r}\frac{dr}{r}=2K\pi\int_{h_0}^{h}h\,dh\rightarrow Q\ln\ r|_{R_0}^{r}=2K\pi\cdot\frac{h^2}{2}\bigg|_{h_0}^{h}\rightarrow Q\ln\frac{r}{R_0}=2K\pi\left(\frac{h^2}{2}-\frac{h_0^2}{2}\right)$$

$$\therefore Q=\frac{K\pi}{\ln\frac{r}{R_0}}(h^2-h_0^2)\cong\frac{K\pi}{2.303\log\frac{r}{R_0}}(h^2-h_0^2)$$

The previous equation can also be written in terms of the depressions of level, s. For this purpose, it is done:

$$h=m-s\wedge h_0=m-s_0$$

where,

$$Q=\frac{K\pi}{\ln\frac{r}{R_0}}[(m-s)^2-(m-s_0)^2]\cong\frac{K\pi}{2.303\log\frac{r}{R_0}}[(m-s)^2-(m-s_0)^2]$$

Expression for the radius of influence, R_i (Freatic aquifer)

To obtain an expression for the influence radius R_i, the equation $Q=2K\pi r\cdot h\,dh/dr$ is integrated in the interval (R_0, h_0) to (R_i, m):

$$Q=\frac{K\pi}{\ln\frac{R_i}{R_0}}(m^2-h_0^2)=\frac{K\pi}{\ln\frac{R_i}{R_0}}[m^2-(m-s_0)^2]$$

where,

$$\ln\frac{R_i}{R_0}=\frac{K\pi}{Q}[m^2-(m-s_0)^2]=\frac{K\pi}{Q}(2m-s_0)\cdot s_0$$

$$\therefore\ \ln R_i=\ln R_0+\frac{K\pi}{Q}(2m-s_0)\cdot s_0$$

Which allows to obtain R_i, from known values of R_0, Q, K, m, s_0 (Júnior, 2015).

Expression for the coefficient of permeability, K (Freatic aquifer)

Consider pumping a phreatic well with a flow rate Q, and reductions s_1 and s_2 in equilibrium regime, measured in the observation wells PO1 and PO2 (Figure 108). The integration of the equation $Q=2K\pi r\cdot h\,dh/dr$ between the limits (R_1, h_1) and (R_2, h_2) allows to write:

$$Q=\frac{K\pi}{\ln\frac{R_2}{R_1}}(h_2^2-h_1^2)$$

Or,

$$K = \frac{Q \cdot \ln(R_2/R_1)}{\pi(h_2^2 - h_1^2)} = \frac{Q \cdot \ln(R_2/R_1)}{\pi[(m - s_2)^2 - (m - s_1)^2]} \cong \frac{2.303Q \cdot \log(R_2/R_1)}{\pi[(m - s_2)^2 - (m - s_1)^2]}$$

which is the expression of calculus of the coefficient K in the groundwater aquifer, based on the reduction of two observation wells (which operates as piezometers) (Júnior, 2015).

In the case of the well PO_1 is confused with a well under pumping, the distance R_1 would becomes the raidus of the well R_0 and the depression s_1 turns into the depression of dynamic level of equilibrium s_0 for the flow rate Q. In such case, K is calculated according to the expression:

$$K \cong \frac{2.303Q \cdot \log(R/R_0)}{\pi[(m - s)^2 - (m - s_0)^2]}$$

In which R and s are refered to only the well of operation.

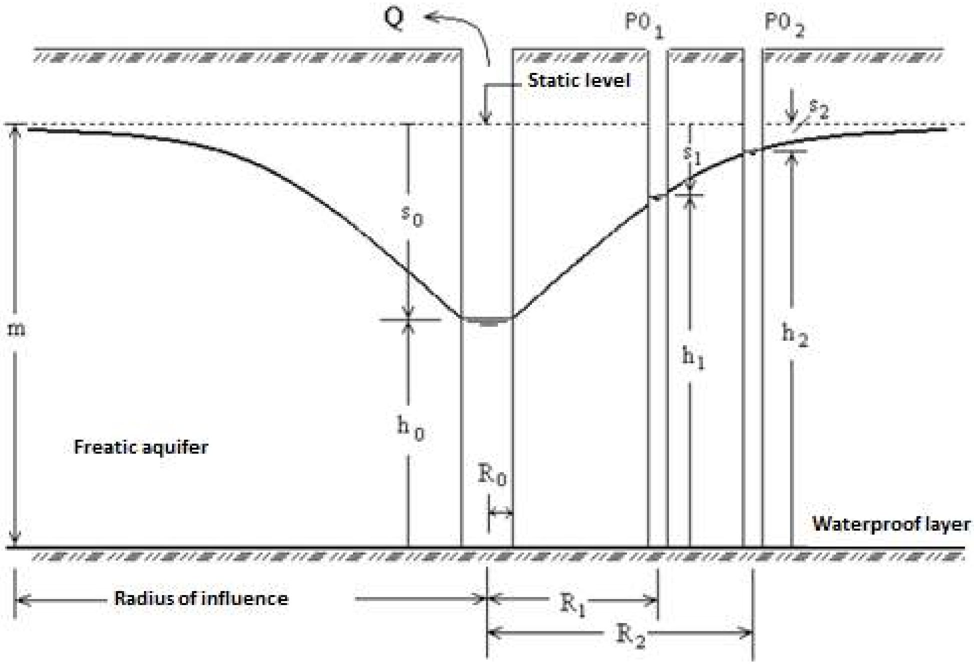

Figure 108 – Pumping go the phreatic well. Calculus of the permeability coefficient based in two wells of observation (Júnior, 2015).

Observations:

i) The values of the permeability coefficient K are generally more accurate when defined by the determinations related to two observation wells, due to the loss of load at the input of the pumped well. However, the use of a single observation well, rather than two, is more comfortable and economical. This observation also applies to the case of wells in artesian aquifers;

ii) Good practice suggests obtaining an average permeability coefficient ($\overline{K}$). For this, several piezometers (observation wells) are required, as shown in Figure

109. In the case of using 4 piezometers, it is recommended to dispose the following conditions: the first at 1m of the axis of the pumped well; the second at 2m from first piezometer; the third at 5m separation of the second; and the last one at 10m from the third piezometer.

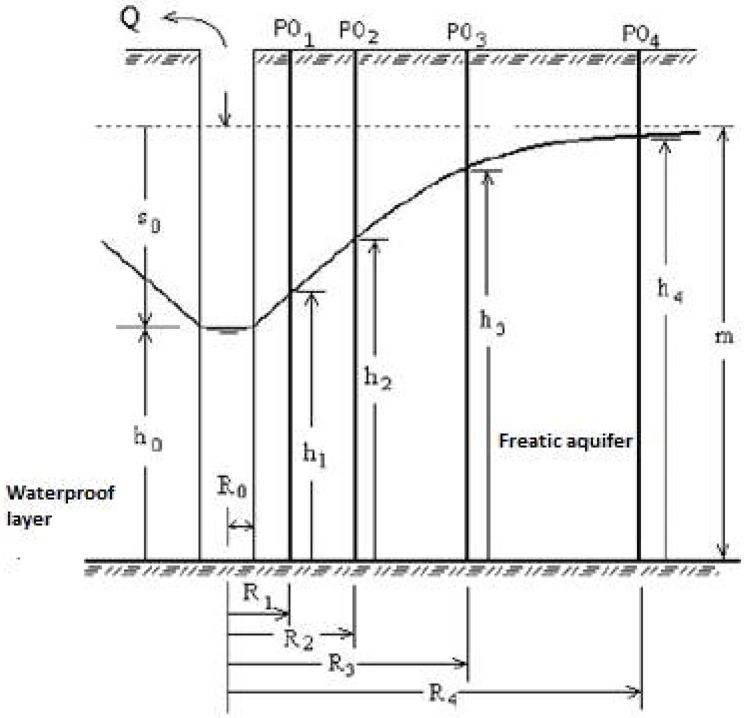

Figure 109 – Scheme of the system of 4 wells of observation to obtain an average coefficient of permeability of the aquifer (Júnior, 2015).

Applying successively an expression of calculus for the coefficient K in a phreatic aquifer for each pair of piezometers i and j (1 & 2, 1 & 3, 1 & 4, 2 & 3, 2 & 4, 3 & 4), it can be determined various values of $K_{i,j}$, that allows to obtain the average coefficient of permeability. For four piezometers, as in Figure 109,

$$\bar{K} = \frac{1}{6}\left(K_{1,2} + K_{1,3} + K_{1,4} + K_{2,3} + K_{2,4} + K_{3,4}\right)$$

In general, for N observation wells,

$$\bar{K} = \frac{1}{\dfrac{N!}{2!\,(N-2)!}} \sum K_{i,j}$$

Equilibrium regime. Artesian well

Figure 110 Represents an artesian well during pumping with a constant flow rate Q, in equilibrium regime: the reduction of the piezometric surface, in each position r, remains invariable in time. The cone of depression represented in the figure constitutes, in fact, an imaginary surface (although this surface can be materialized through the installation of piezometers in the aquifer: the piezometers allows to obtain virtual levels above of the aquifer, similarly to a phreatic well) (Júnior, 2015).

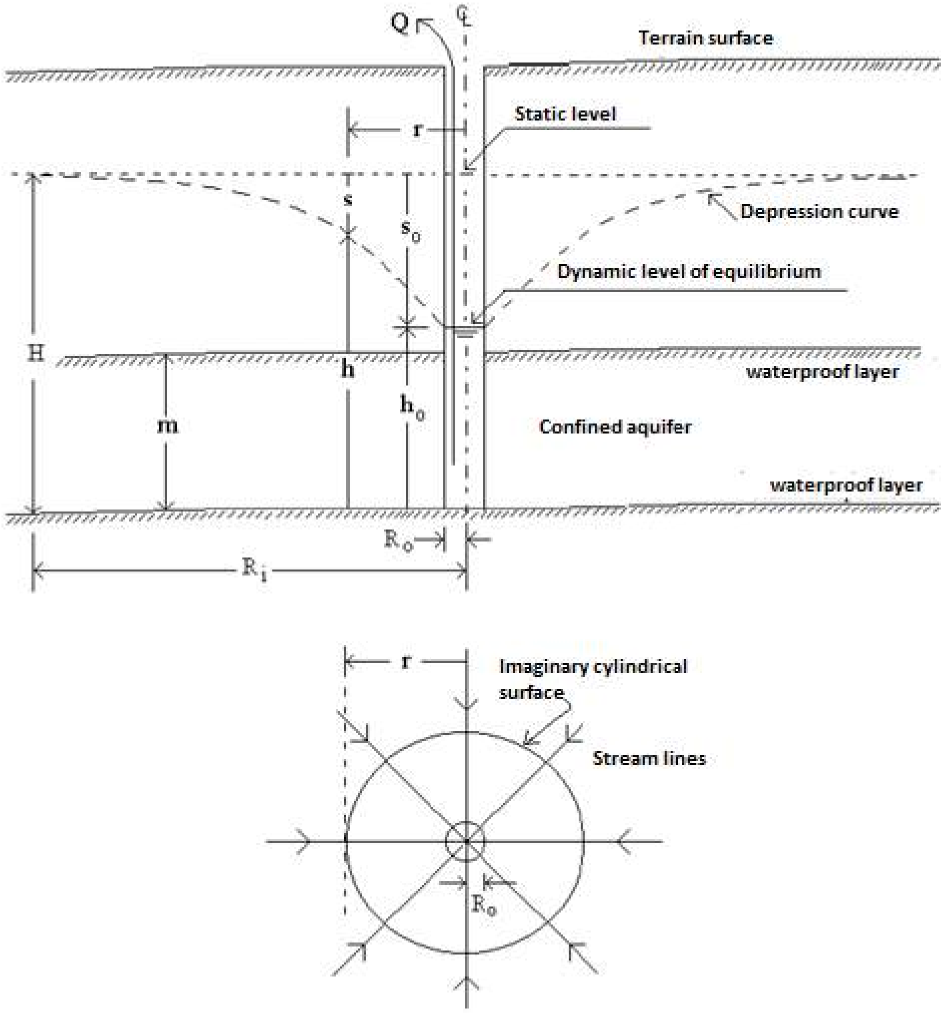

Figure 110 – Artesian well under pumping with constant flow rate (Júnior, 2015).

According to Darcy's law applied to a cylindrical surface located at a distance r from the well axis (Figure 110), which through water flows with a flow rate equal to the pumping flow rate (equilibrium regime),it can be stated that:

$$Q = V \cdot A = K \cdot i \cdot A, where \begin{cases} i = dh/dr \\ A = 2\pi \cdot r \cdot m \end{cases}$$

$$Q = 2K\pi mr \cdot \frac{dh}{dr} \rightarrow Q\frac{dr}{r} = 2K\pi m \cdot dh$$

If the previous equation is integrated between the well boundaries, (R_0, h_0), and a region that suffers the influence of pumping, (r, h), then:

$$Q\int_{R_0}^{r}\frac{dr}{r} = 2K\pi \cdot m\int_{h_0}^{h} dh \rightarrow Q\ln\frac{r}{R_0} = 2K\pi \cdot m(h - h_0)$$

In terms of the depression of the piezometric surfaces:

$$h_0 = H - s_0; h = H - s \rightarrow h - h_0 = s_0 - s$$

Then,

$$Q = \frac{2K\pi m}{\ln \frac{r}{R_0}}(s_0 - s) \cong \frac{2K\pi m}{2.303 \log \frac{r}{R_0}}(s_0 - s)$$

The upper equation is known as **the equation of Thiem for artesian aquifers** (Júnior, 2015).

Expression for the radius of influence, R_i (artesian aquifer)

For the equation $Q = 2K\pi mr \cdot dh/dr$ integrated between the limits (R_0, h_0) & (R_i, H):

$$Q = \frac{2K\pi m}{\ln \frac{R_i}{R_0}}(H - h_0)$$

But, $H - h_0 = s_0$, which correspond to the reduction of the level of dynamic equilibrium. Then:

$$Q = \frac{2K\pi m}{\ln \frac{R_i}{R_0}} s_0$$

Ii is an expression that shows the flow rate, possible to obtain from an artesian well, it would be proportional to the difference $s_0 = (H - h_0)$. However, these equations are only applicable for relatively weak (depression) levels and less than $1/4 \cdot (H - m)$, which is for $s_0 < (H - m)/4$ (Júnior, 2015).

The expression for the radius of influence, based on the previous equation, gives:

$$\ln \frac{R_i}{R_0} = \frac{2k\pi m}{Q} s_0 \rightarrow R_i = R_0 \times e^{\left(\frac{2k\pi m}{Q} s_0\right)}$$

Expression for the coefficient of permeability, K (artesian well)

In Figure 111 represents an artesian well under pumping and two observation wells (Piezometers), PO1 & PO2, with a distance of R_1 and R_2 from the pumped well axis. The reduction of the piezometric surfaces corresponding to PO1 and PO2 are respectively, $s_1 = H - h_1$ and $s_2 = H - h_2$, being H the height of the static charging plain referred to the inferior waterproof layer of the artesian aquifer (Júnior, 2015).

For the equation $Q = 2K\pi mr \cdot dh/dr$ integrated between limits (R_1, h_1) and (R_2, h_2), it is obtained:

$$Q = \frac{2K\pi m(h_2 - h_1)}{\ln \frac{R_2}{R_1}} = \frac{2K\pi m(s_1 - s_2)}{\ln \frac{R_2}{R_1}} = \frac{2K\pi m(s_1 - s_2)}{2.303 \log \frac{R_2}{R_1}}$$

because, $(h_2 - h_1) = (H - s_2) - (H - s_1) = (s_1 - s_2)$ (Júnior, 2015).

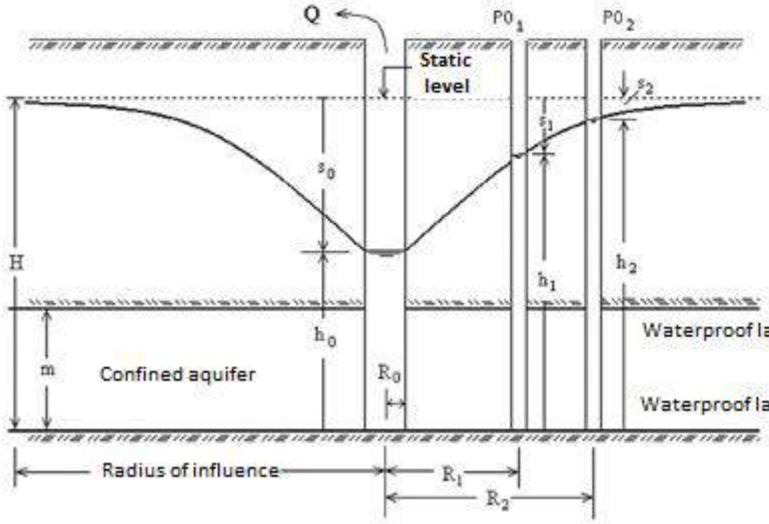

Figure 111 – Pumping of an artesian well. Obtaining the permeability coefficient based on the readings of two observation wells (Júnior, 2015).

Detailing in terms of K:

$$K = \frac{Q \cdot \ln(R_2/R_1)}{2\pi m(s_1 - s_2)} = \frac{2.303Q \cdot \log(R_2/R_1)}{2\pi m(s_1 - s_2)}$$

that is the expression for the calculus of the permeability coefficient K of and artesian aquifer based on the reductions of 2 observation wells (Júnior, 2015).

Again, here are the same observations made in the study of the permeability of the groundwater aquifer. The equations for obtaining an average permeability coefficient are also valid when using several observation wells (Júnior, 2015).

Interference between water wells

The interference between two wells occurs when, being both subjected to pumping, their zones of influence partially coincide. In practice, in order to avoid interference between two wells that will operate simultaneously with the same Q, flow rate, it is sought to determine the minimum distance that must exist between them. Thiem equations are used to obtain the influence radius R_i, as the aquifer is phreatic or artesian. In order to have a well not located in the region of influence of the other, the minimum distance between, must be $2 \times R_i$. If the wells distance each other from a value less than $2 \times R_i$, there will be interference (Júnior, 2015).

Non-equilibrated regime

The unbalanced regime (non-permanent regime), which starts with pumping, is characterized by the lowering of the dynamic level and ends when the equilibrium regime is reached: the water level of the well, initially at the static level, stabilizes at the dynamic level of equilibrium under constant pumping flow (Júnior, 2015).

For permanent flows, admitting water as incompressible and the structure of the aquifer as undeformable, it can be proved that the Laplacian of the hydraulic load is null: $\nabla^2 h = 0$ (aquifer of constant thickness and permeability). Or, in Cartesian coordinates,

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

In polar coordinates,

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = 0$$

When the water from an artesian aquifer starts to be used, an important part of the well feeding, comes from the decompression of water in the zone of pressure reduction and compaction of the saturated state. This action gradually reaches the furthest regions from the pumping site, as the process of water extraction proceeds in time. In an aquifer of infinite extension, the equilibrium conditions cannot not be reached in a finite time (Júnior, 2015).

For non-permanent flow conditions (unbalanced regime) in a compressible aquifer, the application of the continuity equation to a concentric control volume of a well, produce a differential equation:

$$T\nabla^2 h = S\frac{\partial h}{\partial t}$$

In cylindrical coordinates:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t}$$

where:
S - coefficient of storage, dimensionless;
T - coefficient of transmissivity, $[T] = L^2T^{-1}$;
h - hydraulic load or charge $(h = z + p/\gamma)$, $[h] = L$.

The last equation can be written in terms of reduction s ($s = H - h$, for an artesian aquifer). Them:

$$\frac{\partial^2 s}{\partial r^2} + \frac{1}{r}\frac{\partial s}{\partial r} = \frac{S}{T}\frac{\partial s}{\partial t}$$

Formula of Theis

The result of the integration, which expresses the reduction of the piezometric surface in an observation well located at a distance r from the pumping point (Figure 112), as a function of time, known as Theis formula (formula obtained by Charles Vernon Theis in a paper developed for the US Geological Survey in 1935, based on existing literature of heat transfer, with the mathematical help of CI Lubin),it is derived from the analogy between groundwater flow and heat conduction, considering the initial conditions and of contour:

i) $s(r,0) = 0$;
ii) $s(\infty, t) = 0$;
iii) $\lim_{r\to 0} r(\partial s/\partial r) = -(Q/2\pi T)$.

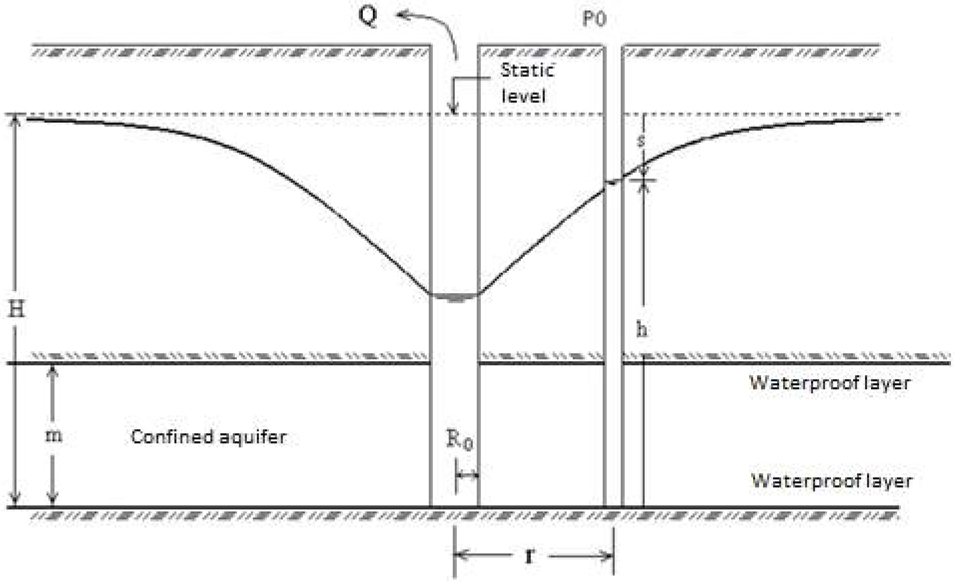

Figure 112 – Artesian well pumped under constant flow rate and lowering observed in an observation well located at distance r from pumped well axis (Júnior, 2015).

The classic solution presented by Theis is:

$$s = H - h = \frac{Q}{4\pi T}\int_u^{\infty}\frac{e^{-u}}{u}du$$

Or,

$$s = \frac{Q}{4\pi T}W(u)$$

Where:

$$W(u) = function\ of\ the\ well = \int_u^{\infty}\frac{e^{-u}}{u}du$$

Being:

$$u = \frac{r^2 S}{4Tt}$$

The values of $W(u)$ can be found by the development of the convergent series:

$$W(u) = \int_u^\infty \frac{e^{-u}}{u} du = -0.5772 - \ln u + u - u^2 + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} + \cdots$$

Based in such series, it is possible to build tables of such function $W(u)$ in function of the variable u, defined by the equation $r^2S/4Tt$. One of the most used is Table 30 of Wenzel (Júnior, 2015).

Table 30 – Table of Wenzel (1942) for values of the function of a well, W(u), in terms of u (Júnior, 2015).

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
x 1	0.219	0.049	0.013	0.0038	0.00114	0.00036	0.00012	0.000038	0.000012
x 10^{-1}	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
x 10^{-2}	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
x 10^{-3}	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
x 10^{-4}	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
x 10^{-5}	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
x 10^{-6}	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
x 10^{-7}	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
x 10^{-8}	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
x 10^{-9}	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
x 10^{-10}	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
x 10^{-11}	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
x 10^{-12}	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
x 10^{-13}	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
x 10^{-14}	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
x 10^{-15}	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

Formula of Theis modified by Jacob

Studies realized by C. E. Jacob (1940) in turn of the equation of Theis, $s = Q/4\pi T\, W(u)$, for a non-equilibrated regime, lead him to conclude that for sufficient small values of u, it can be consider with a good approximation, the convergent series limited to the two first term:

$$W(u) = \int_u^\infty \frac{e^{-u}}{u} du \cong -0.5772 - \ln u$$

Thus, for a sufficiently long time (which is equivalent to a small u), Jacob re-wrote the Theis equation in the approximated form:

$$s = \frac{Q}{4\pi T} W(u) \cong \frac{Q}{4\pi T}(-0.5772 - \ln u)$$

Doing $-0.5772 = \ln x$, it is obtained $x = 0.56147$. Then,

$$-0.5772 - \ln u = \ln 0.56147 - \ln u = \ln(0.56147/u)$$

Thus,

$$s = \frac{Q}{4\pi T} \ln\left(\frac{0.56147}{u}\right) = \frac{Q}{4\pi T} \frac{\log(0.56147/u)}{\log e} = \frac{2.303}{4\pi} \frac{Q}{T} \log\left(\frac{0.56147}{u}\right)$$

Remembering that $u = r^2S/4Tt$, then:

$$s = \frac{0.183Q}{T}\log\left(\frac{4 \times 0.56147 \times T \times t}{r^2 \times S}\right)$$

Or,

$$s = \frac{0.183Q}{T}\log\frac{2.25Tt}{r^2S}$$

The above equation is the Jacob's simplified Theis formula for reduction in a well of observation at the distance r from the well under pumping. The formula holds for a t large enough (or small). In practice, for $u < 0.01$, the values of this equation are practically identical to the classical solution presented by Theis (Júnior, 2015).

Determination of the coefficients of transmissivity (*T*) and storage (*S*) based on the formula of Theis simplified by Jacob

Time-lowering process

The characteristics of an aquifer can be determined from the collection of a set of pairs of values of reduction and corresponding time, (s_i, t_i), the time being counted from the beginning of the pumping. This method of determining the characteristics of the aquifer is known as the time-lowering process (Júnior, 2015).

For a convenient graphical representation, observed reduction as a function of time are plotted on monolog paper: the values of the reductions (s) are plotted on the arithmetic scale and the values of observation times (t) in abscissa, on the logarithmic scale. For large periods of time (which implies small values of u), the data is arranged in a straight line (Júnior, 2015).

Indeed, by rewriting the formula of **Theis simplified by Jacob**,

$$s = \frac{0.183Q}{T}\log t + \frac{0.183Q}{T}\log\frac{2.25T}{r^2S}$$

Which it is of kind, $y = ax + b$. By the last equation, the line slope in the graphic of s *versus* $\log t$ is equal to $0.183Q/T$. The coefficient of transmissivity can be calculated from the pairs of values of s and t, located on the line $s = f(\log t)$:

$$\begin{cases} For\ the\ instant\ t_1:\ s_1 = \dfrac{0.183Q}{T}\log t_1 + \dfrac{0.183Q}{T}\log\dfrac{2.25T}{r^2S} \\ For\ the\ instant\ t_2:\ s_2 = \dfrac{0.183Q}{T}\log t_2 + \dfrac{0.183Q}{T}\log\dfrac{2.25T}{r^2S} \end{cases}$$

Doing $s_2 - s_1$, results:

$$s_2 - s_1 = \frac{0.183Q}{T}\log\frac{t_2}{t_1}$$

And,

$$T = \frac{0.183Q}{s_2 - s_1} \log \frac{t_2}{t_1}$$

If it is chosen by convenience, $t_2 = 10t_1$,

$$T = \frac{0.183Q}{s_2 - s_1}$$

Figure 113 shows the reductions s_1 and s_2 in the observation wells corresponding to instants t_1 and t_2 as showed above (Júnior, 2015).

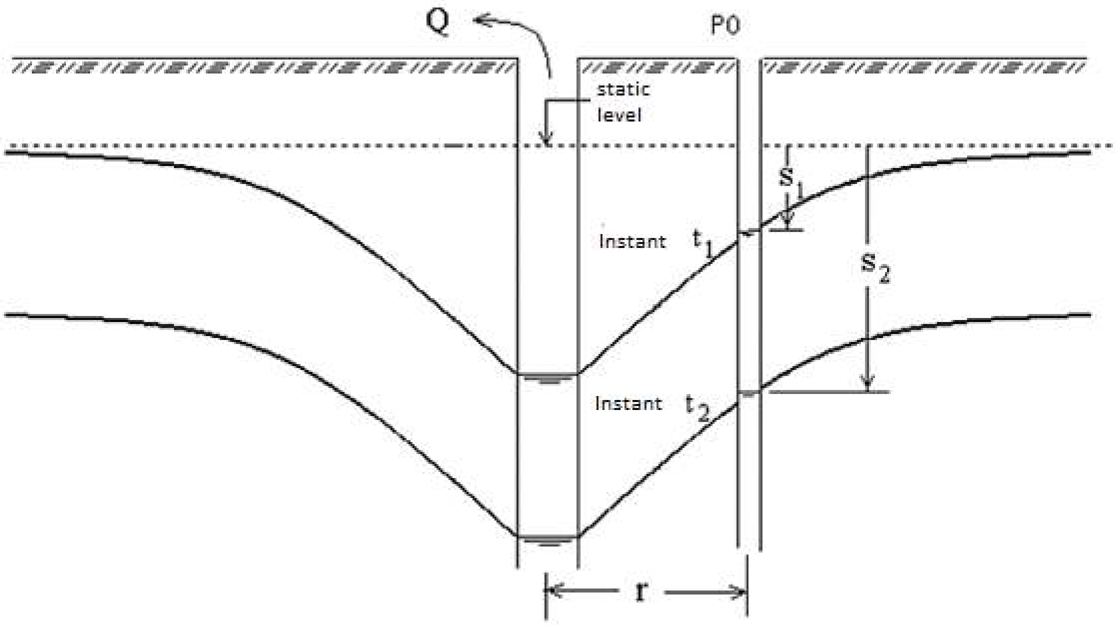

Figure 113 – Reduction observed in an observation well in two successive instants (Júnior, 2015).

The storage coefficient can also be estimated based on the graphical construction of s *versus* $\log t$ (or of s *versus* t, in a monolog graph). As for example, the graphic in the *monolog paper* (Figure 114), extrapolate the linear tendency to obtain the moment t_0 corresponding to the reduction of $s = 0$. Then, based in the **formula of Theis simplified by Jacob**, for $s = 0$ (null reduction), then:

$$\frac{2.25Tt_0}{r^2 S} = 1$$

And once $Q \neq 0$. Knowing the coefficient T, it is possible to write:

$$S = \frac{2.25Tt_0}{r^2}$$

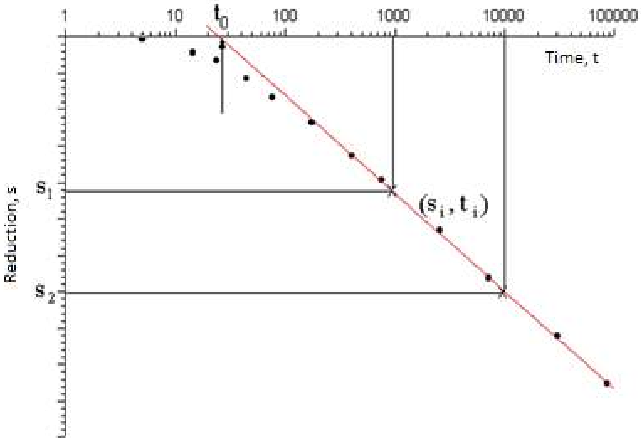

Figure 114 – The reduction curve versus time in monolog paper to obtain the coefficients of transmissivity, T, and storage, S (Júnior, 2015).

For its simplicity, the equations for determination of T and S due to Jacob, constitute a useful tool for determining the aquifer characteristics. In an alternative to graphical construction, it is also possible to employ a regression analysis to obtain the coefficients T and S, provided that sufficiently large values of t guarantee $u < 0.01$ (Júnior, 2015).

Determination of the transmissivity coefficients (T) and storage coefficient (S) based in the general expression of Theis

When the duration of pumping is not long enough to allow the definition of the logarithmic asymptote (see Figure 114), it must be used the general expression of Theis (Júnior, 2015).

For such condition, Theis developed a graphical method based in the proportionality between u and r^2/t:

$$s = \frac{Q}{4\pi T} W(u) \rightarrow W(u) = C_1 \cdot s$$

$$u = \frac{r^2}{t} \frac{S}{4T} \rightarrow u = C_2 \cdot \frac{r^2}{t}$$

According to Júnior (2015), the method consists in comparing the descriptive curve of the behaviour of $W(u)$ in function of u (called "standart curve"), plotted on log-log paper (or bi-log paper) with the experimental curve of s in function of r^2/t, drawn on the same scale. Thus, at the end of the pumping test, with the pairs of reduction values as a function of time obtained in the observation well, it is proceeded as follows:

a) on transparent log-log paper, the pairs of values of $\boldsymbol{s}$ and $\boldsymbol{r^2/t}$, where r is the distance between the axis of the wells of observation and pumping, and t is the time at which reduction s is measured;
b) In opaque log-log paper, the pairs of values of $\boldsymbol{W(u)}$ and $\boldsymbol{u}$, that is, construction of the "standard curve" (note that the size of each "cycle" of the log-log paper must be equal to the corresponding of the previous chart);

c) Then, overlap both charts (of course, with the transparent paper on the opaque paper), keeping the axes $W(u)$ and $s(t)$ parallel. The transparent paper is adjusted until most of the observed reductions fall on the "standard curve" (Note that the corresponding axes must remain parallel during the displacement in order to search the best fit);
d) Selects an arbitrary point (not necessarily on the "standard curve") and notes, for this point, the values of u and $W(u)$ of the opaque paper, and the corresponding r^2/t and s of the transparent paper. Such points are designated u_0 and $W_0(u)$, and $(r^2/t)_0$ and s_0;
e) Finally, the transmissivity (T) and storage (S) coefficients are calculated using the equations for such effect and the above-determined coordinates:

$$T = \frac{Q}{4\pi s_0} W_0(u)$$

And,

$$S = \frac{4Tu_0}{(r^2/t)_0}$$

It should be noted that the methods of Theis and Jacob apply, strictly, to artesian aquifers. Its use in free aquifers (groundwater or phreatic wells) may provide acceptable values if the reduction of level is small relatively to the thickness of the water table (Júnior, 2015).

11.4.5 Aquifers. Transmissive Function

According to the hydrodynamic parameters obtained in pumping tests that characterize the transmissive function of aquifers, it is possible to qualitatively classified in terms of hydraulic conductivity (K) and transmissivity (T) by using tables (based on vast testing) as follows:

Table 31 – Values of hydraulic conductivity (K) of aquifers and its classification.

K (m/day)	Classification
$K < 10^{-2}$	Very low
$10^{-2} < K < 1$	Low
$1 < K < 10$	Medium
$10 < K < 100$	High
$K > 100$	Very high

Table 32 – Values of transmissivity (T) of aquifers and its classification.

T (m²/day)	Classification
$T < 10$	Very low
$10 < T < 100$	Low
$100 < T < 500$	Medium
$500 < T < 1000$	High
$K > 1000$	Very high

CHAPTER 12 - NOTIONS OF STATISTICS

12.1 Introduction

According to ISEL (2015), the hydrological phenomenon depends of physical laws, but are tide to multiple random factors (casual), which leads to the use of **Probability Theory** and **Statistical Methods**, to predict or infer its behaviour:

- **Probability Theory** - to build a model of distribution of the results of observations of such phenomenon;
- **Statistical Methods** - to infer, with major or minor inference, the results that could happen in a near future, starting from the observation of the phenomenon and obtain a measure of that uncertainty.

12.2 Random variable, frequency and probability

Consider an experience constituted by the release of a perfect die (without vices). In this case, we know the set *Ω* of the possible results of the experiment (1,2,3,4,5,6), but it is not always known the exact result that will occur in each execution of the experiment (ISEL, 2015).

It is called **random experience** to an experience where:

- The set of all possible outcomes is known;
- It is not possible to know, before the experience, the result that will occur.

It is called, **frequency** of a given event (face output 6, for example):

- Quotient between the number of times it occurs and the total number of repetitions of the experiment.

It is called **the probability** of a given event:

- Limits that tends to the frequency of the event when the number of repetitions tends to infinity.

The set *Ω* of the possible results of the experiment (1, 2, 3, 4, 5, 6) is called **population** (or universe) (ISEL, 2015).

Sample of the population *Ω* is any finite arrangement of *Ω* elements. The elements of a sample are referred as observations (ISEL, 2015).

The number of observations that constitute the sample is called size (or dimension or length) of the sample (ISEL, 2015).

The results of the experiment, whose values are not known before the match or occurrence, constitute a **random variable**. A random variable is called:

- **Discrete** if it can only take discontinuous values (for example, the number of days that rains each year);

- **Continuous** when it can take any x value of a range of values, limited or not (for example, the average annual rainfall at a udometric station).

12.3 Frequency distributions

To summarize large amounts of data it is usual to distribute them in classes and determine the number of individuals belonging to each class, which is designated as **absolute frequency** of the class (ISEL, 2015).

The ratio between the frequency of the class and the total frequency is called relative frequency, with the sum of all relative frequencies equal to 1 (ISEL, 2015).

The distribution of data in classes with the respective frequencies is called **frequency distribution**. The graphic representation of a frequency distribution is called **histogram** (ISEL, 2015).

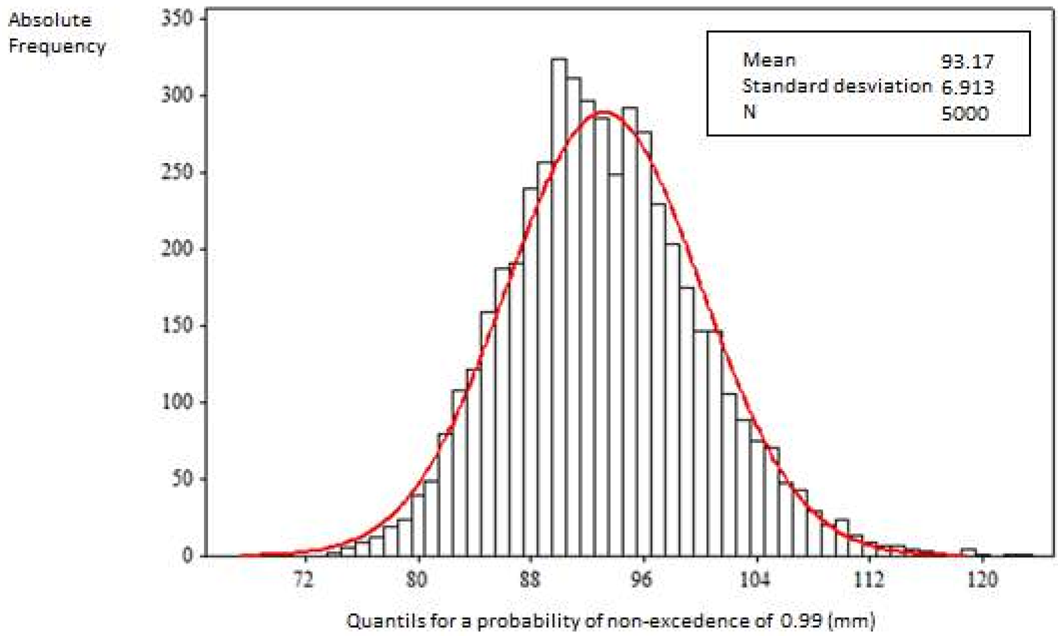

Figure 115 – Histogram of estimates provided by the synthetic series (in number of W = 5000) of the annual maximum daily precipitation at the Udometric station of Pavia (20I/01G) for a probability of non-exceedance of 99% (Naghettini & Portela, 2011).

12.4 Distribution and Duration functions

Suppose that, there is a sample with many observations of a given phenomenon X (for example the annual rainfall in a hydrographic basin), and the values are classified in ascending order:

- Sample: $x_1, x_2, x_3, x_4, x_5, \ldots, x_i, \ldots, x_n$ (x_n is the highest value of the sample).

The frequency of not exceeding the value of the event of order i of the sample, will be i/n:

$$F(x_i) = \frac{i}{n}$$

where:
i - order number of the sample elements sorted in ascending order;
n - total number of sample elements.

The defined function is called the empirical distribution function (or empirical frequency function), and is designated as $F(x)$:

$$F(x) = P(X \leq x), probability\ of\ \ X \leq x$$

If the sample values are sorted in a descending order, it is defined the frequency of equalled or exceeded the x value of the event of order i, as:

$$G(x) = P(X \geq x), probability\ of\ \ X \geq x$$

The function $G(x)$ is known as **function of empirical duration** (ISEL, 2015).

It should be noted that, when such concept is applied to discrete random variables, the **sum of the functions $\boldsymbol{F(x) + G(x)}$** is always greater than 1 (ISEL, 2015). As for example:

- 10 values are ordered in an ascending order. The probability of not exceeding the value of order 7 is $7/10$.

1	2	3	4	5	6	7	8	9	10

- The values are now ordered in a descending order. The same value will occupy position 4, so the probability of being equated or exceeded is $4/10$.

10	9	8	7	6	5	4	3	2	1

- The sum will be $7/10 + 4/10 = 1.1$.

Therefore, in practice, for the calculation of **"empirical probability"**, instead of $F(x_i) = i/n$, it is used $F(x_i) = i/n + 1$ (it can be demonstrated that this expression gives the average value of the distribution function in the various samples taken from the same population) (ISEL, 2015).

In the case of a **continuous random variable,** a probability density is defined as the function:

$$f(x) = \lim_{\Delta x \to 0} \frac{\Delta F(x)}{\Delta x} = \frac{dF(x)}{dx}$$

Which results in the distribution function,

$$F(x) = \int_{-\infty}^{x} f(x)dx$$

And the duration function,

$$G(x) = \int_{x}^{+\infty} f(x)dx$$

Being:

$$F(x) + G(x) = \int_{-\infty}^{+\infty} f(x)dx = 1$$

Then, the probability of X being between $\boldsymbol{x}$ and $\boldsymbol{x + \delta x}$ is :

$$P(x \leq X \leq x + \delta x) = f(x)dx$$

The probability of X to take a value between $\boldsymbol{a}$ and $\boldsymbol{b}$ is:

$$P(a \leq X \leq b) = F(b) - F(a) = \int_{a}^{b} f(x)dx$$

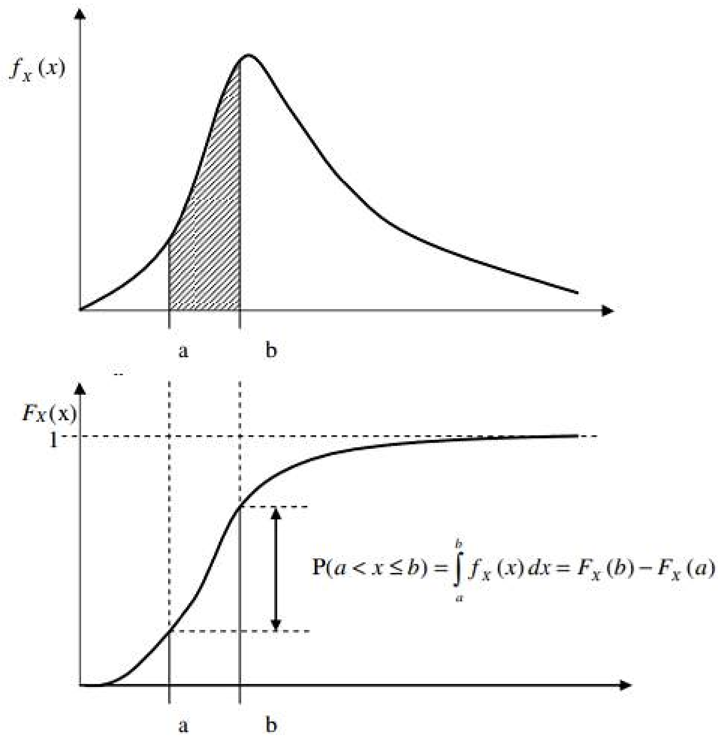

Figure 116 – Density and accumulated functions of probability of a continuous variable (Naghettini & Portela, 2011).

12.5 Return period and Risk

12.5.1 Return period

For annual values of the hydrological variables, it is more convenient to associate to a given value x of such variable, the **return period** $T(x)$, instead of the probability of occurrence $F(x)$ (ISEL, 2015).

Being $G(x)$ the probability of a value x of the variable X being exceeded in a given year:

$$G(x) = 1 - F(x)$$

It is defined as a **return period** (or interval of recurrence), $T(x)$, expressed in years, the inverse of this probability:

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

The return period of a given value x shows the number of years (average) that separates the occurrence of values higher than x. **It is indispensable to bear in mind that the concept of return period is not associated with any idea of a cyclical repetition** (ISEL, 2015).

Then, it can occur that in two successive years, the values of the variable that exceed x_{100} corresponding to a period of 100 years; such occurrence is unlikely to happen, but it is not impossible (ISEL, 2015).

What actually defines the return period, is the **average interval** that would separate the occurrence of values of the variable, upper than x_{100} , if there exists the possibility of having a sufficiently long number of years of observation of such variable (ISEL, 2015).

12.5.2 Risk

Since $F(x)$ is the probability of a value x of not being exceeded in a certain year, it is possible to define the probability of x of not being exceeded in n successive years, by:

$$F_1(x) \times F_2(x) \times F_3(x) \times \ldots \times F_n(x) = F^n(x) = \left(1 - \frac{1}{T}\right)^n$$

It is defined as risk, R, of the event x being exceeded, the probability that x is exceeded at least one time in n successive years:

$$R(x) = 1 - F^n(x) = 1 - \left(1 - \frac{1}{T}\right)^n$$

Being R function of T and n, it can be formed a table that relates such values:

Table 33 – Return period associated to different grades of risk and period of projects (useful time of projects) (ISEL, 2015).

Risk, R (%)	Period of a project, n (years)							
	2	5	10	15	20	25	50	100
75	2.00	4.13	7.73	11.3	14.9	18.5	36.6	72.6
50	3.41	7.73	14.9	22.1	29.4	36.6	72.6	144.8
40	4.44	10.3	20.1	29.9	39.7	49.4	98.4	196.3
30	6.12	14.5	28.5	42.6	56.6	70.6	140.7	281
25	7.46	17.9	35.3	52.6	70.0	87.4	174.3	348
20	9.47	22.9	45.3	67.7	90.1	112.5	224.6	449
15	12.8	31.3	62.0	92.8	123.6	154.3	308	616
10	19.5	48.0	95.4	142.9	190.3	238	475	950
5	39.5	98.0	195.5	292.9	390	488	975	1950
2	99.5	248	495	743	990	1238	2475	4950
1	199.5	498	995	1493	1990	2488	4975	9950

For example: Determine the full return period in order to design a marginal protection dam, if we wish that in the next 10 years the risk (probability) of being burst or drowned, it's does not exceed 0.20 (ISEL, 2015).

$$Given\ n = 10\ and\ R = 0.20, from\ the\ upper\ table\ , T = 45\ years.$$

12.6 Statistical parameters of populations and samples

The main parameters that are considered in the statistical distributions and the expressions for calculating the corresponding values in samples are, as follows:

- Measures of central tendency;
- Dispersion measures;
- Asymmetry measures.

12.6.1 Measures of central tendency

Average (expected value or arithmetic average)

It is defined in continuous distributions by:

$$\mu = \int_{-\infty}^{+\infty} xf(x)dx$$

In a finite sample with n elements, it is defined by:

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

Median

In a continuous distribution, it is defined by the equation:

$$\int_{-\infty}^{\tilde{\mu}} f(x)dx = \int_{\tilde{\mu}}^{+\infty} f(x)dx \Rightarrow F(\tilde{\mu}) = G(\tilde{\mu})$$

In a finite sample, its value is given by x_m, such that:

$$\sum_{i=1}^{m} P(x_i) = \sum_{i=m}^{n} P(x_i)$$

Fashion or main tendency (or more frequent value)

Corresponds to the maximum probability of the density function (ISEL, 2015). It is obtained by operating:

$$\frac{df(x)}{dx} = 0$$

In a finite sample its value $\hat{x}$, is such that:

$$P(\hat{x}) = m\acute{a}xP(x_i)$$

12.6.2 Dispersion measures

Variance (Central moment of 2nd order)

For a continuous function, the variance is defined as:

$$\sigma^2 = \int_{-\infty}^{+\infty} (x - \mu_x)^2 f(x)dx$$

In a sample its value is calculated by:

$$s^2 = \frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}$$

Standard deviation

It is the square root of the variance that, for a continuous function, the variance is defined as:

$$\sigma = \sqrt{\sigma^2}$$

In a sample its value is calculated by:

$$s' = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}} = \sqrt{\frac{\sum_{i=1}^{n}({x_i}^2)}{n-1} - \frac{\left(\sum_{i=1}^{n} x_i\right)^2}{n(n-1)}}$$

Coefficient of variation

It is the quotient between the standard deviation and the average that, for a continuous function, the variance is defined as:

$$\eta_v = \frac{\sigma}{\mu}$$

In a sample its value is calculated by:

$$c_v = \frac{s}{\bar{x}}$$

c_v relativizes the value of the standard deviation by dividing it by the average value of the magnitude. In the following figure, it is possible to observe the difference between two distributions of the same type with the same average, but different standard deviation (and consequently coefficient of variation) (ISEL, 2015).

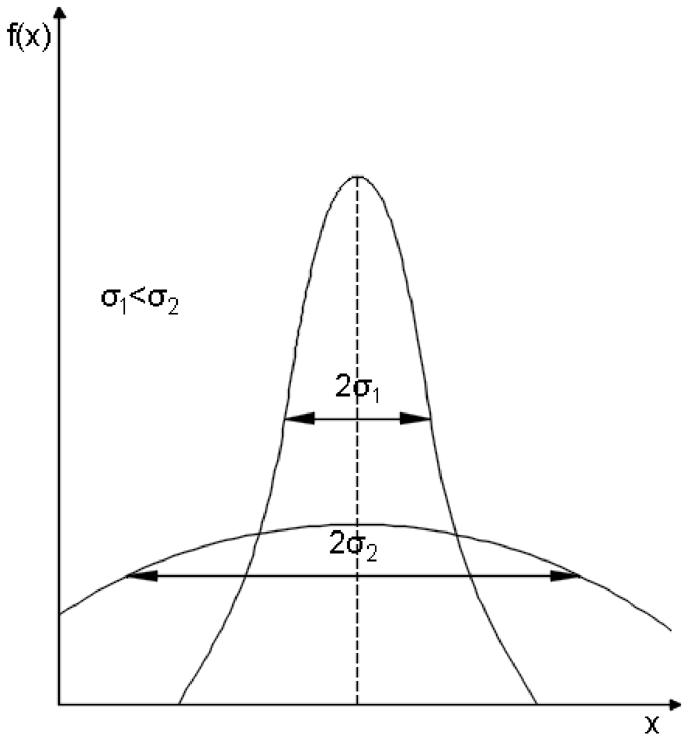

Figure 117 – Difference between two distributions of the same type with the same average, but different standard deviation (source: Author).

12.6.3 Asymmetry measures

Asymmetry (Central moment of 3rd order)

It is defined by:

$$\mu^3 = \int_{-\infty}^{+\infty} (x-\mu_x)^3 f(x)dx$$

In a sample, its values are calculated by:

$$m^3 = \frac{n}{(n-1)(n-2)}\sum_{i=1}^{n}(x_i-\bar{x})^3$$

Coefficient of asymmetry

It is defined by:

$$\gamma = \frac{\mu^3}{\sigma^3}$$

In a sample, its value is calculated by:

$$c_a = \frac{m^3}{s^3}$$

A null asymmetry coefficient indicates a symmetrical distribution, where the average and median coincide: $\mu = \tilde{u}$ (ISEL, 2015).

A positive asymmetry coefficient indicates a distribution with positive asymmetry, in which the average value is higher than the median: $\mu > \tilde{u}$ (ISEL, 2015).

A negative asymmetry coefficient indicates a negative asymmetry distribution, in which the average value is lower than that of the median: $\mu < \tilde{u}$ (ISEL, 2015).

12.7 Distribution models (Probability laws)

Exists several distribution functions (also known as probability laws), which seek to describe the frequencies of the experimental results, of which the following laws of continuous distributions stand out:

- Normal distribution (Gauss);
- Logarithmic-Normal distribution (Galton);
- Gumbel distribution.

12.7.1 Normal distribution (or Gauss's law)

It is the most common and important statistical distribution. It adapts well to many natural phenomena, namely some hydrological parameters, such as annual rainfall and the average annual flow rate (ISEL, 2015).

This distribution law establishes, that the most frequent values (which correspond to the highest probabilities) are around the average of the random variable; as more distant the values are from the average, the less frequent are such values (ISEL, 2015).

A random variable X has a normal distribution, if the **probability density function** is given by:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}}$$

where μ and σ are parameters, respectively the average and standard deviation of the distribution (ISEL, 2015).

Schematically, this distribution has a bell-shaped **probability density curve**, symmetric around the average, with horizontal asymptote on the abscissa axis (ISEL, 2015).

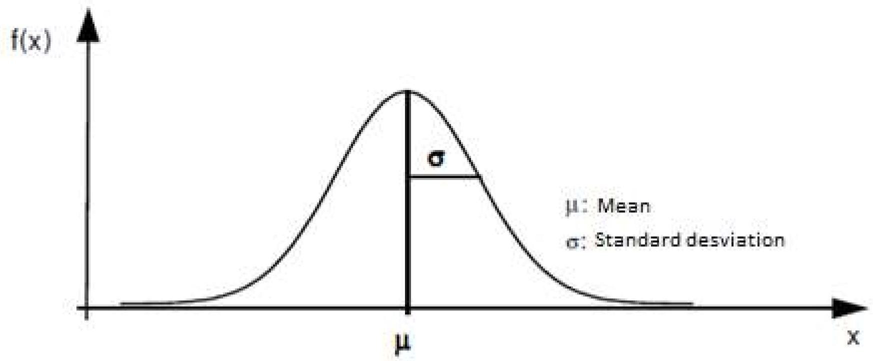

Figure 118 – Normal distribution (ISEL, 2015).

Since the normal law is symmetric, its coefficient of asymmetry is zero (ISEL, 2015).

The **distribution function** is the integral of the probability density function:

$$F(x) = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{x} e^{-\frac{(x-\mu)^2}{2\sigma^2}} dx$$

The following figure shows the probability density curve of the normal law and its distribution function, for some combinations of μ and σ^2 (ISEL, 2015).

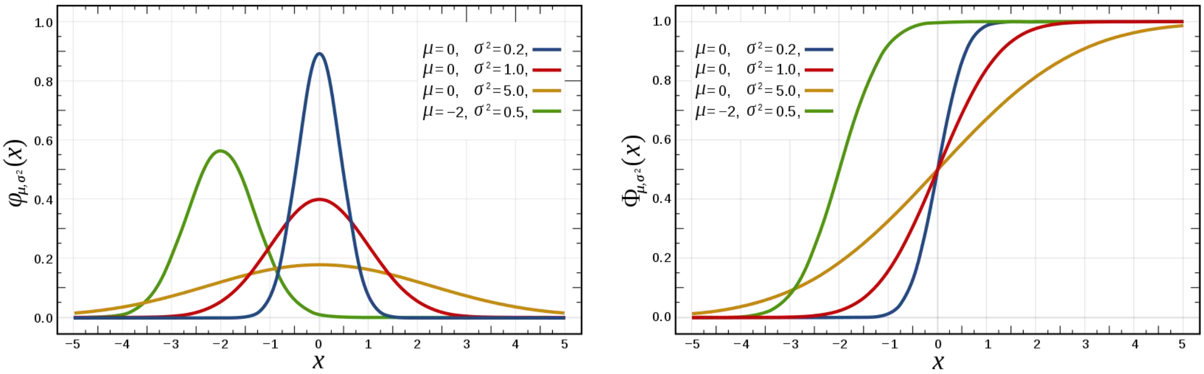

Figure 119 – Density of probability and respective distribution functions (http://www.wikiwand.com).

The parameter μ "locates" the curve above the xx axis, and the parameter σ define the "form" of such curve (ISEL, 2015).

Considering a variable $u = x - \mu/\sigma$ (called **reduced variable**), the distribution function takes the form:

$$F(x) = \varphi(u) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} e^{-\frac{u^2}{2}} du$$

Standard normal distribution is the normal distribution with a null average and unit standard deviation, that is, $\mu = 0$ and $\sigma = 1$ (in the previous figure corresponds to the continuous and red colour curves) (ISEL, 2015).

This standard normal distribution can be tabulated, with the values of $\varphi(u)$ as a function of u, allowing the calculus of the value of $F(x)$ (probability for a variable X to take values $\leq x$), since it is possible to estimate from the sample, the values for the average μ and for the standard deviation σ, (ISEL, 2015).

The estimate x of the average μ and the estimate s' of the standard σ of such distribution are calculated from the values of the sample, by:

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

$$s' = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}}$$

Instead of tables, you can also use numeric calculation, using an expression that gives the approximate value of $\varphi(u)$:

$$\varphi(u) = 1 - f(u)(0.4361836t - 0.120167t^2 + 0.9372980t^3)$$

$$f(u) = \frac{1}{\sqrt{2\pi}} e^{-\frac{u^2}{2}};\ t = \frac{1}{1 + 0.33267|u|}$$

Table 34 – Values of φ(u)=F(x), for the distribution function of the normal law (μ=0; σ=1) (ISEL, 2015).

u	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998

For negative values of u, use the arithmetic complement for one of the values of $\varphi(u)$ corresponding to a positive value:

$$\varphi(-u) = 1 - \varphi(u)$$

Example:

$$\varphi(-1) = 1 - \varphi(1) = 1 - 0.8413 = 0.1587$$

For values of $\varphi(u) < 0.5$, calculate $1 - \varphi(u)$, check the value of u and turn it negative (ISEL, 2015).

Example:

$$\varphi(u) = 0.0668; 1 - \varphi(u) = 0.9332; u = -1.5$$

12.7.2 Logarithmic-normal distribution (Galton)

A variable follows Galton's law or log-normal law, when its y transformed, defined by $y = \log x$, shows a normal distribution (ISEL, 2015).

The distribution function of variable X is:

$$F(x) = \varphi(u) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} e^{-\frac{u^2}{2}} du$$

Such variable X can take values between zero and the corresponding to the field or domain of the transformed variable $(-\infty, +\infty)$, where the reduced normal variable is defined by:

$$u = \frac{y - \mu_y}{\sigma_y}$$

The logarithmic-normal law can be represented by a straight line in a logarithmic-normal graph with the axis of normal gradation probabilities and the second logarithmic graduation axis, where x is marked (ISEL, 2015).

The estimator M and S of the parameter μ_y and σ_y, have the following expression:

$$M = \frac{\sum_{i=1}^{n} \log x_i}{n}; \ S = \sqrt{\frac{\sum_{i=1}^{n} \log^2 x_i}{n-1} - \frac{(\sum_{i=1}^{n} \log x_i)^2}{n(n-1)}}$$

And the estimative of x of a value of probability(x) is such that:

$$\log x = M + uS$$

where u is the value of the reduced normal variable corresponding to the probability $F(x)$ (ISEL, 2015).

In summary, the application of the logarithmic-normal law to values of a sample of variable X, coincides with the application of the normal law to the logarithms of those values (ISEL, 2015)**.**

12.7.3 Gumbel Law

Gumbel's law has been adopted to represent the distribution of the annual maximum values of the peak flow rate (or the annual maximum daily flow rate) or intense precipitations with a certain duration (ISEL, 2015).

The distribution function is expressed by:

$$F(x) = e^{-e^{-0{,}577-\frac{k\pi}{\sqrt{6}}}}$$

If the parameters of the distribution are estimated according to the method of moments, the law is expressed as:

$$x = \mu + k\sigma$$

where μ and σ has the typical averaging, and are estimated by $\bar{x}$ and s', and k is the factor of probability, which depends of $F(x)$, and have an analogous averaging to the reduced variable u of the normal law (ISEL, 2015). The factor k is calculated by:

$$k = \frac{\sqrt{6}}{\pi}(\ln\ln\frac{1}{F(x)} - 0{,}577216)$$

which is already calculated for different values of $F(x)$ in Table 35 (ISEL, 2015).

Table 35 – Factor of probability (k) of the Gumbel Law (x=μ+kσ) (ISEL, 2015).

$F(x)$	k	$F(x)$	k	$F(x)$	k	$F(x)$	k
0.0001	-2.181	**0.10**	-1.100	**0.80**	0.719	**0.9998**	6.191
0.0002	-2.120	**0.20**	-0.821	**0.90**	1.305	**0.9999**	6.731
0.0005	-2.032	**0.25**	-0.705	**0.95**	1.866		
0.001	-1.957	**0.30**	-0.595	**0.98**	2.593		
0.002	-1.874	**0.40**	-0.382	**0.99**	3.137		
0.005	-1.750	**0.50**	-0.164	**0.995**	3.679		
0.01	-1.641	**0.60**	0.074	**0.998**	4.395		
0.02	-1.514	**0.70**	0.354	**0.999**	4.936		
0.05	-1.306	**0.75**	0.521	**0.9995**	5.476		

12.8 Statistical analysis of random hydrological variables

Statistical methods are only applicable to random variables (ISEL, 2015). The following series of values are in such condition (among others):

- With a distribution proximal to a normal law:
  - Annual precipitation;
  - Precipitation in a certain month of the calendar;
  - Annual flow;
  - Flow in a certain month of the calendar.
- With distribution proximal to Gumbel Law:
  - Annual maximum precipitation with a certain duration;
  - Maximum instantaneous flow rate;
  - Daily average maximum flow rate.

REFERENCES

Agostinho, N. S. (2009). *Instrumentos Meteorológicos de uma Estação Clássica Virtual no Ensino das Ciências.* Aveiro: Universidade de Aveiro. Obtained from https://core.ac.uk/download/pdf/15562878.pdf

Almeida, A. B. (2001). *Curso sobre Operação e Segurança de Barragens. Capítulo 7. Emergências e Gestão do Risco.* Lisboa. Obtained from http://www.civil.ist.utl.pt/~joana/artigos%20risco%20ABA/pub-2001/capitulo-7-livro-curso%20INAG2001.pdf

Andjelkovic, I. (2001). *Guidelines on non-structural measures in urban flood management.* Paris: UNESCO. Obtained from http://unesdoc.unesco.org/images/0012/001240/124004e.pdf

Andrade, F. (2014). *CC54Z - Hidrologia. Geração de escoamento e análise de hidrograma.* Curitiba: UTFPR. Obtained from http://paginapessoal.utfpr.edu.br/fandrade/teaching/files/aula_8_escoamento.pdf

ANPC. (2018). *Riscos e Prevenção. Cheias*. Obtained from ProCiv: http://www.prociv.pt/pt-pt/RISCOSPREV/RISCOSNAT/CHEIAS/Paginas/default.aspx

APA. (2014). *Questões Significativas da Gestão da Água (QSiGA). Região Hidrográfica do Sado e Mira (RH6).* Amadora: APA. Obtained from https://www.apambiente.pt/_zdata/Politicas/Agua/PlaneamentoeGestao/PGRH/PGRH_ParticipacaoPublica/PGRH_2_QSiGA/RH6_QSiGA.pdf

Area Mecânica. (2011). *Ingeniería Mecánica: Curvas características de una bomba centrífuga (II).* Obtained from Area Mecânica: https://areamecanica.wordpress.com/2011/06/16/ingenieria-mecanica-curvas-caracteristicas-de-una-bomba-centrifuga-ii/

ARM. (2018). *Fornecimento de água aos municípios.* Obtained from ARM - Águas e Resíduos da Madeira, S.A.: http://www.aguasdamadeira.pt/

Armand Colin. (2006). *L’Atlas du Monde diplomatique.* Paris: Armand Colin.

Baliño, J. L. (2017). *Equações de Navier-Stokes.* São Paulo, Brasil: USP.

Barbosa, J. N. (1982). *Mecânica dos fluidos e Hidráulica Geral.* Porto Editora.

Barbosa, J. N. (1985). *Mecânica dos fluidos e Hidráulica Geral 2.* Porto Editora.

Braga, J. (2014). *Hidráulica II.* U.M.A.

Bruijnzeel, L. A. (2001). Hydrology of tropical montane cloud forests: A Reassessment. *Land Use and Water Resources Research*, 1.1-1.18.

Brunetti, F. (2008). *Mecânica dos Fluídos* (2nd ed.). São Paulo: Pearson. Prentice Hall.

C., F., S., P., Sequeira, M., Perestrelo, A., & Silva, M. O. (2006). *Estudo da precipitação oculta em dois tipos de vegetação da ilha da Madeira.* Coimbra: APRH. Obtained from http://www3.uma.pt/sprada/documentos/academicos/8%20congresso%20da%20agua.pdf

Camacho, R. F. (2015). *Caracterização, simulação (à escala) e modelação do escoamento em canais artificiais: aplicação a caso de estudo.* Funchal: UMa. Obtained from https://digituma.uma.pt/handle/10400.13/898

Carneiro, C. (2007). Viagem virtual ao Aqüífero Guarani em Botucatu (SP): Formações Pirambóia e Botucatu, Bacia do Paraná. *Terræ Didatica, 3*(1), 50-73. Obtained from https://edisciplinas.usp.br/pluginfile.php/1216748/mod_resource/content/1/Bacia%20do%20Paran%C3%A1.pdf

Castro, S. B., & Carvalho, T. M. (2009). Análise morfométrica e geomorfologia da bacia hidrográfica do rio Turvo - GO, através de técnicas de sensoriamento remoto e geoprocessamento. *Scientia Plena*, 1-7. Obtained from https://ufrr.br/mepa/phocadownload/morfometria%20bacia%20castro%20e%20carvalho%202009.pdf

Cecílio, R. A., Martinez, M. A., Pruski, F. F., & Silva, D. D. (2013). Modelo para estimativa da infiltração de água e perfil de umidade do solo. *Revista Brasileira de Ciência do Solo*. Obtained from http://www.scielo.br/scielo.php?script=sci_arttext&pid=S0100-06832013000200012

Ciência Viva. (2006). *Hidrogeologia. Água Subterrânea - Conhecer para Proteger e Preservar.* Lisboa: Ciência Viva.

Coelho, J. G., Brasil Junior, A. C., & Noleto, L. (2006). *Escoamento Turbulento em Difusores.* Rio de Janeiro: EPTT.

Costa, A. S. (2003). *Turbinas Hidráulicas e Condutos Forçados.* Santa Catarina: UFSC. Obtained from http://www.labspot.ufsc.br/~simoes/dincont/turb-hidr-2003.pdf

Costa, T. d., & Lança, R. (2011). *Capítulo I. Hidrologia de Superfície.* Faro: Instituto Superior de Engenharia da Universidade do Algarve. Obtained from http://w3.ualg.pt/~rlanca/sebenta-hid-aplicada/sebenta_de_hidraulica_aplicada_2011_03_01.pdf

CPRM. (2004). *Definição da planície de inundação da cidade de Valadares.* Belo Horizonte. Obtained from http://www.cprm.gov.br/publique/media/def_gov_vala.pdf

CPRM. (2008). *Hidrologia - Conceitos e Aplicações.* Brasil: CPRM. Obtained from rigeo.cprm.gov.br/jspui/bitstream/doc/14818/3/livro_hidrogeologia_conceitos.pdf

Crockford, R. H., & RichardsonI, D. P. (2000). Partitioning of rainfall into throughfall, stemflow and interception: effect of forest type, ground cover and climate. *Hydrol. Process*, 2903-2920.

Cruz, J. (2006). *Aproveitamentos Hidroelétricos.* Lisboa: IST.

DRHGSA. (2007). *Capítulo II. Orifícios.* Piauí, Brasil: UFPI. Obtained from http://leg.ufpi.br/subsiteFiles/ct/arquivos/files/pasta/CAP%202.pdf

Eisberg, R., & Resnick, R. (1979). *Física Quântica - Átomos, Moléculas, Sólidos, Núcleos e Partículas* (13th ed.). Editora Campus.

Escola da Vida. (2018). *Análise dimensional e leis de semelhança aplicadas as bombas hidráulicas.* Obtained from Escola da Vida: http://www.escoladavida.eng.br/mecfluquimica/analise_dimensional.htm

Estúdio Conejo. (2014). *Geografia.* Zênite. Obtained from https://issuu.com/estudioconejo/docs/book_fc30dd0ea6b760/41

Filho, J. G. (2015). *Turbinas Hidráulicas - Tipos e Usos.* Campinas: Universidade Estadual de Campinas. Obtained from https://docslide.com.br/documents/turbinas-hidraulicas-55ef3ff5660bb.html

Fletcher, B. P., & Grace, J. L. (1972). Practical guidance for estimating and controlling erosion at culvert outlets. *U.S. Army Engineer Waterways Experiment Station*.

Gaspar, R. (2005). *Mecânica dos Materiais.* São Paulo, Brasil: Centro Universitário Nove de Julho.

Gobbi, M., Dias, N. L., Mascarenhas, F., & Valentine, E. (2011). *Introdução à Mecânica dos Fluídos e aos Fenómenos de Transporte.* Paraná, Brasil: UFRP.

Gonzalez, J. (2000). Monitoring cloud water interception in a tropical montane cloud forest of the south-western Colombian Andes. *Adv. Environ. Monitoring and Modelling*, 97-117.

Henn, É. A. (2006). *Máquinas de Fluido* (2nd ed.). Santa Maria: UFSM.

Henriques, J. D., Palma, J. C., & Ribeiro, Á. S. (2006). *Medição de flow rate em sistemas de abastecimento de água e de saneamento de águas residuais.* Lisboa: LNEC.

Hipólito, J. R., & Vaz, Á. C. (2011). *Hidrologia e Recursos Hídricos.* Lisboa: IST.

Holder, C. D. (2003). Fog precipitation in the Sierra de las Minas Biosphere Reserve, Guatemala. *Hydrol. Process.*, 2001-2010.

Ignácio, R. F., & Nóbrega, R. L. (2004). *Mecânica dos Fluídos. Análise Dimensional e Semelhança Dinâmica.* Paraíba, Brasil: AERH. UFCG.

ISEL. (2015). *Hidrologia.* Lisboa: ISEL. Obtained from https://issuu.com/joanadecastrocortesao/docs/hidrologia_resumo_luis_mendes

IST. (2018). *Hidrologia e Recursos Hídricos. Análise estatística aplicada à hidrologia.* Lisboa: IST. Obtained from http://www.civil.ist.utl.pt/~mps/HRH/2017_2018/HRH_17_Estatistica_Aplicada_Hidrologia_30_4_2018.pdf

IST. (2018). *Hidrologia e Recursos Hídricos. Balanço hidrológico de uma bacia hidrográfica.* Lisboa: IST. Obtained from http://www.civil.ist.utl.pt/~mps/HRH/2017_2018/HRH_5_Balanco%20Hidrologico_5_3_2018.pdf

IST. (2018). *Hidrologia e Recursos Hídricos. Ciclo hidrológico - distribuição global da água.* Lisboa: IST. Obtained from http://www.civil.ist.utl.pt/~mps/HRH/2017_2018/HRH_3_Ciclo_Hidrologico_26_02_2018.pdf

Júnior, A. R. (2015). *11. Água Subterrânea - Hidráilica de Poços.* Obtained from https://pt.slideshare.net/FbioSouza1/11-hidraulica-de-pocos

Junior, L. B. (2005). *Orifícios, Bocais e Tubos curtos.* Goiânia - GO, Brasil: PUC Goiás.

Junior, N. V., & Colvara, L. D. (2010). Os modelos mentais de alunos em relação a vetores em duas e três dimensões: uma análise da dinâmica da aprendizagem e da inadequação das avaliações tradicionais. *Ciências & Cognição*, 55-69. Obtained from http://pepsic.bvsalud.org/pdf/cc/v15n2/v15n2a06.pdf

KSB. (2003). *Manual de Treinamento. Seleção e Aplicação de Bombas Centrífugas.* KSB. Obtained from https://pt.slideshare.net/EvandroTP/ksb-manual-de-selecao-e-aplicacao

Lencastre, A. (1972). *Manual de Hidráulica Geral.*

Lencastre, A. (1983). *Hidráulica Geral.* Lisboa: Hidroprojeto.

Lewis, E. V. (1988). *PNA - Principles of Naval Architecture, Volume I - Stability and Strength.* New Jersey, USA: SNAME.

Manzanares, A., & Quintela, A. (1980). *Hidráulica Geral II. Escoamentos Líquidos.* Lisboa: AEIST.

Martins, M. R. (2010). *Hidrostática e Estabilidade.* São Paulo, Brasil: USP. Obtained from http://www.ndf.poli.usp.br/~gassi/disciplinas/pnv2341/Martins_2010_Hidrost%C3%A1tica_e_Estabilidade_PNV2341.pdf

Mata-Lima, H. (2010). *Apontamentos de Hidráulica.* Funchal: UMa.

Mendonça, F. C. (2015). *Hidrometria.* São Paulo, Brasil: USP. Obtained from http://www.esalq.usp.br/departamentos/leb/disciplinas/Fernando/leb472/Aula_12/Aula%2012_Hidrometria.pdf

Miranda, J. M., & Baptista, M. A. (2006). *Riscos Naturais.* Lisboa: Instituto Dom Luiz.

Montgomery, C., & Reichard, J. (2007). *Environmental Geology.* United States: McGraw Hill.

Naghettini, M. (2012). *Introdução à Hidrologia Aplicada.* Belo Horizonte: UFMG. Obtained from https://www.researchgate.net/profile/Mauro_Naghettini/publication/306959699_Introducao_a_Hidrologia_Aplicada_ApostilaSebentaDraft/links/57c08ab808aeb95224d4a362/Introducao-a-Hidrologia-Aplicada-Apostila-Sebenta-Draft.pdf

Naghettini, M., & Portela, M. M. (2011). *Probabilidade e Estatística aplicadas à Hidrologia.* Lisboa: IST. Obtained from http://www.civil.ist.utl.pt/~mps/Mod_hid/Teoria/Cap%C3%ADtulo%20HE%20Final%20PT.pdf

Nalluri, C., & Featherstone, R. (2001). *Civil engineering hydraulics: essential theory with worked examples.* Wiley-Blackwell.

NETeF. (2012). *Análise Dimensional e Semelhança.* São Paulo, Brasil: USP. Obtained from http://www2.eesc.usp.br/netef/Oscar/Aula23

Netto, A. (1998). *Manual de Hidraulica.* Edgard Blucher Ltda.

Oliveira, R. J. (2007). *Cavitação: Como entender este fenômeno? Como identificar?* Minas Gerais: MGS - Tecnologia.

Oliveira, R. P. (2009). *Caracterização geomorfológica de uma bacia hidrográfica.* Lisboa: IST. Obtained from https://fenix.tecnico.ulisboa.pt/downloadFile/3779573911187/IST_HRH_200910_Trab1_v1.pdf

Pacífico, A. L. (2016). *Introdução à Cinemática dos Fluídos.* São Paulo, Brasil: USP.

Pelaez, J. R. (1978). Calculo Hidrometeorologico de Flow ratees Maximos en Pequenas Cuencas Naturales. Espanha: Ministerio de Obras Publicas y Urbanismo.

Peterka, A. J. (1958). *Hydraulic design of stilling basins and energy dissipators.*

Pinho, J. L., Vieira, J. M., & Lima, M. M. (2011). *Apontamentos para as Aulas de Hidráulica Geral I & II.* Minho: Universidade do Minho.

Pontes, J. d., & Mangiavacchi, N. (2013). *Fenômenos de Transferência Com Aplicações às Ciências Físicas e à Engenharia* (Vol. I). Rio de Janeiro: UFRJ.

Pordeus, R. V. (2015). *Nota de Aula: Fenômenos de Transporte. Mecânica dos Fluidos. Tipos de Regime de Escoamento .* Rio Grande do Norte, Brasil: UFERSA. Obtained from http://www2.ufersa.edu.br/portal/view/uploads/setores/111/CAP%20V%20TIPOS%20E%20REGIME%20DE%20ESCOAMENTO.pdf

Portela, M. M. (2005). *Modelação Hidrológica.* Lisboa: IST. Obtained from http://www.civil.ist.utl.pt/~mps/Mod_hid/Teoria/teoria%202007%20para%20jpg.pdf

Porto, R. L., & Filho, K. Z. (2005). *PHD 2307 - Hidrologia Aplicada.* São Paulo: Escola Politécnica da USP. Obtained from www.pha.poli.usp.br/LeArq.aspx?id_arq=6535

Porto, R. L., Filho, K. Z., & Marcellini, S. S. (1999). *PHD 307- Hidrologia Aplicada.* São Paulo: USP. Obtained from www.pha.poli.usp.br/LeArq.aspx?id_arq=7814

Possas, M. (2 de novembro de 2011). *Água: uso consuntivo e não-consuntivo.* Obtained from Fonte Hídrica: http://fontehidrica.blogspot.com/2011/11/agua-uso-consuntivo-e-nao-consuntivo.html

Prada, S., Perestrelo, A., Sequeira, M., Nunes, A., Figueira, C., & Cruz, J. V. (2005). Disponibilidades Hídricas da Ilha da Madeira. *AQUAMAC - Técnicas y métodos para la gestion sostenible del agua en la Macaronesia*, 261-294. Obtained from https://geodiversidade.madeira.gov.pt/images/documentos/Publicacoes/Madeira/Disponibilidades_Hidricas_da_Ilha_da_Madeira.pdf

Queiroz, G. (2017). *Vertedores.* Minas Gerais, Brasil: UFOP. Obtained from http://www.em.ufop.br/deciv/departamento/~gilbertoqueiroz/CIV225-Vertedores.pdf

Quintela, A. C. (1996). *Hidrologia e Recursos Hídricos.* Lisboa: IST. Obtained March 4th, 2017, from http://www3.uma.pt/sprada/documentos/aulas/Hidrologia_Recursos_Hidricos/biblio/HRH_Quintela.pdf

Quintela, A. d. (2005). *Hidráulica* (9th ed.). Lisboa: Fundação Calouste Gulbenkian.

Ramalho, F. J., Nicolau, G. F., & Toledo, P. A. (2015). *Os Fundamentos da Física* (11th ed.). São Paulo, Brasil: Moderna.

Rijo, M. (2010). *Canais de Adução - Projecto, Operação, Controlo e Modernização* (1st ed.). Lisboa: Edições Sílabo.

Rodrigues, C. M., Moreira, M., & Guimarães, R. C. (2011). *Apontamentos para as aulas de hidrologia.* Évora: Departamento de Engenharia Rural. Obtained from https://dspace.uevora.pt/rdpc/bitstream/10174/4721/1/Apontamentos_hidrologia.pdf

Rodrigues, S. P. (2017). *Aplicação de metodologias SIG à avaliação da perigosidade de inundação fluvial. O caso da cidade de Tomar (estudos preliminares).* Tomar: Instituto Politécnico de Tomar. Obtained from https://comum.rcaap.pt/bitstream/10400.26/.../relatorio%20Sergio%20Rodrigues.pdf

Sansone, E. C. (2014). *Porosidade das Rochas.* Brasil: USP. Obtained from https://edisciplinas.usp.br/pluginfile.php/1678273/mod_resource/content/1/03%20-%20PMI1673%20-%202014%20-%20Porosidade%20das%20Rochas.pdf

Silva, G. Q. (2014). *Estudo dos Orifícios e Bocais.* Minas Gerais, Brasil: UFOP.

Simões, J. G. (2011). *Condutos livres. Escoamento uniforme em canais.* São Paulo, Brasil: Unisanta.

Smith, H. (1886). *Hydraulics: The Flow of Water Through Orifices, Over Weirs, and Through Open Conduits and Pipes.* Truebner and co.

SNIRH. (2018). *Sistemas Aquíferos.* Obtained from SNIRH: https://snirh.apambiente.pt/index.php?idMain=4&idItem=3&idISubtem=link2

Soares Júnior, R. L. (2013). *Projeto Conceitual de uma Turbina Hidráulica a ser Utilizada na Usina Hidrelétrica Externa de Henry Borden.* Rio de Janeiro: UFRJ.

Soares, A. S. (2014). *Análise Experimental das Condições de Escoamento Superficial em Zonas Urbanas na Presença de Viaturas.* Lisboa: UNL.

Soares, H. (2011). *Hidráulica Geral.* Minas Gerais, Brasil: UFJF. Obtained from http://slideplayer.com.br/slide/5382800/

Sousa, E. R. (2001). *Saneamento Ambiental I. Captações de água.* Lisboa: IST. Obtained from https://fenix.tecnico.ulisboa.pt/downloadFile/3779573893840/AG_Captacoes.pdf

Tavares, I. T. (2014). *A Influência de Difusor na Eficiência de uma Turbina Hidráulica.* Brasília, DF: UnB.

Teixeira, E. (2003). *Previsão dos valores de pressão junto ao fundo em bacias de dissipação por ressalto hidráulico.* Rio Grande do Sul, Brasil: UFRGS.

UALG. (2018). *Guia do princípio de Arquimedes.* Obtained from UALG. Guias: https://pt.wikipedia.org/wiki/Impuls%C3%A3o

UEL. (2017). *Resumo de Hidráulica - Introdução.* Obtained from ebah: http://www.ebah.pt/content/ABAAAAICkAF/resumo-hidraulica-introducao?part=3

UN/SA Population Division. (2015). *World Population Prospects: The 2015 Revision.* New York: UN. Obtained from https://esa.un.org/unpd/wpp/publications/files/key_findings_wpp_2015.pdf

Universidade Federal do ABC. (2013). *Alimentação via turbo-bombas.* Universidade Federal do ABC. Obtained from https://pt.slideshare.net/edpackness/liquid-propellant-rocket-engine-motor-foguete-liquido-part11

UTFPR. (2005). *Hidrologia. Capítulo 5 - Medição de Vazão.* Curitiba, Paraná, Brasil: UTFPR. Obtained from http://paginapessoal.utfpr.edu.br/eudesarantes/disciplinas/hidrologia/Medicaodavazao.pdf/at_download/file.

Vasconcelos, M. M. (2005). *Hidráulica Geral I.* Évora: Universidade de évora.

Villa, A. A. (2011). *Análise Dimensional e Semelhança.* Pernambuco, Brasil: UFPEP.

White, H. E. (1948). *Modern College Physics.* van Nostrand.

Winck, N. B. (2015). *Avaliação hidrogeológica, hidroquímica e da ocorrência de Ferro e Manganês nas águas subterrâneas do distrito de Ipiranga, região noroeste do município de Gravataí.* Porto Alegre: UFRGS. Obtained from https://www.lume.ufrgs.br/bitstream/handle/10183/170419/001052403.pdf?sequence=1