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Ministry of Higher Education and Scientific Research

University of Science and Technology of Oran Mohamed-Boudiaf

Faculty of Architecture and Civil Engineering

Department of Architecture

Handout

Roads and Miscellaneous Networks

Drinking Water Distribution and Wastewater and Rainwater Evacuation System

Dr Salima REZAK

Objectives

This course is aimed specifically at students of architecture and hydraulics, with a view to preparing them to master the intricacies of water-related systems in complex urban environments. We will delve into three crucial aspects, namely *Drinking water distribution*, *Urban wastewater disposal*, and *Rainwater management*.

In the first part which focuses on drinking water supply systems, we will begin with a first chapter devoted to the fundamental notions of fluids and their properties. We felt that architecture students often lack prior knowledge about the behavior of fluids, particularly water. This chapter aims to provide basic knowledge of hydraulics which will serve as a basis for developing subsequent chapters on urban hydraulics.

In the second chapter, we will examine the characteristics of gravity and pressure flows. This chapter is essential for introducing students to fluid dynamics, a necessary prior knowledge for the following chapters (third, fourth, and fifth). They will address water catchment, the design of drinking water supply networks, and will broaden the discussion to sanitation in the second part, which focuses on wastewater evacuation systems. This fundamental approach will allow students to grasp the principles of hydraulics essential to urban planning.

The final chapter addresses the treatment of wastewater, enlightening students on the importance of this step in safeguarding the environment and the health of populations. It highlights the methods required to remove contaminants, particularly organic materials. Thanks to purification that complies with discharge standards, treated water can be reintroduced into the natural hydrological cycle without harming the ecosystem.

At the end of this course, students should be able to:

- Read and interpret hydraulic data.
- Understand the hydraulic and hydrological phenomena to be taken into consideration in construction (buildings).
- Design water storage structures for the supply of drinking water.
- Establish drinking water distribution and wastewater and rainwater evacuation systems, so as to avoid water stagnation and flooding.

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General Introduction

Water and the building

The urban hydraulics course is highly important for any architecture student. Indeed, the architect must first collect as much data as possible regarding water, in all its forms (drinking, used, rain), on the construction site, in order to find solutions and to be able to make decisions.

The choice of location of new residential buildings or commercial buildings cannot be made superficially. On the contrary, multiple criteria must be taken into account and studied in order to provide the best possible public service and to strengthen urban cohesion and improve the quality of the landscape. It is always useful to find out about the site and know all the details relating to water. To do this, we can then ask ourselves the following questions:

Do the networks to which the future equipment will be connected have sufficient capacity? How far away are they located? (For example, insufficient networks to meet the needs of a building for its drinking water supply, sanitation network to collect wastewater, etc.).

Is the water pressure on the ground sufficient (tap pressure on a network)? When the pressure delivered by the drinking water network is insufficient, it is recommended to install a pumping system). It is useful to know that the public network delivers drinking water at an average pressure of around 3 bars. Above 5 bars, the installation of a pressure reducer is necessary in order to protect the domestic installation. Below 3 bars, a suppressor can be used to increase the pressure.

What is the degree of permeability of the soil? Is it possible to control rainwater by infiltration? Is it necessary to provide a retention system? (A rainwater harvesting system may be installed to prevent flooding in the event of heavy rain. This rain can be used for watering or other purposes).

Is there a groundwater table under the building site (depth of the groundwater, risk of its contamination, and risk of its rise)?

Are there any pollutant discharges nearby? What are they? How polluted are they? (For example: release of toxic products into waterways upstream of the building site).

What are the levels of groundwater and ground pollution? (Reduce the risks of contact with toxic materials).

Is the site subject to a natural risk prevention plan?

Has it been exposed to disasters or natural hazards (Floods or others)?

According to the Official Journal of the Algerian Republic No. 07, Article 43 stipulates that the building permit application must be accompanied by the following files:

I/ An architectural file including, among other things:

- The indication of the viability networks serving the land with their main technical characteristics as well as the connection points and the layout of the roads and networks projected on the land.

II/ A technical file including, among other things:

- The summary description of the electricity, gas, heating, drinking water supply, sanitation and ventilation systems,
- Plans of wastewater evacuation networks,
- The nature and quantities of liquid, solid or gaseous substances harmful to public health, agriculture and the environment, contained in discharged wastewater as well as gaseous emissions. The wastewater treatment, storage and filtering system must be described.

Part 1

Drinking water distribution system

Chapter I: Basic concepts of hydraulics

I.1 Introduction

In the field of urban hydraulics, a thorough understanding of the properties of liquids, especially water, is crucial. This knowledge is essential for the efficient design and management of drinking water supply, rainwater drainage and wastewater treatment systems in urban environments. Furthermore, water, due to its ability to deform and flow under external forces, occupies a central place in water infrastructure that ensures safe water supply and sustainable use of water resources in an urban environment. Before diving into the details of drinking water distribution systems, wastewater evacuation, and rainwater management in urban areas, it is therefore essential to understand certain basic concepts about liquids.

I.2 Definition of a fluid

Fluids, whether liquid or gas, are distinguished by their ability to deform and flow under external pressure, which means that any extremely small effect can result in an indefinite relative displacement of their particles.

I.2.1 Perfect Fluids - Real Fluids

In a *Perfect Fluid*, the absence of viscosity implies that there are no tangent forces, whether the fluid is stationary or moving. This means that the fluid particles can move without encountering internal resistance.

In a *Real Fluid*, frictional forces resulting from molecules sliding past each other create viscosity, which affects motion and requires energy. This is what defines a fluid as real.
A fluid which can be in a liquid or gaseous form consists of a set of microscopic particles occupying a volume whose shape adjusts to that of the container which retains it.

I.2.2 Gaseous state

In gases, particles have the ability to move freely in all directions and with great speed. The forces of attraction are zero between these particles which tend to uniformly occupy the entire volume around them **Figure 1 a)**.

I.2.3 Liquid state

The liquids take on the shape of the vase which encloses them. In the static state, there is a free surface for separation between the liquid and the air. It is flat and horizontal **Figure 1 b)**.

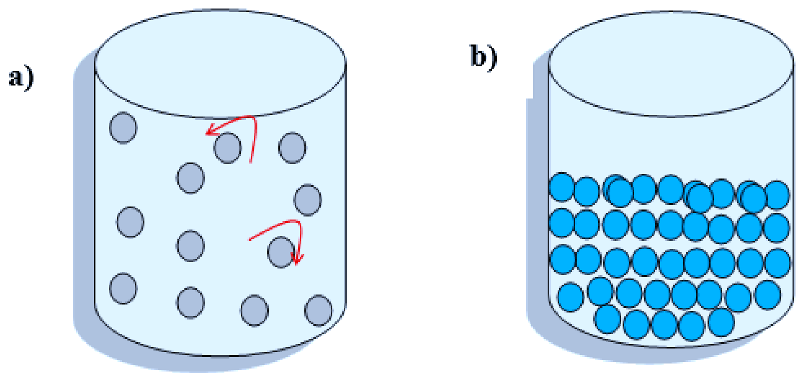


Figure 1: a) Gazeous state; b) Liquid state

I.3 Main properties of liquids

We limit ourselves here to recalling the definition as well as the main characteristics of liquids. The properties of liquids must be well mastered in order to be able to analyze the flows in a hydraulic circuit.

I.3.1 Specific weight

This is the most important property in hydrostatics, i.e. fluid at rest. It is designated by ω and is equal to the weight of the unit volume of the liquid considered. It is given by the following expression:

$$\omega = G/V$$
$$\omega = mg/V\ [\mathrm{kg.m/m^2s^2}]$$

$$[\mathrm{kg.m/m^2s^2}] = [N/m^2]$$

Here G is the weight of the liquid and V is the volume of the liquid.

I.3.2 The density (ρ)

The density (ρ) is the ratio of the mass M to the volume V of the liquid considered.

$$\rho = M / V$$

The unit of density ρ is [kg/m^3].

For liquids, the volume is practically insensitive to variations in pressure applied to it. In the majority of cases, it increases slightly with increasing temperature. Below the temperature of 4°C, water is an exception to this rule.

Here are some examples of density for various liquids:

ρ_{water} = 1000 kg/m^3

$\rho_{mercury}$ = 13546 kg/m^3

$\rho_{dry\ air}$ = 1.205 kg/m^3

I.3.3 Density

In hydrodynamics, density and viscosity are fundamental parameters. The density of a fluid is expressed as the ratio between the mass of a specific volume of this fluid and that of the same volume of a reference body. It is given by:

$$d = \frac{m}{m_0}$$

Where d represents the density of the fluid, m is the mass of the fluid, and m_0 is the mass of the same volume of a reference body.

In the expression: $d = [m/V]/[m_0/V]$, (m/V) is the volume density of the fluid, (m_0/V) is the volume density of the reference body, and d is the density of the fluid.

Likewise, we have also: $d = \rho/\rho 0$, with ρ representing the density of the fluid and ρ_0 the density of the reference body.

The density of a liquid is generally expressed about that of water at a temperature of 4 °C and an atmospheric pressure of 1013 mbar. As for the density of a gas, it is usually related to that of air under the same temperature and pressure conditions as the gas in question.

I.3.4 Viscosity

I.3.4.1 Dynamic viscosity

Viscosity is defined as the ability of a fluid to oppose internal relative movement. Consider a fluid confined between two parallel planes. One of the planes is fixed while the other can move in its own plane with uniform rectilinear motion (**Figure 2)**. In this configuration, the fluid particles follow rectilinear and parallel trajectories.

It can be shown that the tangential or shear stress is proportional to the velocity gradient (dV/dy). It is given by:

$$\tau = \mu(dV/dy)$$

Where dV represents the variation in speed between two streams of fluid infinitely close and separated by dy and μ is the absolute dynamic viscosity coefficient of the liquid.

The usual unit of dynamic viscosity is the poiseuille which is equal to 1 Ns/m^2.

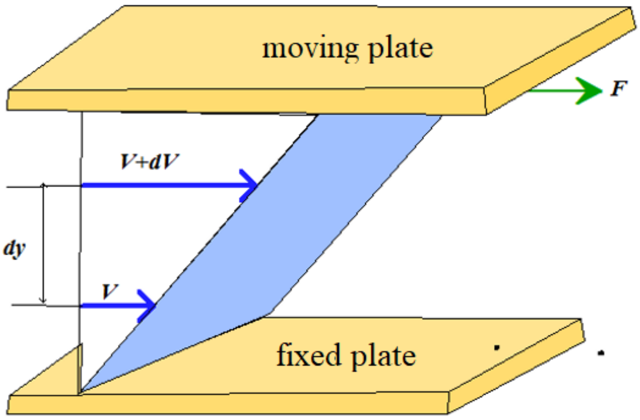


Figure 2: Distribution of speed in a liquid between two parallel plates. The upper plate can move relative to the lower plate which is fixed.

I.3.4.2 Kinematic viscosity

Kinematic viscosity, represented by ν, is defined as the ability of a fluid to resist internal relative motion. It is defined as the ratio between the dynamic viscosity μ and the density ρ of the fluid.

$$\nu = \mu/\rho$$

Here μ is expressed in $[Ns/m^2]$ and ρ in $[kg/m^3]$, which means that the unit of ν is $[m^2/s]$.

I.3.5 Compressibility

Compressibility is the property of bodies that can be compressed. This property is characterized by the compressibility coefficient β which indicates the variation in volume V relative to the unit of pressure.

$$\beta = -1\ dv/V\ dp$$

The minus sign (-) in this formula indicates that for any increase in pressure, there is a decrease in volume V.

I.3.6 Dilation

The increase in the volume of a body following an increase in its temperature is characterized by a coefficient called the coefficient of thermal expansion. Since liquids do not have linear dimensions, only their volume expansion can be considered in this case. The volume expansion coefficient β_t of a liquid represents the increase undergone by the unit of volume when its temperature t increases by 1°C.

$$\beta t = \frac{1}{V}\frac{dV}{dt}$$

I.3.7 Flow rate

Flow rate is defined as the quantity of liquid passing through a given cross section of the flow. It is billed as the Volume on Time. The flow rate is equal to the product of the cross section and the speed V of the liquid passing through this section. It is by:

$$Q = SV$$

This flow rate, as defined, is called volume flow rate. On the other hand, the mass flow rate Q_m is calculated using the expression:

$$Q_m = \rho Q = \rho VS$$

The mass flow rate Q_m is expressed in units of mass per unit of time (kg/s).

Application 1

Consider a volume of 6 m^3 of oil, having a weight of 47 KN. Find the density of the oil.

Answer

$\rho = \frac{M}{V}1$

$G = M \times g$

$M = \frac{G}{g} \rightarrow \rho \times V = \frac{G}{g}$

$\rho = \frac{G}{g \times V}$

$\rho = \frac{47 \times 10^3}{9.81 \times 6}$

$\rho = 798.5\ \frac{kg}{m^3}$

Oil is therefore lighter than water

Application 2

At pressure $P_1 = 34.5\ bars$, the volume is $V_1 = 28.32\ dm^3$, and $P_2 = 241.3\ bars$, the volume is $V_2 = 28.05\ dm^3$.

Calculate the compressibility modulus of this liquid.

Answer

Note

$1\ dm^3 = 1\ liter$; $1\ bar = 100\ 000\ Pa$; $1m^3 = 1000\ liters$

$$\beta = -1 \frac{\frac{dv}{V}}{dp}$$

$$\beta = -1 \frac{\frac{(V_2 - V_1)}{V_1}}{P_2 - P_1}$$

$$\beta = 4.61\ 10^{-10} Pa^{-1}$$

Application 3

Find the weight G_0 of a volume V = 3 liters of oil with density d = 0.918. Gravity is given by g = 9.81 m/s^2

Answer

$$\begin{cases} G_0 = mg \\ \rho = \frac{m}{V} \Rightarrow m = \rho V \end{cases} \Rightarrow G_0 = \rho_{huile} Vg$$

$$G_0 = d \times \rho_{ref} \times g$$

Numerical application

$$\rho_{ref} = \rho_{eau} = 1000 \frac{kg}{m^3}$$

$$G_0 = 27N$$

Application 4

Determine the viscosity of a liquid whose density is equal to 0.918 and its kinematic viscosity is ν = 1.089 stokes.

Note:

1 stoke = 10^{-4} m^2/s

Answer

$\rho_{Liquid} = d\rho_{water}$

$\nu = \mu/\rho$

$\mu = \nu\rho$

$\mu = \nu\rho_{water}d$

$\mu = 1.098 \times 10^{-4} \times 10^{3} \times 0.918$

$\mu = 0.1\ Pa$

I.4 Hydrostatics

Hydrostatics is the science that studies fluids at rest. A liquid can be in absolute equilibrium (with respect to the earth) or relative equilibrium if its particles do not move relative to each other. In this case, the liquid is subject to the action of volume (gravity) and surface (pressure) forces.

I.4.1 General equation of hydrostatics

Let's consider an infinitely small parallelepiped-shaped fluid element with sides dx, dy, and dz. Let ρ be the density of the liquid and m is its mass (**Figure 3**). We can then write:

$$m = \rho \times dx \times dy \times dz$$

As the parallelepiped is in equilibrium, we can then say that the sum of the volume forces and the pressure forces applied on all 6 faces are equal to zero.

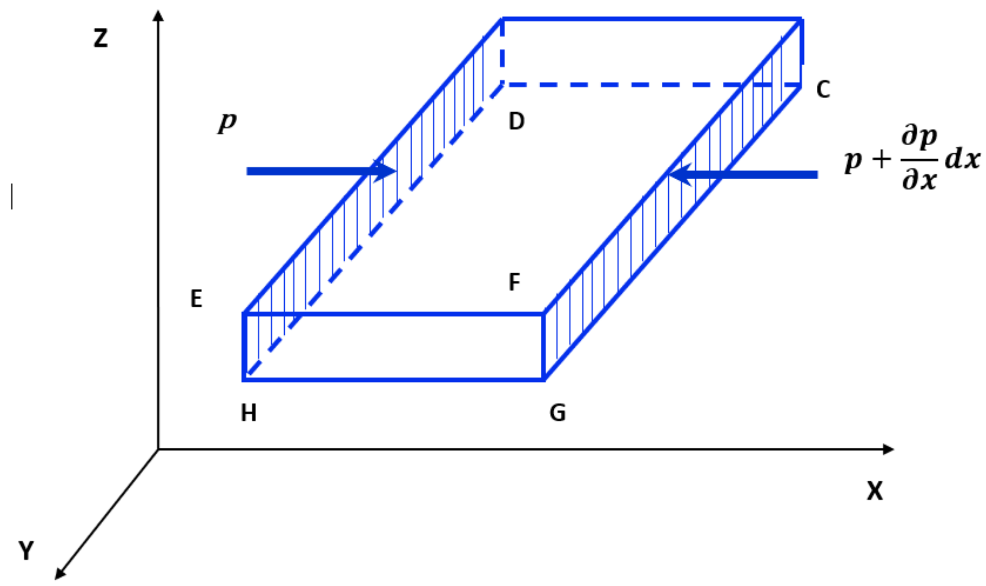


Figure 3: Elementary parallelepiped of liquid subjected to volume and surface forces

I.4.1.1 Volume forces

We consider $\vec{F}$ the resultant of the volume forces per unit mass

$$m\vec{F}\begin{cases} mf_x = \rho dxdydzf_x \\ mf_y = \rho dxdydzf_y \\ mf_z = \rho dxdydzf_z \end{cases}$$

I.4.1.2 Surface forces

The pressure p is applied on ADEH and $\left(-p + \frac{\partial p}{\partial x}dx\right)$ on BCFG.

The total pressure along X is then given by: $p - \left(p + \frac{\partial p}{\partial x}dx\right) = -\frac{\partial p}{\partial x}dx$

The force along X is: $-\frac{\partial p}{\partial x}dxdydz$

The total pressure along Y is given by: $p - \left(p + \frac{\partial p}{\partial y}dy\right) = -\frac{\partial p}{\partial y}dy$

The force along Y is: $-\frac{\partial p}{\partial y}dydxdz$

The total pressure along Z is: $p - \left(p + \frac{\partial p}{\partial z}dz\right) = -\frac{\partial p}{\partial z}dz$

The force along X is: $-\frac{\partial p}{\partial z}dzdxdy$

Given that the parallelepiped is in equilibrium, we can then write:

$$\begin{cases} \sum F/X = 0 \\ \sum F/Y = 0 \\ \sum F/Z = 0 \end{cases}$$

$\sum F/X = 0 \Leftrightarrow \rho dxdydzf_x - \frac{\partial p}{\partial x}dxdydz = 0$

$$\rho f_x = \frac{\partial p}{\partial x}$$

$$f_x = \frac{1}{\rho}\frac{\partial p}{\partial x}$$

$\sum F/Y = 0 \Leftrightarrow \rho dxdydzf_y - \frac{\partial p}{\partial x}dxdydz = 0$

$$\rho f_y = \frac{\partial p}{\partial Y}$$

$$f_y = \frac{1}{\rho}\frac{\partial p}{\partial Y}$$

$\sum F/Z = 0 \Leftrightarrow \rho dxdydzf_z - \frac{\partial p}{\partial z}dxdydz = 0$

$$\rho f_z = \frac{\partial p}{\partial Z}$$

$$f_z = \frac{1}{\rho}\frac{\partial p}{\partial Z}$$

$$\begin{cases} f_x = \frac{1}{\rho}\frac{\partial p}{\partial x} \\ f_y = \frac{1}{\rho}\frac{\partial p}{\partial Y} \Leftrightarrow \vec{F} = \frac{1}{\rho}\overrightarrow{grad}\ \mathrm{p} \\ f_z = \frac{1}{\rho}\frac{\partial p}{\partial Z} \end{cases}$$

$$\begin{cases} f_x dx = \frac{1}{\rho}\frac{\partial p}{\partial x}dx \\ f_y dy = \frac{1}{\rho}\frac{\partial p}{\partial Y}dy \\ f_z dz = \frac{1}{\rho}\frac{\partial p}{\partial Z}dz \end{cases}$$

$$\frac{1}{\rho}\left(\frac{\partial p}{\partial x}dx + \frac{\partial p}{\partial Y}dy + \frac{\partial p}{\partial Z}dz\right) = f_x dx + f_y dy + f_z dz$$

dP is the total derivative

$$dP = \frac{\partial p}{\partial x}dx + \frac{\partial p}{\partial Y}dy + \frac{\partial p}{\partial Z}dz$$

$$\frac{1}{\rho}dP = f_x dx + f_y dy + f_z dz$$

This is the second form of the hydrostatic equation.

Case of a fluid subject to the sole action of gravity

$$\vec{F}\begin{cases} f_x \\ f_y \\ f_z \end{cases} f_x = 0\ ,\ f_y = 0,\ f_z = -g$$

$$\frac{1}{\rho}dP = -gdz \Rightarrow dP = -\rho g dz$$

$$dP + \rho g dz = 0$$

I.4.2 Constant pressure (or isobaric) surface

$dP = 0 \Rightarrow \rho g dz = 0$

$dz = 0 \Rightarrow$ z = constant $\Rightarrow$ these are horizontal planes that correspond to surfaces with equal pressures.

Case of an incompressible fluid

The density is constant ($\rho = cste$)

The integral of the equation:

$(\rho = cste)$

$dP + \rho g dz = 0 \Rightarrow P + \rho g z = cste$

Absolute pressure and effective pressure

By integration between A and M (**Figure 4**)

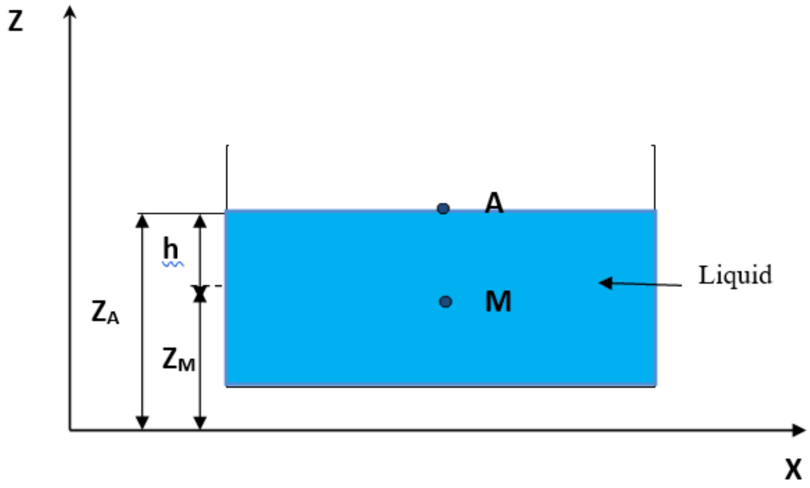


Figure 4: Scheme of an open water tank containing a liquid

$P_M - P_A = -\rho g(Z_M - Z_A)$

$$\begin{cases} P_A + \rho g(Z_A) = constant \dots\dots\dots\dots\dots\dots\dots\dots.(1) \\ P_M + \rho g(Z_M) = constant \dots\dots\dots\dots\dots\dots\dots\dots(2) \end{cases}$$

(2)-(1) $\Rightarrow P_M + \rho g(Z_M) - \left(P_A + \rho g(Z_A)\right) = 0$

$(P_M - P_A) = \rho g(Z_A - Z_M)$

$(P_M - P_A) = \rho g(h)$

$P_A = P_a = Atmospheric\ pressure$

$(P_M - P_a) = \rho g(h) \Rightarrow P_M = P_a + \rho g(h)$

$$\begin{cases} P_M = absolute\ pressure\ at\ point\ M \\ P_a = atmospheric\ pressure \\ \rho g(h) = \text{effective pressure} \end{cases}$$

I.4.3 Pascal's principle

The integration between M_1 and M_2 gives (**Figure 5**):

$P_{M2} - P_{M1} = -\rho g(Z_2 - Z_1)$ (1)

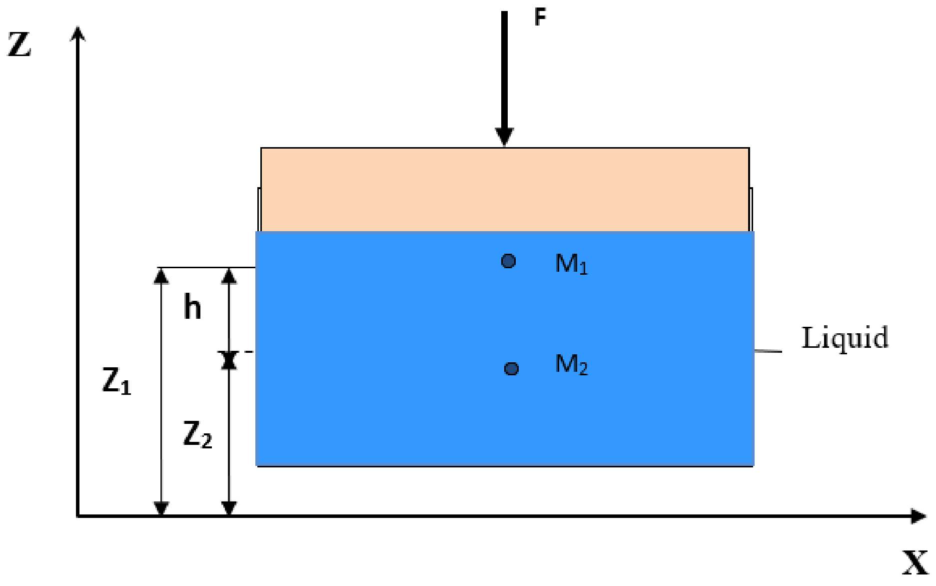


Figure 5: Scheme of Pascal's principle

If the pressure at M_1 is increased by ΔP_1 then the pressure at that point is $P_1 + \Delta P_1$

If the pressure at M_2 is increased then the pressure at that point is $P_2 + \Delta P_2$

Equation (1) then becomes :

$$(P_{M2} + \Delta P_2) - (P_{M1} + \Delta P_1) = -\rho g(Z_2 - Z_1) \qquad (2)$$

Then, putting (2) - (1) gives:

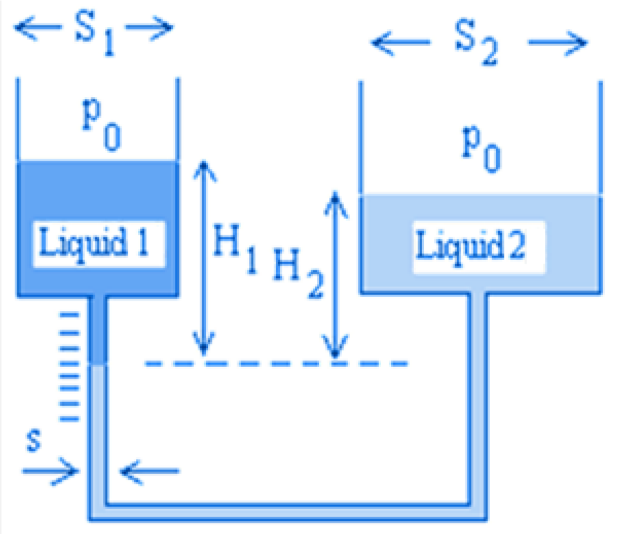
$$P_{M2} + \Delta P_2 - P_{M1} - \Delta P_1 - P_{M2} + P_{M1} = -\rho g(Z_2 - Z_1) + \rho g(Z_2 - Z_1) \Delta P_2 - \Delta P_1 = 0$$

$$\Delta P_1 = \Delta P_2$$

The pressure variations within a liquid in equilibrium are entirely transmitted to every point of the liquid mass.

Application 1

A differential pressure gauge consists of two cylindrical containers, of respective straight sections S_1 and S_2, connected by a tube of constant internal section, S. The system contains two immiscible liquids with densities ρ_1 and ρ_2.



Initially, the pressure above the two liquids is the same and is equals to P_0. The surface of separation is defined by H_1 and H_2. Deduce a relationship between ρ_1, ρ_2, H_1 and H_2.

An overpressure ΔP is created above liquid 1 and the separation surface of the two liquids moves by ΔH.

Deduce the sensitivity$\frac{\Delta H}{\Delta P}$.

N. A.:

$\rho_1 = 998\frac{kg}{m^3}; \rho_2 = 1024 = kg/m^3 ;S_1 = S_2 = 100s$

Answer

$P_0 + \rho_1 g H_1 = P_0 + \rho_2 g H_2 \Rightarrow \rho_1 H_1 = \rho_2 H_2$

When the pressure increases on side 1, the separation surface of the two liquids decreases by Δh, the free surface of liquid 1 decreases by $h_1 = \frac{s\Delta h}{S_1}$, and that of liquid 2 increases by $h_2 = \frac{s\Delta h}{S_2}$ Putting that the pressures at the separation surface of the two liquids are equal then gives:

$$P_0 + \Delta P + \rho_1 g(H_1 - h_1 + \Delta h) = P_0 + \rho_2 g(H_2 + h_2 + \Delta h)$$

Therefore,

$$\Delta P + \rho_1 g(-h_1 + \Delta h) = \rho_2 g(h_2 + \Delta h)$$

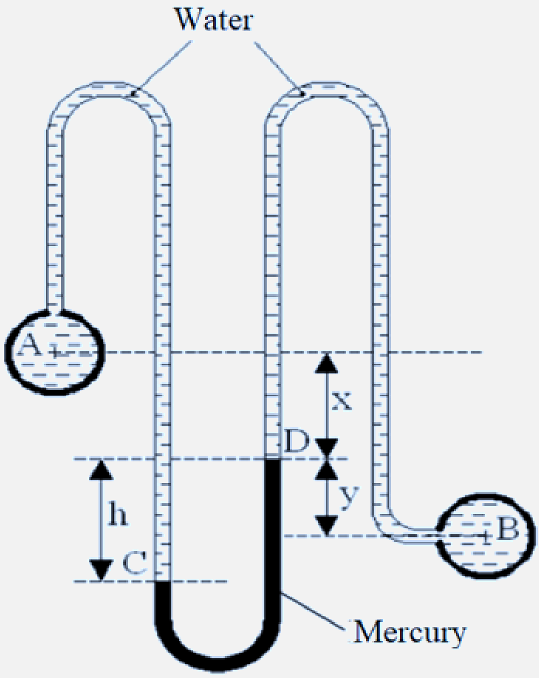
$$\Delta P = (\rho_2 - \rho_{21})g\Delta h + g(\rho_1 h_1 + \rho_2 h_2) = (\rho_2 - \rho_1)g\Delta h + gs\Delta h\left(\frac{\rho_1}{S_1} + \frac{\rho_2}{S_2}\right)$$

$$= g\Delta h\left[(\rho_2 - \rho_1) + s\left(\frac{\rho_1}{S_1} + \frac{\rho_2}{S_2}\right)\right]$$

$$\frac{\Delta h}{\Delta P} = \frac{1}{g\left[\rho_2 - \rho_1 + s\left(\frac{\rho_1}{S_1} + \frac{\rho_2}{S_2}\right)\right]} = 2.2mm/Pa$$

Application 2

Containers A and B contain water at pressures of 2.80 bars and 1.40 bars, respectively. Calculate the difference in height h of the mercury of the differential pressure gauge. Let x + y = 2m and the density of mercury is d = 13.57



Applying the hydrostatic equation between points A and C gives:

$$P_C - P_A = \rho_{water} g(Z_C - Z_A)$$

$$P_C = P_A + \rho_{water} g(h + x)$$

The hydrostatic equation is then applied between points C and D:

$$P_D - P_C = \rho_{mercury} g(Z_D - Z_C)$$

$$P_D = P_C + d\rho_{water} g(-h)$$

$$P_D = P_A + \rho_{water} g(h + x) + d\rho_{water} g(-h)$$

$$P_D = P_A + \rho_{water} g(x) + h\rho_{water} g(1 - d)$$

Applying the hydrostatic equation between points D and B gives:

$$P_B - P_D = \rho_{water} g(Z_B - Z_D)$$

$$P_B = P_D + \rho_{water} gy$$

$$P_B = P_A + \rho_{water} g x + h\rho_{water} g(1-d) + \rho_{water} g y$$

$$P_B = P_A + \rho_{water} g(x+y) + h\rho_{water} g(1-d)$$

$$h = \frac{p_B - \left(p_A + \rho_{water} g(x+y)\right)}{\rho_{water} g(1-d)}$$

$$h = 1.272\ m$$

Application 3

The Figure below shows an open tank with two piezometer tubes. It is filled with two immiscible liquids, namely oil with density ρ_1=850 kg/m^3 over a height h_1= 6 m and water of density ρ_1=1000 kg/m^3 over a height h_2 = 5 m.

We designate by:

- A a point on the free surface of oil,
- B a point at the interface between the two liquids,
- C a point belonging to the bottom of the tank
- D and E the points representing the levels in the piezometer tubes,
- (O, $\vec{Z}$) is a vertical axis such that ZC = O.

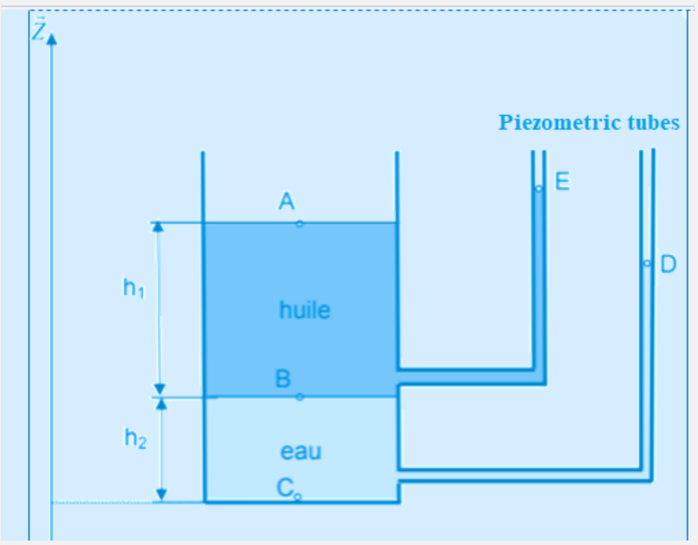
Apply the fundamental equation of hydrostatics (FEH) between the points:

1) B and A, and deduce the pressure PB (in bar) at point B.

2) A and E, and deduce the oil level ZE in the piezometer tube.

3) C and B, and deduce the pressure PC (in bar) at point C.

4) C and D, and deduce the water level ZD in the piezometer tube.



Answer :

Applying the fundamental equation of hydrostatics (FEH) between B and A gives:

$P_B - P_A = \rho_1 g(Z_A - Z_B)$ Or $P_A = P_{atm}$et $Z_A - Z_B = h_1$

Then: $P_B = P_{atm} + \rho_1 g h_1$

N.A.:

$$P_B = 10^5 + 850 \times 9.81 \times 6 = 150031Pa = 1.5\ bars$$

Applying FEH between A and E gives:

$P_A - P_E = \rho_1 g(Z_E - Z_A)$ Or $P_A = P_E = P_{atm}$

Therefore : $Z_E = Z_A = h_1 + h_2$ AN :$Z_E = Z_A = 6 + 5 = 11m$

Applying FEH between C and B gives:

$P_C - P_B = \rho_2 g(Z_B - Z_C)$ Or $Z_B - Z_C = h_2$

Then: $P_C = P_B + \rho_2 g h_2$

$$P_C = 150031 + 1000 \times 9.81 \times 5 = 199081Pa = 2\ bars$$

Applying FEH between C and D gives:

$P_C - P_D = \rho_2 g(Z_D - Z_C$ Or $P_D = P_{atm}\ and\ Z_C = 0$

Hence : $Z_D = \frac{P_C - P_{atm}}{\rho_2 g}$

$$Z_D = \frac{199081 - 10^5}{1000 \times 9.81} = 10.1m$$

Chapter II: Water Catchment for the Supply of Drinking Water

II.1 The Water Cycle - Introduction

The natural water cycle begins with precipitation in which water drops as rain or snow. This water then flows through streams, rivers and lakes, forming fresh water reserves. Part of this water infiltrates the ground to replenish groundwater, while the other part evaporates from land surfaces and bodies of water and is transported into the atmosphere in the form of water vapor. It should be noted that this evaporation process is also influenced by evapotranspiration since the water released by plants through transpiration evaporates into the atmosphere. Thus, the natural water cycle is completed since the water will return to the atmosphere to form new precipitation. In addition to these processes, the natural water cycle also includes percolation where water seeps into the ground and recharges groundwater. Water interception occurs when it is held by vegetation or other surfaces before falling to the ground, thereby delaying its flow. Sublimation is the process by which ice or snow goes directly from a solid state to a water vapor state without first melting. Finally, condensation occurs when water vapor in the atmosphere cools and condenses into water droplets or ice crystals, forming clouds (**Figure 6).**

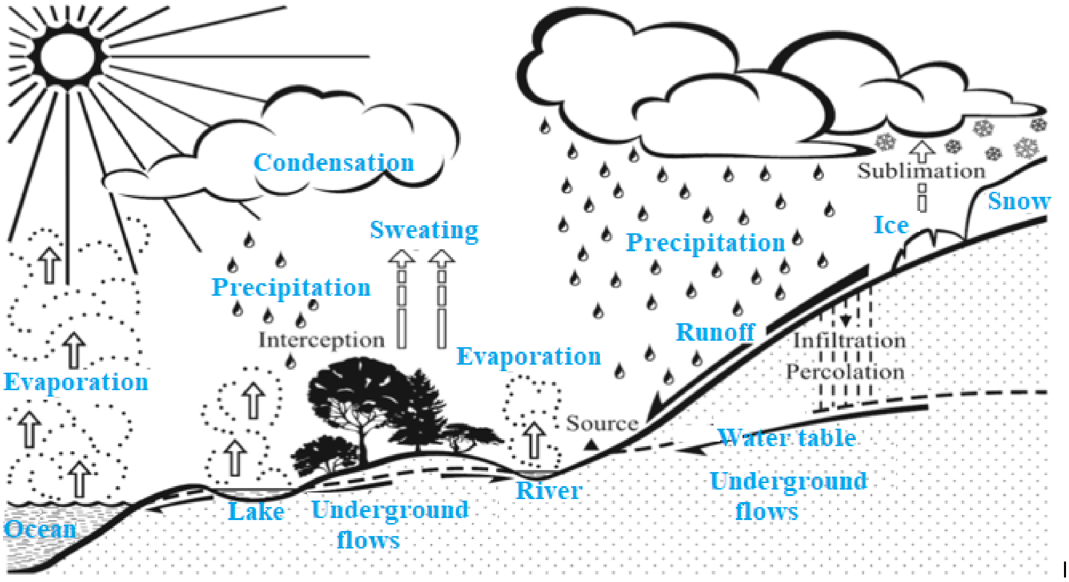


Figure 6: Water cycle (Anctil et al., 2012)

II.2 Urban water cycle

Water is a vital resource. Its cycle plays a crucial role in maintaining life on Earth. In urban environments, this cycle must be controlled very carefully in order to meet the drinking water needs and efficiently dispose of wastewater. The urban water cycle begins with the establishment of a catchment station whose role is to take water from natural sources such as a river, a water table or others (**Figure 7**). Then, this water is transported through conveyance systems to raw water storage tanks for further treatment to remove impurities and make it potable and fit for human consumption. After treatment, the water is then stored in reservoirs before it is distributed through a distribution network for use by urban residents. Once used, the water is collected via a drainage network and directed to wastewater treatment plants, where it is cleaned before being released into the natural environment. This closed process ensures the continued availability of drinking water while preserving the health of aquatic ecosystems in urban environments.

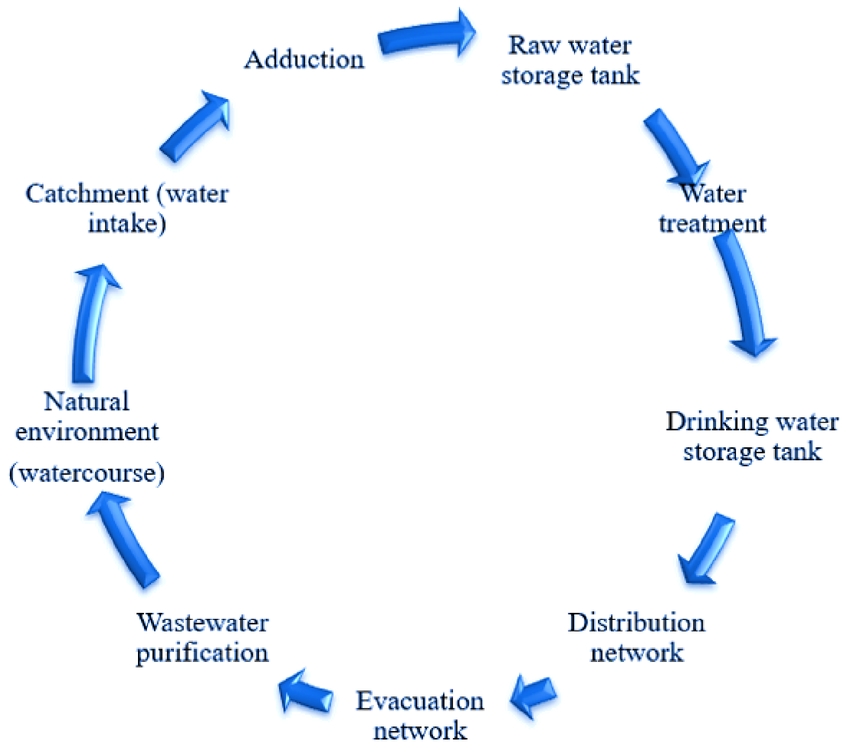


Figure 7: Water Route - From Nature to Consumption, Treatment and Return to Source

Drinking water is generally collected from surface water sources such as lakes or rivers. This process begins with the capture and conveyance of water, using a water intake and a conveyance pipe, to a reservoir which receives the water in its raw state. Then, this water is regularly and continuously drawn to be directed to a purification station. In the event of unavailability of water or heavy pollution of surface water sources, often subject to contamination and seasonal fluctuations, it is necessary to draw water from groundwater through wells or boreholes.

II.2 Water catchment

II.2.1 Collection of surface water

The collection of water from rivers or lakes is often preferred, particularly to supply large cities located near a river. It is essential to first check that the low flow of the river, that is to say the minimum flow during low water periods, is sufficient to meet the water needs.

The water of lakes or reservoir dams has the same composition as the water of the watercourses that feed them. Its composition is similar to that of the rocks constituting the basin of the lake. However, it should be noted that the lake acts as a natural settling basin, so that the water, although often contaminated, is less contaminated towards the outlet of the lake than towards the entrance. Therefore, it is better to draw water from lakes, avoiding areas close to the banks and choosing points located a few meters both from the bottom and from the minimum level of the free surface, i.e. approximately halfway up. It is important to note that convection currents can disrupt the water settling process, depending on the season. Often, the presence of plankton makes the water cloudy.

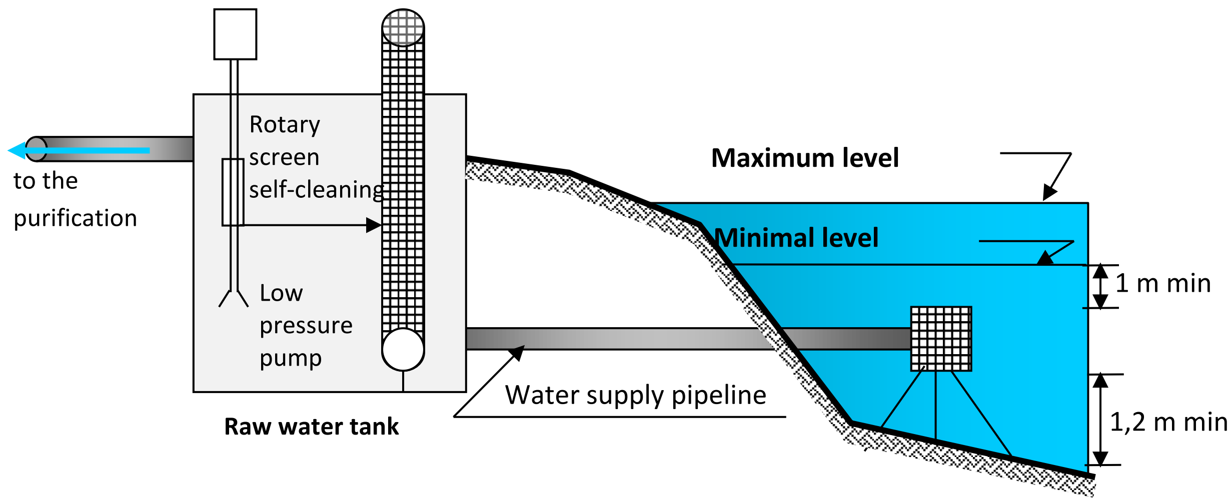


Figure 8 : Surface water intake, supply pipe and raw water tank

II.2.1.1 River catchment

Catchment is done by a water intake, a structure designed specifically to capture natural or raw water from the lake or river where it is submerged. This structure must be installed in an area where the raw water collected is of the best possible quality, while avoiding any potential source of pollution. It is preferable to select the collection point upstream of the main sources of pollution such as towns or unsanitary industrial installations. The main challenges are to avoid the entry of solid or floating materials into the water intake. This can be built on a bank, in the river bed, under the river bed or in the alluvium constituting the bank, at an appropriate distance from the watercourse.

II.2.1.1.1 Reservoir catchment

The water intake is usually placed at a reasonable distance from the bank to avoid stagnant water that is often contaminated near the banks.

The type of device to use actually depends on several factors, including the flow rate required and the expected duration of the work.

For a summary water intake (see **Figure 9**):

1. The flow rate required is simple,
2. The installation is temporary,
3. The installation is quick.

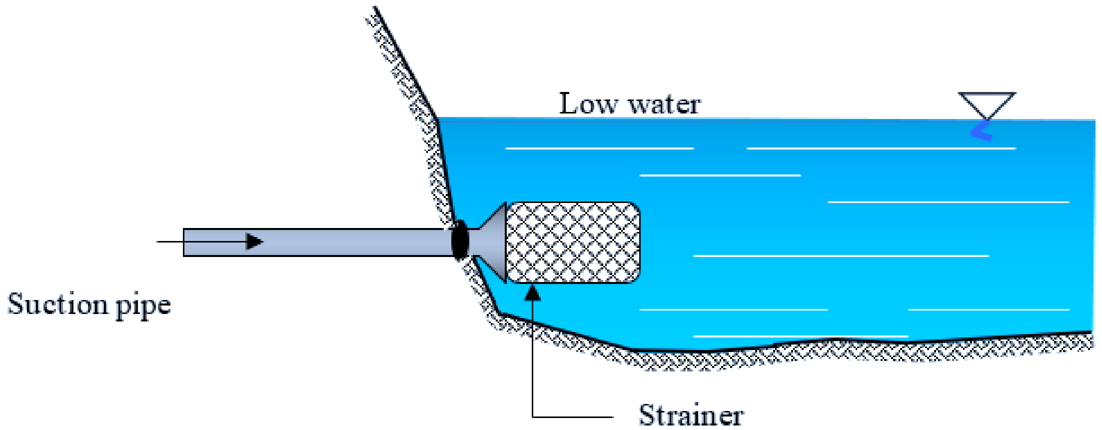


Figure 9: Water intake from the reservoir

Other devices for permanent installations but at moderate flow (**Figures 10**, **11** and **12**)

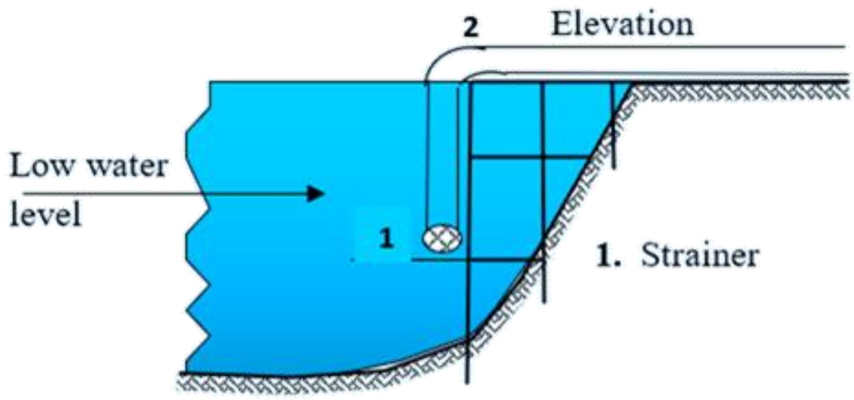


Figure 10: Water intake from the pier

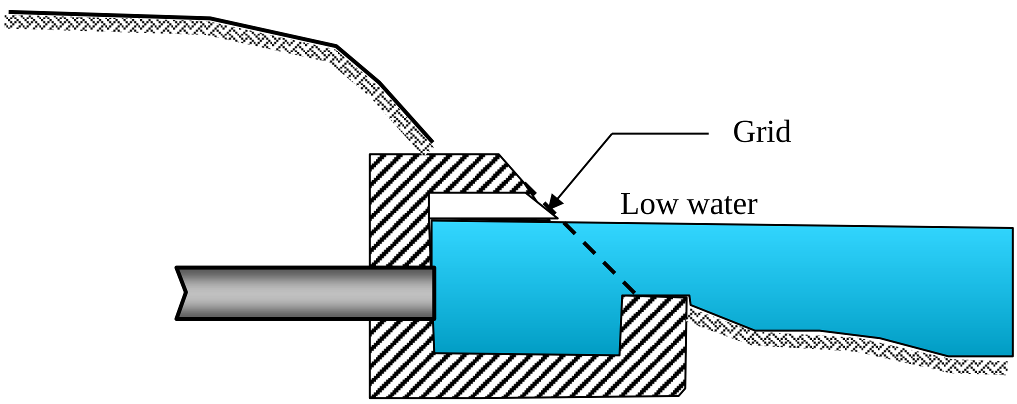


Figure 11: Water intake from a masonry bank

This sump must be dug to an appropriate depth so that the suction strainer always remains at least 0.80 m below the water table surface in order to avoid any risk of unpriming of the pump which can be caused by the drawdown of the water table. In addition, the strainer must be positioned at least 0.80 m from the bottom to avoid sucking up the settled sludge. These preventive measures apply to all water intakes located along the river.

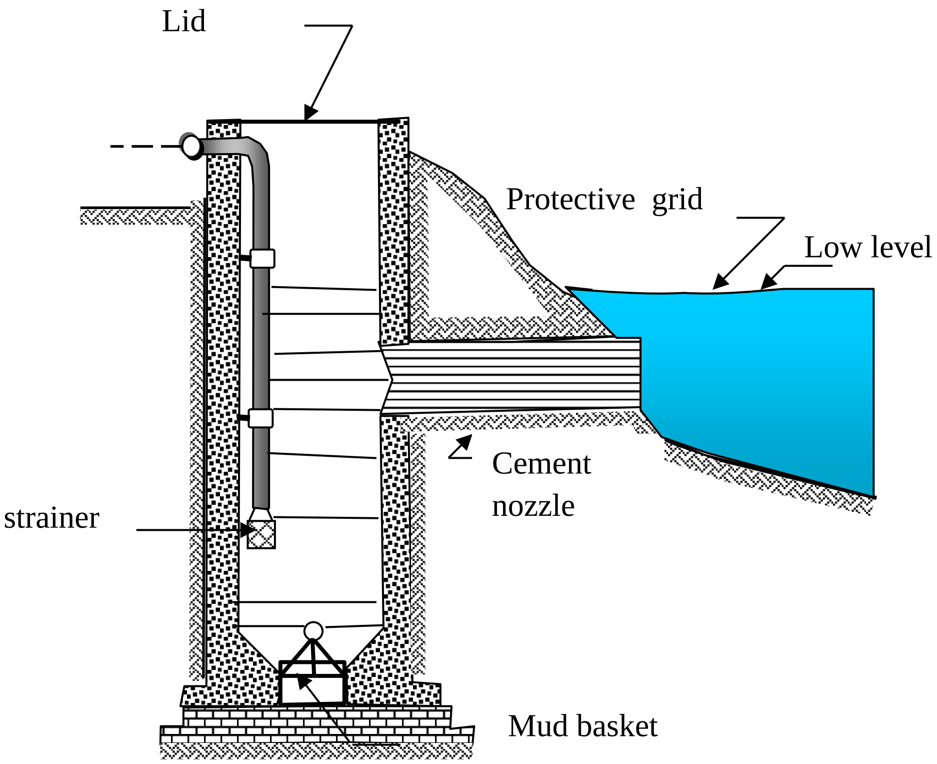


Figure 12: Water intake from bank through a sump

II.2.1.1.2 Free water intake

The water intake can be performed either at the bottom of the river bed or between two water levels. In this case, it is crucial to take into account low water levels. The devices used here are particularly suitable for slow-moving rivers (**Figures 13** and **14**).

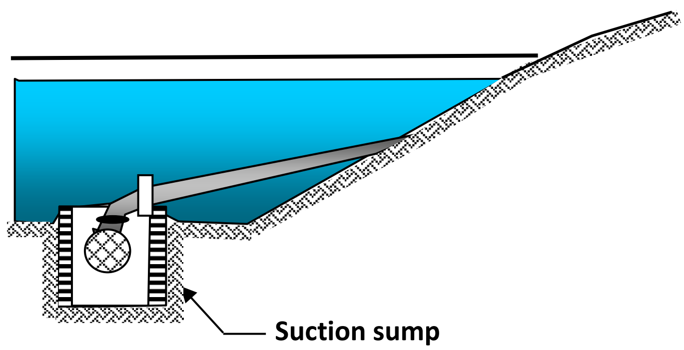


Figure 13: Water intake from the bottom

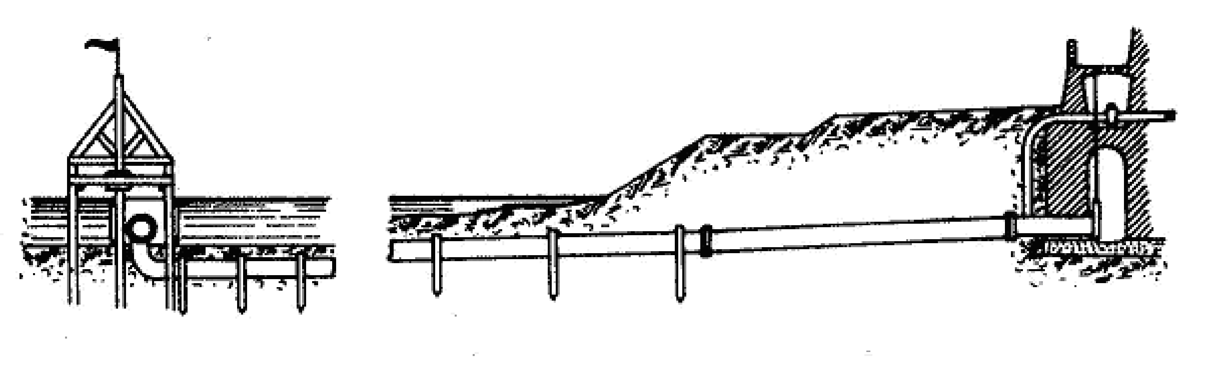


Figure 14: Water intake from the middle of the current

II.2.1.1.3 Water intake from below the bottom

A trench perpendicular to the flow must be dug in the river bed if the catchment is carried out at the bottom of the river. A suction strainer is then placed in this trench and connected to the bank by pipes. Then, the trench is filled with gravel which protects the strainer while allowing the passage of water. This capture method is suitable for rivers whose flow is neither too slow nor too fast. Indeed, slow-moving watercourses can deposit fine elements which risk clogging the intake structure, while fast-moving rivers and torrents can scour it or degrade it quickly.

This process is used for torrential rivers (**Figure 15**).

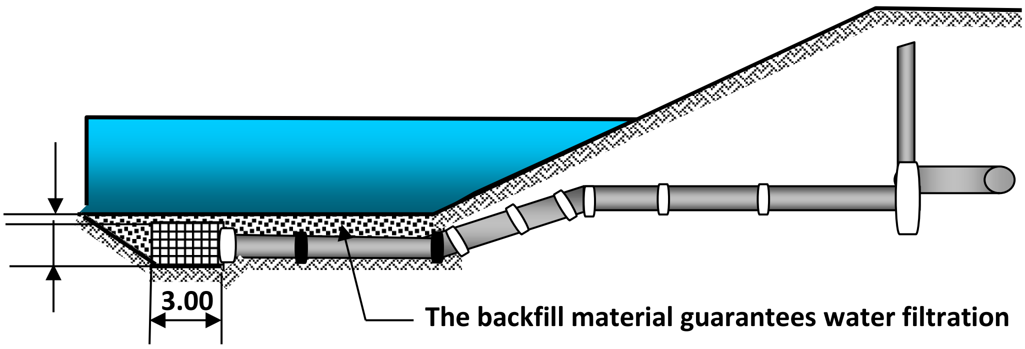


Figure 15: Fast-flowing River water intake

It is also possible to build chambers buried at the bottom of the river for water intake.

II.1.1.1.4 Water intake from alluvia

When the banks are formed of permeable alluvium, it is possible to capture the water from the groundwater which forms there once it has passed through the ground and undergone significant filtration. This filtration generally gives the water good clarity, which can help simplify the treatment procedures.

The catchment device may consist of a well or a series of wells aligned along the bank of the river to avoid interference. Alternatively, this series of wells can be replaced by a collection gallery, parallel to the bank. However, the construction of such a gallery is generally more complex and therefore more expensive.

II.2.1.2 Water collection from a lake

Since lakes act as settling ponds, their water is often of better quality than that of rivers. In addition, the quality of lake water usually does not change much as compared to that of rivers. However, in deep lakes, it is common for water quality to vary significantly with depth. So, in summer, the first meter of water below the surface may contain algae that can disrupt the water purification process due to bad tastes. Algae may sometimes clog filters. On the other hand, near the bottom, the water is cold and devoid of dissolved oxygen, which can lead to the dissolution of certain metals present in the bottom sludge that is formed by deposits of various materials, thus making the water less suitable to purification. Additionally, in deep lakes, thermal stratification favors seasonal overturning. It should also be noted that the observations made on the behavior of the water accumulated behind a dam are also valid for that of the water of a lake. It is worth noting that this water differs from that carried by a river by two main characteristics, namely temperature stratification and water composition stratification. The density of water reaches its maximum value at the temperature of 4°C. Between 0°C and 4°C, the density increases, but beyond 4°C, it decreases with temperature.

In early spring, just after the snow melts, the surface water temperature is 0°C, while the deeper layers are at 4°C because they are denser at that temperature. It is important to specify that the temperature does not vary linearly from 0 to 4°C between the surface and the bottom. On the contrary, it forms distinct layers with sometimes significant temperature differences between them. Additionally, although the temperature generally remains constant, or varies linearly within a layer, this thermal stratification creates significant variation in temperature at this time of year.

In spring, as the surface water warms, movement occurs in the water mass until all the water reaches a temperature of 4°C. When the surface water goes from 0 to 4°C, it becomes denser and sinks. Consequently, at this stage, the slightest breath of wind can agitate the whole which, in this case, can be considered as being in an unstable equilibrium.

As summer arrives, the surface water warms up considerably. However, this heat has difficulty penetrating deep, which causes a sudden variation in the temperature curve at a certain distance below the surface (**Figure 16**). This distance varies depending on the transparency of the water. In temperate zones, it is generally around ten meters.

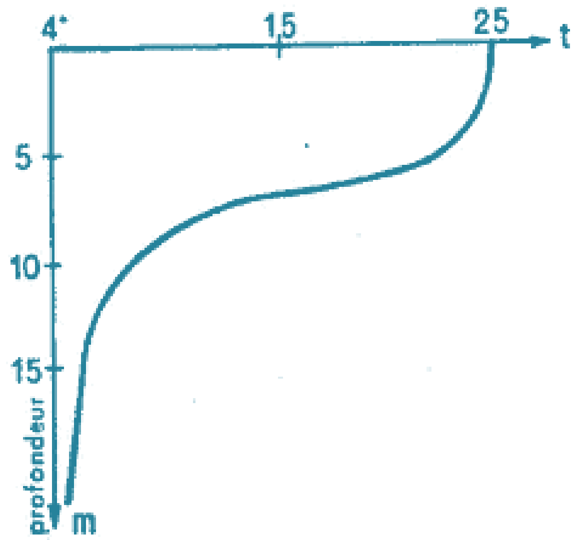


Figure 16: In summer, the temperature away from the surface decreases abruptly

Two layers, presenting very different temperatures, can thus be distinguished. The upper layer, where the water experiences large temperature variations, is called the epilimnion, while the lower layer is called the hypolimnion. The separation surface of the two layers is called the thermocline. The thermal stratification persists throughout the summer season.

In the fall season, the cooling surface waters become denser and sink. At the same time, warmer, lighter water from the bottom rises to the surface, carrying with it the polluted sludge from the bottom. A new thermal equilibrium is then established at 4°C, similar to the one observed in spring, with the resulting instability.

Beside this temperature stratification, there is also a stratification of the water composition (**Figure 17**). It should be noted that the fauna and flora intervene differently depending on the depth. On the surface, in the illuminated layers, the flora releases oxygen and consumes CO_2, while at depth, fish and bacteria consume oxygen and release CO_2, which leads to the stratification of the composition of water. Indeed, the illuminated layer, over a variable thickness, is well oxygenated and relatively purified, while the deep zone is rich in organic matter and CO_2, but can be very poor in oxygen.

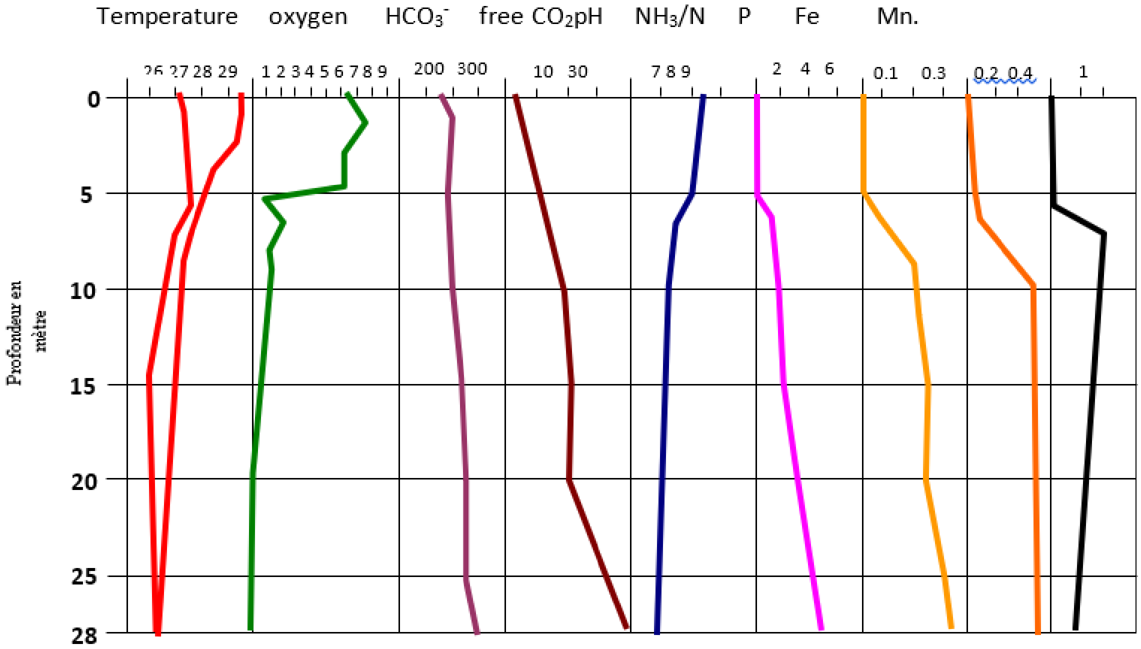


Figure 17 : Variation of water quality with depth in a deep lake (Indonesia)

Designing the water intake for a dam may require the installation of a tower on the height of the dike, with water intakes spaced approximately 1.50 meters apart and equipped with wall valves operable from the top (**Figure 18**). This configuration allows the operator to freely choose the most suitable slice of water, depending on the seasons.

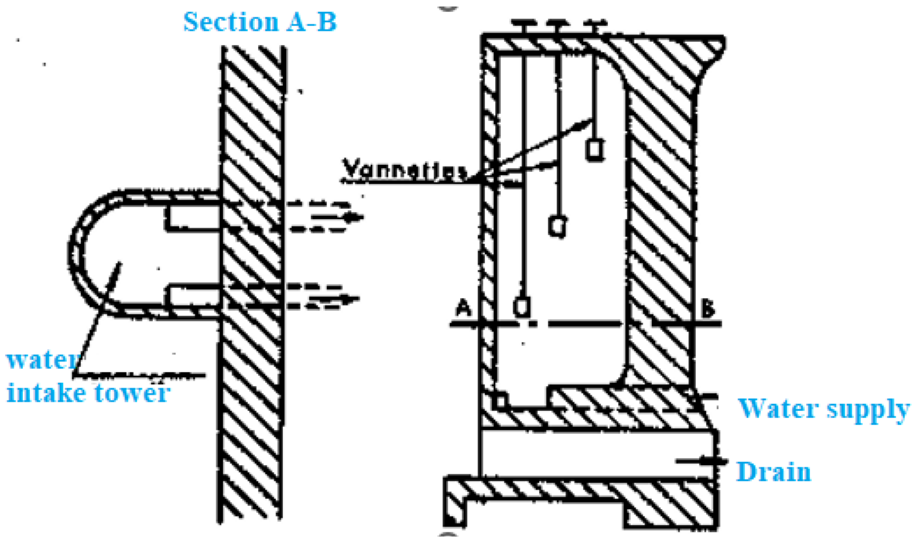


Figure 18: Cross section of the dam intake

It is useful to add the following points:

1. No eutrophication problem is to be feared if the reservoir is partially, or even better completely, emptied at least once a year.

2. No deterioration of water quality is to be feared if the watercourse feeding the lake does not receive any discharge of waste or residual water, in accordance with the requirements of the circular mentioned above.

3. It is highly recommended to maintain a retention depth of around 10 to 20 meters in order to guarantee an adequate level of oxygenation of the dam water.

If it is impossible to create such a reservoir and if the option is towards an artificial lake, non-curable or difficult to cure, it is advisable to carry out a limnological, ecological and biological study of the biotope which will develop.

It is worth indicating that the above considerations highlight the challenges that operators may face when they plan to draw water from a natural lake, although, initially, this option was more attractive. Natural lakes may have eutrophication issues, which may require a carefully planned approach. For a sufficiently deep lake, the best conditions seem to be achieved by taking the sample far from the banks, about thirty meters below the surface, by means of a pipe laid, in dredged soil when possible, or directly on the bottom. It is essential that the sampling point is located at a sufficient distance, generally 5 to 6 meters above the bottom, to avoid potential problems. The pipe thus installed then rises in a pipe at this place and is held in that position by a metal boom.

II.2.2 Groundwater capture

It should be highlighted that the use of groundwater is often considered in the absence of a sufficient quantity of water or if it is not of good quality. Groundwater is generally preferred because of its freshness and its chemical and bacteriological qualities. When water characteristics remain relatively constant during multiple analyses, this suggests slow water circulation in environments where there are limited exchanges with the outside world. This also attests to the absence of direct introduction of foreign surface water or other water into the underground reservoir.

These waters can be captured in several ways:

1. Directly from their natural source, such as springs.

2. Inside the water table, when embedded in loose elements, such as sand and gravel.

3. In the deposit, for water circulating in fissured grounds.

4. It is sometimes necessary to capture water circulating at great depth.

It should also be noted that the water collection methods vary depending on the circumstances. However, it is essential to respect the principle of drawing water from a sufficient depth in its geological deposit and to carry out the collection operation in such a way as to preserve the water from contamination, particularly when it is close to the ground surface.

II.2.2.1 Circulation of water in the ground

Water from precipitation does not flow entirely to the surface since some of it infiltrates through permeable soils until it encounters an impermeable layer on which it accumulates to form what is called a groundwater table.

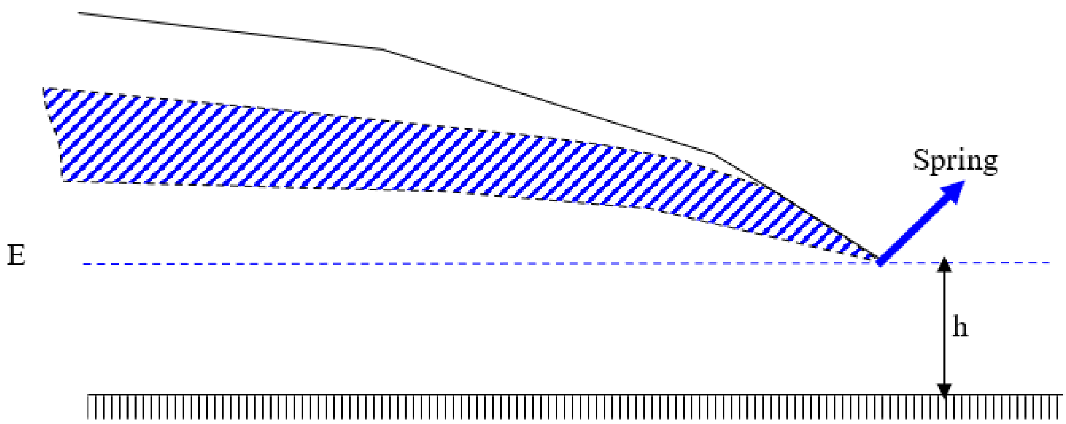


Figure 19 : Different parts of a body of water

This water flows slowly towards a more or less distant outlet. When the free surface of the water table reaches the surface of the ground, a source is then formed. In the figure, the lowest load line, denoted N', represents the level at which the water table is still flush with the ground surface. Between this line and the maximum line N, the water contained in the hatched area can normally be exploited. Below this level, the water source becomes depleted. The part of the water table, which is above E, is called an active water table, while the lower part is called a passive water table (**Figure 19**).

When underground flow occurs near a river, the latter can be fed by the water table, but the variations in level of each water table or river can reverse this flow. In particular, during a rapid flood of the river or during a prolonged drought which affects the level of the water table more than that of the river, the water table then becomes fed rather by the river.

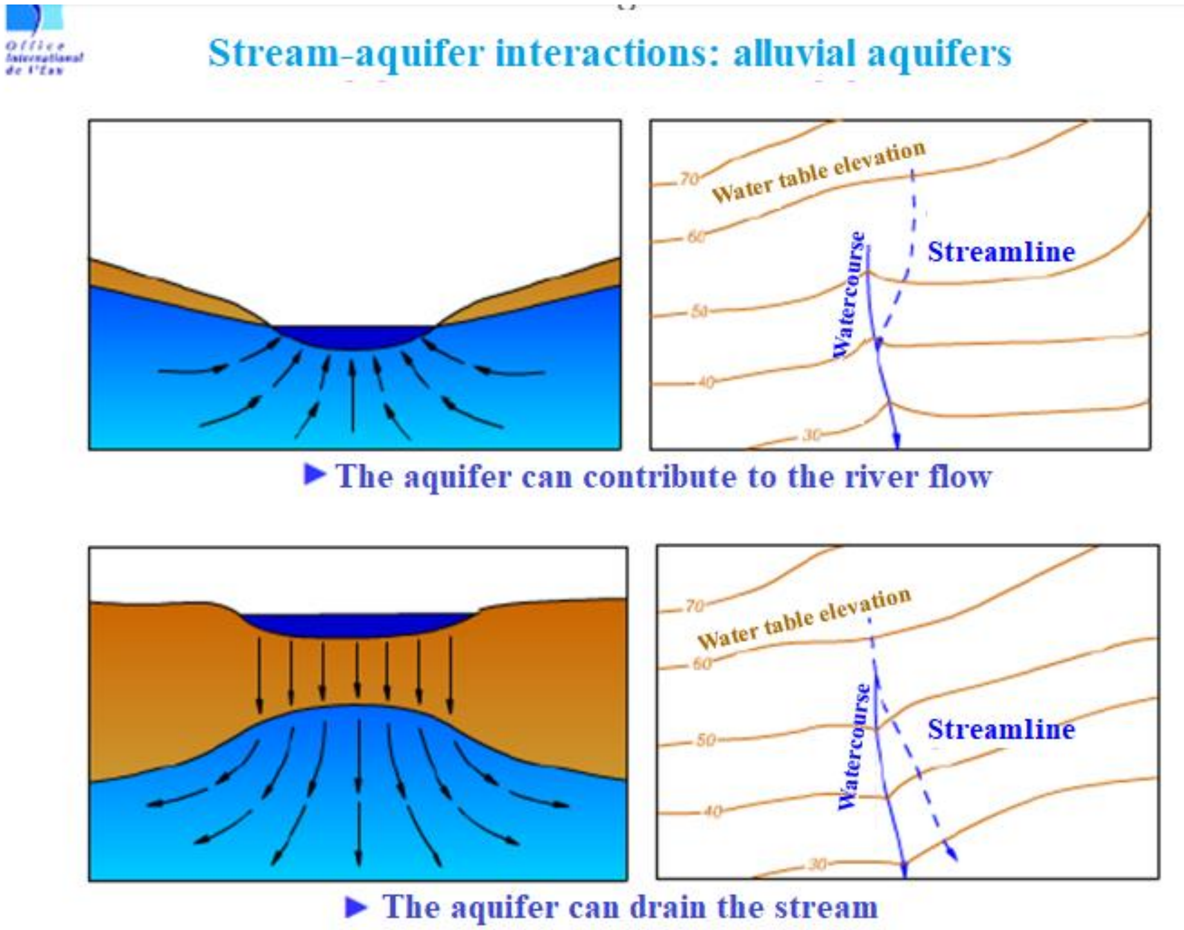


Figure 20: *Normal case*: The aquifer feeds the river ; *Case of drought or flood* : The river feeds the water table.

When the flow lines of the aquifer reach the watercourse, the aquifer feeds the river, which is observed during periods of high water (winter). On the other hand, when these flow lines flow from the river towards the aquifer, the river feeds the aquifer, which occurs during periods of low water (**Figure 20**).

II.2.2.2 The main types of groundwater

Groundwater is neither lakes nor underground watercourses. They contain water which fills the pores or cracks of the rocks saturated by infiltrated rainwater (AEAG, 2012). It should be noted that the porosity and structure of the soil determine the type of water table and the way in which water circulates underground. The term *Aquifer* or *Water table* simply refers to a layer of soil that contains water and can be used as a source of water (Arjen, 2010).

There are several types of aquifers depending on the characteristics of the rock reservoirs and the nature of the substrate.

II.2.2.2.1 Free water table - When the permeable substrate containing an aquifer is not covered by an impermeable layer, the water table can develop freely upwards. In this case, it is a free water table. Free water tables are common, especially in porous rock formations, such as sand, chalk or limestone, which have generally retained their permeability all the way to the ground surface. They are fed directly through the infiltration of runoff water (**Figure 21**). The waters of these aquifers are not kept under pressure by a layer that is less permeable than the formation that contains these waters (Arjen, 2010).

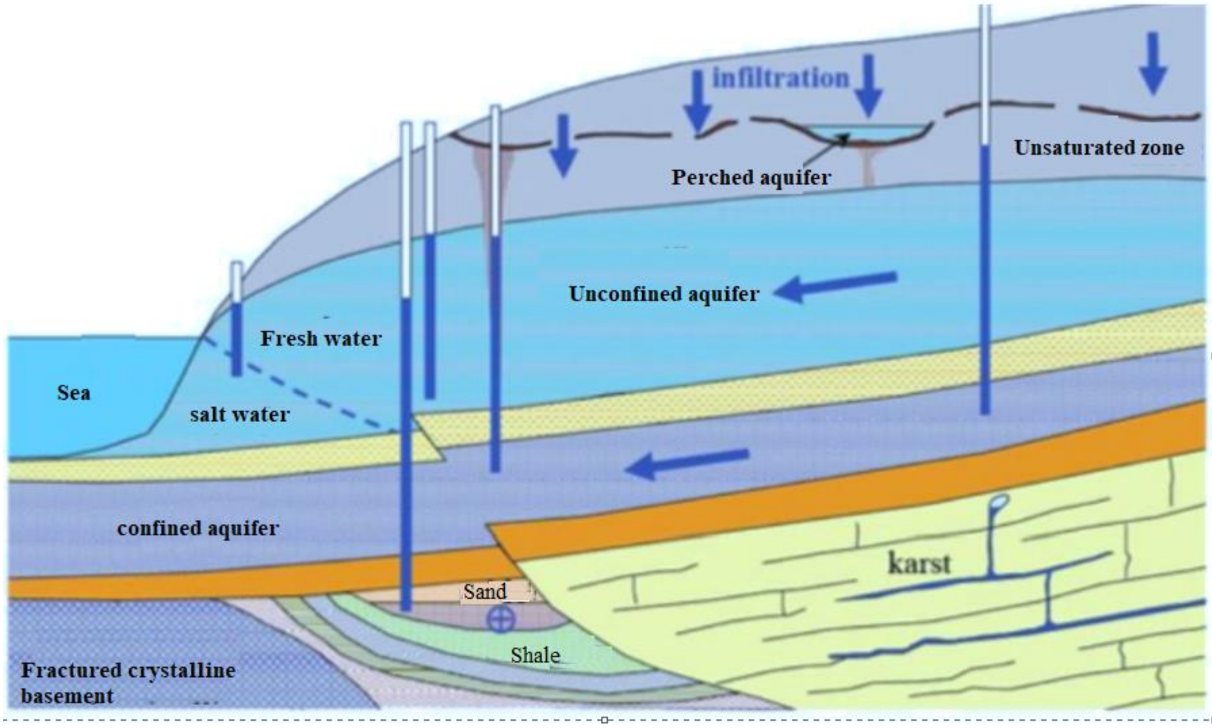


Figure 21: Different types of aquifers (Source: KESSASRA, 2016)

II.2.2.2.2 Captive water table:

When the permeable layer is enclosed between two impermeable layers, the water table cannot expand freely upwards; it is a captive water table. Sometimes, it can even be located below the ground surface locally, thus forming an artesian well, fed only by a captive aquifer. Confined aquifers are often deep and sometimes even very deep (more than 1000 meters). These layers are isolated from the ground surface by an impermeable layer (**Figure 21**). They are therefore not directly supplied through the ground and are found at significant depths. This makes these aquifers less sensitive to pollution (Degremont, 2005).

II.2.2.2.3 Groundwater:

Groundwater lies on the first impermeable layer, not far from the ground surface. They are always free and often contaminated. These aquifers are crucial for water supplies, as they constitute the main reserves of drinking water in many parts of the world. Sometimes, a water table can emerge on the surface of the ground, in the form of a source, as it can also be exploited by means of wells (Arjen, 2010).

II.2.2.2.4 Karst water table:

Karst water tables develop in limestone formations. Waters dissolve limestone through pre-existing cracks, creating voids into which water can flow. These voids can reach considerable dimensions, such as chasms and caverns. In these conduits, water can circulate quickly and form underground watercourses (**Figure 21**).

II.2.2.2.5 Alluvial water table:

The alluvial water table forms in the vast expanses of sand, gravel, and pebbles of rivers. It constitutes a crucial point of exchange with watercourses and wetlands. This type of aquifer can be fed by floods and, conversely, return water to rivers during periods of drought (**Figure 21**).

II.2.2.2.6 Perched water table:

It is a free underground water table, permanent or not, generally of modest dimensions, located above an unsaturated zone. The level of perched aquifers can thus be higher than the surface of neighboring watercourses, as shown in Figure 21. The presence of perched aquifers is often indicated by the presence of springs (Wikhydro, 2021).

II.2.2.3 Spring water collection

Springs are mainly found in mountainous regions and hilly areas (**Figure 22**). A spring can be defined as a place where groundwater naturally flows to the surface. Typically, a spring is fed by a formation of sand or gravel that holds water, called an aquifer, or by flow through fissured rock. When compacted or clayey soil layers prevent underground flow, water can be pushed back to appear on the surface. This is how it manifests itself in the open air.

The quality of water from a spring can be highly variable depending on a variety of factors, including the type of groundwater flow, the length of time it stays underground, the geological composition of the rocks it passes through, and weather conditions. All of these factors can cause variations in water quality over time.

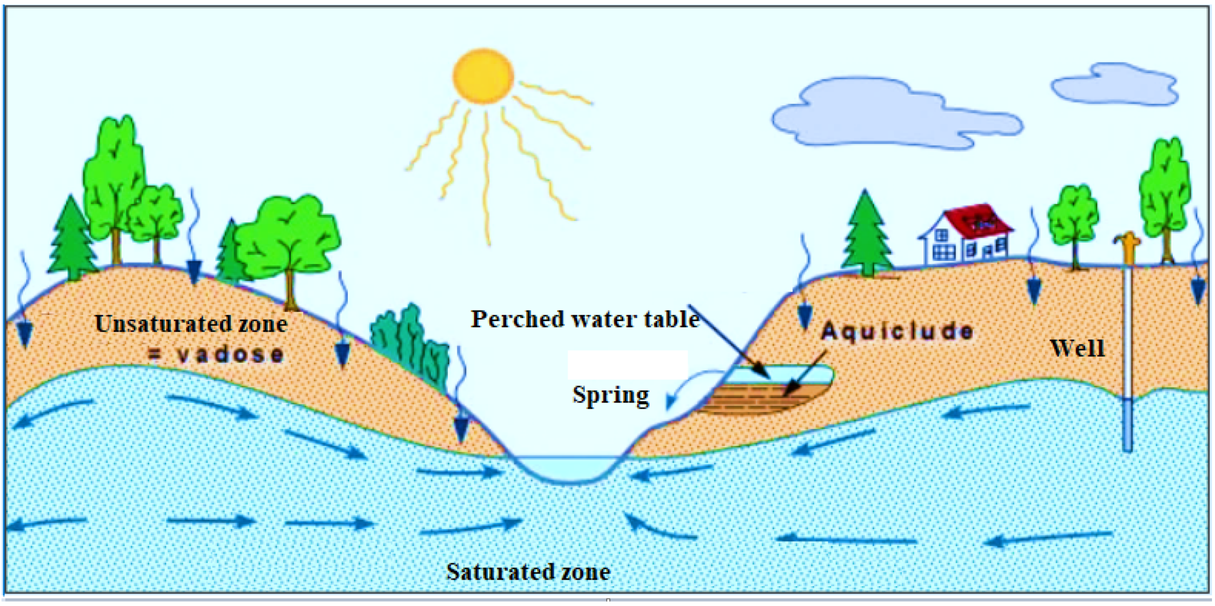


Figure 22: Appearance of water springs

(http://www2.ggl.ulaval.ca/personnel/bourque/s3/eaux.souterraines.html)

II.2.2.3.1 Emerging water springs

The first step consists of removing the cover soil in order to expose and clearly characterize the crack(s) through which water accesses the surface. To collect this water in its geological site, it is important that the sampling structure completely crosses the alluvium and is embedded deep enough in the layer containing the water table whose source is an emergence, so as to avoid any mixing of the water collected with the waters that flow in the alluvium (**Figure 23**).

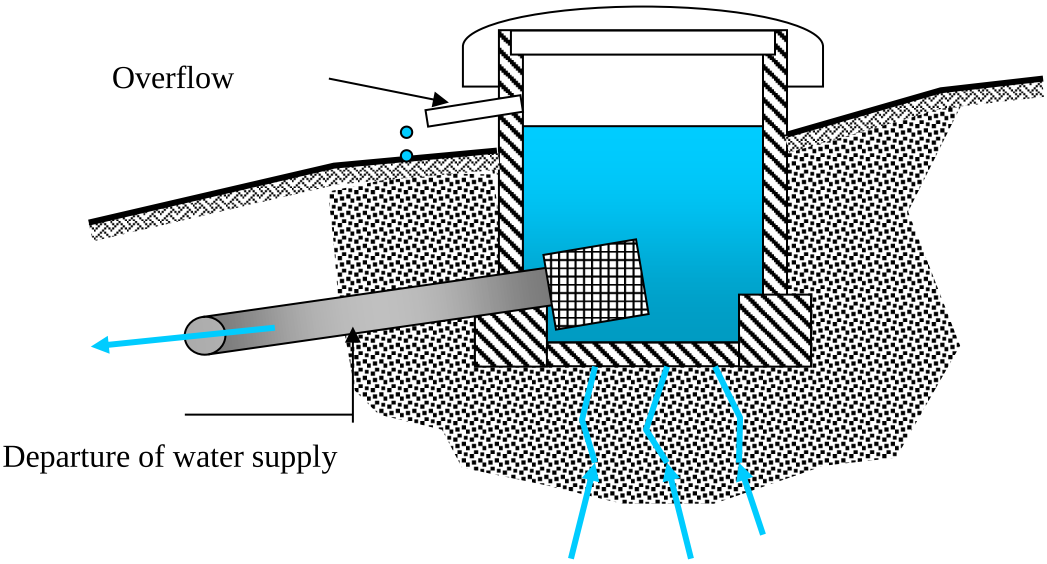


Figure 23: Water intake process in emerging springs

II.2.2.3.2 Outcrop and spill springs

These springs are generally formed by several streams which, after having grouped together, appear in a more or less deep cavity, often on the side of a hill. The process of water harvesting involves clearing these streams, collecting their waters using a branched system, pooling them, and transferring them to a strategically located receiving chamber (**Figure 24**).

This structure is in the form of a network comprising stones, drains, aqueducts and galleries that are structured into a draining system that follows the general direction of the streams, and into an intersecting system that is perpendicular to the first one and which intercepts the streams escaping the first system. This configuration makes it possible to extract the maximum water flow from the land.

Different draining systems are then used, depending on the flow rate:

Stones: Bleedings practiced along streams,

Drains: Used in the same way as stones.

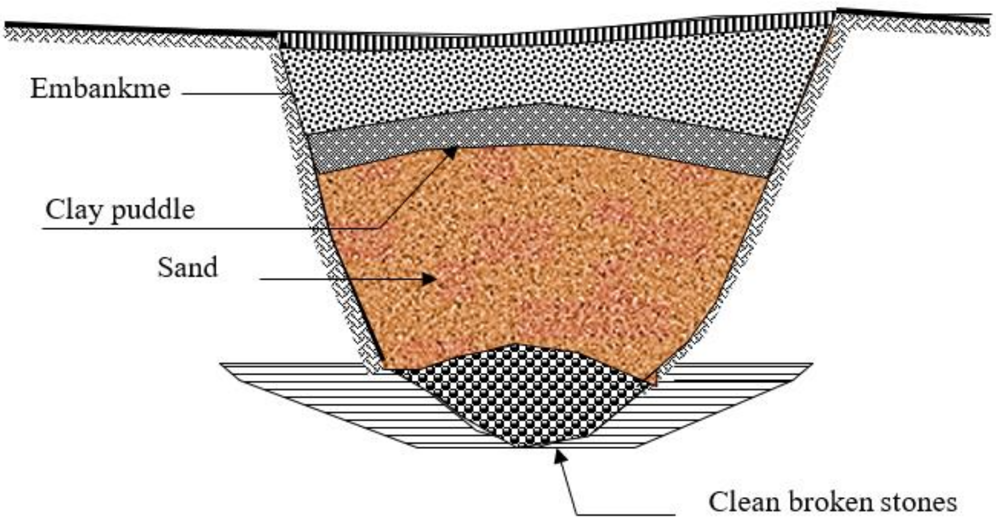


Figure 24: Process of Water Withdrawal from Outcrop and Discharge Sources

II.2.2.4 Groundwater collection

II.2.2.4.1 Groundwater collection in sand and gravel

We will explore some water withdrawal techniques within the water table itself, using:

a. Vertical wells

b. Horizontal drain wells

c. Radiant drain wells

Vertical wells

The energy required in this case is expensive if the diameters of the wells are large. Indeed, frictions increase when the diameter of the well gets larger, which means that more energy is needed. In practice, the useful diameter of the well, which is not to be confused with the internal diameter, corresponds to that of the excavation in the aquifer formation. It generally varies between 1.80 m for wells located in alluvium and 2.50 m for those in fine formations such as dune sands.

Horizontal drain wells

Horizontal drains are water intake structures established within the water table, following a slightly inclined profile towards a a waterproof end equipped with pumping devices. The length of these drains depends on the flow rate to be extracted. It should be noted that it is not uncommon to find drains that exceed 100 meters. However, drains are not suitable for all water tables. For practical purposes, the water table must be relatively close to the ground surface. In addition, drains must always be submerged to ensure their permanent effectiveness.

This requirement assumes a powerful and stable water table in terms of level. These two conditions are often encountered only in the alluvial layers of large rivers. Conceptually, drains are similar to catchment wells since they have a catchment wall surrounded by gravel. They can be simply collecting or both collecting and accessible. In the latter case, they are structures of significant importance, intended mainly for the water supply of large cities. Drain dimensions may generally vary between 0.80 and 1.50 meters.

Radiant drain wells

An innovative method mainly consists of capturing water using horizontal drains, starting from a vertical well, which is not a collecting well, but serves as a collector for water coming from the drains (**Figure 25).** The pumping station is installed directly above the well, with all the necessary measures to prevent water contamination.

The central well is made of reinforced concrete and has an internal diameter of approximately 4.00 meters, with a thickness of approximately 0.45 m. For greater depths, more imposing structures, with a diameter of up to 6.00 meters and a thickness of approximately 0.50 m, may be required.

The length and number of drains depend on the power of the water table and on the water needs to be met as well. The length of the drains can go up to approximately 80.00 meters. Until now, this type of installation remains relatively uncommon. Furthermore, its relatively high costs seem to be one of the main reasons for this situation.

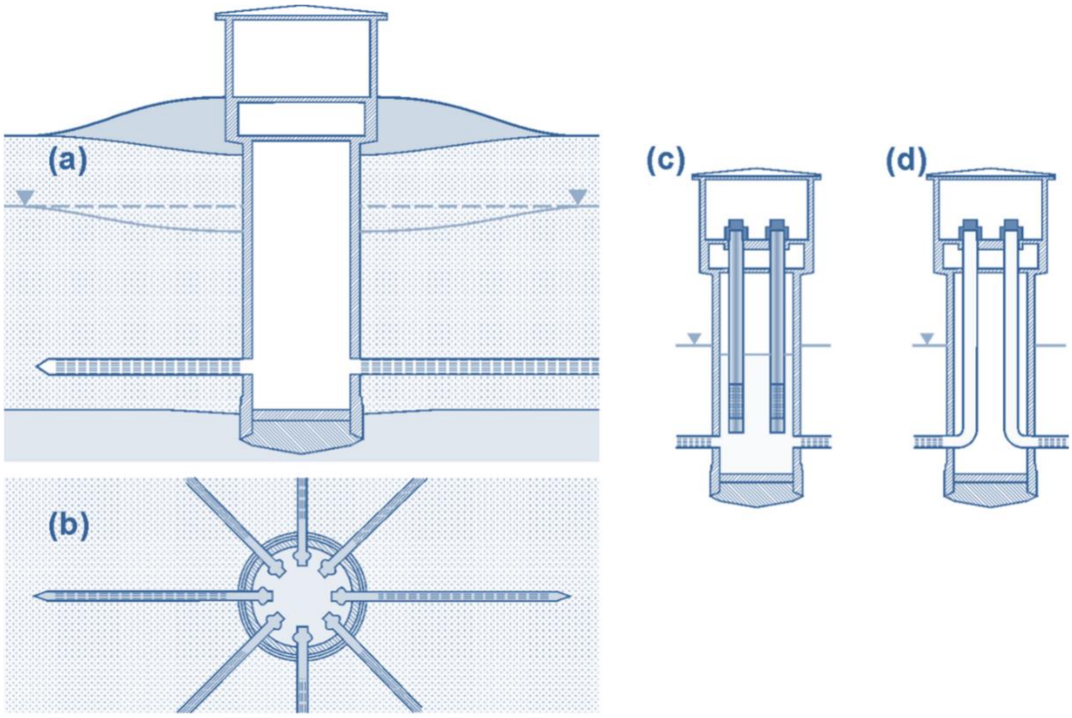


Figure 25: Typical set-up of a radial collector well: (a) Cross section, (b) Plan view, (c) Wet shaft and (d) Dry shaft (modified after Houben and Treskatis, 2007) (source: Houben *et al.*, 2022).

II.2.2.4.2 Collection of deep waters

Access to deep water is generally achieved through drilling, an artificial technique involving the creation of a circular hole. This hole, generally of modest diameter to limit the quantity of excavation, is often dug vertically, although oblique or sinuous surveys can be carried out depending on geological requirements. The diameters of the boreholes commonly vary from a few centimeters to a few tens of centimeters. However, they can exceptionally reach up to one meter. In terms of depth, drilling can extend over several thousand meters.

II.1.2.4.3 Use of Brackish Water and Sea Water as Alternative Solutions in Case of Water Shortage

When surface and groundwater resources are unavailable or insufficient to meet the water needs, an alternative solution is to use brackish water or seawater. The salinity of brackish water is higher than that of fresh water but lower than that of sea water. Brackish water is generally found in areas where fresh water mixes with sea water, such as estuaries or coastal areas.

The use of brackish water often requires specific desalination or treatment technologies that render this water potable or suitable for specific uses. You should know that salinity can affect the quality of water and make it unsuitable for human consumption or for certain agricultural or industrial activities.

As for seawater, it is widely available in coastal regions and represents an abundant resource. However, due to the high salinity of sea waters, they require more advanced desalination processes in order to make them usable for human consumption or other uses. These desalination processes can be expensive and energy-intensive, but they offer a viable solution when other water sources are limited or unavailable.

Algeria has opted for seawater desalination to guarantee the supply of drinking water in coastal regions, thus exploiting its 1 600 km of coastline. This strategic choice aims to reduce dependence on precipitation for the supply of drinking water, a major challenge in the face of population growth. This initiative is of particular importance in the west of the country, where the rainfall deficit is particularly worrying (**Figure 26** and **27**).

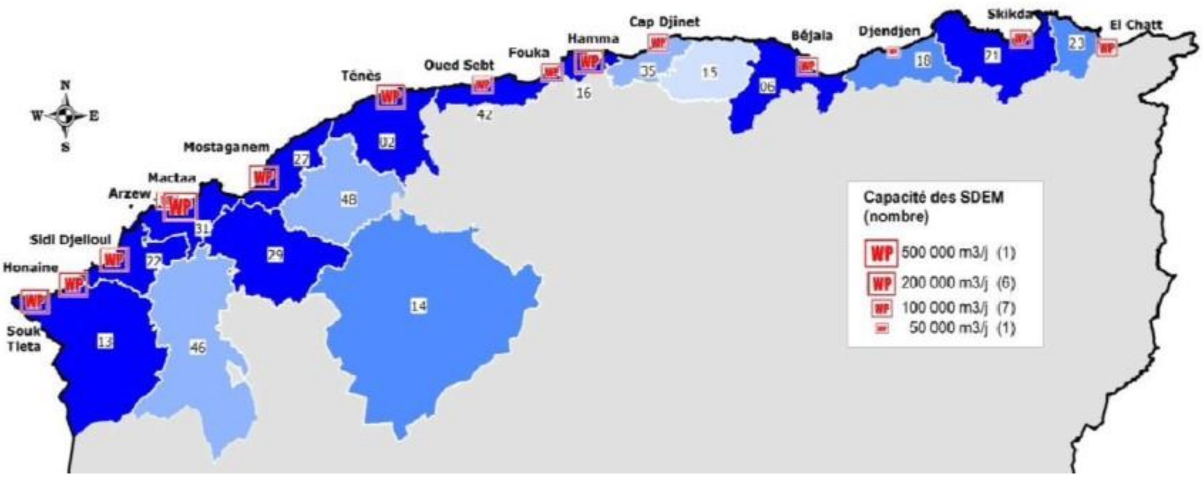


Figure 26: Distribution of seawater desalination stations in Algeria (Source: Djaffar Sabrina & Kettab Ahmed)

Figure 27 Example of a desalination station (source: https://lecarrefourdalgerie.dz/stations-de-dessalement-de-leau-de-mer-unique-solution-face-aux-perturbations/)

II.3 Adduction pipe

The water supply pipe, a crucial element of the system, establishes a vital link between the water intake and the raw water tank (**Figure 8**). This infrastructure allows water to move under the force of gravity, without the need for pumps, ensuring its smooth and natural circulation. The diameter of the conveyance pipe must be carefully sized to maintain the water flow velocities within the recommended limits, as shown in **Table 1**.

Table 1: Water flow speeds in water supply pipes

Criterion	**Speed (m/s)**	
	Min.	**Max.**
Objective	0.9	1.22
Limit	0.6	1.85

If the flow speeds are too low, deposits, particularly sand, may accumulate in the pipes, leading to a silting phenomenon. On the other hand, flow speeds that are too high can cause premature wear of pipes, thus reducing their service life, and lead to pressure losses due to excessive friction with the walls of the pipes. In addition, it is essential that the slope of the pipe remains constant to avoid the accumulation of sediments at low points and the formation of air pockets at high points.

II.4 Raw water tank

The raw water line usually terminates on one side of the raw water tank, rather than directly at the bottom. This arrangement allows the raw water to be screened before it is pumped and sent to the purification station. This sieving step aims to eliminate debris and unwanted particles present in the raw water, thus ensuring optimal quality of the water to be treated. Once sieving is complete, the water is ready to be pumped and routed to the purification system (**Figure 8**).

II.5 Low pressure pump

Nowadays, motor pumps are increasingly used for extracting water from raw water tanks and then for transporting it to the purification station. The electric motors of these pumps are mounted on the slab that covers the top of the raw water tank. The maximum flow rate of pumped water must correspond to the maximum daily consumption (**Figure 8**).

II.6 Underground hydraulics reminders

II.6.1 Essential concepts

II.6.1.1 DARCY experience

Darcy's Law uses one of the fundamental equations used in hydrogeology to describe the movement of water through an aquifer. This law states that the water flow is proportional to the hydraulic gradient and depends on the aquifer material.

In the DARCY experiment, the flow of water occurs through a permeable medium, following a single direction, and with a constant section S. We then measure the difference h in hydraulic pressure between the two ends of a homogeneous section of length L, as well as the flow rate Q going through this section (**Figure 28**).

The fictitious filtration speed is given by $V = \frac{Q}{A}$. This is the average speed that the flow whose flow rate is Q would have if the impermeable medium were absent.

On the other hand, the dimensionless quantity $i = \frac{\Delta h}{L}$ is used to denote the slope of the load line.

Darcy observed, at least for low flow rates, that the fictitious filtration speed is linearly linked to the slope j of the load line by:

$$V = Ki$$

Here K is a constant of proportionality. It corresponds to the permeability of the environment.

The value of the permeability K thus defined is between 3.10^{-3} to 10^{-4} m/s in sand. It can go down to 10^{-7} or even 10^{-9}.

Darcy's law is given as follows:

$$Q = KiA$$

Where Q is the flow rate (m^3/s), K is the hydraulic conductivity (m/s), j is the hydraulic gradient (m/m), and finally A is the cross sectional area (m²).

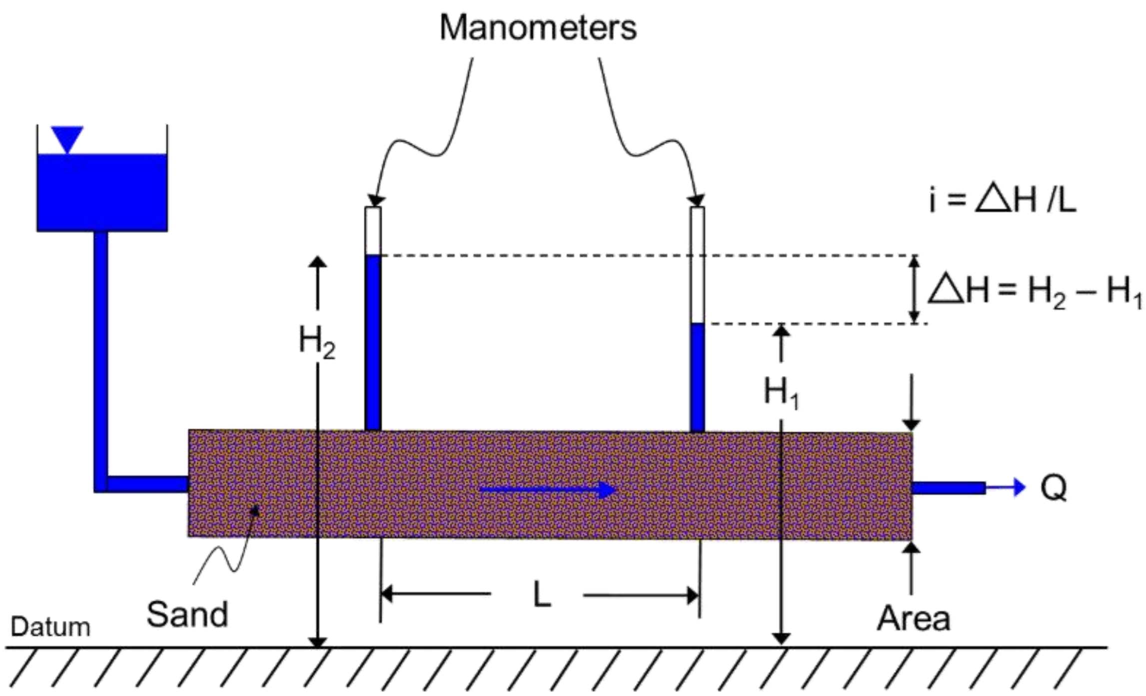


Figure 28: Darcy's law (source : https://www2.deq.idaho.gov/admin/LEIA/api/document/)

II.6.1.2 Filtration flow rate

II.6.1.2.1. Well capturing a captive water table

An artesian well is schematized by the flow of water in an indefinite permeable layer, of constant thickness e and permeability K, enclosed between two impermeable layers. The well, of radius r, crosses the entire permeable layer (**Figure 2929**).

In addition, h is the water level in the well, and H is the height of the free water table at a distance R. Therefore:

$$Q = 2K\pi e\left(\frac{H-h}{ln\frac{R}{r}}\right)$$

$$Q = 2K\pi e\left(\frac{H-h}{2.3Log_{10}\frac{R}{r}}\right)$$

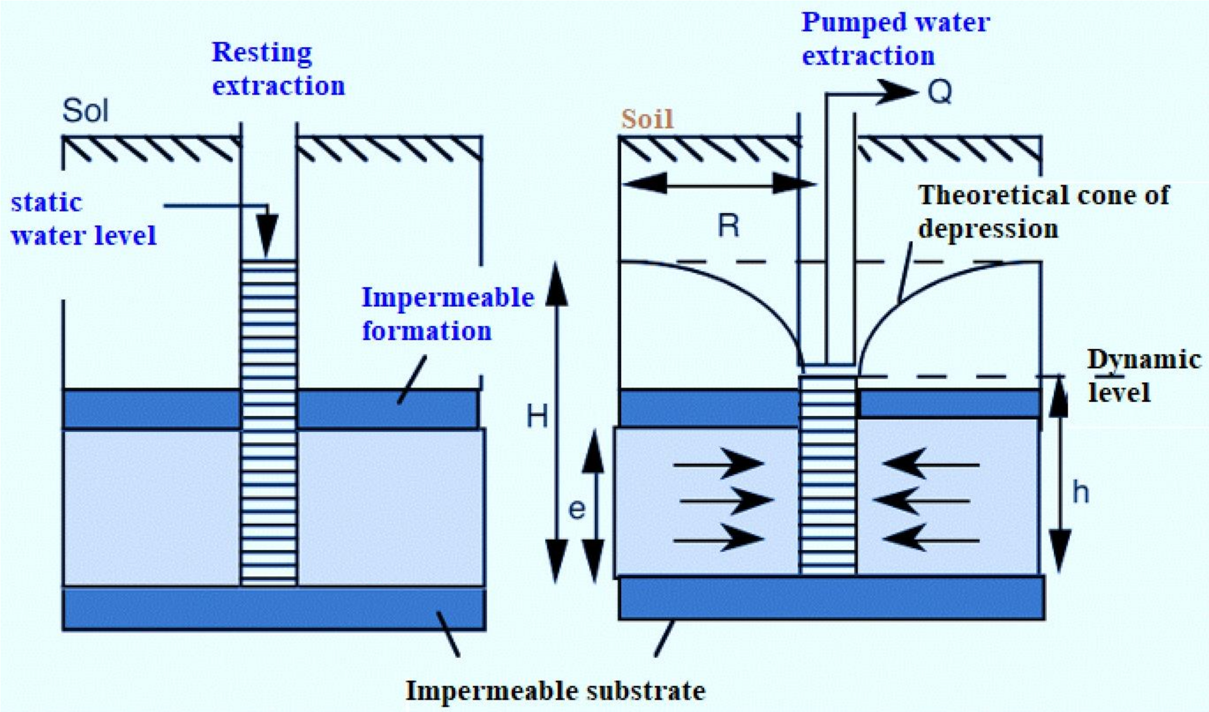


Figure 30: Operating principle of groundwater collection in a confined water table (non-spouting artesian drilling).

II.6.1.2.2. Open water filter well

The thickness of the aquifer is not constant. It is limited by an impermeable horizontal bottom, but has, at its upper part, a free surface, at height *H*, and at a distance *R* from the axis of the well (**Figure 30**).

$$Q = \frac{K\pi\left(H^2 - h^2\right)}{ln\frac{R}{r}}$$

$$Q = \frac{K\pi\left(H^2 - h^2\right)}{2.3\, log_{10}\frac{R}{r}}$$

Where Q is the pumping rate (m^3/h), K is the hydraulic conductivity of the medium (permeability coefficient of the water table), e the thickness of the aquifer layer (confined water table) through which the water flows , H the static level of the water table, i.e. the height of the water at the bottom of the well (m), h the dynamic height of the water table, i.e. the variation in the height of the water in the well (m), R is the radius of action (of influence), i.e. the horizontal distance from the well to the point where the pumping effect is negligible (m), and r is the radius of the well, i.e. the distance from the center from the well to its wall (m).

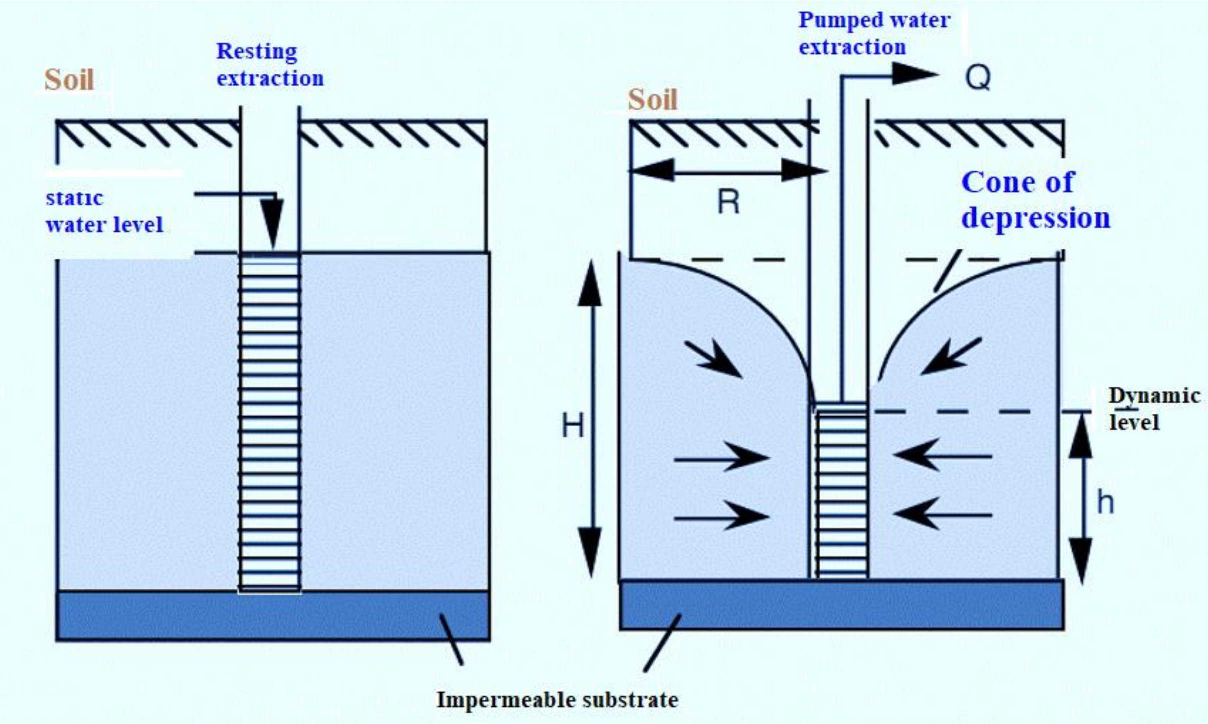


Figure 31: Operating principle of a groundwater collection in free water table

II.6.1.3 Determination of the radius of influence

The radius of influence, noted R, can be evaluated in the field using piezometers placed at increasingly greater distances from the well or drilling. This radius can also be estimated approximately using various formulas, as follows:

Vibert admits that: $\frac{R}{r} = 100 \div 300$

Sichardt gives a roughly approximate law: $R = 3000(H - h)\sqrt{K}$

From the log approximation formula: $R = 1.5\sqrt{\frac{Tt}{S}}$

As for Choultse, he considers that: $R = \sqrt{\frac{6Ht}{n}}$

Where T is the transmissivity, H the height of the aquifer, t the pumping time, and n is the porosity of the drainage.

II.6.1.4 Maximum filtration speed

If the filtration speed is too high, the water can then carry away the finer grains of sand. This is known as the "*Fox Phenomenon*". Consequently, the fictitious filtration speed must not exceed the critical speed.

$$V_c = \frac{\sqrt{K}}{15}$$

Therefore, the maximum (critical) flow can then be expressed as:

$$Q_c = 2\pi rh\frac{\sqrt{K}}{15}$$

(K and h are expressed in meters, k in m⁄s, and Q_c in m^3⁄s).

II.6.1.5 Consequences of Exceeding the Critical Pumping Speed of a Well

Exceeding the critical pumping speed of a well may engender several undesirable consequences, including:

- The static level of the aquifer drops, which can lead to partial or total drying of the well.
- The energy costs related to pumping increase because excessive pumping speed requires more energy to extract the same amount of water.
- Salty or brackish water may penetrate into the well, especially in coastal regions where the aquifers are in contact with sea water.
- The long-term well productivity decreases due to the fact that fine soil or sand particles are carried by water, which can block cracks or pores in the aquifer.
- Ecological imbalance may occur in local ecosystems due to the reduction in the flow of watercourses fed by overexploited groundwater.
- The water quality can deteriorate due to the intrusion of contaminants from external sources like industrial or agricultural pollutants which can be drawn into the well at high speeds.

II.6.1.6 Pumping in non-steady state

When it comes to non-permanent pumping, the radius of influence can be defined and calculated using the following formula:

$$R = 1.5\sqrt{\frac{KH}{n}t}$$

For $t \leq$ 24h

Here, *R* designates the radius of influence at time *t*, *K* is the permeability of the massif, *H* the height of the water table above the bedrock, *n* the effective porosity of the ground.

Likewise, the water table lowering can be calculated using the following formula:

$$Z' = s = \frac{Q}{2\pi KH}ln(^R/_r)$$

Chapter III: Hydraulic characteristics of flows in drinking water and sanitation distribution pipes

III.1 Introduction

This chapter presents the main hydraulic concepts necessary for calculating flow rates, velocities and water levels in sewer pipes and water distribution pipes, as well as for calculating load slopes or energy and piezometric lines, as well as pressure losses.

In hydraulics, which is a branch of engineering devoted to the study of fluid movements, we can distinguish two categories of flows, with different characteristics:

1. Load flows where the pipe is completely filled with water. These flows are commonly found in drinking water distribution networks and pressure pipes.
2. Free surface flows, which are characterized by an interface delimiting water and air. These flows are particularly observed in natural watercourses, irrigation canals, and sanitation networks.

III.2 Definition of free surface flows

Free surface flows are characterized by fluid flow with a free surface in direct contact with the atmosphere. In this type of flow, the piezometric line is aligned with the surface of the liquid, where the atmospheric pressure is predominant (**Figure 31**).

In nature, rivers exhibit free-surface flow where water flows freely while in contact with the atmosphere. In urban areas, the flow of wastewater in separate or unitary systems also occurs on a free surface.

Figure 32: An example of free surface flow in an irrigation canal (source: http://www.vitaminech.com/annuaire/canal-d-irrigation/Photos_15108_48969_0_1.html)

III.2.1 Sizing free surface flows

III.2.1.1 Basic equation (Chézy's formula)

Chézy proposed an equation based on experimentation. This Formula allows calculating the speed of uniform flows in canals. Uniform flows are characterized by a constant longitudinal water velocity throughout the canal. The Chézy equation is given by:

$$V = C_c\sqrt{R_h i}$$

Here, V is the average longitudinal speed of the flowing water (m⁄s), C_c is the Chézy coefficient which depends on the friction of water against the walls, and R_h the hydraulic radius (m) which is given by:

$$R_h = \frac{A}{WP}$$

where A is the area of the flow section (m^2). In the case of a circular pipe in which the water circulates at full flow, $A = \frac{\pi d^2}{4}$ (m^2), and *WP* is the wetted perimeter (m), which corresponds to the length of the water contact line with the wall. In the case of a circular pipe where the water flows at full flow, we have: $WP = \pi d$ (**Figure 32**).

With *d* the internal diameter of the pipe (m) and *i* the slope of the energy line (m/m).

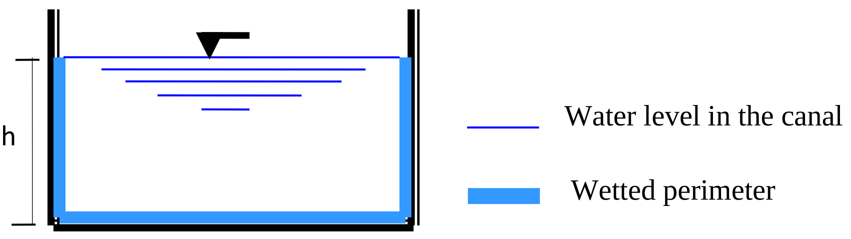


Figure 33 : Example of a section of a rectangular channel with a free-surface flow

Several researchers have used the Chézy equation while trying to give a better evaluation of the coefficient C by expressing it as a function of the roughness of the pipe walls and the hydraulic radius R_h.

III.2.1.2 Estimation of the Chezy coefficient C using the Bazin formula

In practice, Bazin's formula is the one that best estimates the Chézy coefficient C. It is expressed as a function of the wall roughness and hydraulic radius R_h. It is given as:

$$Cc = \frac{87}{1 + \frac{\gamma}{\sqrt{R_h}}}$$

Here, γ is the Bazin roughness coefficient. It depends on the nature of the wall.
The table below gives some values of the roughness coefficient, which depends on the nature of the wall of the pipe.

Table 2: Roughness coefficient, which depends on the nature of the wall

Nature des parois	γ (m1/2)
Parois très unies (ciment, bois raboté...)	0,06
Parois unies (planches, briques, pierres de taille...)	0,16
Parois en maçonnerie de moellons	0,46
Parois de nature mixte (section en terre, très irrégulières)	0,85
Canaux en terre dans les conditions ordinaires	1,30
Canaux en terre, avec fond de galets, parois herbées	1,75

Bazin roughness coefficient in sanitation

Rainwater with sand and suspended matter: $\gamma = 0.46$

Rainwater with decarnization zone and smooth materials: $\gamma = 0.3$

Rainwater and wastewater in buildings: $\gamma = 0.16$

III.2.1.3Gaukler, Manning and Strikler's formula

The Manning equation can be used to calculate the velocity of free surface flow.

The speed of flow along a pipe in which the water is subjected to atmospheric pressure is given by:

$$v = \frac{{R_h}^{2/3} \times i^{i/2}}{n}$$

The Chézy coefficient can then be expressed as follows:

$$C_c = \frac{{R_h}^{1/6}}{n}$$

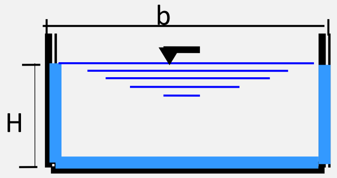
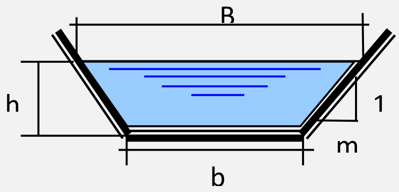
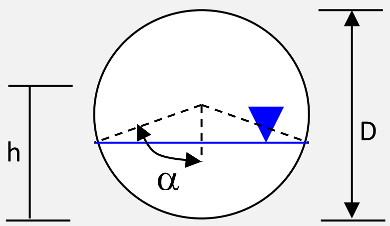
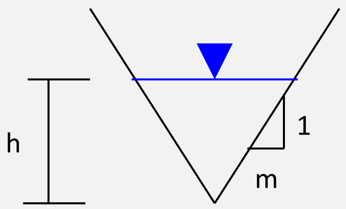
The Manning formula is used for calculating the gravity flows and, particularly, for sizing wastewater and rainwater evacuation pipes.

$$Q = K \times {R_h}^{2/3} \times i^{i/2} \times S_m$$

Here Q is the flow rate (m^3/s), V is the speed of water (m/s), S_m is the wetted cross-sectional area (m^2), and R_h is the hydraulic radius (S_m /P_m) in meters.

Also, I is the longitudinal slope (m/m), P is the wetted perimeter (m), and C the coefficient deduced from the BAZIN formula.

Table 3 Example of wetted areas and perimeters of some geometric shapes

Geometric shape	Wetted section	Wetted perimeter	Hydraulic radius
	$A = b\ h$ $A\ (m^2)$ $b\ (m)$ h = Draught (m)	$P = b + 2h$ $P = (m)$	$R_h = b\ h\ /(b + h)$ $R_h\ (m)$
	$A = h(b+B)/\ 2$	$P = b + 2\ (((B - b)\ /\ 2)^2 + h^2)^{1/2}$	$R_h = (h\ (b + B)\ /\ 2)\ /\ (b + 2\ (((B - b)\ /\ 2)^2 + h^2)^{1/2})$
	$A = D^2/4\ (\alpha - \sin(2\alpha)/2)$	$P = \alpha\ D$	$R_h = D/4\ [1 - \sin(2\alpha)\ /\ (2\ \alpha)]$
	$A = m\ h^2$	$P = 2\ h\ (1 + m^2)^{1/2}$	$R_h = m\ h\ /\ 2\ (1 + m^2)^{1/2}$

Application

Calculate the hydraulic radius of a circular section pipe, in two distinct cases. First case: the section is completely filled; second case: the section is half filled.

Answer

The hydraulic radius is given as :

$$R_h = \frac{A}{PM}$$

Then, for the :

Fully filled section: $R_h = \pi r^2/2\pi r\ \rightarrow R_h = r/2\ \rightarrow R_h = \emptyset/4$

Half-filled section: $R_h = (\pi r^2/2\,)/(\,\pi r) \rightarrow R_h = r/2\ \rightarrow R_h = \emptyset/4$

Table 4: Values of the Manning coefficient *n*, depending on the nature of the material and the condition of the wall

Nature of surfaces	**Wall condition**			
	Perfect	**Good**	**Fairly good**	**Bad**
Artificial canals				
Smoothened cement	0.010	0.011	0.012	0.013
Cement mortar	0.011	0.012	0.013	0.015
Planed wooden aqueducts	0.010	0.012	0.013	0.014
Unplaned wooden aqueducts	0.011	0.013	0.014	0.015
Channels lined with concrete	0.012	0.014	0.016	0.018
Raw rubble stones	0.017	0.020	0.025	0.030
Dry stones	0.025	0.030	0.033	0.035
Dressed rubble stones	0.013	0.014	0.015	0.017
Metal aqueducts with smooth semi-circular section	0.011	0.012	0.013	0.015
Metal aqueducts with pleated semi-circular section	0.0225	0.025	0.0275	0.030
Earth canals, straight and uniform	0.017	0.020	0.0225	0.025
Canals with stones, smooth and uniform	0.025	0.030	0.033	0.035
Canals with stones, rough and irregular	0.035	0.040	0.045	-
Wide meandering earthen canals	0.0225	0.025	0.0275	0.030
Dredged earthen canals	0.025	0.0275	0.030	0.033
Canals with earthen bottom, sides with stones	0.028	0.030	0.033	0.035
Natural watercourses				
1) Clean, straight-line banks	0.025	0.0275	0.030	0.033
2) Same as 1, with some herbs and stones	0.030	0.033	0.035	0.040
3) With meanders, with some ponds and shallow places,	0.035	0.040	0.045	0.050
4) Same as 3, water at low flow, with lower slope and	0.040	0.045	0.050	0.055
5) Same as 3, with some herbs and stones	0.033	0.035	0.040	0.045
6) Same as 4, with stones	0.045	0.050	0.055	0.060
7) Areas with slow flowing water, grass or very deep pits	0.050	0.060	0.070	0.080
8) Areas with lots of weeds	0.075	0.100	0.125	0.150

Source:

http://sites.uclouvain.be/gce/~hydraulique/enseignement/didacti/lecon02/manning.html

Table 5: Manning coefficient *n* for different pipe materials

Conduit Type	n
Welded steel	0.012
Polyethylene (PE)	0.009
PVC	0.009
Asbestos cement	0.011
Ductile iron	0.015
Cast iron	0.014
Wood	0.012
Concrete (metal forms with smooth joints)	0.014

Source: https://www.reseau-cicle.org/wp-content/uploads/riaed/pdf/4-_Methodologies_d_evaluation_des_sites.pdf

III.3 Sizing of pipes under pressure

III.3.1 Definition of flow under pressure

There are two types of flow:

When the flow is subjected to pressure, the water fills the entire section of the pipe. This type of flow is particularly suitable for drinking water networks.

Energy losses are manifested by pressure drops due to friction of the fluid against the pipe walls. Estimating these losses is essential for the sizing of a hydraulic circuit, particularly with regard to the choice of diameter, pump characteristics, pressure, etc.

Pressure losses, which result in a progressive reduction in pressure along a pipe, can be classified into two categories, namely linear pressure losses and singular pressure losses. These losses result from energy dissipation, due to internal friction against the pipe walls, leading to the transformation of part of the mechanical energy into heat. In hydraulics, this energy is referred to as *Pressure*.

III.3.2 Linear pressure loss

Linear pressure losses take place along pipes, such as water supply or drinking water supply pipes. Researchers Darcy and Weisbach developed a specific equation that is applicable to circular pipes subjected to pressure in order to estimate these losses. This equation is formulated as follows:

$$\Delta H_{lin} = \lambda \frac{Lv^2}{2gd}$$

Where ΔH represents the frictional pressure losses in a pipe of length L (m), λ is the friction coefficient which depends on the Reynolds number and the roughness of the pipe wall, and g is the acceleration due to gravity (g = 9.81 $\frac{m}{s^2}$).

The Reynolds number R_e for a circular pipe subjected to pressure depends on the pipe diameter d, the flow speed v, and the dynamic viscosity μ. It is given by the following relation:

$$R_e = \rho_e \frac{Vd}{\nu}$$

Where ρ_e is the density of water $\left[\frac{kg}{m^3}\right]$, and μ is the dynamic viscosity of water $[\frac{Ns}{m^2}]$.

III.3.2.1 Reynolds experiment

In Reynolds' experiment, which is shown in **Figure 33**, a colored liquid stream is introduced into a horizontal flow. The shape and behavior of the colored net allows identifying the different flow regimes. In the laminar mode, the colored net retains its initial shape and then follows the movement of the liquid mass without mixing with it. One should know that this mode is generally observed when the flow speed is low or when the liquid has a high viscosity. In contrast, in the turbulent regime, which is characterized by a high flow velocity, the colored net disperses and mixes with the surrounding water, thus giving the flow an irregular and rough appearance.

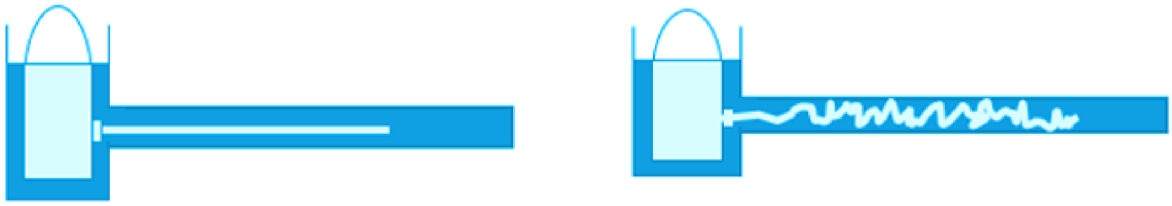


Figure 34: Different flow regimes. Left: laminar regime; Right: turbulent regime

In the case of a laminar regime ($R_e < 2000$), λ is independent of the wall roughness and depends only on the Reynolds number. It can be expressed as follows:

$$\lambda = \frac{64}{R_e}$$

In the case of the turbulent regime ($R_e > 2000$), several formulas may be utilized to deduce the value of λ.

Likewise, The Colebrook-White formula may be employed to estimate the pressure loss coefficient in pipes. This formula can be applied to turbulent regimes where the Reynolds number is greater than 2300. Around the 1940s, Colebrook and White observed that the transition zone could be represented by a monotonically decreasing curve that is asymptotic to the other two domains. They then proposed the following expression:

$$\frac{1}{\sqrt{\lambda}} = -2\,log\left[\frac{\varepsilon}{3.7d} + \frac{2.51}{R_e\sqrt{\lambda}}\right]$$

III.3.2.2 Prandtl formula

This formula may be applied to rough pipes:

$$\frac{1}{\sqrt{\lambda}} = 2\,log\frac{d}{2\varepsilon} + 1.74$$

Here ε is an approximate dimension of the internal roughness of the pipe.

The estimation of this coefficient from these formulas is only possible by an iterative calculation.

Some values of the roughness of the pipe, depending on its nature, are presented in the following Table.

Table 6: Some usual values of roughness ε (mm)

	Nature of the internal surface	Roughness index ε
1	Copper, lead, brass, stainless steel	0.001 à 0.002
2	PVC pipe	0.0015
3	Stainless steel	0.015
4	Commercial steel tube	0.045 à 0.09
5	Drawn steel	0.015
6	Welded steel	0.045
7	Galvanized steel	0.15
8	Rusty steel	0.1 à 1
9	New cast iron	0.25 à 0.8
10	Used cast iron	0.8 à 1.5
11	Inlaid cast iron	1.5 à 2.5
12	Asphalted sheet metal or cast iron	0.01 à 0.015
13	Well smoothed cement	0.3
14	Ordinary concrete	1
15	Rough concrete	5
16	Well planed wood	5
17	Ordinary wood	1

Source: http://www.thermexcel.com/french/ressourc/pdc_line.htm

III.3.2.2 Blasius formula

This formula can be used for smooth pipes:

$$\lambda = \frac{0.316}{{R_e}^{1/4}}$$

In practice, the value of λ can be determined using the Moody diagram given in **Figure 34**. This diagram is essentially used in the case of a pipe under pressure. It also serves to estimate the pressure loss coefficient as a function of the Reynolds number, for different values of the relative roughness coefficient $\frac{\varepsilon}{d}$.

The Moody diagram shows that for a turbulent regime in a pipe with rough walls, the pressure loss coefficient depends only on the relative roughness coefficient.

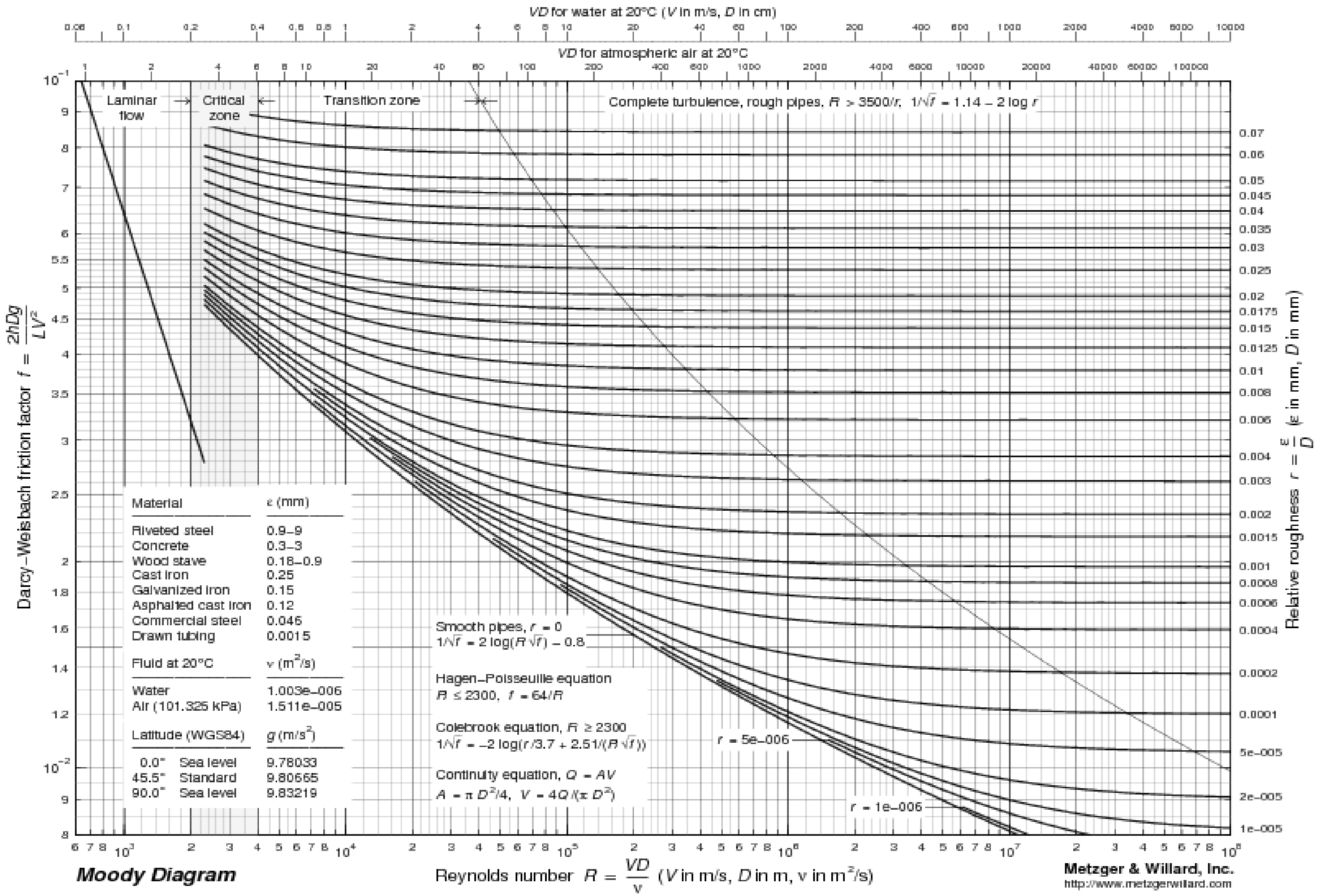


Figure 35 : Moody Diagram

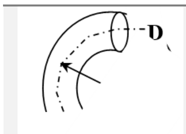
III.3.3 Singular pressure losses

The singular pressure losses are due to changes in direction, narrowing or widening of the pipe, and to the presence of valves, elbows, check valves, or any other regulating or joining device in the network. Every time water encounters one of these special parts, some of its energy is dissipated in the form of turbulence, resulting in a pressure loss. These singular pressure losses are generally proportional to the square of water speed at the point where this pressure loss occurs. Therefore, the higher the water speed, the greater the singular pressure losses. It is consequently highly recommended to carefully evaluate these elements when designing and modelling water distribution networks.

$$\Delta H_{sin\,g} = \frac{K_l V^2}{2g}$$

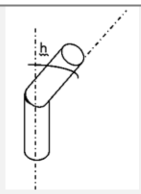
Here $\Delta H_{sin\,g}$ is the pressure loss due to an obstacle (m), K_l is the coefficient specific to the obstacle or the situation, V is the speed of the water circulating around or near the obstacle (m/s), and g is the acceleration due to gravity (m/s^2).

Table 7 summarizes the values of K_l for some specific structures.



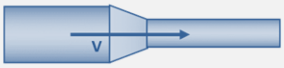
Rounded elbow

R/D	K_L
0.5	0.900
0.75	0.450
1.00	0.350
1.50	0.250
2.00	0.200



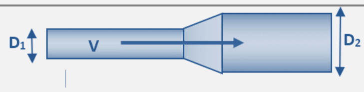
Sharp-angled elbow

α	K_L
15°	0.1
30°	0.2
45°	0.5
60°	0.7
90°	1.3



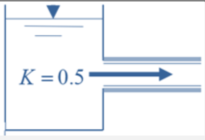
converging cone

- $\boldsymbol{K \approx 0}$

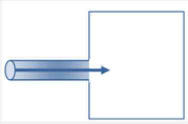


Diverging cone

- $\boldsymbol{K = 0.2\left(1 - \frac{D_1{}^4}{D_2^4}\right);\alpha < 10°}$
- $K\left(1 - \frac{D_1^2}{D_2^2}\right)^2 ;\alpha > 10°$



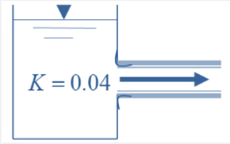
Water outlet



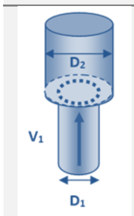
$\Delta h_{sin\,g} = \frac{V^2}{2g}$, k=1

Water inlet

Sharp corner connection

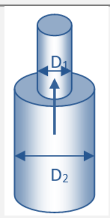


Rounded profile connection



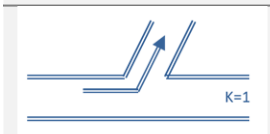
D_1/D_2	K_L
0.10	1.00
0.20	0.90
0.40	0.70
0.60	0.40
0.80	0.20

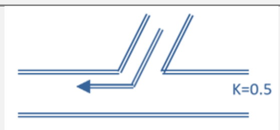
Sudden enlargement



D_2/D_1	K_L
0.10	0.6
0.20	0.5
0.40	0.4
0.60	0.3
0.80	0.20

Sudden enlargement





Reservoir outlet

Careful observation of the reservoir outlet allows noticing an alteration in the fluid velocity distribution and the emergence of a zone of separation of the liquid mass. This modification results in a tightening of streamlines, which represent the envelope of the velocity field (**Figure 35**). The presence of this recirculation zone results in a slight reduction in the flow passage section.

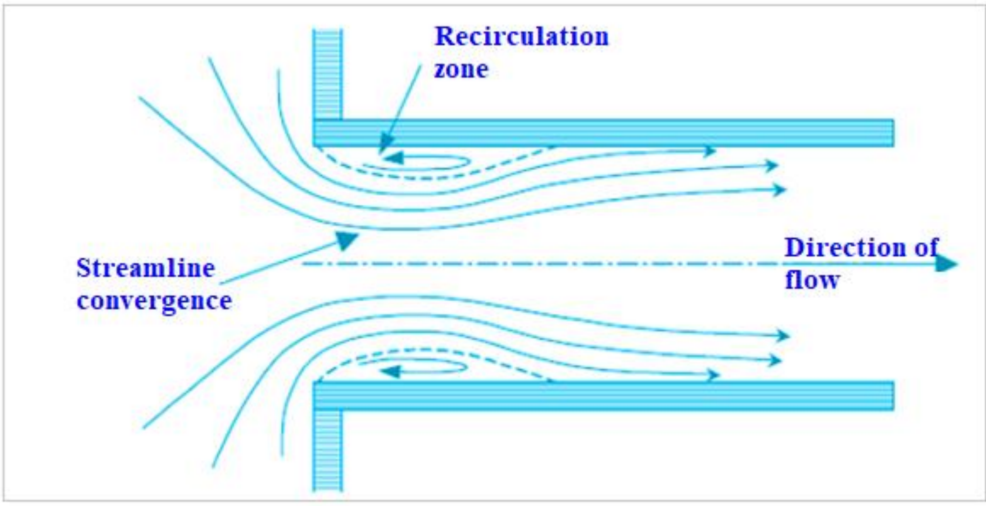


Figure 36: Effects of water leaving a tank on the speed distribution and formation of recirculation zones.

III.3.4 General expression of pressure loss

The linear pressure losses are always of the following form:

$\Delta H = f\frac{Lv^2}{2gd}; V = \frac{Q}{S}$

$\Rightarrow \Delta H = f\frac{LQ^2}{2gDS^2}; \quad m = \frac{f}{2gDS^2}$

$\Rightarrow \Delta H = mLQ^2$.

The singular pressure losses are always given as:

$H_{sin\,g} = \frac{K_l V^2}{2g}; \qquad V = \frac{Q}{S}$

$\Rightarrow \Delta H = \frac{K_L Q^2}{2gS^2}; \quad m' = \frac{1}{2gS^2}$

$\Delta H_{sin\,g.} = m' K_L Q^2$

The general expression of the total pressure loss is therefore given by:

$\Delta H_{total} = \Delta H_{Lin} + \Delta H_{sin\,g.}$

$\Delta H_{total} = Q^2(\sum mL + \sum m' K_L)$.

The total pressure loss in a system of pipes connected in series is the sum of the pressure losses of each pipe. This means that when water passes through several successive pipes, the total pressure loss is determined by adding the individual pressure losses of each section of the pipe. In a network comprising k pipes, noted from 1 to k, the total pressure loss can be calculated using the following formula:

$$\Delta H = \sum_{i=1}^{k} \left(\lambda_i \frac{L_i {v_i}^2}{2gd_i} + k_i \frac{v_i^2}{2g} \right)$$

III.4 Bernoulli's principle

The Bernoulli equation is viewed as the theoretical basis for the description of physical liquid flow phenomena. The energy of a water particle moving inside a water pipeline is the strict combination of potential energy, kinetic energy and pressure energy. According to the principle of conservation of energy, energy is neither created nor destroyed; it can only be transformed. Thus, the energy of a water particle remains constant as it passes through different points inside the pipeline, whether at point 1, point 2, or any other point along the water path.

Consider two sections, 1 and 2, in a tube through which a steady state flow rate Q flows (the physical quantities are independent of time).

$$\frac{\partial P}{\partial t} = 0; \quad \frac{\partial V}{\partial t} = 0; \quad \frac{\partial \rho}{\partial t} = 0$$

If the fluid is incompressible, then its density remains constant regardless of the pressure it is subjected to. In this case, one can write that the flow rates in two sections of a flow tube are equal, which can be mathematically expressed by the equation:

$$Q = S_1 V_1 = S_2 V_2$$

This equation expresses the principle of continuity for an incompressible flow.

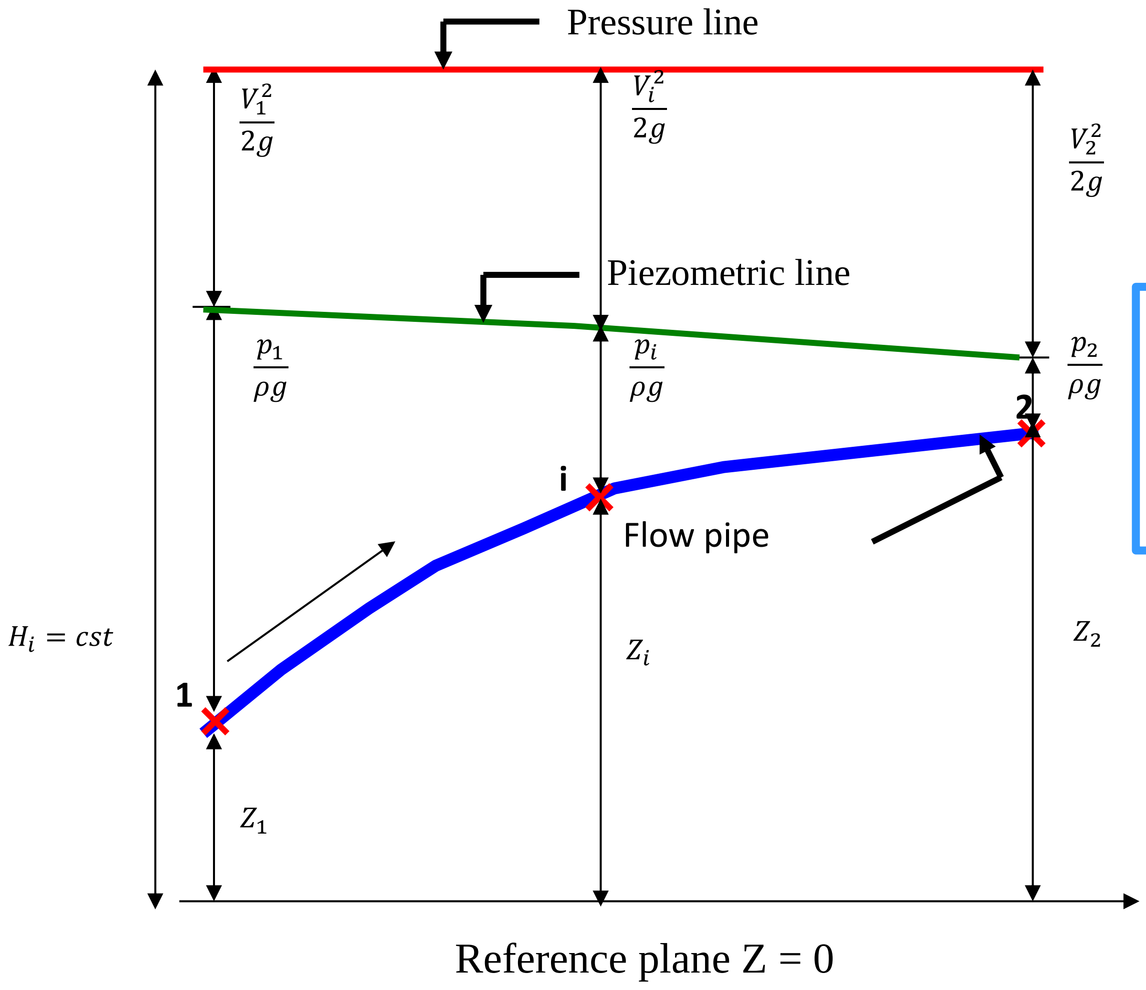
Additionally, if we assume that there are no frictional forces, then the sum of potential, pressure, and kinetic energies remains constant along the streamline. This principle can be formulated as follows (**Figure 36**):

Potential Energy + Pressure Energy + Kinetic Energy = constant

This essentially amounts to applying the principle of conservation of energy in the context of the incompressible fluid flow. In this case, the variation of each form of energy is compensated by those of the other forms.

In a more elaborate form, one can then write:

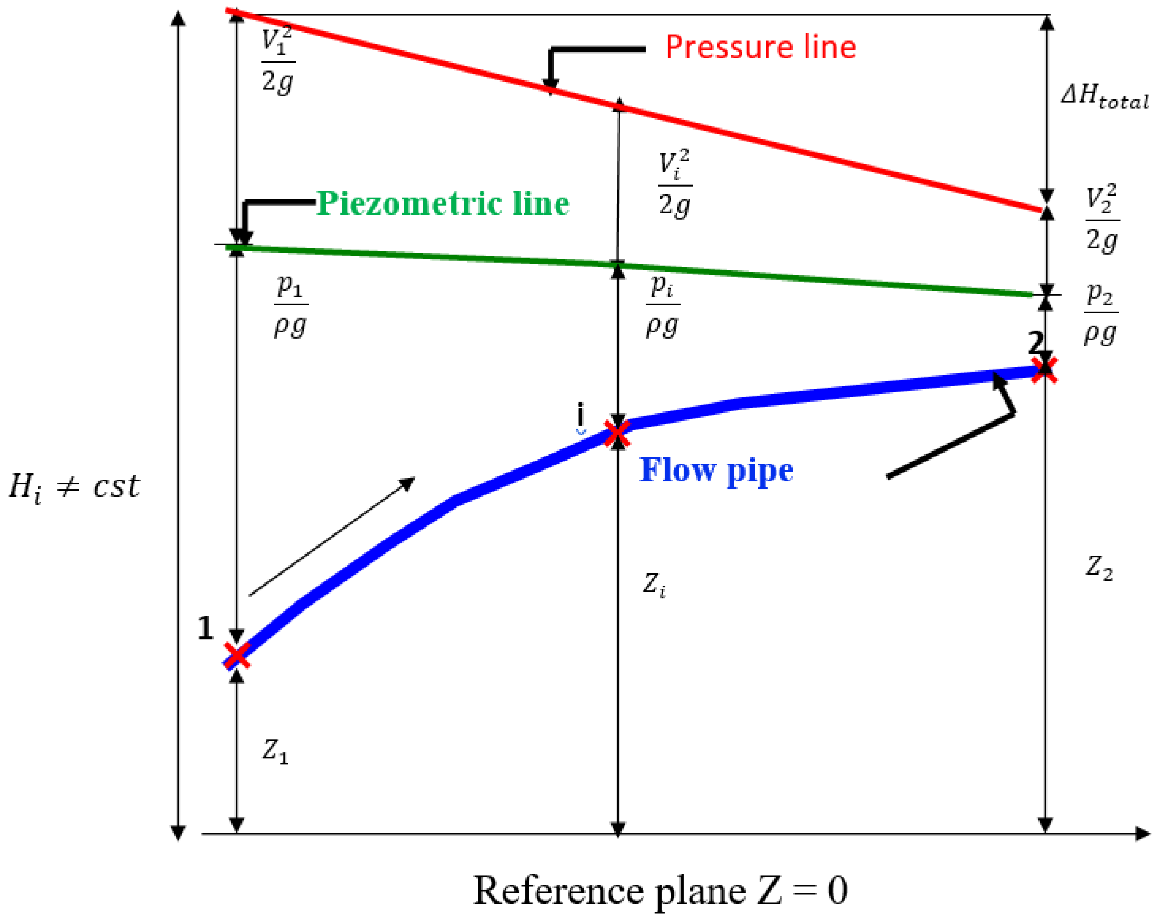
$$\frac{P_i}{\rho g} + \frac{V_i^2}{2g} + Z_i = cons\,tan\,t \qquad (1)$$



$$\frac{P_i}{\rho g} + \frac{V_i^2}{2g} + Z_i = cons\tan t\, e$$

$$\frac{p_1}{\rho g} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\rho g} + Z_2 + \frac{V_2^2}{2g} = cste$$

Figure 37 : Conservation of energy: Incompressible fluid flow



$$\frac{p_i}{\rho g} + Z_i + \frac{V_i^2}{2g} \neq cste$$

$$\frac{p_1}{\rho g} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\rho g} + Z_2 + \frac{V_2^2}{2g} + \Delta H_{totale}$$

Figure 38 **:** General form of Bernoulli's equation with regular and singular pressure losses

Here Z_i is the elevation of the water particle (m), P_i is the pressure (Pa or N/m^2), ρ is the density of the liquid (kg/m^3), V is the speed of the particle (m/s), and g the acceleration of gravity (g = 9.81 m.s^{-2}).

The above equation expresses the conservation of the total energy of a particle of a fluid during its movement.

Equation (1) can also be expressed as follows by considering two arbitrary points 1 and 2 of a pipeline.

$$\frac{p_1}{\rho g} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\rho g} + Z_2 + \frac{V_2^2}{2g} = cst$$

III.4.1 Interpretation of Bernoulli's equation

III.4.1.1 Geometric interpretation

$$\frac{P_i}{\rho g} + \frac{V_i^2}{2g} + Z_i = cst$$

Here Z_i is the position height, $\frac{pi}{\rho g}$ is the pressure height or piezometric height, and $\frac{V_i^2}{2g}$ is the the height due to the speed or kinetic height.

III.4.1.2 Energy interpretation

$$gZ_i + \frac{p_i}{\rho} + \frac{V_i^2}{2} = cst$$

Where gZ_i is the specific potential energy of position, $\frac{p}{\rho}$ is the specific potential energy of pressure, $gZ_i + \frac{p_i}{\rho}$ refers to the specific potential energy of the liquid, and $\frac{V_i^2}{2}$ relates to the specific kinetic energy.

In reality, some energy losses due to friction occur throughout the path (**Figure 37**). These energy losses, or pressure losses, are taken into account in equation (2):

$$\frac{p_1}{\rho g} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\rho g} + Z_2 + \frac{V_2^2}{2g} + \Delta H_{total}$$

ΔH_{Total} represents all energy losses due to friction of water particles against the walls of the pipe.

$$\Delta H_{total} = \Delta H_{Lin} + \Delta H_{sin\,g.}$$

This is the general form of Bernoulli's equation with regular and singular pressure losses.

$$\alpha_1 \frac{v_{1m}^2}{2g} + \frac{p_1}{\rho g} + z_1 = \alpha_2 \frac{v_{2m}^2}{2g} + \frac{p_2}{\rho g} + z_2 + \sum_i \lambda_i \frac{L_i}{D_i} \frac{v_i^2}{2g} + \sum_j k_j \frac{v_j^2}{2g}$$

Application 1

Water at 20°C flows through a circular tube with a diameter of 60 mm. Calculate the largest flow rate for which laminar flow can occur.

The kinematic viscosity of water at 20°C is equal to $\nu = 1 * 10^{-6}$ m²/s.

Solution

If the minimum value of R_e is 2000, then

$V = 2000/10^6 * 0.06 = 0.033$ m/s

$Q = AV = \pi/4 * 0.06^2 * 0.033 = 3.73 * 10^{-4}$ m³/s $= 0.373$ l/s

Application 2

Using the Moody diagram, calculate the linear pressure loss in a pipe with a diameter of 900 mm, over a length of 500 m, with a flow rate equal to 2.3m³/s. The absolute roughness is $\varepsilon = 0.6$mm, and the kinematic viscosity of water is given as $\nu = 1.31 * 10^{-6}$.

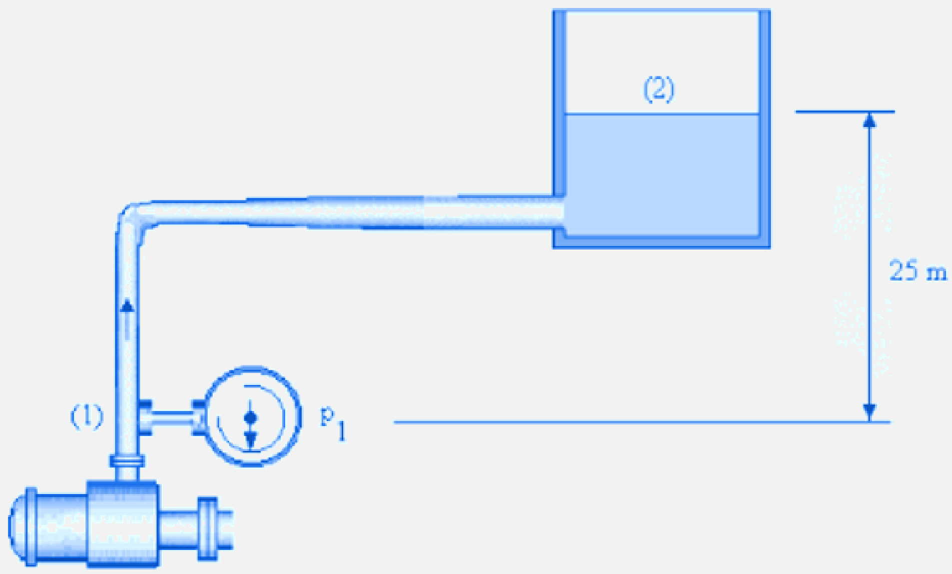
Solution

The Moody diagram allows writing that for $\varepsilon/D = 0.00062$ and $Re = 1.3 * 10^6$, one can say that: $\lambda = 0.019$

$$\lambda = 0.019 \times \frac{500}{0.9} \times \frac{1.886^2}{2 \times 9.81} = 1.91m$$

Application 3 : (source : © D.J.Dunnwww.freestudy.co.uk)

The figure below illustrates a pump delivering water to a tank via a 30 mm diameter pipe. Find the pressure at point (1) when the flow rate is 1.4dm³/s, knowing that the density of water is 1000 kg/m³ and the pressure loss due to friction is 50 kPa.



Solution

The surface area is: $A = \pi x 0.03^2/4 = 706.8x10^{-6}\ m^2$.

The flow rate is $Q = 1.4dm^3/s = 0.0014m^3/s$

The average speed inside the pipe is $V= Q/A= 1.98m^3/s$

Applying Bernoulli's theorem between point (1) and point (2) on the free surface of the tank then gives:

$$\frac{p_1}{\rho g} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\rho g} + Z_2 + \frac{V_2^2}{2g} + \Delta H_{total}$$

The reference plane goes through point (1), which implies that $Z_1 = 0$ and $Z_2 = 25m$.

The gauge pressure on the surface is equal to zero.

The pressure loss is 50KPa = 50000Pa

The velocity at point (1) is $V_1 = 1.98$ m/s, but at the surface of the tank it is $V_2 = 0$

Based on the above, one can write:

$$p_1 + 0 + \frac{1000 \times 1.98^2}{2} = 0 + 1000 \times 9.9125 + 0 + 50000$$

$$p_1 = 293.29kPa$$

Chapter IV: Drinking water distribution

IV.1 Introduction

The water distribution system, or water distribution network, is intended to transport water from the water source, possibly treated, to points where it is accessible to consumers. A community's water needs vary greatly throughout the day. Water demands are generally highest during times when water is used for personal hygiene, laundry, meal preparation, and household cleaning purposes. On the other hand, these are significantly reduced during the night.

Water is stored in reservoirs overnight so that high water needs can be met during peak hours of the day, as water demand varies during the 24 hours. For this, the pipe diameters must be carefully dimensioned in order to guarantee an optimal water flow while maintaining a ground pressure compatible with the height of the buildings.

IV.2 Water storage tanks

Once treated, drinking water is stored in structures called Tower Tanks, commonly called Water Towers, or in buried tanks that are located in an area of high topography. The function of storage tanks lies in temporarily storing water while awaiting distribution. They also play a vital role in regulating the water flow.

IV.2 .1 Principle of operation

The principle of operation is as follows:

- *Catchment*: Water is pumped by means of a powerful motor either from groundwater, or from treatment plants, or from a reservoir, using large pipes.
- *Storage*: The water captured with pumps is intended to fill the tank which is located at a higher point.
- *Distribution or supply*: The water leaves the tank, which is located at a specific height. It is then supplied to the agglomerations through pipes with a constant pressure. For example, a water tower is generally placed on a geographical peak in such a way that it is located above the highest faucet. The water tower is intended to supply the drinking water distribution network.

▪

IV.2 .2 The roles of water tanks

Drinking water reservoirs play a vital role in meeting the water demands. They are placed at various locations of the water distribution network and perform several crucial functions, namely:

1. Store water during periods when consumption is lower than production,
2. Restore water when consumption exceeds production,
3. Regulate the pressure and flow of water between the production and consumption phases,
4. Separate the distribution network from the delivery network,
5. Ensure the distribution of water in urban areas,
6. Facilitate the upkeep and maintenance of the pumping stations.

IV.2 .3 Types of tanks

When designing a drinking water reservoir, it is imperative to guarantee the stability and durability of the structure, as well as the quality of the previously treated water. Tanks can be classified into several types depending on:

IV.2 .3 .1 The topography of the reservoir location.

We therefore can mention the:

IV.2 .3 .1.1 Elevated tanks

These are reservoirs that ensure the distribution of water in urban areas by gravity. They are placed on a tower or on pillars or posts. The shapes of vats of the reservoirs are generally circular for a volume less than 1000 m^3 and frustoconical for a volume greater than 1000 m^3. The height of water in the reservoir varies between 5 and 6 meters (**Figure 38**).

IV.2 .3 .1.2 Tanks located on the ground

If the location of the tank is favourable for ensuring gravity distribution, it is placed directly on the ground. The section of these reservoirs is generally rectangular for a volume greater than 300 m^3, with a water height varying between 4 and 5 meters. If the section is circular, the volume is less than 3000 m^3.

IV.2 .3 .2 The materials the tank is made of.

In this case, three types of tanks are worth mentioning, namely:

a. Metal tanks

b. Polystyrene tanks c. Reinforced concrete tanks

IV.2 .3 .2 .1 Metal tanks

These tanks are easy to install as they offer the advantage of rapid construction and low water load. However, they have some disadvantages such as the increased difficulty of maintenance, high costs for maintenance and labour, as well as complex implementation.

IV.2 .3 .2 .2 Polystyrene tanks

These tanks have some advantages, like the fact that this excellent quality material offers great freedom in creating various tank shapes. Additionally, these tanks require minimal maintenance and are relatively inexpensive. However, their storage capacity is limited. Also, they are generally difficult to repair in the event of a crack.

IV.2 .3 .2 .3 Reinforced concrete tank

Reinforced concrete tanks offer several advantages, including on-site availability of aggregates, material durability, minimal maintenance, and large storage capacity. However, their construction requires careful installation of concrete. Inadequate implementation can lead to a risk of poor waterproofing. In addition, cracks are difficult to repair because, in this case, a suitable thin and flexible waterproofing coating must be installed.

IV.2 .3 .3 The shape of tank

Tanks can be classified according to the shape of their vat. The shape of these tanks may be circular, rectangular, or other. Generally, circular tanks offer several advantages in terms of sizing. However, rectangular tanks are generally easier to manufacture because they do not require the use of curved formworks.

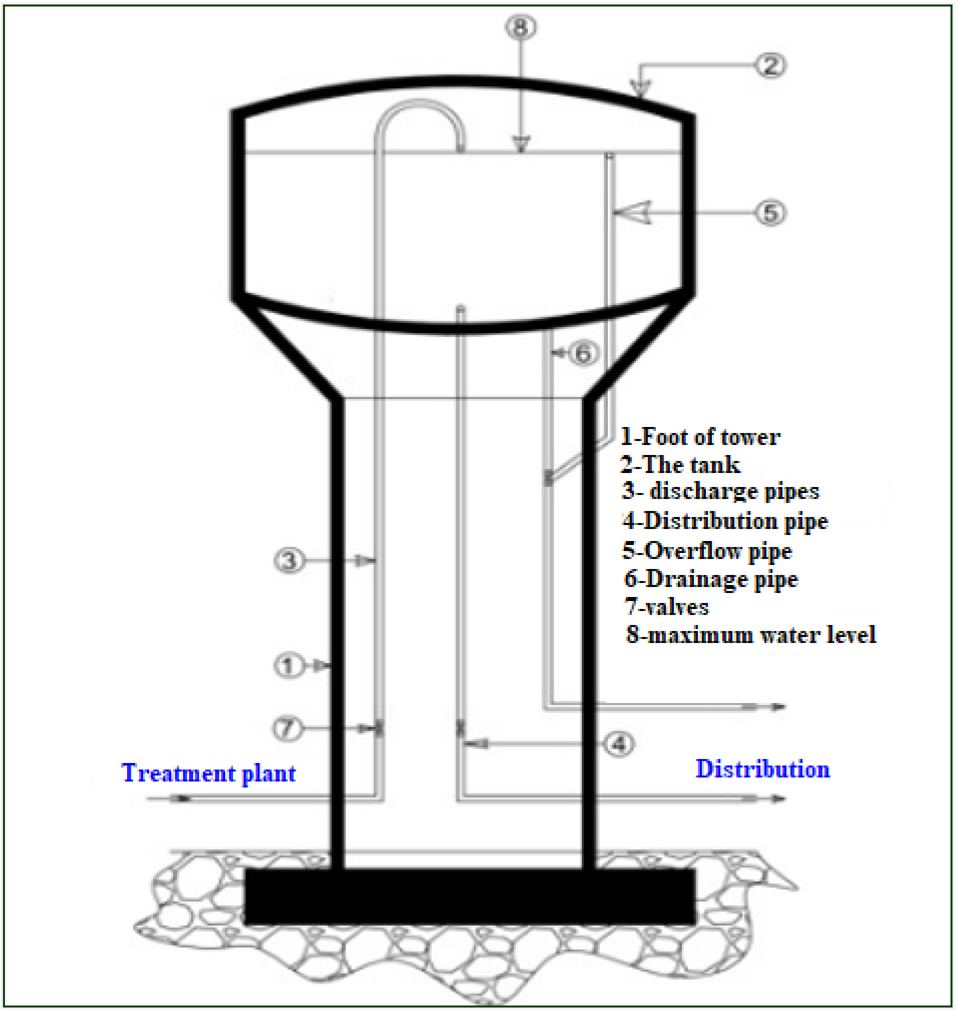


Figure 39: Water tower **(Source : https://fasoeducation.net/espace_eleves/secondaire/eftp/bac_technologique/travails_stockage/co/grain_dispositions_constructives.html)**

IV.2 .3 .4 Criteria to consider for siting a water storage tank

The choice of the site to install a storage tank is based on two fundamental parameters which are the altitude of the natural terrain (TN) and the proximity of the structure with respect to the town to be provided with water.

IV.2 .3 .4 .1 Searching for an elevated position

Raised areas are ideal locations for the installation of water storage structures. The high altitude of the natural terrain offers significant advantages in terms of construction, with a reduction in the height that is required for the structure, as this contributes to saving on the materials used. In addition, these high zones make it possible to increase the pressure in the distribution network pipes, which results in a reduction in the energy required for pumping water.

IV.2 .3 .4 .2 Proximity

The proximity of the water reservoir to urban areas is a very important criterion when choosing the location. Proximity helps reduce investment costs of the water distribution network. The final choice of site must be based on an economic analysis that takes into account the two main criteria mentioned above. Other elements, such as the nature of the foundation and the accessibility of the site, can also influence the decision regarding the choice of the site.

IV.2 .3 .5 Adjusting the drinking water pressure

The reservoir acts as a buffer zone between the flow provided by the water treatment plant and the user demands. It also plays a crucial role in pressurizing water. When water is released from the water tower, it is under high pressure. For example, a water tower located at an altitude of 100 meters provides water at a pressure of 10 bars, which allows distributing this water with an average pressure of 2.5 bars to buildings located several kilometers away. Drinking water suppliers must guarantee a pressure of at least 1 bar to their subscribers. Sometimes pumps are required to provide sufficient pressure.

In addition, routine sanitary inspections ought to be regularly carried out in these storage areas in order to guarantee the good quality of the water distributed to subscribers. The distribution of drinking water, from its source to the tap, is the responsibility of local authorities as a public service.

IV.2 .3 .6 Flow control and regulation

It is worth emphasizing that monitoring and controlling water flows is essential to ensure a constant and continuous drinking water distribution. A typical water tower can hold around 1000 cubic meters of water, but larger ones can store up to tens of thousands. Devices, such as flow meters and counters, are installed at the inlet and outlet in order to accurately measure and control flows. In general, the flow is higher during the day, when water demand peaks with the activity of city residents. The peak in water consumption is often recorded at the end of the day, when users return home. For example, for a town of 8000 inhabitants, the average flow during the day is around 60 cubic meters per hour, while it can be between 6 and 9 cubic meters per hour during the night. Excessive nighttime flow may indicate leaks in the network, requiring rapid intervention to prevent wasting water.

Thanks to technological advances, above-ground and underground reservoirs, as well as the entire distribution network, are now equipped with automated local stations which make it possible to monitor and control the incoming and outgoing flows in real time, while adjusting supply according to needs. Additionally, these stations can detect and report any anomalies in flow, making it easier to detect potential problems in advance.

IV.3 Classifications of drinking water distribution networks

Two main types of water distribution networks are worth mentioning, namely:

IV.3 .1 The branched network

Branched networks are mainly used in small communities due to their simple and cost-effective design. Their lower initial cost makes them attractive to these communities. However, despite this economy, branched networks have some significant disadvantages. In the event of a failure or breakdown in the main pipe, all downstream subscribers may be deprived of water, which means that this type of network lacks security and resilience. Additionally, their limited capacity can pose problems in rapidly expanding areas where water demand is increasing rapidly. Moreover, due to their design, branched systems may be more vulnerable to contamination of their water in the event of leaks or pipe damage (**Figure 39)**.

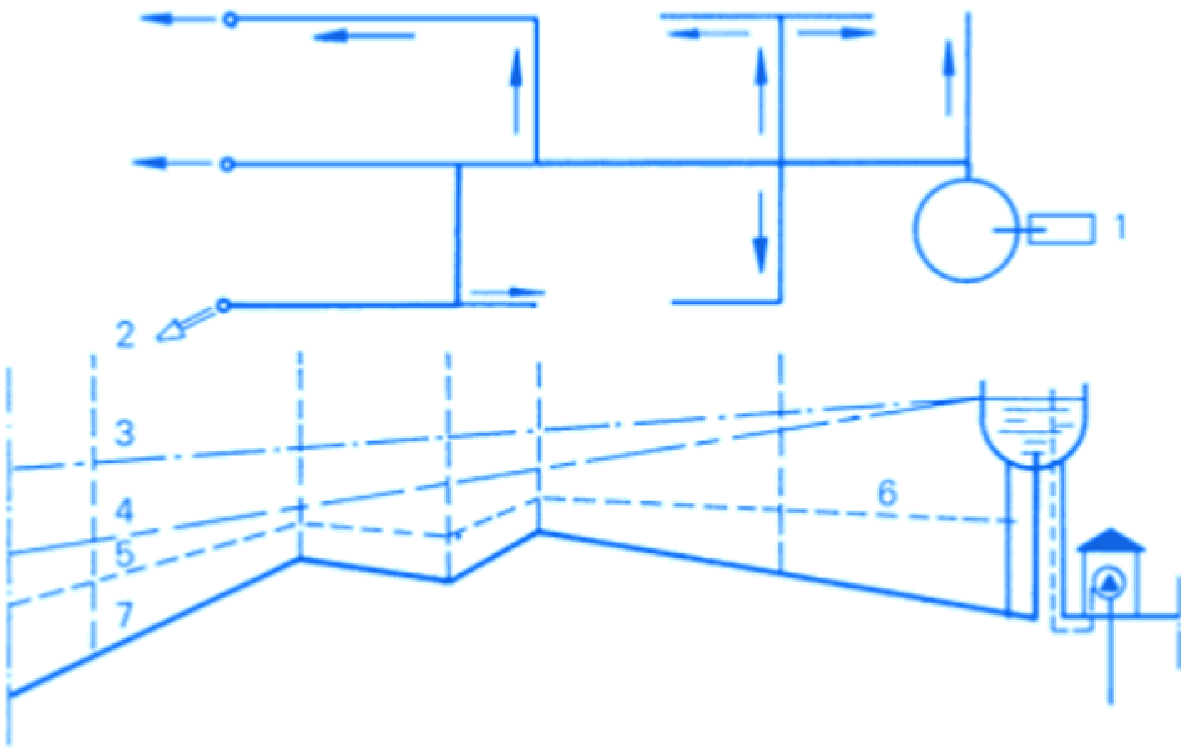


Figure 40: Diagram of a branched water distribution network : 1. Pump, tank; 2. Network expansion; 3. Minimum flow; 4. Maximum flow; 6. Minimum required; 7. Hydraulic profile: calculation by approximations based on the required residual pressures and the flow rates of the connections. (Vittone, 2010)

IV.3 .2 The mesh network

Mesh networks, generally preferred for larger drinking water distributions due to their supply through a loop, offer greater security and better flexibility in the event of a break compared to branched networks. It should be noted that thanks to the mesh configuration, a simple manipulation of taps makes it possible to isolate the damaged section while continuing to supply water to downstream subscribers (**Figure 40**). Although they are more expensive to set up due to the complexity of the system and to the additional equipment required for loopback, mesh networks are generally preferred because of the security they offer. However, their complexity can make maintenance more difficult and require higher maintenance costs in the long term**.**

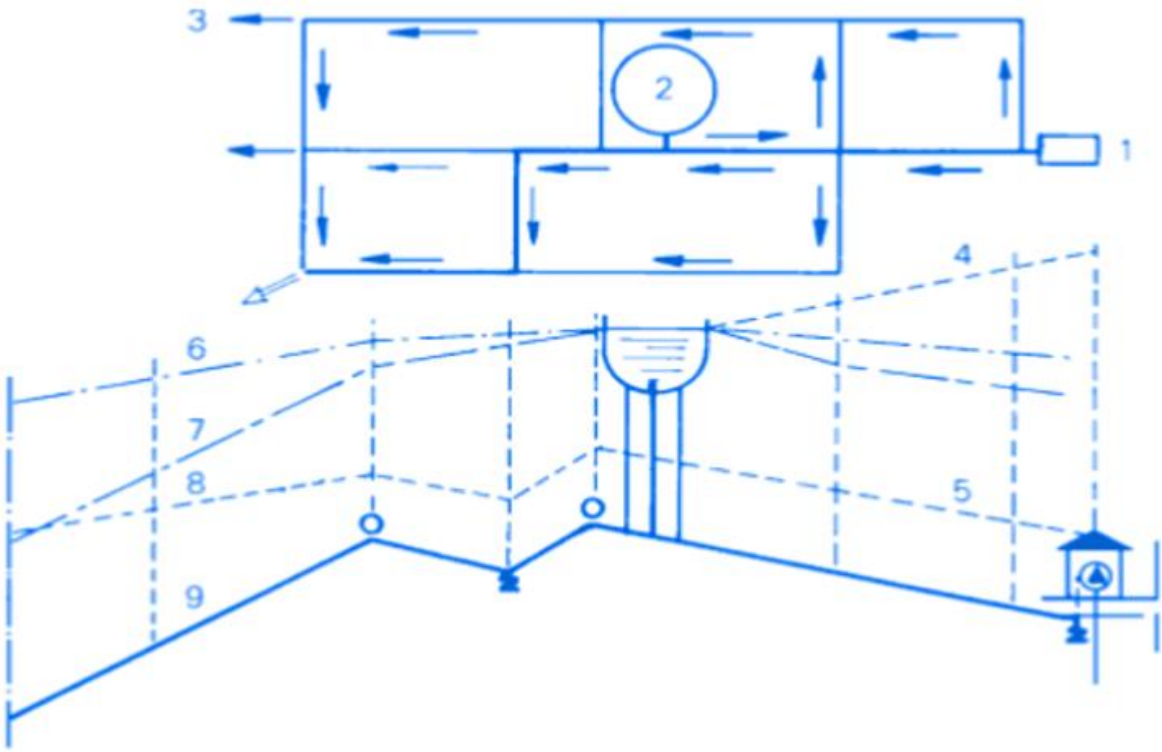


Figure 41 : Diagram of a mesh water distribution network : 1. Pump; 2. Tank; 3. Forecast requested; 4. Maximum pump; 5. Residual pressure; 6. Load line and minimum flow; 7. Load line and maximum flow; 8. Minimum required for fire; 9. Pipe and hydraulic profile of the central reservoir (Vittone, 2010).

IV.3 .3 Tank capacity

Drinking water tanks, placed on the ground or elevated, are directly subject to atmospheric influences, mainly to temperature rises in summer and to the risk of freezing in winter, hence the need for thermal insulation.

The so-called drinking water, suitable for human consumption, does not attack concrete as it is almost always slightly alkaline. Therefore, the tanks only need to be watertight. Different processes are hence used:

Waterproofing is applied to the mass of the concrete itself, with a dosage of around 400 kg/m^3, with an appropriate particle size and implementation. It is therefore necessary to prepare solid concrete, and not just compact concrete which could be hollow, with a minimum of fine grains. All the voids in the solid concrete are filled with solid mortar in which the cement paste is in sufficient quantity and therefore allows filling the voids with the sand, and also reducing as much as possible, if not completely, the re-casting. In addition, low shrinkage cement must be used. Finally, rapid filling of the tank, immediately after the concrete has hardened, is favorable to the compactness of the concrete.

 The cement mortar coating, 15 to 25 mm thick, is executed in two layers. The first layer forms the roughing while the second is the actual coating. Moreover, the dosage, previously classic, was between 1000 and 1200 kg of cement per m^3 of mortar. However, today it seems that it is possible to reduce this dosage to 600 or 800 kg of cement per m3 of mortar. The dosage of 500 kg sometimes used is undoubtedly a minimum.

 Same coating as above, but with the incorporation of a water repellent and a plasticizer. In the past, it was common to add black soap to the mixing water, at a rate of 9 kg per 100 liters. Nowadays, very varied and more effective products can be found in the market.

IV.3 .4 Sea water tank

It is well known that sea water is harmful due to the presence of magnesium salts, sulfate and chloride salts, and sometimes sulfate hydrogen (Mediterranean Sea).

This is how concrete and coatings are treated. This technique has become quite classical for sea structures. We can then consider:

- Special cement, which is rich in alumina and low in lime, can be a solution that is well suited to constructions subject to marine conditions. This cement offers increased strength and durability regarding the challenges posed by salt water.

- Very solid concretes and mortars, which are designed to minimize recovery, offer high density. This reduces the risk of voids or air pockets appearing during pouring or application, and hence helps to enhance the strength and durability of structures.
- Concrete or mortar requires extensive curing before being put into service, which means that this material requires a significant amount of time to reach its optimum strength before being fully operational or usable.
- Serious protection of the reinforcements using concrete with a thickness of 4 to 5 cm of . This operation is necessary in order to guarantee adequate coverage and protect the reinforcements against corrosion and external damage.
- The use of a bituminous coating is necessary to provide additional protection against humidity and water infiltration. This helps reinforce the durability and strength of the structure.

IV.4 Estimation of drinking water needs

Effective water management is crucial for both rural and urban populations. While the water needs of rural communities are often assessed on a lump sum basis, urban areas require a more complex approach which depends on the size of communities and on their population density.

IV.4 .1 Water needs in rural communities

Effective water management is essential to meet the needs of dispersed rural populations. In these areas, water needs can be assessed on a flat rate basis, based on the number of inhabitants, with adjustments for specific activities such as intensive livestock farming. For example, for towns with less than 2000 inhabitants, a water allocation (unit flow) of 125 liters per day per inhabitant should be planned, but this flow must be increased to 200 l/d/inhabitant in the case of intensive livestock farming.

IV.4 .2 Water requirement in urban areas

In urban areas where the population density is higher, water needs vary depending on the size of the city. Specific guidelines must then be established for different categories of cities, ranging from small towns to metropolises, to ensure sufficient water supply for all citizens. For example, for urban distribution, calculations are based on the size of the agglomeration, with water allocations varying from 150 to 200 l/d/inhabitant for towns of less than 20000 inhabitants, from 200 to 300 l/d/inhabitant for towns with 20000 to 100000 inhabitants, and from 300 to 400 l/d/inhabitant for towns with more than 100000 inhabitants.

IV.4 .3 Evolution of the population

The projection of long-term population growth, which is based on the formula for population growth over a specific horizon, is defined by:

$$N_a = N_0(1 + i)^a$$

where N_a represents the future population for a given horizon, N_0 is the population of the reference year, i the population growth rate (for example, i = 3%), and a the number of years separating l reference year of the horizon considered. This calculation allows planners to accurately forecast the needs for infrastructure, services and resources, including water needs, to meet the future demands of society.

IV.4 .4 Needs calculation procedures

IV.4 .4 .1 Current population

The current population depends on density (P), which represents the number of people per hectare, as well as occupied area (S), which indicates the total inhabited area. Thus, the total number of people in the region (N_0) is found by multiplying the population density by the occupied surface area.

$$N_0 = PS$$

IV.4 .4 .2 Future population

The population growth is given as:

$N_a = N_0(1 + i)^a$

The average daily water flow is determined by:

$Q_j^{Avge} = N_a q_0$

Where N_a is the number of inhabitants and q0 the unit flow (water supply in liters/day/inhabitant)

The maximum daily flow is given by:

$Q_j^{max} = k_j Q_j^{Avge}$

Here k_j is the daily coefficient of variation k_j = (1.1 – 1.3)

The maximum hourly flow is equal to: $Q_h^{max} = \frac{Q_j^{max}}{24} \times k_h$

$Q_h^{max} = Q_p$

k_h is the hourly coefficient of variation.

$$k_h^{max} = \alpha \times \beta$$

Where $\alpha = 1.2 - 1.4$.

The values of β are given according to the number of inhabitants

Nhab (10^3)	1	1.5	2.5	4	6	10	20	50	100	200	>300
β_max	2	1.8	1.6	1.5	1.4	1.3	1.2	1.15	1.1	1.05	1

Note: The sizing of the storage tank is carried out using the maximum daily flow (Q_j^max), as for the sizing of the distribution network, it is done with the peak flow (Qp).

The distribution tank serves to balance the supply of water from the source and treatment with the varying water demand from the network. It stores water when demand is low and releases it when demand is high, ensuring a constant supply of water to users.The storage volume must be large enough to balance the differences between production and demand.

The necessary storage volume can be read from the following graph (**Figure 41**):

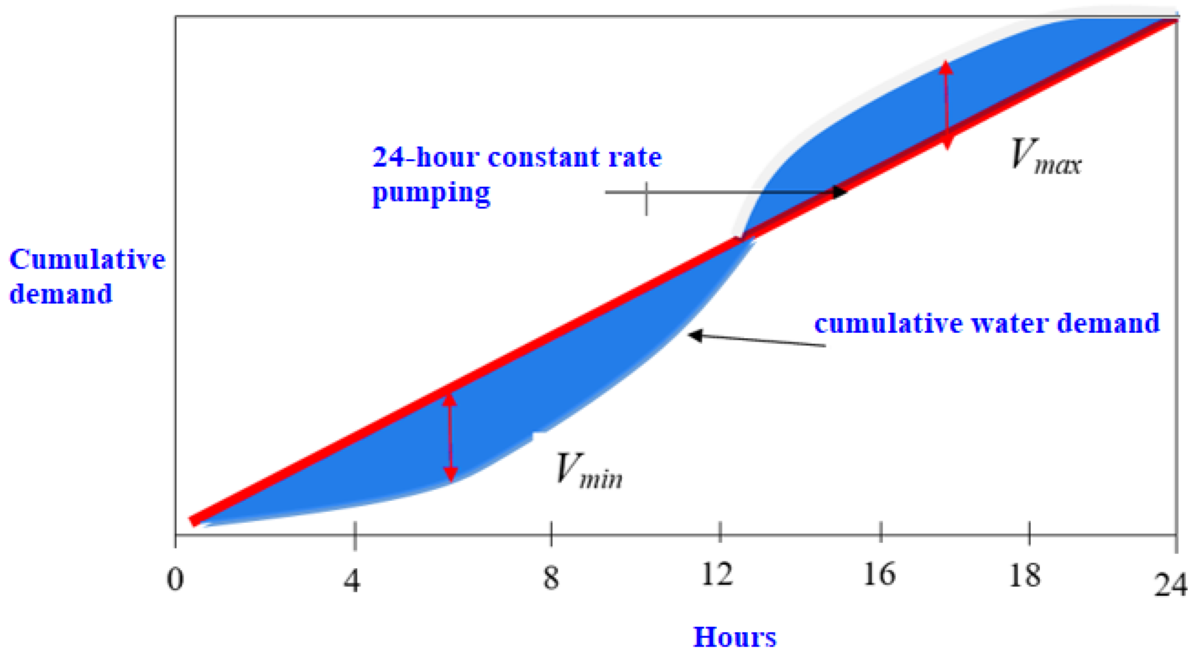


Figure 42: Variation in demand for water stored in the reservoir over a 24-hour period

It is worth adding that each tank must be equipped with a fire reserve with a minimum required capacity of 120 cubic meters. This water reserve is essential to guarantee a sufficient quantity of water in the event of an emergency, particularly for firefighting. It ensures a rapid and effective response to critical situations, in order to guarantee the safety of property and people.

The total volume of the tank is given by:

$$V_{tank} = |V_{max}|^{+} + |V_{min}|^{-} + V_{fire}$$

IV.5 Calculation of a mesh network using the Hardy-Cross method

The calculation of a mesh network is carried out by successive approximations using the Hardy-Cross method, which is based on the following two laws (**Figure 42**):

First law

The sum of the flows arriving at any node of the pipe is equal to the sum of those leaving that node.

For node A, shown in Figure 39, and for the flow direction indicated by the arrows, we have:

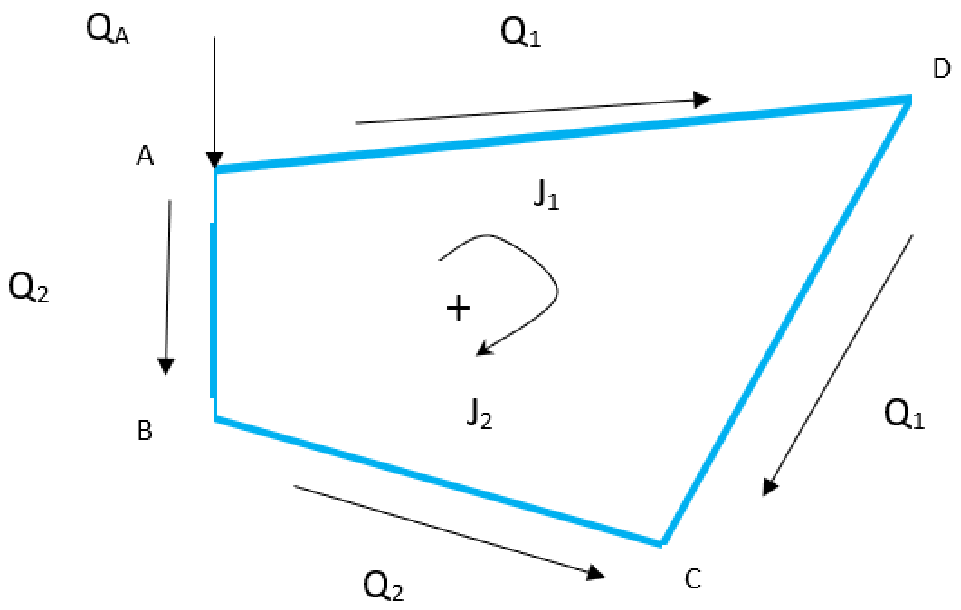


Figure 43: Example of a simple mesh

$Q_A = Q_1 + Q_2$…………………………………..(1)

where Q_A is arbitrarily decomposed into Q1 and Q2.

Let us therefore choose the diameters allowing the flow rates Q1 and Q2 to pass through.

These flow rates generate the pressure losses J1 and J2.

Second law

Along an oriented and closed path, the algebraic sum of the pressure losses is equal to zero.

$J_1 + J_2 = 0 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots . (2)$

This equality is not commonly verified the first time, and it is therefore necessary to modify the assumed initial distribution of the flow rates Q1 and Q2 in order to rectify the values of J1 and J2 accordingly.

Furthermore, as the pressure losses are proportional to the square of the flow rates, it is then possible to write that:

$$J_1 = R_1 Q_1^2$$

$$J_2 = R_2 Q_2^2$$

Here R_1 and R_2 represent the resistances of the pipes along the lengths L_1 and L_2.

If $Q1$ is modified (increased or reduced) by the quantity $\Delta Q1$, then $Q2$ must also be modified (increased or decreased) so that the sum ($Q1 + Q2$) remains constant.

Consequently, the second law applied to rectified flow rates gives:

$$R_1(Q_1 + \Delta Q_1)^2 - R_2(Q_2 + \Delta Q_1)^2 = 0$$

$$(\Delta Q_1)^2 \approx 0$$

Therefore:

$$\Delta Q_1 = \frac{-R_1 Q_1^2 + R_2 Q_2^2}{2(R_1 Q_1 + R_2 Q_2)}$$

Since:

$$R_1 = \frac{J_1}{Q_1^2} \text{ and } R_2 = \frac{J_2}{Q_2^2}$$

Then $$\Delta Q_1 = \frac{-J_1 + J_2}{2\left(\frac{J_1}{Q_1} + \frac{J_2}{Q_2}\right)}$$

$$\Delta Q_1 = -\frac{J_1 - J_2}{2\left(\frac{J_1}{Q_1} + \frac{J_2}{Q_2}\right)}$$

Therefore, the general expression is:

$$\Delta Q = \frac{-\sum J}{2\sum \frac{J_1}{Q_1}}$$

Consequently, in a first approximation, the new flow rates then become in the chosen example: (Q1+ΔQ1) and (Q2-ΔQ1)

We continue the approximations until the values of ΔQ are close to zero (particularly, ΔQ=0 when ΔQ < 0.4) and until the pressure losses on the contour are less than about 0.5 m.

Application 1

Consider an agglomeration of 1639 inhabitants. The number of years separating the reference year from the horizon year is 28. In addition, the water supply is 150 l/day/inhabitant and the population growth rate is i = 3%.

Furthermore, the school water supply is 50 l/day/student, the number of students is 240 students.

The allocation of commercial premises is 100 liters per day per premise, the number of commercial premises is n = 9. Regarding the water needs for the socio-cultural structures, there is a mosque with a water allocation of 50 liters per day per person with an average number of people of 300.

Calculate the total daily average flow

Calculate the total daily maximum flow

Calculate the maximum hourly flow (peak Q)

Solution

Evolution of the population

$$N_a = N_0(1 + i)^a$$

a = 28 years

Therefore $N_a = 1639(1 + 0.03)^{28} \rightarrow N_a = 3750\ inhabitants$

The average daily domestic flow:

$$Q_j^{Avge} = \frac{3750 \times 150}{1000}$$

$$Q_j^{Avge} = 562.5 m^3/d$$

Water needs for schools:

$$Q_j^{Avge} = N_{pupil} q$$

$$Q_j^{Avge} = \frac{50 \times 240}{1000}$$

$$Q_j^{Avge} = 12\ m^3/d$$

Water needs for commercial premises:

$$Q_j^{Avge} = N_{loc} q$$

$$Q_j^{Avge} = \frac{9 \times 100}{1000}$$

$$Q_j^{Avge} = 0.9\ m^3/d$$

Needs of sociocultural structures:

$$\boldsymbol{Q_j^{Avge} = N_{per} q}$$

$$Q_j^{Avge} = \frac{50 \times 300}{1000}$$

$$Q_j^{Avge} = 15\ m^3/d$$

Total average daily flow:

$$Q_{j\ tot}^{Avge} = 562.5 + 12 + 0.9 + 15$$

$$Q_{j\ tot}^{Avge} = 590.4\ m^3/d$$

Maximum daily flow:

$Q_j^{max} = k_j Q_j^{Avge}$; k_j= (1.1 – 1.3)

$Q_j^{max} = 1.2 \times 590.4$

$$Q_j^{max} = 708.48 m^3/d$$

Maximum hourly flow:

$$Q_h^{max} = \frac{Q_j^{max}}{24} \times k_h$$

$$Q_h^{max} = Q_p$$

Here k_h is the hourly variation coefficient:

$$k_h^{max} = \alpha \times \beta$$

Where :

$$\alpha = (1.2 - 1.4)$$

$$\alpha = 1.3$$

$$\beta_{max}(Number\ of\ inhabitants)$$

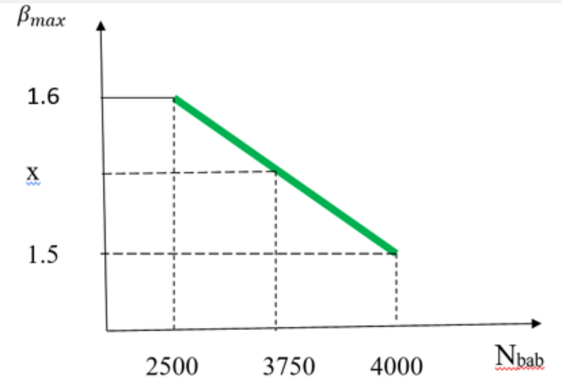
N=4000→$\beta_{max} = 1.5$

N=2500→$\beta_{max} = 1.6$

N=3750→β_{max} =?

$$\frac{1.6-1.5}{4000-2500}=\frac{1.6-X}{3750-2500}$$

X=β_{max} =1.52



$$Q_h^{max}=1.52\times1.30\times708.48$$

$$Q_h^{max}=1399.96\text{ m3/d}$$

Débits	m³/j	m³/h	l/s
Q_j^moy	590.4	24.6	6.83
Q_j^max	708.48	29.52	8.2
Q_h^max	1399.96	58.33	16.203

Application 2

Knowing that the maximum daily flow is 230.69 m^3/d, and the population is 202 inhabitants, determine the required capacity of the reservoir, while considering the hourly consumption rates provided in Table 1.

Solution

The pumped flow is given by:

$Q_p = \frac{Q_d^{max}}{24} \rightarrow Q_p = \frac{230.69}{24}$

$$Q_p = 9.61\,\frac{m^3}{h}$$

The flow consumed is then:

$Q_j^{max} = 230.69 \rightarrow 100$

$Q_c = ? \rightarrow 3.35$

$Q_c = \frac{230.69 \times 3.35}{100} \rightarrow Q_c = 7.73\, m^3/h$

The results are presented in the table below.

The capacity of the reservoir is:

$$W = 12.89 + 1.98$$

$$W = 14.76\, m^3$$

If the volume of water intended for fire fighting is taken into account, then:

$$W_{Total} = 120 + 14.76$$

$$W_{Total} = 120 + 14.76$$

$$W_{Total} = 134.76\, m^3$$

Hour	%	Q_{pumped} (Entering)	$Q_{consumed}$ (Leaving)	Q Pumped and Accumulated	Q consumed and Accumulated	Q^+	Q^-
0-1	3 .35	9.61	7.74	9.61	7.73	1.88	--
1-2	3.25	9.61	7.49	19.22	15.22	4.00	--
2-3	3.30	9.61	7.61	28.83	22.83	6.00	--
3-4	3.20	9.61	7.58	38.44	30.41	8.83	--
4-5	3.25	9.61	7.45	48.05	37.86	10.39	--
5-6	3.40	9.61	7.84	57.66	45.7	12.16	--
6-7	3.85	9.61	8.88	67.27	54.58	12.89	
7-8	4.45	9.61	10.26	76.88	64.84	12.24	
8-9	5.20	9.61	11.99	86.49	76.83	9.86	
9-10	5.05	9.61	11.65	96.1	88.48	7.82	
10-11	4.85	9.61	11.18	105.71	99.66	6.25	
11-12	4.60	9.61	10.61	115.32	110.27	5.27	
12-13	4.60	9.61	10.61	124.93	120.88	4.25	
13-14	4.55	9.61	10.49	134.54	131.37	3.37	
14-15	4.75	9.61	10.95	144.15	142.32	2.03	
15-16	4.70	9.61	10.84	153.76	153.16	0.8	
16-17	4.65	9.61	10.73	163.37	163.89		0.32
17-18	4.35	9.61	10.03	172.98	173.91		0.74
18-19	4.40	9.61	10.15	182.59	184.06		1.28
19-20	4.30	9.61	9.92	192.2	193.98		1.59
20-21	4.30	9.61	9.92	201.81	203.90		1.90
21-22	4.20	9.61	9.69	211.42	213.6		1.98
22-23	3.75	9.61	8.65	221.03	222.24		1.02
23-24	3.70	9.61	8.53	230.64	230.64		0.00
Sum	**100%**	**230.69**	**230.64**				

Application 3

The drinking water distribution network defined by the diagram below shows the length of each section of the pipe. If the maximum hourly flow is 200 m^3/h and the concentrated flow at node C is 30m^3/h (K = 10-4), then:

Calculate the corrected flow of each section.

$$Q = \frac{200 \times 1000}{3600} = 55.55 l/s$$

Calculation of total route flow:

$Q_r = Q_p - \sum Q_c$

$$Q_{Concentrated\ C} = \frac{30 \times 1000}{3600} = 8.33 l/s$$

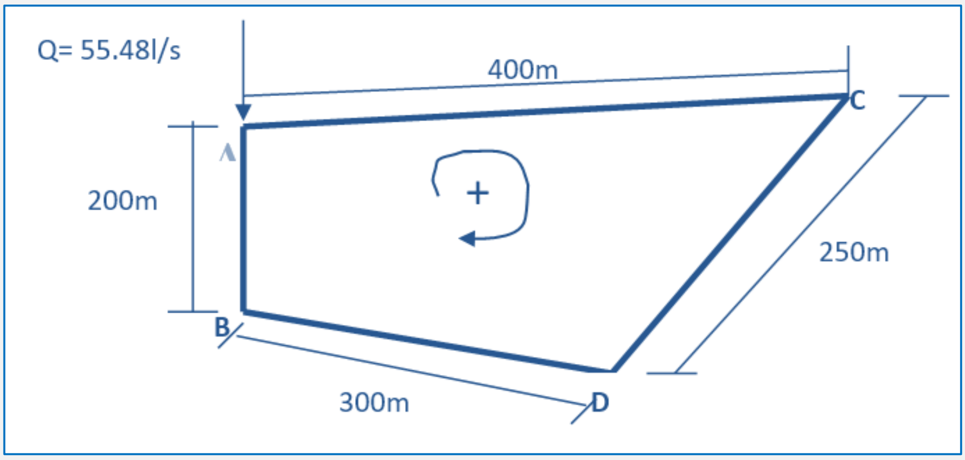
$Q_r = 55.55 - 8.33$

$Q_r = 47.2166$ l/s

Calculation of the specific flow

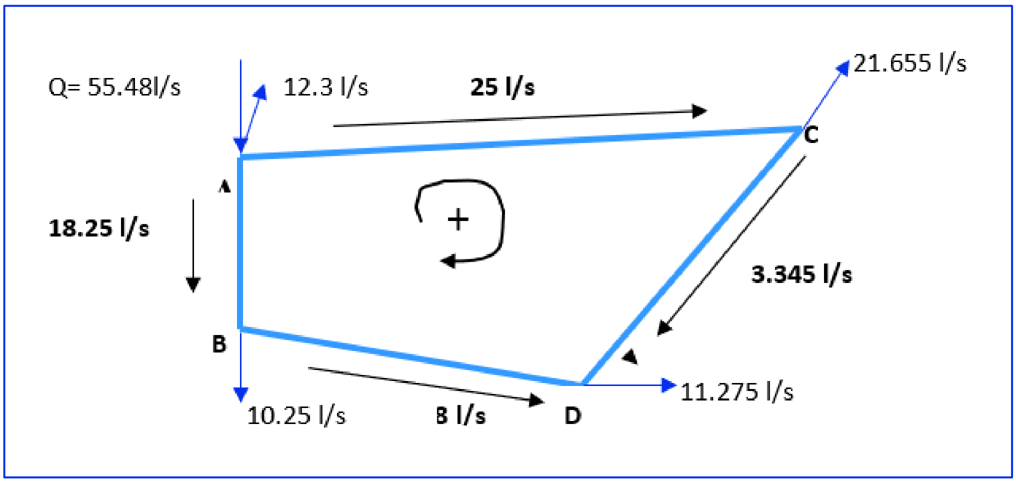
$Q_{sp} = \frac{Q_r}{\sum L_i}$ avec $\sum L_i = 400 + 200 + 300 + 250$

$\sum L_i = 1150$ m



$$Q_{sp} = \frac{47.21}{1150} = 0.041 l/s/ml$$

N° nodes	N° section	L (m)	Q_{sp} (l/s/ml)	Q_r (l/s)	Q_c	$Q_n = 0.5\sum Q_r + Q_c$ (l/s)
A	**AB**	200	0.041	8.2		12.3
	AC	400	0.041	16.4		
B	**BD**	300	0.041	12.3		10.25
	BA	200	0.041	8.2		
C	**CA**	400	0.041	16.4		21.655
	CD	250	0.041	10.25	8.33	
D	**DB**	300	0.041	12.3		11.275
	DC	250	0.041	10.25		



Calculation of the corrected flow

$$Q_c = \frac{-\sum J}{2\sum \frac{J}{Q_0}}$$

N° mesh	Section	Q_0 (l/s)	D_0 (mm)	J (10^{-3})	L (m)	J (m)	J/Q_0 (10^{-2})	Δq	Q_c
I	AB	-18.25	150	7.803	200	-1.5606	8.551	0.76094	-17.489
	AC	+25	200	3.34	400	+1.336	5.344	0.76094	+25.760
	CD	+3.345	80	8.353	250	+2.0882	62.428	0.76094	+4.106
	BD	-8	100	12.453	300	-3.7359	46.698	0.76094	-7.2394
						∑J =-1.87225	**∑J/Q_0=1.2302**		

J (10^{-3})	J (m)	J/Q_0 (10^{-2})	Δq	Q_c
7.165	-1.433	8.19	0.147	-17,342
3.546	+1.418	5.506	0.147	+25,907
10.998	+2.749	66.963	0.147	+4,253
10.336	-3.100	42.821	0.147	-7,0924
	∑J =-0.3651	**∑J/Q_0= 1.23021**		

Application

The drinking water distribution network (branched network) defined by the diagram below shows the number of inhabitants for each section of pipe. The total number of inhabitants is equal to 17700.

$\beta = 1.2$,

Water supply: q = 250 l/inhab./day

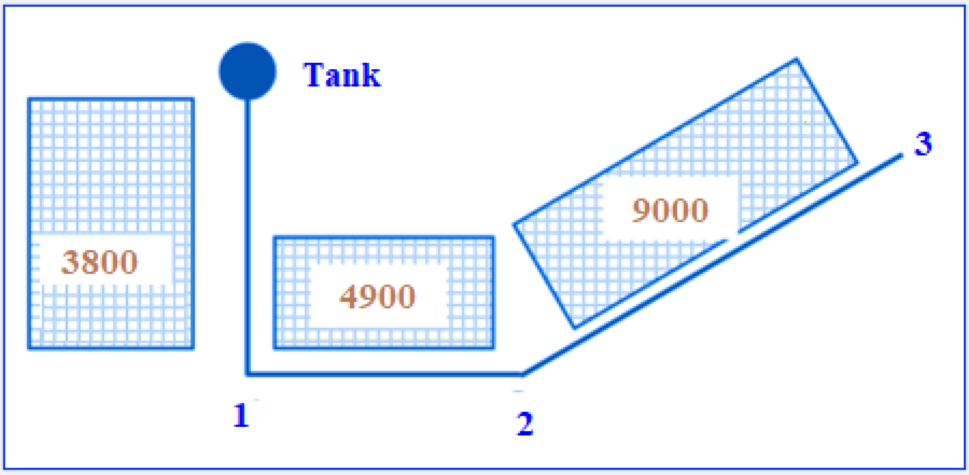
$K_j = 1.3$,

$\alpha = 1.3$,

Pipe lengths:

$L_{R-1} = 350$m, $L_{1-2} = 400$m, $L_{2-3} = 700$m.

- Determine the equivalent flow rate of each pipe section of the branched network.
- Calculate the diameter, pressure loss, and speed for each segment of the pipe by referring to the pressure loss tables. The values in the diagram below represent the number of inhabitants.



Solution

The daily flow is:

$Q_j^{Avge} = 17700 \times 250$

$Q_j^{Avge} = 4425 \text{ m}^3/\text{day}$

The maximum daily flow is:

$Q_j^{max} = Kj \times Q_d^{moy}$

Kj= 1.3

$Q_j^{max} = 1.3 \times 4425$

$Q_j^{max} = 5752\ m^3/day$

The maximum hourly flow is:

$Q_h^{max} = (K_h \times Q_j^{max})/24$

$K_h = 1.2 \times 1.3$

$K_h = 1.56$

$Q_h^{max} = (1.56 \times 5752)/24$

$Q_h^{max} = 373{,}9125\ m^3/h$

$Q_h^{max} = 103.86\ l/s$

The specific flow is:

$Q_{sp.} = (Q_h^{max} / N_{inhab.total})$

$Q_{sp} = (103.86/17700)$

$Q_{sp} = 0.005868\ l/s/inhab.$

The route flow is:

$Q_r = Q_{sp} N_{inhab}$

The equivalent flow is:

$Q_{eq} = Q_{downstream} + 0.55 Q_r$

Calculation of pressure losses :

$J = j \times L$

$J = RQ^2$

J is the pressure loss per meter of the pipe length (See the pressure loss tables)

$$\begin{cases} j = RQ^2 \\ j = j' (Q^2 / Q'^2) \end{cases}$$

$j' = RQ'^2$

Summary table of equivalent flow rates for each section

Section	N_{inhab}	Q_{sp} *(l/s/inhab)*	Q_r*(l/s)*	$Q_{upstream}$ *(l/s)*	$Q_{downstream}$ *(l/s)*	Q_{eq} *(l/s)*
R-1	3800	0.005868	22.2984	103.86	81.5616	93.82572
1-2	4900	0.005868	28.7532	81.5616	52.8084	68.62266
2-3	9000	0.005868	52.812	52.8084	0	29.04462

Summary table of characteristics, such as the diameter, speed and pressure loss, for each section

Section	Q_{eq} *(l/s)*	*Ø(mm)*	*V (m/s)*	$j*10^{-3}$ $(K=10^{-4})$	*L (m)*	*J (m)* $(K=10^{-4})$
R-1	93.82572	300	1.3051	0.005397	350	1.889
1-2	68.62266	300	1.4507	0.007419	400	2.9617
2-3	29.04462	200	1.4609	0.004411	900	3.9699

Part 2

Wastewater disposal system

Chapter V : Sanitation

V.1 Definition of sanitation

Sanitation consists of collecting all wastewater, whether domestic, industrial or rainwater, and directing it to natural discharge points, while respecting public and environmental health standards. It is important to emphasize that before this wastewater is discharged into the natural environment, it is treated in treatment plants to ensure that it complies with the provisions of the receiving ecosystems, such as rivers, lakes or sea. This treatment phase is essential because it helps to reduce the risks of waterborne diseases, especially in urban areas, and also to protect the quality of aquatic environments. The main objective of collective sanitation is to gather all wastewater from an urban area and send it to a treatment plant.

V.2 Definition of a sanitation network

A sanitation network involves all the pipes, equipment and structures that are intended to collect, transport, and treat wastewater and rainwater from a given urbanized area. There are two categories of sanitation networks: the collective sanitation network and the non-collective sanitation network.

The collective sanitation network, also known as sewer, is specifically designed for urban areas. It is used to collect and transport waste and rainwater from densely populated areas, businesses, industries, as well as public infrastructure such as mosques, schools, hospitals and Hammams (traditional baths). On the other hand, a non-collective sanitation is mainly intended for rural regions. It controls the evacuation of wastewater from housing that is not connected to collective sanitation networks, typically via the use of septic tanks.

Furthermore, sewer networks are designed only to evacuate runoff water as well as roof water, which is discharged into gutters, and sewage water which is collected in septic tanks. This could cause severe health risks. With the evolution towards modern sewerage, all wastewater is first purified in a treatment plant and is then transported to a receiving environment. On the other hand, runoff water can either be evacuated separately from wastewater or mixed.

V.2.1Components of a sewer network

A sewer network is mainly composed of interconnected pipes in which hydraulic parameters, such as diameter and water velocity, can vary at different points of the network (**Figure 43**). To facilitate the description and calculation of the network, the concepts of bifurcation points are then introduced. These are points where the different behaviors meet. It should be noted that branches or sections can be found between two bifurcation points.

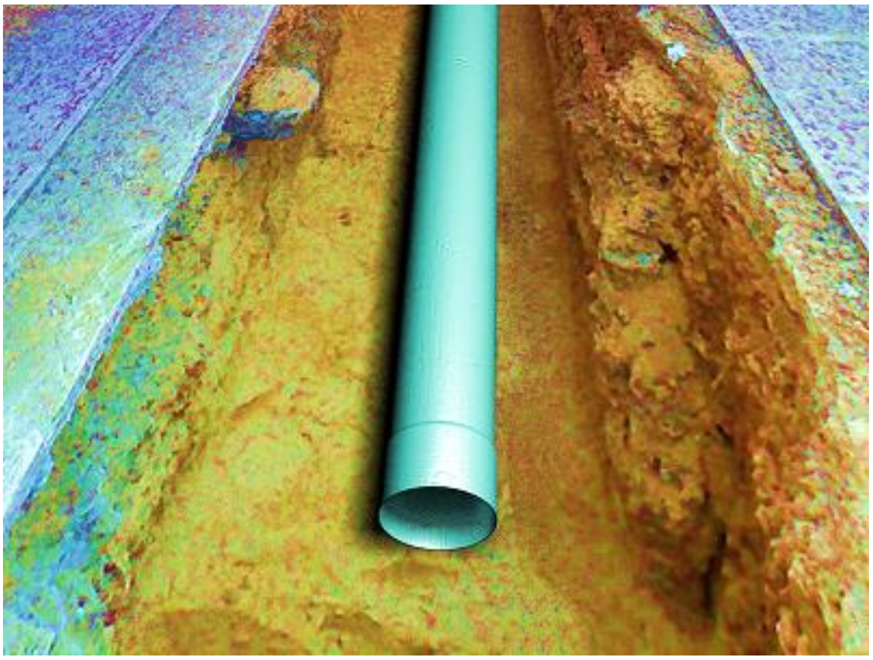


Figure 44: Sewer pipe

V.2.2 Types of sewer pipes

There are generally five types of pipes in a sewer system:

1. Local Sewer Line - As the name suggests, this line is intended to carry wastewater from one single area or from a small group of streets.

2. The collector - It carries wastewater from several local sewer pipes or several collectors. It constitutes the main evacuation axis of a watershed.

3. The interceptor - This pipe receives all or only part of the water conveyed by the collectors and transports it to the treatment plant. It must normally be buried deep enough in order to capture the water intended for it by gravity.

4. The outfall - This pipe is used to evacuate wastewater to the receiver.

5. The sewer manhole - This is a work of primary importance in a sewer network because it allows access to a pipe to carry out maintenance tasks (**Figure 44**). It also ensures ventilation in the network, thus facilitating the evacuation of gases, some of which are toxic and explosive (H_2S, NH_3, CH_4, CO_2, etc.).

A sewer manhole must be installed in the following situations:

- At a point where the diameter or the slope of a pipe has to be changed.
- At the junction of pipes oriented in different directions.
- At the start of a network, that is to say at the head of the section located furthest upstream of a local sewer.
- At the junction of two pipes located at significantly different depth. In addition, in the event of a significant level difference, the installation of a drop manhole is necessary.

Typically, manholes are separated by a distance between 40 and 50 meters.

Drop or fall manhole

A drop manhole is installed when the invert of the pipe, which carries the wastewater towards the sewer manhole, is located more than 600mm from the crown of the pipe that evacuates this wastewater (**Figure 45**).

Figure 45: Sewer manhole and its access chimney

(**Source**: Quebec Standardization Bureau - Water and sewer pipes - Quebec Publications, NQ 1809-300, 1987, P. 75.)

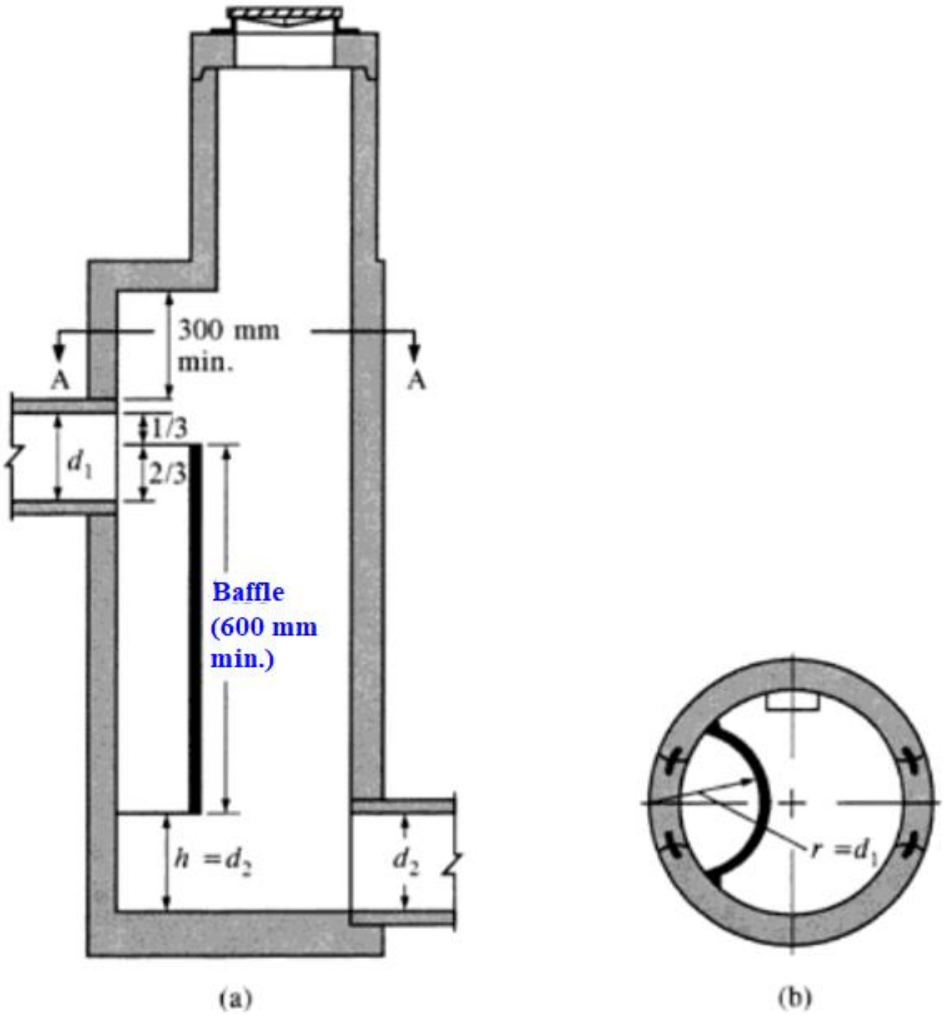


Figure 46: Drop manhole: (a) Vertical cut view; (b) Horizontal cut view (A-A)

(Source: Quebec Standardization Bureau - Water and sewer pipes - Quebec Publications, NQ 1809-300, 1987, P. 58)

6. Connection boxes, which are located on sidewalks, connect users to the urban network.
7. The manhole, also called a street catch basin, collects runoff water from the edges of the streets and directs it to the storm sewer or combined sewer. It is positioned in areas presenting a risk of stagnation of rainwater. It generally consists of the three following elements (**Figure 46** and **47**):
    - ***The drain***, an opening equipped with a selective grid located on the road, allows the passage of water while retaining coarse solids such as leaves, branches and debris.
    - ***The well***, also known as a *Cunette*, can be direct passage or decantation.

- *The connections to the collector*, which connect users to the urban network, are located on the sidewalks.

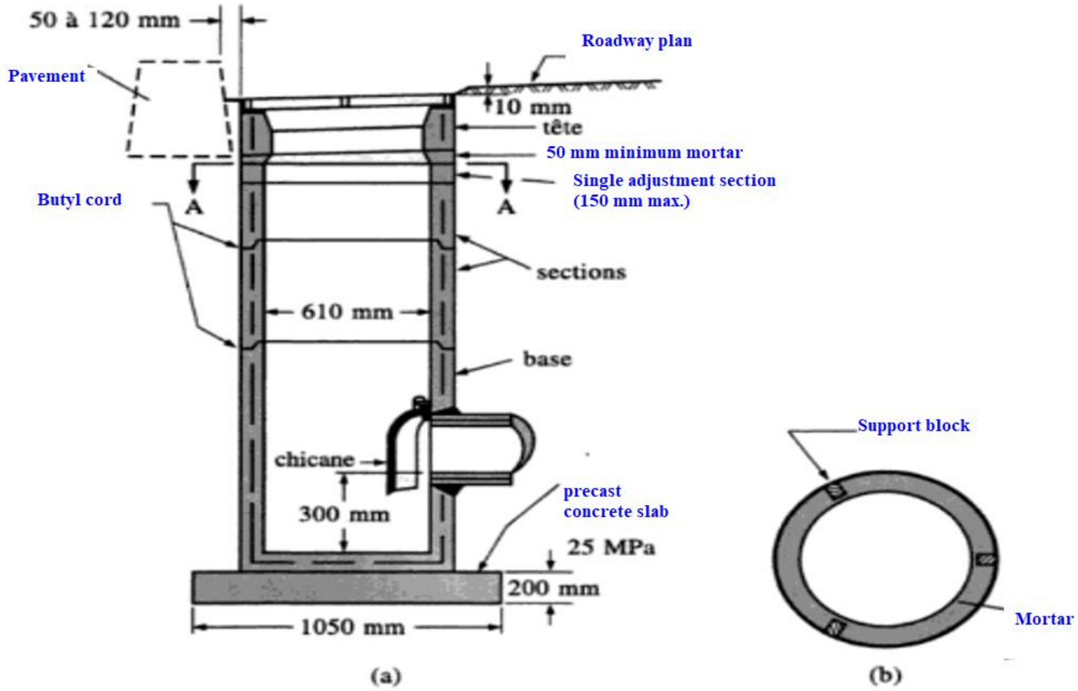


Figure 47: Drop manhole : (a) Vertical cut view ; (b) Horizontal cut view (A-A)

(Source: Quebec Standardization Bureau - Water and sewer pipes - Quebec Publications, NQ 1809-300, 1987, P. 78)

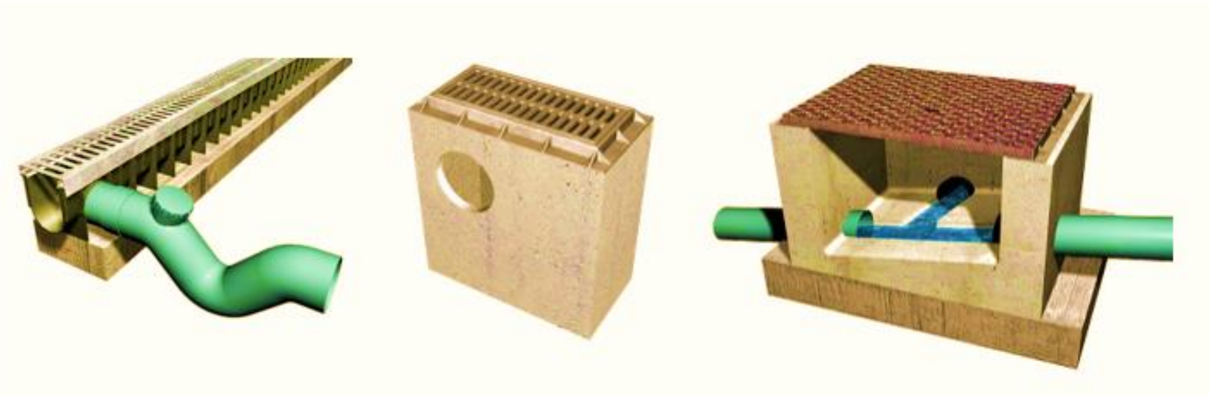


Figure 48: Solid concrete manhole, cast “in situ”

8. Storm overflow - Wastewater treatment plants can, generally speaking, only accommodate triple or, at most, quadruple the dry weather flow without, however, dropping the speed of the flow to limit settling suspended solids present in the effluent. The storm overflows, which are installed along the collectors, are therefore intended to allow a fraction of the storm flow to pass towards a natural outlet.

Figure 48 : Effluent exiting a wastewater discharge pipe

https://www.aquaportail.com/definition-4163-effluent.html

Effluent means a waste fluid, whether treated or not, originating from agricultural, industrial, or urban sources, and discharged, directly or indirectly, into the environment from a natural body of water or structure human. Wastewater is one example of an effluent (**Figure 48**).

This discharge generally concerns the slice of water greater than the sum of the flow of wastewater and that of a small rain. Storm overflows also prevent the entry of water from the natural environment (**Figure 49**). A storm spillway consists of a diversion structure that receives water from an upstream collector and then returns it to the downstream collector. It directs the excess water to a discharge collector. Spills can be directed to storm or decontamination ponds, or directly into the natural environment, such as rivers and bodies of water.

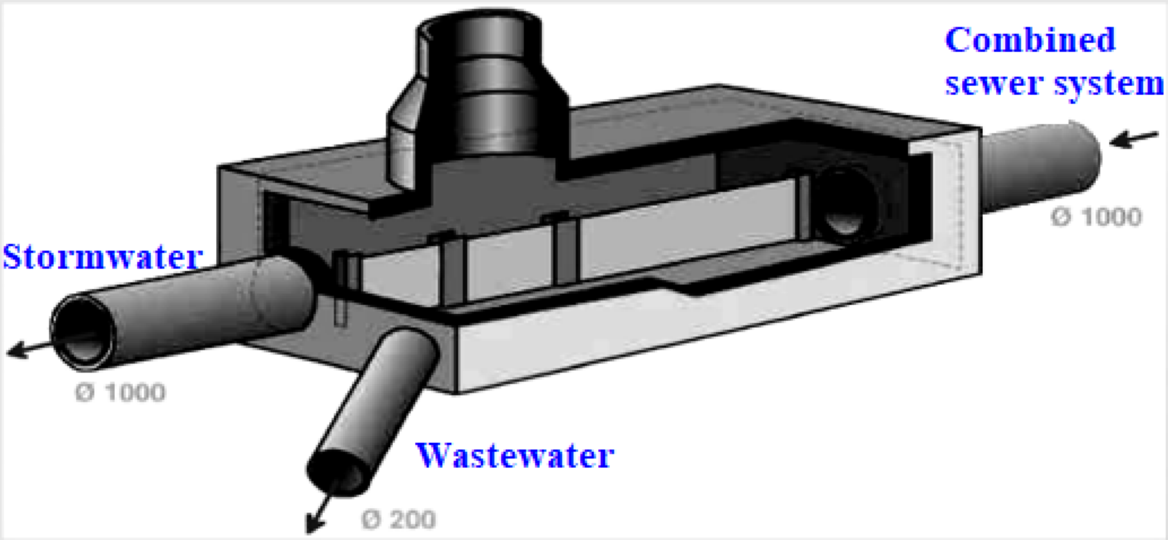


Figure 49: Diagram of the storm spillway.

V.2.3 Sewer pipe manufacturing materials

Sewer pipes can be made from a variety of materials. The choice of these materials depends on several factors:

- The roughness of the material,
- The lifespan of the material and elements constituting the sewer network,
- The resistance of the material to erosion, abrasion, acids, bases, gases, solvents, and others.
- The Ease of handling and installing pipes,
- The resistance of the pipe to the different loads it is subjected to,
- The sealing of joints,
- The availability of special connection parts.
- The purchase costs

The durability of a sanitation network actually depends on the materials chosen for its construction, as well as on the quality of its installation and maintenance. Among the materials most used for the construction of sewer pipes, one can mention:

- *Asbestos cement* - Although this material was widely used in the past for its strength and low cost, it is now less common due to the health risks associated with exposure to asbestos.

- *Unreinforced concrete* - This type of concrete is economical and easy to manufacture, but less resistant to loads and tensions than reinforced concrete.
- *Reinforced concrete* - This concrete, which can be prefabricated or cast on site, offers better resistance to loads thanks to the incorporation of reinforcing steel bars. However, it is more expensive than unreinforced concrete.
- *Cast iron* - It is very resistant to corrosion and can withstand heavy loads, but its cost is relatively high.
- *Plastic* - Plastic pipes, such as PVC or HDPE, are lightweight, corrosion resistant and easy to install. They are often used in new installations due to their longevity and affordability.

Each material has its own specific areas of application which depend on the environment, type of soil, presence of chemicals, temperature and other factors. It should also be noted that implementation and maintenance costs vary, which influences the final choice of material for a given project.

A priori, there is no interest in establishing deeply buried structures due to the increased cost price and the risk of sinking into the water table. However, a minimum depth must be respected in order to:

- Enable correct construction of particular connections whose slope must not, in principle, fall below 3%, in order to avoid the risk of waterlogging,
- Avoid any risk of sewers being crushed under the effect of rolling loads. In practice, the depth of the laying trenches commonly reaches 2 meters,
- Avoid any risk of intercommunication between the sanitation network and a neighbouring drinking water supply pipe. The sanitation works must always be established at a level lower than the drinking water supply.

V.3 Sewer network diagram

V.3 .1 Perpendicular equipment diagram

In this equipment diagram, which is typical of rainwater networks in a separative system, the flow takes place directly in the watercourse. This diagram may also be used as a unitary system if no treatment is necessary or if the watercourse used temporarily as a collector has a run-of-river treatment station downstream.

V.3 .2 Lateral movement scheme

It is possible to modify the perpendicular route in order to direct all the water towards a single purification point P1, by installing a lateral collector along the river. If the slope of the river is sufficient, this collector can be realized at lower cost. Otherwise, it can be quite expensive because, in this case, it requires the adoption of a transverse or oblique collector.

V.3 .3 Transverse or oblique collector

When the slope of the river is insufficient, an oblique collector can be installed, which makes it possible to benefit from the natural slope of the land towards the river. This allows increasing the slope of the collector.

V.3 .4 Level collector

When the slope is low with an extensive settlement along the river, it may be necessary to carry out sanitation in several levels, thus multiplying the purification points and increasing the overall cost.

V.3.5 Radial collector

On a flat terrain without relief, it is necessary to create a slope for the sewers by adjusting the depth of the trenches. The radial sewers then converge towards a central point (P) from which the wastewater is pumped and conveyed under pressure to a suitable outlet.

V.3 .6 multi-radial network

For a city extending over a horizontal plain, it is necessary to multiply the lifting stations (5p). This makes the multi-radial network particularly expensive.

It is worth emphasizing that the objective of the lifting stations is to raise the level of wastewater, i.e. to install the pipes at a shallower depth while maintaining a sufficient slope. This elevation process is also used in low areas in order to reach the network located above. Water lifting stations are sensitive points in the network. Indeed, pumps, which are electromechanical equipment subject to breakdowns, require regular maintenance. Any interruption of a lifting station risks causing, through the rise of effluent, a wastewater overflow at the station itself. If this occurs upstream in the pipeline, it may present a major health risk.

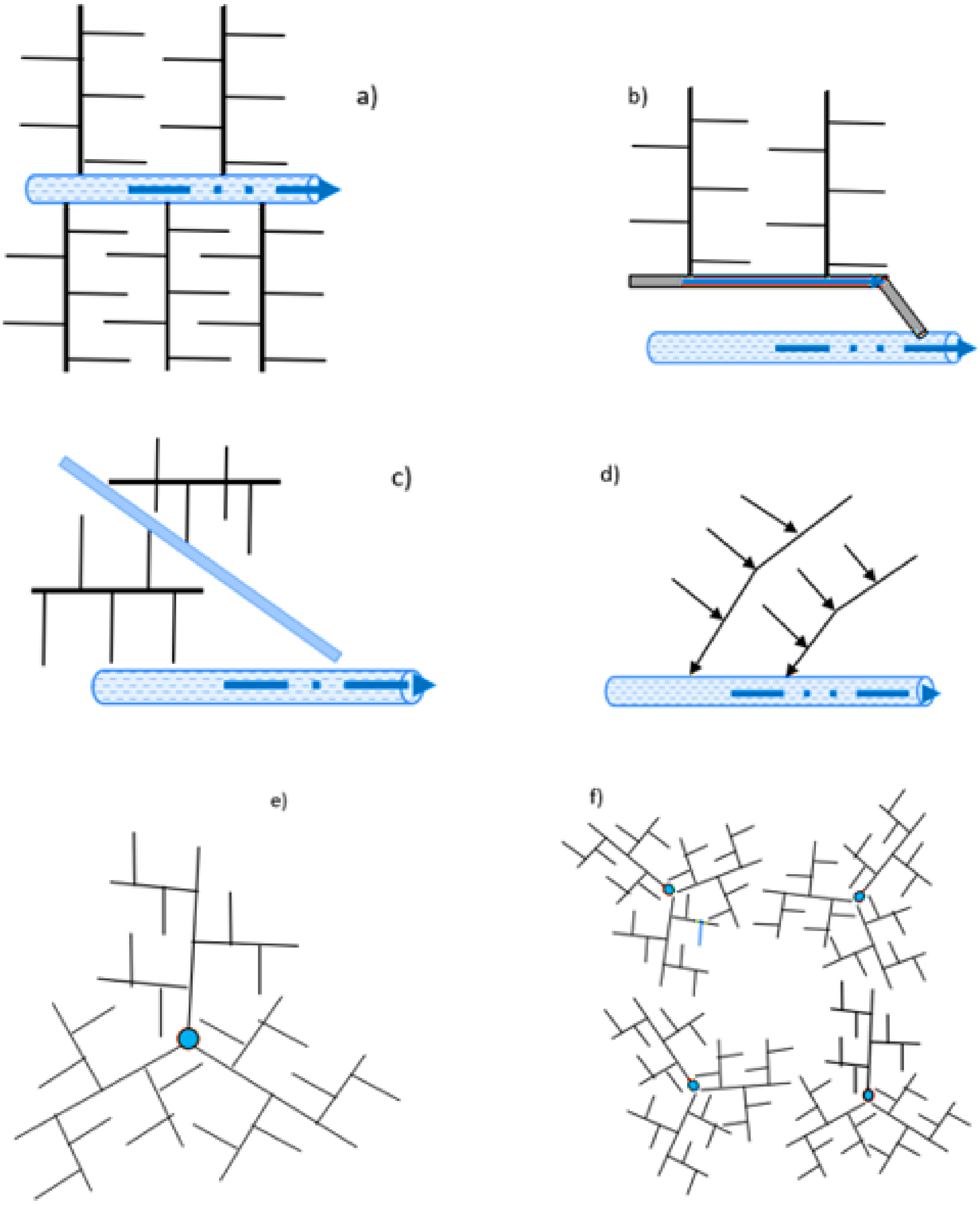


Figure 50: Types of sewerage network diagram

V.4 Wastewater disposal systems

The wastewater and rainwater evacuation network allows rapid evacuation and prevents stagnation of water. It thus ensures health protection of urban areas, fight against flooding, and protection of the environment against pollution. Sewerage systems in large cities generally come in three forms, namely the separative system, the unitary system, and the pseudo-separative system. The choice of an evacuation system is based on several factors, including (**Figure 50**):

- The design techniques,
- The economic aspect,
- The impact of the system on the natural environment.

V.4.1 Unitary network

The unitary network, also called all-in-sewer, has the role of collecting wastewater and rainwater in the same pipe. In this case, it is necessary to install a storm overflow downstream of the network to allow direct discharge, by overflow, of part of the water into the receiving environment during significant rain events. The unitary network is recommended when the separating network is not economically viable (**Figure 50**).

This type of network is better suited to high-density urban environments. However, it presents some self-cleaning problems in dry periods, as it requires maintenance interventions to prevent the accumulation of sediment and debris.

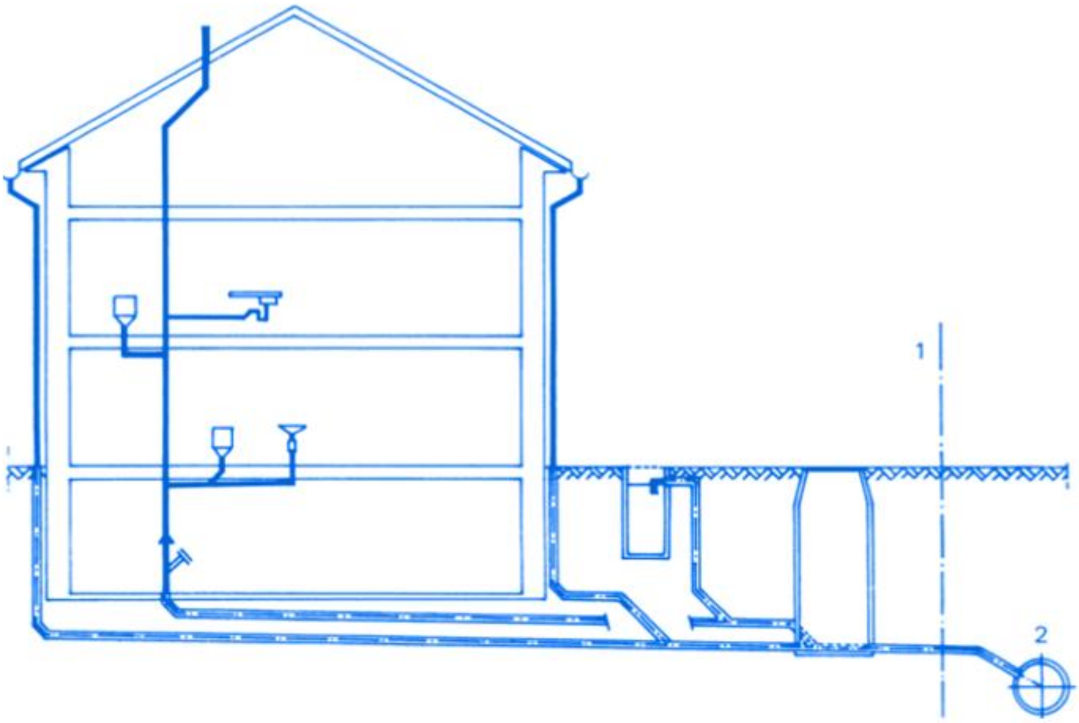


Figure 51: Unitary system (Vittone, 2010)

V.4.1.1 Advantages

Economically, the unitary network is less expensive than the separative system because the evacuation of wastewater and rainwater is ensured by one single pipe. Maintenance of the unitary network is also more economical because rainwater makes its self-cleaning easier. In addition, these waters prevent the accumulation of fecal matter and waste in the sewer system. Moreover, the large pipe diameter promotes ventilation and self-cleaning of the network.

V.4.1.2 Disadvantages

The capacity of the treatment plants does not make it possible to treat all the flows from rainwater and, consequently, this requires the installation of rainwater separation systems before their arrival at the treatment plant. This task is ensured by storm overflows. It is worth indicating that the discharge from the spillways is polluting, especially during the first minutes of the storm. Its effects can be limited by installing stormwater treatment basins. However, certain types of contamination can reach the receiving environment.

Extra costs, linked to the installations necessary for the degradation of organic matter and the dilution of urban solids, such as stormwater treatment basins and the wastewater treatment plant, can be added.

V.4.2 Separative network

In a separative network, rainwater and wastewater are evacuated in two separate pipes. This network is preferable in small and medium-sized towns, as well as in extensions of large cities, because it allows better management of different types of water. It also helps reduce the risk of pollution (**Figure 52**).

V.4.2 .1 Advantages

- The purification regime is more regular because the rains do not alter it.
- Wastewater and rainwater do not mix, thus avoiding discharge of polluted water.
- Water treatment costs are generally lower because the treatment process is simplified.

V.4.2 .2 Disadvantages

- Network maintenance and cleaning costs are high due to the management of two separate systems.
- Stormwater from urban areas requires minimal treatment even if it does not mix with wastewater. The reason is that stormwater can include a lot of pollutants.

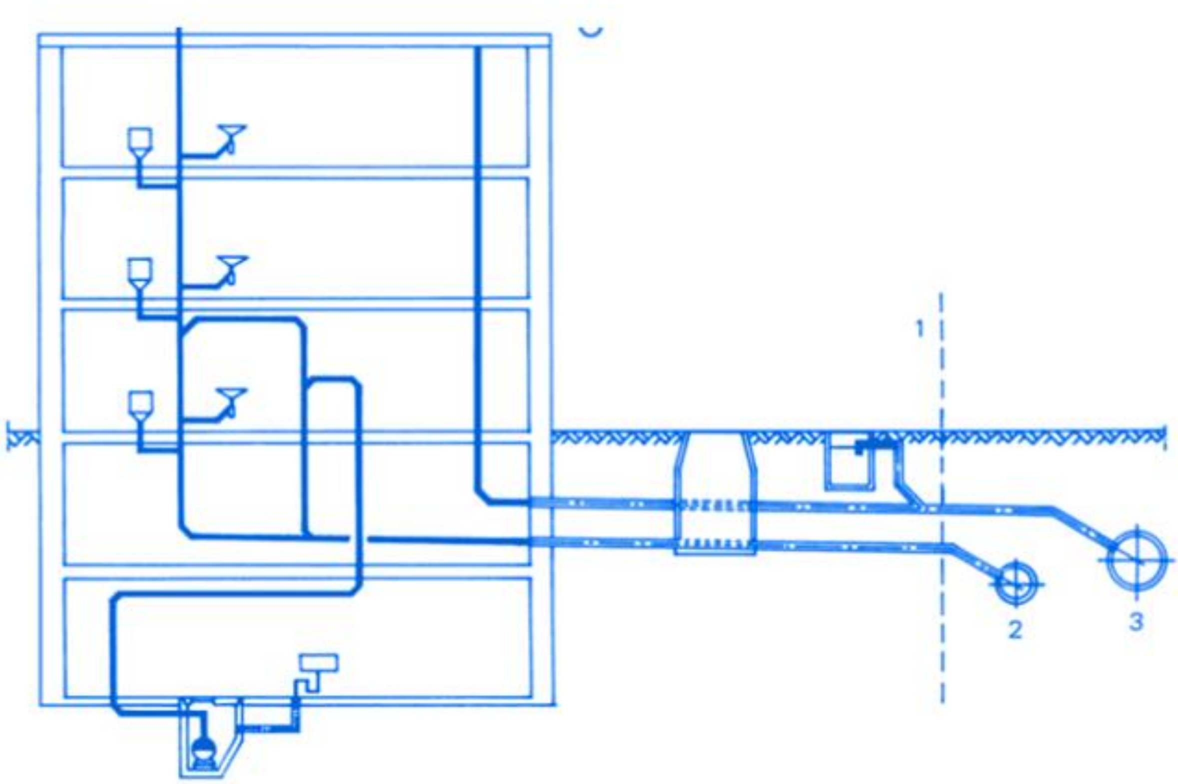


Figure 52: Separative system (Vittone, 2010)

V.4.3 Pseudo-separative network

This network ensures the evacuation of wastewater with a portion of rainwater coming from terraces and courtyards. Runoff water is discharged directly into the natural environment through gutters and ditches (**Figure 53**).

The pseudo-separative network ensures the evacuation of domestic wastewater as well as part of the rainwater, generally that coming from terraces and watercourses. This system makes it possible to reduce the load on wastewater treatment plants by evacuating part of the rainwater directly into the natural environment through gutters and ditches.

V.4.3 .1 Advantages

- Reduction of the hydraulic load on treatment plants since part of the rainwater is evacuated separately.
- Compared to the unitary network, there is a lesser impact on the treatment of wastewater during rainy events.
- Installation and maintenance costs are potentially reduced compared to those of a complete separative network because the pseudo-separative network requires fewer pipes.

V.4.3 .2 Disadvantages

- Risk of pollution of the natural environment by runoff water which may contain pollutants.
- Complex stormwater management, which requires careful design to avoid overflows and contamination.
- Regular maintenance of gutters and ditches is required to ensure their proper functioning and avoid obstructions.

It should be noted that this type of network is particularly suitable for areas where the complete separation of waste and rainwater is not possible or is not economically viable. This network requires careful planning and management in order to balance cost and efficiency benefits with the environmental imperatives.

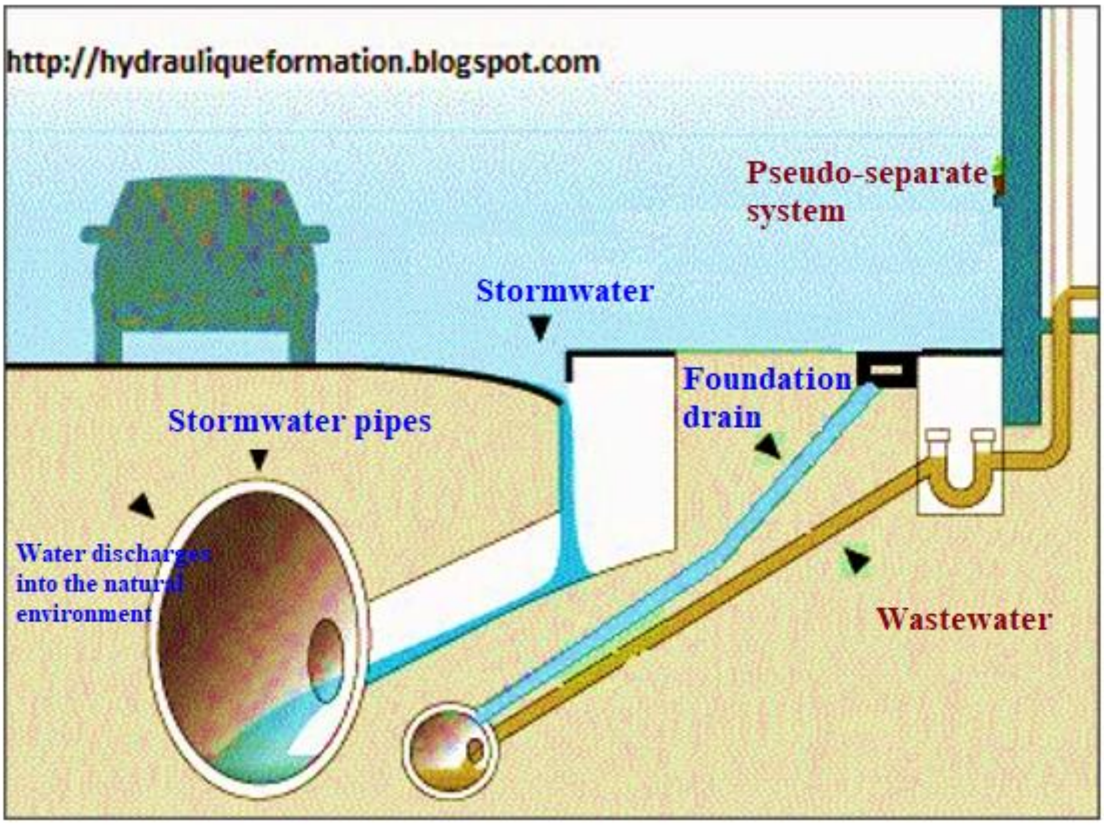


Figure 53: Pseudo-separative network

V.5 Method for assessing the flow rates of water to be evacuated

V.5.1 Determination of flow rates in urbanized watersheds using the superficial method

Water evacuation structures are designed for precipitation whose occurrence probability is determined by probabilistic methods. In hydrology, rain events are considered random events. A rainy event is characterized by its excess frequency (F) or its return period (T), which is the inverse of the frequency (T=1/F).

The return period (T) is often used to design evacuation systems capable of handling rain events of a specific intensity, which are only likely to occur once every T years on average. For example, an event with a return period of 100 years is an event that has, in theory, a 1% chance of occurring each year.

Application 1

Consider that a rainy event, with a height h = 20 mm, occurred 5 times, in 24 hours, over 50 years.

Determine the probability of exceedance and the return period of this event.

Solution

Probability of exceedance:

$$p = \frac{5}{50} = 0.1$$

Return period:

$$T = \frac{1}{F} = \frac{1}{0.1} = 10\ years$$

The return period of a 20 mm 24-hour precipitation event in Oran is 10 years, which subsequently allows concluding that the 10-year, 24-hour precipitation is 20 mm.

The probability of exceeding p is equal to the reciprocal of the return period T, that is to say:

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

Here T > 1.

Probability is a dimensionless number that is always greater than zero and less than one.

If the precipitation during 24 hours is 20 mm over a period of 10 years, then there is a probability of 1/10 = 0.1 =10% that a height of 20mm or more occurs in a 24-hour period in a given year. In other words, if a 10-year rainfall event has already been exceeded this year, then the probability that it will be exceeded again next year is still 10%.

V.5.2 Estimation of hydrological risk

Determining the capacity of structures for rainwater management is generally based on rainfall events. However, due to the probabilistic nature of precipitation, there is always a risk that the capacity of the structure will be exceeded.

This risk is defined as the probability that the rainy event will be exceeded once or more during the lifespan of the structure. The hydrological risk can then be calculated as follows:

$$J = 1 - \left(1 - \frac{1}{T}\right)^{N}$$

Here J is the hydrological risk, T is the return period of the event which serves as a basis for the design of the structures, and N is the lifespan of the storm structure.

Application 2

A road culvert is designed to withstand a storm that occurs once every 25 years. Determine the hydrological risk of this design if the expected life of the culvert is 30 years.

Solution

The hydrological risk is:

$$J = 1 - \left(1 - \frac{1}{25}\right)^{30} = 0.71 = 71\%$$

V.5.3 Definition of culverts

Culverts are constructions that allow the free circulation of runoff water or a watercourse under a structure dedicated to circulation (**Figure 54**). They are part of the structures that allow the management of rainwater in an area. They have an important role since they help reduce the risk of flooding and allow traffic on public roads (Valois, 2008).

Figure 54: Pierreville culvert after completion of work. Source:

Application 3

Consider the culvert in the example given above. What return period should be used as a basis for the design of the structure if the permitted hydrological risk is 0.10 (or 10%)?

Solution

The hydrological risk is given as:

$$0.1 = 1 - \left(1 - \frac{1}{T}\right)^{30}$$

Then, solving for T, we obtain T = 285 years. In practice, for economic reasons, it is preferable to use a return period of 2 to 50 years in the design of rainwater structures which allow hydrological risks well above 10%. Therefore, flooding a stormwater structure, such as a culvert or street entrance, does not always mean that the structure was poorly designed.

V.6 Estimation of rainwater flows

Rainwater consists of water from rain runoff on plots of land, whether built on or not, as well as surface water runoff. Waterproofed surfaces, such as urban paving, result in increased stormwater runoff, which leads to an increased volume of water flowing and, therefore, an increased risk of flooding downstream. This situation can endanger the safety of populations, their property, as well as the integrity of the receiving environment.

V.6 .1 Superficial method

V.6 .1 .1 Intensity-Duration-Frequency (IDF) curves

The IDF curves illustrate the variation of the maximum average rain intensity as a function of its duration, for short-term precipitation (less than 3 hours, and often, even 1 hour) and for various return periods. **Figure 57** presents the IDF curves established based on the analysis of rainfall recorded over a long period, i.e. from 1971 to 1999, at a rainfall station located in the Tafna watershed. The most common recurrence intervals are 25 years, 15 years, 10 years, 2 years, 1 year, 6 months, and 3 months. Several formulas can be used to plot IDF curves.

The Montana formula is the most frequently used. It was established in 1904 by Professor Talbot. It is expressed as follows:

$$i_M(t,T) = a(T).t^{b(T)} \text{et} h = at^{(1-b)}$$

With $i_M(t,T)$is the average maximum intensity over the duration *t*, with a return period *T*, *a* and *b* are two numerical adjustment parameters that depend on *T*, and *h* is the amount or height of rain (mm).

V.6 .1 .2 Estimation of Montana parameters

The Montana coefficients are necessary for the calculation of water flows. The Montana model makes it possible to estimate the average intensity of the rain *i (d, T)*, (or the precipitation height h(d, T)) over a duration d, as a function of two parameters, i.e. *a(T)* and *b(T)*, which both depend on the return period *T*, the inverse of the frequency.

Montana's formula therefore makes it possible to make a relationship between the rain intensity *i(t)* collected during a rainy episode and its duration *t*, through the expression:

$$\boldsymbol{i}(\boldsymbol{t}) = at^{-b}$$

The Montana law described by the Technical Instruction of 1977 (Ministry of Equipment and Territorial Planning, 1977) is given by:

$$\boldsymbol{i}(\boldsymbol{t}) = \boldsymbol{a}\boldsymbol{t}_{\boldsymbol{c}}^{\boldsymbol{b}}$$

Here *a* and *b* are the Montana coefficients and *tc* the concentration time (minutes).

The Montana coefficients (a, b) are calculated by a statistical adjustment between the durations and intensities of rain, for a given return period T.

V.6 .1.3 Gumbel's law

V.6 .1.3.1 Statistical adjustment of a rain shower – Gumbel’s law

The frequency analysis of a long series of maximum annual precipitation amounts makes it possible to estimate the return time of a particular precipitation value. The Gumbel statistical distribution (double exponential law or Law of Extreme Values type I) is among the statistical distributions which allow this analysis to be carried out. The Gumbel model is a frequency model which makes it possible to estimate the probability of occurrence of an event of a given value.

The distribution function of Gumbel’s law F(x) is written as follows:

$$F(x) = exp\left(-exp\left(-\frac{x-a}{b}\right)\right)$$

Setting $u = \frac{x-a}{b}$ with u a reduced variable, we can then write:

$F(x) = exp\big(-exp(-u)\big) \Rightarrow u = -ln(-ln(F(x)))$

Gumbel's law is a law with 2 parameters *a* and *b* which have the same dimension as x.

The probability of non-exceeding *F(xi)*, assigned to each value xi, can be determined by numerous empirical frequency formulas which consist of sorting the series into increasing values and assigning a rank *r* to each value. Hazen's empirical frequency formula is the most used in the case of the Gumbel distribution. It is given by:

$$F(x) = \frac{r-0.5}{n}$$

Here *r* is the rank in ascending order in the data series, and *n* is the sample size.

As previously stated, the return time T of an event is defined as the inverse of the frequency of appearance of the event, which allows us to write:

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

Additionally, the reduced variable can be used to estimate the following linear quantile:

$$x_q = a + bu_q$$

Its density can then be written as: $f(x) = \frac{1}{b}\exp\left(-\frac{x-a}{b}\right).exp\left(-exp\left(-\frac{x-a}{b}\right)\right)$

The maximum of this density is obtained for x = a.

V.6 .1.3.2 Estimation of parameters (a and b) of Gumbel's law using the Method of Moments

The Method of Moments is used to calibrate the parameters of Gumbel's law. Parameters a and b are calculated based on the following formulas:

$$b = \frac{\sqrt{6}}{\pi}\sigma \Rightarrow b = 0.7797\sigma$$

Where σ is the standard deviation of the values making up the sample.

$$\sigma = \sqrt{\frac{\sum(x-\mu)^2}{n}}$$

$$a = \mu - b\gamma$$

μ: the sample mean.

$\gamma = 0.5772$ is Euler's constant

$$a = \mu - 0.5772b$$

$$a = \mu - 0.444\,\sigma$$

Application 4

Maximum annual rainfall amounts are available for a period of 29 years, for durations of 1, 3, 6, 12 and 24 hours. These amounts are presented in descending order of magnitude in columns 1, 2, 3, 4 and 5 in **Table 8** for the given durations. We are asked to develop the IDF curves for this area.

Table 8: Maximum annual rainfall amounts

1h **h (mm)**	3h **h (mm)**	6h **h (mm)**	12h **h (mm)**	24h **h (mm)**
4.06	**8.09**	14.93	15	15
4.36	9.27	15	18.01	**20.5**
4.46	9.67	15.2	20.28	21.7
4.75	10.03	15.66	**20.38**	22
4.87	10.47	**15.71**	21	29.94
4.97	10.59	16.76	25.15	32.1
5.3	11.96	18.01	27.62	32.8
5.52	12.35	20.75	27.64	36.1
5.54	12.51	21.72	28.32	38.6
5.57	12.65	22.76	28.57	40.3
6	13.62	22.89	28.97	40.99
6.68	13.74	24.18	32.1	42.71
7.06	15.8	24.88	32.8	43.46
7.66	16.63	25	33.8	43.7

8.04	17.77	25.51	34.43	44
8.39	18	25.54	36.13	44.93
8.58	19.25	26.83	38.06	47.39
8.88	19.46	27.21	41.66	48
9.19	20.26	28.02	41.73	54.24
10.22	20.85	28.61	45.22	55.5
10.49	21.24	29.6	45.91	56.3
11.25	21.63	35.02	48.65	56.6
11.32	21.9	35.62	50.84	60.4
11.41	23.54	38.36	55.57	64.4
11.49	30.77	44.82	59.38	69.11
11.84	30.88	45.1	62.38	77.1
14.83	31.12	51.4	64.04	78.2
17.19	35.25	55.25	74.8	99.92
21.48	43.8	57.62	85.99	119.04

Example of calculation for a downpour duration of 1 hour:

The empirical frequency *F(x)* is calculated using Hazen's formula as follows:

$$F(x) = \frac{r - 0.5}{n}$$

Where *r* is the rank in the data series classified in ascending order, and *n* is the size of the series. In our case, *n* = 29.

Example: *r* = 2, *n* = 29

$F(x) = \frac{2-0.5}{29} \Rightarrow F(x) = 0.0517$

Calculation of the reduced Gumbel variable:

$u = -ln\left(-ln\left(F(x)\right)\right)$

Example:

F(x) =0.052 $\Rightarrow u = -ln(-ln(0.0517))$

$$u = -1.0858$$

Estimation of adjusted precipitation runoff

Calculation of the observed average precipitation runoff (column D)

$$Moy_{LPO} = \frac{\sum_{i=1}^{n} LPO_1}{n}$$

$$Moy_{LPO} = 8.6689\ mm$$

Calculation of the standard deviation of the observed precipitation runoff (column D)

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}(LPO_i - Moy_{LPO})^2}{n-1}}$$

$$\sigma = 4.1264mm$$

Estimation of parameters (a and b) of Gumbel's law using the Method of Moments

Parameters (a and b) of Gumbel's law are calculated using the formulas:

$$b = \frac{\sqrt{6}}{\pi}\sigma$$

σ is the standard deviation of the values of the sample.

$a = \mu - b\gamma$

μ is the average of the sample.

With $\mu = Moy_{LPO}$ and $\gamma = 0.5772$

Example:

$b = \frac{\sqrt{6}}{\pi}4.1264 \Rightarrow b = 3.2173$

$a = 8.6689 - 3.2173(0.5772\ ) \Rightarrow a = 6.8119$

It is therefore possible to estimate the adjusted precipitation runoff using the following formula:

$LPA = a + bu$

Example for calculating the value of the adjusted precipitation runoff (**Table 9**: line 2 and column E)

$LPA = 6.8119 + 3.2173(-1.0858)$

$LPA = 3.3185\ mm$

Table 8 : Adjusted precipitation runoff

A	B	C	D	E
Rank	F(x) = Hazen's empirical frequency	u = Gumber's reduced variable	Observed precipitation runoff (mm)	Estimated precipitation runoff (mm)
1	0.0172	-1.4013	4.06	2.3034
2	**0.0517**	**-1.0858**	**4.36**	**3.3185**
3	0.0862	-0.8965	4.46	3.9275
4	0.1207	-0.7488	4.75	4.4026
5	0.1552	-0.6223	4.87	4.8097
6	0.1897	-0.5084	4.97	5.1763
7	0.2241	-0.4025	5.3	5.5171
8	0.2586	-0.3019	5.52	5.8407
9	0.2931	-0.2048	5.54	6.1531
10	0.3276	-0.1098	5.57	6.4588
11	0.3621	-0.0158	6	6.7611
12	0.3966	0.078	6.68	7.0629
13	0.431	0.1725	7.06	7.3669
14	0.4655	0.2684	7.66	7.6754
15	0.5	0.3665	8.04	7.9911
16	0.5345	0.4677	8.39	8.3166
17	0.569	0.5728	8.58	8.6549
18	0.6034	0.683	8.88	9.0094
19	0.6379	0.7996	9.19	9.3844
20	0.6724	0.9241	10.22	9.7851
21	0.7069	1.0588	10.49	10.2185
22	0.7414	1.2065	11.25	10.6937
23	0.7759	1.3713	11.32	11.2238
24	0.8103	1.5592	11.41	11.8286
25	0.8448	1.7801	11.49	12.5391
26	0.8793	2.0509	11.84	13.4104
27	0.9138	2.4063	14.83	14.5538
28	0.9483	2.9354	17.19	16.2561
29	0.9828	4.0518	21.48	19.8479

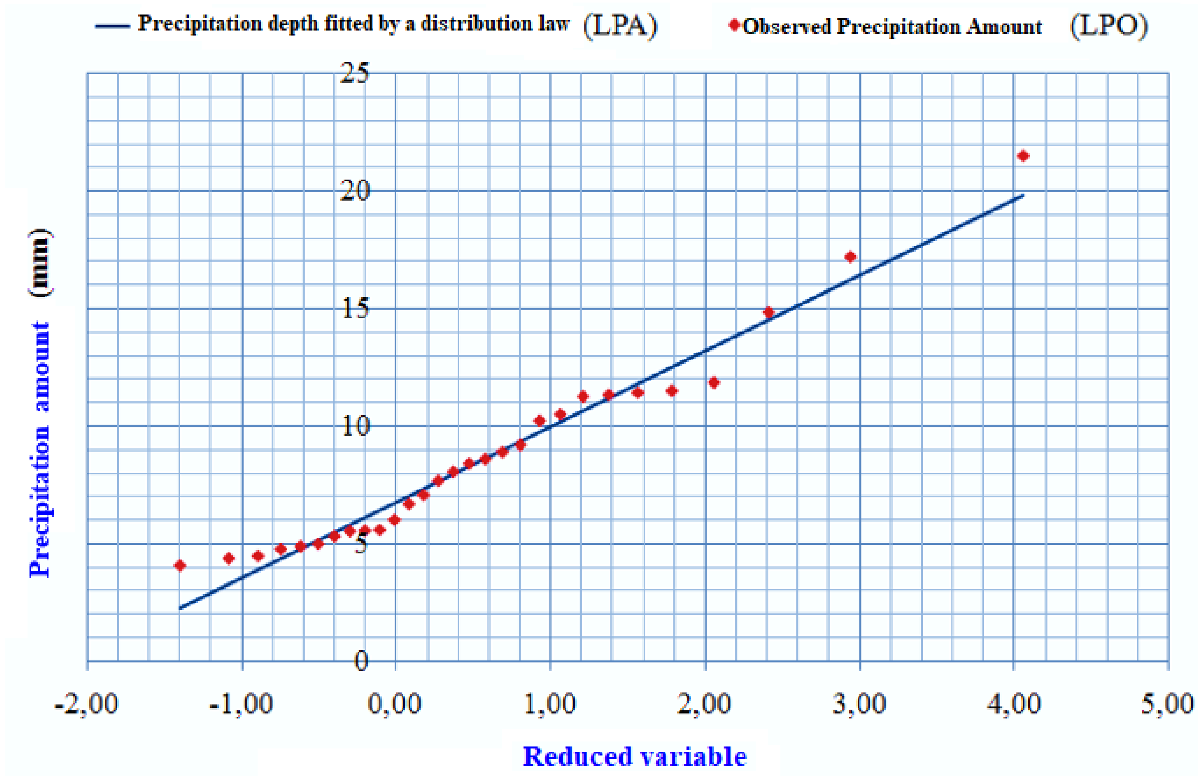


Figure 55: Observed and adjusted precipitation runoffs for a rainfall duration of 1 hour.

The same approaches are followed to determine the adjusted precipitation runoff for other downpour durations (3h, 6h, 12h, and 24h). **Table 10** summarizes the parameters a and b of the adjustment lines obtained by the method of moments, for all precipitation durations under consideration.

Table 9 Parameters a and b of the adjustment lines obtained by the method of moments.

	1h	3h	6h	12h	24h
b	3.2173	6.7935	9.5241	13.6930	18.0452
a	6.8119	14.8063	23.053	31.5594	39.0680

Estimation of the probability of non-exceedance for different return periods

As the return time T of an event is defined as the inverse of the frequency of occurrence of the event according to the following formula, then:

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

Example of calculation

For T= 2 years, $F(x_i) = 1 - \frac{1}{2} \Rightarrow F(x_i) = 0.5$

For T= 50 years, $F(x_i) = 1 - \frac{1}{50} \Rightarrow F(x_i) = 0.98$

Calculation of the reduced Gumbel variable (**Table 11**)

$$u = -ln\left(-ln(F(x))\right)$$

Example:

$$u = -ln(-ln(0.50)) \Rightarrow u = 0.37$$

$$u = -ln(-ln(0.98)) \Rightarrow u = 3.90$$

Table 10: Non-exceedance probability and reduced Gumbel variable

Return time (years)	**2**	**5**	**10**	**20**	**50**
Non-exceedance probability (F(x))	0.50	0.80	0.90	0.95	0.98
Reduced Gumbel variable (u)	0.37	1.50	2.25	2.97	3.90

Estimated precipitation runoffs for the return periods considered (**Table 12)**

$$LPA = a + bu$$

$$LPA(1h \text{ and } T = 2 \text{ years}) = 6.8119 + 3.2173(0.37)$$

$$LPA(1h \text{ and } T = 2 \text{ years}) = 8\, mm$$

$$LPA(1h \text{ and } T = 50 \text{ years}) = 6.8119 + 3.2173(3.90)$$

$$LPA(12h \text{ and } T = 2 \text{ years}) = 31.5594 + 13.6930(0.37)$$

$$LPA(12h \text{ and } T = 2 \text{ years}) = 36.62 \cong 37\, mm$$

Table 11: Precipitation runoffs

Duration of rain (h)	LPA (mm) T = 2 years	LPA (mm) T = 5 years	LPA (mm) T = 10 years	LPA (mm) T = 20 years	LPA (mm) T = 50 years
1	8	12	14	16	19
3	17	25	30	35	41
6	27	37	44	51	60
12	37	52	62	72	85
24	46	66	80	93	109

Estimated rainfall intensities: **(Table 13)**

$$i_{Avge} = \frac{LPA}{Duration}$$

Example

LPA = 37 mm

Duration = 12 h

$$i_{Avge} = \frac{37}{12} = 3.08 \text{ mm/h}$$

Table 12: Rainfall intensities

Duration of rain (h)	I_{Avge} [mm/h]	I_{Avge} [mm/h]	I_{Avge} [mm/h]	I_{Avge} [mm/h]	I_{Avge} [mm/h]
1	7.99110774	11.6377592	14.0521598	16.3681109	19.3658719
3	5.7654277	8.33207407	10.0314173	11.6614681	13.7714017
6	4.42395619	6.22310421	7.41429675	8.55691724	10.0359222
12	3.04817892	4.34152181	5.19782733	6.01921615	7.08241973
24	1.903412	2.75562018	3.31985615	3.86108484	4.56164986

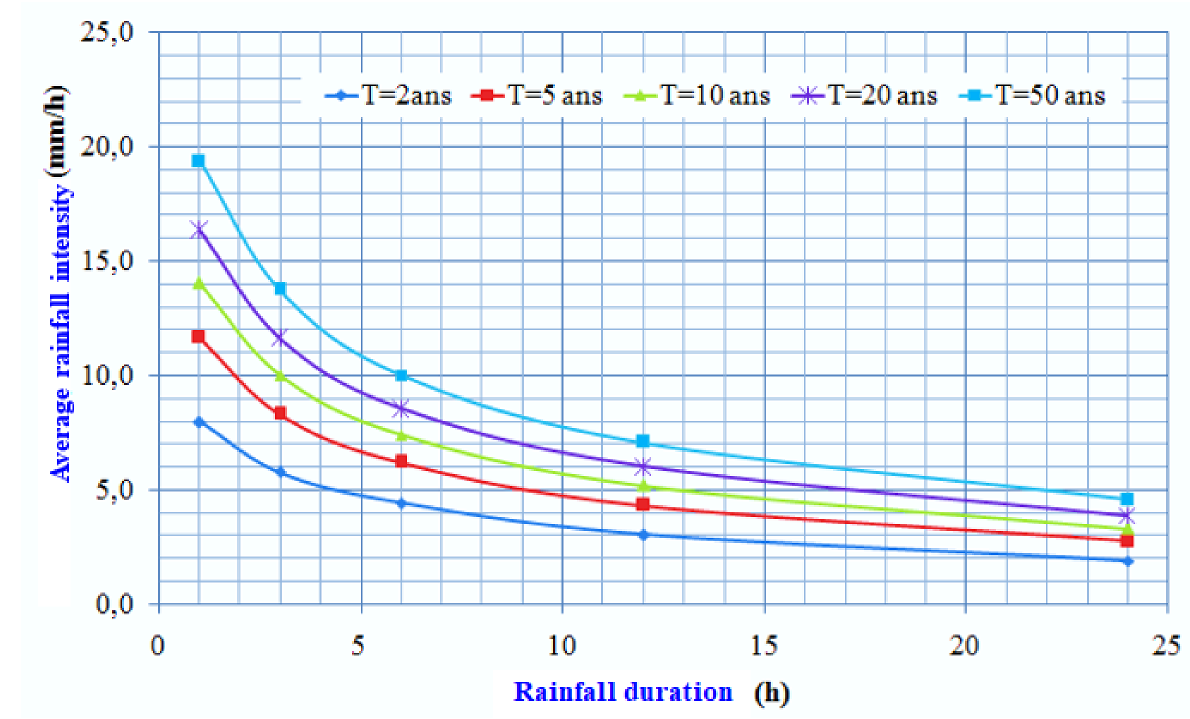


Figure 56 Average precipitation intensity curves

Estimation of Montana's law parameters:

	A	B	C	D	E	F
	Ln(t)	**Ln(i)**	**Ln(i)**	**Ln(i)**	**Ln(i)**	**Ln(i)**
1	0.00	2.08	2.45	2.64	2.80	2.96
2	1.10	1.75	2.12	2.31	2.46	2.62
3	1.79	1.49	1.83	2.00	2.15	2.31
4	2.48	1.11	1.47	1.65	1.79	1.96
5	3.18	0.64	1.01	1.20	1.35	1.52

The Excel's "slope" function = slope(B1:B5;$A$1:$A$5) = -0.45 can be used to calculate the slope of the line.

The Excel “slope” function = ORDINATE.ORIGIN (B1:B5;$A$1:$A$5) = 2.18 can be used to calculate the intercept of the line (**Table 14**).

Table 13 Slope and Ordinate

Parameter "Slope"	-0.45	-0.45	-0.45	-0.45	-0.45
Parameter "Ordinate" l'origine"	2.18	2.55	2.73	2.88	3.05

Montana’s parameter b = -(-0.45) = 0.45

Montana’s parameter a = EXP(2.18) = **8.82**

Table 14: Montana coefficient for the station 160401

Montana’s parameter b	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**
Montana’s parameter a	**8.82**	**12.75**	**15.35**	**17.85**	**21.08**

In this example, a frequency analysis of maximum annual rainfall amounts was carried out for the duration of 1 hour. To obtain the IDF curves, the same procedure is applied to the maximum annual data, for the other durations considered. The calculations are summarized in Tables 14 and 15, and the results are shown in **Figure 57**.

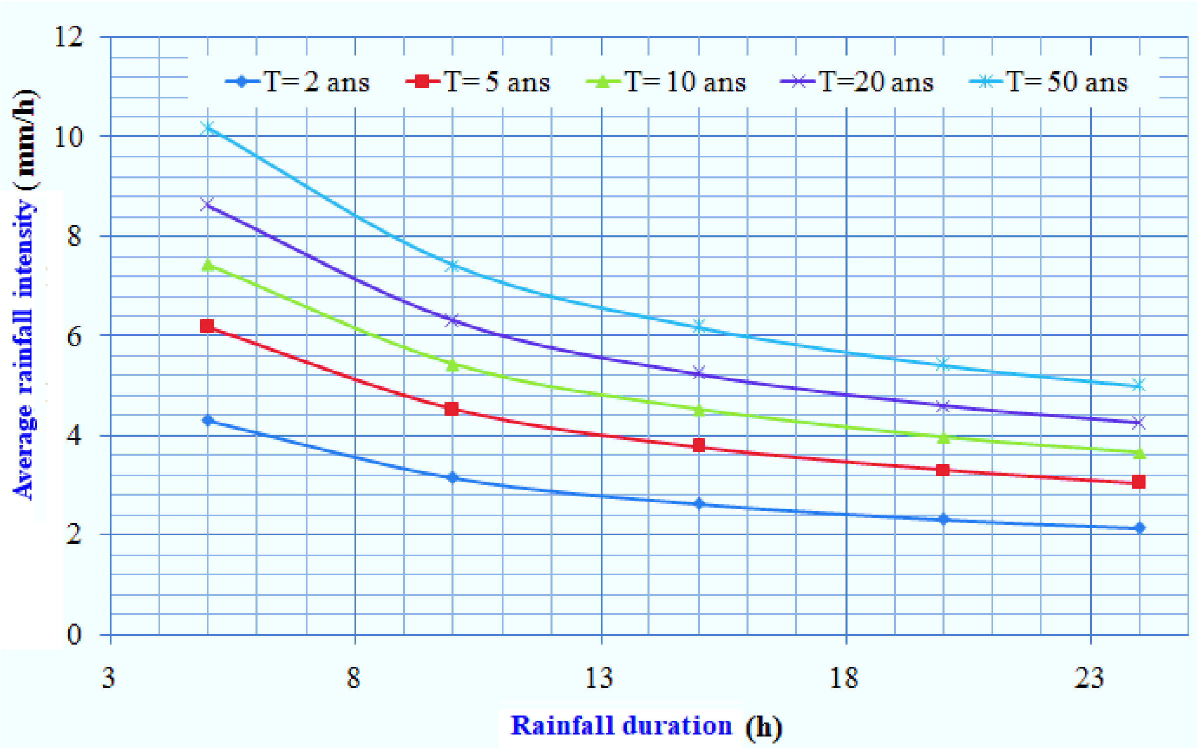


Figure 57 IDF curves using the Montana method

V.6.2 Calculation of storm flows using the Caquot method (Superficial method, 1949)

This method is used to calculate the maximum flows of an urban watershed. It is described in the Technical Instruction of 1977 (Ministry of Equipment and Regional Planning, 1977):

The value of rainwater flow coming from an urbanized watershed, for a given frequency F, is given by M. Caquot's formula which is valid for watersheds of average elongation M=2.

$$\boldsymbol{Q(F) = [K]^{\frac{1}{u}}. C^{\frac{1}{u}}. I^{\frac{v}{u}}. A^{\frac{w}{u}}}$$

The different parameters used in this formula are functions of a(F) and b(F) (Montana parameters of the function: $i_M(t,T) = a(T).t^{b(T)}$).

Where $Q(F)$ is the exceedance frequency flow (F) expressed in cubic meter per second, C is the runoff coefficient that represents the imperviousness rate. It is calculated by the following formula: $= \frac{\grave{A}}{A}$. Here À is the urbanized area and A is the total surface area of the watershed. I is the average slope of the urbanized watershed collector (m/m).

The average slope can be calculated with the following formula: $I = \left[\frac{L}{\frac{L_1}{\sqrt{I_1}}+\frac{L_2}{\sqrt{I_2}}+\cdots\frac{L_n}{\sqrt{I_n}}}\right]^2$ which can also be written as: $I = \left[\frac{L}{\sum\frac{L_k}{\sqrt{I_k}}}\right]^2$ Where L is the length of the longest path of the pipe. The lengths of the sections composing this long path are $L_1, L_2, \ldots, L_k$.

Also, A is the area (ha) of the watershed, k is the coefficient calculated from the following expression $\frac{0.5^{b(F)}a(F)}{6.6}$, u is the coefficient calculated from the following expression : $1 + 0.287\, b(F)$, v is the coefficient calculated from the following expression:

$-0.41\, b(F)$, and w is the coefficient calculated from the following expression: $0.95 + 0.507\, b(F)$.

V.6 .2.1 Elongation of the basin (M)

The elongation factor is the ratio of the length of the longest hydraulic path L (hectometers) to the surface area A of the basin considered (hectares).

$$M = \frac{L}{\sqrt{A}} \geq 0.8$$

It should be noted that L is the length of the longest hydraulic path (hectometers), A is the equivalent area (hectares), and M is the form factor of the basin.

If the average elongation factor M of the basin is different from 2, the flow correction must be carried out.

V.6 .2.2 Basin elongation coefficient (m)

$$\boldsymbol{m} = \left(\frac{\boldsymbol{M}}{\boldsymbol{2}}\right)^{\frac{\mathbf{0.84}\boldsymbol{b}(\boldsymbol{F})}{\mathbf{1+0.287}\boldsymbol{b}(\boldsymbol{F})}}$$

This coefficient can also be calculated from the following formula:

$$\boldsymbol{m} = \left(\frac{\boldsymbol{M}}{\boldsymbol{2}}\right)^{\mathbf{0.7}\boldsymbol{b}(\boldsymbol{F})}$$

Where **m** is the elongation coefficient, **b(F)** is the Montana coefficient in the study region per a given return period, and **M** is the basin form factor.

V.6 .2.3 Corrected basin flow (Qc)

It is expressed as :

$$Q_c = mQ_p$$

Here **m** is the elongation coefficient, $\mathbf{Q_p}$ is the peak flow (m^3/s), and $\mathbf{Q_c}$ is the corrected flow (m^3/s).

V.6 .2.4 Concentration time (Tc)

The drop of water falling at a point M in the watershed flows along the path Mm (gutters, gutters, etc.) for a time t' and flows into the collector (**Figure 58**), between points m and O, for a time t''. The total duration of the flow is t = t' + t''.

The maximum duration of flow in the basin is called concentration time tc = max (t'+t'').

On a practical level, this time can be measured using tracers injected into water (fluorescein).

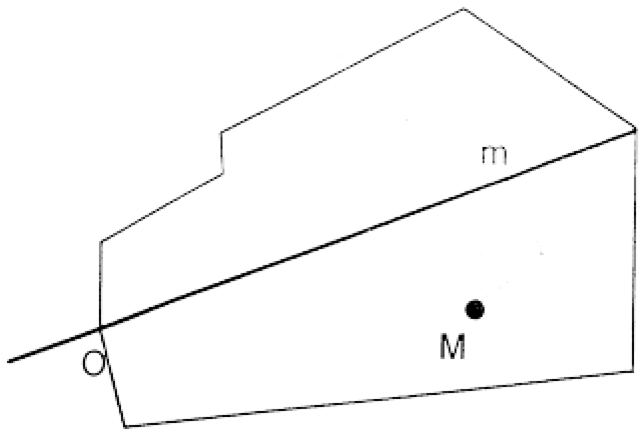


Figure 58 : Urbanized basin, Concentration time

Concentration time (t_c)

$$\boldsymbol{T_c = \mu \times I^c \times A^d \times Q_p}$$

Here T_c is the concentration time (minutes), I is the slope along the path of water (m/m), A is on the basin surface area (hectares), and Q_p is the peak flow (m^3/s).

$\boldsymbol{c} = -0.41$ $\boldsymbol{d} = 0.507$ $\boldsymbol{f} = -0.287$ $\mu = 0.28M^{0.84}$

V.6 .2.4 Caquot's general formula

$$Q(F) = \left[\frac{a.\mu^b}{6(\beta+\delta)}\right]^{\frac{1}{1-b.f}} . C^{\frac{1}{1-b.f}} . I^{\frac{c.b}{1-b.f}} . A^{\frac{1-\varepsilon+d.b}{1-b.f}}$$

Q_P is the peak flow (m^3/s), I is the average slope of the hydraulic path (m/m), C is the runoff coefficient > 0.20, and A is the area of the watershed (ha).

μ= 0.5 **β + δ** = 1.1 **ε** = 0.05 **c** = -0.41 **d** = 0.507 **f** = -0.287

Also, *i* is the rainfall intensity (mm/minute), and *t* is the time (between 5 and 120 minutes). β and δ characterize the rainfall-runoff relationship.

Table 15: Estimation of runoff coefficient depending on urbanization type or land use (Source: PHASE 2 REPORT: STUDY OF SOLUTIONS - Rainwater sanitation master plan (October 2004, https://www.langueux.fr/medias/2016/08/rapport.pdf).

Désignation du type d'urbanisation ou d'occupation du sol	Coefficient de ruissellement moyen [1]
Centre ville d'agglomération importante, habitat très dense, « Vieille ville »	0,80 - 0,95
Zones d'habitat collectif, banlieue sans jardins ni espaces verts	0,60 - 0,80
Zones d'habitat semi-collectif, quartiers récents avec espaces verts	0,40 - 0,60
Zones résidentielles ou pavillonnaires	0,25 - 0,45
Centre d'agglomération rurale	0,15 - 0,35
Zone artisanale	0,30 - 0,80
Zone industrielle	0,50 - 0,80
Zone portuaire	0,70 - 0,90
Zone ferroviaire	0,20 - 0,35
Terrain de sports et de jeux	0,20 - 0,40
Cimetières	0,4
Chaussées, parkings, voies piétonnes	0,70 - 0,90
Espaces verts	0,10 - 0,25
Jardins et parcs	0,05 - 0,20
Bocage	0,04 - 0,08
Zones cultivées	0,06 - 0,10
Forêts, terrains incultes	0,01 - 0,10

The runoff coefficient, which depends on many factors such as the nature of the soil, determines the fraction of precipitation which will run off the ground and therefore end up in the retention structure.

Table 16 : A runoff coefficient is assigned to each type of surface

https://www.giser.be/wp-content/uploads/2019/05/Guide-technique-Eaux-pluviales.pdf

Nature de la surface	**Valeur du coefficient de ruissellement**
forêts, bois	0,05
prairies, jardins, zones enherbées, pelouses, parcs,...	0,15
champs cultivés, landes, broussailles, toitures vertes >10cm, cimetières, dalles empierrement,...	0,25
dalles gazon,...	0,4
terres battues, chemins de terre,...	0,5
pavés à joints écartés, pavés drainants,...	0,7
allées pavées, trottoirs pavés, parkings, terrains imperméabilisés,...	0,9
toitures, routes, plans d'eau,...	1
Autres (à justifier)	

Basins 1 and 2 are in parallel

BV1+BV2 = BV12 equivalent

Basin 3 is in series with the equivalent BV12 basin

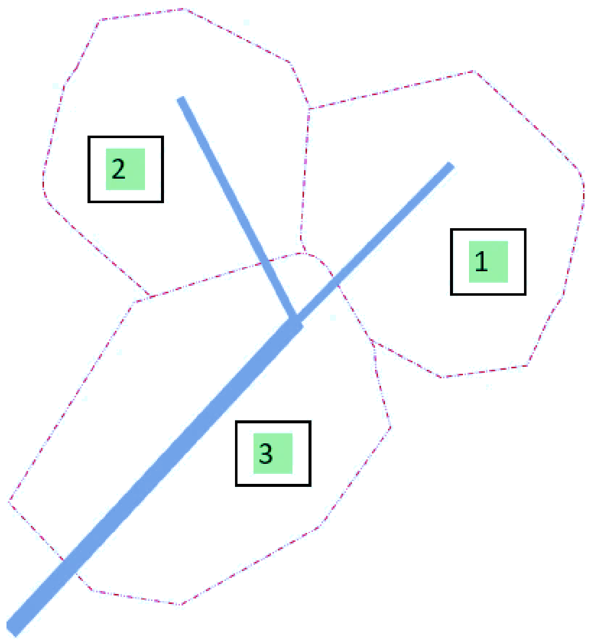


Figure 59: Grouping of basins

Equivalent parameters	A_{eq}	C_{eq}	I_{eq}	M_{eq}

Basins in series	$\sum \boldsymbol{A_j}$	$\frac{\sum \boldsymbol{C_j A_j}}{\sum \boldsymbol{A_j}}$	$\left(\frac{\sum \boldsymbol{L_j}}{\sum \frac{\boldsymbol{L_j}}{\sqrt{\boldsymbol{I_j}}}}\right)^{\mathbf{2}}$	$\left(\frac{\sum \boldsymbol{L_j}}{\sqrt{\sum \boldsymbol{A_j}}}\right)$
Basins in parallel	$\sum A_j$	$\frac{\sum C_j A_j}{\sum A_j}$	$\frac{\sum I_j Q_{pj}}{\sum Q_{pj}}$	$\frac{L(T_{cMax})}{\sqrt{\sum A_j}}$

Note that A_j is the surface area of the elementary basins (m^2), C_j is the elementary runoff coefficient of the elementary basins, L_j is the hydraulic path of the elementary basins (m), I_j is the slope of the elementary basins (m/m), Q_{pj} is the peak flow of the elementary basins (m^3/s), and $L(T_{cMax})$ is the hydraulic path according to the elementary basin which has the highest concentration time.

Note

When assembling watersheds, the following rules must be respected:

1- ***For series watersheds*** - If the flow of the equivalent watershed is less than the maximum of the elementary flow of the sub-basins, the equivalent flow must be taken equal to the maximum flow of the elementary basins,

2- ***For parallel watersheds*** - If the flow of the equivalent watershed is greater than the sum of the flow of the elementary basins, then the equivalent flow must be taken equal to the sum of the flow of the elementary basins.

V.7 Calculation of peak flows using the rational method

The rational method is based on the following assumptions:

- Rain is constant and uniform across an entire watershed
- All sections of the watershed contribute to the flow
- The duration of the rain is equal to the concentration time.
- This method does not take into account the heterogeneity of rainfall and tends to overestimate the peak flow.

The peak stormwater runoff flow can be calculated from the following formula:

$Q_p = K \times C \times I \times A$

Here Q_p is the peak flow (m^3/s), C is the runoff coefficient (non-dimensional), I is the rainfall intensity (m^3/s/ha) for the frequency of occurrence retained, A is the elementary area of the watershed (ha), and K is the conversion factor (K = 2.75.10^{-3}).

V.7 .1 Determination of the pipe section

1.7.1.1 - Manning-Strickler formula (Uniform flow)

The Manning-Strickler formula may be applied to calculate the sectional area of pipes in each portion as a function of the calculated flow rates. Hence:

$$V = K \times R_h^{2/3} \times I^{1/2}$$

$$Q = VS_m$$

$$Q = K \times R_h^{2/3} \times I^{1/2} \times S_m$$

Here Q is the flow rate in the pipes (m^3/s), R_h is the average hydraulic radius which is given by the ratio between the flow section (m^2) and the wetted perimeter (m) as $R_h = S_m/P_m$, I is the slope of the pipe (m/m), and K the Manning-Strickler coefficient.

The Manning coefficient depends on the nature of the gutter covering. Its usual values are presented in Table Y, Annex Y.

1.7.1.1 Sizing the wastewater network

1.7.1.1.1 Calculating the wastewater flows

The wastewater flow is estimated from the total water consumption of an urban area and/or an industry, while taking into account the peak volume distributed.

1.7.1.1.2 Drinking water consumption of the population

The sanitary sewer is designed to evacuate domestic wastewater, as well as the wastewater coming from businesses, industries and public facilities such as hospitals, mosques, schools, etc. Sizing the wastewater network involves evaluating the flow rates of all these sources. The average daily consumption per capita is between 100 and 250 liters per day.

For areas including establishments with specific activities, such as hospitals, canteens, schools, military barracks, etc., the designer may refer to a table which presents average daily consumption values and peak coefficients for different establishments.

Table 17: Daily water consumption with peak coefficient of some establishments

ACTIVITES	CONSOMMATION JOURNALIERE	COEFFICIENT DE POINTE
Cantines	10 l. par rationnaire	10
Internat	150 l. par élève	6
Ecoles	60 l. par élève	6
Ateliers et bureaux	60 l. par personne	4
Casernes	90 l. par soldat	3
Hôpitaux	400 l. par lit	3
Hôtels	500 l. par chambre	4
Gymnase	20 l. par usager	2
Centres commerciaux	5 l. par m^2	2.5

1.7.1.1.3 Determination of the average flow (Q_m)

The average daily flow of domestic wastewater, noted Q_{jm}, is calculated on the basis of the water supply, which corresponds to the unit flow per inhabitant, noted q_u. For example, if q_u is 150 litres per capita per day, and N the number of inhabitants, then the average daily flow Q_{jm} can be calculated as follows:

$$Q_{Avge} = \frac{q_u \times N_{inhab}}{86400}$$

Where Q_{Avge} is the average flow (l/s), q_u is the water supply or unit flow (l/d/inhabitant), and N_{inhab} is the number of inhabitants

1.7.1.1.4 Determining the peak coefficient

The peak coefficient P, which is used in designing sanitation networks to take into account flow variations, may be determined using the following formula:

$$P = a + \frac{b}{\sqrt{Q_m}}$$

$$1.5 \leq P = 1.5 + \frac{2.5}{\sqrt{Q_m}} \leq 4$$

Finally, the peak flow Q_P can be calculated from the average daily flow Q_{jm} by applying the peak coefficient

$Q_P = Q_{jm} \times P$

Q_P is the peak flow (l/s).

1.7.1.1.5 Case of a separative sanitation network

Here, the rainwater flow is not considered:

$$Q = KSI^{0.5}R_h^{2/3}$$

Q is the wastewater flow (m^3/s), K is the Strickler coefficient, which is linked to the roughness of the pipe, S is the sectional area of the pipe (m^2), I is the slope (m/m), and R_h the hydraulic radius (m).

From the above, one may deduce the theoretical pipe diameter D through which this flow passes:

$$D = 4^{5/8}(Q/(\pi I^{0.5}K))^{3/8}$$

1.7.1.1.6 Checking self-cleaning conditions

In addition to the slope criterion, the self-cleaning conditions to be checked are as follows:

- At full section, the flow must ensure a minimum speed of 0.7 m/s.
- For a filling of 2/10 of the diameter, the flow speed must be equal to at least 0.30 m/s.

These conditions must be verified for the entire network.

These criteria are highly important to ensure that sediment and waste do not build up in pipes, as this could impede the flow of wastewater and therefore require more frequent maintenance.

Application 1

In a circular pipe, with a nominal diameter ϕ = 300 mm, a flow rate flows over a height h= 210 mm with a slope i = 5‰.

- Calculate the total filling height fraction
- Determine the flow speed and flow rate.
- Calculate the hydraulic radius.

Solution

The chart Ab.3 gives:

$$\begin{cases} \emptyset = 300\ mm \\ i = 5‰ \end{cases} \rightarrow \begin{cases} Q_{ps} = 65\ l/s \\ V_{ps} = 0.9\ m/s \end{cases}$$

The total filling height fraction is:

$r_h = \frac{h}{\emptyset} = \frac{210}{300} = 0.7$

The chart Ab.5 gives:

$$r_h \rightarrow \begin{cases} r_Q = 0.837 \\ r_V = 1.12 \end{cases} \rightarrow \left\{ r_Q = \frac{Q_e}{Q_{Ps}} \right.$$

$$Q_e = r_Q \times Q_{Ps}$$

$$Q_e = 0.837 \times 65$$

$$Q_e = 54.40\ l/s$$

The flow speed is:

$$r_V = \frac{V_e}{V_{Ps}} \rightarrow V_e = r_V V_{Ps}$$

$$V_e = 1.12 \times 0.9$$
$$V_e = 1.008\ m/s$$

The hydraulic radius is:

$$V_e = \frac{1}{n} R_h^{2/3} i^{1/2}$$
$$R_h^{2/3} = \frac{V_e}{\frac{1}{n} i^{1/2}}$$
$$R_h = \left(\frac{1.008}{75(0.005)^{1/2}}\right)^{3/2}$$

$$R_h = 83\ mm$$

Application 2

Consider a separate sewerage collector system carrying an average flow Q_{Avge} = 18l/s, using a slope i = 8 per thousand

1. Determine the collector diameter
2. Check the self-cleaning conditions

Solution

$$P = 1.5 + \frac{2.5}{\sqrt{Q_{Avge}}}$$

$P = 1.5 + \frac{2.5}{\sqrt{18}} \Rightarrow$P = 2.08

$Q_e = P \times Q_{Avge} \Rightarrow Q_e = 2.08 \times 18$

$$Q_e = 37.60\ l/s$$

The chart Ab.3 gives: $Q_{Ps} = 50\ l/s$ and $\emptyset = 250\ mm$

Checking the self-cleaning conditions

$r_H = \frac{H}{\emptyset} \Rightarrow$

$r_H = 0.5 \Rightarrow V_e \geq 0.5\ m/s$

$r_V = \frac{V_e}{V_{ps}} \Rightarrow V_e = r_V V_{ps}$

With:

$V_{ps} = 1\ m/s \Rightarrow V_e > 0.5 \frac{m}{s} \Rightarrow$ The first condition is satisfied.

2) $r_H = 0.2 \Rightarrow V_e \geq 0.3\ m/s$

The chart gives:

$r_H = 0.5 \Rightarrow r_V = 0.6$

Hence:

$V_e = 0.6 \times 1 \Rightarrow V_e = 0.6\ m/s \Rightarrow 0.6 > 0.3 \frac{m}{s} \Rightarrow$ The second condition is satisfied.

3) $Q_{Avge} = 18 l/s$

$r_H = 0.2 \Rightarrow Q_{Avge} \geq Q_e$

The chart Ab.5 gives :

$r_H = 0.2 \Rightarrow r_Q = 0.12$

Then: $r_Q = \frac{Q_e}{Q_{ps}} \Rightarrow Q_e = r_Q Q_{ps}$

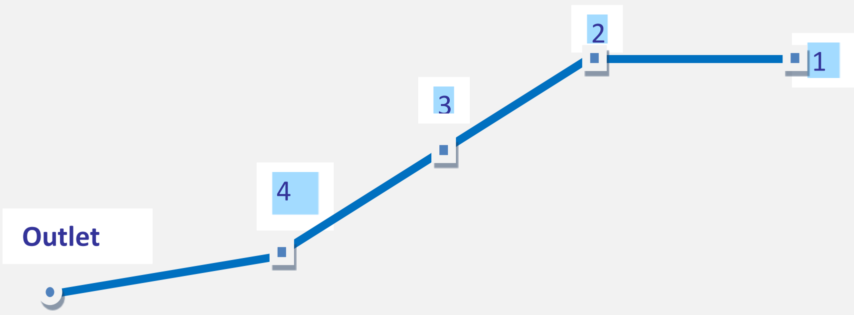
$$Q_e = 0.12 \times 50$$

$Q_e = 6\frac{l}{s} \Rightarrow$ The third condition is satisfied.

Application 3

I. I. Let's consider the sewer collector (wastewater) in a separate system described below:

$L_{1-2} = 45m$
$L_{2-3} = 50m$
$L_{3-4} = 55m$
$CTN_1 = 509.1m$
$CTN_2 = 508.8m$
$CTN_3 = 507.2m$
$CTN_4 = 506.5m$



Basic data:

1) Number of housing units: 2005 housing units
2) Number of inhabitants per accommodation: 7 people
3) Population growth rate: 28‰
4) Water supply: 200 l/d/inhabitant
5) Leak percentage: 20%
6) Future evolution of the population by 2020

Question: Size and check the self-cleaning conditions of this collector.
Solution:

- $P = P_0 (1 + t)^n$
  $P_0 = 2005 * 7 = 14035$ pers
- $P_{2020} = 14035(1 + 0.028)^{18}$
  $= 23073$ pers.
- $Q_{Avge} = (23073*0.8*200)/(3600*24) = 42.73$ l/s

Calculation of peak flow:

$P = 1.5 + (2.5)/ (Q_{Avge.})^{1/2}$
$P = 1.5 + (2.5)/(42.73)^{1/2} = 1.88$
$Q_{pt} = 1.88*42.73$
$= 80.33$l/s

Calculation of specific flow:

$\sum L_i = 150m$
- $Q_{sp} = (80.33)/150 = 0.535$l/s/ml

Calculation of the flow in the sections:

Section	Q_{sp}	L_i	Q_{tr}	$Q_{cumulative}$ (l/s)
1-2	0.535	45	24.099	24.099
2-3	0.535	50	26.75	50.849
3-4	0.535	55	29.425	80.274

	$Q_{cumulative}$ (l/s)	I_H	D mm	Q_{ps} l/s	V_{ps} m/s	r_q	r_v	V_e m/s	Checking self-cleaning conditions						
									$r_h=0.5 \Rightarrow v_e>0.5$m/s		$r_h=0.2 \Rightarrow v_e>0.3$m/s		$r_h=0.2 \Rightarrow Q_{Avge}>Q_r$l /s		
									r_v	V_e	r_v	V_e	r_q	Q_{moy}	Q_r
1-2	24.099	1%	200	30	0.97	0.8033	1.11	1.0767	1.01	0.9797	0.61	0.5917	0.12	12.81	3.6
2-3	50.849	0.002	400	85	0.69	0.5982	1.05	0.7245	1.01	0.6969	0.61	0.4209	0.12	27.04	10.2
3-4	80.274	6.10188	400	85	0.69	0.9444	1.14	0.7866	1.01	0.6969	0.61	0.4209	0.12	42.69	10.2

$Q_{Avge} = Q_{cumulative}/P$

$Q_r = Q_{ps} * r_q$

Chapter VI : Wastewater treatment

VI.1 Introduction

It is highly recommended to present the different pollutants present in wastewater in order to proceed with their elimination before addressing the different processes implemented in a wastewater treatment plant. These pollutants can come from several sources. They can come from detergents used to wash dishes. Other pollutants can come from clothing, or even from sewage and road washing. These pollutants present a major risk to the environment when released into the natural environment.

Floating materials consist of bulky objects such as plastics, rags, dead leaves, plant debris and cans. These coarse wastes are generally not taken into account in standard water analyses. Floating materials represent a risk of visual pollution. Indeed, the direct release of these materials into the natural water environment leads to their floating on the surface, which disrupts air-water exchanges and deprives the aquatic environment of its sources of oxygen. These materials are often removed during the initial stages of wastewater treatment to avoid damaging or clogging treatment plant equipment.

Suspended matter primarily includes sand from road erosion. These sands are generally characterized by a large particle size and a fairly high density. In addition to sand, water also carries suspended mineral and organic matter. Due to their density, these materials are likely to settle when hydraulic conditions permit, and therefore to separate from the water. The evacuation of water loaded with suspended particles risks causing the accumulation of sediment at the bottom of watercourses and on the banks, which could block any contact with the substrate. The accumulation of these particles, which include organic components, is likely to promote excessive growth of bacteria, which could lead to excessive oxygen consumption. This could also lead to deoxygenation and anaerobiosis of the watercourse. Additionally, the presence of microscopic contaminants, such as heavy metals, in these particles raises concerns about persistent toxicity.

The dissolved and non-settleable substances are mainly organic particles which are extremely fine, known as colloids. They are also dissolved compounds that are primarily made up of carbon, nitrogen and phosphorus. It was revealed that dissolved materials in water reduce light penetration and photosynthesis, providing a breeding ground for oxygen-depleting bacteria. This leads to eutrophication, and consequently to plant proliferation, oxygen depletion, and an imbalance in the ecosystem. Finally, pollution alters the oxygen balance of the river, leading to the disappearance of fish, particularly the most sensitive species.

These substances require specific treatment methods to be removed from wastewater because they are not easily separated by settling because they are very small and dissolved.

VI.2 Pollution indicators

Suspended solids (S.S.) are all those particles which cloud the water, which can be separated by filtration or decantation, and are classified as mineral or volatile matter. Their dry weight indicates their potential for deposition and impact on water quality.

The 5-day Biochemical Oxygen Demand (BOD5) expresses the quantity of oxygen that microorganisms must consume to decompose the organic matter present in wastewater, over a period of five days. It is an important indicator of water quality and the pollutant load of wastewater.

Chemical Oxygen Demand (COD) measures the total amount of oxygen needed to oxidize chemicals present in water. It includes both biodegradable and non-biodegradable materials, and hence provides a comprehensive assessment of the pollutant load contained in water. The COD is an indicator of prime importance that allows assessing the impact of wastewater and its potential treatment on the environment.

It is worth emphasizing that **nitrogenous materials** in wastewater, mainly in ammoniacal and organic forms, consume a lot of oxygen when transformed into nitrates in rivers. To avoid this problem and prevent the toxicity of ammonia, it is deemed necessary to carry out nitrification in the treatment plant. Denitrification can further reduce nitrate releases. It should be noted that excess nitrates and phosphates can lead to eutrophication which is harmful to aquatic life and river flow.

VI.3 Preliminary wastewater treatment

Pretreatment is a crucial step in wastewater treatment. Its main purpose is to eliminate pollution and protect processes, such as filtration, decantation, and disinfection, against obstructions and wear.

VI.3.1 Screening

Screening, which is designed to retain bulky waste, is the first filtration step in wastewater treatment. It generally consists of two phases. The 1st phase is coarse screening (bar spacing of 8 to 10 cm) for the largest elements, and the 2nd phase is fine screening (bar spacing of 2 cm). Sometimes even finer sieving is added. The screens are cleaned automatically, often with a mechanical comb, to prevent clogging.

VI.3.2 Desanding,

Desanding, also called desilting, is a wastewater treatment process in which sand and mineral particles are retained to prevent abrasion of equipment and blockage of pipes. In addition, water speed is controlled to allow sedimentation of sand without affecting suspended organic matter. Moreover, aerated desilting, which is a more recent method, uses the blowing of air which serves to separate lighter particles from the sand which is then collected, washed, and drained. Desilting basins, either elongated or circular, use centrifugal force to improve the efficiency of the process.

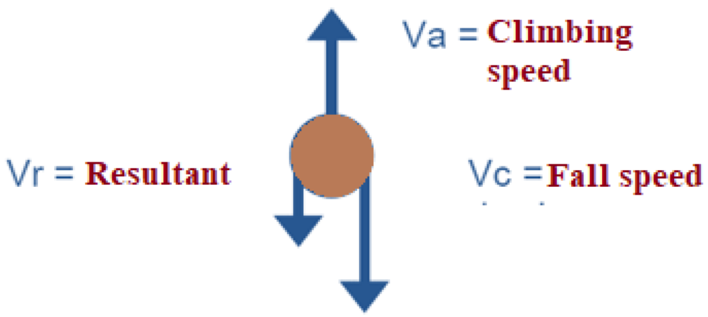
VI.3.3 Degreasing and de-oiling

Degreasing and de-oiling in wastewater treatment involves separating products with a density lower than water, or much lighter than water. By slowing the flow, the particles present in the water rise and are captured by a partition. For the purpose of preventing capillarity and improving separation from water, fine air bubbles are introduced, which accelerates the flotation of fats which are then removed by scraping.

VI.4 Primary decantation

Primary decantation is a physical separation method that uses gravity to remove suspended solids (S.S.) from wastewater. Decanters, circular or rectangular, are designed so that the falling speed of the particles is greater than the rising speed of the water, thus allowing the particles to settle to the bottom. The formula for the climbing speed is:

$Va(m/h)=Q(m^3/h)/surface(m^2)$



It should be noted that circular decanters are easier to scrape thanks to a central rotating bridge. However, rectangular decanters optimize space but require more complex scraping. The collected sludge is then treated or dehydrated, and any fats that may have escaped the pre-treatment phase are then recovered and returned for treatment.

In addition, biological purification is a stage of wastewater treatment which uses bacteria to eliminate dissolved pollution and fine particles. After primary settling, chemical reagents could be used to agglomerate the particles and improve settling. However, this method is expensive and not very effective against dissolved pollution. Bacteria, under optimal mixing and aeration conditions, transform pollution into biomass, which is then separated from the water by decantation. The flocculated structure of bacterial colonies, due to the secretion of exopolymers, facilitates this process.

VI.5 Biological purification processes

VI.5.1 bacterial bed

The bacterial bed is a biological wastewater treatment system in which microorganisms, mainly bacteria, are fixed on a support which is composed of porous or hollow materials and which is regularly irrigated by wastewater previously clarified by decantation. Aerobic bacteria, located on the surface of the biological film, and anaerobic bacteria, deeper, work in synergy to degrade pollutants. Moreover, natural or mechanical aeration of the system is crucial because it provides the necessary oxygen for aerobic bacteria. The contaminants are then absorbed by the biological film which forms on the support. This biofiltration process effectively eliminates organic matter which is often recovered in a secondary decanter where it is separated from the purified water. The waste and carbon dioxide resulting from the purification are then evacuated with the liquid and gaseous effluents, thus completing the purification cycle (**Figure 60**).

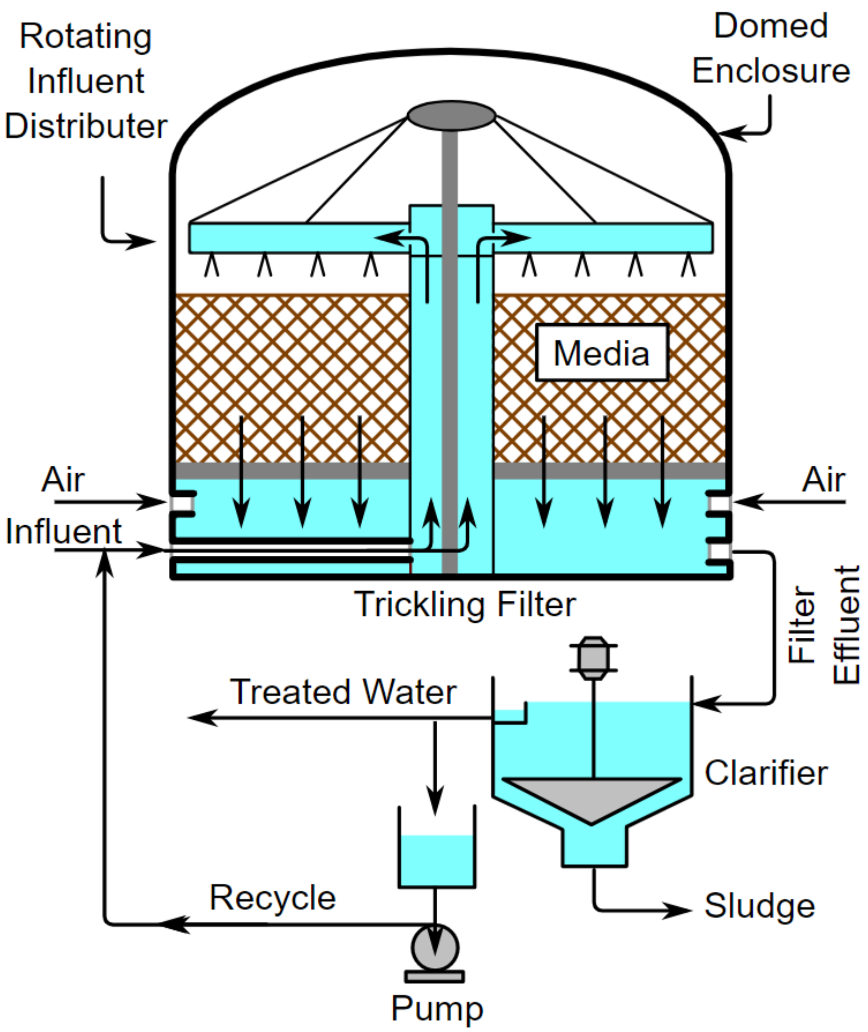


Figure 60 :Treatment plants with bacteria filter

VI.5.2 Biological disks or Rotating Biological Contactors

Rotating biological contactors are a purification system that uses cultures immobilized on rotating disks. Microorganisms grow on these contactors and then create a biological film intended to treat wastewater (**Figure 61**). The partial rotation of the discs in the water allows not only their immersion but also the oxygenation of the biomass which adheres to them. For the purpose of guaranteeing the proper functioning of this installation, it is essential to:

- Check the mechanical reliability of the structure, including, in particular, gradual start-up and solid fixation of the support on its axis.

- Ensure that the surface area of the disks is sufficiently large, providing significant safety margins in order to anticipate any variation in load.

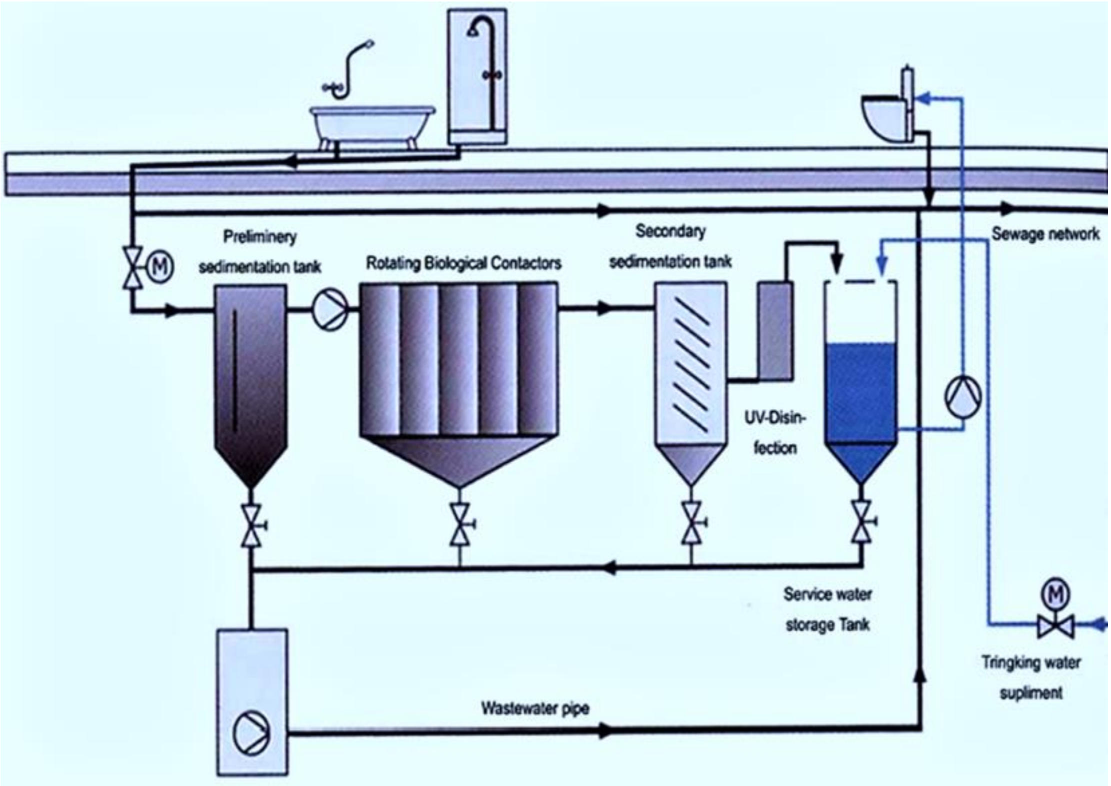


Figure 61: Rotating biological contactors

Source: https://sswm.info/water-nutrient-cycle/wastewater-treatment/hardwares/semi-centralised-wastewater-treatments/rotating-biological-contactors

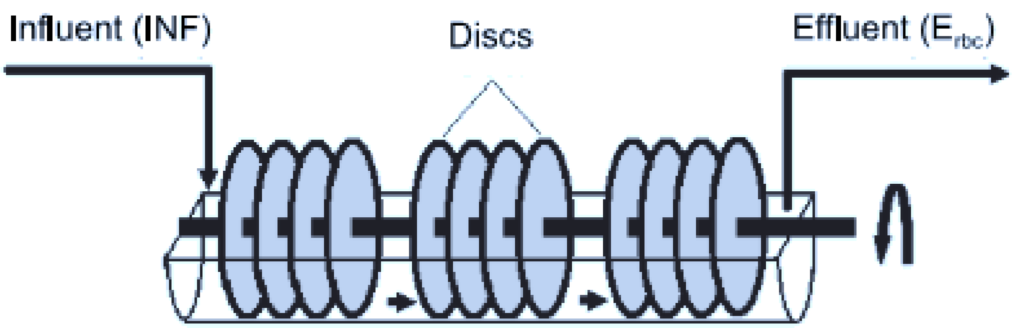


Figure 62: Schematic diagram of a rotating biological contactor (RBC) system (Tomas Zewski et al., 2015)

VI.5.3 Principle of activated sludge

This principle, which uses biology for the purification of wastewater, is based on the use of suspended microbial cultures. In a water treatment system, this principle is part of the secondary treatment processes. It involves the use of pools populated with bacteria and other microorganisms which evolve freely and aggregate into flakes to form suspended particles. The activated sludge process takes place in an aerobic biological reactor in which the microorganisms spontaneously group together into small aggregates called *bioflocs*. The combination of wastewater and bioflocs is referred to as mixed liquor.

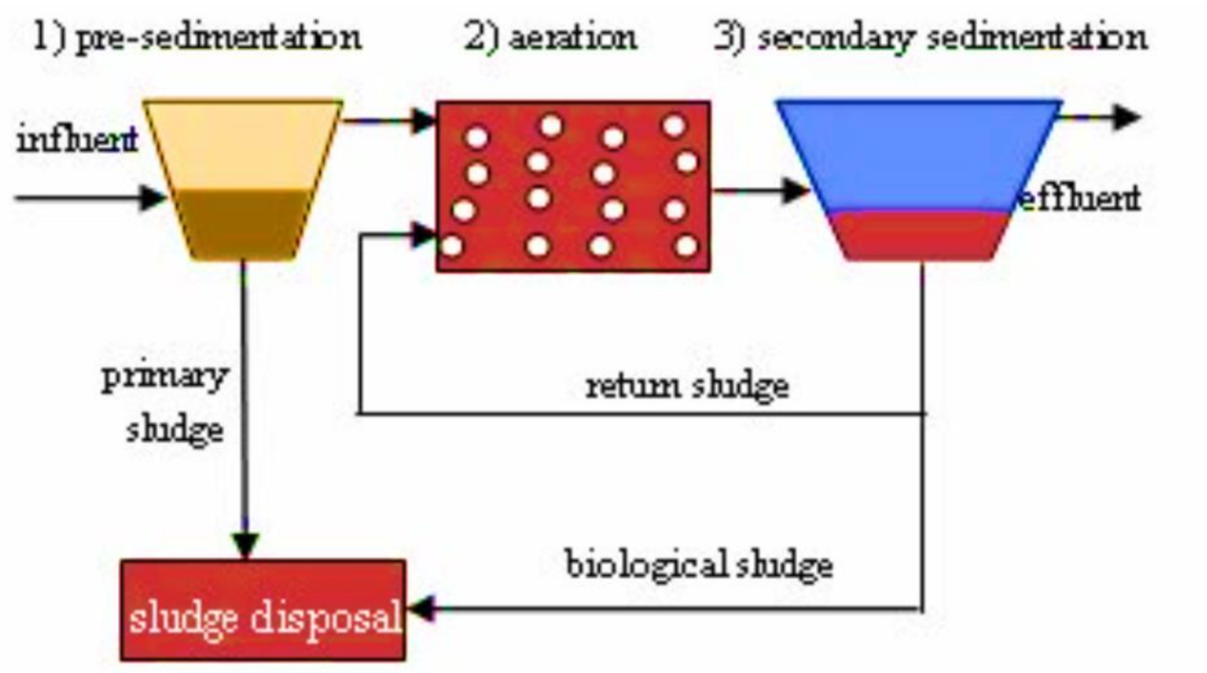


Figure 63 : The activated sludge treatment process with three main stages; 1) Presedimentation, 2) Aeration and 3) Secondary sedimentation (Heikkinen, et al., 2008)

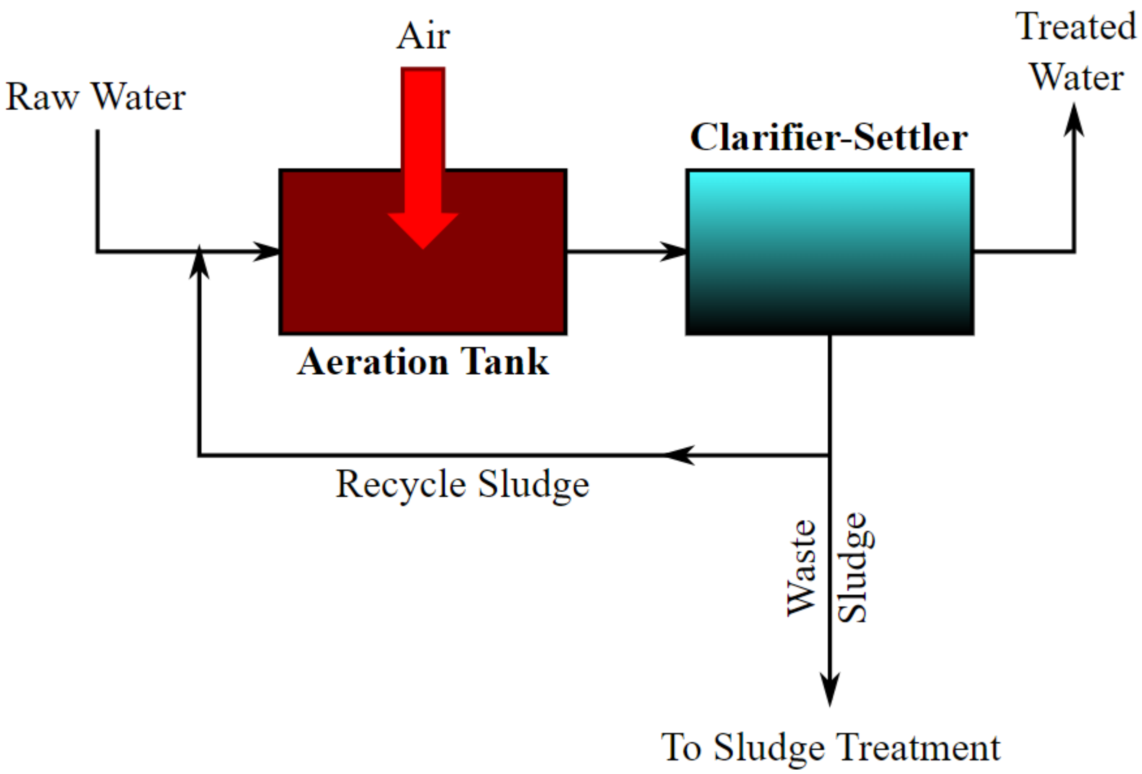


Figure 64: A generalized schematic diagram of an activated sludge process.

https://commons.wikimedia.org/

VI.5.4 The lagooning technique

The lagooning process purifies wastewater using sealed basins populated with microorganisms, algae, and aquatic plants. This biological treatment, which uses only solar energy, is renowned for being ecological and respectful of the environment. It is a more economical alternative to traditional physicochemical treatment methods. It turned out that the lagooning technique is particularly effective in eliminating pathogens (**Figure 65**).

Lagoon systems not only treat pollution; they also help to enrich the ecosystem. These installations, composed of artificial ponds, can operate independently or connected in series, which helps to increase their efficiency in the treatment of wastewater.

Figure 65 Lagoon systems **https://www.spicergroup.com/wastewater-collectiontreatment-1/oscoda-township-wastewater-lagoon-improvements**

Once the purification process is completed and the pollution levels meet the established discharge standards, the treated water is released into the natural environment, such as rivers or lakes. These waters then join the great hydrological cycle of nature. This cycle includes evaporation, condensation and precipitation. It thus allows purified water to circulate again in ecosystems, to contribute to aquatic and terrestrial life, and then eventually return to be used by humans and other living beings. The hydrological cycle is a crucial element in sustainable water management because it ensures that water remains clean and safe for all future uses.

Bibliographic references

Guyard, Christian. "Canalisations d'assainissement : à chaque matériau ses avantages." L'Eau, l'industrie, les nuisances 363 (2013) : 47-62.

Gabert, J. (2018). *Mémento de l'assainissement* (p. 844). Editions Quae.

Akan, A. O., & Houghtalen, R. J. (2003). *Urban hydrology, hydraulics, and stormwater quality: engineering applications and computer modeling*. John Wiley & Sons.

Valois, I. (2008). *Aménagement et entretien des ponceaux municipaux* (Doctoral dissertation, éditeur non identifié).

Commune de Saint-Philbert-de-Grand- (2018). Zonage d'assainissement pluvial, Notice de zonage. https : //www.stphilbert.fr/medias/2018/07/6.C.2-170674_StPhilbert_notice-de-zonage_V3.pdf

Guide méthodologique pour la gestion des eaux pluviales dans les projets d'aménagement (Dossier « loi sur l'eau » rubrique 2.1.5.0). Tome 2: Méthode d'investigation et de dimensionnement ; direction départementale des territoires et de la Mer de l'Hérault ; Février 2014.

H. Guerree., C. Gomella. (1982). Les eaux usées dans les agglomérations urbaines ou rurales. 2e édition, Paris, p146.

C. Coste., M. Loudet. (1980). Guide de l'assainissement en milieu urbain et rural. Edition du Moniteur, p 8 et p 9.

C. ROUYRRE., (2003). Guide de l'eau : Comment moins la polluer ? ; Comment la préserver ? Editions du seuil, février 2003, p 99.

Coopération des eaux usées ALGERO-ALLEMANDE., (2001). Epuration des eaux usées par procédé extensif: Le lagunage naturel. Ministère des ressources en eau, office national de l'assainissement, Alger.

Brière, François G. *Distribution et collecte des eaux*. Presses inter Polytechnique, 2012.

Hamouda, R. B. (2009). *Notions de mécanique des fluides : cours et exercices corrigés*. UVT.

Pamphlet, A. (1971). Engineering design handbook: Hydraulic fluids. *Headquarters, US army material command*.

Anctil, F., Rousselle, J., & Lauzon, N. (2012). *Hydrologie: cheminements de l'eau*. Presses inter Polytechnique.

Thomas, Pierre-François. Précis de Physique-chimie: cours et exercices. Editions Bréal, 2006.

Fauduet, Henri. *Mécanique des fluides et des solides appliquée à la chimie*. Lavoisier, 2011.

Dubaj, Denis, and Stéphane Gottar. "60 khôlles de Physique: Questions de cours et exercices incontournables: Classes préparatoires PCSI, MPSI et PTSI." (2019): 1-480.

Houben, G.J., Collins, S., Bakker, M. *et al.* Review: Horizontal, directionallydrilled and radial collector wells. *Hydrogeol J* **30**, 329–357 (2022). https://doi.org/10.1007/s10040-021-02425-w

Brière, F. G. (2012). *Distribution et collecte des eaux*. Presses inter Polytechnique.

Bélizaire, D. (1955). *Premier cours national post gradué sur l'irrigation, le drainage et la gestion des ressources hydriques*. ISSN 606-0534-5391. Révision et Edition: Humberto Pizarro.

Djaffar, Sabrina., & Kettab, A. H. M. E. D. (2015). Simulation stratégique, politique des ressources en eau en Algérie à l'horizon 2030. *Laboratoire de Recherches en Sciences de l'Eau (LRS-EAU) Ecole Nationale Polytechnique, 10*.

Vittone, R. (2010). *Bâtir: manuel de la construction*. PPUR Presses polytechniques.

Grosclaude, G. (1999). *L'eau: Tome 2: Usages et polluants*. Quae.

Tomaszewski, M., Pypeć, K., Zgórska, A., & Ziembińska - Buczyńska, A. (2015). Toxicity analysis of coke wastewater treated in a rotating biological contactor and a membrane bioreactor. *Environmental Biotechnology*, *11*(2), 34-40.

Heikkinen, M., Heikkinen, T., &Hiltunen, Y. (2008, October). Process states and their sub-models using self-organizing maps in an activated sludge treatment plant. In *Proc. 48th Scandinavian Conference on Simulation and Modeling (SIMS 2008)[CD-ROM], Oslo University College*.

Sites Web :

https://www.sciences.univ-nantes.fr/sites/claude_saintblanquet/synophys/211exsfl/211exsfl.htm

https://f2school.com/wp-content/uploads/2019/11/M%C3%A9canique-des-fluides-TDcorr-01.pdf

https://sigesaqi.brgm.fr/

Table 18 : Rugosités de Manning (https://sites.uclouvain.be/didacticiel-hydraulique/lecons/lecon_ii_1/tableau_n.htm**)**

Nature of surfaces	Wall condition			
	Perfect	Good	Quite good	Bad
A) Artificial canals				
Smoothed Cement	0.01	0.011	0.012	0.013
Cement mortar	0.011	0.012	0.013	0.015
Planed wooden aqueducts	0.01	0.012	0.013	0.014
Unplaned wooden aqueducts	0.011	0.013	0.014	0.015
Channels lined with concrete	0.012	0.014	0.016	0.018
Raw rubble stone	0.017	0.02	0.025	0.03
Dry stones	0.025	0.03	0.033	0.035
Dressed rubble stones	0.013	0.014	0.015	0.017
Metal aqueducts with smooth semi-circular section	0.011	0.012	0.013	0.015
Metal aqueducts with pleated semi-circular section	0.0225	0.025	0.0275	0.030
Straight and uniform earth canals	0.017	0.020	0.0225	0.025
Canals with stones, smooth and uniform	0.025	0.030	0.033	0.035
Canals with stones, rough and irregular	0.035	0.040	0.045	-
Wide meandering earthen canals	0.0225	0.025	0.0275	0.030
Dredged earthen canals	0.025	0.027	0.030	0.033
Stone-sided and earth-bottomed canals	0.028	0.030	0.033	0.035
B) Natural watercourses				
1) Clean, straight-line banks	0.025	0.027	0.030	0.033
2) Same as 1, with a few herbs and stones	0.030	0.033	0.035	0.040
3) With meanders, with some ponds and shallow places, clean	0.035	0.040	0.045	0.050
4) Same as 3, water at low flow, slope and lower sections	0.040	0.045	0.050	0.055
5) Same as 3, with a few herbs and stones	0.033	0.035	0.040	0.045
6) Same as 4, with stones	0.045	0.050	0.055	0.060
7) Areas with slow flowing water, with grass or very deep pits	0.050	0.060	0.070	0.080
8) Areas with lots of weeds	0.075	0.100	0.125	0.150

V.5 TABLES DE COLEBROOK

Pertes de charge linéaires j évaluées par la formule de Colebrook pour de l'eau à 10°C.

J est donné en mètre par mètre pour deux rugosités équivalentes :

k = 0,0001 mm valeur préconisée pour toutes les conduites de conception récente en service

k = 0,002 mm valeur possible pour des conduites très anciennes (fonte non revêtue...)

D= 40 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
0,6	0,0093	0,0212	0,48	0,01
0,7	0,0124	0,0288	0,56	0,02
0,8	0,0159	0,0375	0,64	0,02
0,9	0,0197	0,0474	0,72	0,03
1,0	0,0240	0,0584	0,80	0,03
1,1	0,0287	0,0706	0,88	0,04
1,2	0,0338	0,0839	0,95	0,05
1,3	0,0393	0,0984	1,03	0,05
1,4	0,0453	0,1141	1,11	0,06
1,5	0,0516	0,1309	1,19	0,07
1,6	0,0583	0,1488	1,27	0,08
1,7	0,0654	0,1679	1,35	0,09
1,8	0,0729	0,1882	1,43	0,10
1,9	0,0809	0,2096	1,51	0,12
2,0	0,0892	0,2322	1,59	0,13
2,1	0,0979	0,2559	1,67	0,14
2,2	0,1071	0,2808	1,75	0,16
2,3	0,1166	0,3068	1,83	0,17
2,4	0,1265	0,3340	1,91	0,19
2,5	0,1369	0,3623	1,99	0,20
2,6	0,1476	0,3918	2,07	0,22
2,7	0,1588	0,4224	2,15	0,24
2,8	0,1703	0,4542	2,23	0,25
2,9	0,1823	0,4871	2,31	0,27
3,0	0,1946	0,5212	2,39	0,29
3,1	0,2074	0,5565	2,47	0,31
3,2	0,2205	0,5929	2,55	0,33
3,3	0,2341	0,6304	2,63	0,35
3,4	0,2480	0,6692	2,71	0,37
3,5	0,2624	0,7090	2,79	0,40
3,6	0,2772	0,7500	2,86	0,42
3,7	0,2923	0,7922	2,94	0,44
3,8	0,3079	0,8355	3,02	0,47
3,9	0,3239	0,8800	3,10	0,49
4,0	0,3402	0,9256	3,18	0,52

D= 50 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
0,8	0,0052	0,0112	0,41	0,01
1,0	0,0079	0,0174	0,51	0,01
1,2	0,0111	0,0250	0,61	0,02
1,4	0,0147	0,0340	0,71	0,03
1,6	0,0189	0,0443	0,81	0,03
1,8	0,0236	0,0560	0,92	0,04
2,0	0,0288	0,0690	1,02	0,05
2,2	0,0344	0,0834	1,12	0,06
2,4	0,0406	0,0992	1,22	0,08
2,6	0,0473	0,1164	1,32	0,09
2,8	0,0544	0,1349	1,43	0,10
3,0	0,0621	0,1548	1,53	0,12
3,2	0,0703	0,1760	1,63	0,14
3,4	0,0789	0,1986	1,73	0,15
3,6	0,0881	0,2226	1,83	0,17
3,8	0,0977	0,2480	1,94	0,19
4,0	0,1079	0,2747	2,04	0,21
4,2	0,1185	0,3027	2,14	0,23
4,4	0,1297	0,3322	2,24	0,26
4,6	0,1413	0,3630	2,34	0,28
4,8	0,1534	0,3952	2,44	0,30
5,0	0,1661	0,4287	2,55	0,33
5,2	0,1792	0,4636	2,65	0,36
5,4	0,1928	0,4999	2,75	0,39
5,6	0,2069	0,5376	2,85	0,41
5,8	0,2216	0,5766	2,95	0,44
6,0	0,2367	0,6170	3,06	0,48
6,2	0,2523	0,6587	3,16	0,51
6,4	0,2684	0,7018	3,26	0,54
6,6	0,2850	0,7463	3,36	0,58
6,8	0,3020	0,7921	3,46	0,61
7,0	0,3196	0,8393	3,57	0,65
7,2	0,3377	0,8879	3,67	0,69
7,4	0,3563	0,9379	3,77	0,72
7,6	0,3753	0,9892	3,87	0,76

D= 60 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
1,4	0,0059	0,0127	0,50	0,01
1,6	0,0076	0,0165	0,57	0,02
1,8	0,0095	0,0209	0,64	0,02
2,0	0,0115	0,0257	0,71	0,03
2,2	0,0138	0,0311	0,78	0,03
2,4	0,0162	0,0370	0,85	0,04
2,6	0,0188	0,0434	0,92	0,04
2,8	0,0216	0,0503	0,99	0,05
3,0	0,0246	0,0576	1,06	0,06
3,2	0,0279	0,0655	1,13	0,07
3,4	0,0312	0,0740	1,20	0,07
3,6	0,0348	0,0829	1,27	0,08
3,8	0,0386	0,0923	1,34	0,09
4,0	0,0426	0,1022	1,41	0,10
4,2	0,0467	0,1127	1,49	0,11
4,4	0,0511	0,1236	1,56	0,12
4,6	0,0556	0,1351	1,63	0,13
4,8	0,0603	0,1470	1,70	0,15
5,0	0,0653	0,1595	1,77	0,16
5,2	0,0704	0,1725	1,84	0,17
5,4	0,0757	0,1860	1,91	0,19
5,6	0,0812	0,2000	1,98	0,20
5,8	0,0868	0,2145	2,05	0,21
6,0	0,0927	0,2295	2,12	0,23
6,2	0,0988	0,2450	2,19	0,25
6,4	0,1050	0,2610	2,26	0,26
6,6	0,1114	0,2775	2,33	0,28
6,8	0,1181	0,2946	2,41	0,29
7,0	0,1249	0,3121	2,48	0,31
7,2	0,1319	0,3302	2,55	0,33
7,4	0,1391	0,3487	2,62	0,35
7,6	0,1465	0,3678	2,69	0,37
7,8	0,1541	0,3874	2,76	0,39
8,0	0,1618	0,4074	2,83	0,41
8,2	0,1698	0,4280	2,90	0,43

D= 80 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
2,5	0,0042	0,0085	0,50	0,01
3,0	0,0058	0,0122	0,60	0,02
3,5	0,0078	0,0166	0,70	0,02
4,0	0,0100	0,0217	0,80	0,03
4,5	0,0125	0,0274	0,90	0,04
5,0	0,0152	0,0338	0,99	0,05
5,5	0,0182	0,0408	1,09	0,06
6,0	0,0215	0,0486	1,19	0,07
6,5	0,0250	0,0569	1,29	0,09
7,0	0,0288	0,0660	1,39	0,10
7,5	0,0328	0,0757	1,49	0,11
8,0	0,0371	0,0861	1,59	0,13
8,5	0,0417	0,0972	1,69	0,15
9,0	0,0466	0,1089	1,79	0,16
9,5	0,0517	0,1213	1,89	0,18
10,0	0,0570	0,1344	1,99	0,20
10,5	0,0627	0,1481	2,09	0,22
11,0	0,0685	0,1625	2,19	0,24
11,5	0,0747	0,1776	2,29	0,27
12,0	0,0811	0,1934	2,39	0,29
12,5	0,0878	0,2098	2,49	0,32
13,0	0,0947	0,2269	2,59	0,34
13,5	0,1019	0,2446	2,69	0,37
14,0	0,1094	0,2630	2,79	0,40
14,5	0,1171	0,2821	2,88	0,42
15,0	0,1251	0,3019	2,98	0,45
15,5	0,1333	0,3223	3,08	0,48
16,0	0,1418	0,3434	3,18	0,52
16,5	0,1506	0,3651	3,28	0,55
17,0	0,1596	0,3876	3,38	0,58
17,5	0,1689	0,4107	3,48	0,62
18,0	0,1785	0,4344	3,58	0,65
18,5	0,1883	0,4589	3,68	0,69
19,0	0,1984	0,4840	3,78	0,73
19,5	0,2087	0,5098	3,88	0,77

D= 100 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
3,5	0,0026	0,0050	0,45	0,01
4,0	0,0033	0,0066	0,51	0,01
4,5	0,0041	0,0083	0,57	0,02
5,0	0,0050	0,0102	0,64	0,02
5,5	0,0059	0,0123	0,70	0,02
6,0	0,0070	0,0146	0,76	0,03
6,5	0,0081	0,0172	0,83	0,03
7,0	0,0093	0,0199	0,89	0,04
7,5	0,0106	0,0228	0,95	0,05
8,0	0,0120	0,0260	1,02	0,05
8,5	0,0135	0,0293	1,08	0,06
9,0	0,0150	0,0328	1,15	0,07
9,5	0,0166	0,0365	1,21	0,07
10,0	0,0184	0,0405	1,27	0,08
10,5	0,0201	0,0446	1,34	0,09
11,0	0,0220	0,0489	1,40	0,10
11,5	0,0240	0,0535	1,46	0,11
12,0	0,0260	0,0582	1,53	0,12
12,5	0,0281	0,0631	1,59	0,13
13,0	0,0303	0,0683	1,66	0,14
13,5	0,0326	0,0736	1,72	0,15
14,0	0,0349	0,0791	1,78	0,16
14,5	0,0374	0,0849	1,85	0,17
15,0	0,0399	0,0908	1,91	0,19
15,5	0,0425	0,0970	1,97	0,20
16,0	0,0452	0,1033	2,04	0,21
16,5	0,0479	0,1098	2,10	0,22
17,0	0,0508	0,1166	2,16	0,24
17,5	0,0537	0,1235	2,23	0,25
18,0	0,0567	0,1307	2,29	0,27
18,5	0,0598	0,1380	2,36	0,28
19,0	0,0630	0,1456	2,42	0,30
19,5	0,0662	0,1533	2,48	0,31
20,0	0,0696	0,1612	2,55	0,33
20,5	0,0730	0,1694	2,61	0,35

D= 125 mm				
	k= 0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
6,0	0,0023	0,0044	0,49	0,01
7,0	0,0031	0,0060	0,57	0,02
8,0	0,0039	0,0079	0,65	0,02
9,0	0,0049	0,0099	0,73	0,03
10,0	0,0060	0,0123	0,81	0,03
11,0	0,0072	0,0148	0,90	0,04
12,0	0,0084	0,0176	0,98	0,05
13,0	0,0098	0,0206	1,06	0,06
14,0	0,0113	0,0239	1,14	0,07
15,0	0,0129	0,0275	1,22	0,08
16,0	0,0145	0,0312	1,30	0,09
17,0	0,0163	0,0352	1,39	0,10
18,0	0,0182	0,0395	1,47	0,11
19,0	0,0202	0,0440	1,55	0,12
20,0	0,0223	0,0487	1,63	0,14
21,0	0,0245	0,0537	1,71	0,15
22,0	0,0268	0,0589	1,79	0,16
23,0	0,0292	0,0644	1,87	0,18
24,0	0,0317	0,0701	1,96	0,19
25,0	0,0342	0,0760	2,04	0,21
26,0	0,0369	0,0822	2,12	0,23
27,0	0,0397	0,0886	2,20	0,25
28,0	0,0426	0,0953	2,28	0,27
29,0	0,0456	0,1022	2,36	0,28
30,0	0,0487	0,1094	2,44	0,30
31,0	0,0519	0,1168	2,53	0,33
32,0	0,0552	0,1244	2,61	0,35
33,0	0,0586	0,1323	2,69	0,37
34,0	0,0621	0,1404	2,77	0,39
35,0	0,0657	0,1488	2,85	0,41
36,0	0,0694	0,1574	2,93	0,44
37,0	0,0732	0,1662	3,02	0,46
38,0	0,0771	0,1753	3,10	0,49
39,0	0,0811	0,1846	3,18	0,51
40,0	0,0852	0,1942	3,26	0,54

Pertes de charge linéaires j évaluées par la formule de Colebrook pour de l'eau à 10°C.

J est donné en mètre par mètre pour deux rugosités équivalentes :

k = 0,0001 mm valeur préconisée pour toutes les conduites de conception récente en service

k = 0,002 mm valeur possible pour des conduites très anciennes (fonte non revêtue...)

D= 150 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
8,0	0,0016	0,0030	0,45	0,01
10,0	0,0024	0,0046	0,57	0,02
12,0	0,0034	0,0067	0,68	0,02
14,0	0,0045	0,0090	0,79	0,03
16,0	0,0058	0,0118	0,91	0,04
18,0	0,0073	0,0149	1,02	0,05
20,0	0,0089	0,0184	1,13	0,07
22,0	0,0106	0,0222	1,24	0,08
24,0	0,0126	0,0264	1,36	0,09
26,0	0,0146	0,0310	1,47	0,11
28,0	0,0169	0,0359	1,58	0,13
30,0	0,0192	0,0412	1,70	0,15
32,0	0,0218	0,0469	1,81	0,17
34,0	0,0245	0,0529	1,92	0,19
36,0	0,0273	0,0593	2,04	0,21
38,0	0,0303	0,0661	2,15	0,24
40,0	0,0335	0,0732	2,26	0,26
42,0	0,0368	0,0807	2,38	0,29
44,0	0,0403	0,0885	2,49	0,32
46,0	0,0439	0,0967	2,60	0,35
48,0	0,0477	0,1053	2,72	0,38
50,0	0,0517	0,1142	2,83	0,41
52,0	0,0558	0,1235	2,94	0,44
54,0	0,0600	0,1332	3,06	0,48
56,0	0,0644	0,1432	3,17	0,51
58,0	0,0690	0,1536	3,28	0,55
60,0	0,0737	0,1644	3,40	0,59
62,0	0,0786	0,1755	3,51	0,63
64,0	0,0836	0,1870	3,62	0,67
66,0	0,0888	0,1989	3,73	0,71
68,0	0,0941	0,2111	3,85	0,75
70,0	0,0996	0,2237	3,96	0,80
72,0	0,1053	0,2366	4,07	0,85
74,0	0,1111	0,2499	4,19	0,89
76,0	0,1170	0,2636	4,30	0,94

D= 200 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
15,0	0,0012	0,0022	0,48	0,01
17,5	0,0016	0,0030	0,56	0,02
20,0	0,0021	0,0040	0,64	0,02
22,5	0,0026	0,0050	0,72	0,03
25,0	0,0032	0,0062	0,80	0,03
27,5	0,0038	0,0075	0,88	0,04
30,0	0,0045	0,0089	0,95	0,05
32,5	0,0052	0,0104	1,03	0,05
35,0	0,0060	0,0121	1,11	0,06
37,5	0,0069	0,0139	1,19	0,07
40,0	0,0078	0,0158	1,27	0,08
42,5	0,0087	0,0178	1,35	0,09
45,0	0,0098	0,0199	1,43	0,10
47,5	0,0108	0,0222	1,51	0,12
50,0	0,0119	0,0246	1,59	0,13
52,5	0,0131	0,0271	1,67	0,14
55,0	0,0143	0,0298	1,75	0,16
57,5	0,0156	0,0325	1,83	0,17
60,0	0,0169	0,0354	1,91	0,19
62,5	0,0183	0,0384	1,99	0,20
65,0	0,0198	0,0415	2,07	0,22
67,5	0,0213	0,0448	2,15	0,24
70,0	0,0228	0,0481	2,23	0,25
72,5	0,0244	0,0516	2,31	0,27
75,0	0,0261	0,0552	2,39	0,29
77,5	0,0278	0,0590	2,47	0,31
80,0	0,0296	0,0628	2,55	0,33
82,5	0,0314	0,0668	2,63	0,35
85,0	0,0333	0,0709	2,71	0,37
87,5	0,0352	0,0751	2,79	0,40
90,0	0,0372	0,0795	2,86	0,42
92,5	0,0392	0,0840	2,94	0,44
95,0	0,0413	0,0886	3,02	0,47
97,5	0,0434	0,0933	3,10	0,49
100,0	0,0456	0,0981	3,18	0,52

D= 250 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
24,0	0,0010	0,0017	0,49	0,01
28,0	0,0013	0,0024	0,57	0,02
32,0	0,0017	0,0031	0,65	0,02
36,0	0,0021	0,0039	0,73	0,03
40,0	0,0025	0,0048	0,81	0,03
44,0	0,0030	0,0058	0,90	0,04
48,0	0,0036	0,0069	0,98	0,05
52,0	0,0042	0,0081	1,06	0,06
56,0	0,0048	0,0094	1,14	0,07
60,0	0,0055	0,0108	1,22	0,08
64,0	0,0062	0,0123	1,30	0,09
68,0	0,0070	0,0139	1,39	0,10
72,0	0,0078	0,0155	1,47	0,11
76,0	0,0086	0,0173	1,55	0,12
80,0	0,0095	0,0192	1,63	0,14
84,0	0,0104	0,0211	1,71	0,15
88,0	0,0114	0,0232	1,79	0,16
92,0	0,0124	0,0253	1,87	0,18
96,0	0,0135	0,0276	1,96	0,19
100,0	0,0146	0,0299	2,04	0,21
104,0	0,0158	0,0323	2,12	0,23
108,0	0,0170	0,0349	2,20	0,25
112,0	0,0182	0,0375	2,28	0,27
116,0	0,0195	0,0402	2,36	0,28
120,0	0,0208	0,0430	2,44	0,30
124,0	0,0222	0,0459	2,53	0,33
128,0	0,0236	0,0489	2,61	0,35
132,0	0,0250	0,0520	2,69	0,37
136,0	0,0265	0,0552	2,77	0,39
140,0	0,0280	0,0585	2,85	0,41
144,0	0,0296	0,0619	2,93	0,44
148,0	0,0313	0,0654	3,02	0,46
152,0	0,0329	0,0690	3,10	0,49
156,0	0,0346	0,0726	3,18	0,51
160,0	0,0364	0,0764	3,26	0,54

D= 300 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
35,0	0,00080	0,00141	0,50	0,01
40,0	0,00103	0,00184	0,57	0,02
45,0	0,00128	0,00232	0,64	0,02
50,0	0,00156	0,00286	0,71	0,03
55,0	0,00187	0,00346	0,78	0,03
60,0	0,00220	0,00411	0,85	0,04
65,0	0,00256	0,00482	0,92	0,04
70,0	0,00294	0,00558	0,99	0,05
75,0	0,00335	0,00641	1,06	0,06
80,0	0,00379	0,00728	1,13	0,07
85,0	0,00425	0,00822	1,20	0,07
90,0	0,00474	0,00921	1,27	0,08
95,0	0,00526	0,01026	1,34	0,09
100,0	0,00580	0,01136	1,41	0,10
105,0	0,00637	0,01252	1,49	0,11
110,0	0,00696	0,01374	1,56	0,12
115,0	0,00759	0,01501	1,63	0,13
120,0	0,00823	0,01634	1,70	0,15
125,0	0,00890	0,01773	1,77	0,16
130,0	0,00960	0,01917	1,84	0,17
135,0	0,01033	0,02067	1,91	0,19
140,0	0,01108	0,02222	1,98	0,20
145,0	0,01186	0,02383	2,05	0,21
150,0	0,01266	0,02550	2,12	0,23
155,0	0,01349	0,02722	2,19	0,25
160,0	0,01434	0,02900	2,26	0,26
165,0	0,01522	0,03084	2,33	0,28
170,0	0,01613	0,03273	2,41	0,29
175,0	0,01706	0,03468	2,48	0,31
180,0	0,01802	0,03669	2,55	0,33
185,0	0,01901	0,03875	2,62	0,35
190,0	0,02002	0,04087	2,69	0,37
195,0	0,02106	0,04305	2,76	0,39
200,0	0,02212	0,04528	2,83	0,41
205,0	0,02321	0,04757	2,90	0,43

D= 350 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
40,0	0,00048	0,00082	0,42	0,01
50,0	0,00073	0,00127	0,52	0,01
60,0	0,00102	0,00182	0,62	0,02
70,0	0,00136	0,00247	0,73	0,03
80,0	0,00175	0,00322	0,83	0,04
90,0	0,00219	0,00407	0,94	0,04
100,0	0,00267	0,00502	1,04	0,06
110,0	0,00320	0,00607	1,14	0,07
120,0	0,00378	0,00722	1,25	0,08
130,0	0,00441	0,00847	1,35	0,09
140,0	0,00508	0,00982	1,46	0,11
150,0	0,00580	0,01126	1,56	0,12
160,0	0,00656	0,01281	1,66	0,14
170,0	0,00737	0,01445	1,77	0,16
180,0	0,00823	0,01620	1,87	0,18
190,0	0,00914	0,01805	1,97	0,20
200,0	0,01009	0,01999	2,08	0,22
210,0	0,01109	0,02203	2,18	0,24
220,0	0,01213	0,02418	2,29	0,27
230,0	0,01322	0,02642	2,39	0,29
240,0	0,01436	0,02876	2,49	0,32
250,0	0,01554	0,03120	2,60	0,34
260,0	0,01678	0,03374	2,70	0,37
270,0	0,01805	0,03638	2,81	0,40
280,0	0,01938	0,03912	2,91	0,43
290,0	0,02075	0,04196	3,01	0,46
300,0	0,02217	0,04490	3,12	0,50
310,0	0,02363	0,04794	3,22	0,53
320,0	0,02514	0,05108	3,33	0,56
330,0	0,02670	0,05431	3,43	0,60
340,0	0,02830	0,05765	3,53	0,64
350,0	0,02995	0,06109	3,64	0,67
360,0	0,03165	0,06462	3,74	0,71
370,0	0,03339	0,06826	3,85	0,75
380,0	0,03518	0,07199	3,95	0,80

D= 400 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
60,0	0,00053	0,00090	0,48	0,01
70,0	0,00070	0,00122	0,56	0,02
80,0	0,00090	0,00159	0,64	0,02
90,0	0,00112	0,00201	0,72	0,03
100,0	0,00137	0,00248	0,80	0,03
110,0	0,00164	0,00300	0,88	0,04
120,0	0,00194	0,00356	0,95	0,05
130,0	0,00225	0,00418	1,03	0,05
140,0	0,00259	0,00484	1,11	0,06
150,0	0,00296	0,00556	1,19	0,07
160,0	0,00335	0,00632	1,27	0,08
170,0	0,00376	0,00713	1,35	0,09
180,0	0,00419	0,00799	1,43	0,10
190,0	0,00465	0,00890	1,51	0,12
200,0	0,00513	0,00986	1,59	0,13
210,0	0,00563	0,01086	1,67	0,14
220,0	0,00616	0,01192	1,75	0,16
230,0	0,00671	0,01303	1,83	0,17
240,0	0,00729	0,01418	1,91	0,19
250,0	0,00788	0,01538	1,99	0,20
260,0	0,00850	0,01663	2,07	0,22
270,0	0,00915	0,01794	2,15	0,24
280,0	0,00981	0,01929	2,23	0,25
290,0	0,01051	0,02068	2,31	0,27
300,0	0,01122	0,02213	2,39	0,29
310,0	0,01196	0,02363	2,47	0,31
320,0	0,01272	0,02517	2,55	0,33
330,0	0,01350	0,02677	2,63	0,35
340,0	0,01431	0,02841	2,71	0,37
350,0	0,01514	0,03011	2,79	0,40
360,0	0,01599	0,03185	2,86	0,42
370,0	0,01686	0,03364	2,94	0,44
380,0	0,01776	0,03548	3,02	0,47
390,0	0,01869	0,03737	3,10	0,49
400,0	0,01963	0,03930	3,18	0,52

Pertes de charge linéaires j évaluées par la formule de Colebrook pour de l'eau à 10°C.

J est donné en mètre par mètre pour deux rugosités équivalentes :
k = 0,0001 mm valeur préconisée pour toutes les conduites de conception récente en service
k = 0,002 mm valeur possible pour des conduites très anciennes (fonte non revêtue...)

D= 500 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
80,0	0,00030	0,00049	0,41	0,01
100,0	0,00045	0,00077	0,51	0,01
120,0	0,00064	0,00110	0,61	0,02
140,0	0,00085	0,00149	0,71	0,03
160,0	0,00109	0,00195	0,81	0,03
180,0	0,00137	0,00246	0,92	0,04
200,0	0,00167	0,00303	1,02	0,05
220,0	0,00200	0,00367	1,12	0,06
240,0	0,00236	0,00436	1,22	0,08
260,0	0,00275	0,00511	1,32	0,09
280,0	0,00317	0,00593	1,43	0,10
300,0	0,00362	0,00680	1,53	0,12
320,0	0,00410	0,00774	1,63	0,14
340,0	0,00461	0,00873	1,73	0,15
360,0	0,00514	0,00978	1,83	0,17
380,0	0,00571	0,01090	1,94	0,19
400,0	0,00630	0,01207	2,04	0,21
420,0	0,00693	0,01331	2,14	0,23
440,0	0,00758	0,01460	2,24	0,26
460,0	0,00826	0,01596	2,34	0,28
480,0	0,00897	0,01737	2,44	0,30
500,0	0,00971	0,01885	2,55	0,33
520,0	0,01048	0,02038	2,65	0,36
540,0	0,01128	0,02197	2,75	0,39
560,0	0,01211	0,02363	2,85	0,41
580,0	0,01296	0,02534	2,95	0,44
600,0	0,01385	0,02712	3,06	0,48
620,0	0,01476	0,02895	3,16	0,51
640,0	0,01571	0,03085	3,26	0,54
660,0	0,01668	0,03280	3,36	0,58
680,0	0,01768	0,03482	3,46	0,61
700,0	0,01871	0,03689	3,57	0,65
720,0	0,01977	0,03903	3,67	0,69
740,0	0,02086	0,04122	3,77	0,72
760,0	0,02198	0,04348	3,87	0,76

D= 600 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
125,0	0,00028	0,00046	0,44	0,01
150,0	0,00039	0,00066	0,53	0,01
175,0	0,00052	0,00089	0,62	0,02
200,0	0,00067	0,00116	0,71	0,03
225,0	0,00084	0,00147	0,80	0,03
250,0	0,00102	0,00181	0,88	0,04
275,0	0,00123	0,00219	0,97	0,05
300,0	0,00145	0,00260	1,06	0,06
325,0	0,00169	0,00305	1,15	0,07
350,0	0,00194	0,00354	1,24	0,08
375,0	0,00222	0,00406	1,33	0,09
400,0	0,00251	0,00461	1,41	0,10
425,0	0,00282	0,00521	1,50	0,12
450,0	0,00315	0,00583	1,59	0,13
475,0	0,00349	0,00650	1,68	0,14
500,0	0,00385	0,00720	1,77	0,16
525,0	0,00423	0,00793	1,86	0,18
550,0	0,00463	0,00870	1,95	0,19
575,0	0,00505	0,00951	2,03	0,21
600,0	0,00548	0,01035	2,12	0,23
625,0	0,00593	0,01123	2,21	0,25
650,0	0,00640	0,01215	2,30	0,27
675,0	0,00688	0,01310	2,39	0,29
700,0	0,00739	0,01408	2,48	0,31
725,0	0,00791	0,01511	2,56	0,34
750,0	0,00845	0,01616	2,65	0,36
775,0	0,00900	0,01726	2,74	0,38
800,0	0,00957	0,01839	2,83	0,41
825,0	0,01017	0,01955	2,92	0,43
850,0	0,01078	0,02075	3,01	0,46
875,0	0,01140	0,02199	3,09	0,49
900,0	0,01205	0,02326	3,18	0,52
925,0	0,01271	0,02457	3,27	0,55
950,0	0,01339	0,02591	3,36	0,58
975,0	0,01408	0,02729	3,45	0,61

D= 700 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
150	0,00018	0,00029	0,39	0,01
200	0,00031	0,00052	0,52	0,01
250	0,00048	0,00080	0,65	0,02
300	0,00067	0,00116	0,78	0,03
350	0,00090	0,00157	0,91	0,04
400	0,00116	0,00205	1,04	0,06
450	0,00145	0,00259	1,17	0,07
500	0,00177	0,00320	1,30	0,09
550	0,00213	0,00386	1,43	0,10
600	0,00251	0,00459	1,56	0,12
650	0,00293	0,00539	1,69	0,15
700	0,00338	0,00625	1,82	0,17
750	0,00386	0,00717	1,95	0,19
800	0,00438	0,00816	2,08	0,22
850	0,00492	0,00920	2,21	0,25
900	0,00550	0,01032	2,34	0,28
950	0,00610	0,01149	2,47	0,31
1 000	0,00674	0,01273	2,60	0,34
1 050	0,00742	0,01403	2,73	0,38
1 100	0,00812	0,01540	2,86	0,42
1 150	0,00885	0,01683	2,99	0,46
1 200	0,00962	0,01832	3,12	0,50
1 250	0,01042	0,01987	3,25	0,54
1 300	0,01124	0,02149	3,38	0,58
1 350	0,01211	0,02318	3,51	0,63
1 400	0,01300	0,02492	3,64	0,67
1 450	0,01392	0,02673	3,77	0,72
1 500	0,01488	0,02860	3,90	0,77
1 550	0,01586	0,03054	4,03	0,83
1 600	0,01688	0,03254	4,16	0,88
1 650	0,01793	0,03460	4,29	0,94
1 700	0,01901	0,03673	4,42	0,99
1 750	0,02013	0,03892	4,55	1,05
1 800	0,02127	0,04117	4,68	1,11
1 850	0,02245	0,04349	4,81	1,18

D= 800 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
250	0,00025	0,00040	0,50	0,01
300	0,00035	0,00057	0,60	0,02
350	0,00046	0,00078	0,70	0,02
400	0,00059	0,00102	0,80	0,03
450	0,00074	0,00128	0,90	0,04
500	0,00091	0,00158	0,99	0,05
550	0,00109	0,00191	1,09	0,06
600	0,00128	0,00228	1,19	0,07
650	0,00150	0,00267	1,29	0,09
700	0,00172	0,00309	1,39	0,10
750	0,00197	0,00355	1,49	0,11
800	0,00223	0,00404	1,59	0,13
850	0,00250	0,00456	1,69	0,15
900	0,00279	0,00511	1,79	0,16
950	0,00310	0,00569	1,89	0,18
1 000	0,00343	0,00630	1,99	0,20
1 050	0,00376	0,00694	2,09	0,22
1 100	0,00412	0,00762	2,19	0,24
1 150	0,00449	0,00833	2,29	0,27
1 200	0,00488	0,00906	2,39	0,29
1 250	0,00528	0,00983	2,49	0,32
1 300	0,00570	0,01064	2,59	0,34
1 350	0,00613	0,01147	2,69	0,37
1 400	0,00658	0,01233	2,79	0,40
1 450	0,00704	0,01323	2,88	0,42
1 500	0,00753	0,01415	2,98	0,45
1 550	0,00802	0,01511	3,08	0,48
1 600	0,00853	0,01610	3,18	0,52
1 650	0,00906	0,01712	3,28	0,55
1 700	0,00961	0,01817	3,38	0,58
1 750	0,01017	0,01925	3,48	0,62
1 800	0,01074	0,02037	3,58	0,65
1 850	0,01133	0,02151	3,68	0,69
1 900	0,01194	0,02269	3,78	0,73
1 950	0,01256	0,02390	3,88	0,77

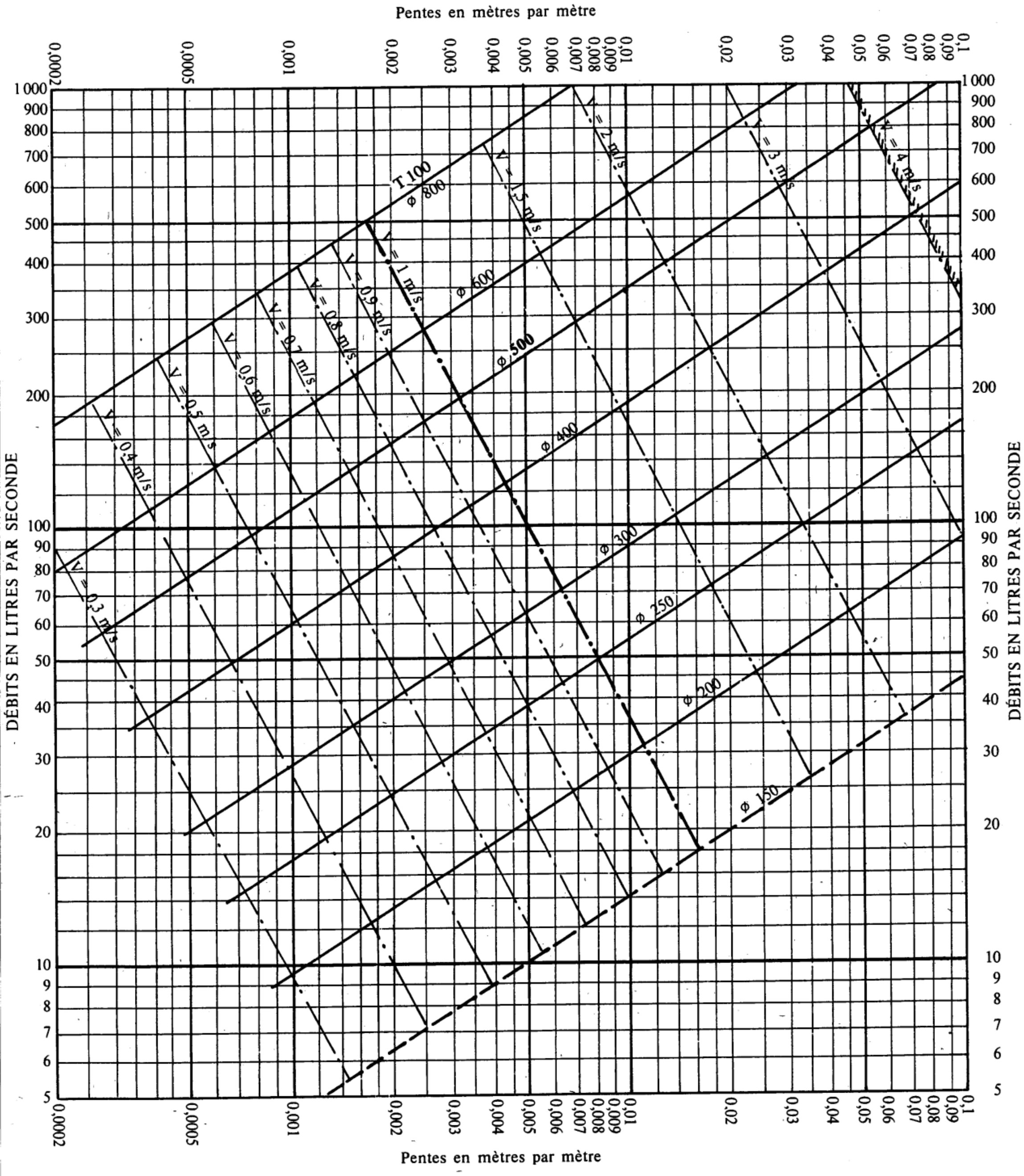
D= 900 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
300	0,00019	0,00031	0,47	0,01
350	0,00026	0,00042	0,55	0,02
400	0,00033	0,00055	0,63	0,02
450	0,00041	0,00069	0,71	0,03
500	0,00050	0,00085	0,79	0,03
550	0,00060	0,00103	0,86	0,04
600	0,00071	0,00123	0,94	0,05
650	0,00083	0,00144	1,02	0,05
700	0,00095	0,00167	1,10	0,06
750	0,00109	0,00191	1,18	0,07
800	0,00123	0,00217	1,26	0,08
850	0,00138	0,00245	1,34	0,09
900	0,00154	0,00275	1,41	0,10
950	0,00171	0,00306	1,49	0,11
1 000	0,00189	0,00339	1,57	0,13
1 050	0,00208	0,00374	1,65	0,14
1 100	0,00227	0,00410	1,73	0,15
1 150	0,00247	0,00448	1,81	0,17
1 200	0,00268	0,00488	1,89	0,18
1 250	0,00291	0,00529	1,96	0,20
1 300	0,00313	0,00572	2,04	0,21
1 350	0,00337	0,00617	2,12	0,23
1 400	0,00362	0,00663	2,20	0,25
1 450	0,00387	0,00711	2,28	0,26
1 500	0,00414	0,00761	2,36	0,28
1 550	0,00441	0,00813	2,44	0,30
1 600	0,00469	0,00866	2,52	0,32
1 650	0,00498	0,00921	2,59	0,34
1 700	0,00527	0,00977	2,67	0,36
1 750	0,00558	0,01036	2,75	0,39
1 800	0,00589	0,01095	2,83	0,41
1 850	0,00622	0,01157	2,91	0,43
1 900	0,00655	0,01220	2,99	0,45
1 950	0,00689	0,01285	3,07	0,48
2 000	0,00724	0,01352	3,14	0,50

D= 1000 mm				
k=	0,0001	0,002		
q (l/s)	j(m/m)	j(m/m)	V m/s	$V^2/2g$
350	0,00015	0,00024	0,45	0,01
400	0,00020	0,00032	0,51	0,01
450	0,00025	0,00040	0,57	0,02
500	0,00030	[illegible]	0,64	0,02
550	[illegible]	[illegible]	0,70	0,02
600	0,00042	[illegible]	0,76	0,03
650	0,00049	0,00083	0,83	0,03
700	0,00056	0,00096	0,89	0,04
750	0,00064	0,00110	0,95	0,05
800	0,00073	0,00125	1,02	0,05
850	0,00082	0,00141	1,08	0,06
900	0,00091	0,00158	1,15	0,07
950	0,00101	0,00176	1,21	0,07
1 000	0,00111	0,00195	1,27	0,08
1 050	0,00122	0,00215	1,34	0,09
1 100	0,00134	0,00236	1,40	0,10
1 150	0,00145	0,00258	1,46	0,11
1 200	0,00158	0,00280	1,53	0,12
1 250	0,00171	0,00304	1,59	0,13
1 300	0,00184	0,00329	1,66	0,14
1 350	0,00198	0,00355	1,72	0,15
1 400	0,00212	0,00381	1,78	0,16
1 450	0,00227	0,00409	1,85	0,17
1 500	0,00243	0,00437	1,91	0,19
1 550	0,00258	0,00467	1,97	0,20
1 600	0,00275	0,00498	2,04	0,21
1 650	0,00292	0,00529	2,10	0,22
1 700	0,00309	0,00562	2,16	0,24
1 750	0,00327	0,00595	2,23	0,25
1 800	0,00345	0,00629	2,29	0,27
1 850	0,00364	0,00665	2,36	0,28
1 900	0,00383	0,00701	2,42	0,30
1 950	0,00403	0,00738	2,48	0,31
2 000	0,00423	0,00777	2,55	0,33
2 050	0,00444	0,00816	2,61	0,35

ABAQUE Ab. 3

Ab. 3

RÉSEAUX D'EAUX USÉES EN SYSTÈME SÉPARATIF



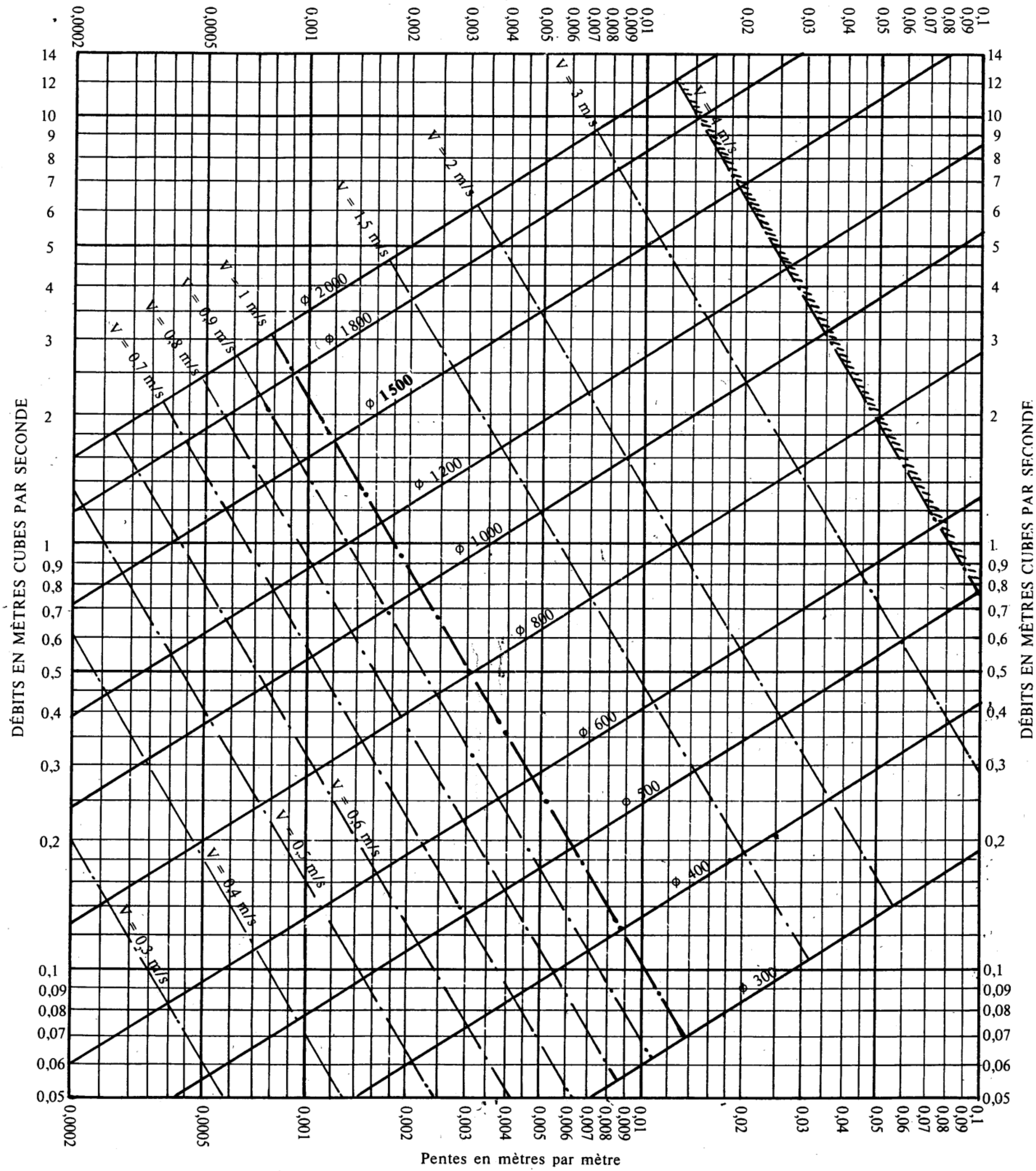
Nota. – La valeur du coefficient de Bazin a été prise égale à 0,25. Lorsque la pose des canalisations aura été particulièrement soignée, et surtout si le réseau est bien entretenu, les débits pourront être majorés de 20 % (γ = 0,16). A débit égal, les pentes pourront être réduites d'un tiers.

ABAQUE Ab. 4 a

Ab. 4a

RÉSEAUX PLUVIAUX EN SYSTÈME UNITAIRE OU SÉPARATIF

(Canalisations circulaires)

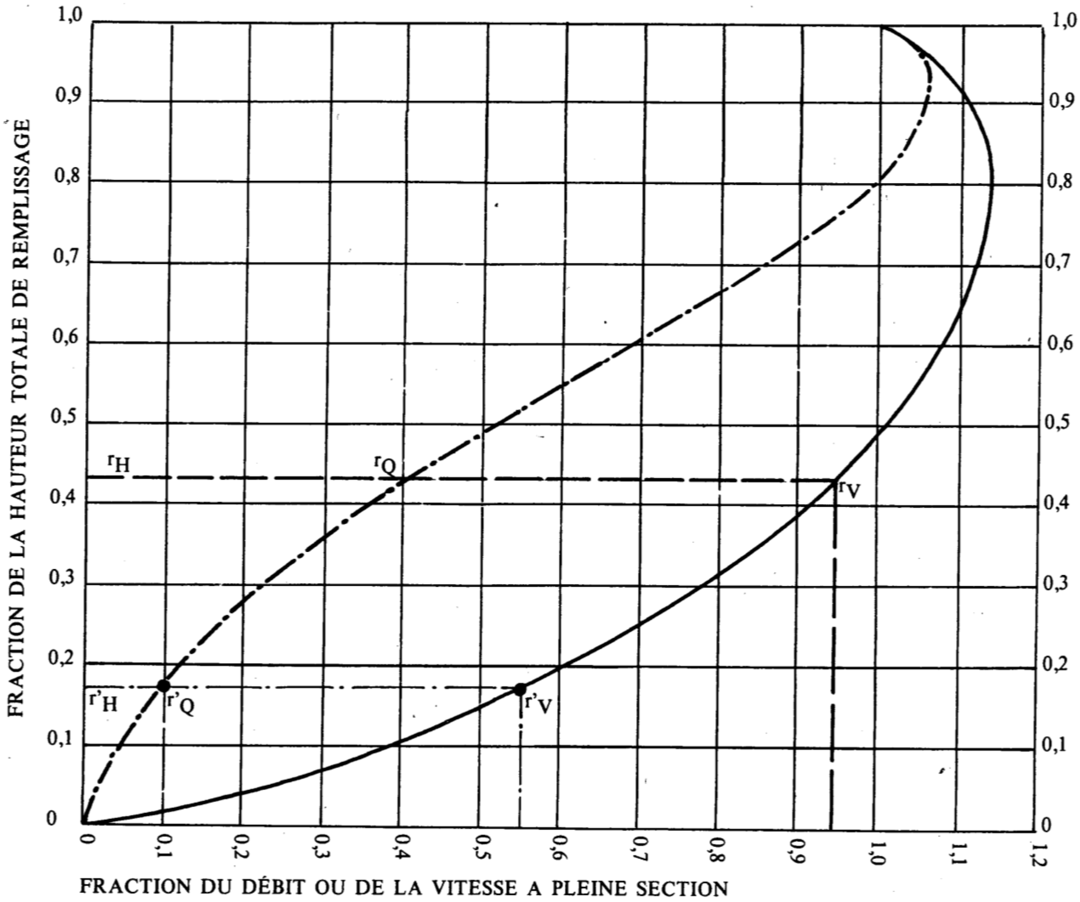


Nota. – La valeur du coefficient de Bazin a été prise égale à 0,46. Lorsque la pose des canalisations aura été particulièrement soignée, et surtout si le réseau est bien entretenu, les débits pourront être majorés de 20 % (γ = 0,30). A débit égal, les pentes pourront être réduites d'un tiers.

ABAQUE Ab. 5 — Ab. 5 *(a)*

VARIATIONS DES DÉBITS ET DES VITESSES EN FONCTION DU REMPLISSAGE

a) Ouvrages circulaires



MODE D'EMPLOI.

Les abaques Ab. 3 et Ab. 4 (*a* et *b*) utilisés pour le choix des sections d'ouvrages, compte tenu de la pente et du débit, permettent d'évaluer la vitesse d'écoulement à pleine section.

Pour l'évaluation des caractéristiques capacitaires des conduites, ou pour apprécier les possibilités d'autocurage, le nomogramme ci-dessus permet de connaître la vitesse atteinte en régime uniforme pour un débit inférieur à celui déterminé à pleine section.

Les correspondances s'établissent, soit en fonction de la fraction du débit à pleine section, soit en fonction de la hauteur de remplissage de l'ouvrage.

Exemples :

Pour r_Q = 0,40, on obtient r_V = 0,95 et r_H = 0,43.

Pour $Q_{PS}/10$, on obtient **r'_V = 0,55 et r'_H = 0,17 (autocurage).**

Nota. — Pour un débit égal au débit à pleine section, la valeur du rapport r_Q = 1,00 est obtenue avec r_H = 0,80.

Le débit maximum (r_Q = 1,07) est obtenu avec r_H = 0,95.

La vitesse maximum (r_V = 1,14) est obtenue avec r_H = 0,80.

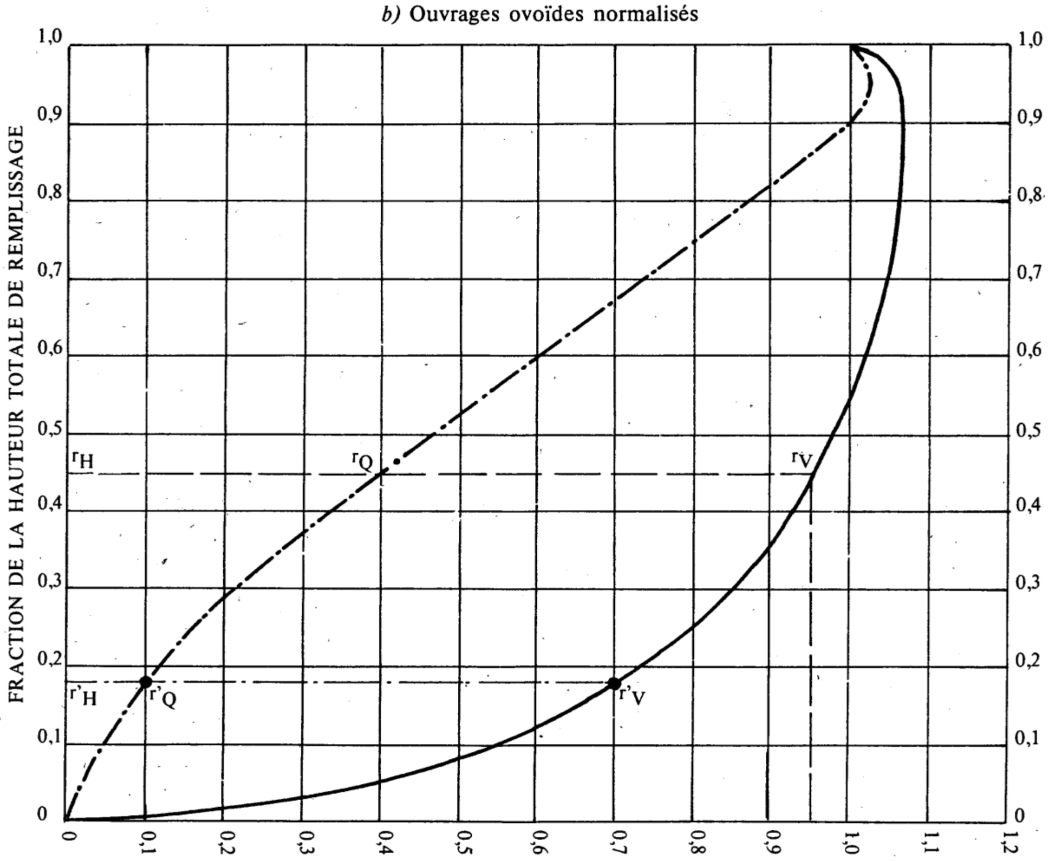
Ces dernières conditions d'écoulement à caractère assez théorique ne peuvent être obtenues que dans des conditions très particulières d'expérimentation.

ABAQUE Ab. 5

Ab. 5 *(b)*

VARIATIONS DES DÉBITS ET DES VITESSES EN FONCTION DU REMPLISSAGE

b) Ouvrages ovoïdes normalisés



FRACTION DU DÉBIT OU DE LA VITESSE A PLEINE SECTION

MODE D'EMPLOI.

Les abaques Ab. 3 et Ab. 4 (*a* et *b*) utilisés pour le choix des sections d'ouvrages, compte tenu de la pente et du débit, permettent d'évaluer la vitesse d'écoulement à pleine section.

Pour l'évaluation des caractéristiques capacitaires des conduites, ou pour apprécier les possibilités d'autocurage, le nomogramme ci-dessus permet de connaître la vitesse atteinte en régime uniforme pour un débit inférieur à celui déterminé à pleine section.

Les correspondances s'établissent, soit en fonction de la fraction du débit à pleine section, soit en fonction de la hauteur de remplissage de l'ouvrage.

Exemples :

Pour r_Q = 0,40, on obtient r_V = 0,95 et r_H = 0,45.

Pour $Q_{PS}/10$, on obtient r'_V = 0,70 et r'_H = 0,18 (autocurage).

Nota. – Pour un débit égal au débit à pleine section, la valeur du rapport r_Q = 1,00 est obtenue avec r_H = 0,90.

Le débit maximum (r_Q = 1,03) est obtenu avec r_H = 0,95.

La vitesse maximum (r_V = 1,07) est obtenue avec r_H = 0,90.

Ces dernières conditions d'écoulement à caractère assez théorique ne peuvent être obtenues que dans des conditions très particulières d'expérimentation.