
**Practical task n.º 1 Flood analysis. Statistical analysis. Separation of the components
of the flood hydrograph and of the rainfall hyetograph.****I) Estimation of peak flood discharges**

Consider a watershed (**case study**) with an area of 40 km² located relatively close to the river gauge station of Castro Daire (R.E.) (08J/01H) (Paiva River, Douro watershed). The case study watershed is totally comprehended in the influence area of the rain gauge of Sernancelhe (08M/02) (according to the Thiessen method).

- a) Check for stationarity of the sample of maximum annual instantaneous discharge at Castro Daire (R.E.) by applying the moving average technique with a running length of $n=20$. A sample is said to be stationary if the natural prevailing processes and the sampling errors do not change over time (homogeneous and consistent sample). The climate change is a lack of homogeneity (by means of trends) and, therefore, of stationarity.
- b) Perform the statistical analysis of the previous sample considering the Normal, Gumbel and Pearson III laws and estimate the 100-years maximum annual peak instantaneous discharge (return period, $T = 100$ years) according to the law with best adjustment.
- c) By combining the previous estimate with a regionalization model, compute the 100-year peak flood discharge at the case study.

II) Baseflow recession model

Table 1¹ presents the discharges at the case study measured during three flood events. By applying the Horton's model to the previous flood hydrographs estimate the recession constant applicable to the posterior recession in the watershed.

II) Separation between the direct runoff and the baseflow. Total and excess rainfall. Rainfall abstractions

Table 2¹ presents the hyetograph registered at the rain gauge of Sernancelhe (08M/02) during the first flood of Table 1 (November 1996). Based on such flood compute the excess or net rainfall hyetograph and separate the direct runoff from the base flow. For that purpose consider that:

- (1) the recession constant is applicable until the instant of occurrence of the peak flood discharge;
- (2) the time of concentration of the watershed, t_c , is 7.5 h;
- (3) the initial loss is equal to the total amount of rainfall until the moment from which the discharges of the observed flood hydrograph increase;
- (4) the variation of the baseflow between the moment when the peak flood discharge occurs and the end of the direct runoff is linear. To identify the moment of the end of the direct runoff, you should assume a value for the constant loss, ϕ , and make it vary until the volume of the effective runoff is equal to the volume of the direct runoff.

¹ Available in the webpage of the course.

All the computations must be organized in table. Figures representing the different rainfall hyetographs (total, excess and losses) as well as the different flood hydrographs (for total and direct runoff and for baseflow, superimposed and separated) are also mandatory.

Practical task n.º 2 Unit hydrograph model

Under the conditions of the Practical task n.º1, apply the least square method to compute the unit hydrograph for the duration of 0.5 h, HUD for $D=0.5$ h, and by doing the discrete convolution of the effective rainfall hyetograph and of the previous unit hydrograph, compare the direct runoff flood hydrograph thus obtained with the one used as data.

Present the results in the form of tables and figures. The comparison should be based on the correlation coefficient and on another error measure chosen and justified by you (Mean absolute error (MAE), Root mean squared error (RMSE), Nash–Sutcliffe model efficiency coefficient (NSE), Pearson coefficient, etc.).

Practical task n.º 3 Design rainfalls and hyetographs at the watershed of the case study

Taking into account the proximity between the watershed of the case study and the rain gauge of Sernancelhe (08M/02), select, from the set of 27 rain gauges with IDF curves schematically located in the next figure, the one in which area of influence the watershed of the case study is located.



Based on the that rain gauge and on the series of annual maximum daily rainfall, Pamd, at the rain gauge of Sernancelhe (08M/02) establish the design hyetographs applicable to the watershed of the case study for the durations of the rainfall equal and twice the time of concentration, t_c (7.5 and 15.0 h). For each duration you should present two hyetographs, one uniform and another built upon 3 and 6 alternating blocks, for t_c and twice t_c , respectively.

Prior to the statistical analysis of the Pamd sample at Sernancelhe you should check the sample for outliers (“valores espúrios/atípicos”) by applying the test of page 287 of Naghettini & Pinto, 2007, corrected according to the corresponding erratum (both available in the web page of the course and reproduced in the next pages).

Page 287 of Naghettini & Pinto, 2007

7.5 – Teste para Detecção e Identificação de Pontos Atípicos (*outliers*)

Em uma certa amostra de observações, um elemento ou ponto amostral é considerado atípico, ou um *outlier*, do ponto de vista estatístico, quando ele se desvia *significativamente* do conjunto dos outros pontos. Esse desvio pode ter origem em erros de medição ou de processamento, mas também pode ser o produto de causas naturais indeterminadas. Em qualquer caso, a presença de pontos atípicos em uma dada amostra, pode afetar drasticamente o ajuste de uma certa distribuição de probabilidades àqueles dados. No item 2.1.4 do capítulo 2, foi descrito um procedimento de identificação de pontos atípicos, por meio dos quartis amostrais e da amplitude inter-quartis. Este procedimento, embora bastante útil, é meramente exploratório e não constitui, do ponto de vista estatístico, um teste de hipótese, com um nível de significância previamente estabelecido.

Entre os diversos testes de hipóteses para detecção e identificação de pontos atípicos, o teste de Grubbs e Beck, descrito por Grubbs (1950, 1969) e estendido por Grubbs e Beck (1972), encontra-se entre os mais frequentemente empregados. De acordo com esse teste, as quantidades x_s e x_l definem, respectivamente, os limites superior e inferior, acima e abaixo dos quais, os pontos atípicos, eventualmente presentes em uma amostra, são detectados e identificados. Essas quantidades são definidas pelas seguintes expressões:

$$x_s = \exp(\bar{x} + k_{N,\alpha} s_x) \quad (7.34)$$

e

$$x_l = \exp(\bar{x} - k_{N,\alpha} s_x) \quad (7.35)$$

onde \bar{x} e s_x representam, respectivamente, a média aritmética e o desvio-padrão de uma amostra de tamanho N , de uma variável aleatória X , e $k_{N,\alpha}$ denota o valor crítico da estatística de Grubbs e Beck, para um nível de significância α . Para $100\alpha = 10\%$, Pilon et al. (1985) propõem a seguinte aproximação para o valor crítico da estatística de Grubbs e Beck:

$$k_{N,\alpha=0,10} = -3,62201 + 6,28446 N^{1/4} - 2,49835 N^{1/2} + 0,491436 N^{3/4} - 0,037911 N \quad (7.36)$$

De acordo com o teste de Grubbs e Beck, a um nível $\alpha = 0,10$ e $k_{N,0,10}$ dado pela equação 7.36, as observações eventualmente superiores a x_s , e/ou inferiores a x_l , estariam se desviando significativamente do conjunto dos dados e deveriam ser consideradas como *outliers*.

Erratum

- 30) Página 287, corrigir equação 7.34 para $x_s = \exp(\bar{y} + k_{N,\alpha} s_Y)$
- 31) Página 287, corrigir equação 7.35 para $x_I = \exp(\bar{y} - k_{N,\alpha} s_Y)$
- 32) Página 287, 23ª linha: corrigir para “onde \bar{y} e s_Y representam ...”
- 33) Página 287, 24ª linha: corrigir para “... N , dos logaritmos neperianos da variável aleatória X , e ...”

Practical task n.º 4 Application of the program HEC-HMS. Simulation mode and optimization mode

In a given watershed, the HEC-HMS program can be applied in two different modes: 1) the **simulation mode**, aiming at computing design hydrographs based on design hyetographs, and 2) the **optimization mode** having in view the calibration of parameters of the models by minimizing the errors between observed values and simulated ones through an objective function. The optimization mode always requires an initial solution given by a simulation run. The parameters are calibrated will be used to simulate design conditions.

1) Parameters calibration

Consider the **total precipitation, the initial losses the and direct runoff discharges** of the flood of November 1996 at the watershed of the case study (1st Practical task). Based on those results and assuming that the impervious areas are negligible, apply the HEC-HMS program to optimize:

- (i) the lag time, t_{lag} , of the Soil Conservation Service, SCS, synthetic unit hydrograph model, and the rainfall constant loss rate;
- (i) the lag time, t_{lag} , of the Soil Conservation Service, SCS, synthetic unit hydrograph, and the curve number, CN, of the rainfall loss model also of the SCS;

2) Computation of design flood hydrographs

Apply the HEC-HMS program to obtain the design flood hydrographs for the four design hyetographs defined in the scope of the 3rd Practical task. Adopt the values previously calibrated for the rainfall constant loss rate and for lag time of the SCS synthetic unit hydrograph. You should compare and comment on the flood hydrographs you obtained.

Practical task n.º 5 Application of the program HEC-HMS to flood routing in artificial reservoirs

At the outlet section of the watershed of the case study a dam will be built aiming at flood control. The dam has an uncontrolled (without gates) broad-crested spillway (*descarregador em soleira espessa horizontal*). The length of the crest of the spillway is 10 m and it is located at the elevation 95 (full water supply level, FSL – *nível de pleno armazenamento, NPA*). The elevation-storage curve above the FSL is presented in the table. The discharge rating curve of the spillway is given by

$$Q = C L \sqrt{2 g} H^{1.5}$$

where Q is the outflow (m³/s); C the discharge coefficient (with a constant value of 0.45); L the length of the spillway crest (10 m); g the gravitational acceleration; and H the head over the spillway crest.

By applying the HEC-HMS program analyzed the “damping” in the reservoir of the four design flood hydrographs that you obtain in the 4th Practical task. For each design hydrograph you should include in the report the schematic representation of the inflow and outflow hydrographs.

Comment on the results, namely in what concerns the comparison between t_c and twice t_c and the effect of considering the temporal variability of the design hyetograph.

Analyze the effect of increasing or decreasing the length of the spillway crest in: a) the “damping” of the peak inflow discharges (that is, in the peak outflow discharges); b) the differences in the maximum flood levels; and c) the resulting cost (related to the dam and to the spillway channel).

Table – Volume stored above the FSL

Elevation	Volume stored above the FSL (hm ³)
95	0
96	0.1
97	0.2
98	0.5
99	0.9
100	1.3
101	1.8
102	2.5
103	3.3
104	4.1
105	5.2

**Practical task n.º 6 Application of the program HEC-RAS to the computation of the
river surface profile under gradually steady flow conditions**

A given river reach is characterized by the eleven cross sections given in the Excel file.

Section 1 is the most downstream section and section 11, the most upstream one. Some of the areas of the cross sections are only water accumulation areas (ineffective), is described in the file.

In the river reach there are two bridges located between cross sections 3 and 4 (bridge A) and between cross sections 9 and 11 (bridge B). The information regarding the bridges – i.e., the boundary cross sections, the distance to the upstream cross sections, the widths, the high and the low chords and the stations defining the beginning (LOB) and the end (ROB) of the bridge roadways – are also provided in the table.

The Manning-Strickler coefficient is $30 \text{ m}^{1/3} \text{ s}^{-1}$. The downstream boundary condition is normal depth (*escoamento uniforme*, in Portuguese) for an average slope of the river bed of 6.5 m/km.

- a) Explain why the establishment of a boundary condition of normal depth requires the knowledge of the average slope of the river bed.
- a) Under the previous conditions compute the maximum discharge along the river reach that ensures a free gap between the water surface elevation and the low chord of any of the bridges of at least 20 cm.
- b) Confirm that the flow conditions do not change significantly if the boundary condition at the downstream cross section was critical depth. What could cause a critical depth boundary condition at the most downstream section of the river reach?

Practical task n.º 7 Computation of the energy production in a small hydropower scheme with run-of-river exploitation

A small hydropower scheme, SHP, with run-of-river exploitation will be constructed at the outlet section of the watershed of the case study.

The gross topographic head of the SHP will be $\Delta H = 180$ m. The net heads to consider in the computation of the mean annual energy production (ΔH_{ene} , eqn. 1) and of the installed capacity (ΔH_{pot} , eqn. 2) can be considered to be approx. 97.5 and 95% of the gross topographic head, respectively. The average efficiency of the powerhouse, η , is 88%.

$$E = \frac{V \Delta H_{ene}}{\frac{3600}{9.8 \eta}} \quad (\text{eqn. 1})$$

$$P = 9.8 \eta \Delta H_{pot} Q_{max} \quad (\text{eqn. 2})$$

In the previous equations E refers to the mean annual energy production (kWh), V to the mean annual turbined volume (m^3), P (kW) to the installed capacity, Q_{max} to the design discharge and the net heads ΔH_{ene} , ΔH_{pot} are expressed in meters.

The daily discharges, Q (m^3/s) at the water intake of the SHP in the 21-year period, from 1998/99 to 2018/2019 (hydrological years starting 1st October) are given in an Excel file (the discharges at the 29th of February of the leap years were discarded).

For the next three possible design discharges, Q_{max} , compute the mean annual energy productions of the SHP, based on its daily exploitation: $Q_{max} = 1.6 Q_{mod}$; $Q_{max} = 1.8 Q_{mod}$; $Q_{max} = 2.0 Q_{mod}$. Q_{mod} stands for the long term mean daily discharge (or modulus) at the water intake of the SHP.

The ecological discharge, Q_{eco} , will be 5% of Q_{mod} ($Q_{eco} = 0.05 Q_{mod}$).

Practical task n.º 8

Economic analysis criteria. Application to the design of a small hydropower scheme.

The layout of the small at small hydropower scheme (SHP) that will be implemented at the outlet section of the watershed of the case study includes a small weir, a canal with L_{can} (m) long, a surge tank or forebay that promotes the transition from the free surface flow in the canal to the pressurized flow in the penstock, this penstock with L_{pen} (m) long and finally the powerhouse where a Pelton turbine is installed – see next figure.

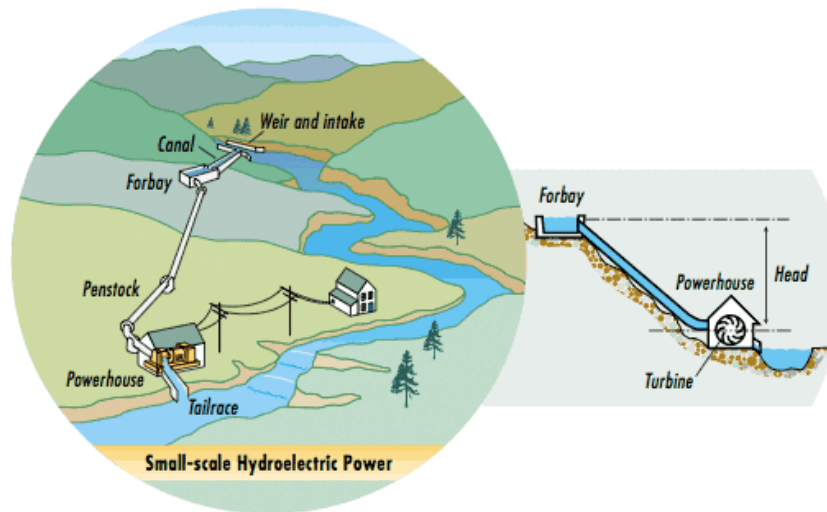


Figure – Schematic layout of the SHP.

As stated in the Practical Task n.º 7, the gross topographic head of the SHP will be $\Delta H = 180$ m and the net heads to compute the energy production (ΔH_{ene}) and the installed capacity (ΔH_{pot}) approx. 97.5 and 95% of the gross topographic head, respectively. The average efficiency of the powerhouse, η , is 88%.

In the present practical task you should accomplish the economic analysis of the SHP aiming at to select its design discharge out of the three hypothesis that were considered in the previous practical task. The selection should be based on the comparison of the relative values of the economic indicators that characterize each solution (net present value, NPV, benefit/cost ratio, B/C, payback period and internal rate of return, IRT).

Regarding the costs of the different components of the SHP that need to be consider in the economic analysis, some of them are almost independent from the design discharge, Q_{max} (e.g. the weir, the forebay, the roads, etc.) while others are closely related to the design discharge (e.g. the canal, the penstock, the powerhouse, etc.). The mean annual energy production and the corresponding income also depend on the design discharge.

The criteria for the costs that depend on Q_{max} are next presented. Regarding the remaining costs and the incomes, their values or the criteria required to compute them are specified in Table 3, included in the Excel file.

- Open air powerhouse, including the electrical substation:

$$ct = K P^{0.7} H_p^{-0.35}$$

where ct (€) is the total cost, K is a coefficient, P (MW) the installed capacity and ΔH_{pot} (m) the net head for computing P . From the previous cost, 75% relates to the mechanical, electromechanical and electrical equipment and 25% to the civil construction. The installed capacity, P (MW), is given by:

$$P = (9.8 \eta Q_{max} \Delta H_{pot}) / 1000$$

where the meanings of the variables were previously presented.

- Unit cost of the penstock: consider the cost resulting from Table 1, included in the Excel file.
- Unit cost of the canal (civil construction): interpolate between costs of Table 2 (also provided in the excel file), according to the value of Q_{max} .

The costs and the criteria presented assumed a constant market price system referred to the first semester of 2020. The costs that need to be computed in order to accomplish the economic analysis as well as their temporal occurrence are highlighted in green in Table 3 (in the Excel file) which was already organized in order to allow developing the analysis. In year 25 it is necessary to allocate costs to replace half of the equipment, as it is also highlighted in the table.

The comparison of the different values of Q_{max} should be made based on economic indicators assuming, when relevant, three discount rates 3, 5 and 7%. Consider that the implementation of the scheme will take place during years -2 and -1 followed by 35 years of exploitation (from year 1 to year 35). The reference instant for the economic analysis purposes is the beginning of year 1 (start of the scheme exploitation).

Practical Task n.º 9 Design of artificial reservoirs. Deterministic approach

Consider that at the outlet section of the watershed of the case study a dam will be built to create a reservoir to supply **x% of mean annual inflow volume** of the river at that section. The daily discharges, Q (m^3/s) during a period of 21 years at the same section are those provided in the scope of the Practical task n.º 7.

The purpose of the present practical task is to design the storage capacity of the reservoir that ensures a given demand with different temporal patterns. Neglect the water losses from the reservoir due to evaporation and infiltration.

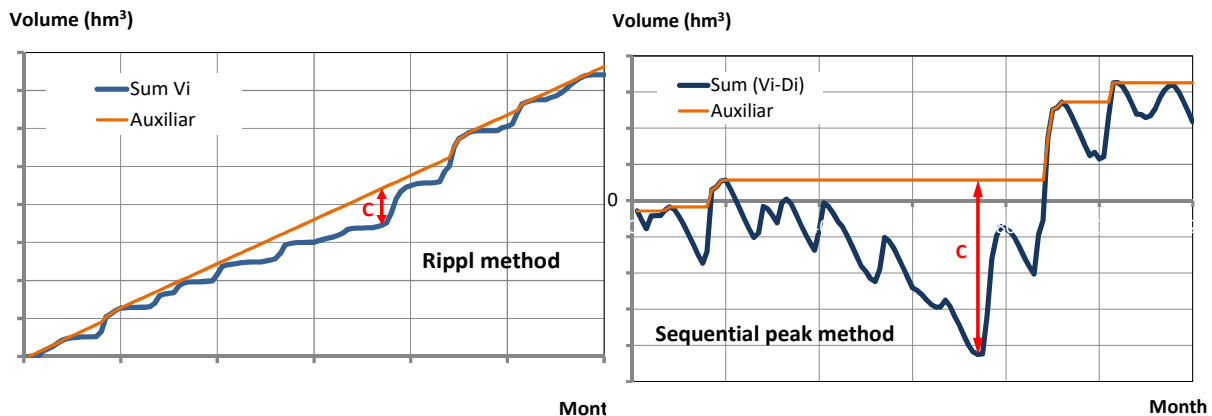
The hypothesis to analyze concerning the demand and the models to apply in the design of the reservoir are as follows:

- Hypothesis 1** Uniform demand – x% of the mean annual inflow volume uniformly distributed by the 12 months of the year. Design based on the Sequential peak and Rippl methods.
- Hypothesis 2** Variable demand mimicking/reproducing the natural variability of the flow regime – in each of the 12 months x% of the mean inflow volume in that month. Design based on the Sequential peak method.
- Hypothesis 3** Demand distributed uniformly during the irrigation season, from March to August – x% of the mean inflow volume uniformly distributed by the 6 months of this period. Design based on the Sequential peak method.

By adopting the demand of **Hypothesis 1** as well the corresponding storage capacity, apply the simulation model to compute the reliability of the reservoir considering that the reservoir is initially at its full capacity (**Hypothesis 4** with $V_0=C$) or at its half capacity (**Hypothesis 5** with $V_0=C/2$).

For each hypothesis, you should present a table with the results, at least, for the first and last year. Additionally you should present the complete results from **Hypothesis 1** in the form of the graphs exemplified in the next figure. You should also present a table summarizing the main results from the different hypothesis.

Comment on the effect (i) of the temporal pattern of the demand on the value of the storage capacity (results from **Hypothesis 1** to **3**) and (ii) of the initial stage on the performance of the reservoir (results from **Hypothesis 4** and **5**).



Note:

The work should be systematized according to the Excel file made available in the web page of the course.

The values of x are as follows: Group 1 – 72.5%

Group 2 – 75.0%

Group 3 – 77.5%

Group 4 – 80.0%

Group 5 – 82.5%

Practical Task n.º 10 Generation of synthetic flow series. Application to the design of artificial reservoirs. Probabilistic approach

The present practical task aims at exemplifying the generation of synthetic flow series and its application to the design of reservoirs under a probabilistic perspective. For this purpose you should adopt the series of monthly and annual inflow volumes at the outlet section of the watershed of the case study, used in the Practical Task n.º 9.

- a) Check the previous annual sample for outliers based on the test of page 287 of Naghettini & Pinto, 2007, corrected according to the corresponding erratum (see Practical Task n.º 3).
- b) Confirm that from the Galton (or log-Normal) and log-Pearson III laws one that best fits the sample of annual inflow volumes is the Galton law. Include in your report the figure that supports the appraisal of the visual adjustment.
- c) Based on the law with best adjustment, generate $M = 50^2$ synthetic series of N years of annual inflow volumes and by applying the method of the fragments the corresponding synthetic series of monthly inflow volumes. The length N should be equal to the length of the sample ($N=21$ years). Consider that the number of classes of fragments is five and that the classes are defined by a constant non-exceedance probability increment of the annual inflow volumes of $100/5=20\%$ (**the limits of the classes should be round to units**).

If V denotes the annual inflow volume comprehended in a given class with limits V_i and V_{i+1} , the definition of the classes should be as follows (you should include in your answer the definition of the classes):

- $V \leq V_1$
- $V_1 < V \leq V_2$
-
- $V > V_k$

- d) Confirm that the statistical characteristics of the historical series of annual and monthly inflow volumes are preserved by the synthetic series for significance level of $\alpha = 95\%$ (**see next page**).

The historical inflow series and the complete set of 50 synthetic inflow series, as well as the results from the analysis of the preservation of the historical statistical characteristic must be organized in an Excel file exemplified in the last page of the Protocol. The file must be delivered along with the written report of the Practical Task.

- e) Based on the sequential peaks method, compute the 50 storage capacities required to meet the uniform demand of x of the mean annual volume in the 21-year period to which the records refer. Consider that the evaporation losses and other losses are already included in the demand.
- f) By applying to the previous storage capacities the same statistical law previously identified as having the best adjustment to the annual inflow volumes, compute the storage capacity that ensures the demand in 90% of the occurrences (non-exceedance probability).

² For each group, 30 of those series are given in the website of the course.

Note:

The values of x are as follows: Group 1 – 72.5%
Group 2 – 75.0%
Group 3 – 77.5%
Group 4 – 80.0%
Group 5 – 82.5%

Preservation by the synthetic flow series of the historical characteristics (see also page 24 of the MSc
Thesis of Silva, A.T., 2010)

The synthetic flow series obtained based on a given sample are considered alternative events, as probable as the historical one, provided they preserve the historical statistical characteristics.

Let $\hat{\theta}^{(m)}$ denote a statistical characteristic (average, standard deviation and skewness coefficient, the last two, with bias correction) obtained based on each of the m synthetic flow series, with $m = 1, \dots, M$ (in the case of the 10th Practical Task, $M = 50$). Let $\bar{\hat{\theta}}$ and $s_{\hat{\theta}}$ denote the average and the standard deviation (with bias correction) of the set of M values of $\hat{\theta}^{(m)}$ ($m=1, \dots, M$)

The same historical statistical characteristic, θ , is said to be preserved for a given significance level, α , if it is comprehended in the following interval, where $(1-\alpha)$ is the confidence level and z is the standard normal variable

$$\theta \in \left[\bar{\hat{\theta}} - z_{\left(1-\frac{\alpha}{2}\right)} s_{\hat{\theta}} ; \bar{\hat{\theta}} + z_{\left(1-\frac{\alpha}{2}\right)} s_{\hat{\theta}} \right]$$

HISTORICAL SAMPLE

		Oct	Nov	Dec	Jan	Sep	Year
HISTORIAL SERIES	1							
	2							

	N							
	Average (hm ³)							
	Standard deviation (hm ³)							
	Skewness coefficient							

SYNTHETIC SERIES N.º 1	1							
	2							

	N							
	Average (hm ³)							
	Standard deviation (hm ³)							
	Skewness coefficient							
SYNTHETIC SERIES N.º 2	1							
	2							

	N							
	Average (hm ³)							
	Standard deviation (hm ³)							
	Skewness coefficient							

.....

SYNTHETIC SERIES N.º M	1							
	2							

	N							
	Average (hm ³)							
	Standard deviation (hm ³)							
	Skewness coefficient							

M SYNTHETIC SERIES

		Oct	Nov	Dec	Jan	Sep	Year
Average	Average (hm ³)							
	Standard deviation (hm ³)							
	Limits of the confidence interval							
	Lower (hm ³)							
	Upper (hm ³)							
Std. dev.	Preserved - Y - or not - N							
	Average (hm ³)							
	Standard deviation (hm ³)							
	Limits of the confidence interval							
	Lower (hm ³)							
Skewness	Upper (hm ³)							
	Preserved - Y - or not - N							
	Average							
	Standard deviation							
	Limits of the confidence interval							
	Lower							
	Upper							
	Preserved - Y - or not - N							