Part II : URBAN SANITATION

Chapter objective

The objective of this chapter is to:

- Understand the main hydrological phenomena that govern the water cycle (precipitation, infiltration, runoff, evapotranspiration, groundwater and surface flow).
- Identify the factors that influence these processes (climate, topography, soil, vegetation, human activities).
- Introduce the basics of hydrological modelling, an essential tool for forecasting water resource availability, analyzing floods, managing reservoirs and planning the sustainable use of water resources.

Introduction

Hydrology is the science that studies water in the environment: its distribution, circulation, and interactions with the soil, atmosphere, and living organisms.

Hydrological phenomena, such as rainfall, infiltration, runoff, and evaporation, play a crucial role in water resource management. Understanding these mechanisms is essential for :

- The drinking water supply,
- Agricultural irrigation,
- Flood prevention,
- The protection of ecosystems.

However, climate variability and the impact of human activities complicate this analysis. Therefore, hydrological modeling is used to reproduce the behavior of watersheds and predict the evolution of water resources under different conditions.

VI.1 Some definitions

The model: Coming from the Latin " modulus ", the word model is defined by the Grand Larousse as "that which is given to you to serve as a reference".

The model is first and foremost a mock-up which represents, in a simplified way, the properties of an object or a natural phenomenon.

Modeling stems from a desire to represent, in a more or less simple way, a reality that is too complex. It can therefore be defined as a schematic representation of a physical phenomenon created for the purpose of studying it.

The scientist then attempts to reproduce the response of a phenomenon by simplifying the environment of their study. This approach is called "modeling".

Surface hydrology: studies runoff, erosion phenomena, stream flows, and flooding.
Groundwater hydrology or hydrogeology: focuses on subsurface resources, their capture, protection, and replenishment.
Urban hydrology: A branch of hydrology concerned with the part of the water cycle affected by urbanization or impacting the functioning of the city: water infiltration into soils and groundwater flow, surface runoff, drinking water production and distribution, wastewater and stormwater collection and treatment… **(Beven, K. 2012).**

VI.1.1 Principles of hydrological modeling

Hydrological models can have several objectives:

1. Understanding how a watershed functions,
2. Assessing the risks of flooding and low water levels,
3. Estimating flow rates on ungauged watercourses,
4. Simulate the impact of development scenarios,
5. Simulating the impact of climate scenarios, …

VI.2 Why urban sanitation?

The purpose of urban sanitation, as defined in this instruction, is to ensure the evacuation of all rainwater and wastewater, as well as their discharge into natural outlets in ways compatible with the requirements of public health and the environment.

-A sanitation system allows the collection and treatment of wastewater from the urban area in order to release treated water, that is to say less harmful, into the environment.

VI.2.1 Types of wastewaters

Wastewater, as its name suggests, is the result of using clean water for various purposes. After use in the kitchen, bathroom, toilet, the water is drained away through appropriate pipes.

VI.2.1.1 Domestic wastewater consists of:

1. Household wastewater from the kitchen and bathroom. It contains organic matter that can ferment (grease, food scraps) and washing products.
2. Wastewater from toilets is very rich in organic matter that can ferment and contains pathogenic germs.
3. Industrial wastewater is discharged by the various industries located in the urban area.

These industries can be very varied and therefore produce wastewater of very different compositions: food, clothing, mechanical industries, etc.

In general, the law requires these industries to treat their wastewater before releasing it into the public network or into nature.

VI.2.1.2 Rainwater (runoff water) : These are the waters from atmospheric precipitation (rain, snow, hail) that run off the roofs, the ground and the facades.

Wastewater and rainwater are collected through what is called a sanitation network, the purpose of which is to collect wastewater from the different districts of an urban area and transport it to a treatment plant, where it will be treated to then either be reused or discharged into the environment.

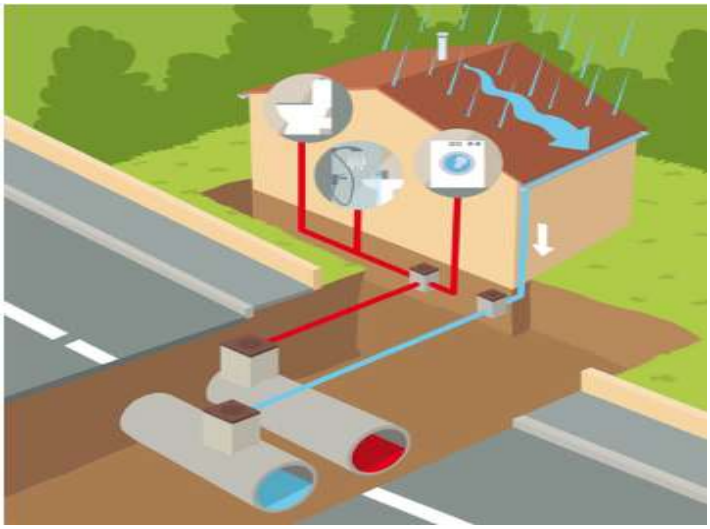
VI.2.2 Types of sanitation systems, their advantages and disadvantages

The most common sanitation systems are:

VI.2.2.1 Separation System

The separate system consists of two networks: a network for wastewater and a network for rainwater.

The separate system allows for better operation of wastewater treatment plants and more economical sizing of the sanitation network since rainwater is evacuated by a separate network. (See figure VI.01).



Figures VI. 01: Schematic diagram of the connection of a dwelling to a separate sanitation network. **(Beven, K. 2012).**

VI.2.2.2 Combined system (sewer system)

Which collects and transports all water (rainwater and wastewater),

It is recognized that the unitary system is attractive due to its simplicity, since it only requires a single pipe in each public road and a single connection for each dwelling. (See figure VI.02).



Figures VI.02 Principle of wastewater and rainwater collection in a combined network **(Beven, K. 2012).**

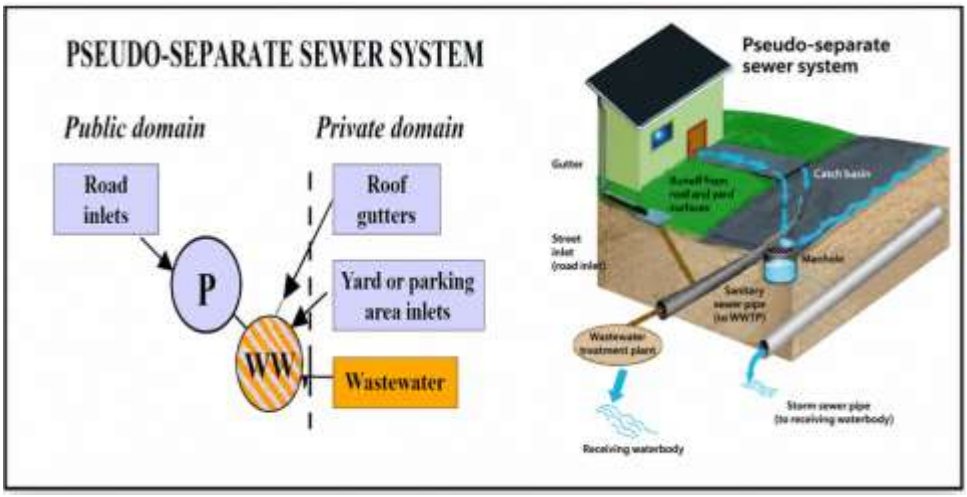
VI.2.2.3 Pseudo-separator system

The meteoric waters are divided into two parts:

· On the one hand, water from road surfaces that flows through structures designed for this purpose: gutters, ditches, etc...

· On the other hand, water from roofs, courtyards, and gardens flows into the sanitation network using the same connections as domestic wastewater.

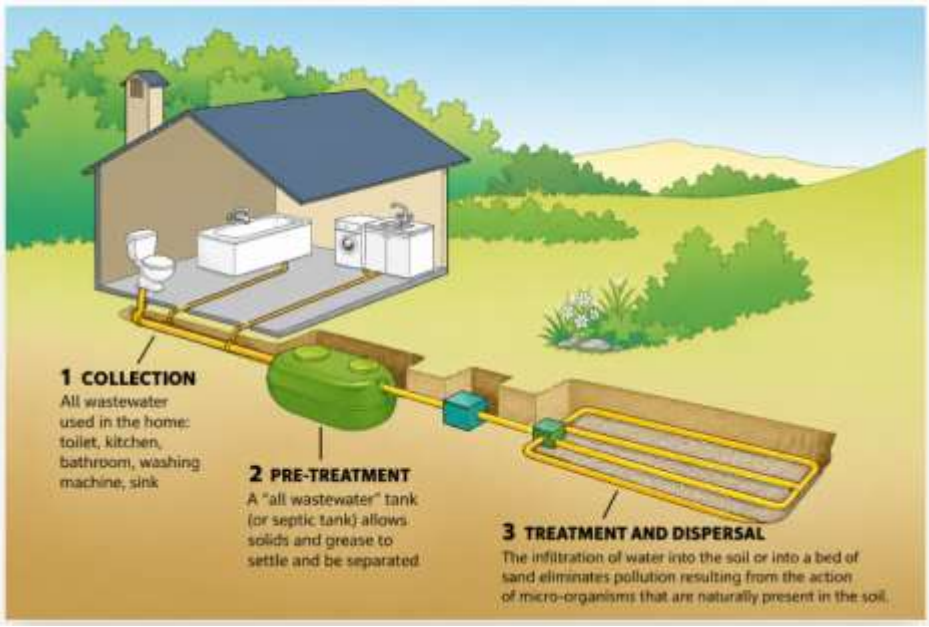
This system is useful when collective impermeable surfaces (roads, parking lots, etc.) cover a large area with steep slopes. (See figure VI.03). **(CEREMA, 2017).**



Figures VI.03. Pseudo-separator system.

VI.2.2.4. Individual or non-collective sanitation

Individual sanitation is the system used in low-density urban areas where wastewater from a dwelling is disposed of either at the dwelling itself or outside on an adjacent property. (See figure VI.04).

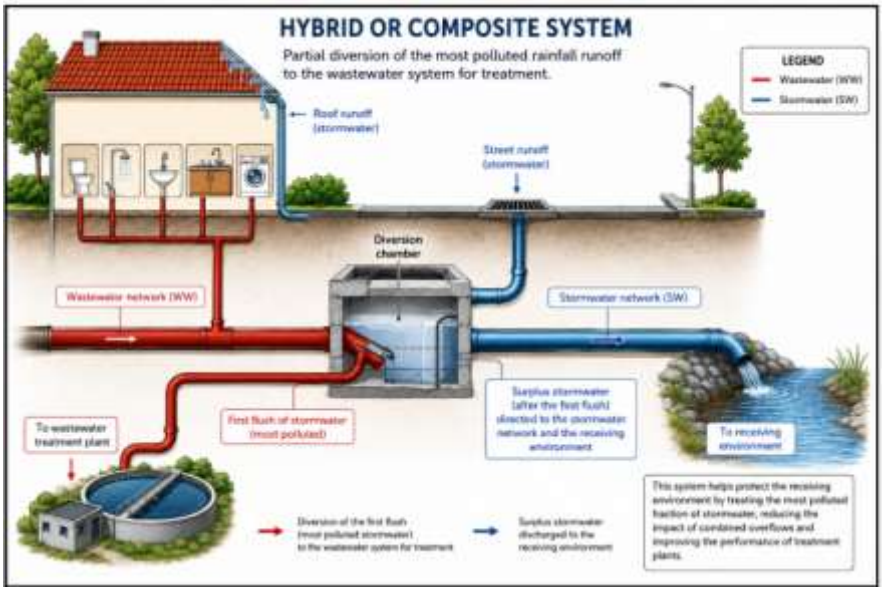


Figures VI.04 Schematic diagram of a non-collective sanitation system. (CEREMA, 2017).

VI.2.2.5 The hybrid or composite system

This system provides for the partial diversion of rainwater to wastewater by means of various devices.

This is a variant of the separate system which, through various modifications, provides for a partial diversion of the most polluted water from the stormwater network to the wastewater network for treatment. (See figure VI.05). **(CEREMA, 2017).**



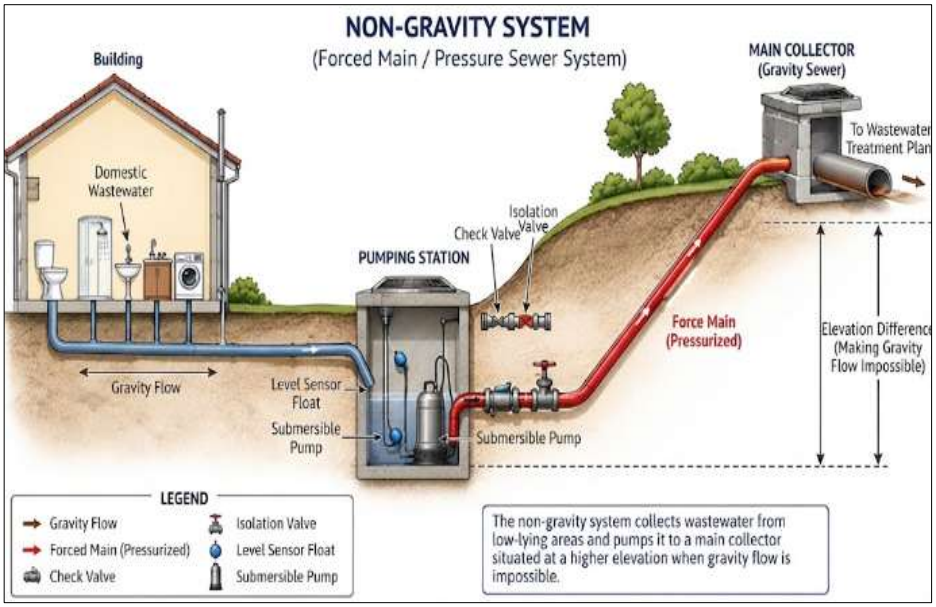
Figures VI.05: Schematic diagram of the hybrid or composite sanitation system,

VI.2.2.6 The non-gravitational system

Depending on the site's topography, this type of system avoids the need for deep pipe laying, resulting in a clear economic advantage.

It also allows for inter-municipal connections, enabling the grouping of municipalities and thus reducing the need for wastewater treatment plants.

Furthermore, it is important to emphasize the significantly higher watertightness of these networks compared to gravity-fed systems, thereby limiting the risk of pollution to the natural environment and the risk of extraneous water infiltration into the network. (See Figure VI.06). **(CEREMA, 2017).**



Figures VI.06: Schematic diagram of a non-gravity sanitation system

VI.3 choice of sanitation network

The choice of network type must meet the following criteria:

- the type of system already existing and to which a connection is possible.
- The cost of implementation
- The existence or absence of a wastewater treatment plant
- The topography of the terrain (gravity flow or pressurized flow)
- Urbanization density: in an agglomeration with dense urbanization, the unitary system is generally the most used. **(CEREMA, 2017).**

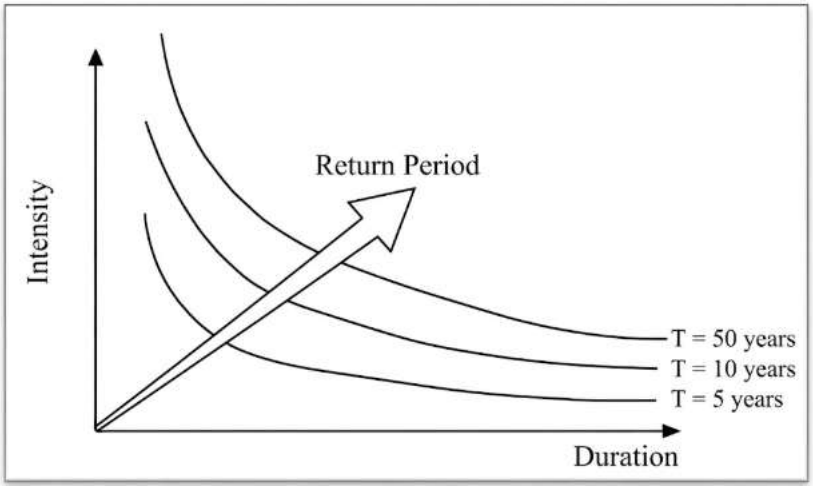
VI.4 Concept of a downpour and intensity

We generally use the term "shower" to refer to a series of rain events associated with a well-defined weather disturbance. The duration of a shower can therefore vary from a few minutes to a hundred hours.

a) General principle

Intensity-Duration-Frequency (IDF) curves are essential tools for sizing drainage networks and hydraulic structures. They relate the maximum intensity of rainfall to its duration over a given return period (2, 5, 10, 20, 50, or 100 years). They are established from the statistical analysis of rainfall observed over different time intervals. (See Figure VI.07).

b) Two fundamental laws characterize rainfall: with a constant return period, intensity increases as duration decreases **;** with a constant duration, intensity increases with the return period . These relationships allow for the determination of design flows and ensure reliable and economically optimal sizing of drainage structures. **(WMO, 2021).**



Figures VI.07: Intensity–Duration–Frequency (IDF) curves of rainfall.

VI.4.1 Methodology for constructing IDF curves

The methodology for constructing IDF curves comprises three (03) steps:

-We calculate the empirical probability of the sample and then fit the sample with several distributions to validate the most suitable distribution.

-We determine the quantiles corresponding to chosen return periods (2, 5, 10 or 20 years)

-We fit an empirical equation on the quantiles, which allows us to link each chosen intensity to the corresponding return time.

$$I = a * t^{-b} \qquad \text{VI.01}$$

With:

I(mm/h): Maximum rainfall intensity,

A and b are the adjustment coefficients

t (min): duration of the rain

VI.4.2 Application areas of IDF curves

The applications of IDF curves are numerous and varied, including:

✓ Stormwater drainage,

✓ Public Works: sizing of structures, infrastructure

Forecasting and prediction: flood risk management (floods, low water levels)

✓ Water resource planning and management,

✓ Agribusiness,

VI.4.3 Construction of IDF curves

IDF curves are established based on the analysis of rainfall recorded at a station over a long period. The resulting curves can therefore be constructed analytically. **(Chow et al. (1988); WMO, 2021).**

VI.4.4 Mathematical models of IDF curves

IDF curves can be represented by several empirical models linking rainfall intensity to its duration:

1. **Montana's Law:** **Montana, M. (1927).**

$$I = a\,t^{-b} \qquad \textbf{VI.02}$$

2. **Talbot's Law:** **Talbot, AN (1881)**

$$I = \frac{a}{t+b} \qquad \textbf{VI.03}$$

3. **Sherman's Law:** **Sherman, CW (1931)**

$$I = \frac{a}{t^{n}} \qquad \textbf{VI.04}$$

The application of the Intensity-Duration-Frequency (IDF) curves consists of determining, for each sub-watershed, the rainfall intensity corresponding to the time of concentration and the chosen return period.

This intensity allows for the calculation of the design flow rate and the sizing of the pipes. For sub-basins located downstream, the calculations take into account the cumulative contributions from upstream basins to ensure consistent hydraulic sizing of the entire network.

VI.5 Calculation of stormwater flow rates

The basic calculation for sizing a stormwater network is based on the heaviest rainfall event likely to occur within a 10-year period (10-year flood event). During a rainfall event, only the runoff portion is relevant for sizing a specific structure designed to adequately handle the water flow from that portion of a given catchment area.

The calculation of runoff water flow rate takes into account:

• characteristics of the rain (intensity, duration and spatio-temporal distribution);

• characteristics of the catchment area surface (nature, degree of permeability, slope and development);

• Hydraulic laws relating to free-channel flows. **Chow et al. (1988).**

VI.5.1 Choice of calculation model

The sizing of sanitation networks is based on several hydrological models that allow for the estimation of design flow rates.

These models, which are more or less empirical, are generally developed from statistical analyses and assumptions representative of the characteristics of the watershed and the urban environment. Their selection depends on the level of precision required and the available data.

Among the most commonly used models are:

- The rational model;
- Caquot's superficial model;
- The Mac-Math model;
- The Mallet-Gauthier model.
- Soil Conservation Service – Curve Number method);

Choosing the most appropriate model depends primarily on several factors, including:

- The surface area of the catchment area;
- The nature and permeability of the soil;
- The slope of the land;
- The hydraulic characteristics of the structures (roughness, geometry, etc.).

VI.5.2 The rational method

This method is used for limited areas (generally less than 10 hectares). The result is better for even smaller areas, due to the accurate estimation of the runoff coefficient.

The determined flow rate is proportional to the average intensity, the runoff coefficient and the swept area. **Kuichling , E. (1889).**

$$Q = kCiA \qquad \text{VI.05}$$

With:

Q: Rainfall flow rate (m^3/s)

k: intensity correction coefficient taking into account rainfall in space, the determination of which depends on the basin's elongation

C: runoff coefficient specific to the sub-basin

A: basin area (ha)

I: maximum rainfall intensity (in l/s/ha)

VI.5.2.1 The runoff coefficient C r

It is the ratio of the volume of water that runs off a surface to the volume of water that falls on that same surface. **Chow et al. (1988).**

It plays a key role in assessing peak rainfall flows, which are used to size the network. Its value varies from 0.05 to 1, depending on several factors:

- ✓ The nature of the soil
- ✓ The slope of the land
- ✓ Land use patterns
- ✓ Population density
- ✓ The duration of the rain
- ✓ Air humidity

In general terms, the runoff coefficient can be considered as the rate of impermeability of the sub-basin.

$$C_r = \frac{A_{imp}}{A_t} \qquad \text{VI.06}$$

With:

A_{imp}: Impermeable surface

A_t: Total surface area of the sub-basin

The runoff coefficient C must be determined from tables of values calculated according to the nature of the soil; (See Table VI.01).

Table VI.1: Runoff coefficients according to the nature of the surface. **Chow et al. (1988).**

Nature of the surface	Runoff coefficient (C)
Roofs, waterproof pavements	0.90
Wide joint paving	0.60
Unpaved roads	0.35
Gravel paths	0.20
Wooded areas	0.05

The method can, however, be used without elementary area decomposition by using the following coefficients (see Table VI.02):

Table VI.2: Runoff coefficients according to land cover type Chow et al. (1988).

Land use type	Runoff coefficient (C)
Very dense housing	0.90
Dense housing	0.60 – 0.70
Less dense housing	0.40 – 0.50
Residential neighborhoods	0.20 – 0.30
Squares, gardens and meadows*	0.05 – 0.20

- **Determination of the weighted runoff coefficient**

In the case where the surface of the basin considered is made up of several catchment areas "Ai " to which runoff coefficients "Cri" are assigned, the weighted runoff coefficient is calculated:

$$C_{rp} = \frac{\sum C_{ri} A_i}{A_t} \qquad \text{VI.07}$$

With:

C $_{rp}$: weighted runoff coefficient

A $_i$ **:** elementary surface

Cry: runoff coefficient corresponding to A $_i$

VI.5.2.2 Rainfall Intensity

The average precipitation intensity is the amount of water that fell during a unit of time.

a. Determining the intensity

Intensity is expressed as a function of parameters **a** and **b** by the "Montana" formula:

$$\mathbf{i\ (T,\ F)\ (mm/h) = a.tc^{-\ b}} \qquad \mathbf{VI.08}$$

a and b Montana parameter function of rainfall valid for a return period T and a given rainfall duration tc .

Note = Montana coefficients vary according to region and return period. Chow et al. (1988).

Example:

T = 10 years (10-year rainfall event)

Time interval tc = 15 mm

- regional values of a and b:

a = 6.7 b = 0.55

i = 1.51 mm/h

b) Maximum annual daily rainfall

$$Pj = \frac{pjmoy}{\sqrt{Cv^2+1}} \times \exp(u \times \sqrt{\ln(Cv^2+1)}) \qquad \text{VI.09}$$

Pjmoy : average daily rainfall (mm).

Cv: Coefficient of variation.

U: Gaussian variable.

ln : Natural logarithm.

Table VI.03: Relationship between the frequency of overshoot, the return time and the Gaussian variable Chow et al. (1988).

Frequency of exceedance (%)	50	20	10	5	2	1
Return period (years)	2	5	10	20	50	100
Gaussian variable (U)	0	0.841	1.282	1.645	2.057	2.327

For drainage works, we adopt a return period of 10 years.

c) Rainfall frequency calculation

The frequency of showers is given by the following formula:

$$Pt(\%) = Pj\ \% \times \left(\frac{Tc}{24}\right)^{b} \qquad \text{VI.10}$$

Pj : Maximum daily rainfall height (mm).

b : Climatic exponent.

Pt: maximum annual daily rainfall.

tc : Concentration time (hour).

VI.5. 2.3 Concentration Time

The duration t of the downpour which produces the maximum flow rate Q is taken to be equal to the time of concentration.

The time of concentration is defined as the time required for a water particle to travel the longest hydraulic path from the basin boundary to the outlet. Chow et al. (1988).

$$Tc = 0.127.\sqrt{A/P} \qquad \text{VI.11}$$

Depending on the characteristics of the drained basin, the concentration time is estimated respectively based on:

a. VENTURA (Ray):

$$Tc = 0,13\frac{\sqrt{S}}{P} \qquad \text{VI.12}$$

Tc: time of concentration in hours;

S : catchment area in km²;

p : slope in m/m

Area of validity between 1 and 20 km² or greater than 10 km² depending on the works.

b) TURRAZA:

$$Tc = 0,108\frac{\sqrt[3]{S*L}}{\sqrt{P}} \qquad \text{VI.13}$$

Tc: time of concentration in minutes;

S : catchment area in hectares;

L : longest hydraulic path in meters;

p : slope in m/m

c) VEN TE CHOW:

$$Tc = 0,096 * L^{0,096}P^{-0,32} \qquad \text{VI.14}$$

Tc: concentration time in minutes;

L: longest hydraulic path in meters

p: slope in m/m Range of validity for agricultural catchment areas of 1 to 2 hectares.

Note: It is complicated to find the range of validity of concentration time formulas, especially since, according to the literature, the validity times vary for the same formula.

Concentration time t_c is defined as the time required for a drop of water to travel the longest hydraulic path between the furthest point in the catchment area and its outlet.

It represents the time from which the entire basin simultaneously contributes to the peak flow.

Concentration time consists of several components:

- **Time** t_1: the time it takes for water to flow through the pipes.

$$t_1 = \frac{L}{V} \qquad \text{VI.15}$$

With:

L: pipe length (m);

V: average speed of the water in the pipe (m/s).

- **Time** t_2: time required for water to reach the first intake structure (gully or sewer inlet) by surface runoff.

Desbordes' (or Caquot's) formula:

$$t_2 = I_p^{-4/11} \qquad \text{VI.16}$$

Or:

I_p: average slope of the hydraulic path at the ground surface (m/m).

- **Time** t_3: runoff time over a basin without a network of pipes.

$$t_3 = \frac{L}{11\sqrt{I_p}} \qquad \text{VI.1 7}$$

L: length of the hydraulic path (m);

I_p: average slope of the basin (m/m).

➢ **Case study: Calculating concentration time**

Depending on the configuration of the catchment area, the time of concentration is expressed as follows:

Concentration time can therefore have three aspects:

The basin has no pipes: **tc = t 3**

The basin consists of a surface section followed by a pipe: $t_c = t_3 + t_1$

The basin is urbanized and includes a main pipeline and tertiary connections: $t_c = t_2 + t_1$.

- Simplified method for calculating [value] t_c for an urbanized area:

We assume a surface circulation time equal to 5 mnand a sewer velocity equal to 1 m/s:

$$t_c = t_1 + t_2 = \frac{L}{V} + 5 \text{ mn} \qquad \text{VI.1 8}$$

$$t_c \text{ (mn)} = \frac{L \text{ (m)}}{60 \text{ m/mn}} + 5 \text{ mn} \qquad \text{VI.1 9}$$

VI.5.3 Caquot's Method

a. definition

A comprehensive method for calculating the maximum flow rate corresponding to a given return period at the outlet of an urban catchment. The use of the Caquot method was recommended in France as early as 1949. **Caquot, A. (1949).**

$$Q\,(m^3/s) = K^{I/U} I^{V/U} C^{1/U} A^{W/U} \qquad \text{VI.20}$$

With:

Q: flow rate in m³/s

I: average slope of the catchment area (m/m)

C: runoff coefficient of the watershed

A: surface area of the BV in hectares

In which the parameters (K, U, V and W) are functions of the Montana coefficients a(T) and b(T) and the latter depend on the return period, represented in the following table:

VI.04. Calculation parameters *U*, *V* and *W* their determination formulas

Settings	The formulas
U	U = 1 + 0.287, b(T)
V	V = -0.41 ,b (T)
W	W = 0.95 + 0.507,b (T)
K	K = 0.5 b(T) × a(T) / 6.6

Note: The validity limits of the CAQUOT formula are:

1. Basin area or group of basins $\Sigma A \leq 200$ ha
2. Slope $0.0002 < I < 0.05$
3. Runoff coefficient $0.2 < C < 1$
4. The elongation coefficient: $M \geq 0.80$

K, u, v, w: are coefficients that vary according to the frequency and rainfall region considered.

Table VI.05: The different forms of the formula as a function of the return period

Return period	The formula
10 years	$Q = 1.430\ I^{0.29}\ Cr^{1.20}\ A^{0.78}$
5 years	$Q = 1.192\ I^{0.30}\ Cr^{1.21}\ A^{0.78}$
2 years	$Q = 0.834\ I^{0.834}\ Cr^{1.22}\ A^{0.77}$

A year	$Q = 0.682\ I^{0.32}\ Cr^{1.23}\ A^{0.77}$

For the region, they would meditate and have a return period of 10 years; the previous formula is written

$$Q = 1.29\ I^{0.21}\ \ Cr^{1.14}\ A^{0.83} \qquad \text{VI.21}$$

b. Calculation principle

The surface method leads to determining the flow rate from the physical characteristics of the catchment area.

c. Determination of parameters

- **The average slope**

The average slope of this network depends not on the widths of these sections but on the flow rates caused by each of the sections.

$$Imoy = \sum_{i}^{n} \frac{Qi \times Li}{Qi} \qquad \text{VI.22}$$

Li: the longest hydraulic path of the basin (case of series arrangement).

II: slope of the basin

- **Lengthening of watersheds**

The elongation “M” is defined as the ratio of the longest hydraulic path “L” to the square root of the area of the basin considered. Its expression is as follows:

$$M = \frac{L}{\sqrt{A}} \geq 0.8 \qquad \text{VI.23}$$

L: length (in hectometers) of the longest hydraulic path
A: surface area of the basin (in hectares)

When greater accuracy is required in the assessment of flows, particularly for the sizing of large structures or for very compact or very elongated catchments, the calculated flow can be corrected by an influence coefficient **"m"** linked to the elongation coefficient **"M"** of the catchment.

This coefficient reflects the inverse variation of the flow rate with the length of the basin for the same surface area. When **M ≠ 2,** the flow rate is corrected by applying the factor **m.** which is given by the formula:

$$m = \left[\frac{M}{2}\right]^{[0.84\, b(F)/(1-b(F)F)]} = \left[\frac{M}{2}\right]^{[-0.42]} \qquad \text{VI.24}$$

With:

M: coefficient of elongation

b (F): parameter for expressing rainfall

f: the adjustment factor for peak flow in the expression for time of concentration

(F = -0.287)

Q_p corresponds to a raw value; this must take into account the coefficient **m** of the shape of the basin.

$$Q_{p\ corrected} = m\ Q_{p\ raw} \qquad \text{VI.25}$$

$Q_{p\ corrected}$: corrected flow rate;

Q_{pbrute} : flow rate calculated initially;

m: influence coefficient related to the lengthening of the catchment area;

M: catchment area lengthening coefficient.

- **Evaluation of equivalent parameters for a group of basins**

The surface formula developed above is valid for a basin with homogeneous physical characteristics.

Applying the model to a group of heterogeneous sub-basins with individual parameters Aj, Cj , Ij, Lj (length of the main drain), Qpj (peak flow of the basin considered alone), requires the use of equivalence formulas for the parameters "A, C, I and M" of the grouping.

These formulas, which differ depending on whether the basins constituting the grouping are in *"series"* or in *"parallel",* are expressed below:

Table VI.06: Characteristics of each sub-basin grouping

Mass assembly	Parallel assembly
$A = \sum_{i=1}^{N} Ai$	$A = \sum_{i=1}^{N} Ai$
$Creq = \frac{\sum_{i=1}^{N} Cri.Ai}{\sum_{i=1}^{N} Ai}$	$Creq = \frac{\sum_{i=1}^{N} Cri.Ai}{\sum_{i=1}^{N} Ai}$

$Iéq = \left[\frac{\sum_{i=1}^{N} Li}{\sum_{i=1}^{N} (\frac{Li}{\sqrt{Ii}})} \right]^2$	$Iéq = \left[\frac{\sum_{i=1}^{N} IQi}{\sum_{i=1}^{N} Qi} \right]$
$M = \frac{\sum L_j}{\sqrt{\sum A_j}}$	$M = \frac{L}{\sqrt{\sum A}} (Q \max)$

A. Manning & Strickler formula (1890) Chow, VT (1959).

The formula used for sizing the collectors is:

$$Q = K*S*R^{n} * I^{½} \qquad \text{VI.26}$$

Q: is the flow rate to be transferred in m^3/s.

K= 60 for concrete pipes in a combined sewer system or for rainwater in a separate sewer system.

70 for concrete wastewater pipes in a separate sewer system.

K = 100 for high-density polyethylene (HDPE) or polyvinyl chloride (PVC) piping.

n = 3/4 for a combined network or rainwater from a separate system.

n = 2/3 for wastewater.

R: is the hydraulic radius in m

I: is the slope of the collector in m/m

S: is the wetted section in m^2.

B. Mac Math formula (McMath, JC (1951).

The statement of MacMath's formula for estimating return period flood flows

Test:

$$QT = K*P*A^{0.58}*I^{0.42} \qquad \text{VI.27}$$

QT: peak flow rate in m^3/s with a return period of [number of years]

K: coefficient dependent on the nature of the catchment area (vegetation cover, topography),

P: maximum rainfall over 24 hours in meters, return period: Tanned

A: catchment area in km^2

I: average slope of the catchment area.

C. Use of empirical formulas

The one that gave the best results is known as the Mallet-Gauthier method. For calculating the 100-year flood, we have: **Gauthier, J. (1968).**

$$C = 2k \log(1 + A) S \frac{\sqrt{1 + 4\log T - \log S}}{\sqrt{L}} \qquad \text{VI.28}$$

The coefficients k and A depend on the topographic and climatological characteristics of the terrain. MEDINGUER admits that for wadis descending from the Atlasic domain and flowing over the Hamada, k = 0.8 and A = 20 can be taken.

Conclusion

Urban hydrology allows us to analyze phenomena related to rainfall (precipitation, runoff, infiltration) on impermeable surfaces.

Intensity-Duration-Frequency (IDF) curves are essential for assessing the probability of extreme rainfall and properly sizing drainage structures.

Calculating rainfall flows, using hydrological methods and models, provides engineers with the data needed to design reliable networks, reduce flood risks, and ensure the proper functioning of urban infrastructure.