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HYDROLOGY, WATER RESOURCES AND ENVIRONMENT

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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 amount
ρ	Density

ABBREVIATIONS

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

EXERCISES

Chapter 1 - Hydrologic Cycle

EXERCISES

1.1. The total volume of freshwater available on Earth is around of $35 \times 10^6\ km^3$. From that volume, approximately 30% remains approximately an average of $1400\ years$ in aquifers and 0.006% remains approximately an average of $16\ days$ in rivers. Calculate the average volume of annual renewal in both reservoirs (aquifers and rivers), and based in the obtained result, determinate which reservoir could be used permanently (most quantity of water).

1.2. The average annual flow of continents is about $316\ mm$. Knowing that the area of the continents is $150 \times 10^6\ km^2$ and that the flow of the Amazon river corresponds to about 12% of it, estimate the average annual flow rate of such river in m^3/s.

1.3. In Portugal Mainland, with an area of approximately $89000\ km^2$ and a population of about $10000000\ people$, the public supply of water is an average of approximately $200\ L/hab/day$. Estimate in $mm/year$ the supplied annual volume.

Chapter 2 - Hydrographic Basins

EXERCISES

2.1. In a specific basin were obtained the following data for a relief analysis:

Table 1 – Data from a basin.

Height (m)	**204**	**220**	**240**	**260**	**280**	**300**	**306**
Area (km^2)	23.05	22.84	16.81	9.32	2.07	0.57	0.00

Calculate the basin average height.

2.2. The area of a basin is $102\ km^2$ and the sum of the lenght of its water courses is $300\ km$, in a certain cartographic scale. Estimate the average extension of a superficial flow.

2.3. The next table shows the number of watercourses segments by order according to Strahler's stream order.

Table 2 – Number of watercourses of order i.

Order, i	**1**	**2**	**3**	**4**	**5**
Number, N_i	139	46	11	3	1

Determinate the average bifurcation ratio.

2.4. In order to trace a longitudinal profile of a given water course, the following data was collected:

Table 3 – Data for a longitudinal profile.

Distance to the reference section (km)	**0**	**2**	**4**	**7**
Height (m)	103	110	130	205

Determinate the water course average slope and equivalent slope.

Chapter 3 - Hydrologic balance of a basin

EXERCISES

3.1. In a $100\ km^2$ hydrographic basin, in which are transferred approximately $8\ hm^3$ per month from a neighboring basin, the precipitation and flow of a certain hydrologic year were $1000\ mm$ and $1300\ mm$, respectively. Estimate in mm, the value of real evapotranspiration of such year. Justify your answer.

3.2. It is pretended to transfer water from a $100\ km^2$ hydrographic basin to a neighboring basin. Knowing that the average annual precipitation and evapotranspiration of the basin of origin are, respectively, $1000\ mm$ and $700\ mm$, estimate the maximum average flow rate available to transfer in m^3/s. Justify your answer.

3.3. The average annual values of precipitation and flow deficit in a $40\ km^2$ basin, were estimated as $1500\ mm$ and $850\ mm$, respectively. Determine the average annual flow rate at the reference section in such basin (m^3/s).

Chapter 4 - Precipitation

EXERCISES

4.1. The following figure shows a daily log of a siphon udograph. Knowing that the vertical scale corresponds to $10\ mm$ of precipitation, estimate the precipitation in that day.

Figure 1 – Udograph.

4.2. The accumulated hyetograph of a given precipitation is represented in the following table:

Table 4 – Data of a hyetograph.

$t\ (min)$	0	10	20	30	40	50	60
$P\ (mm)$	0	15	35	41	45	47	47

Determine the maximum average intensity of precipitation, $I\ (mm/h)$, in half an hour.

4.3. In three udometric stations with areas of influence of 10, 20 and $30\ km^2$ in a given basin, it has been registered precipitations of $12\ mm$, $18\ mm$ and $23\ mm$ in a given interval of time, respectively. Estimate the precipitation over the basin in that span of time by using the Thiessen's Method.

Chapter 5 - Superficial Flow

THEORETICAL SUPPORT

5.1 Flow in an inclined plane

Consider an inclined plane (Figure 2), of undefined width and waterproofed surface, in which occurs a precipitation with intensity equal to p (Hipólito & Vaz, 2011).

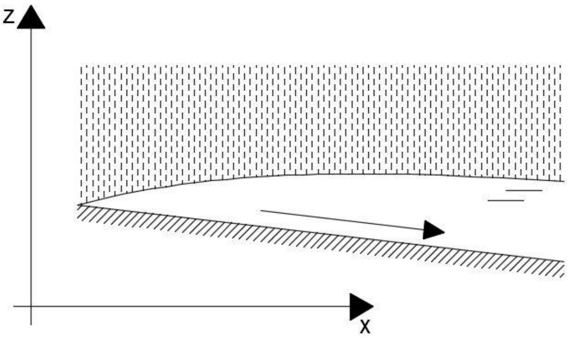


Figure 2 – Example of an inclined plane.

Admit that the **flow rate per unit of length**, q (m^2/s), is defined by

$$q = \alpha h^m$$

Where h represents the flow height, and considering the formula of Manning-Strickler,

$$\alpha = K_S S_f^{\frac{1}{2}}$$

$$m = \frac{5}{3}$$

or, when considered the formula of Chézy:

$$\alpha = C S_f^{\frac{1}{2}}$$

$$m = \frac{3}{2}$$

Although, flows on natural slopes are turbulent and very tortuous, for a laminar flow on an inclined plane, it would be obtained:

$$\alpha = \frac{\gamma}{2\mu} S_f$$

$$m = 3$$

where γ represents the water volumetric weight, and μ, its dynamic viscosity. It is verified that the exponent m, for a flow on an inclined plane, must vary between 1.5 and 3.0 (Hipólito & Vaz, 2011). S_f represents the inclination.

It is called **time of concentration (t_c)** to the time span from the beginning of a precipitation with constant intensity and undefined duration, until the created flow reaches the plane terminal section. It can be calculated by:

$$t_c = \left(\frac{L}{\alpha p^{m-1}}\right)^{\frac{1}{m}} \ ou \ t_c = \frac{h_L}{p}$$

Where:
L is the traveled distance (m);
p is the intensity of precipitation (m/h);
$h_L = h(L, t_c)$ represents the maximum flow height in the plane terminal section.

The previous equation shows that the time of concentration for a constant intensity of precipitation is as greater as the roughness of the plane (lowers K_S or C); as lower the slope of such plane, as greater as the length of such plane and smaller as the intensity of precipitation (Hipólito & Vaz, 2011). For a given flow height, the first two factors correspond to a minor **average velocity**:

$$u = \frac{q}{h} = \alpha h^{m-1}$$

The **maximum flow rate** is achieved when the duration of precipitation, with a constant intensity, is at least equal to the time of concentration:

$$q_{max} = pL$$

5.2 Measurement of superficial flows

In water resources, the volume of water that cross a certain transversal section of a water course in a given time span it is called **flow**, occurring in that time span. Thus, the volume of water that flows in a day, month or year, in a given section it is called, respectively, **daily**, **monthly** or **annual flow**. Usually, the volume of water is divided by the area of the hydrographic basin of origin, being expressed as height of water uniformly distributed in that basin. Thereby, it can be compared or related to other magnitudes as precipitation or evaporation in the same time span, and of course, also expressed in height (Hipólito & Vaz, 2011).

It is called as **average flow rate** in a given time span, the ratio between the flow, expressed in volume, and the time span in which it occurs, obtaining respectively, and according to a daily, monthly or annual flow, a **daily, monthly** or **annual average flow rate** (Hipólito & Vaz, 2011).

Thin-walled spillways

In small dimension channels as those used in watering and drainage of agriculture fields, or in small water courses, the flow rate is measured with some frequency using thin-walled spillways, rectangular or triangular, as shown in Figure 3 (Hipólito & Vaz, 2011).

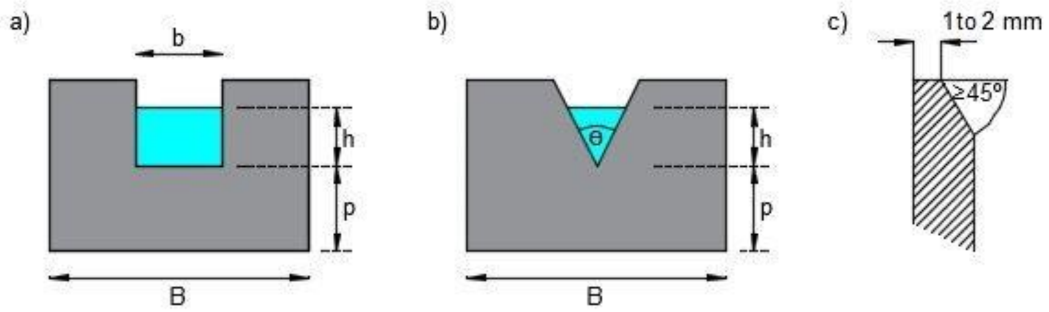


Figure 3 – Thin-walled spillways: a) rectangular; b) triangular; c) normal cut to notch edges.

The equation of a rectangular thin-walled spillways, working with free surface conditions and with guaranteed ventilation of the liquid vein in its fully contour, according to Kindsvater and Carter (ISO, 2008), is:

$$Q = C\frac{2}{3}\sqrt{2g}(b + k_b)(h + k_h)^{\frac{3}{2}}$$

Where:
Q is the flow rate that crosses the spillway in permanent regime (m^3/s);
C is the flow coefficient of the spillway (-);
g is the gravitational acceleration (m^2/s);
b is the width of the rectangular notch (m);
h is the charge above the spillway crest (m);
k_b and k_h (m) are compensation parameters for viscosity and superficial tension effects. The factors $b + k_b$ and $h + k_h$ are called effective width and effective charge, respectively.

The flow coefficient (C), function of b/B and h/p, where B is the width of the rectangular channel at upstream of the spillway, and p is the height of the thin wall from the bottom of the channel until the base of the notch. Such coefficient is defined as:

$$C = a\left(\frac{b}{B}\right) + a'\left(\frac{b}{B}\right)\frac{h}{p}$$

where a and a' are functions of b/B empirically determined, as the flow coefficient C itself. Table 5 presents the values of a and a' (Hipólito & Vaz, 2011).

Table 5 – Values of a and a' in function of b/B.

$\frac{b}{B}$	a	a'
1.0	0.602	0.075
0.9	0.598	0.064
0.8	0.596	0.045
0.7	0.594	0.030
0.6	0.593	0.018
0.5	0.592	0.010
0.4	0.591	0.0058
0.2	0.589	-0.0018
0.0	0.587	-0.0023

For different values of b/B from those presented in Table 5, the values of a and a' can be found by interpolation (Hipólito & Vaz, 2011). Alternatively, it can be used the following polynomial expressions that results from an adjustment to the referred values in Table 5:

$$a = 0{,}0367\left(\frac{b}{B}\right)^4 - 0{,}0507\left(\frac{b}{B}\right)^3 + 0{,}0218\left(\frac{b}{B}\right)^2 + 0{,}0072\left(\frac{b}{B}\right) + 0{,}587$$

$$a' = -0{,}2252\left(\frac{b}{B}\right)^4 + 0{,}4608\left(\frac{b}{B}\right)^3 - 0{,}189\left(\frac{b}{B}\right)^2 + 0{,}0320\left(\frac{b}{B}\right) - 0{,}0026$$

Table 6 presents the values of k_b in function of b/B (ISSO, 2008) (Hipólito & Vaz, 2011).

Table 6 – Values of k_b in fucntion of b/B.

$\frac{b}{B}$	k_b
1.0	-0.9
0.8	4.2
0.6	3.6
0.4	2.7
0.2	2.4
0.0	2.4

For values of b/B different from those presented in Table 6, the values of k_b can be found by interpolation. Alternatively, it can be used the following polynomial expression that results from an adjustment to the referred values in Table 6:

$$k_b = -85{,}938\left(\frac{b}{B}\right)^5 + 140{,}63\left(\frac{b}{B}\right)^4 - 76{,}563\left(\frac{b}{B}\right)^3 + 20{,}625\left(\frac{b}{B}\right)^2 - 2{,}05\left(\frac{b}{B}\right) + 2{,}4$$

Where results k_b in mm. According to the experimental results, k_h has a constant value of $0.001\ m$ (Hipólito & Vaz, 2011).

Chemical tracers

When it is not possible to measure by any other method, because the flow height are is too low, or flow velocity or turbulence are too high, it is common to use chemical tracers to estimate the flow rate in a given section of a water course (Figure 4). In this method, at an upstream section is injected a solution with a known concentration of a given solute and it is monitored, at a downstream sampling section, the evolution of this solute concentration (Hipólito & Vaz, 2011).

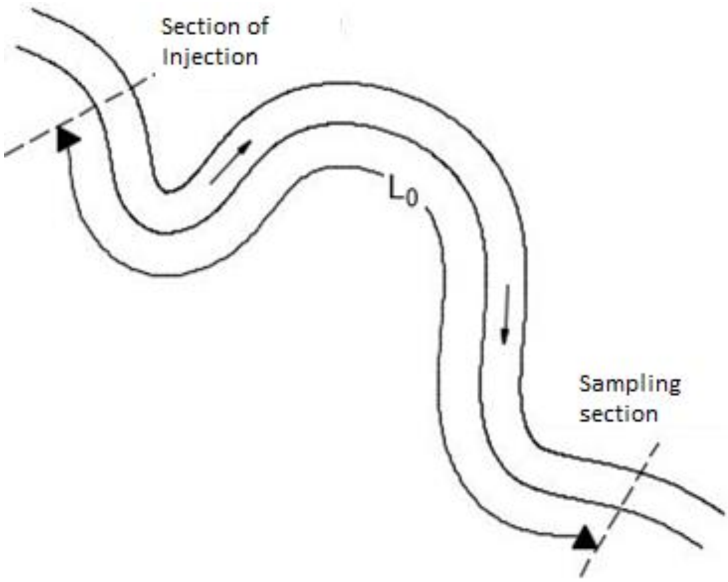


Figure 4 – Example of a section of a river used for chemical tracers.

The injection can be made abruptly or uniformly distributed over a time span and only at one point, central or marginal, or in the center of equal partitions of the width of the injection section (Hipólito & Vaz, 2011).

The monitoring of the solute concentration should be done in a section sufficiently distant from the injection point, so that the solute is already fully diluted in that cross section. To confirm this, three or more points of the sampling section may be monitored (Hipólito & Vaz, 2011).

The distance L_0 between two sections, which at least guarantees the complete dissolution of the solute, can be estimated by (Dingman, 1994):

$$L_0 = K\frac{C\bar{B}^2}{\sqrt{g\bar{y}}}$$

Where:
C represents the Chézy's coefficient ($C = K_S R^{1/6}, em\, m^{1/2}/s$);
$\bar{B}$ is the average length of the section (m);
$\bar{y}$ is the average deepness of the section (m);
g is the gravitational acceleration (m^2/s);
K is a factor that depends on the number of points and local of injections at upstream section (Table 7).

Table 7 – Factor K for the calculus of L_0.

Number of points and local of injections	$K\ (-)$
1 point in the center of the section	0.500
2 points, one in the center of each half of width	0.125
3 points, one in the center of each 1/3 of width	0.055
1 point in the margin section	2.000

The substance used as a tracer should be easily soluble, have zero or low concentration in the flow of the analyzed section, not be chemically reactive or physically absorbable by organic or mineral substances existing in the section, easy to detect even at reduced concentrations, be harmless to the observer and to aquatic life and must have a low cost. One of the most commonly used substances has been the common salt (NaCl), which can be easily detectable by calibration with the electrical conductivity of water (Hipólito & Vaz, 2011).

Assuming that the concentration of a tracer in the flow before the injection section is C_b, base concentration, and it is permanently injected a total flow rate Q_t where the concentration of the tracer or solute is C_t, when in the sampling section the balance concentration is C_e, then, the flow rate at the sampling section, Q, is:

$$Q = Q_t \frac{C_t - C_e}{C_e - C_b}$$

EXERCISES

5.1. On a certain inclined plane, with a slope of 0.05, a development according to the greater slope lines of $20\,m$ and a Strickler's coefficient of $45\,m^{1/3}/s$, occurs a precipitation with constant intensity of $40\,mm/h$. Determine the time of concentration for that precipitation and the maximum flow rate that flows by unit of width.

5.2. To measure the flow rate in a watering channel, a rectangular thin-walled spillway was installed, with $B = 0.5\,m$, $b = 0.3\,m$ and $p = 0.5\,m$. Knowing that the load over the spillway crest was $0.35\,m$, determine the flow rate of that channel.

5.3. In a given segment of a river, with an average width of $4\,m$, an average depth of $0.15\,m$ and a Chézy's coefficient of $22\,m^{1/2}/s$, a salt (NaCl) dilution was used to estimate the flow rate in the segment. The salt initial or base concentration in the river was $0.1\,g/L$, the salt concentration in the solution injected in the middle of the section in a continuous mode was $200\,g/L$, and the concentration measured in the sampling section was $2\,g/L$. Knowing that the injected flow rate was $5\,L/s$, estimate the flow rate in the segment and the amount of salt required to perform the measurement. Comment.

Chapter 6 - Flow rates

THEORETICAL SUPPORT

6.1 Study of Flow rates

Most of the major achievements in the field of hydraulic engineering relate to the exploitation of natural flow rates. Only for this reason, it is enough to make an idea of the importance of knowledge of flow rates

Absolute flow rate or caudal (Q) is the volume of water that crosses a given section per unit of time (m^3/s or L/s).

Specific flow rate or caudal (q), where A is the area of the region that contributes to create an absolute flow rate in a given section of a hydrographic or drainage basin ($m^3/s/ha$; $m^3/s/km^2$; $L/s/ha$).

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

Integral or accumulate flow rate is the volume that crosses in a given section in a given time span.

$$\int_{t_0}^{t_1} Q(t) \cdot dt = V$$

6.2 Study of the flow rate curves

What is it measured? Height of water at a given section of the river (limnigraphs), knowing a **flow rate curve** (relates the height of water measured in a section of the river with the corresponding flow rates).

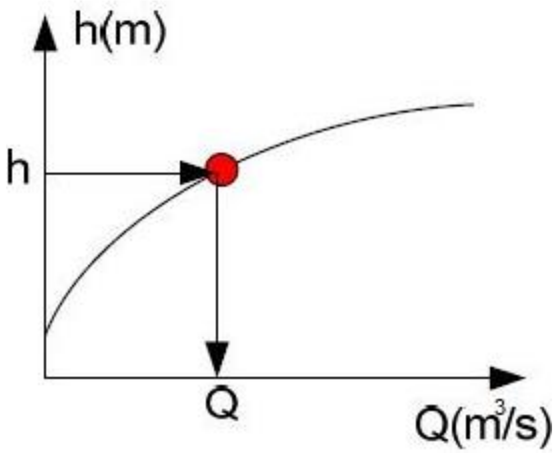


Figure 5 – Flow rate curve.

Gathering the readings in the limnigraphs with the flow curve, corresponding to that section of the river, it is obtained the **instantaneous chronological flow rates curve.**

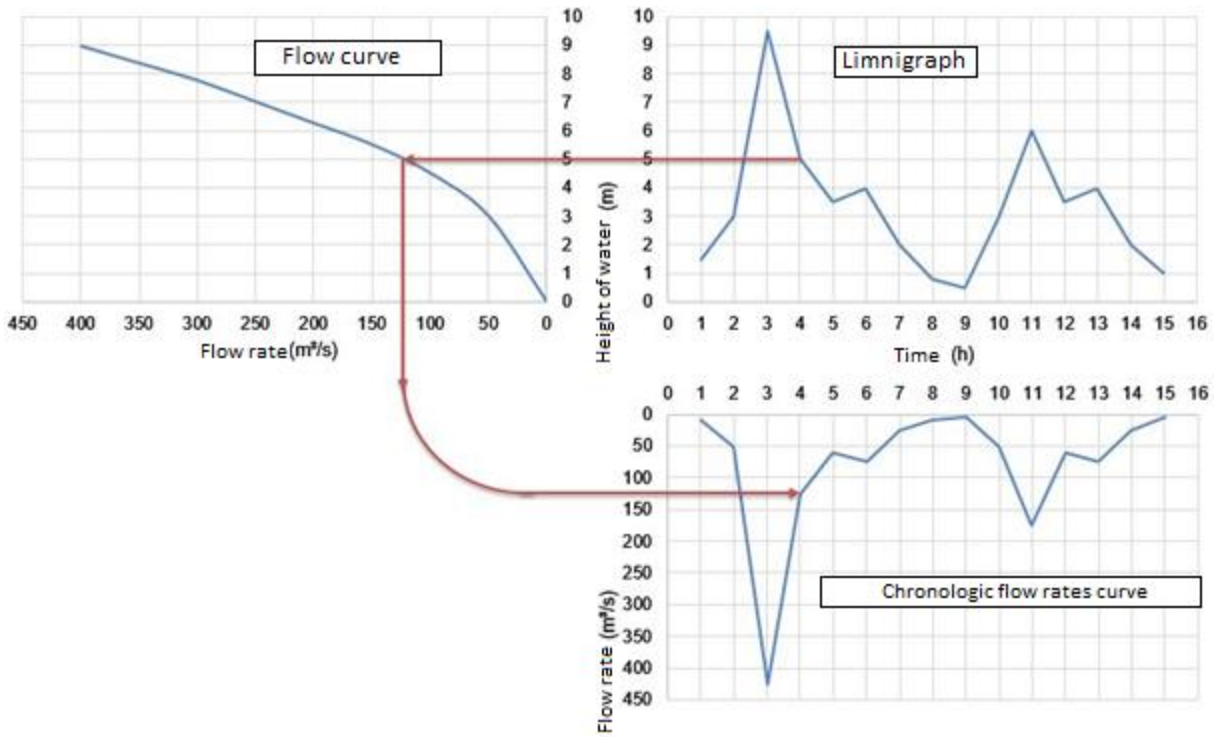


Figure 6 – Chronological instantaneous flow rates curve.

Average chronological flow rates curve (daily, monthly or annual)

For a given section it is defined as the average flow rate (Q_m), at a given time span, the constant flow rate that generates the same volume of water in that same time interval:

$$Q_m = \frac{\int_{t_0}^{t_1} Q(t) \cdot dt}{t_1 - t_0}$$

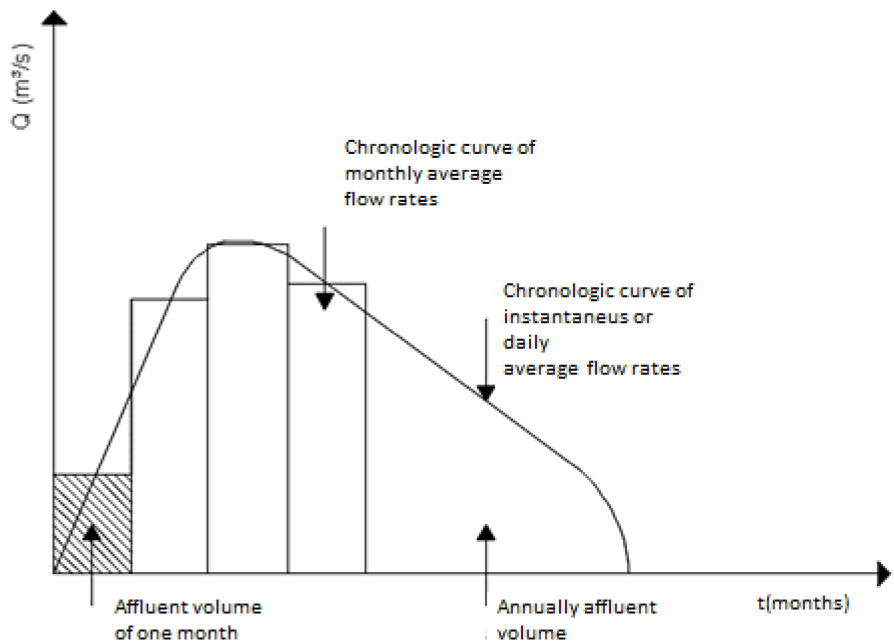


Figure 7 – Average chronological flow rates curve.

The average flow rate is such that the area of the rectangle defined by it, equals the area implied by the instantaneous flow rates diagram.

Classified flow rates curve

It is the curve that for a given period indicates the time interval in which in this period a given flow rate was exceeded or equaled. This curve is obtained from the chronological flow rates curve:

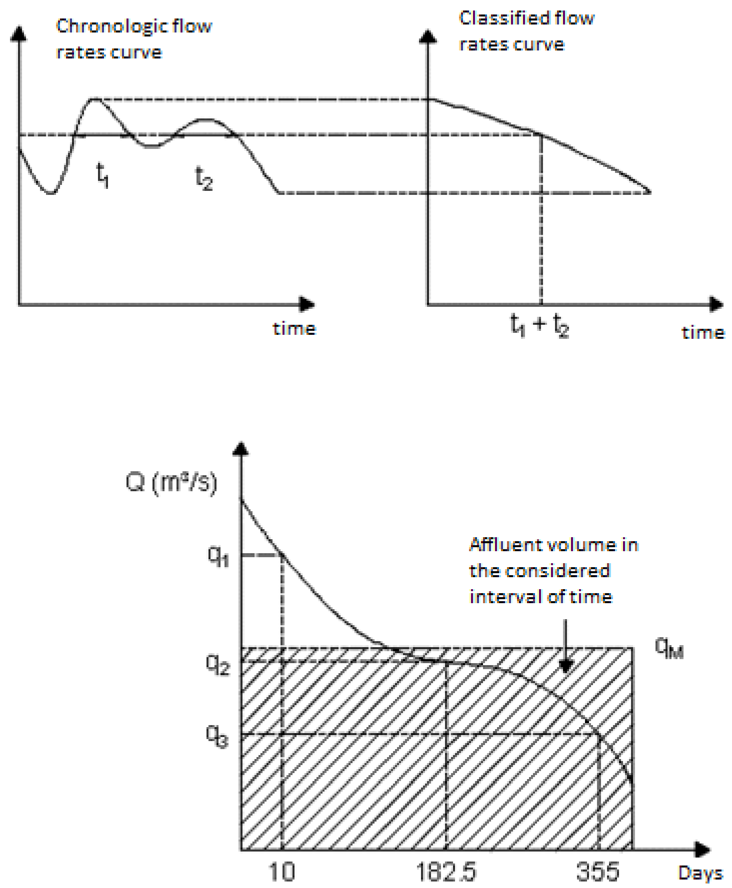


Figure 8 – Classified flow rates curve.

This curve allows to define some parameters of interest, such as the **characteristic flow rates** (well-defined values of the classified flow rates curve - Figure 8):

- **Semi-permanent flow rate** is the flow rate equaled or exceeded in 6 months of a given year (q_2);
- **Maximum characteristic flow rate** is the flow rate equaled or exceed in 10 days of a given year (q_1);
- **Minimal characteristic flow rate** is the flow rate that it is only not exceeded in 10 days of a given year (q_3);
- **Module of classified flow rates or modular flow rate** is the flow rate that defines a rectangle with the same area and base as the classified flow rates curve (q_M).

6.3 Harnessing energy (or other purposes)

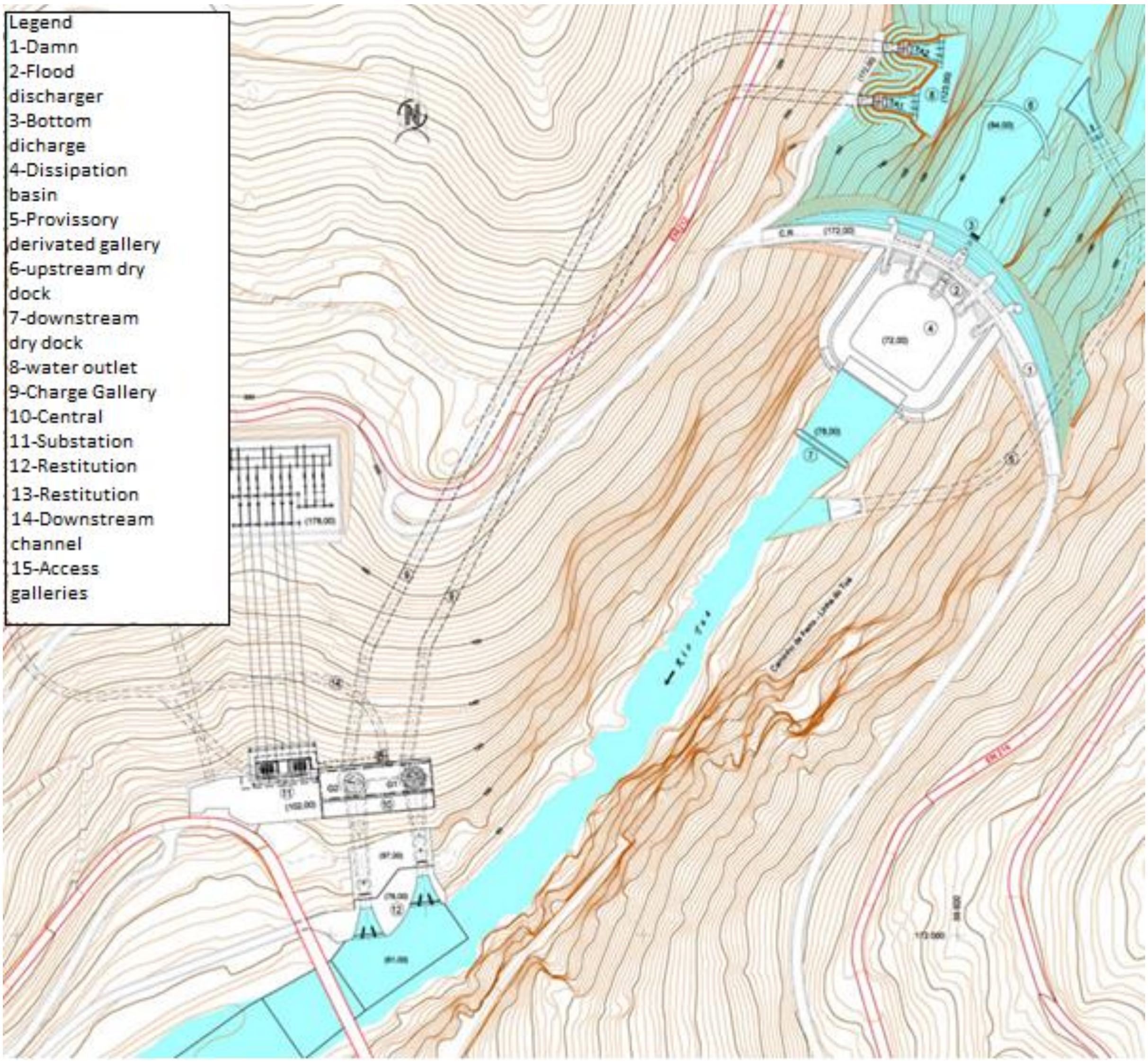


Figure 9 – Hydraulic circuit.

Hydraulic circuit (example of hydroelectric energy production):

1. Water outlet;
2. Adduction works;
3. Central;
4. Restitution works;
5. Restitution nozzle.

Types of harnessing:

- **Water thread:** There is a subjection to the natural flow rates, as they reach the section of the dam;
- **Reservoir:** it can be stored water in excess for a given time and use it later when necessary.

Harnessing by water thread

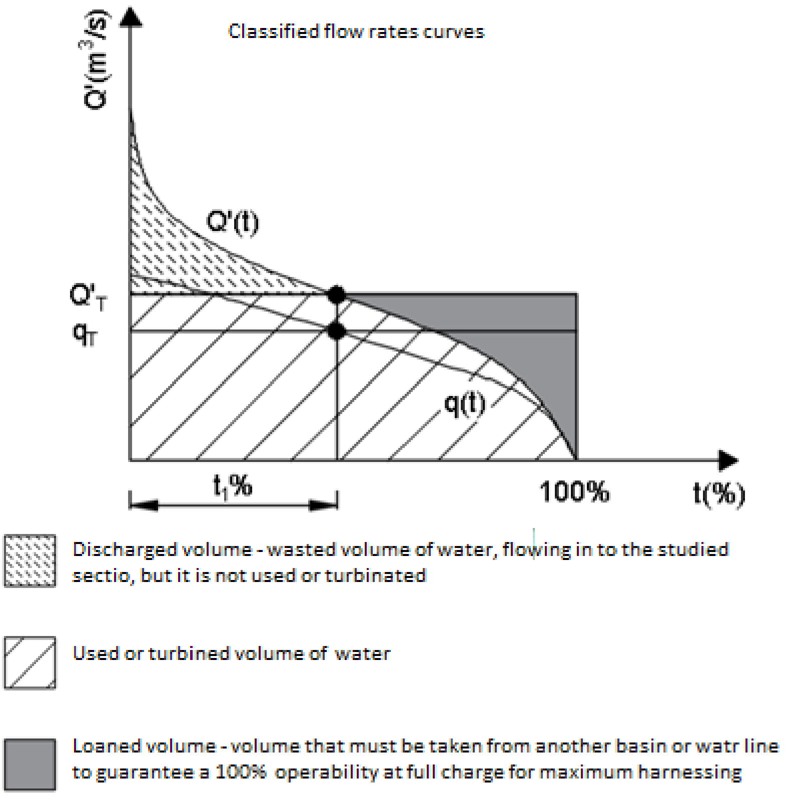


Figure 10 – Classified flow rates curve.

$$V_{used/turbined} + V_{discharged} = V_{flowed}$$

The operation at full load, that is, in which the used/turbined flow rate is the maximum usable/turbinable, occurs in $t_1\%$ of the year, period in which there is waste of water; in the remaining time, $100 - t_1\%$ of the time, the operation is done with a flow rate below the maximum usable/turbinable, that is, there is a lack of water. If the harnessing worked at full load during the entire considered period in order to consume the same volume of water as previously, it would have to consider a constant flow rate q_t. This value is designated as a module of the usable/turbinable flow rates or average usable/turbinable flow rate and delimits a rectangle with an area equal to that defined by the classified flow rates curve, below the installed flow, also designated as the classified used/turbined flow rates curve.

To each value of the installed flow rate corresponds a module of used/turbined flow rates, which can be defined as a curve, which provides the average usable/turbinable flow rates, according to the installed flow rates, which is designated as, **hydrological characteristic curve**, which derives from the classified flow rates curve relative for a given section of the river in which it is installed a harnessing procedure without storage capacity with maximum usable/turbinable flow rate or installed flow rate equal to Q_t'.

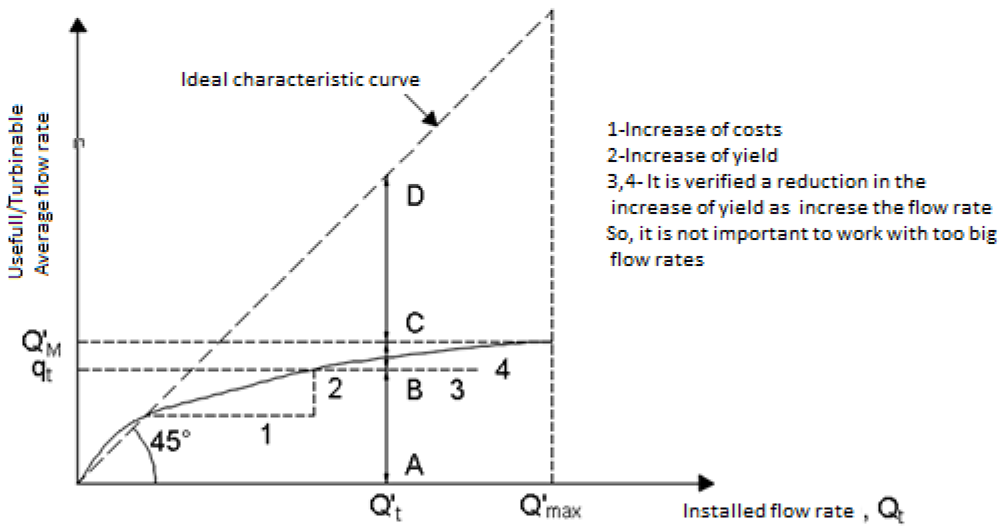


Figure 11 – Hydrological characteristic curve.

Legend:
AC - Modular flow rate;
AD - Installed flow rate;
AB - Average usable/turbinable flow rate, proportional to the volume of water used/turbined;
BC - Proportional to the discharged volume, in other words, the volume of water wasted because of an installed flow rate less than Q'_{max};
BD - Proportional to a borrowed volume, that is, the volume of water that would be necessary to provide so that the harnessing at full load should always be possible.

Observation: Allows to make a study in terms of energy and costs.

It is a curve obtained from the **classified flow rates curve**, which is the **concentration curve**. Such curve results from the integration in order to time and from the end to the origin of the classified flow rates curve. This is an accumulated flow rates curve (Volumes) that always has the concavity facing upwards.

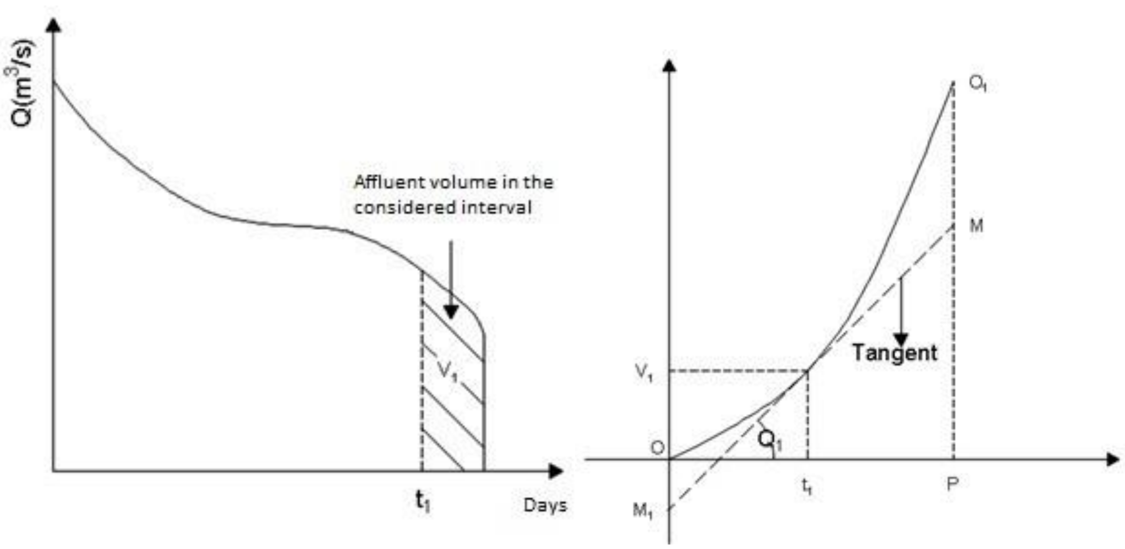


Figure 12 – Concentration curve.

In the previous graph, the marked points have the following meanings:
PO_1 - Total volume of water attributed to the section;
PM - Volume of water consumed for maximum turbinable flow rate Q_1;
MO_1 - Volume discharged, volume of water wasted by that installation;
OM_1 - Borrowing volume, volume of water that would need to be borrowed from another water line so that it can be used/turbined the flow rate Q_1, all the time.

Yield or efficiency of an installation (η)

Despite of the method or curve used, the yield is always defined as the ratio between the used volume and the tributed volume. So, based in the concentration curve, it would be:

$$\frac{PM}{PO_1} = \eta$$

Harnessing with reservoir

Accumulated or integral flow rates curve

It is the integral curve of the instantaneous flow rates. It gives in a given section and at each instant the accumulated volume from the "origin of the times":

$$V = \int_0^t Q(t) \cdot dt$$

This curve gives an idea of the possible storage to be verified in a reservoir that controls the concerned section.

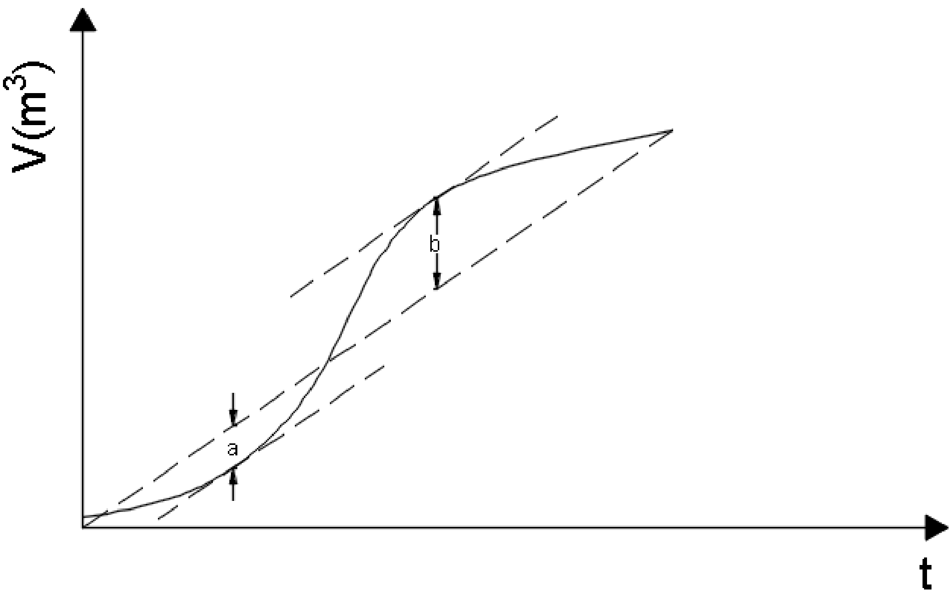


Figure 13 – Watercourse Regularization Study.

$$C = a + b$$

EXERCISES

6.1. Consider a given section of a river characterized by a flow rate curve represented by the following ratio between water height, h (m), and a flow rate, Q (m^3/s): $Q = 5 \cdot h^2$. Admit that, in a given hydrological year, the average height of water in the four trimesters of such year were successively 7, 4, 1 and 2 m; represent the following curve graphically:
a) Chronological flow rates curve;
b) Integral or accumulated flow rates curve;
c) Classified flow rates curve;
d) Hydrological characteristic curve;
e) Concentration curve.

6.2. Consider the chronological flow rates curve, Figure 14:

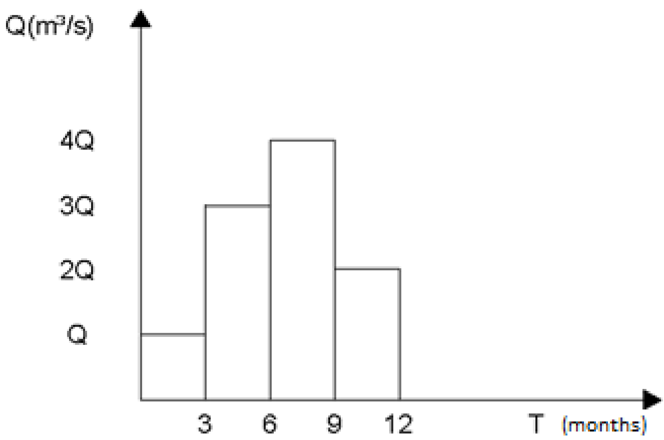


Figure 14 – Chronologic flow rates curve.

a) Determine the value of Q for an installed flow rate equal to $3Q$ and a module of used flow rates of $22.5\ m^3/s$;
b) Determine the yield of such installation using different procedures (classified flow rates curve, hydrological characteristic curve), and consider for a used/turbined volume a corresponding flow rate of $30.0\ m^3/s$.

6.3. Consider the classified flow rates curve, Figure 15:

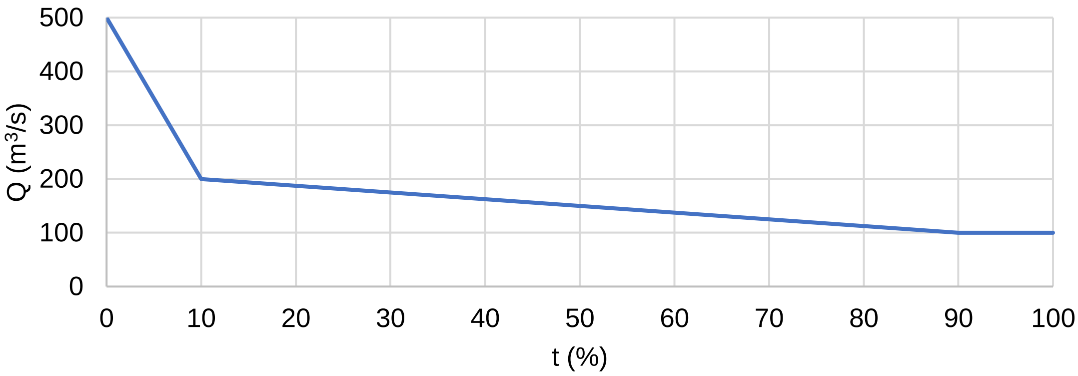


Figure 15 – Classified flow rates curve.

a) Calculate the module. Calculate the value of the semipermanent flow rate:
b) Draw the hydrological characteristic curve;
c) What range of flow rates makes the installation more economical;
d) Trace the concentration curve;
e) Knowing that the installed flow rate is equal to the semipermanent and knowing the volume annually discharged, calculate the yield of the installation;
f) Which volume would have to be borrowed from another basin so that the harnessing works all year at full load.

Chapter 7 - Study of floods

THEORICAL SUPPORT

7.1 Flood hydrographs

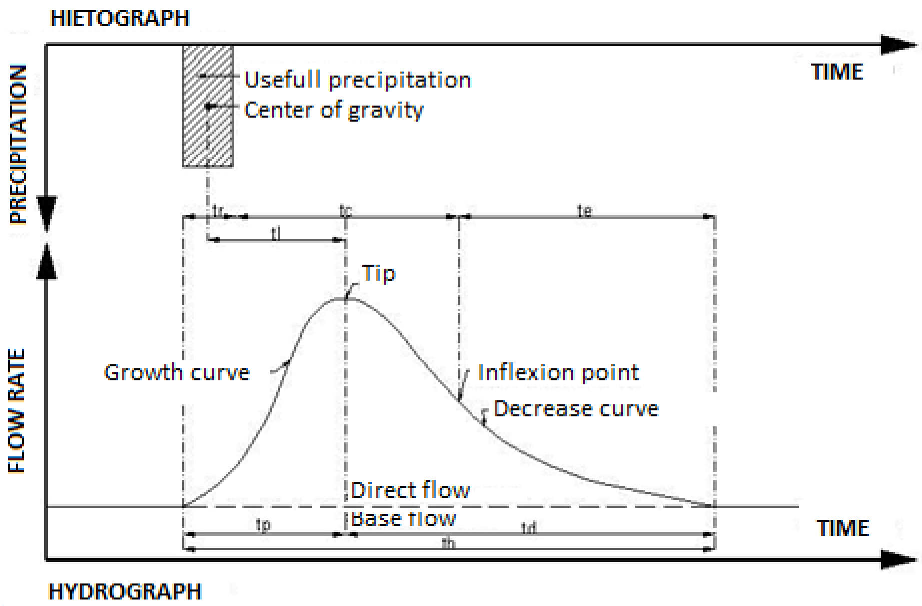


Figure 16 – Flood hydrograph (flow associated to an intense precipitation in function of time).

There are different methods to determine the hydrograph corresponding to a given precipitation, which will be addressed and analyzed.

Unitary hydrograph method

The principles used for this method only apply, exclusively, to the superficial flow fraction. The considered precipitation only represents a useful precipitation. A unitary hydrograph, to a given section of a watercourse, is the flood hydrograph produced by an effective uniform precipitation of unitary intensity and unitary precipitation time.

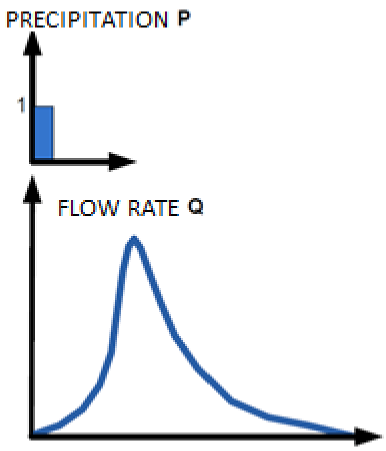


Figure 17 – Unitary precipitation time.

The precipitation time must be equal or lower than a 1/3 or 1/5 of the time of concentration t_c, which occurs between the end of the rain and the restoration of the pre-existing flow rate.

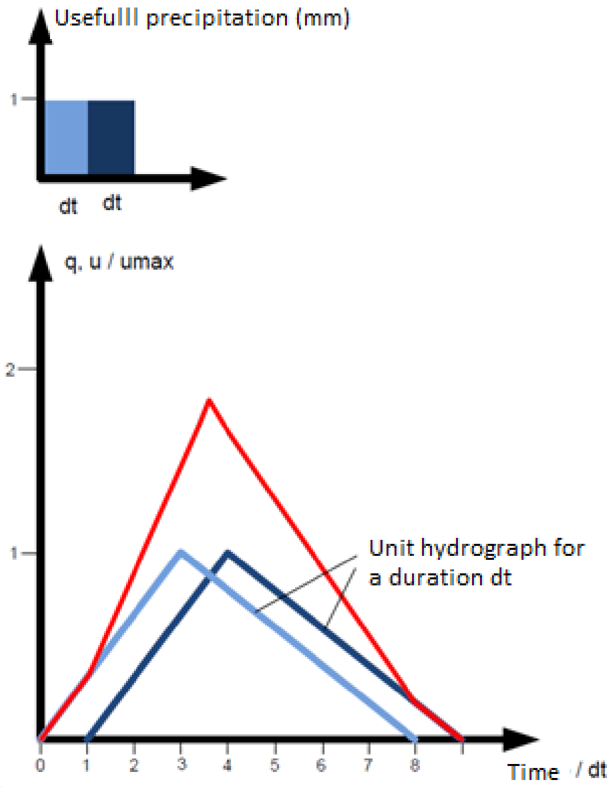


Figure 18 – Unitary precipitation flow.

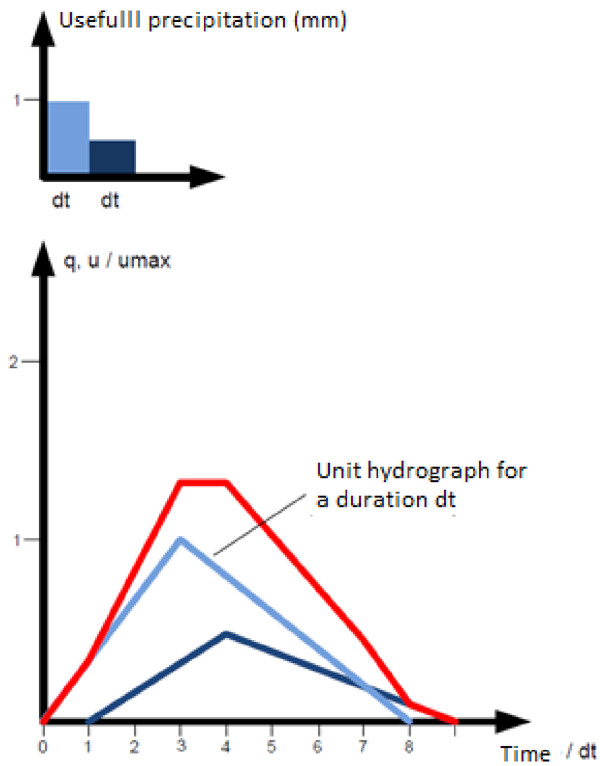


Figure 19 – Construction of a flood hydrograph based on the unitary hydrograph.

$$\begin{cases} q_1 = i_1 \cdot y_1 \\ q_2 = i_1 \cdot y_2 + i_2 \cdot y_1 \\ q_2 = i_1 \cdot y_3 + i_2 \cdot y_2 + i_3 \cdot y_1 \\ q_4 = i_1 \cdot y_4 + i_2 \cdot y_3 + i_3 \cdot y_2 \end{cases}$$

Conversion of a unitary hydrograph into a S hydrograph

The S hydrograph is the resulting hydrograph from a series of rainfalls, juxtaposed, each with unitary useful precipitation. Therefore, it is obtained from the sum of unitary hydrographs for the duration *D* of the useful precipitation, each offset *D* from the previous hydrograph (Figure 20):

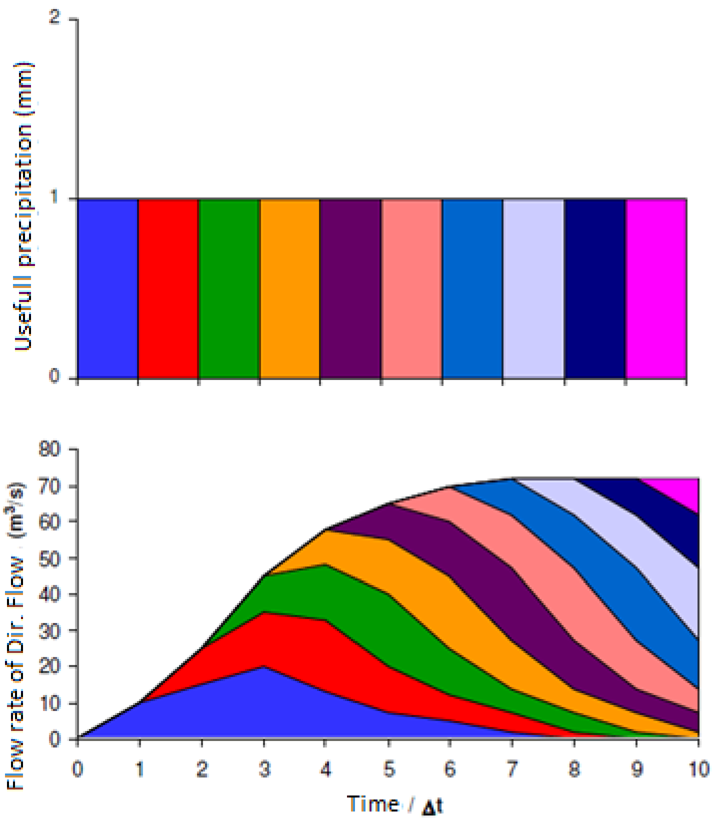


Figure 20 – Construction of a S hydrograph.

The S hydrograph has a level corresponding to the equilibrium flow rate. The smallest number of unitary hydrographs that becomes necessary to sum in order to achieve the equilibrium flow rate of the S hydrograph is equal to T/D, being T the time interval during which direct flow occurs in each unitary hydrograph.

The equilibrium flow rate of a S hydrograph is the result of the contribution of the entire area of the hydrographic basin with the intensity of useful precipitation (I_u), for which such hydrograph is defined as:

$$Q_e = I_u \cdot A$$

From the S hydrograph, the corresponding unitary hydrograph can also be obtained.

Giandotti's Method

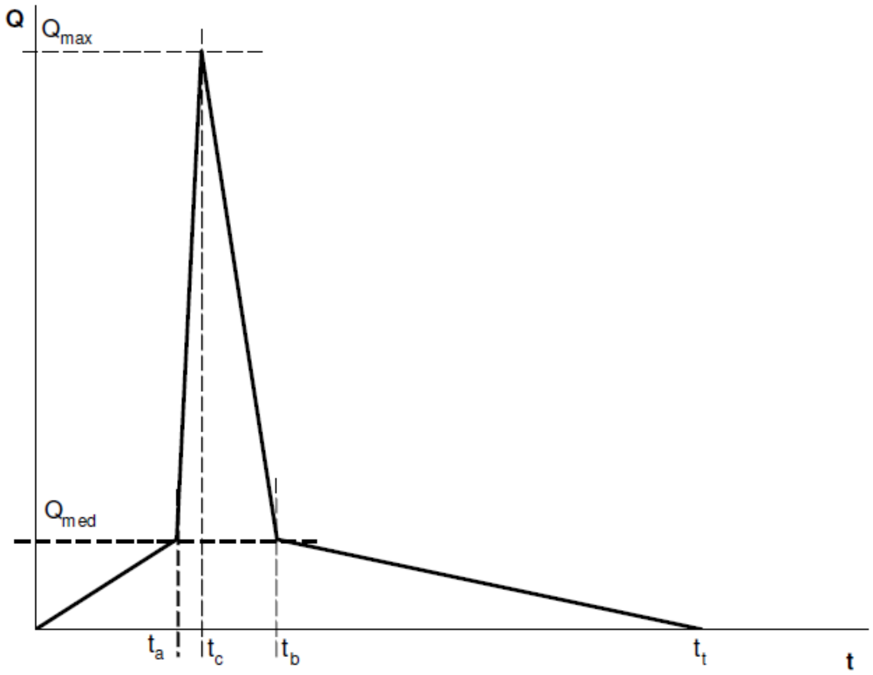


Figure 21 – Giandotti's method.

Table 8 – Giandotti's method.

t	Q
$\mathbf{0}$	0
t_a	Q_{avg}
t_c	Q_{max}
t_b	Q_{avg}
t_t	0

$$Q_{max} = \frac{277 \cdot \Psi \cdot \gamma \cdot P \cdot A}{\lambda \cdot t_c}$$

$$t_c = \frac{4 \cdot \sqrt{A} + 1.5 \cdot L}{0.8 \cdot \sqrt{h_m}}$$

$$t_a = \left(1 - \frac{1}{\gamma}\right) \cdot t_c$$

$$t_b = \left(\frac{\lambda - 1}{\gamma} + 1\right) \cdot t_c$$

Where:
P - precipitation (m);
A - area of the basin (km^2);
t_c - time of concentration;
L - length of the watercourse from upstream until the section of reference (km);

h_m - average height of the basin (m);
Ψ - coefficient of flow (-);
γ - coefficient of peak rainfall (-):

$$\gamma = \frac{Q_{max}}{Q_{avg}}$$

λ - coefficient of flood duration (-):

$$\frac{1}{\lambda} = \frac{t_c}{t_t}$$

t_t - duration of flood.

Table 9 – Giandotti's method (coefficient).

A (km^2)	γ	λ	Ψ
< 300	10.0	4.0	0.50
300 - 500	8.0	4.0	0.50
500 - 1000	8.0	4.5	0.40
1000 - 8000	6.0	5.0	0.30
8000 - 20000	6.0	5.5	0.25
20000 - 70000	6.0	6.0	0.20

EXERCISES

7.1. Consider the unitary hydrograph, Figure 22, corresponding to a useful precipitation of A mm.

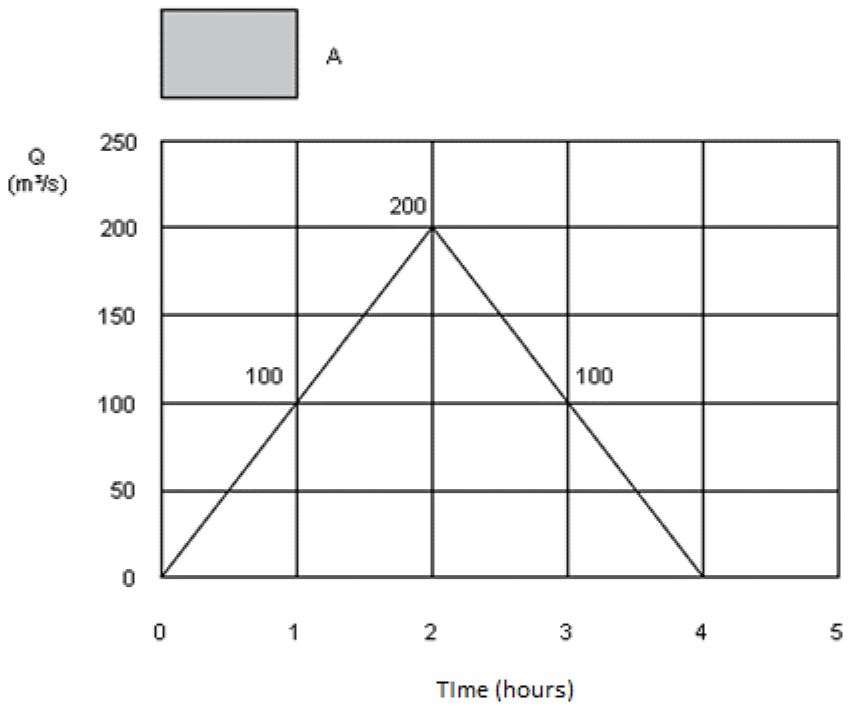


Figure 22 – Unitary hydrograph.

For the unitary hydrograph presented, determine the flood peak flow rate for the following conditions of effective precipitation:
A) Precipitation with the same duration and twice the intensity;
b) Precipitation with the same intensity and twice the duration;
c) Precipitation with twice the intensity and lasting 4 hours;
D) Precipitation with the following temporal distribution:

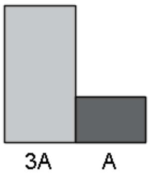


Figure 23 – Temporal distribution of a precipitation.

7.2. Consider the unitary hydrograph shown in Figure 24 corresponding to an effective precipitation of A mm.

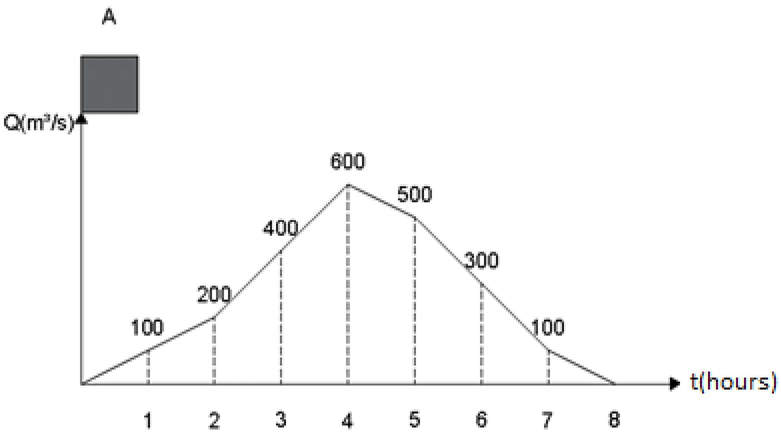


Figure 24 – Unitary hydrograph.

a) From the data given by the graph, design the flood hydrograph resulting from a precipitation B:

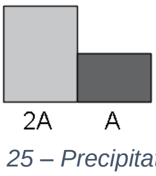


Figure 25 – Precipitation B.

b) What is the value of the peak flow rate (m^3/s) and the time of concentration (h);
c) Solve the previous two points considering the following precipitation C:

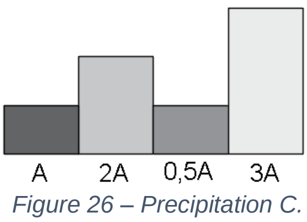


Figure 26 – Precipitation C.

7.3. Determine the unitary hydrograph corresponding to a flood hydrograph shown in Figure 27:

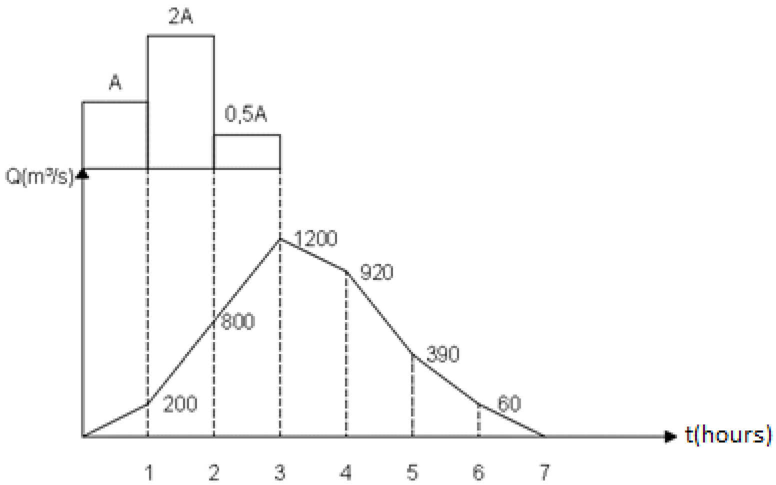


Figure 27 – Flood hydrograph.

7.4. Design the S hydrograph corresponding to the unitary hydrograph of Figure 27.

7.5. Consider the S hydrograph shown in Figure 28, corresponding to a useful or effective precipitation of A mm.

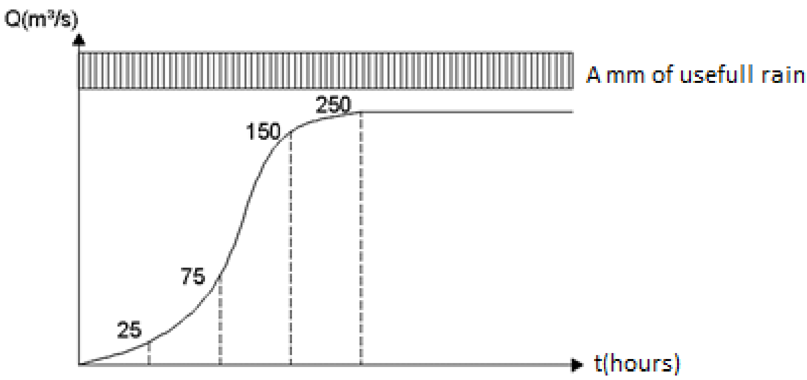


Figure 28 – S Hydrograph.

a) Design the corresponding unitary hydrograph for a unit of time;
b) Ascertain which flood peak corresponds to the unitary hydrograph.

7.6. Design the flood hydrograph by Giandotti's method, for a hydrographic basin with the following characteristics:

Table 10 – Characteristics of a hydrographic basin.

Parameters	
Area (km^2)	362.0
Length of the watercourse (km)	33.8
Average height of the basin (m)	240.0
Maximum precipitation height (mm)	109.0

7.7. The application of Giandotti's method for a hydrographic basin with an area lower than $5000\ km^2$, leads to a flood hydrograph which its growth stage has the following appearance shown in Figure 29. It is known that the flood volume is $21.6 \times 10^6\ m^3$ and the precipitation that caused such flood has a value of $48\ mm$.

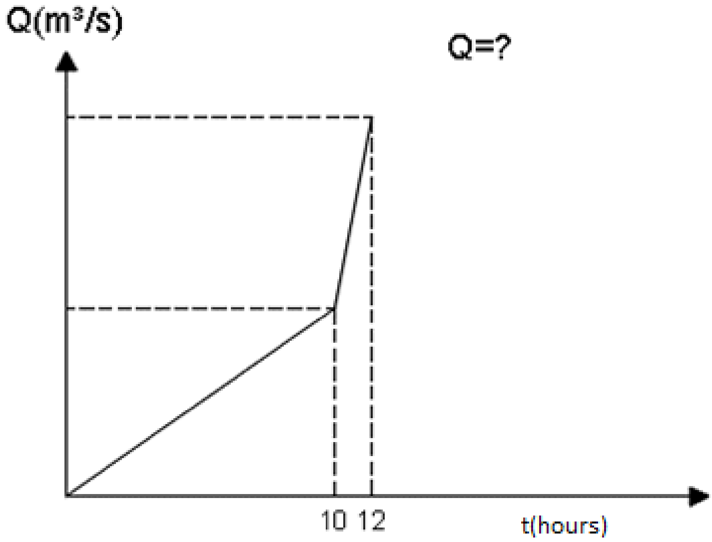


Figure 29 – Flood hydrograph

a) Complete the flood hydrograph;
b) Determine the area of the basin and the flow rate after $30\ hours$;
c) Calculate the maximum flow rate by using an empirical formula.

7.8. In a $54\ km^2$ basin, it was recorded a uniform precipitation with total intensity of $25 mm/h$ and duration of $1\ hour$. The flood hydrograph is shown in Figure 30, knowing that it has a volume of $0.54 \times 10^6\ m^3$. Also, the precipitation of Figure 31, leads to the presented hydrograph.

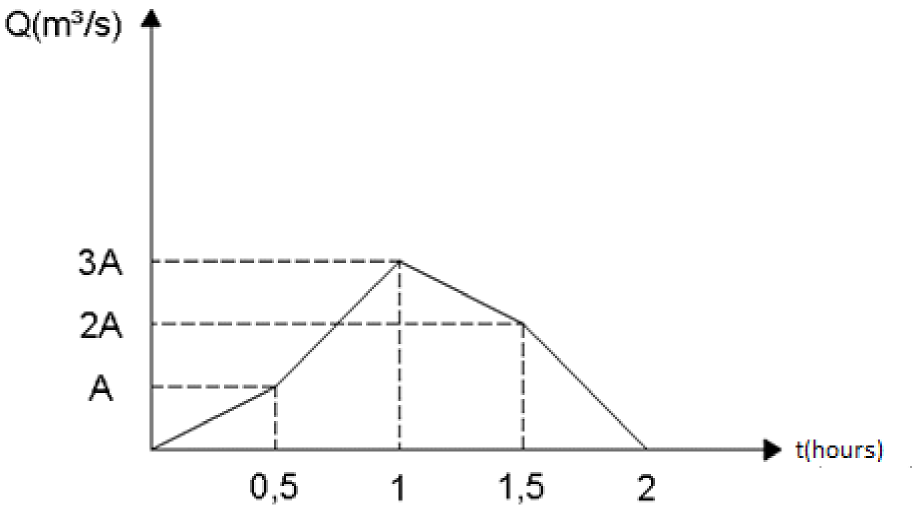


Figure 30 – Flood hydrograph.

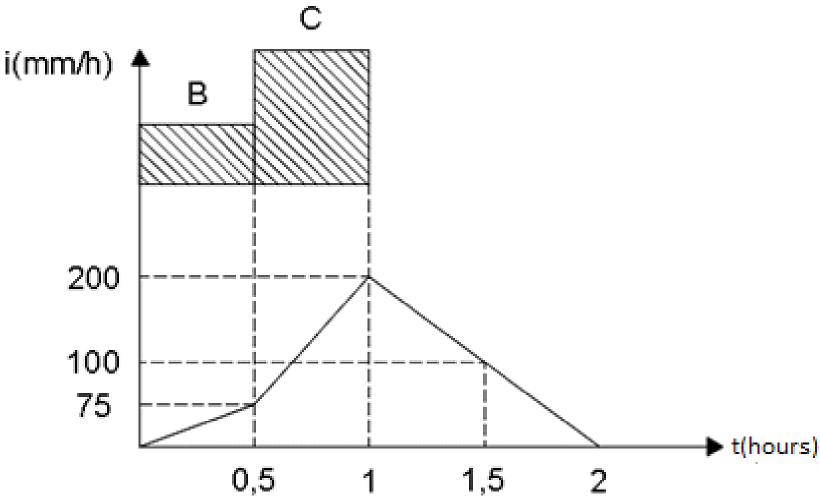


Figure 31 – Hydrograph.

a) Determine the flow coefficient of the basin and mention what is the valued of the maximum flow rate that can occur on it, based on a precipitation with total intensity of $20\ mm/h$ (justify your answer);
b) Calculate the values of B and C.

Chapter 8 - Regularization effect of a reservoir on flood peak damping

8.1 Effects of reservoirs on floods

The continuous rise of the level of water free surface, in a reservoir, means that the discharged flow rate (effluent) is inferior to the affluent. The difference between the two flows (affluent - effluent) equals the increase in volume stored in the reservoir. This implies that the discharged flood hydrograph may have a much lower peak than that of the affluent flood. The transformation of the flood hydrograph into a reservoir is referred to as **flood damping**.

Damping interest:

- Decreases the flow rate of the discharge structures;
- Reduces the risk of downstream valley flooding;
- Facilitates defence works.

Damping conditions:

- Requires higher dam height;
- MFL (Maximum Flood Level) > FSL (Full Storage Level).

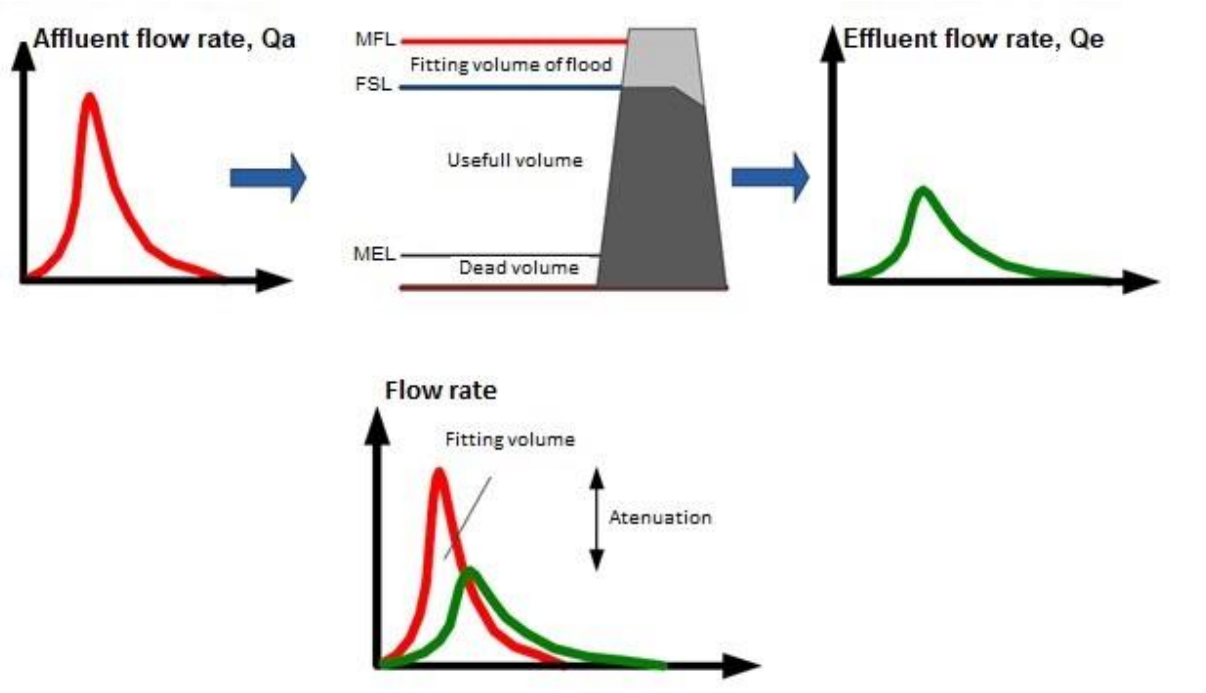


Figure 32 – Analysis of the hydrograph.

The peak of the discharged hydrograph is located at its intersection with the affluent hydrograph on its descending section.

Areas between the two hydrographs equal the volume stored between the FSL and the MFL. MEL stands for Minimum Exploration Level.

8.2 Procedure

When precipitation occurs over a given hydrographic basin, the precipitated water flows naturally to the lowest points of the terrain, finally reaching the streams or rivers that lead them to their final destination. Naturally these precipitations cause variations in flow over time in a given section of the river.

The hydrograph of a flood wave is exactly the representation of the flow variation in a given section of the river, thus representing the effects of the hydrographic basin upstream of this section on the temporal distribution of rainfall.

A flood wave, when passing through a reservoir suffers a damping effect, that is, the volume of the reservoir when retaining part of the affluent flow rate causes the output flow rate to be smaller than the input flow rate, also being lagged in relation to the flow rate input.

For the calculus of the damping of a flood wave in reservoirs, the equation of continuity is used:

$$\int_{t_1}^{t_2} Q_A dt - \int_{t_1}^{t_2} Q_E dt = V_2 - V_1$$

The continuity equation basically expresses that the volume of water stored in a reservoir in a given time span is equal to the total amount of water that arrives at such reservoir (Q_A) minus the total amount of water leaving the same reservoir (Q_E), in that given time span.

In a simplified way, the phenomenon can be described by the following equation:

$$Q_A - Q_E = \frac{\partial V}{\partial t}$$

Q_A represents the known hydrograph of affluent flow rates to the reservoir, Q_E the hydrograph of effluent flow rates from the same reservoir and $\partial V/\partial t$ represent the variation of the volume stored in such reservoir due to the variation of its water level.

The reservoirs are generally deep and not extensive structures, being water velocity consequently low. Thus, without introducing major errors, the water surface can be considered horizontal. Thus, the volume stored in the reservoir is directly proportional to the water level, which facilitates the resolution of the equation.

8.3 Input data

For the calculation of the damping of a flood wave by a reservoir, data of flood hydrograph, reservoir, and upstream and downstream constraints are required in the case of dimensioning the width of a spillway.

The flood or affluent hydrograph to the reservoir may be the one defined by a flood study in the hydrographic basin (unitary hydrograph, rain-flow rate transformation model, etc.), in the case of spillway design. In the application of operating a reservoir, the hydrograph may be a hydrograph already observed (in post-operation analyses) or a predicted hydrograph, in the case of real-time operation.

Data concerning the reservoir are summarized to the knowledge of the height-volume curve of such reservoir and the discharge equations of the overflow structures of the dam.

The height-volume curve consists of the ratio between the water level of the reservoir and the corresponding volume. The capacity of a reservoir built on natural terrain is calculated through a topographic survey, using the same procedures used in the calculation of earthworks volumes. It is mapped by planimetry of the areas between the topographically surveyed level curves, the area-height curve of the reservoir. And by integrating the area-height curve, the volume-height curve of the reservoir is obtained.

Another curve needed to solve the equation is the spillway curve. The spillway is the overflow structure of the reservoir, that is, the structure through which the flow rate exits the reservoir. The spillway curve indicates the effluent flow for different water line heights above the spillway crest.

The following equations present the general forms of the discharge equations for various types of overflow structures:

- Free sill spillway:

$$Q = CLH_1^{3/2}$$

- Spillway with floodgate:

$$Q = \frac{2}{3}\sqrt{2g}cL\left(H_1^{3/2} - H_2^{3/2}\right)$$

- Tulip spillway:

$$Q = C_0(2\pi R_S)H^{3/2}$$

- Orifice:

$$Q = CWD\sqrt{2gH}$$

Where:
Q is the discharge flow rate;
C is the discharge coefficient;
L is the widdth of the spillway crest;
H_1 is the total charge at the cerst of the spillway;
H_2 is the total charge at the top of the oppening;
C_0 is a coefficient that relates H_1 and R_S;

R_S is the opening radius of the spillway;
D is the opening height;
W is the nozzle width.

The external restrictions to the dam area are understood as the maximum flow rate capacity of the channel immediately at downstream of the dam and the maximum water level upstream of the spillway, limited by the height of the dam or the desired maximum floodable area.

8.4 Calculus

The problem of wave damping in the reservoir is solved using basically the continuity equation. The equation can be solved by numerical processes or graphs, as shown below.

Graphical methods

With the current easiness to perform calculations, the graphic processes of solution of the continuity equation fell practically in disuse. Only as a reference to its generic form the **Puls's method** is presented below.

The continuity equation can be written as follows:

$$\frac{Q_{A1} + Q_{A2}}{2} \cdot \Delta t + V_1 - \frac{Q_{E1}}{2} \cdot \Delta t = V_2 + \frac{Q_{E2}}{2} \cdot \Delta t$$

Having:

$$A = [(Q_{A1} + Q_{A2})/2] \cdot \Delta t$$

$$F_1 = V_1 - (Q_{E1}/2) \cdot \Delta t$$

$$F_2 = V_2 + (Q_{E2}/2) \cdot \Delta t$$

Two curves are built, at first ($F_1 \times Q_E$), and secondly ($F_2 \times Q_E$) and by a given value of Q_{E1}, introduced in the first curve and then F_1 is obtained. By introducing the obtained value in the upper equation (once A is also known), the value of F_2 is obtained. And introducing the value obtained in the second curve, value of Q_{E2} is obtained.

Iterative method

It is the most indicated method for its simplicity and speed of convergence.

And as previously mentioned, the height-volume and spillway curves are used in the solution of such problem. In the possession of these two curves and the input hydrograph of the reservoir, the resolution of the continuity equation can be easily obtained.

By reordering the terms of the equation $Q_A - Q_E = \partial V/\partial t$, it is obtained, that:

$$\frac{Q_{A1}}{2} + \frac{Q_{A2}}{2} + \frac{V_1}{\Delta t} - \frac{Q_{E1}}{2} - \frac{Q_{E2}}{2} = \frac{V_2}{\Delta t}$$

Or,

$$Q_{A1} + Q_{A2} + \frac{V_1}{\Delta t/2} - Q_{E1} - Q_{E2} = \frac{V_2}{\Delta t/2}$$

To simplify the resolution of the above equation, it is suggested to assemble a table where each of the terms of the equation is recorded in a column. So:

- **First column:** time interval (Δt)
  - In this column, it is noted the initial and final instants of the time interval considered. It should be remembered that this time interval should be sufficiently small so that the flow variation can be considered linear.
- **Second column:** initial affluent flow (Q_{A1})
  - In this column is noted the flow rate corresponding to the beginning of the interval. This value is obtained by reading the affluent hydrograph in the horizontal axis corresponding to the initial instant of the interval considered.
- **Third column:** final affluent flow (Q_{A2})
  - In this column is noted the flow rate corresponding to the end of the interval. This value is obtained by reading the affluent hydrograph in the horizontal axis corresponding to the final instant of the interval considered. The Q_{A2} flow rate of an interval corresponds to the Q_{A1} flow of the subsequent interval.
- **Fourth Column:** volume stored in the reservoir at the beginning of the time interval divided by half of such time interval $[V_1/(\Delta t/2)]$
  - In this column is noted the value of $[V_1/(\Delta t/2)]$, calculated by the reservoir's height-volume curve.
- **Fifth Column:** initial effluent flow (Q_{E1})
  - In the first interval, it is adopted Q_{E1} corresponding to the initial water level, through the equation of the spillway. If the water level is less than or equal to the position of the crest of the spillway, then, it is imposed $Q_{E1} = 0$. For the remaining intervals, the value of Q_{E1} is always equal to the Q_{E2} value of the previous interval, that is, the output flow from the initial instant of an interval equals to the output flow of the final instant of the previous interval.
- **Sixth Column:** estimated height of the effluent water level at the end of the time interval (Y_{EST2})
  - Corresponds to the estimated height of the effluent water line at the end of a calculation interval. The initial estimate corresponds to the same

level as the beginning of the interval. For the other estimates, it corresponds to the value of the previous Y_{CALC2}

- **Seventh column:** final effluent flow (Q_{E2})
  - Calculated using the spillway equation, with the water level obtained from the sixth column.
- **Eighth Column:** volume stored in the reservoir at the end of the time interval divided by half of such time interval $[V_2/(\Delta t/2)]$
  - This value is calculated by solving the continuity equation rearranged to the iterative method. In terms of the columns described: $[8] = [2] + [3] + [4] - [5] - [7]$
- **Ninth Column:** calculated height of effluent water level (Y_{CALC2})
  - Corresponds to the water level value calculated by the height-volume curve through the volume stored in the reservoir at the end of the time interval.

The calculation must be repeated until the value of Y_{CALC2} minus the value of Y_{EST2}, become, in module, less than or equal to the desired precision. If the condition is not found, it is adopted as a new value of Y_{EST2}, the value obtained from Y_{CALC2}, repeating the calculations of columns six to nine until the condition is observed. In a generic way, two to three iterations ensure the convergence of the process within a reasonable precision.

8.5 Graphical properties

Analyzing the following figure, that shows the input and output hydrographs, overlapped, then:

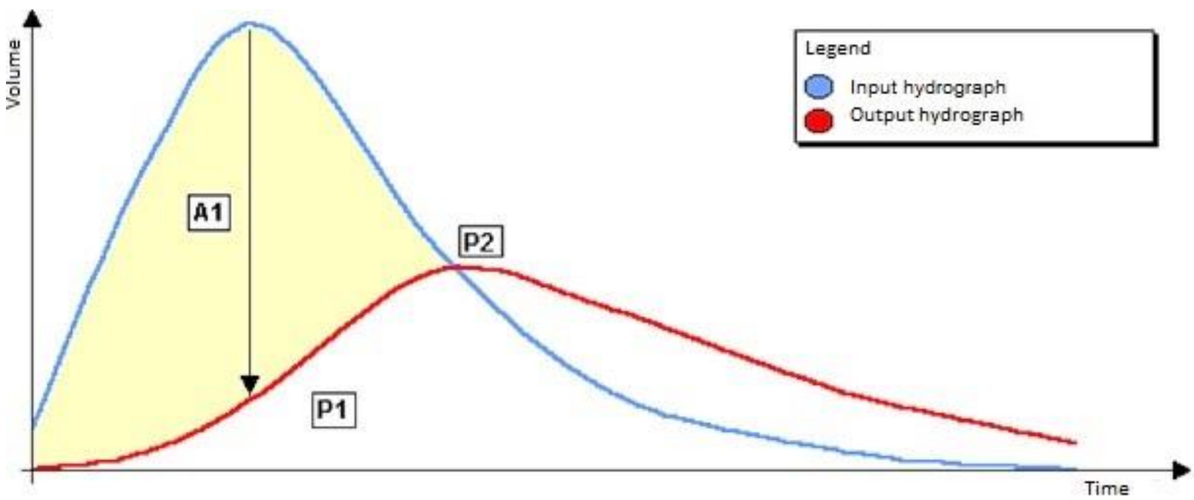


Figure 33 – Flood damping.

First property

Since the integral of the input hydrograph corresponds to a volume that enters into the reservoir and the integral of the output hydrograph corresponds to the volume coming out of it, the **A1** area, corresponds to the volume stored in the reservoir in the considered time span.

Second property

Considering the water level as a function of time, the water level velocity is its first derivative. Between zero and $Q_{E,max}$, such velocity is always positive (rising water level). Between zero and $Q_{A,max}$, the difference between Q_A and Q_0 is increasing, so the rising velocity increases. Between $Q_{E,max}$ and $Q_{A,max}$, the velocity remains positive, but decreases to zero (when Q_A is equal to Q_E). Therefore, at instant $Q_{A,max}$, the velocity variation in time changes signal, which characterizes an inflection point (**P1**) in the output hydrograph.

Third property

The **P2** point, which is the point where the two hydrographs intersect, corresponds to the maximum output flow rate, because the discharge curve is increasing with the water level (volume) and at this point the stored volume is maximum.

EXERCISES

8.1. Study the damping of the peak flood shown in Figure 34, knowing that:

- The flow rate curve of the flood spillway is:

$$Q = 12.5 \cdot H^{\frac{3}{2}}, where\ \ H = Y - 100$$

- The storage volumes in such reservoir are:

Table 11 – Stored volumes.

Height (m)	Volume ($m^3 \times 10^6$)
100.00	1.060
100.50	1.430
101.00	1.820
101.50	2.240
102.00	2.650
102.50	3.060

- The spillway crest is at the same level as the full storage level (FSL) - 100.00.

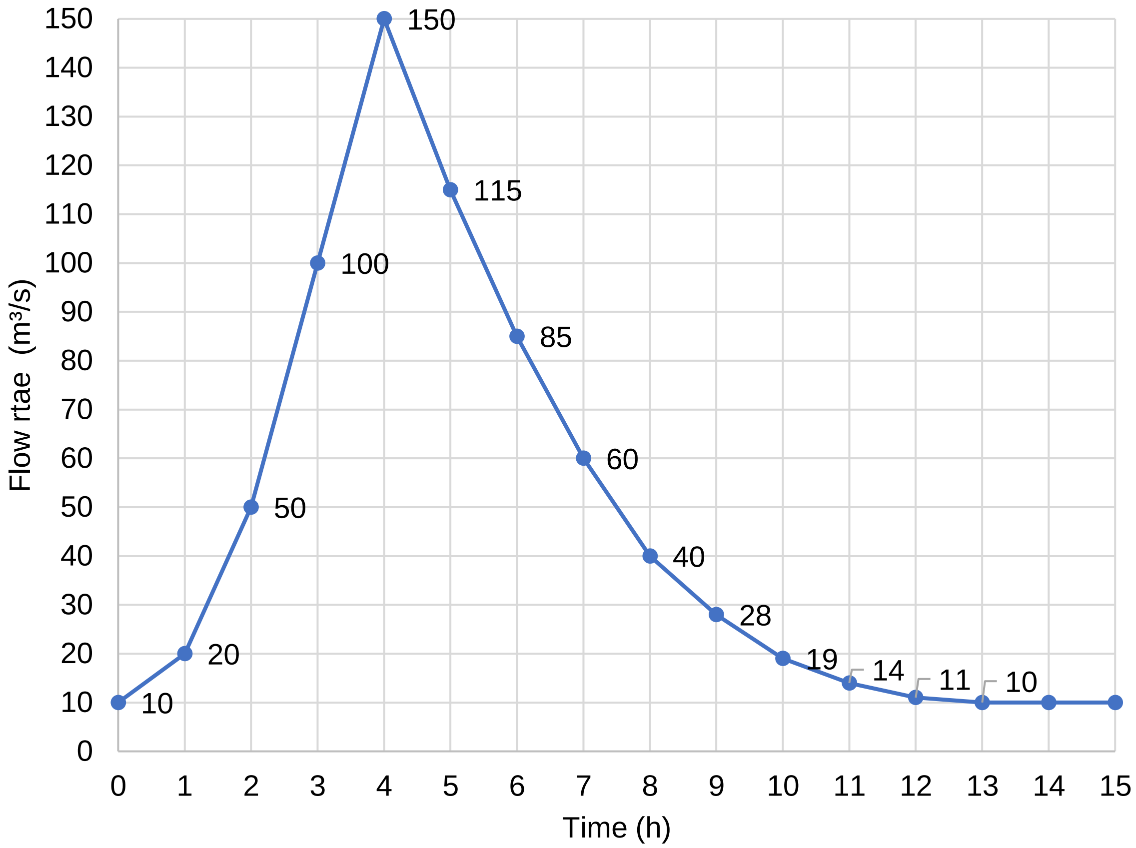


Figure 34 – Flood damping.

Chapter 9 - Evaporation and Evapotranspiration

EXERCISES

9.1. Presented the values of the average monthly temperature (T) and the average daily insolation (n) in a given region at latitude $40° N$.

Table 12 – Average monthly temperature (T) and average daily insolation (n).

Month	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
$\boldsymbol{T}$ (°$\boldsymbol{C}$)	5.1	6.0	8.5	10.6	14.3	18.6	21.7	21.0	18.0	13.6	8.0	5.1
$\boldsymbol{n}$ ($\boldsymbol{h}$)	3.9	5.2	5.7	7.8	10.0	11.6	12.8	12.0	7.8	6.2	4.6	3.2

Estimate by Thornthwaite's and Turc's method, the monthly and annual potential evapotranspiration in such region. Consider the Angstrom's coefficients as $a = 0.23$ and $b = 0.53$, and ignore the effect of relative moisture in Turc's formula.

Chapter 10 - Water in Soils

EXERCISES

10.1. At the beginning of a rainfall, the infiltration capacity is $38\,mm/h$, and after $6\,hours$, such capacity became $8\,mm/h$, where Horton's recession constant is $1.11\,h^{-1}$. Calculate the infiltration capacity after $3\,hours$ from the beginning of such rainfall.

10.2. A sample of a soil occupies a cylinder, $5\,cm$ in diameter and $10\,cm$ high. Knowing that the wet mass and dry mass are respectively, $331.8\,g$ and $302.4\,g$, and that the solids volumetric mass is $2650\,kg/m^3$, determine the volumetric amount of moisture sample and the degree of saturation.

10.3. In a container with an orifice at the bottom there is $5\,L$ of a soil with a volumetric amount of moisture of 0.15. Knowing that the field capacity of such soil corresponds to a volumetric amount of moisture of 0.28, calculate the amount of water that will be discharged by the orifice when is added $1\,L$ of water in the container.

10.4. In a $1\,ha$ plot, an agriculture crop is installed with a radial depth of $0.5\,m$. Knowing that the soil has a field capacity of 0.45 and that, the minimum volumetric amount of moisture usable for production is 0.24, estimate the volume of irrigation required in order to go from such minimum to the field capacity. Knowing that the average evapotranspiration is $3\,mm/d$, estimate also the time interval between two successive irrigations.

Chapter 11 - Groundwater

EXERCISES

11.1. Consider a subsoil with 5 layers:

- Layer 1: medium sand, $K = 5\ m/day$, thickness $10\ m$;
- Layer 2: clay, $K = 0.01\ m/day$, thickness $5\ m$;
- Layer 3: coarse sand, $K = 20\ m/day$, thickness $15\ m$;
- Layer 4: clay, $K = 0.005\ m/day$, thickness $10\ m$;
- Layer 5: thin sand, $K = 1\ m/day$, thickness $30\ m$;

The subsoil is $1\ km$ long (horizontal gradient, $i_{hor} = 0.001$). The total charge loss in the vertical direction is $1\ m$. Calculate for each layer:
a) charge loss Δh;
b) horizontal hydraulic resistance;
c) vertical hydraulic resistance;
d) transmissivity;
e) specific flow rate (horizontal);
f) equivalent permeability (horizontal).

11.2. A confined aquifer is being pumped from a hole at a constant flow rate of $0.1\ m^3/s$, verifying lowering of $10\ m$ and $7\ m$ in two observation wells, located at distances of $10\ m$ and $30\ m$ from the pumping hole. Knowing that the radius of the hole is $0.15\ m$ and that the thickness of the aquifer is $40\ m$, determine the permeability of the aquifer, the theoretical lowering in the well, the distance from which lowering is less than $2\ m$ and the radius of influence of such well.

11.3. The groundwater level of a coastal aquifer is $10\ m$ above sea level, in a hole at a distance of $500\ m$ from the coast. As the aquifer consists of coarse sand, estimate the flow rate per kilometer of coastline that flows into the sea.

SOLUTIONS

Chapter 1 - Hydrologic cycle

1.1. Answer:

The average annual renewal volume of aquifers is calculated as follows:

$$V_{aqus} = \frac{30\% \times V_T}{t_{residence}} = \frac{0.3 \times 35 \times 10^6}{1400} = 7500\ km^3/year$$

And for rivers:

$$t_{residence} = x$$

$$\begin{array}{lcl} 1\ year & \_\_\_ & 365\ days \\ x\ year & \_\_\_ & 16\ days \end{array}$$

$$t_{residence} = 16/365\ year$$

$$V_{rivers} = \frac{0.006\% \times V_T}{t_{residence}} = \frac{0.00006 \times 35 \times 10^6}{16/365} = 47906.25\ km^3/year$$

In comparison to rivers that have a renewal volume of $V_{rivers}/V_{aqus} \approx 6$ times superior than aquifers, being possible to use them permanently.

1.2. Answer:

The average annual flow of the Amazon river can be estimated as follows:

$$Q_{Amaz} = 12\% \times Q_{cont} = 0.12 \times \left(\frac{h_{water} \times A}{365 \times 24 \times 60 \times 60}\right) = 0.12 \times \left(\frac{316 \times 10^{-3} \times 150 \times 10^6}{365 \times 24 \times 60 \times 60}\right)$$

$$Q_{Amaz} \cong 180365.3\ m^3/s\ (In\ comparison\colon Q_{Tejo} = 350\ m^3/s)$$

1.3. Answer:

Daily volume of water supply:

$$V = 200 \times 1 \times 10^7 = 200 \times 10^7\ L/day = 2 \times 10^6\ m^3/d$$

In terms of water height:

$$V = \frac{2 \times 10^6}{89000 \times 10^6} = \frac{2}{89000}\ m/d$$

In $mm/year$:

$$V = \frac{2}{89000} \times 10^3 \times 365 = 8.20\ mm/year$$

Chapter 2 - Hydrographic basins

2.1. Answer:

Expression for the calculus of average height, h_{avg}:

$$h_{avg} = Z_{avg} - Z_{min};\ Z_{avg} = \frac{1}{A_t}\sum_{i=0}^{n-1}\frac{1}{2}(Z_i + Z_{i+1}) \cdot A_i'$$

Table 13 – Organized data for the calculus of average height.

Area above the altitude, A_i (km^2)	**Altitude, Z_i (m)**	**Altitude interval (m)**	**Average altitude, $(Z_i + Z_{i+1})/2$ (m)**	**Area between altitudes, A_i' (km^2)**	**Average altitude *Area between altitudes**
0.00	306	**300;306**	303	0.57	172.71
0.57	300	**280;300**	290	1.50	435.00
2.07	280	**260;280**	270	7.25	1957.50
9.32	260	**240;260**	250	7.49	1872.50
16.81	240	**220;240**	230	6.03	1386.90
22.84	220	**204;220**	212	0.21	44.52
23.05	204	**Sum**		**23.05**	**5869.13**

Procedure to fill Table 13:

- Sort altitudes in descending order;
- Organize altitudes in intervals;
- Calculate the average altitude for each interval: $(Z_i + Z_{i+1})/2$;
- Calculate the area between altitudes: $A_{i+1} - A_i$;
- Product of the average altitude and the area between altitudes.

Then:

$$Z_{avg} = \frac{5869.13}{23.05} \approx 254.6\,m \rightarrow h_{avg} = 254.6 - 204 = 50.6\,m$$

2.2. Answer:

Average extension of a superficial flow:

$$\overline{P_S} = \frac{1}{4D_r};\ D_r = \frac{\sum_{i=1}^{n} L_i}{A}$$

Then:

$$\overline{P_S} = \frac{1}{4D_r} = \frac{1}{4\frac{\sum_{i=1}^{n} L_i}{A}} = \frac{1 \times A}{4\sum_{i=1}^{n} L_i} = \frac{102}{4 \times 300} \cong 0.085\,km$$

2.3. Answer:

Bifurcation ratio:

$$R_b = \frac{N_u}{N_{u+1}}$$

Average bifurcation ratio:

$$\bar{R}_b = \sqrt[n-1]{\prod_{u=1}^{n-1} \frac{N_u}{N_{u+1}}} = \sqrt[n-1]{N_1}$$

Table 14 – Calculus of the bifurcation ratio.

Order, u	N_u	R_b
1	139	3.0
2	46	4.2
3	11	3.7
4	3	3.0
5	1	-

$$\bar{R}_b = \sqrt[5-1]{3.0 \times 4.2 \times 3.7 \times 3.0} = \sqrt[5-1]{139} \cong 3.4$$

2.4. Answer:

Expression for the calculus of average slope:

$$i_{avg} = \frac{Z_{max} - Z_{min}}{L}$$

Expression for the calculus of the equivalent slope:

$$i_{eq} = \frac{Z_{eq} - Z_{min}}{L};\ Z_{eq} = \frac{1}{L}\sum_{i=0}^{n-1}(Z_i + Z_{i+1}) \cdot X'_{i+1} - Z_{min}$$

Table 15 – Organized data for the calculus of the equivalent slope.

Distance to section, X_i (km)	**Altitude, Z_i (m)**	$Z_i + Z_{i+1}$, (m)	**Distance between altitudes, X'_{i+1} (km)**	$(Z_i + Z_{i+1}) \cdot X'_{i+1}$
7	205	335	3	1005
4	130	240	2	480
2	110	213	2	426
0	103	**Sum**	**7**	1911

Procedure to fill Table 15:

- Sort altitudes in descending order;
- Calculate the summation of altitudes: $Z_i + Z_{i+1}$;
- Calculate the distance between altitudes: $X_i - X_{i+1}$;
- Product of the summation of altitudes and the distance between altitudes.

Then:

$$i_{avg} = \frac{205 - 103}{7 \times 10^3} \cong 0.015 = 1.5\%$$

And:

$$Z_{eq} = \frac{1}{7} \times 1911 - 103 = 170\ m \rightarrow i_{eq} = \frac{170 - 103}{7 \times 10^3} \cong 0.010 = 1.0\%$$

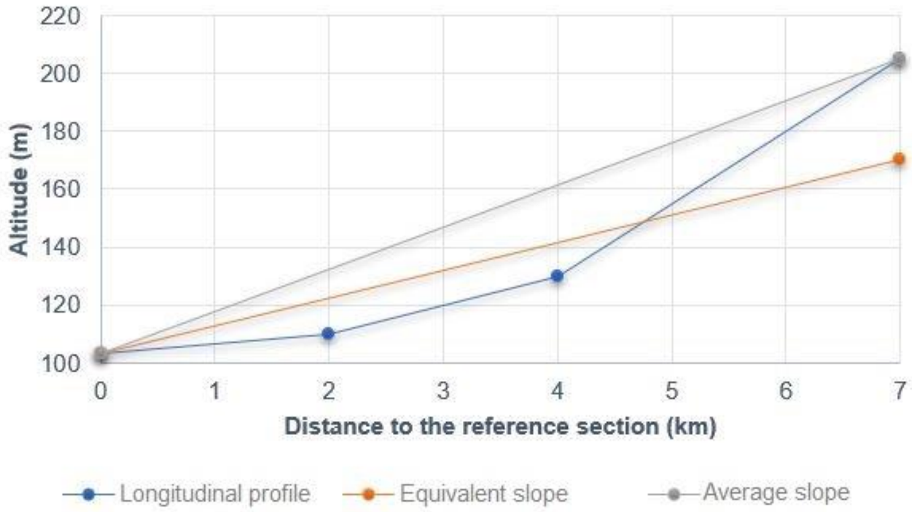


Figure 35 – Graphical representation of the longitudinal profile and the respective calculated slopes.

Chapter 3 - Hydrologic balance of a basin

3.1. Answer:

General expression of hydrologic balance:

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

Where:
P - Precipitation on the basin;
H - Flow at the downstream section of the basin;
E - Evapotranspiration of the basin;
ΔS_P - Variation of the amount of intersection water and storage in riverbeds;
ΔS - Variation of the amount of moisture in soil (water content in the un-saturated area);
ΔS_U - Variation of the amount of water in underground reservoirs;
E_X - Amount of water extracted from the basin by human action;
R - Amount of water inserted in the basin by human action.

Taking into account the given data, the expression becomes:

$$P = H + E - R$$

Then, to estimate the real evapotranspiration in the basin:

$$E = P - H + R = 1000 - 1300 + \frac{8 \times 10^{15} \times 12}{100 \times 10^{12}} = 660\ mm$$

3.2. Answer:

General expression of hydrologic balance:

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

Taking into account the given data, such expression becomes:

$$P = H + E + E_X$$

Then, the maximum amount of water available to extract by human action will be the same as the amount that results from a null flow at the downstream section of the basin:

$$H = 0 \rightarrow P = E + E_X \rightarrow E_X = P - E = 1000 - 700 = 300\ mm$$

The maximum transferable flow rate is:

$$Q_{E_X} = \frac{[(300 \times 10^{-3}) \times (100 \times 10^{6})]}{365 \times 24 \times 60 \times 60} \cong 0.951\ m^3/s$$

3.3. Answer:

General expression for hydrologic balance:

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

Knowing that the difference between precipitation and flow, the flow deficit, equals to the loss of water by evapotranspiration:

$$D = E = P - H$$

Then:

$$H = P - D = 1500 - 850 = 650\ mm$$

The annual average flow rate at the reference section is:

$$Q_H = \frac{[(650 \times 10^{-3}) \times (40 \times 10^{6})]}{365 \times 24 \times 60 \times 60} \cong 0.824\ m^3/s$$

Chapter 4 - Precipitation

4.1. Answer:

Knowing that the vertical scale corresponds to $10\,mm$ of precipitation, then, after 3 complete cycles of loading/unloading (ascending/descending) reaching its maximum, 10 mm, plus 1 cycle of load up to the mark of 6 mm (approximately) results in a daily precipitation estimate of:

$$P = 3 \times 10 + 6 = 36\,mm$$

4.2. Answer:

Average precipitation intensity:

$$I = \frac{\Delta P}{\Delta t}$$

$$I_{30} = \frac{41 - 0}{30 - 0} \times 60 = 82\,mm/h$$

$$I_{40} = \frac{45 - 15}{40 - 10} \times 60 = 60\,mm/h$$

$$I_{50} = \frac{47 - 35}{50 - 20} \times 60 = 24\,mm/h$$

$$I_{60} = \frac{47 - 41}{60 - 30} \times 60 = 12\,mm/h$$

Table 16 – Calculus of the average precipitation intensity in intervals of 30 minutes.

$\boldsymbol{t}\ (\boldsymbol{min})$	**0**	**10**	**20**	**30**	**40**	**50**	**60**
$\boldsymbol{P}\ (\boldsymbol{mm})$	0	15	35	41	45	47	47
$\boldsymbol{I}\ (\boldsymbol{mm/h})$	-	-	-	82	60	24	12

The maximum average precipitation intensity in half hour is $I_{30} = 82\,mm/h$.

4.3. Answer:

Thiessen's method:

$$\bar{P} = \frac{\sum_{i=1}^{n} P_i \cdot A_i}{A_b} = \frac{(12 \times 10) + (18 \times 20) + (23 \times 30)}{10 + 20 + 30} = 19.5\,mm$$

Chapter 5 - Superficial Flow

5.1. Answer:

The time of concentration is expressed as:

$$t_c = \left(\frac{L}{\alpha p^{m-1}}\right)^{\frac{1}{m}}$$

When it’s considered Manning-Strickler's formula:

$$\alpha = K_S S_f^{\frac{1}{2}} = 45 \times 0.05^{1/2} \wedge m = \frac{5}{3}$$

Then:

$$t_c = \left(\frac{20}{45 \times 0.05^{1/2} \times (40 \times 10^{-3})^{5/3-1}}\right)^{\frac{1}{5/3}} \approx 5.472\ hours$$

And:

$$q_{max} = pL = (40 \times 10^{-3}) \times 20 = 0.8\, m^2/s$$

5.2. Answer:

For a rectangular thin-plate weir, the flow rate becomes:

$$Q = C\frac{2}{3}\sqrt{2g}(b + k_b)(h + k_h)^{\frac{3}{2}}$$

The coefficient C can be calculated as follows:

$$C = a\left(\frac{b}{B}\right) + a'\left(\frac{b}{B}\right)\frac{h}{p}$$

Using tables:

$$\frac{b}{B} = \frac{0.3}{0.5} = 0.6 \begin{cases} a = 0.593 \\ a' = 0.018 \end{cases}$$

So, then:

$$C = 0.593 \times 0.6 + 0.018 \times 0.6 \times \frac{0.35}{0.5} = 0.36336$$

The coefficients k_b and k_h take the values of 3.6 (using tables) and $0.001\, m$, respectively.

Then:

$$Q = 0.36336 \times \frac{2}{3}\sqrt{2 \times 10} \times (0.3 + 3.6)(0.35 + k_h)^{\frac{3}{2}} \cong 0.068\ m^3/s$$

Note: Alternatively, the values of a, a' and k_b could be calculated.

5.3. Answer:

$$Q = Q_t \frac{C_t - C_e}{C_e - C_b} = 5 \times \frac{200 - 2}{2 - 0.1} \cong 521\ L/s = 0.521\ m^3/s$$

In order to estimate the necessary quantity of salt for such procedure, it is necessary to know the distance $L_{0,}$ velocity and the time interval for such route L_0:

$$L_0 = K \frac{C\bar{B}^2}{\sqrt{g\bar{y}}} = 0.5 \times \frac{22 \times 4^2}{\sqrt{10} \times 0.15} \approx 371\ m$$

$$v = \frac{0.521}{4 \times 0.15} \approx 0.868\ m/s$$

$$\Delta t = \frac{371}{0.868} \approx 427\ s$$

The amount of salt injected per second is $200 \times 5 = 1000\ g/s = 1\ kg/s$. Thus:

$$m_{salt} = 427 \times 1 = 427\ kg$$

Chapter 6 - Flow rates

6.1. Answer:

SUPPORT CALCULATIONS

Table 17 – Chronological flow rates curve.

Trimester	$h\ (m)$	$Q\ (m^3/s)$
1	7	$\boldsymbol{Q_1 = 5 \times 7^2 = 245}$
2	4	$\boldsymbol{Q_2 = 5 \times 4^2 = 80}$
3	1	$\boldsymbol{Q_3 = 5 \times 1^2 = 5}$
4	2	$\boldsymbol{Q_4 = 5 \times 2^2 = 20}$

Table 18 – Accumulated or integral flow rates curve.

Trimester	$Q\ (m^3/s)$	$Q_{acm}\ (m^3/s)$
1	$Q_1 = 245$	$\boldsymbol{Q_1 = 245}$
2	$Q_2 = 80$	$\boldsymbol{Q_1 + Q_2 = 325}$
3	$Q_3 = 5$	$\boldsymbol{Q_1 + Q_2 + Q_3 = 330}$
4	$Q_4 = 20$	$\boldsymbol{Q_1 + Q_2 + Q_3 + Q_4 = 350}$

Table 19 – Classified flow rates curve.

Trimester	$Q_{cc}\ (m^3/s)$
1	**245**
2	**80**
3	**20**
4	**5**

Table 20 – Hydrologic characteristic curve.

i	$Q\ (m^3/s)$	$q\ (m^3/s)$
0	**0**	$\boldsymbol{q_0 = 0}$
1	**5**	$\boldsymbol{q_1 = 0 + 0.25 \times (4 - 0) \times (5 - 0) = 5}$
2	**20**	$\boldsymbol{q_2 = 5 + 0.25 \times (4 - 1) \times (20 - 5) = 16.25}$
3	**80**	$\boldsymbol{q_3 = 16.25 + 0.25 \times (4 - 2) \times (80 - 20) = 46.25}$
4	**245**	$\boldsymbol{q_4 = 46.25 + 0.25 \times (4 - 3) \times (245 - 80) = 87.5}$

Table 21 – Concentration curve.

$Q\ (m^3/s)$	$t_{cc}(\%)$	$V\ (m^3)$
0	**0**	$\boldsymbol{V_0 = 0}$
5	**25**	$\boldsymbol{V_1 = 0 + 5 \times 0.25 \times (365 \times 24 \times 3600) = 3.942 \times 10^7}$
20	**50**	$\boldsymbol{V_2 = 3.942 \times 10^7 + 20 \times 0.25 \times (365 \times 24 \times 3600) = 1.971 \times 10^8}$
80	**75**	$\boldsymbol{V_3 = 1.971 \times 10^8 + 80 \times 0.25 \times (365 \times 24 \times 3600) = 8.2782 \times 10^8}$
245	**100**	$\boldsymbol{V_4 = 8.2782 \times 10^8 + 245 \times 0.25 \times (365 \times 24 \times 3600) = 2.7594 \times 10^9}$

GRAPHICAL REPRESENTATION

a)

Curve that expresses the flow rate variation in a given section over time:

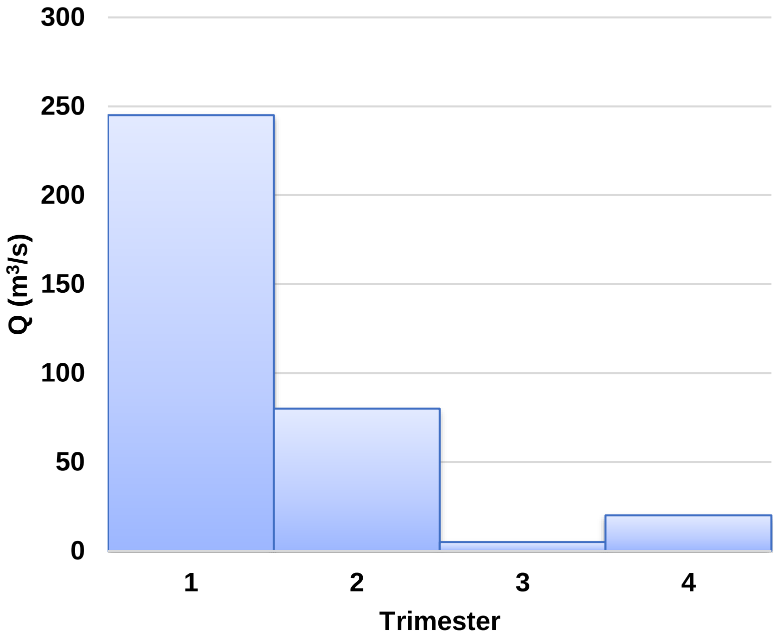


Figure 36 – Chronological flow rates curve.

b)

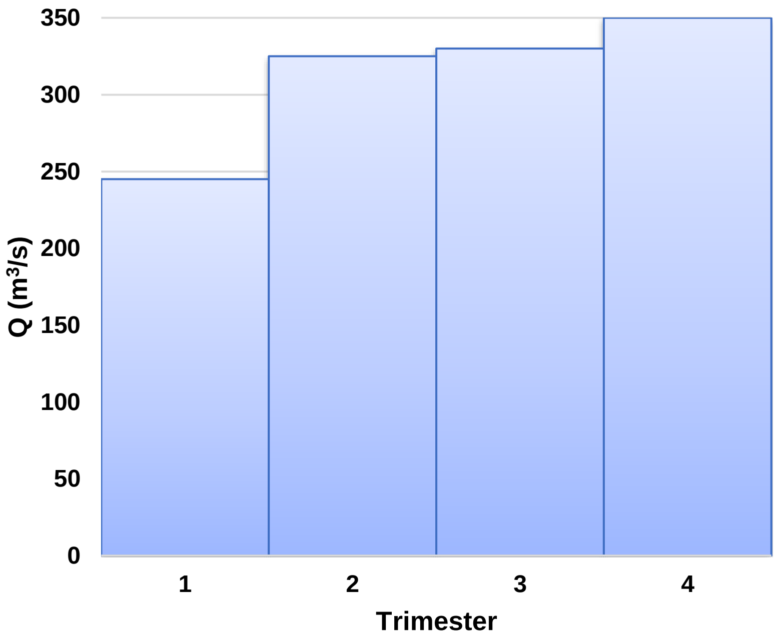


Figure 37 – Accumulated or integral flow rates curve.

c)

The used flow rates can be arranged in ascending or descending order:

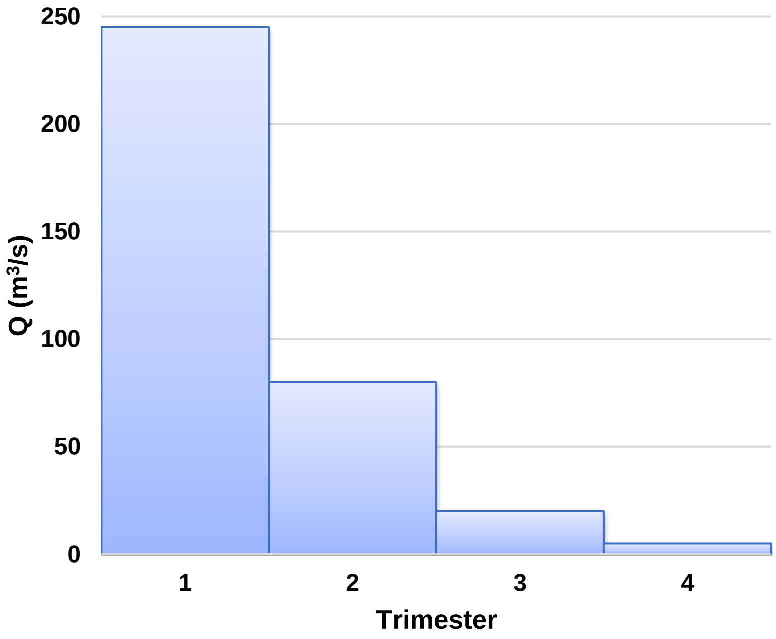


Figure 38 – Classified flow rates curve.

d)

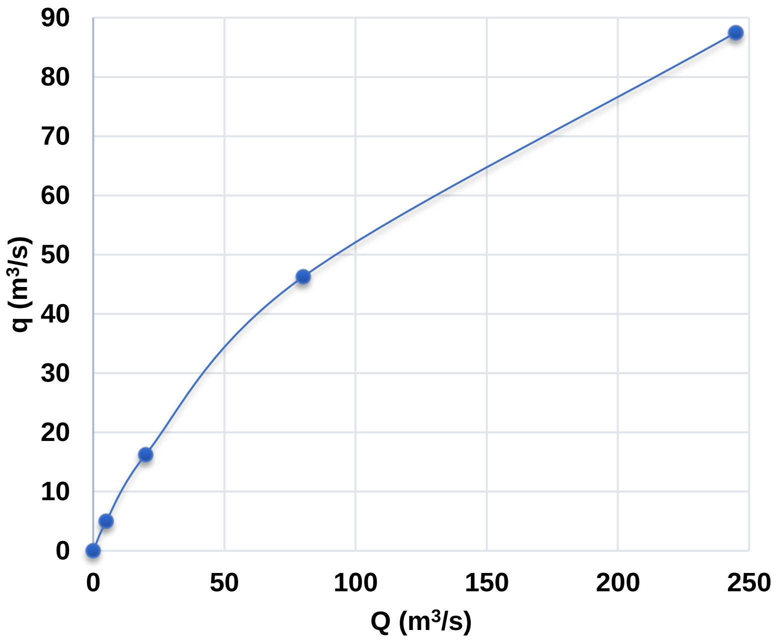


Figure 39 – Hydrologic characteristic curve.

e)

It results from integration in relation to time:

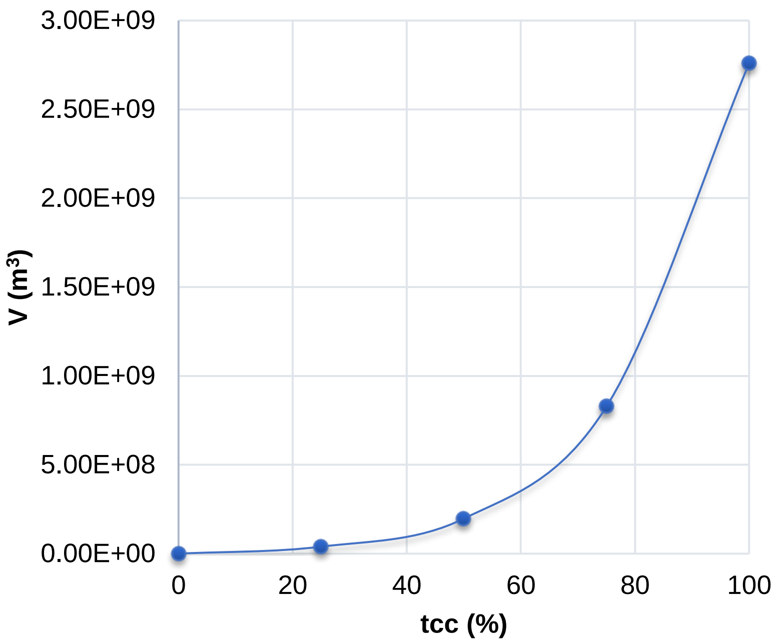


Figure 40 – Concentration curve.

6.2. Answer:

a)

After the construction of the classified flow rates curve:

Table 22 – Construction of the classified flow rates curve.

Trimester	$Q_{cc}\ (m^3/s)$
1	$4Q$
2	$3Q$
3	$2Q$
4	Q

It is calculated value of Q, based in the construction of the hydrologic characteristic curve:

Table 23 – Calculus of Q.

i	$Q\ (m^3/s)$	$q\ (m^3/s)$
0	0	$Q_{installed} = 0 \rightarrow q_0 = 0$
1	Q	$Q_{installed} = Q \rightarrow q_1 = 0 + 0.25 \times (4-0) \times (Q-0) = Q$
2	$2Q$	$Q_{installed} = 2Q \rightarrow q_2 = Q + 0.25 \times (4-3) \times (2Q-Q) = 1.75Q$
3	$3Q$	$Q_{installed} = 3Q \rightarrow q_3 = 1.75Q + 0.25 \times (4-2) \times (3Q-2Q) = 2.25Q$
4	$4Q$	$Q_{installed} = 4Q \rightarrow q_4 = 2.25Q + 0.25 \times (4-3) \times (4Q-3Q) = 2.5Q$

So, for $Q_{installed} = 3Q$:

$$q_3 = 2.25Q = 22.5 \rightarrow Q = 10\ m^3/s$$

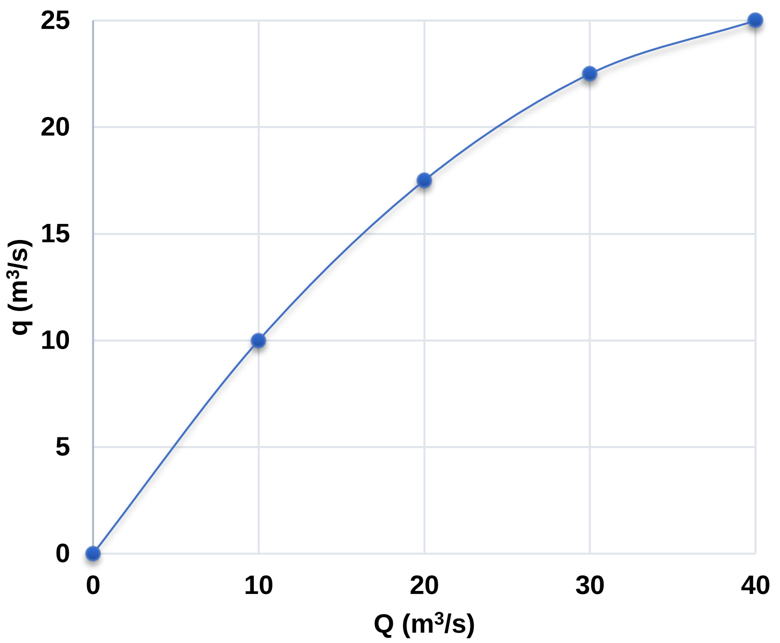


Figure 41 – Hydrologic characteristic curve.

b)

$$Efficiency\ or\ yield \rightarrow \eta = \frac{Used\ volume\ \ (or\ turbined\ volume)}{Affluent\ volume\ \ (or\ installed\ volume)} \times 100$$

Calculus of the used/turbined volume corresponding to a flow rate of $30.0\ m^3/s$:

$$Used\ volume = 0.25 \times t \times \sum_{i=1}^{n=3} Q_i = 0.25 \times (365 \times 24 \times 60 \times 60) \times (10 + 20 + 30)$$

$$Used\ volume = 473040000\ m^3 = 4.7304 \times 10^8\ m^3$$

Calculus of the affluent volume:

$$Affluent\ volume = 0.25 \times t \times \sum_{i=1}^{n=4} Q_i$$

$$Affluent\ volume = 0.25 \times (365 \times 24 \times 60 \times 60) \times (10 + 20 + 30 + 40)$$

$$Affluent\ volume = 788400000\ m^3 = 7.884 \times 10^8\ m^3$$

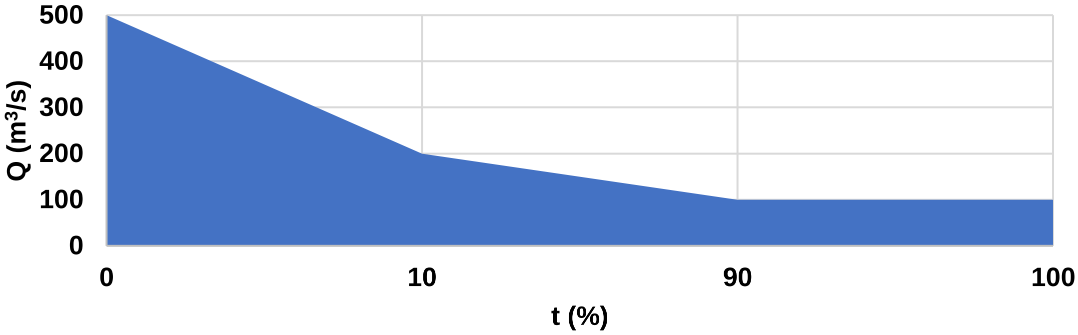
Then:

$$Efficiency\ or\ yield \rightarrow \eta = \frac{4.7304 \times 10^8}{7.884 \times 10^8} \times 100 = 60\%$$

6.3. Answer:

a)

Module - (Module of the classified flow rates) - flow rate that defines a rectangle with the same area of the diagram of classified flow rates:



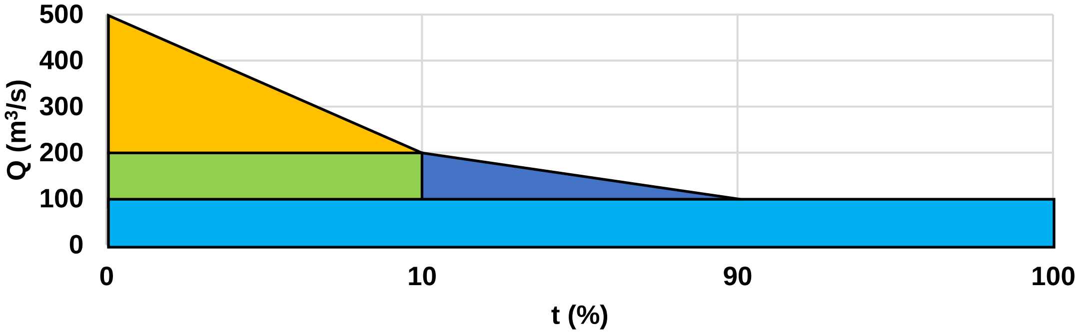


Figure 42 – Area of the diagram of classified flow rates.

$$q_M = |Q| = 1 \times 100 + 0.1 \times (200 - 100) + \frac{(0.9 - 0.1) \times (200 - 100)}{2} + \frac{0.1 \times 500 - 200}{2}$$

$$q_M = |Q| = 165\ m^3/s$$

Semi-permanent flow rate - flow rate equaled or exceeded at 50% of the time:

$$q_2 = \frac{100 - 200}{0.9 - 0.1} \times 0.4 + 200 = 150\ m^3/s$$

b)

Table 24 – Calculus of q.

i	$Q\ (m^3/s)$	$q\ (m^3/s)$
0	$\mathbf{0}$	$q_0 = 0$
1	$\mathbf{100}$	$q_1 = 1 \times 100 = 100$
2	$\mathbf{200}$	$q_2 = 100 + 0.1 \times (200 - 100) + (200 - 100) \times (0.9 - 0.1)/2 = 150$
3	$\mathbf{300}$	$q_3 = 150 + \{[0.1 + (0.1 \times 2/3)] \times (300 - 200)\}/2 = 158.3(3)$
4	$\mathbf{400}$	$q_4 = 158.3(3) + \{[(0.1 \times 1/3) + (0.1 \times 2/3)] \times (400 - 300)\}/2 = 163.3(3)$
5	$\mathbf{500}$	$q_4 = 163.3(3) + [(0.1 \times 1/3) \times (500 - 400)]/2 = 165$

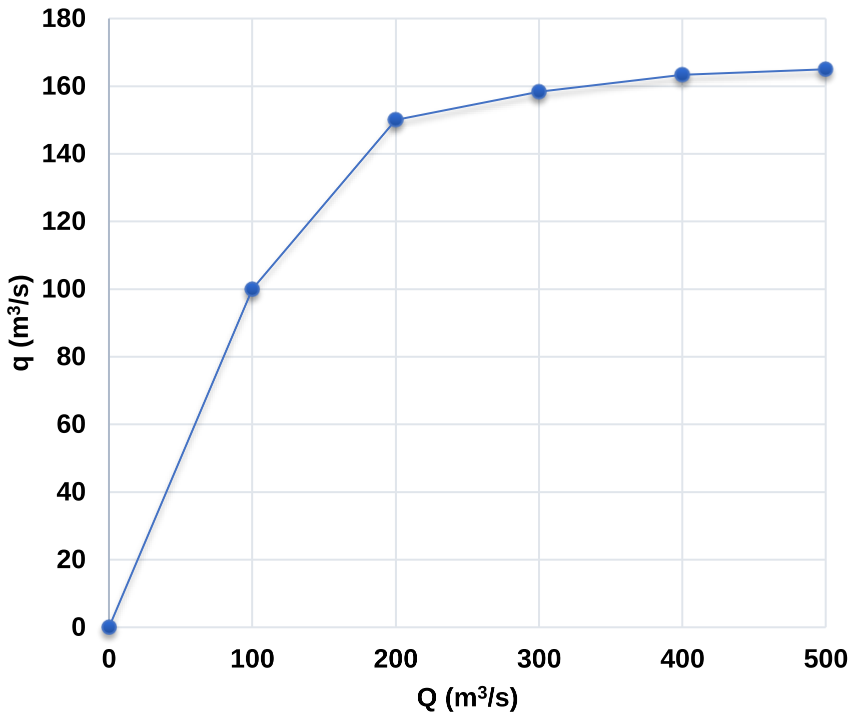


Figure 43 – Curve q(Q).

c)

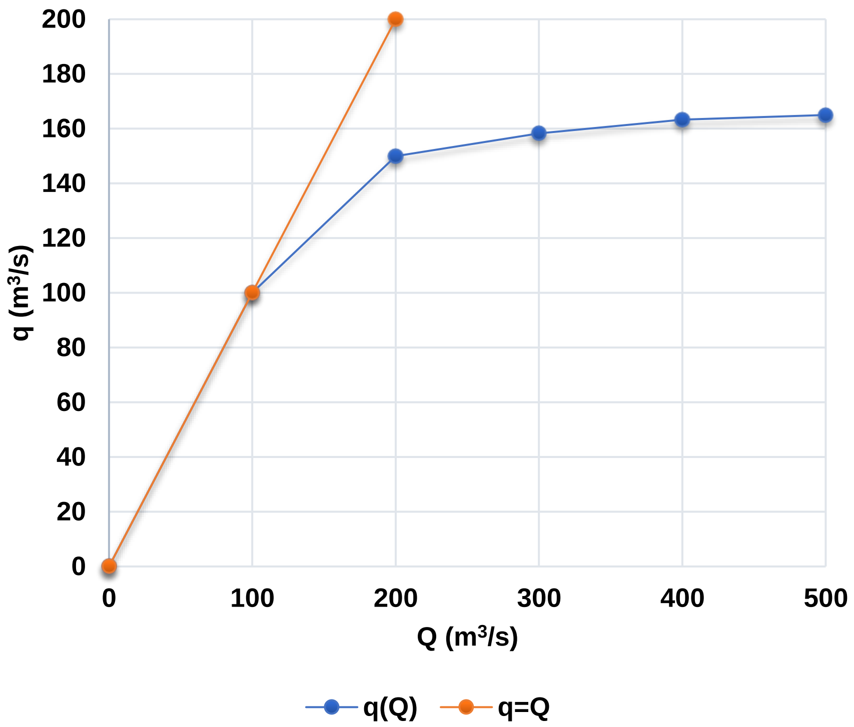


Figure 44 – Economical evaluation.

The hydrological characteristic curve relates the average used flow rates used the maximum turbinable. The closer the curve approaches the line at 45º $(q = Q)$, the more economical the installation will be, because lower will be the difference between the average usable flow rates and the maximum turbinable. This curve allows to evaluate the economy of a facility.

The range of flow rates that makes the installation most economical is $[0; 100]\ m^3/s$.

d)

Concentration curve - integrated curve in relation to time and to the extension between end and beginning of the classified flow rates curve:

Table 25 – Calculus of V.

$t_{cc}(\%)$	$V\ (m^3)$
0	$V_0 = 0$
10	$V_1 = 0 + 100 \times 0.1 \times (365 \times 24 \times 3600) = 3.1536 \times 10^8$
20	$V_2 = [(100 \times 0.2) + (1/8 \times 100 \times 0.1)/2] \times (365 \times 24 \times 3600) = 6.5043 \times 10^8$
30	$V_3 = [(100 \times 0.3) + (2/8 \times 100 \times 0.2)/2] \times (365 \times 24 \times 3600) = 1.02492 \times 10^9$
40	$V_4 = [(100 \times 0.4) + (3/8 \times 100 \times 0.3)/2] \times (365 \times 24 \times 3600) = 1.43883 \times 10^9$
50	$V_5 = [(100 \times 0.5) + (4/8 \times 100 \times 0.4)/2] \times (365 \times 24 \times 3600) = 1.89216 \times 10^9$
60	$V_6 = [(100 \times 0.6) + (5/8 \times 100 \times 0.5)/2] \times (365 \times 24 \times 3600) = 2.38491 \times 10^9$
70	$V_7 = [(100 \times 0.7) + (6/8 \times 100 \times 0.6)/2] \times (365 \times 24 \times 3600) = 2.91708 \times 10^9$
80	$V_8 = [(100 \times 0.8) + (7/8 \times 100 \times 0.7)/2] \times (365 \times 24 \times 3600) = 3.48867 \times 10^9$
90	$V_9 = [(100 \times 0.9) + (100 \times 0.8)/2] \times (365 \times 24 \times 3600) = 4.09968 \times 10^9$
100	$V_{10} = \lvert Q \rvert \times (365 \times 24 \times 3600) = 165 \times (365 \times 24 \times 3600) = 5.20344 \times 10^9$

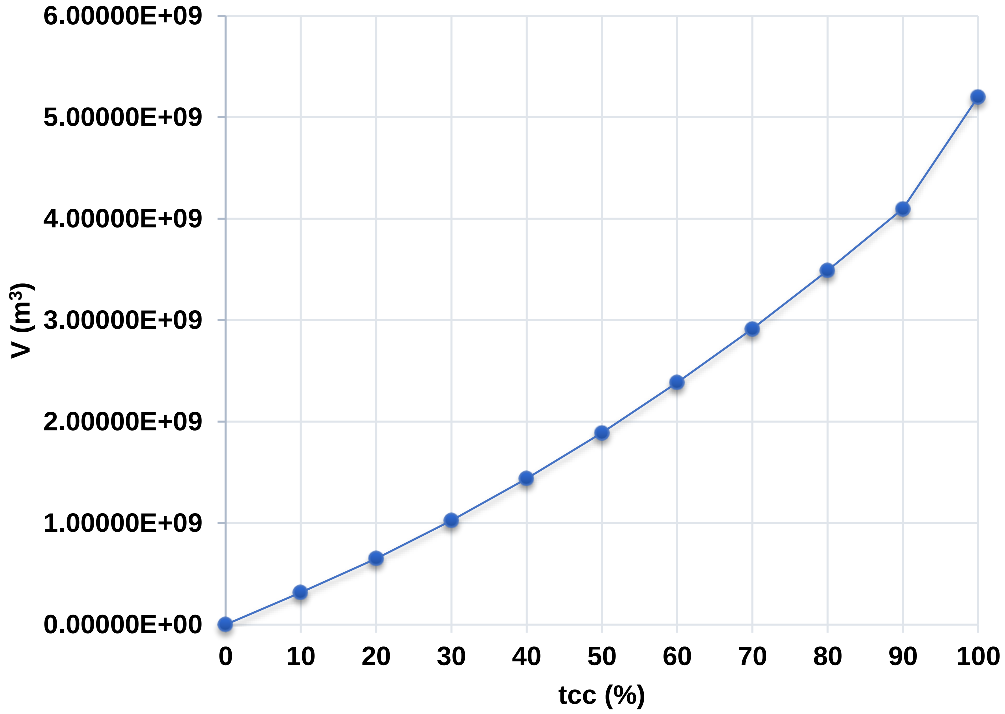


Figure 45 – Concentration curve.

e)

The yield or efficiency of a facility is given by:

$$\eta = \frac{Used\ volume}{Affluent\ volume} \times 100 = \frac{Affluent\ volume - Discharged\ volume}{Affluent\ volume} \times 100$$

Knowing that the installed flow rate is equal to the semi-permanent flow rate, it is possible to trace a tangent line to the concentration curve at the point $(tcc_{50\%}, V_{50\%})$, which express the real consumed volume by the facility:

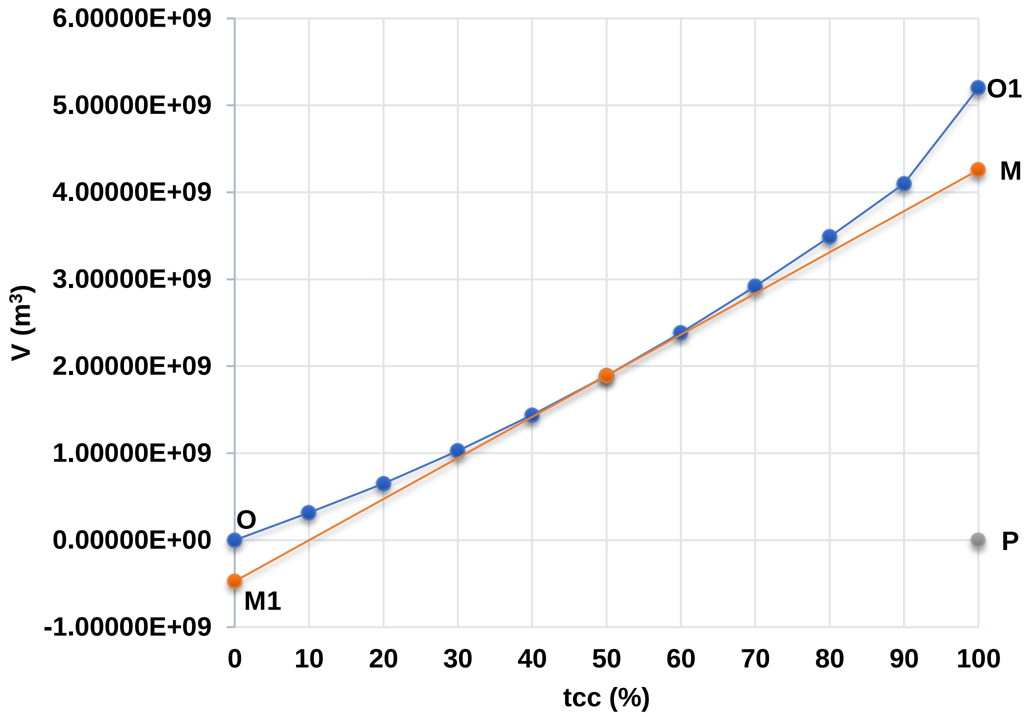


Figure 46 – Concentration curve and real consumed volume by the facility.

The yield calculated based in the such information at the upper graph, results in:

$$\eta = \frac{\overline{PM}}{\overline{PO1}} \times 100 = \frac{\overline{PO1} - \overline{MO1}}{\overline{PO1}} \times 100$$

Where $\overline{MO1} = \overline{PO1} - \overline{PM}$ is the difference between the total affluent volume ($\overline{PO1}$) at the section of the watercourse and the real consumed volume by the facility ($\overline{PM}$). Corresponds to a discharged volume of such facility.

Total affluent volume:

$$\overline{PO1} = 5.20344 \times 10^9\ m^3$$

The discharged volume ($\overline{MO1}$) results from the area of the classified flow rates curve that is above of the installed flow rate of $150\ m^3/s$ (or Zone 1), as in the following figure:

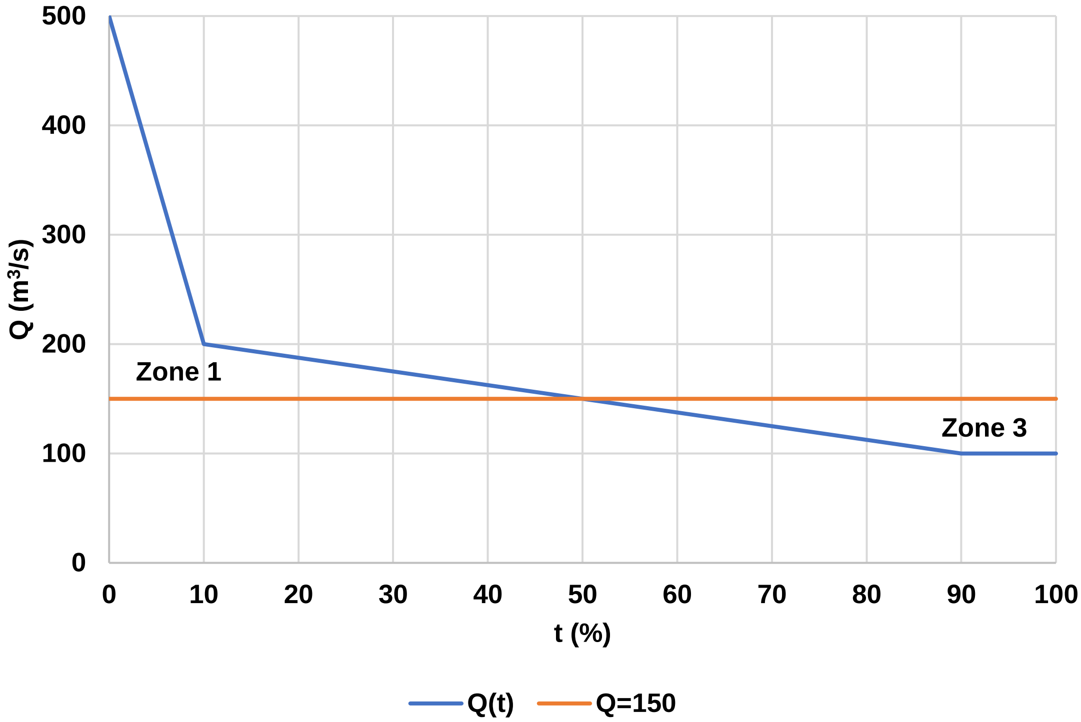


Figure 47 – Classified flow rates curve and installed flow rate.

Then, the discharged volume:

$$\overline{MO1} = \left[\left(\frac{0.4 \times 50}{2}\right) + 0.1 \times 50 + \left(\frac{0.1 \times 300}{2}\right)\right] \times (365 \times 24 \times 3600) = 9.46 \times 10^{8}\ m^{3}$$

Finally, the yield of the facility is:

$$\eta = \frac{5.20344 \times 10^{9} - 9.46 \times 10^{8}}{5.20344 \times 10^{9}} \times 100 \cong 81.8\%$$

f)

By consulting Figure 46, the volume to be obtained from another basin, so that the facility can work all year at full load corresponds to the section $\overline{OM1}$, which represents the total volume deficit in the operation of such facility below the maximum flow rate. It is the volume of water that has to be borrowed from another river to be able to have all year (100%) of harnessed flow rate.

The volume to borrow is $(\overline{OM1})$ and result from the area of the classified flow rates curve that is below the installed flow rate of $150\ m^{3}/s$ (or Zone 3), as in Figure 47. So:

$$\overline{OM1} = \left[0.1 \times 50 + \left(\frac{0.4 \times 50}{2}\right)\right] \times (365 \times 24 \times 3600) = 4.73 \times 10^{8}\ m^{3}$$

Chapter 7 - Study of floods

7.1. Answer:

a)

For a rainfall with the same duration and twice the intensity, the resulting hydrograph is the sum of the unit hydrograph (HU) with a hydrograph with the same duration and intensity of the first ($to\ t_i = t_{i,HU} \rightarrow Q_i = 2Q_{i,HU},$):

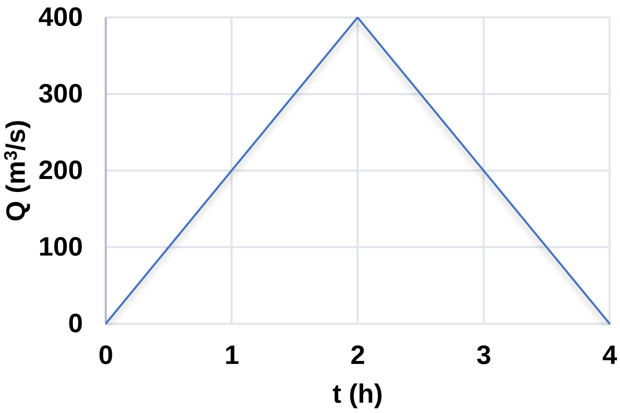


Figure 48 – Rainfall with same duration and twice the intensity.

Peak flow rate is $400\ m^3/s$.

b)

For a rainfall with the same intensity and twice the duration, the resulting hydrograph is the sum of the HU with a hydrograph with the same intensity and lag of $1\ hour$ from the first ($Q_i = Q_{i,HU} + Q_{i,HU+1}$):

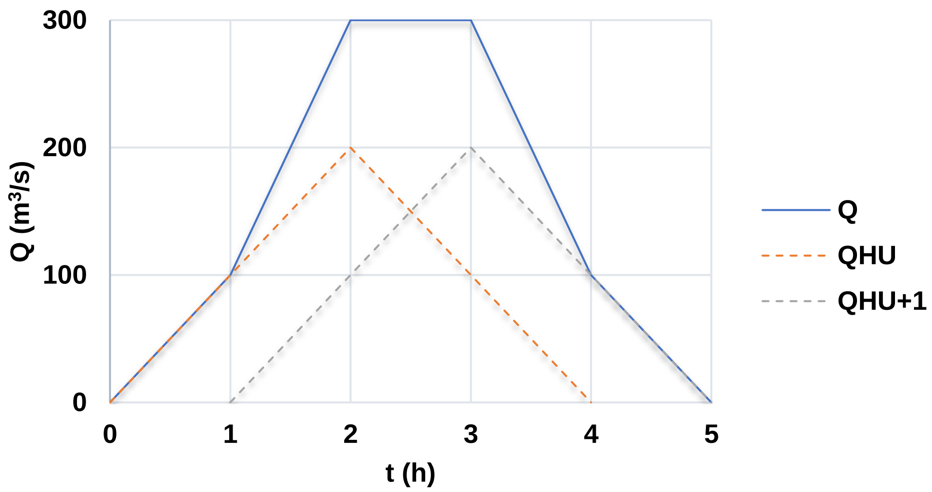


Figure 49 – Rainfall with same intensity and twice the duration.

Peak flow rate is $300\ m^3/s$.

c)

For a rainfall with twice the intensity and a duration of $4\ hours$, the resulting hydrograph is the sum of the HU with twice the original intensity with successive hydrographs with that same intensity, but lagged by 1, 2 and $3\ hours$ from the first one ($Q_i = 2Q_{i,HU} + 2Q_{i,HU+1} + 2Q_{i,HU+2} + 2Q_{i,HU+3}$):

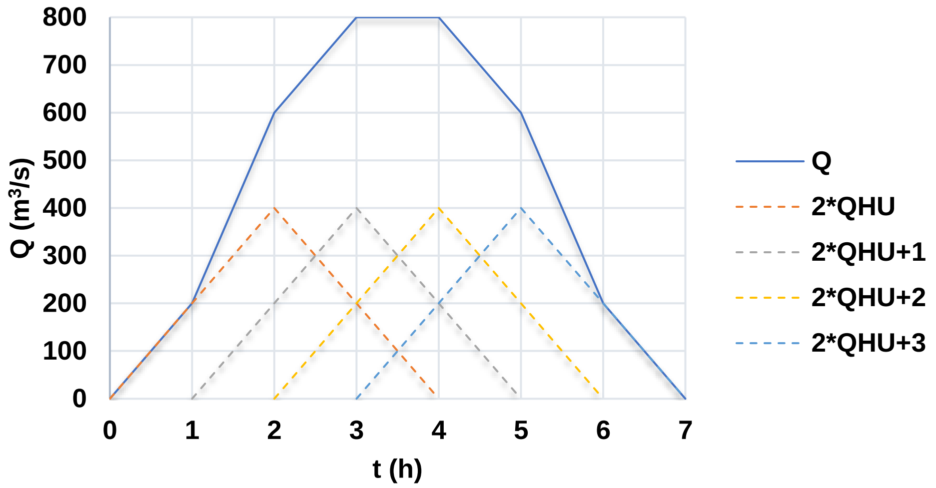


Figure 50 – Rainfall with twice the intensity and duration of 4 hours.

Peak flow rate is $800\ m^3/s$.

d)

For a rainfall with the following temporal distribution:

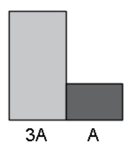


Figure 51 – Temporal distribution of a rainfall.

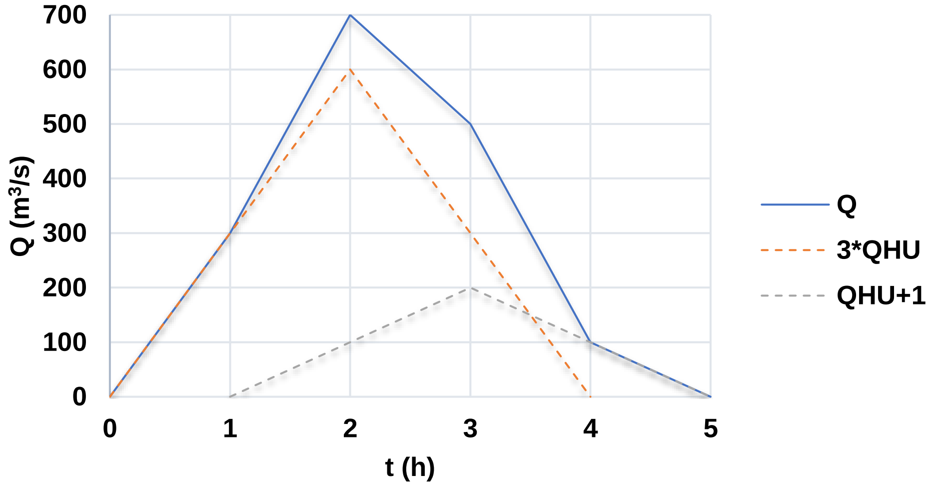


Figure 52 – Rainfall with temporal distribution of 3A+A.

The resulting hydrograph is the sum of the HU with the triple of the original intensity with a hydrograph with the same intensity of the original HU, but lagged by $1\ hour$ from the first one ($Q_i = 3Q_{i,HU} + Q_{i,HU+1}$). The peak flow rate is $700\ m^3/s$.

7.2. Answer:

a)

The resulting hydrograph is the sum of the HU with twice the original intensity with a hydrograph with the same intensity as the original HU, but lagged by $1\ hour$ from the first one ($Q_i = 2Q_{i,HU} + Q_{i,HU+1}$):

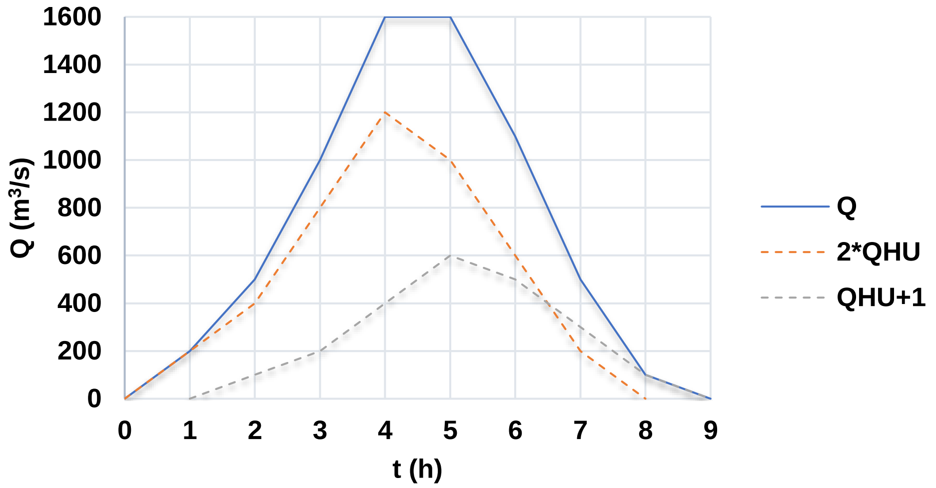


Figure 53 – Flood hydrograph resulting from precipitation B.

b)

The peak flow rate is $1600\ m^3/s$. The time of concentration is $[4{,}5]\ hours$.

c)

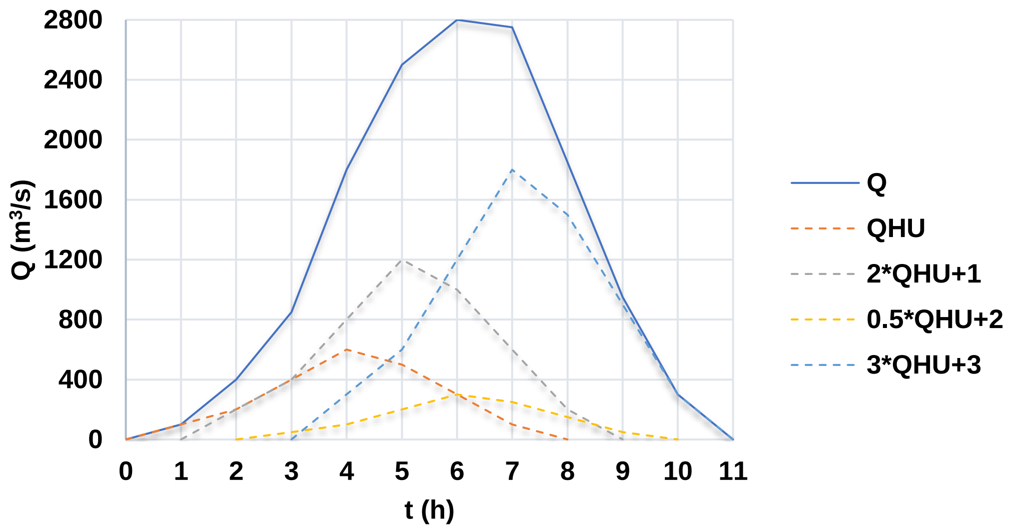


Figure 54 – Flood hydrograph resulting from precipitation C.

The resulting hydrograph is the sum of the HU with successively lagged hydrographs by 1, 2 and 3 $hours$, with intensity of 2, 0.5 and 3 times the original intensity of the HU, respectively ($Q_i = Q_{i,HU} + 2Q_{i,HU+1} + 0.5Q_{i,HU+2} + 3Q_{i,HU+3}$). The peak flow rate is $2800\ m^3/s$.The time of concentration is $6\ hours$.

7.3. Answer:

Base time of the unit hydrograph is $7 - 2 = 5\ hours$.

$$t = 0 \rightarrow Q = 0$$

$$t = 1 \rightarrow Q = 200 = A_1 + 0 + 0 \rightarrow A_1 = 200$$

$$t = 2 \rightarrow Q = 800 = A_2 + 2 \times A_1 + 0 \rightarrow A_2 = 400$$

$$t = 3 \rightarrow Q = 1200 = A_3 + 2 \times A_2 + 0.5 \times A_1 \rightarrow A_3 = 300$$

$$t = 4 \rightarrow Q = 920 = A_4 + 2 \times A_3 + 0.5 \times A_2 \rightarrow A_4 = 120$$

$$t = 5 \rightarrow Q = 390 = A_5 + 2 \times A_4 + 0.5 \times A_3 \rightarrow A_5 = 0$$

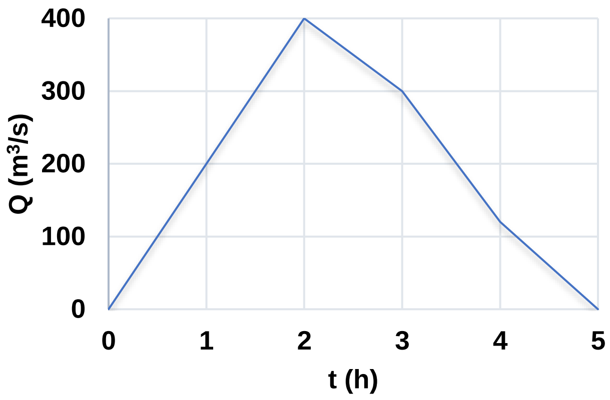


Figure 55 – HU for the given flood hydrograph.

7.4. Answer:

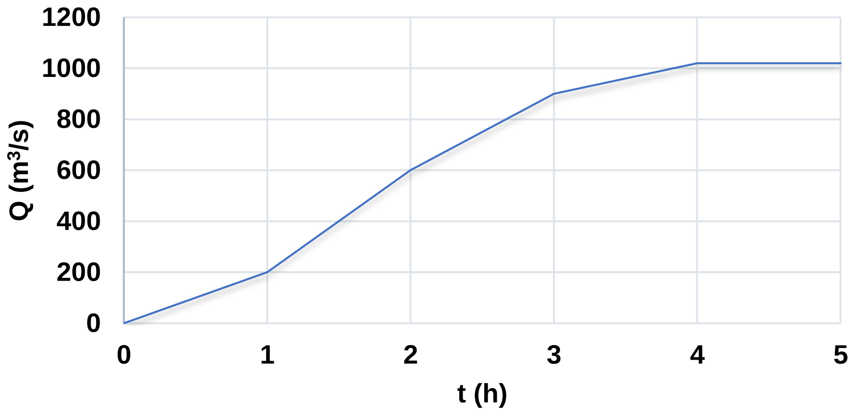


Figure 56 – “S” hydrograph.

The S hydrograph results from a series of rainfalls, juxtaposed, each with a unitary useful precipitation. Therefore, it is obtained from the sum of unitary hydrographs for a duration D of the useful precipitation, each lagged by D from the previous hydrograph.

7.5. Answer:

a)

Base time of the unit hydrograph is $5\ hours$.

$$t = 0 \rightarrow Q = 0 = A_0$$

$$t = 1 \rightarrow Q = 25 = A_0 + A_1 \rightarrow A_1 = 25$$

$$t = 2 \rightarrow Q = 75 = A_0 + A_1 + A_2 \rightarrow A_2 = 50$$

$$t = 3 \rightarrow Q = 150 = A_0 + A_1 + A_2 + A_3 \rightarrow A_3 = 75$$

$$t = 4 \rightarrow Q = 250 = A_0 + A_1 + A_2 + A_3 + A_4 \rightarrow A_4 = 100$$

$$t = 5 \rightarrow Q = 250 = A_0 + A_1 + A_2 + A_3 + A_4 + A_5 \rightarrow A_5 = 0$$

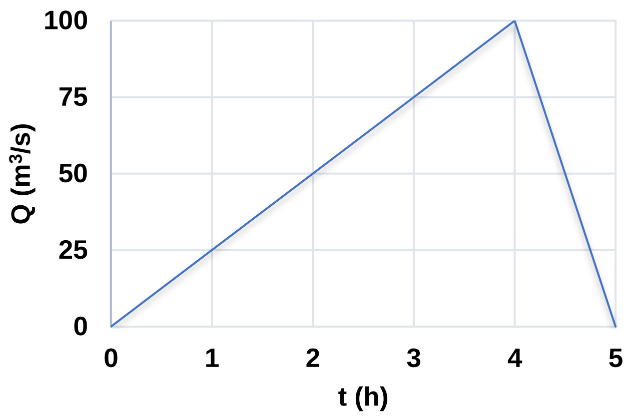


Figure 57 – HU for the given "S" hydrograph.

b)

The peak flow that corresponds to a unitary hydrograph is $100\ m^3/s$.

7.6. Answer:

To draw a flood hydrograph by Giandotti's method, the formulas presented in the theoretical support of Chapter 7 are used, using the following coefficients:

Table 26 – Coefficients.

$A\ (\mathrm{km}^2)$	γ	λ	Ψ
$A_b = 362 \rightarrow [300; 500]$	8.0	4.0	0.50

Table 27 – Points for the construction of a flood hydrograph.

$t\ (h)$	$Q\ (m^3/s)$
$t_0 = 0$	$Q_0 = 0$
$t_a = \left(1 - \frac{1}{8}\right) \times 10.23 = 8.95$	$Q_{avg} = 1068.41/8 = 133.55$
$t_c = \frac{4\sqrt{362} + 1.5 \times 33.8}{0.8 \times \sqrt{240}} = 10.23$	$Q_{max} = \frac{277 \times 0.5 \times 8 \times 0.109 \times 362}{4 \times 10.23} = 1068.41$
$t_b = \left(\frac{4-1}{8} + 1\right) \times 10.23 = 14.07$	$Q_{avg} = 1068.41/8 = 133.55$
$t_t = 4 \times 10.23 = 40.92$	$Q_t = 0$

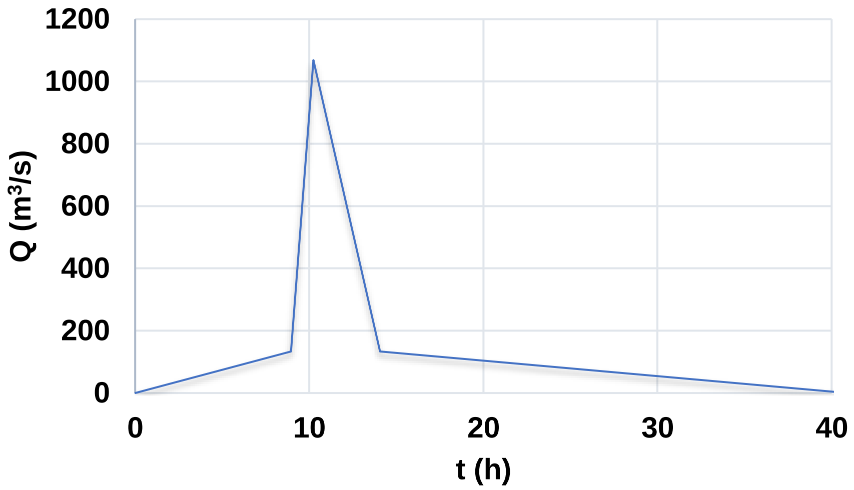


Figure 58 – Flood hydrograph by Giandotti's method.

7.7. Answer:

a)

Obtaining the coefficient:

$$t_a = \left(1 - \frac{1}{\gamma}\right) \times 12 = 10 \rightarrow (\gamma - 1) \times 12 = 10\gamma \rightarrow 12\gamma - 10\gamma = 12 \rightarrow \gamma = \frac{12}{2} = 6$$

Then, as $A_b < 5000\ km^2 \rightarrow [1000; 8000]$ and $\gamma = 6$, then, $\lambda = 5$ and $\Psi = 0.3$.

$$V_{flood} = Q_{avg} \times t_t \rightarrow Q_{avg} = \frac{21.6 \times 10^6}{60 \times 3600} = 100\ m^3/s$$

$$\gamma = \frac{Q_{max}}{Q_{avg}} \rightarrow Q_{max} = 6 \times 100 = 600\ m^3/s$$

Table 28 – Points for the construction of the flood hydrograph.

$t\ (h)$	$Q\ (m^3/s)$
$t_0 = 0$	$Q_0 = 0$
$t_a = 10$	$Q_{avg} = 100$
$t_c = 12$	$Q_{max} = 600$
$t_b = \left(\frac{5-1}{6} + 1\right) \times 12 = 20$	$Q_{avg} = 100$
$t_t = 5 \times 12 = 60$	$Q_t = 0$

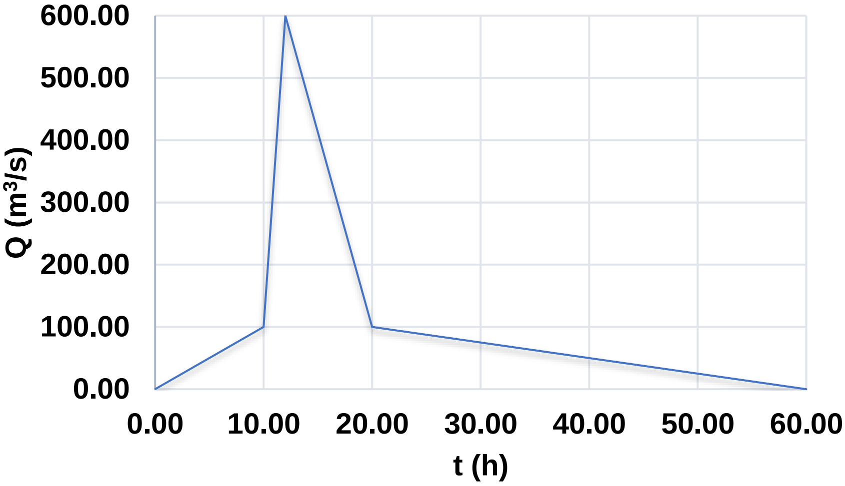


Figure 59 – Complete flood hydrograph.

b)

$$Q_{max} = \frac{277 \times 0.3 \times 6 \times 0.048 \times A}{5 \times 12} = 600 \rightarrow A = \frac{600 \times 5 \times 12}{277 \times 0.3 \times 6 \times 0.048} \cong 1504\ km^2$$

$$\frac{t_b - t_t}{Q_{avg} - 0} = \frac{t_b - 30}{Q_{avg} - Q_{30}} \rightarrow (100 - Q_{30}) \times (20 - 60) = 100 \times (20 - 30) \rightarrow$$

$$\rightarrow -4000 + 40Q_{30} = -1000 \rightarrow Q_{30} = \frac{3000}{40} = 75\ m^3/s$$

c)

$$Giandotti: Q_P = \frac{\lambda \times A \times h}{t_c} = \frac{0.1 \times 1504 \times 48}{12} \cong 602\ m^3/s$$

Note:

- $A = 1504\ km^2 \rightarrow \lambda = 0.1;$
- $t_c = 12\ h;$
- $h = 48\ mm.$

7.8. Answer:

a)

To determine the discharge coefficient:

$$C = \frac{I_{effective}}{I_{total}} = \frac{I_{effective}}{25}$$

Knowing that:

$$V = Q \times t \rightarrow V = area\ of\ the\ diagram$$

$$0.54 \times 10^6 = \left[\left(\frac{0.5 \times A}{2}\right) + \left(\frac{3A + A}{2} \times 0.5\right) + \left(\frac{3A + 2A}{2} \times 0.5\right) + \left(\frac{0.5 \times 2A}{2}\right)\right] \times 3600 \rightarrow$$

$$\rightarrow 0.54 \times 10^6 = (0.25A + A + 1.25A + 0.5A) \times 3600 \rightarrow 0.54 \times 10^6 = 3A \times 3600 \rightarrow$$

$$\rightarrow A = \frac{0.54 \times 10^6}{3 \times 3600} = 50\ m^3/s$$

Then:

$$Q_{max} = 3A = 150\ m^3/s$$

And:

$$Q_{max} = I_{effective} \times A_b \rightarrow I_{effective} = \frac{150}{54 \times 10^6} \times (3600 \times 10^3) = 10\ mm/h$$

Then:

$$C = \frac{10}{25} = 0.4$$

To determine the maximum flow rate for a rain of total intensity of $20\ mm/h$ and adopting $C = 0.4$ (because the discharge coefficient of the basin is an intrinsic property of it):

$$C = \frac{I_{effective}}{I_{total}} \rightarrow 0.4 = \frac{I_{effective}}{20} \rightarrow I_{effective} = 8\ mm/h$$

Then:

$$Q_{max} = I_{effective} \times A_b = 8 \times \left(\frac{10^{-3}}{3600}\right) \times 54 \times 10^6 = 120\ m^3/s$$

b)

The base time of the hydrographs B and C is $t = 2 - 0.5 = 1.5\, hours$, being the hydrograph C lagged by $0.5\, h$. The hydrograph of Figure 31 results from the sum of the hydrographs resulting from precipitations B and C, so:

$$t = 1 \rightarrow 200 = i_B + i_C$$

Adopting $C = 2B, lagged\ by\ 0.5\ hours$, then:

$$t = 1 \rightarrow 200 = i_B + 2i_B(t = 0.5) \rightarrow i_B = 200 - 2 \times 75 = 50\, mm/h \rightarrow$$

$$\rightarrow i_C = 150\, mm/h$$

Table 29 – Hydrographs.

$T\ (hours)$	$B + C\ (mm/h)$	$B\ (mm/h)$	$C\ (mm/h)$
0	0	0	0
0.5	75	75	0
1	200	50	150
1.5	100	0	100
2	0	0	0

The precipitations B and C are calculated as follows:

$$B = i_B(t = 0.5) \times 0.5 = 75 \times 0.5 = 37.5\, mm$$

$$C = 2B = 2 \times 37.5 = 75\, mm$$

Chapter 8 - Regularization effect of a reservoir on flood peak damping

8.1. Answer:

For the study of flood damping, it is necessary to calculate the following parameters:

- $Q_A = Affluent\ flow\ rate = Area\ of\ the\ hydrgraph\ until\ instant\ t$;
- $V_A = Q_A \times 3600$;
- $Y_{EST} = arbitrated\ value$;
- $Q_E = discharged\ flow\ rate\ = flow\ rate\ curve\ of\ the\ spillway$;
- $\bar{Q}_E = average\ discharged\ flow\ rate\ = \bar{Q}_{E,i} + \bar{Q}_{E,i-1}/2$
- $V_E = \bar{Q}_E \times 3600$;
- $V_{stored} = V_A - V_E$;
- $V_{stored,acm} = V_{stored,acm,i-1} + V_{stored,i}$;
- $Y_{CALC} = interpolated\ value\ from\ the\ height - volume\ curve.$

And define a value of accuracy/precision for the calculus:

$$Y_{CALC} - Y_{EST} \leq 0.0005$$

Table 30 – Study of peak flood damping.

t	Δt	Q_A	V_A	Y_{EST}	Q_E	$\bar{Q}_E$	V_E	V_{stored}	$V_{stored.acm}$	Y_{CALC}
0	-	10	-	100.0000	0.00	-	-	-	1060000.00	100.0000
1	1	15	54000.00	100.1000	0.40	0.20	711.51	53288.49	1113288.49	100.0720
				100.0720	0.24	0.12	434.80	53565.20	1113565.20	100.0724
2	1	35	126000.00	100.2000	1.12	0.68	2447.26	123552.74	1237117.95	100.2393
				100.2393	1.46	0.85	3069.48	122930.52	1236495.72	100.2385

For $Y_{CALC} = 100.0724$:

$$Y_{CALC} - Y_{EST} = 100.0724 - 100.0720 = 0.0004 \leq 0.0005 \rightarrow Verifies!$$

And, therefore, $100.0724\ m$ is the value for the point where the two hydrographs (affluent and effluent) intersect, which corresponds to the maximum output flow rate, because the discharge curve increases with the water level (volume) and for which the stored volume is maxed.

Chapter 9 - Evaporation and Evapotranspiration

9.1. Answer:

Table 31 – Estimation of monthly and annual potential evapotranspiration by Turc's method.

Turc									
$\boldsymbol{Month}$	D_m $(days)$	T (°C)	n (h)	H_{0m} (h)	I_0 $(MJ/m^2/d)$	r_H	I'_g $(MJ/m^2/d)$	ETP_d (mm)	$\boldsymbol{ETP_m}$ $\boldsymbol{(mm)}$
Jan	30	5.1	3.9	9.5	15.15	0.41053	6.78082	0.69746	**20.9239**
Feb	30	6	5.2	10.5	20.15	0.49524	9.92340	1.06329	**31.8988**
Mar	30	8.5	5.7	11.7	27.21	0.48718	13.2841	1.72233	**51.66987**
Apr	30	10.6	7.8	13.1	34.32	0.59542	18.7241	2.66886	**80.06568**
May	30	14.3	10	14.2	39.28	0.70423	23.6953	3.89675	**116.9024**
June	30	18.6	11.6	14.8	41.29	0.78378	26.6488	4.92592	**147.7776**
July	30	21.7	12.8	14.5	40.17	0.88276	28.0331	5.51483	**165.4448**
Aug	30	21	12	13.5	35.99	0.88889	25.2330	4.93512	**148.0537**
Sept	30	18	7.8	12.2	29.46	0.63934	16.7584	3.18389	**95.51654**
Oct	30	13.6	6.2	10.9	22.08	0.56881	11.7348	2.03629	**61.08878**
Nov	30	8	4.6	9.7	16.16	0.47423	7.77846	1.06352	**31.90558**
Dec	30	5.1	3.2	9.2	13.61	0.34783	5.63927	0.60781	**18.23432**
a	**0.23**	**b**	**0.53**					**Annual**	**969.4819**

Table 32 – Estimation of monthly and annual potential evapotranspiration by Thornthwaite's method.

Thornthwaite							
$\boldsymbol{Month}$	D_m $(dias)$	T (°C)	n (h)	H_{0m} (h)	N_m $(-)$	i_m $(-)$	$\boldsymbol{ETP_m}$ $\boldsymbol{(mm)}$
Jan	30	5.1	3.9	9.5	0.791667	1.030150	**12.44985**
Feb	30	6	5.2	10.5	0.875000	1.314534	**17.01333**
Mar	30	8.5	5.7	11.7	0.975000	2.216529	**29.87447**
Apr	30	10.6	7.8	13.1	1.091667	3.086767	**44.62597**
May	30	14.3	10	14.2	1.183333	4.836699	**71.51358**
June	30	18.6	11.6	14.8	1.233333	7.174876	**105.0620**
July	30	21.7	12.8	14.5	1.208333	9.041377	**125.8827**
Aug	30	21	12	13.5	1.125000	8.607439	**112.2892**
Sept	30	18	7.8	12.2	1.016667	6.830520	**82.97547**
Oct	30	13.6	6.2	10.9	0.908333	4.485939	**51.41213**
Nov	30	8	4.6	9.7	0.808333	2.023858	**22.88272**
Dec	30	5.1	3.2	9.2	0.766667	1.030150	**12.05670**
I	**0.23**	**a**	**0.53**			**Annual**	**688.0381**

According to some investigations, in Portugal, the Thornthwaite's formula seems to lead to estimates of potential evapotranspiration rather by defect and the Turc's formula, to estimates slightly by excess.

Chapter 10 - Water in soil

10.1. Answer:

Infiltration capacity for a given instant:

$$f(t) = f_c + (f_0 - f_c)e^{-Kt}$$

Then for $t = 3\ h$,

$$f(3) = 8 + (38 - 8)e^{-1.11\times 3} \cong 9.1\ mm/h$$

10.2. Answer:

$$\theta = \frac{V_w}{V_t}$$

Volumetric amount of moisture in the sample:

$$\theta = \frac{V_w}{V_t} = \frac{\frac{m_w}{\rho_w}}{V_{cylinder}} = \frac{\frac{(331.8 - 302.4)}{1000} \times \frac{10^6}{10^3}}{\pi \times \left(\frac{5}{2}\right)^2 \times 10} \cong 0.15$$

Degree of saturation:

$$S = \frac{V_w}{V_f} = \frac{V_w}{V_w + V_a} \cong 0.36$$

$$V_a = V_t - V_w - V_s = \left[\pi \times \left(\frac{5}{2}\right)^2 \times 10\right] - \left[\frac{(331.8 - 302.4)}{1000} \times \frac{10^6}{10^3}\right] - \left[\frac{302.4}{2650} \times \frac{10^6}{10^3}\right]$$

$$V_s = \frac{m_s}{\rho_s} = \frac{302.4}{2650} \times \frac{10^6}{10^3}$$

10.3. Answer:

$$V_{w,out} = V_{w,in} - V_{w,retain} = V_{w,in} - [(\theta_{cc} - \theta) \cdot V_t] =$$

$$V_{w,out} = 1 - [(0.28 - 0.15) \times 5] = 0.35\ L$$

10.4. Answer:

Volume of irrigation required in order to go from minimum volumetric amount of moisture allowable to the field capacity:

$$V_{w,irrigation} = V_{w,cc} - V_{w,min} = [(\theta_{cc} - \theta) \cdot V_t] = [(\theta_{cc} - \theta) \cdot (A \times h)]$$

$$V_{w,irrigation} = [(0.45 - 0.24) \times (1 \times 10^4 \times 0.5)] = 1050\ m^3$$

To determine the time interval, it is used the following data:

$$H_{w,irrigation} = \frac{V_{w,irrigation}}{A} = \frac{1050}{1 \times 10^4} = 0.105\ m$$

$$EVT = 3\ mm/d = 0.003\ m/d$$

Knowing that the average evapotranspiration is $3\ mm/d$, the time interval between two successive irrigations:

$$\begin{array}{rcl} m & - & days \\ EVT = 0.003 & - & 1 \\ H_{w,irrigation} = 0.105 & - & x \end{array}$$

$$x = \frac{0.105 \times 1}{0.003} = 35\ days$$

Chapter 11 - Groundwater

11.1. Answer:

a)

$$\Delta h = i_{hor} \times h$$

Table 33 – Charge loss Δh.

$\boldsymbol{Layer}$	$\boldsymbol{\Delta h}\ (\boldsymbol{m})$
1	0.010
2	0.005
3	0.015
4	0.010
5	0.030

b)

$$c_h = \frac{L}{K}$$

Table 34 – Horizontal hydraulic resistance.

$\boldsymbol{Layer}$	$\boldsymbol{c_h}\ (\boldsymbol{day})$
1	200
2	100000
3	50
4	200000
5	1000

c)

$$c_v = \frac{h}{K}$$

Table 35 – Vertical hydraulic resistance.

$\boldsymbol{Layer}$	$\boldsymbol{c_v}\ (\boldsymbol{day})$
1	2
2	500
3	0.75
4	2000
5	30

d)

$$T = K \times h$$

Table 36 – Transmissivity.

$\boldsymbol{Layer}$	$\boldsymbol{T}\ (\boldsymbol{day}^{-1})$
1	50
2	0.05
3	300
4	0.05
5	30

e)

$$q_{hor} = K \times h \times i_{hor}$$

Table 37 – Specific flow rate (horizontal).

$\boldsymbol{Layer}$	$\boldsymbol{q_{hor}}\ (\boldsymbol{m^2/day})$
1	0.05
2	0.00005
3	0.3
4	0.00005
5	0.03

f)

$$K_{eq,hor} = \frac{1}{h}\sum_j \left(K_j h_j\right)$$

Table 38 – Equivalent permeability (horizontal).

$\boldsymbol{Layer}$	$\boldsymbol{K_{eq,hor}}\ (\boldsymbol{m/day})$
1	0.71429
2	0.00071
3	4.28571
4	0.00071
5	0.42857
$\boldsymbol{Total}$	5.43

11.2. Answer:

Expression for the permeability coefficient (K) for an artesian aquifer:

$$K = \frac{Q \cdot \ln(R_2/R_1)}{2\pi m(s_1 - s_2)} = \frac{0.1 \cdot \ln(30/10)}{2\pi \times 40 \times (10-7)} \approx 0.000146\, m/s \cong 12.6\, m/day$$

Theorical lowering at the well (s_0), from the Thiem's equation for artesian aquifers, with $r, s \rightarrow r_1, s_1$:

$$s_0 = \frac{Q \cdot \ln(r_1/R_0)}{2\pi K m} + s_1 = \frac{0.1 \cdot \ln(10/0.15)}{2\pi \times 0.000146 \times 40} + 10 \cong 21.5\, m$$

Expression for the radius of influence, R_i:

$$R_i = R_0 \times e^{\left(\frac{2\pi K m}{Q} s_0\right)} = 0.15 \times e^{\left(\frac{2\pi \times 0.000146 \times 40}{0.1} \times 21.5\right)} \cong 389.4\, m$$

From the expression to the radius of influence, it is possible to calculate the distance from which the lowering is less than $2\, meters$, doing $R_0, s_0 \rightarrow R, s$:

$$R_i = R \times e^{\left(\frac{2\pi K m}{Q} s\right)} \rightarrow R = \frac{R_i}{e^{\left(\frac{2\pi K m}{Q} s\right)}} = \frac{389.4}{e^{\left(\frac{2\pi \times 0.000146 \times 40}{0.1} \times 2\right)}} \cong 187.2\, m$$

Then, for a lowering lower than $2\, m$, $R > 187.2\, m$.

Note: It is possible to calculate R, without calculating R_i at first. For such procedure, the expression for the radius of influence is $R_i, s_0 \rightarrow R, s_0 - s$:

$$R = R_0 \times e^{\left[\frac{2\pi K m}{Q}(s_0 - s)\right]} = 0.15 \times e^{\left[\frac{2\pi \times 0.000146 \times 40}{0.1} \times (21.5 - 2)\right]} \cong 187.2\, m$$

11.3. Answer:

The flow rate per kilometer of coastline that flows into the sea, based in the solution of Dupuit-Ghyben-Herzberg:

$$q = \frac{1}{2} \times K \times \frac{\rho_s - \rho_w}{\rho_w} \times \frac{z^2}{x}, with\ z \approx 40h$$

$$\bar{K} = 10^{-1}\, cm/s = 86.4\, m/day$$

$$\rho_s = 1025\, kg/m^3\ (sea\ water)$$

$$\rho_w = 1000\, kg/m^3\ (fresh\ water)$$

$$q = \frac{1}{2} \times 86.4 \times \frac{1025 - 1000}{1000} \times \frac{(40 \times 10)^2}{500} = 345.6\, (m^3/day)/m \rightarrow$$

$$\rightarrow q = 345600\ (m^3/day)/km$$

PRACTICAL WORKS

Introduction

Aiming to cover some of the proposed program thematic areas and respecting the new norms of the curricular unit's functioning, the development of a general project is proposed in order to incorporate all the concepts addressed in the classes, theorical and practical aspects related to each theme.

Weekly time will be available relative to the theorical and practical classes for the execution of the practical work. Doubts can be clarified about the work of general interest of the whole class.

The development of weekly work during the theorical and practical classes will be taken into consideration in the final evaluation, being filled a weekly assessment sheet on practical work/student.

It is intended that each working group completely characterizes, according to the notes of the theorical and practical classes and bibliography to be consulted, the topics referred below (the presentation of the theme, chapters and sub-chapters must be respected):

1. Double Accumulation Method;
2. Average Annual Precipitation;
3. Intense Precipitations. DDF Curve;
4. Average Annual Flow and Duration Curve;
5. Evapotranspiration and Irrigation;
6. Infiltration;
7. Peak Flow Rate;
8. Pumping Tests.

They should be presented and structured as follows:

- Report (cover + text):
    - Cover (group identification - full name and mechanographic number);
    - Text (Index, Introduction, Answer to the points established in the objectives of the practical work, Bibliography);
    - The body of the text must be printed in the Arial font, size of 11 points. The main headings must be printed in Arial, 12 points, using uppercase. The subtitles will also be in Arial, 11 points, with the first letter capitalized. The lower-order headings will be in Arial, Italic, 11 points, with the first letter capitalized;
    - All margins must be set at 3 cm;
    - The paragraphs must be aligned with both margins (left and right). The headings must be aligned on the left;
    - Use a 12-point line spacing. Leave a blank line between two successive paragraphs and after each title. Leave two blank lines between a paragraph and the next title;
    - The text should be slightly adjusted in such a way that ensures that there are no isolated lines;

- Figures drawn in AutoCAD format;
- Spreadsheets in Microsoft Excel format;
- Deliver working folder, with identification of practical work and group number, example: "Practical Work Nr. 1 - Group I" (only with files ".doc, .xls and .dwg").

Work Nr. 1: Double Accumulation Method

I - Introduction

The breakdown of homogeneity or consistency of annual series can be detected in many cases by a double accumulated values test.

In order to verify the consistency of the series of annual precipitations in a given udometric station its marked, in a cartesian axis system, in one of the axis, the accumulated values of the annual precipitation at that station, and in the other axis the accumulated values of the sum (or the arithmetic average) of annual precipitation in a group of neighboring stations.

If the annual precipitation values series in a given station are consistent, its obtained points that are sensitively aligned according to a straight line.

If the series in a given station are inconsistent, it is generally obtained two straight segments with a break at the point corresponding to the year in which a significant change was found in the operating conditions of the station.

II - Objective

Table 39 holds the annual precipitation values at a station A and the average annual precipitation in 12 stations of the same region:

1. Verify the consistency of station A records and, in case of inconsistency, present a table with its corrected values.

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. CONSISTENCY VERIFICATION OF STATION A RECORDS
3.1. Station A double mass curve
3.2. Correction of annual precipitations for station A
4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (suggested) and appendices

Table 39 – Annual precipitation (mm) at station A and the average of precipitation.

Year	Station A	Average
41/42	710	768
42/43	518	623
43/44	527	705
44/45	563	631
45/46	549	698
46/47	523	512
47/48	668	785
48/49	750	676
49/50	554	614
50/51	470	609
51/52	549	631
52/53	829	807
53/54	489	638
54/55	422	561
55/56	374	527
56/57	484	745
57/58	297	524
58/59	383	807
59/60	489	698
60/61	414	517
61/62	462	524
62/63	506	745
63/64	489	591
64/65	348	549
65/66	384	631
66/67	371	648
67/68	393	524
68/69	523	830
69/70	314	527
70/71	492	785
71/72	357	638
72/73	506	663
73/74	532	862
74/75	317	512
75/76	323	591
76/77	395	522

Work Nr. 2: Average Annual Precipitation

I - Introduction

Knowledge of punctual precipitations alone has little interest. It is generally important to know the weighted precipitation (weighted average) over a given zone (a hydrographic basin for example, or an irrigation perimeter).

There are 2 main methods to obtain the weighted precipitation on a given zone from precipitation records of udometric stations:

- Thiessen's method or areas of influence;
- Isolines method.

II - Objective

1. Determine the average annual precipitation on a hydrographic basin (plant provided by the professor) using the following methods:

1.1. Thiessen's method or areas of influence;

1.2. Isolines method (Table 41).

Table 40 presents the values of the average annual precipitation at the udometric stations indicated and located on the hydrographic basin (plant provided by the professor) or in its vicinity.

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. ANNUAL AVERAGE PRECIPITATION

3.1. Thiessen's method or areas of influence

3.2. Isolines method

4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (suggested) and appendices

Table 40 – Annual average precipitation for each station.

Station	Precipitation (mm)
A	831
B	976
C	1172
D	864
E	927
F	1053
G	1225
H	1119

Table 41 – Isolines.

Precipitation (mm)
800-850
850-900
900-950
950-1000
1000-1050
1050-1100
1100-1150
1150-1200
1200-1250

Work Nr. 3: Intense Precipitations. DDF Curve

I - Introduction

Intense precipitations play a key role in determining a flood peak flow rate, which is essential in the structural sizing of hydraulic works such as flood defence works. The study of the characteristics of intense precipitations comprises:

- Statistical analysis of precipitations for a given duration;
- The establishment of ratios between precipitation and its duration (in which the probability of occurrence is considered as a parameter), better known as DDF curves.

II - Objective

1. Determine, for the hydrographic basin studied in Work Nr. 2, the maximum daily precipitation corresponding to return periods of 20 and $100\ years$, based on the average and standard deviation of the maximum annual daily precipitation at the nearest udometric station to the centre of the basin (Table 42).
2. Using the quotients between the maximum short duration precipitation and the maximum daily precipitation provided by the professor (Table 43), present a graph on a logarithmic scale relating the maximum precipitation with its duration, for the return periods of 20 and $100\ years$, and determine the parameters of the corresponding DDF curves, for durations between 1 and $6\ h$.
3. Present in the same graph the lines corresponding to the IDF curves proposed by R. Matos and M. Silva, LNEC, 1986, for the same return periods (Figure 60).

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. BASE DATA

3.1. Calculation of maximum daily precipitation for return period of T=100 years and T=20 years

3.2. Estimate of short duration intense precipitation

4. PARAMETERS OF THE DDF CURVES FOR DURATIONS BETWEEN 1 AND 6 HOURS
5. CONCLUSION
6. BIBLIOGRAPHIC REFERENCES
7. APPENDICES

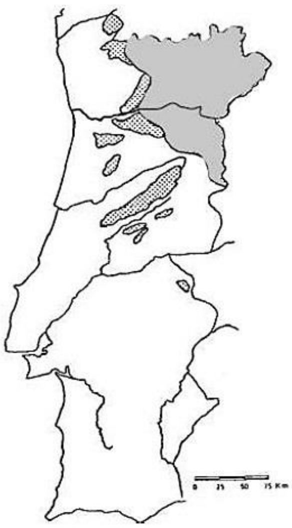
IV - Bibliographic references (recommendations) and appendices

Table 42 – Annual maximum daily precipitation.

Date	Annual maximum daily precipitation (mm)
15-10-1979 09:00	80.4
07-11-1980 09:00	50.2
26-12-1981 09:00	35.0
21-04-1983 09:00	38.9
16-12-1983 09:00	32.0
10-02-1985 09:00	32.1
14-02-1986 09:00	46.1
06-01-1987 09:00	40.0
15-10-1987 09:00	50.0
24-05-1989 09:00	40.2
23-10-1989 09:00	33.1
07-03-1991 09:00	38.1
09-01-1992 09:00	42.1
05-12-1992 09:00	35.1
29-11-1993 09:00	29.8
05-11-1994 09:00	26.3
26-12-1995 09:00	43.5
27-12-1997 09:00	30.5
18-09-1999 09:00	20.1
10-12-1999 09:00	31.0
31-12-2000 09:00	41.2
17-09-2002 09:00	36.5
02-12-2005 09:00	61.2
31-01-2009 09:00	24.0

Table 43 – Quotients between short duration maximum precipitations and maximum daily precipitation.

Duration (n), in minutes	Quotient (P_n/P_{24})
360	0.7700
60	0.4300
30	0.3397
15	0.2451
10	0.1935
5	0.1247



Return Period, T (year)	Pluviometric Region A IDF Curve Lisbon a'	n'	B IDF Curve Lisbon - 20% a'	n'	C IDF Curve Lisbon + 20% a'	n'
2	202,72	-0,577	162,18	-0,577	243,26	-0,577
5	259,26	-0,562	207,41	-0,562	311,11	-0,562
10	290,68	-0,549	232,21	-0,549	348,82	-0,549
20	317,74	-0,538	254,19	-0,538	381,29	-0,538
50	349,54	-0,524	279,63	-0,524	419,45	-0,524
100	365,62	-0,508	292,5	-0,508	438,75	-0,508

Figure 60 – Pluviometric regions and parameters of the IDF curves (adapted from R. Matos and M. Silva, 1986).

Work Nr. 4: Average Annual Flow and Duration Curve

I - Introduction

A flow is the amount of water that crosses a section of a watercourse in a given time interval. Being expressed in volume (m^3) or in water height (mm) uniformly distributed over the area, in plant, of the corresponding hydrographic basin.

The volume of flow in a given time interval is obtained by integrating the flow rates values observed over such time interval. Being more intuitive, it is current to use, instead of the flow in a given time interval, the corresponding average flow rate: fictional flow rate, uniform, that in the same time interval produces a flow volume equal to the actual succession of the flow rate.

In a section of a river, for each hydrological year, the respective average daily flow rate curve is defined and, for an interval of several years, the average annual duration of the average daily flow rate.

By “duration of a given average daily flow rate in a hydrological year”, is meant as the number of days in which the flow rate was equalled or exceeded and by “average annual duration of a given average daily flow rate, in an interval of several years”, is meant the average number of days per year in which this flow was equalled or exceeded (total number of days in the time interval considered divided by the number of years).

A duration curve is obtained by marking the flow rate values at y-axis, and at x-axis the number of days of the year in which these values were equalled or exceeded.

II - Objective

1. Determine the annual average flow and the module in the hydrographic basin from Work Nr. 2, based in:
1.1. Isolines chart of the annual average flow (Figure 61);
1.2. Annual precipitation at the closest meteorological station to the centre of the basin (Table 44);
2. Determine the annual average duration curve of the average daily flow rate for the flow value obtained in 1.2., and determine the median flow rate.

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. LOCATION OF THE STUDIED HYDROGRAPHIC BASIN
4. ANNUAL AVERAGE FLOW AND MODULE IN THE HYDROGRAPHIC BASIN
4.1. Isolines chart of the annual average flow in Portugal
4.2. Annual precipitation at the closest meteorological station to the centre of the basin and Quintela’s regional relations

5. ANNUAL AVERAGE DURATION CURVE OF THE DAILY AVERAGE FLOW RATE AND MEDIAN FLOW RATE
6. CONCLUSION
7. BIBLIOGRAPHIC REFERENCES
8. APPENDICES

IV - Bibliographic references (recommendations) and appendices

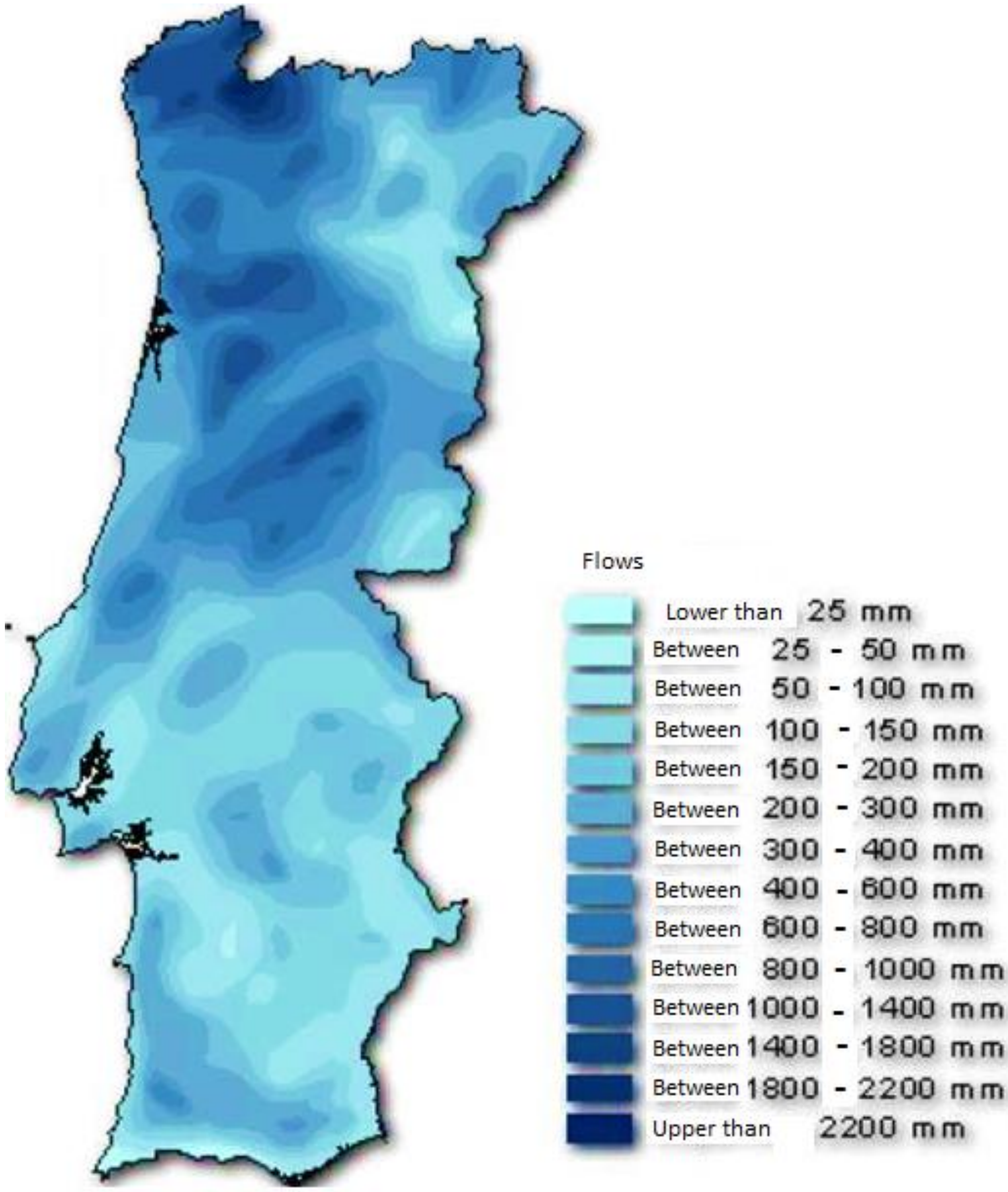


Figure 61 – Isolines chart of annual average flow.

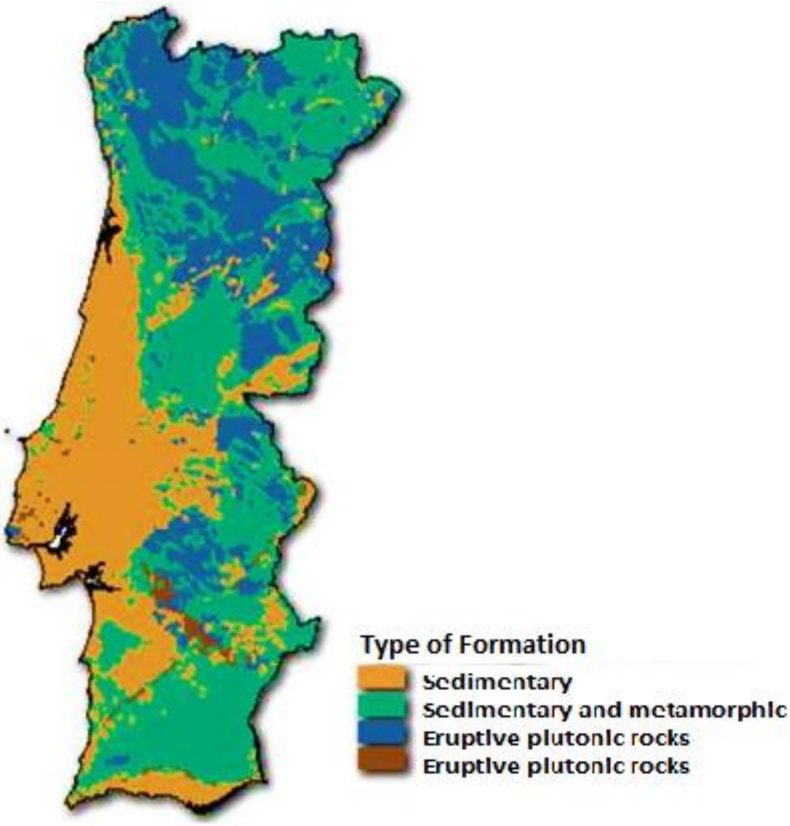


Figure 62 – Soils chart.

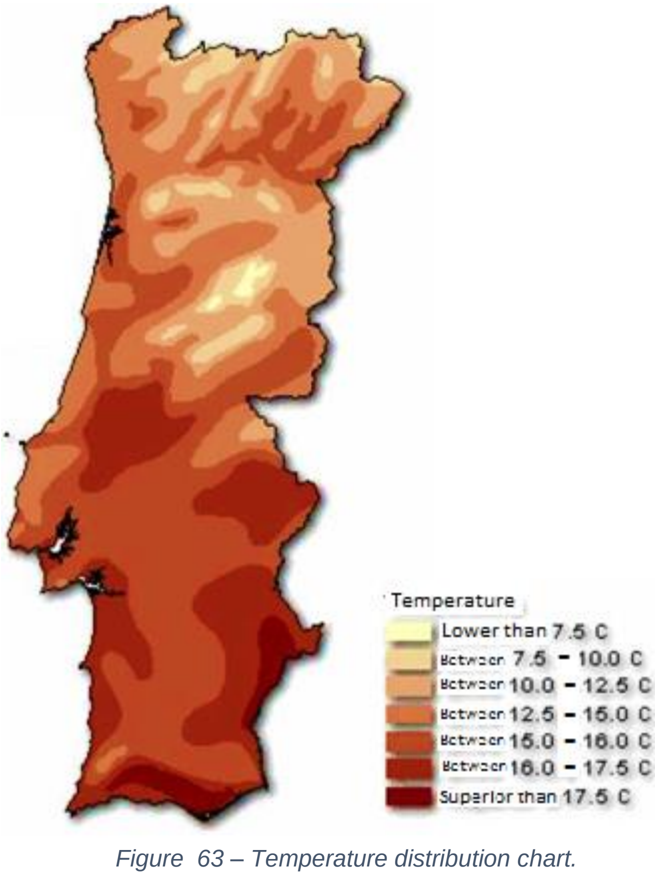


Figure 63 – Temperature distribution chart.

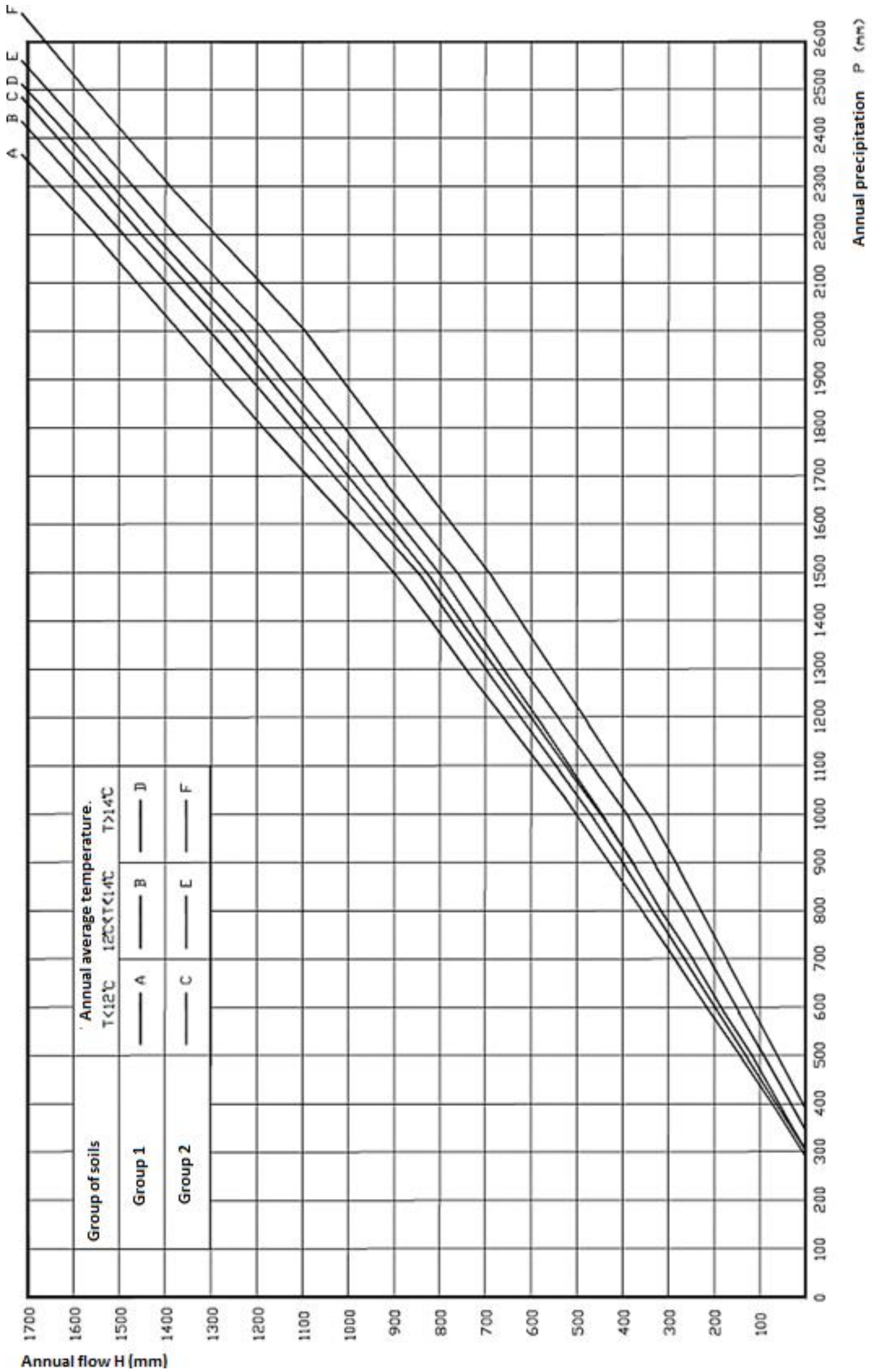


Figure 64 – Regional curves.

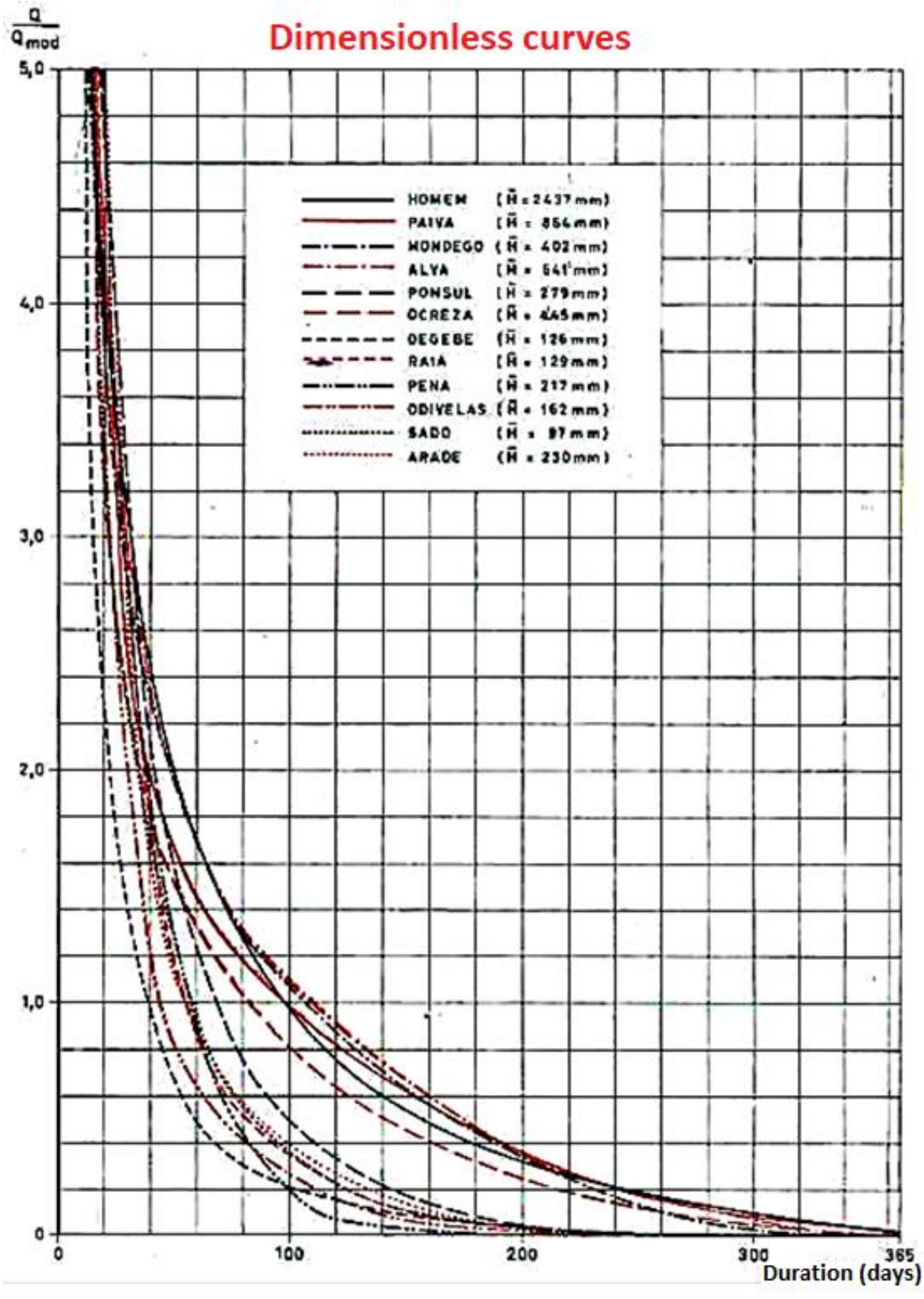


Figure 65 – Annual average duration curves of flows for Portuguese rivers.

Table 44 – Annual average precipitation in the nearest meteorological station to centre of the basin.

Date	Annual precipitation (mm)
01/10/1979 09:00	954.0
01/10/1980 09:00	739.3
01/10/1981 09:00	796.1
01/10/1982 09:00	778.3
01/10/1983 09:00	1163.1
01/10/1984 09:00	1178.4
01/10/1985 09:00	1020.1
01/10/1986 09:00	955.4
01/10/1987 09:00	1277.7
01/10/1988 09:00	691.8
01/10/1989 09:00	887.6
01/10/1990 09:00	973.1
01/10/1991 09:00	625.6
01/10/1992 09:00	898.4
01/10/1993 09:00	804.6
01/10/1994 09:00	553.3
01/10/1995 09:00	1312.3
01/10/1996 09:00	1093.3
01/10/1997 09:00	1576.9
01/10/1998 09:00	376.5
01/10/1999 09:00	1049.5
01/10/2000 09:00	1828.1
01/10/2001 09:00	601.4
01/10/2005 09:00	862.8
01/10/2008 09:00	722.2

Work Nr. 5: Evapotranspiration and Irrigation

I - Introduction

Evapotranspiration is the aggregation of evaporation and transpiration processes. It includes transpiration of plants and evaporation of water from surfaces and moist soils, vegetation and other obstacles that intercept water.

The passage of water from liquid to gas state is made, in hydrological phenomena, with energy consumption of solar origin.

Irrigation is, at the planet level and in most continents and countries, the main consumer of the water resource. According to Gleick (2007), the abstractions of water for irrigation correspond to about 70% of the total abstractions and in Asia it corresponds to about 20% of the average annual flow. Due to the massive impact that this particular water use exerts on hydric resources, the adequate study of irrigation is essential.

This study comprises the principles of hydrology from the following subjects: hydrologic cycle, hydrologic balance, precipitation, evaporation and evapotranspiration.

II - Objective

A farm located at latitude $42°N$ has the characteristics indicated in Table 45. Table 46 presents the monthly precipitation values and the average monthly temperature in the exploration area for a given year:

1. Determine the monthly evapotranspiration of such year, calculated using the formulas of Thornthwaite and Blaney-Criddle.
2. Calculate the monthly volume of water needed for irrigation, in despite of the storage of water in such soil and adopting the monthly evapotranspiration given by the Blaney-Criddle formula.

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. EVAPOTRANSPIRATION AND IRRIGATION

3.1. Determination of monthly evapotranspiration of a given year using Thornthwaite's formula

3.2. Determination of monthly evapotranspiration of a given year using Blaney-Criddle's formula

4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (recommendations) and appendices

Table 45 – Farm at latitude 42ºN.

Crop	Area (ha)	Months	Crop Coef.
Lucerne	20	Feb. - Apr.	0.70
		May - Sept.	0.85
		Oct. - Nov.	0.70
Soy	15	June - Aug.	0.70
Barley	30	Apr. - June	0.75

Table 46 – Monthly average temperature and precipitation.

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
$\boldsymbol{P}$ (mm)	150	100	120	80	70	40	30	10	50	100	130	200
$\boldsymbol{T}$ (°C)	8	10	12	13	15	18	20	23	18	15	12	10

Table 47 – Daily average astronomic insolation (h) (Hipólito & Vaz, 2011).

Lat	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
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

Work Nr. 6: Infiltration

I - Introduction

The admission of water into a soil by its surface constitutes a phenomenon that is generically referred to as infiltration.

The infiltration is by far the most significant process of precipitation losses for a flow:
- Transformation of total precipitation in effective precipitation responsible for the superficial flow (by deducting the losses of precipitation for the flow due to infiltration);
- Establishment of irrigation allocation in order to find the amount of water that could be stored in the soil (at roots depth);
- Analysis of aquifer recharge.

II - Objective

Table 48 presents the values of an experience done with a ring infiltrometer with a diameter of $35\ cm$:
1. Present the graph of infiltration capacity as a function of time.
2. Determine the parameters of the Horton formula and illustrate it in the previous graph.

III - Structure

1. INTRODUCTION
2. GENERAL CONSDERATIONS
3. INFILTRATION
3.1. Capacity of infiltration as a function of time
3.2. Horton's formula
4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (recommendations) and appendices

Table 48 – Experience results.

Total elapsed time (mm)	Total volume added (cm^2)
2	278
5	658
10	1173
20	1924
30	2500
60	3345
90	3879
150	4595

Work Nr. 7: Peak Flow Rate

I - Introduction

From a hydrological point of view, a given flood is known from its hydrograph, where the ascending segment is distinguished, the maximum or peak flow rate and the descending segment. However, for various applications such as the structural sizing of bridges flow sections, aqueducts or rainwater collectors, it is sufficient just to know the peak flow rate.

II - Objective

1. Estimate the time of concentration of the hydrographic basin studied in the Work Nr. 2, using Giandotti's formula;
2. Determine the average intensities of maximum precipitation with duration equal to the time of concentration, for the return periods of 20 and $100\ years$, based on the DDF curves presented in Work Nr. 3;
3. Calculate the peak flow rate for the return periods of 20 and $100\ years$, using the rational formula.

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. PEAK FLOW RATE
3.1. Time of concentration of the hydrographic basin
3.2. Determination of the average intensities of maximum precipitation
3.3. Peak flow rate for the return periods of 20 and 100 years
4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (recommendations) and appendices

Table 49 – Average values of the flow coefficient for urban areas (adapted from Chow, 1964).

Urban areas	
Ground Cover	C
Green areas:	
Lawns in sandy soils	0.05 - 0.20
Lawns in heavy soils	0.15 - 0.35
Parks and cemeteries	0.10 - 0.35
Sports camps	0.20 - 0.35
Commercial areas:	
City centre	0.70 - 0.95
Suburb	0.50 - 0.70
Residential areas:	
Villas in the city centre	0.30 - 0.50
Villas in suburb	0.25 - 0.40
Apartment	0.50 - 0.70
Industrial areas:	
Disperse	0.50 - 0.80
Concentrated	0.60 - 0.90
Railways	0.20 - 0.40
Roads and streets:	
Asphalt	0.70 - 0.90
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Sidewalks	0.85 - 0.85
Roofs	0.75 - 0.95
Vacant lot	0.10 - 0.30

Table 50 – Annual values of the flow coefficient for rural areas (adapted from Chow, 1964).

Rural areas			
Types of soils	C		
	Coverage of the basin		
	Crops	**Grasslands**	**Forests**
Infiltration capacity above average; usually sandy.	0.20	0.15	0.10
Average infiltration capacity; without layers of clay; light soils or similar.	0.40	0.35	0.30
Infiltration capacity below average; heavy clay soil or with a clayey layer close to the surface; slender soils on waterproof rock.	0.50	0.45	0.40

The coefficient C does not correspond necessarialy to a “flow coefficient” (relation between volumes of flow and the precipitation that cause it). The objective is to demonstrate the effects in the peak flow rate, due to superficial retention, infiltration and storage in waterbeds. So, it depends on the hydrologic type of soil, its occupation and return period.

The values of a flow coefficient, previously presented, correspond to a return period between 5 and 10 years; for precipitations of not so frequent intensity, it will be necessary to correct the coefficient by using an adjustment coefficient, C_f, (Wright-McLaughlin, 1969). To refer that the product of the flow coefficient by adjustment coefficient cannot exceed a unit. So, the rational formula is defined as:

$$Q_p = C \cdot C_f \cdot I \cdot A_b$$

Where C_f is the adjustment coefficient defined as follows:

Table 51 – Adjustment coefficient as function of the return period (Wright-McLaughlin, 1969).

Return period, $T\ (years)$	Adjustment coefficient, C_f
25	1.10
50	1.20
100	1.25

Work Nr. 8: Pumping Tests

I - Introduction

The pumping tests are made in order to support the determination of hydraulic parameters of an aquifer, the definition of the exploitation regime and the procedure of testing the state of a catchment. The test is developed in three phases:

- Start the pumping in the hole, for a given time, with a given flow;
- Lowerings are measured either in the extraction hole itself or in piezometers located nearby;
- The hydrodynamic parameters are determined from the pumped flow rate, the measured lowerings and the respective distances or time using appropriate formulas.

II - Objective

1. It was performed a pumping test at constant flow rate of $10\ L/s$, in a confined aquifer, with $12\ m$ of thickness, during $14\ hours$, in which at the end of the test the levels where stabilized. The lowerings were measured in 5 piezometers and the distances to the pumping hole are presented in Table 52. Calculate by Thiem's method, the transmissivity of the aquifer (T), the hydraulic conductivity (K) and the radius of influence (R_i);
2. A pumping test was performed at constant flow rate of $150\ L/s$, in a sandy and confined aquifer. The lowerings were measured in a piezometer, located at $100\ m$ from the pumping hole, and are presented by Table 53. Calculate by Jacob's method, the transmissivity of the aquifer (T) and the coefficient of storage (S).

III - Structure

1. INTRODUCTION
2. GENERAL CONSIDERATIONS
3. INTREPERTATION OF THE PUMPING TEST
3.1 Thiem's method
3.2 Jacob's method
4. CONCLUSION
5. BIBLIOGRAPHIC REFERENCES
6. APPENDICES

IV - Bibliographic references (recommendations) and appendices

Table 52 – Pumping test 1.

Piezometer	Distance to the pumping hole, $r\ (m)$	Lowerings, $s\ (m)$
P1	10	3.10
P2	30	2.35
P3	60	1.70
P4	120	1.32
P5	400	0.59

Table 53 – Pumping test 2.

$t\ (min)$	$s\ (m)$
1	-
5	-
10	-
20	-
50	0.20
60	0.40
70	0.60
80	0.85
90	1.00
100	1.40
120	1.95
150	2.80
200	4.20
300	6.60
400	8.85
600	11.80
800	14.00
1200	17.90
2000	22.60
3000	26.50
4000	29.00

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