People's Democratic Republic of Algeria
Ministry of Higher Education and Scientific Research
Mohamed Khider Biskra University
Faculty of Architecture, Urban Planning,
Civil Engineering and Hydraulics
Department of Civil Engineering and Hydraulics

DRINKING WATER SUPPLY NETWORKS

Courses for students specialising in:
Hydraulics - Urban Management – VRD
(Bachelor's and Master's degrees)

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Official program for the Third Hydraulic level (National Formation) and Third Hydraulic Level (ST)

Semester: 5

Teaching unit: UEF 3.2.1

Subject 2: Drinking water supply

VHS: 45h00 (Class: 1h30, Tutorial: 1h30)

Credits: 4 Coefficient: 2

Teaching objectives:

The student will know the principles of sizing and design of drinking water distribution networks.

Recommended prior knowledge: General hydraulics.

Material content:

Chapter I. General information on the drinking water supply of a city

Chapter 2. Study of needs in water

Chapter 3. Tanks

Chapter 4. Distribution networks

Chapter 5. Water supply

Foreword

This course, entitled: **Drinking water supply networks**, complies with the official syllabus of the Algerian Ministry of Higher Education and Scientific Research, and is intended for students in the third year of the LMD, hydraulics option, and those in the national hydraulics course in the Science and Techniques field at Algerian universities and engineering schools.

The expected aim of this module is to provide the necessary bases for the design and calculation of the various elements that make up a drinking water supply system.

The handout covers five main chapters in accordance with the official syllabus:

A first chapter devoted to **general information on distribution networks**.

The second deals with the study of **water requirements in an urban area**.

A third chapter on storage facilities (**Reservoirs or Tanks**).

The fourth chapter covers **distribution networks (branched, meshed)**, design and calculation, the type and nature of pipes used, and the most common special parts used to connect the various pipes.

Water conveyance from source to reservoir is dealt with in the last section in the fifth chapter, which includes the design of a **water hammer arrester**.

Application examples are given in each chapter to give a better understanding of how to use the formulae and the methods for sizing the various components of a distribution system.

I hope that the content of this pedagogical work will be of useful and valuable help to our students.

I would like to express my sincere gratitude to **Professor OUNOKI Samira** for her efforts in reviewing this work, especially the English language.

For any corrections or suggestions, please contact me on my email: a.bedjaoui@univ-biskra.dz

Chapter 1.
An overview of the drinking water supply

1.1. Introduction

The supply of drinking water to an agglomeration involves the management and transport of water from sources to consumers. This process involves several steps and infrastructure, including water catchments, distribution networks, and water towers. Here are some general points about the drinking water supply of an urban agglomeration:

Water catchments: Water catchments are structures that allow water to be taken from natural environments, either by a water intake (pumping in rivers) or by drilling in groundwater. Catchments intended for the production of drinking water are restricted to protect water sources from contamination.

Distribution networks: Drinking water distribution networks consist of underground pipes that connect storage points and users. In France, the drinking water distribution network is estimated at 875,000 kilometers of pipes and in Algeria 100268 km.

Water towers **and tanks: Water towers** and tanks maintain pressure in the distribution network and ensure sufficient flow at the tap. Water tanks store several hours of consumption, making it possible to adapt to peaks in drinking water use.

Water quality: The quality of drinking water is subject to strict limits and benchmarks to ensure public health. Raw water intended for drinking water is treated to remove impurities and harmful microorganisms.

Crisis management: Drinking water networks are also in charge of managing crises related to water supply, such as disasters or technical problems.

Water distribution and sharing: In some cities, drinking water is supplied by several sources, such as the public water supply network, springs (boreholes or catchments) and groundwater reserves.

In general, the drinking water supply of any agglomeration consists of the following elements (Figure 1.1):

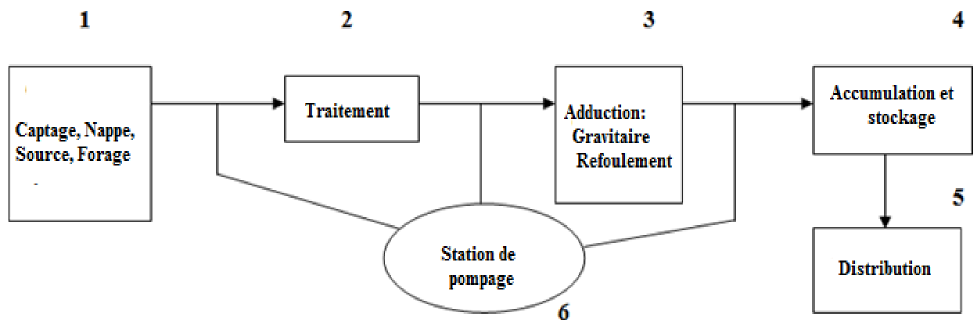


Figure 1.1: Components of a drinking water supply system for a city.

Water Collection

Water collection involves collecting either groundwater (spring, groundwater table, and aquifer) or surface water (rivers, lakes) by means of a water intake and a supply pipe that transports the water to a reservoir that must continuously supply the treatment plant.

The water source to be exploited may be river water (generally fresh water with a salinity of less than 1 g/l), a dam (generally fresh water), groundwater (fresh water or brackish water with a salinity of between 2 and 7 g/l) or seawater (very salty water with a concentration that can reach 35 g/l).

Underground water (groundwater), often freshwater, does not require treatment.

Surface water (rivers or dams), on the other hand, requires physico-chemical treatment to make it drinkable.

Brackish water (salty groundwater or seawater) requires specific treatment (desalination) to reduce the salinity to less than 1 g/l.

Water treatment

Sometimes the collected water does not have the required qualities, so it must be treated. This treatment may also take place after its transportation.

Supply

This is the process of transporting the water from the catchment area to the area where it will be used (distribution). It may take place before treatment.

There are two types of supply:

Gravity-fed supply, where the flow of water at high pressures is caused by the difference in hydraulic levels: the altitude of the source is higher than the altitude of the point of consumption, and therefore moves thanks to the force of gravity, hence its name. This is the principle of the water tower;

Backflow water supply, where pressure is applied to the network and the water is transported using pumps inside pumping stations.

Distribution

Distribution consists of supplying users at all times with the flow of water they need at a suitable pressure. It is carried out by a network of pressure pipes sized to allow the maximum foreseeable flow to pass through each point.

On the previous circuit, the use of pumps is sometimes necessary to solve the problem of unevenness.

1.2 Water distribution

The distribution of water on land can be seen from different points of view:

A quantitative and qualitative distribution of water on a global scale, and in relation to the different components of the hydrological cycle.

A spatial distribution of the water balance on the continents and on the scale of a geographical area. Figure 1.2 shows the distribution of water a round the globe.

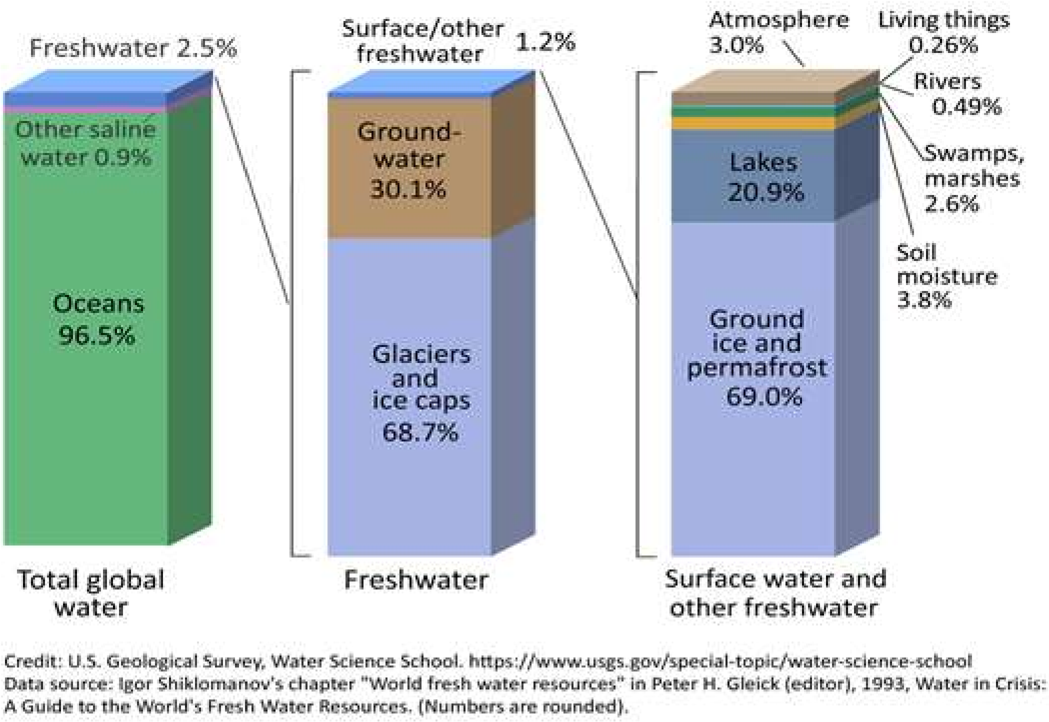


Figure 1.2 Distribution of water around the globe.

- Earth's water is overwhelmingly saline; approximately **97.5% is saltwater** in oceans, leaving only about **2.5% as freshwater**. Of this precious freshwater, the vast majority, roughly **68.7%,** is locked up in ice caps and glaciers. Most of the remaining freshwater is found underground as groundwater (about 30%), with only a tiny fraction (around 1.2%)

available as surface water in rivers and lakes, which is the most accessible form for human and ecological needs.

Breakdown of Earth's Water

Saltwater (Oceans): Accounts for approximately 97.5% of all water on Earth.

Freshwater: Only about 2.5% of Earth's total water is freshwater.

Distribution of Freshwater

Ice and Glaciers: Around 68.7% of all freshwater is frozen in ice caps and glaciers.

Groundwater: A significant portion, about 30%, is stored underground.

Surface Water: A very small amount (just over 1.2%) of freshwater is found in surface sources.

Lakes, Rivers, and Swamps: These sources contain only a tiny fraction of the total freshwater supply but are vital for life.

- On a continental scale, the main elements of water distribution are given by:

- The percentage of precipitation that runs off is greater in the Northern Hemisphere (~40%) than in the Southern Hemisphere (Australia: ~35%, Africa: ~20% and South America: ~10%).

Table 1.1 Distribution of runoff on a continental scale.

Continents	Precipitation	Evaporation	Runoff
Europe	790	507	283
Africa	740	587	153
Asia	740	416	324
North America	76	418	339
South America	1600	910	685
Australia and Oceania	791	511	380
Antarctica	165	0	165
Averages for all continents	800	485	315

1.3. Types of drinking water distribution networks

There are two types of drinking water network:

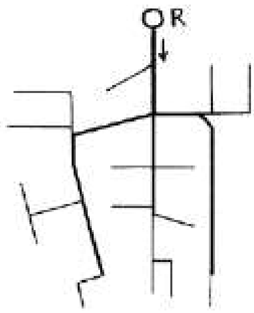
1.3.1. Dead-end or tree network

This is the oldest system, with water flowing in the same direction, i.e. no backflow from the pipes. It is an economic system, but it has a major drawback in terms of safety and flexibility when a stoppage occurs at one point; which requires isolating all or part of a network located downstream. This type of network is easy to be sized and designed (Figure 1.3).

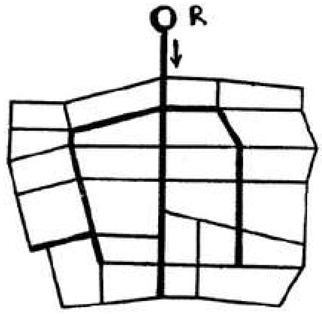
1.3.2. Ring network

The ring network is derived from the tree network by connecting the ends of the pipes (generally up to the level of the tertiary pipes), allowing a return supply. In this way, each point in the network can be supplied with water from two or more sides.

This type of network offers the following advantages: greater security of supply (if one pipe breaks, it simply needs to be isolated and all downstream customers will be supplied by the other pipes) and more even distribution of pressures and flows throughout the network. It is, however, more **expensive and more difficult to calculate.**



Tree network



Ring network

Figure 1.3: Types of drinking water distribution networks

1.3. Conclusion

Supplying a city with drinking water involves managing and transporting water from sources to consumers, through treatment and distribution stages in drinking water distribution networks. This management is subject to strict standards to ensure public health, and is often involved in managing water supply crises.

Drinking water is fresh water that is safe for human consumption. It can be used for domestic and industrial purposes. Drinking water is subject to strict quality standards, and must meet potability criteria defined by the Ministry of Health. These criteria cover physico-chemical parameters, undesirable substances, toxic substances and microbiological parameters.

Drinking water can be distributed in various forms, such as bottled water (mineral water or spring water, still water or sparkling water), tap water or water in tanks for industrial use. In all cases, drinking water must be treated and purified to remove impurities and guarantee its quality.

In most developed countries, the water supplied to households, commerce and industry meets drinking water standards. However, it is important to note that access to clean, high-quality drinking water is a fundamental right for all, and 13% of the world's population, or 884 million people, are still deprived of it.

There are two types of distribution network (branched or meshed, and sometimes combined).

They can be gravity-fed or pressure-fed, in which case a technical and economic study is required to determine the economic diameter.

Chapter 2.
Water demands

2.1 Introduction

The dimensions of structures and installations, the number and power of pumps, the capacity of tanks, the height and capacity of water towers and the diameters of pipes are determined according to the quantity of water to be supplied and the regime of their operation.

The water consumption regime is a major factor determining the operating regime of the water distribution system components. It depends for agglomerations on the number of inhabitants, the type and comfort of the buildings; it is also a function of the variation of the seasons of the year, the days of the week, etc.

The water demand is determined according to:

1- Population and the city size

2- Costums;

3- The seasons;

4- Water sources availability;

5- Lifestyle;

2.2. Population growth rate

Generally, population growth rate is determined for various time period (short, medium and long) depending on the project significance

Short time period: 1 to 10 years

Medium time period: 10 to 20 years

Long time period: 20 to 30 years

There are several formula for short and long time period prediction, including the following:

- Graphic
- Comparison diagram ;
- Geometric progression ;
- Decreasing rate of growth;

- Logistic equation.

It should be noted that all these formula are for guidance only; they must be used with other additional data that may help to determine the population behavior.

2.2.1 Data sources

To estimate the population growth, numerous data are required, such as:

- Censuses.
- Immigration and immigration data at the national level .
- Birth and death records, birth rates and death rates.
- Urban planning maps, which provide data on the current and projected land use and the population density.

2.2.2 Arithmetic progression

This type of progression is affected for old populations where the ratio between population growth and time growth is constant of population increase to time increase is constant.

$$\frac{dP}{dt} = \frac{(P_2 - P_1)}{(t_2 - t_1)} \tag{2.1}$$

$$P_n = P_2 + K_a(t_n - t_2) \tag{2.2}$$

With:

P_1 : Population at time t_1
P_n : Population at time t_n
P_2 : Population at time t_2
K_a : Arithmetic Growth Constant

This formula applies in the case of old and stable populations and in towns with an agricultural character.

2.2.3 Geometric progression

The rate of increase is proportional to the population.

$$\frac{dP}{dt} = K_g . P \tag{2.3}$$

$$K_g = \frac{Ln(P2)-Ln(P1)}{t2-t1} \quad (2.4)$$

$$P_n = P_2\, e^{Kg\,(tn - t2)} \quad (2.5)$$

K_g : Geometric Growth Constant

It applies to young and growing populations. The geometric growth can be expressed using the compound interest equation if the annual growth percentage of the population is available. Then:

$$P_n = P_1(1 + r)^n \quad (2.6)$$

Where:

n: Number of periods (usually years) during i n which there is geometric growth ($t_{n\text{-}t\ 1}$)

r : Growth rate;

P 1 : Population at time t_1

This formula is usually used in Algeria.

2.2.4 Decreasing rate of growth

The rate of growth is proportional to the difference between the population and the saturation population.

$$\frac{dP}{dt} = K(P_s - P) \quad (2.7)$$

$$K = \frac{-Ln\left(\frac{P_S-P2}{P_S-P1}\right)}{t2-t1} \quad (2.8)$$

$$P_n = P_2 + (P_S - P_2)[1 - e^{-k(t_n - t_2)}] \quad (2.9)$$

With :

S : Saturation population, which must be estimated approximately based on population trends and the availability of land in the concerned area.

This formula applies mainly to populations that no longer have room to grow.

Figure 2.1 shows the general shape of the temporal evolution of population.

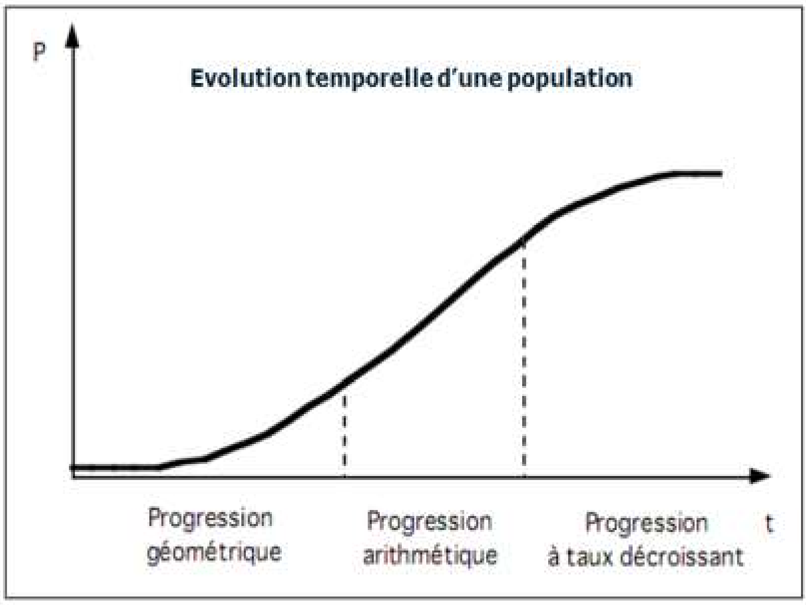


Figure 2.1. Temporal evolution of population

An examination of the world's population over the past 1000 years clearly shows that it it has been growing exponentially. (Figure 2.2).

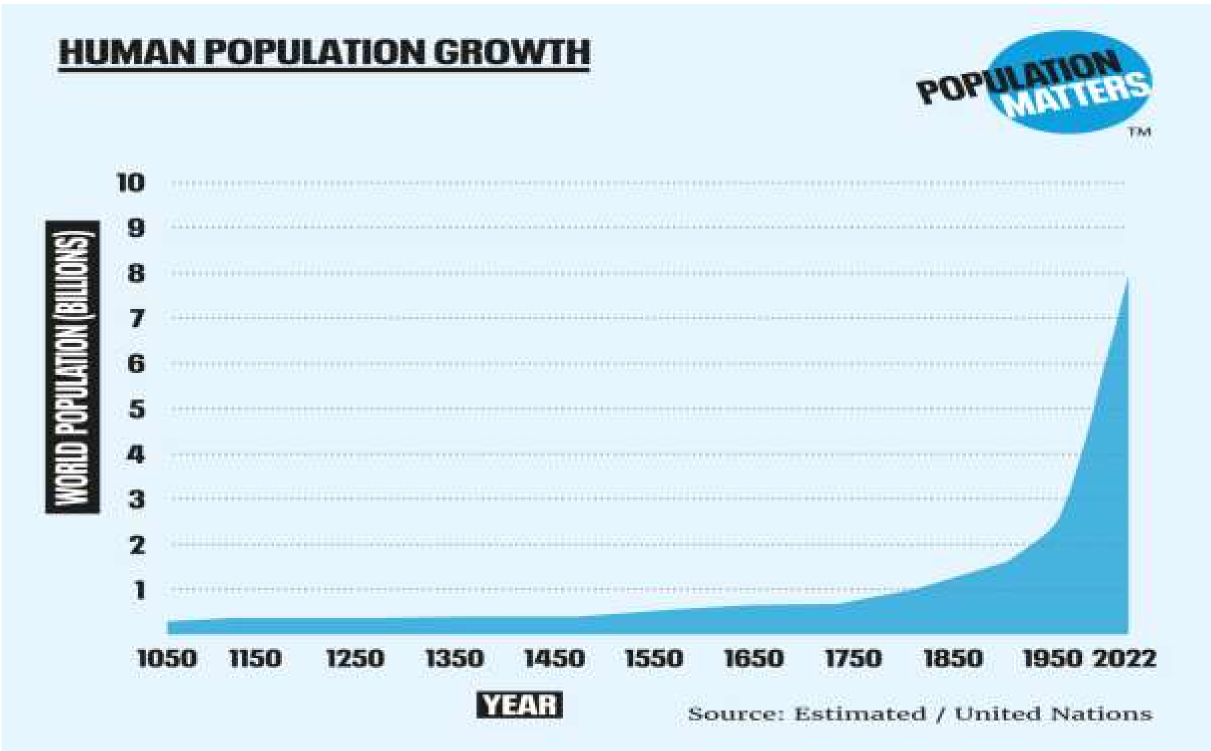


Figure 2.2 world population trends

The world population will reach **9.731 billion in 2050,** compared to 7.141 billion in 2013, according to a biennial study by the French Institute for Demographic Studies (INED). There will be 10 to 11 billion people on the planet by the end of the century, according to projections

by INED, which conducts its own studies in parallel with those carried out by the United Nations, the World Bank and other major national institutes.
Africa will account for about a quarter of the world's population in 2050, with 2.435 billion people on the continent, more than double the 1.1 billion recorded in 2013.
Continental Europe, with a fertility rate of 1.6 children per woman, will be the only region to experience a decline in population, from 740 million inhabitants in 2013 to 726 million in 2050.
America will pass the billion mark, rising from 958 million inhabitants in 2013 to 1.228 billion in 2050.
Asia will jump from 4.305 billion inhabitants in 2013 to 5.284 billion in 2050, and Oceania will grow from 38 to 58 million.

Currently, the 'G7' of the most populous countries consists of China (1.36 billion), ahead of India (1.276 billion), the United States (316.2 million), Indonesia (248.5 million), Brazil (195.5 million), Pakistan (190.7 million) and Nigeria (174.9 million).
By 2050, this ranking is expected to change significantly, with India

By 2050, this ranking is expected to change significantly, with India in the lead (1.65 billion) ahead of a less populous China (1.314 billion) and Nigeria, which, with 444 million inhabitants, will overtake the United States (400 million). Figure 2.3 shows population growth as function of global material reserves.

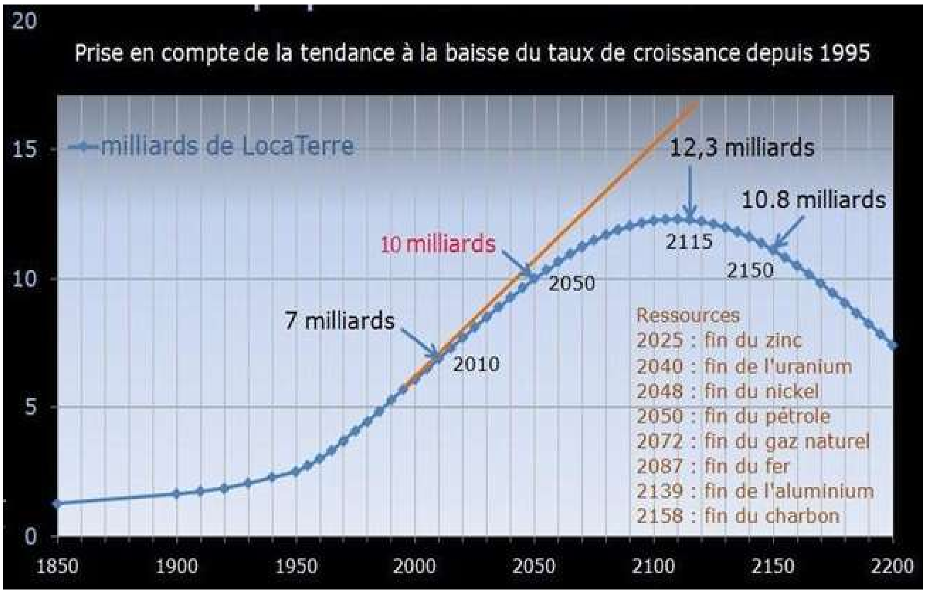


Figure 2.3. Population growth in relation to global reserves of raw materials

2.3 Water demand estimation

Although some regulations exist in certain countries to determine drinking water demand, rigorous quantification of this demand is generally based on statistics.

2.3.1 Types of water requirements

- **Household demands**

Water used for household consumption includes personal hygiene, laundry and lawn watering, often adding to this consumption, water supplied to small businesses.

Average household consumption is generally related to the number of inhabitants, and is then expressed in liters per person per day (l/p/d)).

This consumption varies according to several factors: lifestyle, habits, water availability, season, water price, etc., see Table 2.1.

Table 2.1: Consumption per capita for various population sizes

Population size	Consumption per capita (l/p/d))
more than 100000 inhabitants	120 to 200
For a city of 20,000 to 100,000 inhabitants:	100 to 140
Medium-sized city (5,000 to 20,000 inhabitants)	80 to 120
For a rural area (less than 5,000 inhabitants)	60 to 80
For fire hydrants :	20 to 50

One example is the World Health Organization (WHO) standard. Which sets the minimum household consumption at 55 l/p/d.

- **Public demands**

Public needs include the consumption of offices, educational institutions, municipalities, hospitals, etc.
Table 2.2 displays water public demands.

Table 2.2: Equipment Requirements

Nature of the equipment	Needs
Hospitals :	300 to 600 l/bed/d.
Tourism Needs (Hotels)	400 to 700 l/bed/d (and up to 1200 l/day/bed for luxury hotels).
For Administrations :	100 to 200 l/day/employee.
For Primary Schools :	10 to 20 l/day/student.
For High Schools	20 to 30 l/day/student
For street cleaning and garden watering :	3 to 5 l/day/m^2.

For Faculties and Hostels :	100 to 200 l/day/student.
Mosque :	50 l/d/faithful
Bath-shower :	100 l/d/station
Slaughterhouse :	500 l/day/ head

Table2.3 shows the breakdown of the specific consumption of 100 litres of water at home (1977) :

Table 2.3: Household consumption types

Needs	Quantity (l)
Toilet flushing	25
Baths & Hygiene	30
Laudry	15
Dishwashing	10
Drinking & Cooking	5
Watering and washing the car, etc.	15
Total	100

➢ **Industrial needs**

In general, only the needs of small industries, which consume drinking water and are connected to the city's network, are taken into account.

Currently, large industries are isolated from the city (or located in industrial areas) and fed by independent grids. Those that consume a lot of water must have their own source of water: wells, boreholes, dams, the sea, etc.

It should be noted that industrial consumption depends on the product manufactured and, above all, on the manufacturing process used. Below are some examples of industrial needs.

- **For small industries:**
  - Bakery : 1 l / Kg of bread.
  - Dairy industry : from 5 to 10 liters/ l of milk.
  - fruit or vegetable preserves: from 6 to 15 l/kg of canned goods.

- **For Large Industries :**
  - Sugar refinery from 2 to 15^{m3} /t of beetroot ;
  - Cement works (wet) 2 m^3/t of cement ;

- Tannery: from 20 to 140 ^{m3}/ t of product ;
- Paper mill : from 50 to 300 ^{m3}/t of product ;
- Oil refinery from 1 to 20 ^{m3}/t of oil ;
- Steelworks: from 6 to 300 ^{m3}/t of steel ;
- Power station from 3 to 400 ^{m3}/ MWh.

It should be noted that industrial consumption depends on the product manufactured and especially on the manufacturing process used.

Since it is difficult to accurately estimate all public and industrial needs (small industries), they can be accommodated by slightly increasing domestic needs .

➢ **Water requirements for firefighting**

Every municipality must provide water to fight fires. The amount of water required in this case is relatively small, since this amount is used for very short periods of time, the discharges are high.

Flow rates and volumes of water required are determined according to the suggested standards.

Water tanks and distribution systems are sized to ensure that consumers receive sufficient water at an acceptable pressure:

During the peak consumption day (Qmaxj), when there are one or more fires. During the peak consumption hour (Qh) max, a fire volume of 120 m³ is recommended, which is an adequate volume for extinguishing an average fire lasting two hours.

$$V_{inc} = q_{inc} .. t \qquad (2 .10)$$

V_{inc}: Fire volume (120 m^3)

t : Extinguishing time of an average fire (t=2h)

q_{inc}; Fire flow rate (17 l/s)

Table 2.4 summarizes the main recommended consumption per capita in Algeria.

Table 2.4: Recommended consumption per capita in Algeria

(d) Endowment	Equipment	Nature of the need
100 l/student/d	Primary schools, middle school without boarding	School
120 l/student/d	kindergarten, Schools, middle schools, High Schools, Universities (boarding schools)	
10 l/m^2/d	Cultural Centers, Cinemas, Youth Centers, Stadiums	cultural
5-10 l/m^2/d	Administrative offices (town halls, poste office, Police, Civil Protection, court, etc.)	Administrative
10 l/m^2/d	Clinical, Pharmacy, Polyclinics	Sanitary
(300-400) l/d/bed 500 l/d/bed	Hospitals Maternity	
125 -200 l/d/bed	Hotels	commercial
5 l/d/m^2	Market Clean-up	
150 -200 l/d/post	Hammams	
100 l/d/Comp	Campsites	
50 l/d/station	Showers	
20 l/d/ Swimmer	ج Swimming Pools	
1200 l/d/ Car	Car washes	
500 l/d/head	Slaughterhouses	
1200 l/d/U	Bakeries	
1500 l/d/U	Café shop	
12 l/d/meal	Restaurants	
50 l/d/Fid	Mosques	Mosques

2.4 Study of consumed flow rate variation

Water demand is subject to annual, seasonal, monthly, daily and even hourly variations, hence it is of great interest to study this variation for proper sizing the water supply network and its management, (see Figure 2.4)

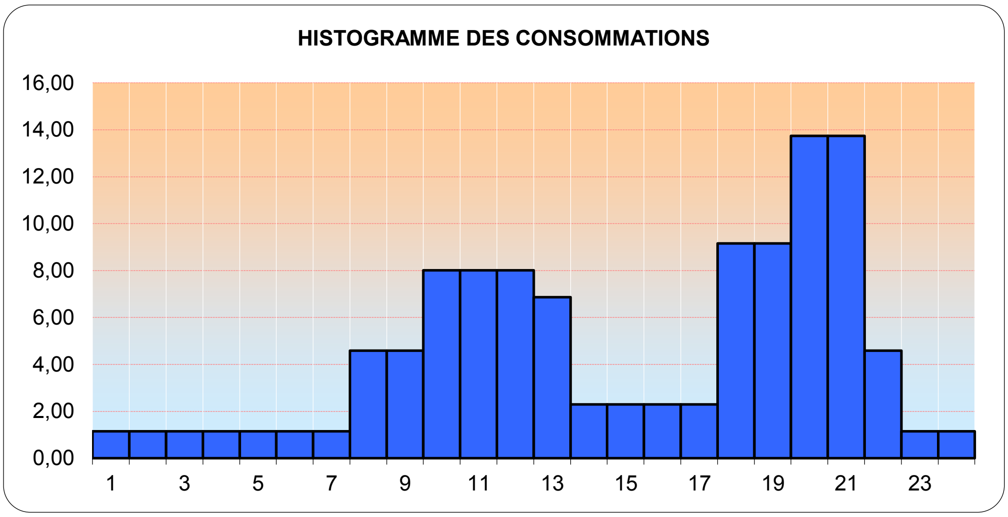


Figure 2.4: The variation in hourly consumption.

2.4.1 Total Average consumption:

The total daily average consumption ($Q_{avg)}$ for an unrban area is given by the following formula:

$$Qmoyj = P_f . d \;\; ; \;\; \text{(l/S)} \tag{2.11}$$

$$Qmoyj = Pf . \frac{d}{10000} \;\; ; \;\; (\text{m}^3/\text{J}) \tag{2.12}$$

With P_f and d are respectively the number of inhabitants at a given time period and the average consumption per capita per day (l/p/d).

2.4.2 Daily peak consumption

Water consumption varies depending on the month (consumption peaks in July and August), the day of the week, it is usually maximum on holidays and weekends (Friday, Eid etc.) and the time of day (it is usually maximum in the morning and around midday). This is the consumption on the day of the year when consumption is highest.

Water intake, treatment and supply facilities (pumping stations, pipes, etc.) must be sized to meet the maximum daily demand (peak day) for the project year. The daily peak coefficient K_j is defined as:

$$K_J = \frac{\text{Consommation journalière max}}{\text{Consommation journalière moy}} = \frac{QmaxJ}{QmoyJ} \tag{2.13}$$

The value of the coefficient K_j is, in principle, determined from the statistics on the daily variation of consumption over the 365 days of the year. Generally, this value of K_j varies from 1.1 to 1.3, depending on the climate and summer activities in the urban area (for example, for a tourist area, K_j is close to 1.1).

2.4.3 Hourly peak consumption

Water distribution structures (network, tanks) must be sized to meet the maximum hourly demand (peak hour) of the day. An hourly peak coefficient Kh or Kp is defined as the greatest quantity of water required by consumers at peak hour, or the greatest demand at any hour of the day.

$$K_h = Kp = \frac{\text{Consommation horaire maximale}}{\text{Consommation horraire moyenne}} = \frac{Qmaxh}{Qmoyh} \tag{2.14}$$

Where:

K_h: irregularity coefficient of hourly consumption $K h = \propto . \beta$

$$Kh=\alpha.\beta \tag{2.15}$$

∝: A coefficient that takes into account the comfort of buildings and ranges from 1.2 to 1.4.
β: a coefficient that takes into account the number of inhabitants is given by Table 2.5

Table 2.5: Values of β Coefficient

P	100.000	50.000	6000	4000	2500	1500	1000	500
β	1,1	1,15	1,4	1,5	1,6	1,8	2,0	2,5

The correlation between the population and β can be written as:

$$\beta = 5.2359^{\,P-0.142}$$
$$R^2 = 0.92$$

❖ Minimum hourly consumption for the year $Q_{Min\ h}$:

The lowest hourly consumption of the year varies between 40 and 80% of the average hourly consumption of the year% .

$$Q_{min.h} = K_{min.h} \cdot Q_{moy.h} \tag{2.16}$$

With:

$Q_{min.h} = K_{min.h} \cdot Q_{moy.h}$

➢ **Water leakage**

In a drinking water supply network, water leakage occurs at various stages: the water intake, the treatment plant, pumping stations, reservoirs, supply and distribution networks, valves, joints, meters, etc.

The volume of water leakage depends on:

- The age and condition of the network;
- The competence and efficiency of the network maintenance service (speed of leak detection, efficiency of execution of the work, human resources, adequate facilities, organization, etc.).

In general, the K-value for leakage ranges from 1.2 to 1.5 :

- o K = 1.2; for a new or well-maintained network .
- o K = 1.25 to 1.35 ; for a moderately maintained network .
- o K = 1.5; for an outdated or poorly maintained network .

- **The calculation flow rate of the various structures in the network :**

The calculation flow rate depends on the type and location of the structure to be sized.

- **The annual volume of** water (V_{tot}) to be expected at the water source (or volume captured):

$$V_{tot} = K . 365 * Q_{moy.J} \quad \text{en m}^3\text{/an} \tag{2.17}$$

The design and calculation flow rate of water supply structures (pumping station, treatment plant, reservoirs, conveyance pipes, etc.) is equal **to the** maximum daily flow rate $Q_{max.J}$

$$Q_{max.j} = K * K_j * Q_{moy.j} \ ; \text{en l/s} \tag{2.18}$$

Q_{maxj} is expressed as a function of the leakage rate K, which varies from (20 to 50)%

$$Q_{maxJ} = K_j Q_{moyjmaj} \tag{2.19}$$

The design and calculation flow rate of the distribution structures (pumping station, elevation of reservoirs, distribution network) **is equal to the maximum hourly flow rate**

$Q_{max.h} = Qp$ **:**

$$Qp = Kp.Q_{moyj.maj} \ ; (l/s) \tag{2.20}$$

Kp: Peak coefficient expressed by:

$$K_p = K_h . Kj \ \text{ avec : } K_h = \alpha . \beta \tag{2.21}$$

$$Kp = 1{,}5 + \frac{2.5}{\sqrt{Qmoymaj}} \tag{2.22}$$

$$Kp = 2.6 - 0.4 \, log \frac{P}{1000} \tag{2.23}$$

$$Kp = 1.8 + \frac{13.7}{\sqrt{n}} + \frac{34.5}{n} \tag{2.24}$$

In which:

Qavg,j,maj : Average daily flow increased

P: supplied population;

n: Number of houses;

Kh: Hourly coefficient of variation;

K*j* : Coefficient of daily variation.

2.5 Application Example 1

Estimate the water needs for 20 53 of an urban area of 10000 inhabitants (2022) with a population growth rate of 1.6%, knowing that the PDAU provides for the following facilities:

N°	**Equipment designations**	**Area/Number of Occupants**	**Consumption per capita**
01	Two primary schools	400 students	100 l/J
02	Middle schools	300 students	100 l/J
03	Bakery	03	1200 l/J/U
04	Mosque	500 Faithful	50 l/J/Fid
05	APC Headquarters	200 m²	10 l/J/m²
06	Gendarmerie headquarters	200 m²	10 l/J/m²
07	Green space	1000 m²	6 l/J/m²
08	Car washing	50 cars	1500 l/J/Car

This urban area will be equipped with a forge with a 30 l/s discharge.

Study the variation in flows (Average Daily, Average Per Day, Max Per Day, and Peak Flow).

Check the pumping time by suggesting a suitable conveyance system.

Solution

1- Study of water demands

(a) household needs

- Estimation of the future population

$$P_{2053}=P_a(1+\tau)^n=10000.\ (1+0,016)^{30} = 16099 \text{ inhabitants}$$

- Evaluation of average daily consumption

$$C_{avgd} = P_{2053.d/1000} = 2414.85\ m^3/J$$

(b) Study of equipment needs

N°	Equipment designations	Area/Number of Occupants	Staffing l/d/....	Requirements (^{m3}/D)
1	Two primary schools	400 students	100 l/J	40,0

2	Middle schools	300 students	100 l/J	30,0
3	Bakery	3	1200 l/J/U	3,6
4	Mosque	500 Faithful	50 l/J/Fid	25,0
5	APC Headquarters	200 m²	10 l/J/m²	2,0
6	AMG Seat	100 m²	5 l/J/m²	1,0
7	Green space	1000 m²	6 l/J/m²	6,0
8	Car washing	50 cars	1500 l/J/Car	75,0
			Total equipment requirements	182,6

Total equipment requirements are estimated at 534.6^{m3}/D

The total average needs of the agglomeration are:

C avg. = C avg.d.population +$C_{avg.d.equi}$ = 2514.85+182.6 = 2597.45 m^3/J

- Evaluation of the increased average daily consumption

$C_{avg.maj=}C_{avg.j.}$ +$\alpha_{C\ avg.}$ = 2,958.45 +1.3. 2,597.45 = 3376.69 m^3/J

2 - Study of the variation of flow rates

- Average daily flow rate: Q avg. = $C_{avg.}$ (l/s) = 30l/S
- Increased average daily flow rate: Q avg. maj = $C_{avg.\ maj}$ (l/s) = 39 l/S
- Max daily flow rate: Q maxj = C_{maxj} (l/s) = 47 l/s
- Peak Flow Rate: Qp=Kp.Qavg.j.maj

With:

K_{P1}= 2.6-0.4log(P2053/1000) = 2.12

K_{P2}= 1.5+2.5/Avg.d.maj 0.5 = 1.9

K_{p3} = Kh.Kj = a.b.Kj = 1,2.1,32.1,2= 2.06

$K_{P\ avg}$ = 2.03 and Qp = 79.2 l/S

Since Q_f=30 l/s < Qmax.j= 47 l/s, there is a water deficit of 17 l/s that needs to be filled.

2.6. Application Example 2

The graph of water consumption for a population is shown in Figure 2.6 It is requested to:

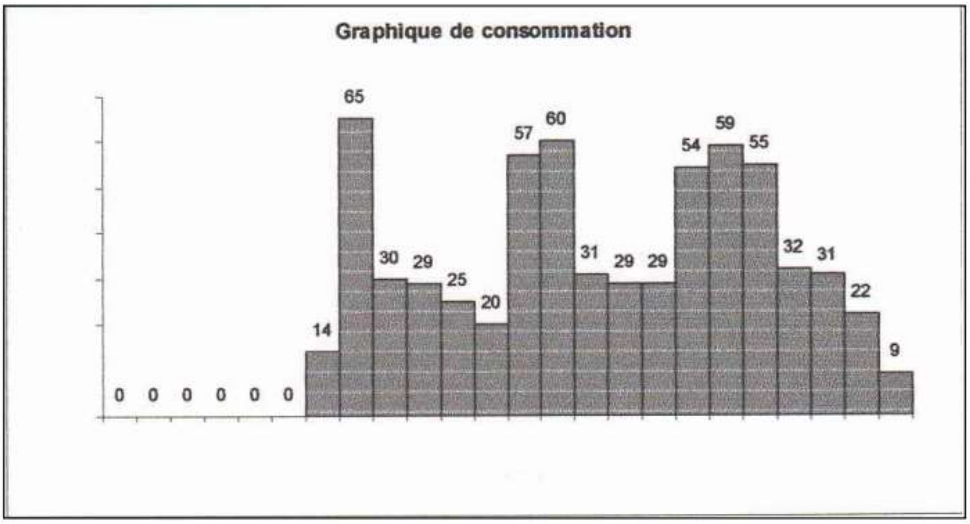


Figure 2.6: Consumption Graph

- Determine the amount of water consumed in 24 hours.
- Calculate the maximum hourly flow rate.
- Calculate the average hourly throughput.
- Calculate the average daily flow rate
- If the maximum instantaneous flow rate is equal to 0.02 l/s, determine the peak coefficient.

Solution

The amount of water consumed during the day is 651 l/d.

Maximum hourly flow rate Qmaxh = 65 l/s

The average hourly flow rate Qavgh = 651/24 = 27.12 l/s

The average daily flow Qavg$j$ = 651 l/d

The instantaneous point coefficient Kp = Q p/Qavgh = 0.02.3600/27.12 = 2.65

2.7. Conclusion

1- The calculation of the maximum daily flow has the following objectives:

- Choosing the conveyance system (Diameter of the discharge line, pumping time)
  - Verification of available resources and needs to be met
  - Tank sizing

2- The peak flow rate is used for the sizing of the distribution network

- The maximum daily flow rate represents the highest demand required in any given day of the year.
- Peak flow is the largest amount of water required by consumers at peak time or the greatest demand at any time of the day

Chapter 3.
Storage facilities: Tanks

3.1. Introduction

A reservoir is used to store water and distribute it to consumers; it connects the supply to the distribution network ensuring a pressure ranging from (1 to 4) bars. Tanks are necessary to adequately supply an urban area with drinking water. They are mainly imposed by the difference between the water discharge flow (which is generally constant) and the water flow consumed in an urban area (which varies depending on the time of day).

3.2. Definition of tanks

A tank is a facility containing a liquid, this liquid is generally water, or drinkable. Among liquids other than water, most often hydrocarbons.
The tanks can be built uncovered or, on the contrary, fitted with a dome or a flat slab. Reservoirs can be simple or complex and made up of several cells. The plan shape can be any. However, most of the time small tanks are square or rectangular, but the circular shape is less expensive.

3.3. Role of the Reservoir

The various functions of tanks can be grouped under five main headings:

- Meet the water needs of the city.
- Allow more uniform operation of the pumps.
- Ensure the maximum flow rates requested during peak hours.
- Regulate pressures in the distribution network.
- Keeping water protected from the risk of contamination and protects it against strong temperature variations.
- An element of safety with regard to the risks of fire, exceptional water demand or temporary break in the supply (breakdown in the pumping station, break in the supply pipe, shutdown of the treatment station ,….)

3.4. Classification of reservoirs

In relation to the supply network, they can be grouped into two types:

- Passage tanks (placed between the catchment and the water distribution network)
- Balance tanks (placed at the end of the distribution network)

According to the nature of the materials, we distinguish:

- Masonry reservoirs.
- Reinforced, ordinary or prestressed concrete tanks.

The tanks are differentiated according to their position relative to the ground, they can be:

- Buried .
- Semi buried .
- Raised on towers (water towers).

3.5. Tank location

The water tank must be located as close as possible to the town to be supplied (on the edge of the town). In fact, given the peak coefficient, the average hourly consumption rate must be used to deduce the maximum hourly consumption (from 1.5 to 3.5), and the pressure loss will generally be greater on the distribution pipe than on the adduction pipe. This means that the further the reservoir is from the town, the higher the level of the body of water must be (hence greater pumping energy).

Gravity distribution

Distribution is gravity-fed when the tank is located at a sufficiently high level in relation to the urban area.

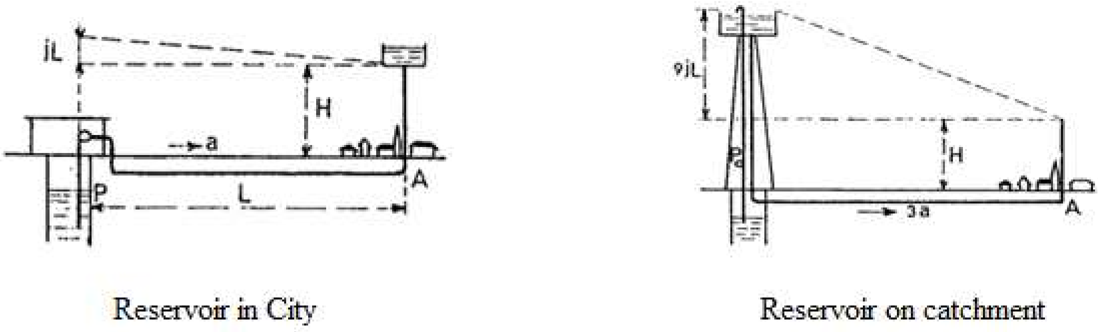


Figure 3.1: Gravity distribution.

Staged distribution

When the city has significant differences in elevation, a staged distribution system can be adopted (see example below). If the urban area extends in a given direction, a single reservoir may become insufficient and provide too low pressure at the end of the network during peak hours. One or more balancing reservoirs can then be added at the other end of the city to ensure acceptable pressure in their area of operation. These balancing reservoirs are connected to the main reservoir and are filled during periods of low consumption (mainly at night).

Example

Figure 2 illustrates a stepped supply between elevations (30.00 m) and (90.00 m). Reservoir R1 is designed to supply the area between elevations (30.00) and (50.00) m. R1 will be located at elevation 70.00 m, and the pressure at ground level will therefore vary from 70 – 30 = 40 m water column to 70 – 50 = 20 m water column. In addition, reservoir R2, located at elevation 90, will supply the area between 50 m and 70 m. These reservoirs can be supplied either by a common station or by different sources. They can also be connected to each other to provide backup if necessary.

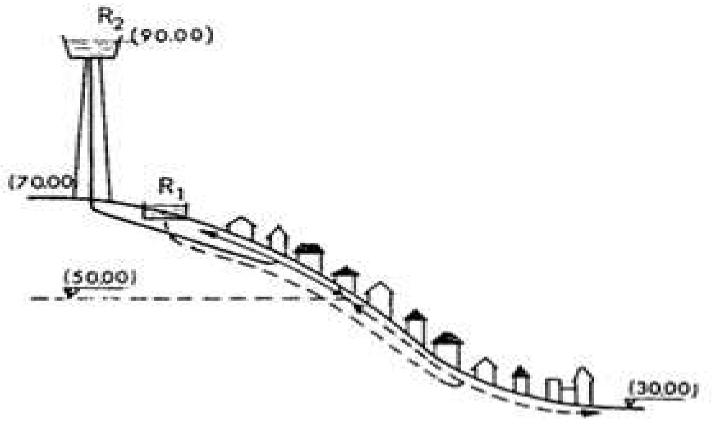


Figure 3.2: Staged distribution

Additional balance tank

If an urban area expands in a given direction, a single reservoir may become insufficient and only provide insufficient pressure at the end of the network during peak hours. In this case, one or more balancing reservoirs are used in conjunction with the main reservoir. These reservoirs fill up during periods of low consumption, mainly at night and, to a lesser extent, during the day.

The connecting pipe, due to its diameter and hydraulic slope, determines the flow rate that will feed the second reservoir (II). This flow rate must be sufficient for (II) to be full at the start of the morning (Figure 3.3). The pressure at point B must be at least equal to the minimum pressure required between the two reservoirs.

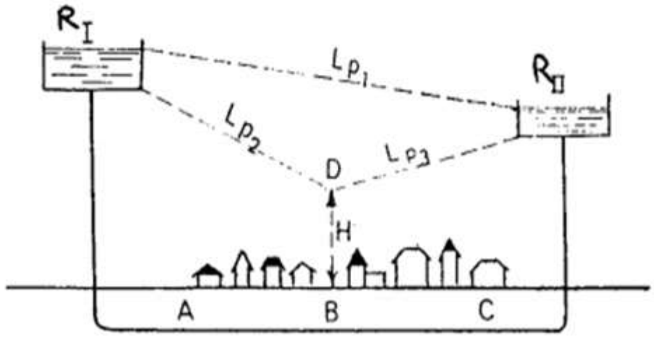


Figure 3.3: Balance tank

In some cases, both types of reservoirs can be used at the same time: (semi-buried and elevated reservoir or water tower). The semi-buried reservoir is supplied by the treatment plant, with or without pumping, at a constant flow rate Qmh. The water tower, located upstream of the distribution system, is supplied by another pumping station SP2 that operates at variable flow rates (see Figure 3.4). Adopting this type of system makes it possible to limit the volume required for the tower reservoir.

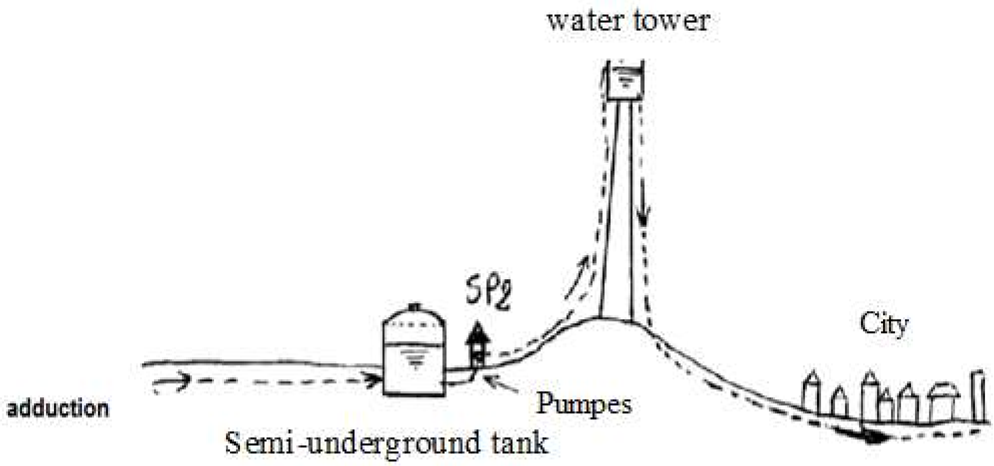


Figure 3.4: Surge tanks.

The location of a reservoir can be summarised as follows: finding a compromise between two factors:

- **Being as close as possible to consumers**

Minimization of the length of main distribution pipes (Technical and economic advantage)

- **Being located at a dominant point**

Reduction of elevation (height)

3.6. Capacity of a tank

The capacity of the tank is determined by taking into account variations in the inflow and outflow of the tank. The volume of the tank is obtained by adding the absolute value of the highest positive value and the highest negative value of the distribution adjustment, plus the volume of the fire reserve and the safety volume, using the following formula:

$V_R = V_u + V_s + V_{inc}$(3.1)

With :

V_R : the volume of the tank m^3

V_u : the useful volume m^3

V_S : the safety volume m^3

V_{inc} : the fire volume m^3

a) Determination of useful volume

The useful volume (or regularized volume) is expressed by the following formula:

$$V_u = |\Delta V_{max}^{+}| + |\Delta V_{max}^{-}| \quad (3.2)$$

With :

$|\Delta V^{+}_{max}|$: Surplus or quantity of water stored m^3

$|\Delta V^{-}_{max}|$: Water deficit m^3

Each of these volumes represents the difference between the cumulative volumes of input and output.

These volumes are calculated respectively by the relations:

$V_{app} = C_{maxd}/24$ (m^3/h)

$V_{dis} = Ch\ \%.\ C_{maxd}$ (m^3/h)

Ch : Hourly coefficients expressing the percentages of distributed water consumption according to the size of the urban areas (see Table 3.1).

Table 3.1 : **Values of hourly coefficients Ch.**

Hours	Less than 10,000	10,000<N<50,000	50,000<N<100,000	N>100,000
0 – 1	1	1.5	3.25	3
1 – 2	1	1.5	3.25	3.1
2 – 3	1	1.5	3.3	3.1
3 – 4	1	1.5	3.2	2.6
4 – 5	2	2.5	3.25	3.5
5 – 6	3	3.5	3.4	4.5
6 – 7	5	4.5	3.85	4.5
7 – 8	6.5	5.5	4.45	4.1
8 – 9	6.5	6.25	5.2	4.9
9 – 10	5.5	6.25	5.05	5.6
10 – 11	4.5	6.25	4.85	4.8
11 – 12	5.5	6.25	4.6	4.7

12 – 13	7	5	4.6	4.4
13 – 14	7	5	4.55	4.1
14 – 15	5.5	5.5	4.75	4.2
15 – 16	4.5	6	4.7	4.5
16 – 17	5	6	4.65	4.4
17 – 18	6.5	5.5	4.35	4.1
18 – 19	6.5	5	4.4	4.5
19 – 20	5	4.5	4.3	4.5
20 – 21	4.5	4	4.3	4.3
21 – 22	3	3	4.2	4.8
22 – 23	2	2	3.75	4.5
23 – 24	1	1.5	3.7	3.3

b) Fire volume calculation

All reservoirs must have a fire reserve, which must be available at all times. The minimum reserve to be provided is 120 m^3 for each tank (the fire-fighting motor pump used by firefighters is 60 m3/h and the approximate duration of extinguishing an average fire is estimated at 2 hours). To calculate the fire volume, the following formula is used:

$$V_{inc} = q_{inc}\, t \tag{3.3}$$

With:

q_{inc} : the fire flow rate l/s.

t : Duration of extinction of an average fire (t = 2 hours).

The fire flow rate is determined based on the size of the city, its architectural texture and existing facilities. Table 3.2 shows fire flow rates based on population size.

Table 3.2: Fire flow values as function of population size (Table without source).

Number of inhabitant	Number of fires at the same time	q_{inc} l/s	
		Building with one floor	Building of 3 floors or more
Less than 5000	1	5	5
5000 – 10,000	1	10	10
10,000 – 25,000	2	10	10
25,000 – 50,000	2	10	15
Greater than 50,000	2	17	17

For urban areas at high risk of fire, the capacity required to deal with a fire could exceed 120 m³. For large cities, the volume required to deal with a fire is generally negligible compared to the total volume of the reservoirs.

c) Safety volume

Water supply interruptions due to system failures such as pipe breaks, power cuts, preventive or corrective maintenance of facilities are poorly tolerated by users who have long enjoyed regular service. Adding an additional volume to the volume normally renewed by the distribution system, known as a safety reserve, makes it possible to limit interruptions by ensuring continuity of service for a certain period of time. The actual volume depends on tolerance, the level of comfort required by users, and the measures taken to prevent deterioration of water quality in the reservoir. Volumes vary from six hours of average consumption to one day's consumption.

This volume is calculated as follow:

$$V_s = (10 - 15)\% \, (V_u + V_{inc}) \tag{3.4}$$

Generally it is recommended to take 12 %, then:

$$V_s = 12\% \, (V_u + V_{inc}) \tag{3.5}$$

Noticed:

According to **A. Dupont,** the volume of the reservoir in the case of continuous pumping is 30 percent of the maximum daily flow, which means:

$$V_r = 30\% \, Q_{maxj} \tag{3.6}$$

The volumes of the most used tanks are: 50; 100; 120; 150; 200; 250; 300; 350; 400; 500; 600; 800; 1000; 1200; 1500; 2000; 2500; 3000; 4000; 5000; 10000 m^3.

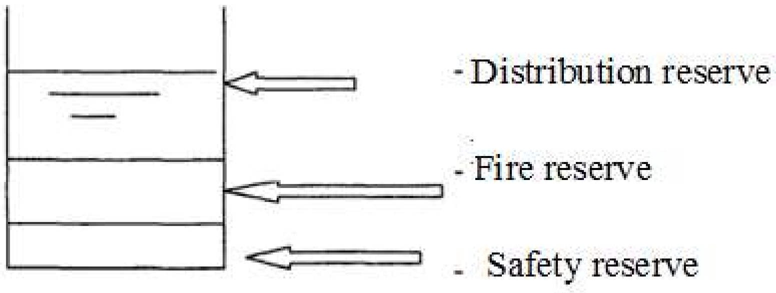


Figure 3.5: Distribution of water reserves in a reservoir

Noticed :

The number of hours of water supply and the times of day during which it is provided have a decisive impact on the size of the distribution reserve. Three methods are used to estimate its volume.

3.7. Shapes and types of Tanks

The shape of the tanks is generally circular, and is rarely square or rectangular. Regarding the water tower, the shape of the tank is also circular; its exterior appearance must adapt to the landscape and requires architecture appropriate to the site so as not to destroy the environment.

The height of water (He) in the reservoirs is between 3 and 6 m, and exceptionally reaches 10 m for large cities. The diameter of the circular tank, imposed by the volume, varies from 1.5 to 2 times the height of the HC tank. Table 3 groups together values for the heights and diameters of the tanks**.**

For economic reasons, tanks are constructed of reinforced concrete up to a volume of 2500 m^3 and of prestressed concrete up to 20000 m^3. For small volumes, and rarely, they can be metallic. Semi-buried tanks are the most used, with a generally vaulted roof, and a 20 to 30 cm covering with earth or sand (thermal insulation of the water).

Semi-buried circular tank

Raised tank Water tower

Figure 3.6: Shapes and types of Tanks

To determine the diameter of a tank, we use the following formula:

$$V_R = S\,H_e = \pi\,\frac{D^2}{4}\,H_e \Rightarrow D = \sqrt{\frac{4\,V_R}{\pi\,H_e}} \quad (3.7)$$

Example:

For a tank volume of 200 m ^{3} , if the water height He= 3 m. The diameter of the tank will be:

$$D = \sqrt{\frac{4.200}{3{,}14.3}} = 9{,}21 \approx 9{,}50\ m$$

3.8 Tank location

One of the main roles of the reservoir is to provide sufficient pressure at ground level (H min) during peak hours at all points in the distribution network (see pressure values below), particularly at the most unfavourable point in the network (the furthest and/or highest point). The altitude of the water tank (specifically the elevation of its base) must therefore be calculated so that, throughout the entire urban area to be supplied, the pressure is at least equal to H min= 10 m.c. water. It is the elevation of the reservoir's base that is taken into account, which corresponds to the most unfavourable supply condition (the reservoir is then almost empty).

It is the calculation of the distribution network during peak hours that makes it possible to determine the various pressure losses and deduce the elevation of the reservoir's base (determination of the critical path).

The value of this rating and the topography of the site determine the type of reservoir to be used (semi-buried or elevated). If the terrain allows, the diameter of the distribution pipes can be increased to reduce pressure losses and avoid having to elevate the reservoir (solution to be justified by an economic calculation).

The location of a tank takes into account the terrain in order to minimise investment and operating costs. Therefore, to determine the elevation of the tank foundation, the following must be taken into account:

• The highest point to be supplied;

• The height of the structures;

• Pressure losses from the tank to the most unfavourable point in the city. Depending on the most critical path.

To determine the ratin of the tank base Cr, we use the following formula:

$$C_r = CTN + H + h_s + P_s + \Delta H_t \tag{3.8}$$

With :

CTN: Natural terrain rating of the most unfavourable point;

H: Height of tallest building (function of number of the floors).

Table 3.3: Values of H (Height of tallest building)

Type of building	Height H(m)	
DRC	3	For each floor we take 3 m
R+1	6	
R+2	9	
R+3	12	

h_s : Singular pressure loss in buildings (2 – 4 m) we generally take h_s = 4 m.

P_s : Additional water column taking into account the equipment using water (3–5 m) we take P_s = 4m

ΔH_t : Linear and singular pressure loss along the most critical path.

The total pressure loss is given by the following formula:

$\Delta H_t = \Delta H_L + \Delta H_s$(3.9)

ΔH_L: linear pressure loss given by (according to ***Darcy):***

$$\Delta H_L = j\, L = \frac{\lambda\, v^2}{2\, g\, D}\, L \tag{3.10}$$

With :

J: Hydraulic pressure loss gradient m/m.

v: Flow velocity m/s.

D: Pipe diameter.

L: Pipe length m.

λ : Friction coefficient expressed by several relationships in turbulent regime Re > 2300:

Colebrook-White : $\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{\varepsilon/D}{3,7} + \frac{2,51}{R\sqrt{\lambda}}\right)$ (3.11)

Nekuradse : $\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{\varepsilon/D}{3,7}\right)$ (3.12)

Achour- Bedjaoui :

$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{\varepsilon/D}{3,7} + \frac{10,04}{\overline{R}}\right)$, With: $\overline{R} \cong 2R\left[-\log\left(\frac{\varepsilon}{3.7D} + \frac{5,5}{R^{0.9}}\right)\right]^{-2}$ (3.13)

Swamee : $\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{5,5}{R^{0.9}}\right)$ (3.14)

Achour et al: $\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3,7D} + \frac{4,5}{Re}Log\frac{Re}{6,97}\right)$ (3.15)

With:

D: Pipe diameter (m);

ε: Roughness coefficient ε = 0.02 mm for HDPE pipes;

R e : *Reynolds* number , given by: $R_e = \frac{v\,D}{\vartheta}$

v: average flow velocity (m/s)

$\boldsymbol{\Delta H_s}$**:** Singular pressure loss expressing the presence of singularities (Tes, Elbows, Sudden widening, narrowing ect ...), in a network where it is practically difficult to count the number and nature of the singularities we propose:

$\Delta H_s = (10 - 15)\%\ \Delta H_L$(3.15)

3.9. Application example

Determine the total pressure loss along a circular HDPE pipe (ε =0.02 mm) 500 m long, diameter D= 1m, conveying at peak hour a flow rate of 20 ls , knowing that the kinematic viscosity of water at 20°C is 10^{-6} m/s ^{2}, singular losses are neglected.

Solution.

The total pressure loss is the sum of the linear and singular pressure loss and is expressed by:

$$\Delta H_t = \Delta H_L + \Delta H_s$$

$$\Delta H_t = \Delta H_L\ bcz\ \Delta H_s = 0$$

ΔH_L : linear pressure loss is given by the following formula (according to **Darcy**):

$$\Delta H_L = j\,L = \frac{\lambda\, v^2}{2\,g\,D}\,L$$

λ : Friction coefficient expressed by the Swamee relation

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{5{,}5}{R^{0.9}}\right) \Rightarrow \lambda = \left[-2\log\left(\frac{\varepsilon}{3.7D} + \frac{5{,}5}{R^{0.9}}\right)\right]^{-2}$$

v: Average flow velocity (m/s) is :$v = \frac{Q}{S} = \frac{4Q}{\pi D^2} = 0{,}64\ \text{m/s}$

R_e : *Reynolds* number , given by:$R_e = \frac{v\,D}{\vartheta} = 1{,}27.10^5$

Which give : $\lambda = \left[-2\log\left(\frac{\varepsilon}{3.7D} + \frac{5{,}5}{R^{0.9}}\right)\right]^{-2}$ = 0,0177 , J = 0.00183 and Δ Ht = 1.83 m

3.10. Constructive provisions

The usable storage volume is obtained by adding the distribution reserve, the safety reserve and the fire reserve. The total capacity of the tank takes into account the headroom between the overflow and the cover to house the control equipment, and the dead volume between the suction strainer and the bottom of the tank, which receives the settled sludge.

The height of the tank is a compromise between the requirements of civil engineering stability and low pressure variation in the networks, and regulation, which works better with a relatively high water level. The optimum height varies between 3 and 6 meters.

Ventilation openings for air renewal shall be protected by fine stainless steel mesh to prevent corrosion by chlorine and its derivatives. Daylight shall be avoided, as it causes algae to grow on the walls of the tank and in the water. The tank cover must have an external slope of 1 to 2% to allow rainwater to run off and limit direct sunlight, which causes the water temperature to rise. The bottom of the tank, which is shaped like a trough, must have a slope of at least 2% to concentrate the sludge and facilitate its removal.

3.11. Tank equipment

A tank must have the following equipment to facilitate its operation.
- A system for stopping its supply: float valve, hydraulically controlled valve or electrically operated valve;

- A distribution feed strainer. The strainer must allow the fire reserve to be renewed without being able to use it during simple distribution;

- A distribution meter, easily accessible to measure the volumes of water distributed.

- A tap for water quality analysis will be placed on the distribution pipe.

- A fire reserve withdrawal pipe whose opening device is at the permanent disposal of the firefighters,

- An overflow pipe.

- A drain pipe fitted with a valve, the operating system of which is protected, is only accessible by agents of the distribution company;

- A by-pass between the supply pipe and the distribution pipe to ensure continuity of service during maintenance of the water tower.

- A system for measuring the volume of water contained in the tank.

Control equipment

The water level in the reservoir, the meter reading and the status of the supply flow shut-off system can be transmitted to the facility control office.

Transmission will be carried out:

- Manually

- By hydraulic transmission

- By radio transmission

- Electrically

- Via the telephone network (analogue, digital).

3.11.1. Supply line

The supply pipe, at its outlet into the reservoir, must be able to be shut off when the water reaches its maximum level in the tank: obstruction by a float valve if the supply is gravity-fed, or a device that stops the motor if the supply is pressure-fed. The supply (see chapter on Supply) is carried out by overflow, either by free fall or by extending the pipe so that its end is always submerged.

The water can also be supplied by passing through the foundation slab. The free fall causes the water to become oxygenated, which can be beneficial for groundwater, which is generally low in dissolved oxygen. Its diameter is the subject of a technical and economic study (see Chapter 5).

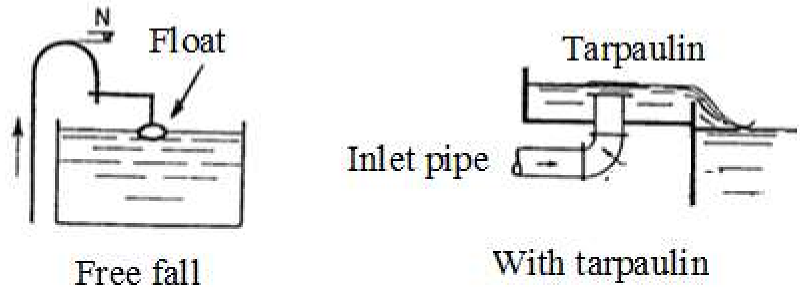


Figure 3.7: Water inlet

3.11.2. Distribution line

The distribution pipe starts 0.15 or 0.20 m above the tank floor to prevent sludge or sand from entering the distribution system, which could potentially settle in the tank.

To facilitate water flow, the outlet should be located opposite the inlet. A tap should be fitted at the outlet of the pipe.

The size of the pipe is determined using the following formula:

$$Q_p = v\,s \tag{3.16}$$

With:

Q_p : Peak flow m^3/s

S: Pipe section

v: Flow velocity, which should vary between 0.5 and 1.5 m/s,

Generally, the diameter is calculated as follows:

An average velocity is assumed (v = 1m/s)

According to the continuity equation: $D = \sqrt{\frac{4\,Q_p}{\pi\,v}}$

A standard diameter is selected with which the velocity is verified and which will be used to calculate the hydraulic gradient and linear head loss. $v = \frac{4\,Q_p}{\pi\,D^2}$

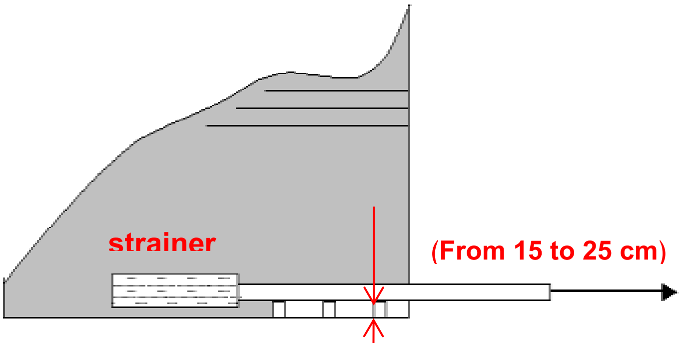


Figure 3.8: Departure from distribution.

3.11.3 Over flow pipe

Most tanks are equipped with an overflow pipe designed to drain excess water when the tank is full. This pipe connects to the bottom drain pipe via a T-joint and is formed by a reduction cone. It is designed to prevent the tank from overflowing. This pipe must be able to drain the entire flow Q arriving at the tank, maintaining the water at the maximum level in the tank. It shall not have any valves along its length. The cross-section will initially feature a truncated cone-shaped flare allowing the flow to pass under a blade height h.

The flow discharged under these conditions is calculated using Lancastre formula:

$$Q = 27.828\, \mu\, r\, h^{3/2} \tag{3.17}$$

With:

h : Water height above overflow.

r : Radius of the opening of the truncated cone of the pipe.

μ : Flow coefficient given as a function of h/r.

Table 3.4: values of μ

h/r	0.2	0.25	0.30	0.40	0.50
μ	0.415	0.414	0.410	0.404	0.394

If we take μ = 0.415 we have h/r = 0.2 therefore h = 0.2 r

$$r = \left[\frac{Q}{27.828\, \mu\, (0.2)^{\frac{3}{2}}}\right]^{2/5} \tag{3.18}$$

The overflow pipe will discharge into a nearby outlet, but there is a risk that this outlet could cause pollution or allow animals or mosquitoes to enter the reservoir. A hydraulic seal consisting of a siphon must be provided to keep the AB section of the overflow pipe filled with water.

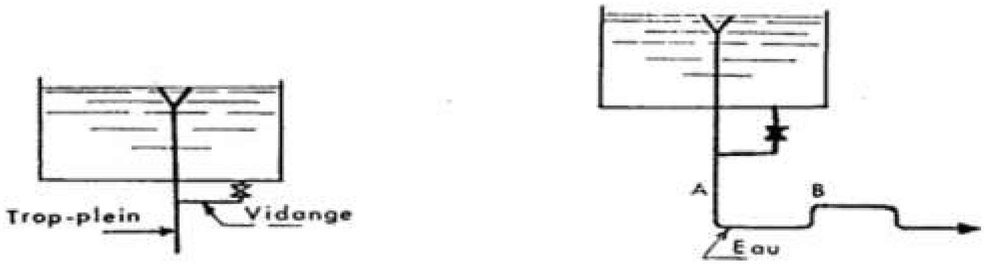


Figure 3.9: Overflow and draining

3.11.4 Drain line

It starts from the lowest point of the tank and connects to the overflow pipe, it has a valve. Draining is essential for tank maintenance (cleaning, repair, etc.)
The diameter of this pipe is determined using the following formula:

$$Q = \mu\, s \sqrt{2\, g\, h} = \mu \left[\frac{\pi\, D^2}{4}\right] \sqrt{2\, g\, h} \rightarrow D = \sqrt{\frac{4\, Q}{\mu\, \pi \sqrt{2\, g\, h}}} \quad (3.19)$$

With :

h : Water height in the tank m.

Q: Inflow m^3/s

μ : Flow coefficient (μ = 0.4)

3.11.5. Application example

Determine the diameter of the drain pipe of tank if its volume is 700 m^3, the height of water in the tank is h = 6 m, and it needs to be drained in 2 hours.

Solution:

- Determination of the draining flow rate if we consider a draining time t of 2 hours

$$Q = \frac{V}{t} = \frac{700}{2.3600} = 0{,}097\ m3/s$$

- Calculation of pipe diameter:

$$D = \sqrt{\frac{4\,Q}{\mu\,\pi\,\sqrt{2\,g\,h}}} = \sqrt{\frac{4.0,097}{0,4.3,14.\sqrt{2.9,81.6}}} = 0,168 \;\approx 200mm$$

- Checking the flow with the new diameter:

$$Q = \;\mu\left[\frac{\pi\,D^2}{4}\right]\sqrt{2\,g\,h} = 0,4\left[\frac{3,14.\,0,2^2}{4}\right]\sqrt{2.9,81.6} = 0,136\;m3/s$$

And the flow velocity: $v = \frac{4\,Q}{\pi\,D^2} = \frac{4.0,136}{3,14.0,2^2} = 4,33\;m/s$

The diameter of the drain pipe is 200 mm which will be capable of discharging a flow rate of 0.136 m^3/s with a speed of 4.33 m/s.

3.11.6 Special devices

3.11.6.1 Bypass

When distribution needs to be ensured during the emptying or cleaning of a tank, all that is required is a bypass connecting the supply pipe to the distribution pipe.

- When using the tank, valves (1) and (2) are open and (3) is closed.
- When cleaning the tank, valve (3) is closed and valves (1) and (2) are open.

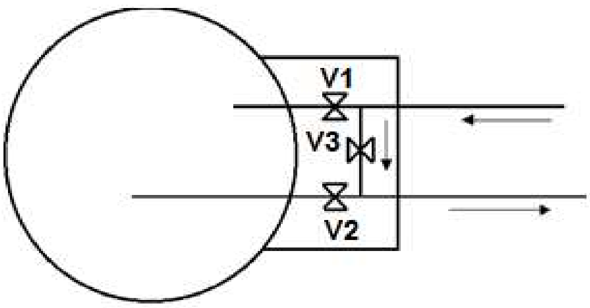


Figure 3.10: by-pass

3.6.11.2. Fire reserve device

The tank always has a certain amount of water, approximately 120 m³, stored in case of fire. A device consisting of a siphon is used, which disengages when the tank level is reached, thanks to the vent open to the open air.

- In normal distribution conditions, (2) is closed and (1) and (3) are opened.
- In the event of a fire, (2) is opened and (1) and (3) are closed.

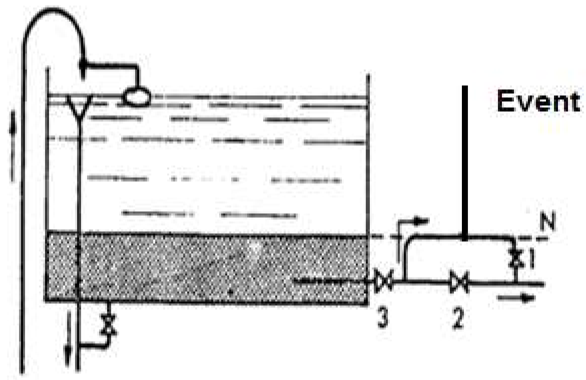


Figure 3.11: Fire reserve device.

3.12. Application Example

What is the volume of a reservoir that supplies a town of 10,000 inhabitants if the pumping flow rate is 21l/s in the following cases:

1. Pumping is continuous 24 hours a day.
2. Pumping is discontinuous during the night 12 hours a day.
3. Pumping is discontinuous during the day 12/24 hours.

Solution.

The volume of the reservoir by definition is: $Vr\ (m^3) = Vu + Vinc + Vs$

With :

Seen: Useful volume (m ^{3}), = Max / Δ V $^+$ / + Max/ Δ V $^-$ /

Vinc : Fire volume (m ^{3}), $V_{inc} (m^3) = qt$

q : Fire flow = 17l/s

t : Extinguishing time for an average fire of 2 hours

Vs: Safety volume = *12% ($V_u + V_{inc}$)* ; (m^3)

The maximum daily consumption is: C_{maxJ} = 21.86400/1000 = 1814.4 m^3/d

The hourly intake volume is: $V_{app} = C_{maxd}/24$ =75.6 (m^3/h)

The hourly volume distributed is: $V_d = Ch.C_{max\ j}$

3.12.1Continuous pumping

a) Determination of useful volume

To determine the useful volume, we proceed to the study of the regularization of water distribution (table 3.5).

Table 3.5: Calculation of useful volume during continuous pumping

Time	Ch %	Partial volume m^3		Cumulative volume m^3		Difference m^3	
		bring	Distributed	Bring	distributed	$\Delta\varsigma$ +	ΔV^-
0 – 1	1	75.6	18.144	75.6	18.144	57,456	
1 – 2	1	75.6	18.144	151.2	36,288	114,912	
2 – 3	1	75.6	18.144	226.8	54,432	172,368	
3 – 4	1	75.6	18.144	302.4	72,576	229,824	
4 – 5	2	75.6	36,288	378	108,864	269.136	
5 – 6	3	75.6	54,432	453.6	163,296	**290.304**	
6 – 7	5	75.6	90.72	529.2	254.016	275.184	
7 – 8	6.5	75.6	117,936	604.8	371,952	232,848	
8 – 9	6.5	75.6	117,936	680.4	489,888	190,512	
9 – 10	5.5	75.6	99,792	756	589.68	166.32	
10 – 11	4.5	75.6	81,648	831.6	671.328	160,272	
11 – 12	5.5	75.6	99,792	907.2	771.12	136.08	
12 – 13	7	75.6	127.008	982.8	898.128	84,672	
13 – 14	7	75.6	127.008	1058.4	1025.14	33,264	
14 – 15	5.5	75.6	99,792	1134	1124.93	9.072	
15 – 16	4.5	75.6	81,648	1209.6	1206.58	3.024	
16 – 17	5	75.6	90.72	1285.2	1297.3		-12.096
17 – 18	6.5	75.6	117,936	1360.8	1415.232		-54.432
18 – 19	6.5	75.6	117,936	1436.4	1533.168		-96.768
19 – 20	5	75.6	90.72	1512	1623.888		-111,888
20 – 21	4.5	75.6	81,648	1587.6	1705.536		**-117,936**
21 – 22	3	75.6	54,432	1663.2	1759.968		-96.768
22 – 23	2	75.6	36,288	1738.8	1796.256		-57.456
23 – 24	1	75.6	18.144	1814.4	1814.4	0	

SO :

$$V_u = |\Delta V_{max}^{+}| + |\Delta V_{max}^{-}| = 290.304 + 117.936 = 408.24\ m^3$$

b) Determination of fire volume

From table 2, the fire flow is 17l/s for t=2h

$$V_{inc} = q_{inc}\, t = 17 * 2 * 3600 = 120\, m^3$$

c) Determining the safety volume

$$V_s = 12\%\, (V_u + V_{inc}) = \frac{12}{100}(408{,}24 + 120\,) \approx 63{,}39\, m^3$$

So the volume of the tank is:

$$V_R = V_u + V_s + V_{inc} = 408{,}24 + 63{,}39 + 120 = 591{,}6 \approx 600\, m^3$$

3.12.2 Discontinuous pumping (nightly), at night 12h/24h

We propose discontinuous pumping from 8 p.m. until 8 a.m., the flow delivered or pumped is:

The maximum daily consumption remains 1814.4 m3/d which amounts to mobilizing an hourly volume 1814.4/12 = 151.2 *m3 / h*

Qapp = 151.2 m* ^{3} */h

The useful volume is calculated in the same way as mentioned above:

Table 3.6: Useful volume in the case of discontinuous pumping at night

Time	ch	Partial volume m ^{3}		Total volume m ^{3}		Difference m ^{3}	
		bring	Distributed	Bring	distributed	Δς +	Δ V $^-$
0 – 1	1	151.2	18,144	756	208,656	547,344	
1 – 2	1	151.2	18,144	907.2	226.8	680.4	
2 – 3	1	151.2	18,144	1058.4	244,944	813,456	
3 – 4	1	151.2	18,144	1209.6	263,088	946,512	
4 – 5	2	151.2	36,288	1360.8	299,376	1061,424	
5 – 6	3	151.2	54,432	1512	353,808	1158,192	
6 – 7	5	151.2	90.72	1663.2	444,528	1218,672	
7 – 8	6.5	151.2	117,936	1814.4	562,464	**1251,936**	
8 – 9	6.5	0	117,936	1814.4	680.4	1134	
9 – 10	5.5	0	99,792	1814.4	780,192	1034,208	
10 – 11	4.5	0	81,648	1814.4	861.84	952.56	
11 – 12	5.5	0	99,792	1814.4	961,632	852,768	
12 – 13	7	0	127,008	1814.4	1088.64	725.76	
13 – 14	7	0	127,008	1814.4	1215,648	598,752	
14 – 15	5.5	0	99,792	1814.4	1315.44	498.96	
15 – 16	4.5	0	81,648	1814.4	1397.088	417,312	
16 – 17	5	0	90.72	1814.4	1487,808	326,592	
17 – 18	6.5	0	117,936	1814.4	1605,744	208,656	
18 – 19	6.5	0	117,936	1814.4	1723.68	90.72	

19 – 20	5	0	90.72	1814.4	1814.4	0	
20 – 21	4.5	151.2	81,648	151.2	81,648	69,552	
21 – 22	3	151.2	54,432	302.4	136.08	166.32	
22 – 23	2	151.2	36,288	453.6	172,368	281,232	
23 – 24	1	151.2	18,144	604.8	190,512	414,288	

SO :

$$V_u = |\Delta\, V_{max}^{+}| + \ |\Delta V_{max}^{-}| = 1251{,}94\ m^3$$

The fire volume still remains120 m^3

The safety volume is:

$$V_s = 12\%\ (V_u +\ V_{inc}) = \frac{12}{100}(1251{,}94\ + 120\ ) = 150{,}2m^3$$

So the volume of the tank is:

$$V_R = V_u +\ V_s +\ V_{inc} = 1251{,}94 + 150{,}2 + 120 = 1522{,}14\ \approx 1500\ m^3$$

3.12.3 Discontinuous pumping during the day 12h/24h

Discontinuous pumping is proposed from 8 a.m. until 8 p.m. with the same inflow rate Qapp = 151.2 m ^{3} /D. Then, the regularized volume will be (See table 3.7):

Table 3.7: Useful volume in the case of discontinuous pumping during the day

Time	Ch	Partial volume m ^{3}		Total volume m ^{3}		Difference m ^{3}	
		bring	Distributed	Bring	distributed	Δς +	Δ V $^-$
0 – 1	1	0	18.144	1814.4	1460.592	353,808	
1 – 2	1	0	18.144	1814.4	1478.736	335,664	
2 – 3	1	0	18.144	1814.4	1496.88	317.52	
3 – 4	1	0	18.144	1814.4	1515.024	299,376	
4 – 5	2	0	36,288	1814.4	1551.312	263,088	
5 – 6	3	0	54,432	1814.4	1605.744	208,656	
6 – 7	5	0	90.72	1814.4	1696.464	117,936	
7 – 8	6.5	0	117,936	1814.4	1814.4	0	
8 – 9	6.5	151.2	117,936	151.2	117,936	33,264	
9 – 10	5.5	151.2	99,792	302.4	217,728	84,672	
10 – 11	4.5	151.2	81,648	453.6	299,376	154.224	
11 – 12	5.5	151.2	99,792	604.8	399,168	205.632	
12 – 13	7	151.2	127.008	756	526.176	229,824	
13 – 14	7	151.2	127.008	907.2	653.184	254.016	
14 – 15	5.5	151.2	99,792	1058.4	752,976	305.424	
15 – 16	4.5	151.2	81,648	1209.6	834.624	374,976	
16 – 17	5	151.2	90.72	1360.8	925.344	435,456	
17 – 18	6.5	151.2	117,936	1512	1043.28	468.72	

18 – 19	6.5	151.2	117,936	1663.2	1161.216	501.984	
19 – 20	5	151.2	90.72	1814.4	1251.936	**562.464**	
20 – 21	4.5	0	81,648	1814.4	1333.584	480,816	
21 – 22	3	0	54,432	1814.4	1388.016	426.384	
22 – 23	2	0	36,288	1814.4	1424.304	390,096	
23 – 24	1	0	18.144	1814.4	1442.448	371,952	

So :

$$V_u = |\Delta\, V_{max}^{+}| + \; |\Delta V_{max}^{-}| = 562{,}64\; m^3$$

The fire volume still remains120 m^3
The safety volume is:

$$V_s = 12\%\; (V_u + \; V_{inc}) = \frac{12}{100}(562.464 + 120\;) \approx 127{,}7\; m^3$$

So the volume of the tank is:

$$V_R = V_u + \; V_s + \; V_{inc} = 562.464 + 127{,}7 + 120 = 810{,}2 \approx 800\; m^3$$

In conclusion, the volume of a tank can be calculated based on the following percentages of the maximum daily consumption and depending on the supply mode.

- In the case of continuous pumping, the tank volume represents 32% of the maximum daily consumption.
- In the case of discontinuous pumping during the day, the reservoir volume represents 44% of the maximum daily consumption.
- In the case of discontinuous pumping during the night, the volume of the reservoir represents 83% of the maximum daily consumption.

3.13. Conclusion

Tanks are mandatory storage structures in a distribution network. They can be classified according to their shape (circular, square, or rectangular) or their position (buried, semi-buried, ground-level or elevated). Each tank is equipped with an inlet pipe, an outlet pipe, an overflow pipe, a bottom drain, a fire reserve, a float valve, etc.). Determining the volume of the tank remains a very delicate task due to variations in hourly consumption. This volume varies from 30 to 83% of maximum daily consumption, depending on the delivery method and the size of the city.

Chapter 4.
Distribution networks

4.1 . Introduction

A distribution network is a set of interconnected pipes operating under pressure that allows variable consumption flow rates to be supplied at a relatively constant pressure (1 to 4) bar and at a flow velocity ranging from 0.3 to 1.75 m/s. The flow velocity must be between a minimum and maximum value. Vmin ensures self-cleaning to prevent deposits (Figure 4.1).

Vmax is related to the erosion of the pipe lining material: this data is provided by the manufacturers.

$V_{min} = 0.3$ [m/s]
$V_{max} \approx 1.00 - 1.20$ [m/s] (PVC)
$V_{max} \approx 1.50 - 1.75$ [m/s] (Cast iron)

Figure 4.1: Scaling due to limestone deposits.

A drinking water supply system consists of (Figure 4.2):

- Main and secondary pipes
- One or more storage facilities (Reservoirs)
- One or more supply pipes (gravity-fed or pumped)
- One or more catchment areas (groundwater or surface water source).

A drinking water distribution network is an infrastructure system that allows the distribution of drinking water to a population of users. It consists of various components, such as pipes, tanks, pumping stations and other equipment. The drinking water distribution network can be managed by local authorities, private companies or public institutions (ADE). The main stages of the drinking water distribution network are as follows:

Water collection: Water is collected from water sources, such as rivers, lakes or aquifers, and delivered to drinking water production plants.

Water treatment: The water is treated to eliminate impurities and achieve the quality standards required by legislation. Treatments include screening, sieving and disinfection.

Distribution: Drinking water is delivered to tanks, which may be buried or elevated. The tanks maintain pressure in the network and supply the taps

The drinking water distribution network is essential for ensuring a sustainable supply of good quality drinking water for the population. It is also important to monitor and control the quality of the water distributed to ensure that it meets the drinking standards established by legislation (see annexes).

A distribution network must meet the following requirements:

- Continuity of supply service in all seasons and at all times;
- Satisfaction of pressure conditions ($P_{service} < P < PN$);
- Coverage of the entire area concerned;
- Transport of peak flows while respecting pressure conditions;
- Compliance with velocity constraints $V_{min} < V < V_{max}$

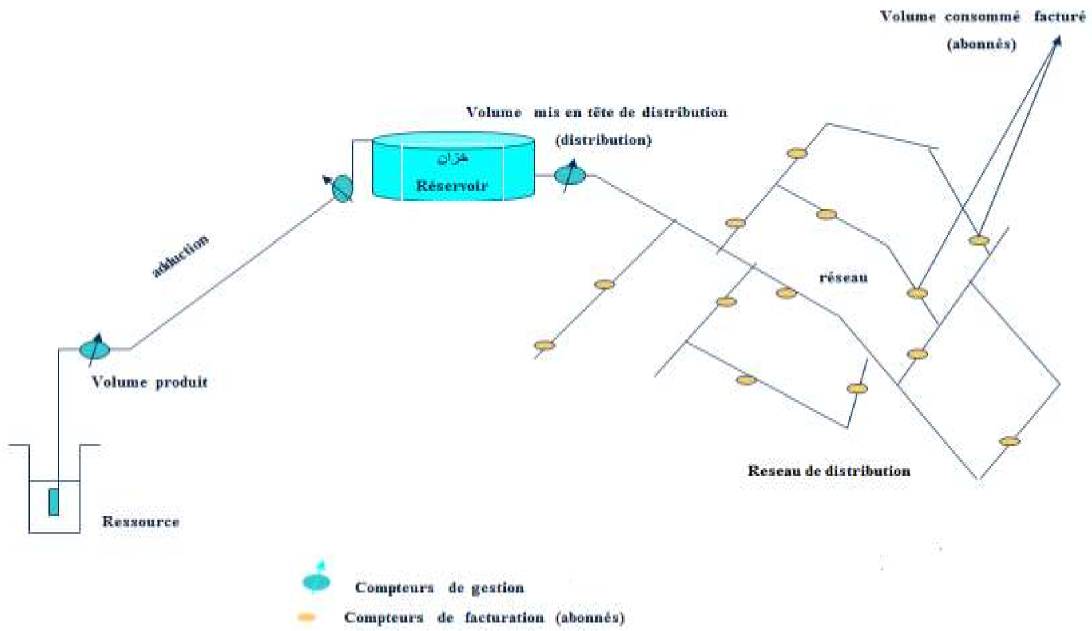


Figure 4.2: Descriptive diagram of water supply system

4.2 Water distribution network types

The distribution network can have a dead-end or tree form or a ring form (Figure 4.3) or mixed which is more common.

A closed circuit composed of constituent elements of a network is called **a ring.**

A place where one or more pipes, pumps, tanks or other equipment are connected is called a **Node.**

Ring network is a secure but uneconomical network

Dead-end or tree network is an insecure but economical network

a) Dead-end or tree network **b) Ring network**

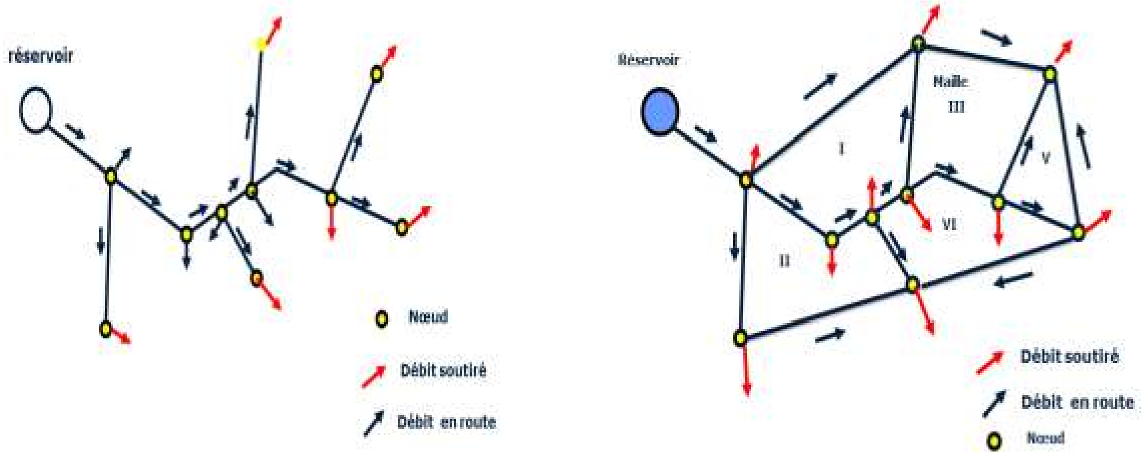


Figure 4.3: Distribution network types

4.3 . Components of a distribution network

A distribution network consists of:

- Main pipes : (The materials used for these pipes are PVC, HDPE, Asbestos cement, Cast iron, (Figure 4.5)
- Secondary pipes: (They are found inside the meshes connecting to the main ones).



PVC pipe HDPE pipe


Cast Iron Pipes Asbestos-Cement Pipes

Figure 4.5: Pipes in a distribution network

Table 4.1 shows the diameters of pipes sold in Algeria.

Table 4.1: Comparaison betwwen several diameters and types of pipes

Pipe type	Diameter	Disadvantages	Benefits
PVC	- 36/40 - 27.2/32 -57/63 - 45.2/50 - 81.4/90 - 67.8/75 – 113/125- 99.4/110 -144.6/160 -126.6/140 – 285/315 -180.8/200 361.8 /400	For the diameters > it is necessary to make a Require a qualified workforce	Available at competitive prices With stand pressures of 6 to 10 bars Setting easy to install
Asbestos cement **Class 20** **Class 30**	-150 – 125 -100 - 80 -400 -300 -250 -200 600 -500	Internal and external deterioration depending on water quality with possibility obstruction	Withstand pressures of 20 to 30 bars Acceptable prices Available on the National market
Steel	-80/90-66/76-50/60 -140-102/114-90/102 170 1200mm	Rapid internal erosion High price Place in non-acidic soil	Withstand pressures up to 40 bars
Melting	-125-100-80-60-50-40 -350-300-250-200-150 -700-600-500-450-400 mm -1000-900-800 1200	Not available on the national market Very price pupil	High resistance to internal and external erosion Lightweight Working at pressures of 40 bars

Table 4.2: groups together the geometric characteristics of PVC, HDPE and asbestos cement conduits.

Table 4.2: Diameter ranges for PVC-HDPE and Asbestos Cement CL20 pipes

PVC pipes PN 10 Bars			HDPE pipes PN 10 bars			Asbestos Cement Pipes CL 20		
DN	e	Dint	DN	e	Dint	DN	E	Dint
Mmm	mm	mm	mm	mm	Mm	Mm	Mm	mm
25	1.5	22	20	2	16	80	14	80
32	2.4	27.2	25	2	21	100	14	100
40	2	36	32	2.4	27.2	125	14	125
50	2.4	45.5	40	3	34	150	15.5	150
63	3	57	50	3.7	42.6	200	19	200
75	3.6	67.8	63	4.7	53.6	250	22	250
90	4.3	81.4	75	5.6	63.8	300	26	300
110	5.3	99.4	90	5.4	79.2	400	35	400
125	6	113	110	6.6	96.8	500	42	500
140	6.7	126.6	125	7.4	110.2	600	50	600
160	7.7	144.6	160	9.5	141			
200	9.6	180.8	200	11.9	176.2			
315	15	285	250	14.8	220.4			
400	19.1	361.8	315	18.7	277.6			
			400	23.7	352.6			
			500	29.7	440.6			
			630	37.4	555.2			

4.4. Main factors generating leaks in a drinking water network

They are diverse and of varying importance depending on the regions and operating conditions. The following are some examples:

► Ground movement: Case of unstable soils such as floodplain, clay fills, etc.

► Corrosion:

Case of insufficiently protected metal equipment, placed in acidic soils, crossed by groundwater or the seat of stray currents

► Rolling loads:

Case of pipes laid under high density road traffic or subject to significant rolling loads.

4.5. Fittings in a drinking water distributions network

In addition to the pipes, the main fittings of a drinking water distribution network include:

- Gate valves: (globe valves - Butterfly valves);

- Air vents

- Valves: (double-flap non-return valves - butterfly non-return valves))

- Elbows: (Flanged elbows or 2-socket elbows)

- Tees: (**Flanged tees** and socket tees)

- Reduction cones : (Reduction cone with two flanges - Reduction cone with 2 sockets)

- Fire hydrants

Most special fittings are made of cast iron, PVC or HDPE, for the dimensions of special fittings (see Appendix). Figure 4.6 shows special fittings found in a distribution network.

a) Gate valves

b) Air valve **c) Non-return valve** **d) Elbows**

e) Tees

f) Reducing cones

g) Fire hydrant

Figure 4.6: Special fittings of pipes

4.6.Fire fighting Recommendations

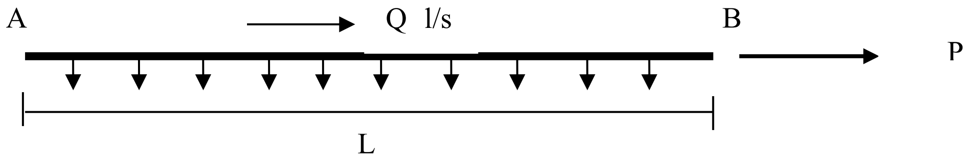
Regulations generally require fire coverage based on providing fire services with 120 m^3 of water for two hours. This fire coverage can be provided by fire hydrants with a diameter of 100 mm delivering a minimum of 60 m^3/h or 17 l/s under a residual pressure of 1 bar (range of 150 m via vehicle access routes), in addition to areas where water can be drawn (ponds, rivers, etc.). (Range of 400 m via roads).

4.7. Calculation of a distribution network

4.7.1. Dead-end or tree network

When it comes to tree network, in addition to the end flow rate, the pipe must be able to distribute water to consumers along its length via the numerous connections attached to it: this is the flow rate along the route. The flow rate along the route is calculated based on the number of users to be served at peak times and assuming that this flow rate is evenly distributed along the street.

In fact, Q is the flow rate thus distributed, the log of a pipe AB of length L, assuming that it must distribute a uniform flow rate along its length and ensure a flow rate P at its end.



The flow rate of the pipe is determined by the following formula:

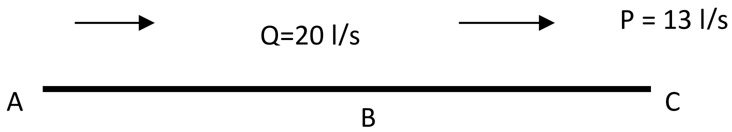
$$\boldsymbol{q = P + 0.55\,Q} \tag{4.1}$$

With :

q : Section flow.
P: Downstream flow.
Q: Flow on route.

Application example

Section ABC, which is supplied in the AC direction, provides en route services with flow rates of Q=20 l/s and P=13 l/s, which are the requirements for sections AB and BC, respectively. What are the proposed flow rates for calculating sections AB and BC?



Section AB:$q = P + 0.55\ Q = 13 + 0.55 * 20 = 24\ l/s$

The BC section:$q = P + 0.55\ Q = 0 + 0.55 * 13 = 7.15\ l/s$

So sections AB and BC will be calculated respectively with flow rates 24 l/S and 7.5 l/s instead of flow rates 33 l/s and 13 l/s

There are three methods for determining this flow:

a) Length method

$q_{sp} = \frac{Qp}{\sum Lg}$ (4.2)

With :

q_{sp} : Specific flow rate (l/s/ml);

Q_p : Peak flow (l/s);

$\sum Lg$: Sum of the geometric lengths of the network.

Therefore, to estimate the flow which represents the needs of the sections, it is enough to apply the following formula (4.3):

$$Qr = q_{sp} L_{i,,section} \quad (4.3)$$

b) Surface method

The specific flow is given by:

$$q_{sp} = \frac{Qp}{At} \quad (4.4)$$

At : Total surface area of the network (ha).

Therefore, the flow, which represents the needs of the sections connected to node i, is determined as follow:

$$Qni = q_{sp} Ai \quad (4.5)$$

Ai : surface corresponds to Node i, (ha).

c) Method based on population

The specific flow is given by:

$$q_{sp} = \frac{Qp}{Pt} \quad (4.6)$$

Pt: Total population of an urban area (inhabitant).

Therefore: the flow rate en route, which represents the needs of the population connected to section I, will be:

$$Qtr = q_{sp} P_{tr.i} \quad (4.7)$$

$P_{tr.i}$: Population connected to section i

4.7.2 Network calculation steps

Determining the outgoing flow rates at each node allows us to set the flow rates for each section (from downstream to upstream).

Knowing the coast of the natural terrain and the dynamic pressure to be observed at each node, we can set the minimum piezometric coast of each node.

Each section is defined by:

- Its flow
- Its diameter
- Its length
- Its upstream load rating

It is then necessary to research the diameters of each section from upstream to downstream by successive tests so that the pressure and velocity constraints are met.

The diameters will be determined according to the following formula:

$$D = 1{,}27.Q^{0,5} \tag{4.8}$$

Generally the steps involved for calculating a dead-end network are:

1- Estimated peak flow for the entire urban area (population and equipment)

2- calculation of the needs of the sections Qtr

a) Specific flow

qsp = Qp / Σ Lg (l/s/ml) or qsp = Qp / Np (l/s/ hab)

Σ Lg: Sum of the geometric lengths of the distribution network (m)

Np : Size of the population to be served

b) **Section needs**

Qtr = qsp .Lg or Qtr = qsp. NPtr (l/s)

3- Calculation of flow requirements based on network configuration, which will be used to calculate pipes diameters.

4- Determination of the diameter for each section (for an average velocity v=1m/s, that is to say:

$$D = \left(\frac{4Q}{v\pi}\right)^{1/2}$$

Or according to the relation (4.8). $D = \mathbf{1{,}27}.Q^{0,5}$

5- A standardized diameter will be adopted in accordance with table 4.2

6- Verification of the average flow velocity according to the relationship:

V=Q/S, which gives: $v = \frac{\boldsymbol{Q}}{\boldsymbol{S}} = \frac{\boldsymbol{4Q}}{\boldsymbol{\pi D^2}}$

This velocity must be between 0.5 and 1.5 m/s

7- Development of a calculation table called Hydraulic calculation table (see table 4.4).

For each chosen diameter the calculation involves the calculation of the hydraulic gradient J, either according to the **Darcy's** relationship (4.9) **Hazen-Williams'** relationship (4.10).

$$J = \frac{fv^2}{2gD} = 8\frac{fQ^2}{\pi^2 gD^5} \tag{4.9}$$

$$J = 10{,}675\left(\frac{Q}{CHW}\right)^{1{,}852}\frac{1}{D^{4{,}75}} \tag{4.10}$$

The friction coefficient *f* will be determined with excellent accuracy according to the relationship of **Achour et al (2002)**

$$\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3{,}7D} + \frac{4{,}5}{Re}Log\frac{Re}{6{,}97}\right) \tag{4.11}$$

$$\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3{,}7D} + \frac{5{,}5}{Re^{0{,}9}}\right) \tag{4.12}$$

In which :

D: Diameter of the section (m);

ε : Absolute roughness of the internal wall of the pipe (m);

Re: Reynolds number given by:

$$Re = \frac{4Q}{\pi D\vartheta} \tag{4.13}$$

Q: Flow carried by each section (m^3/s);

ν : Kinematic viscosity of water ($\nu = 10^{-6}\ m/s^2$)

C_{HW} : **Hazen-Williams** coefficient, which is selected from tables such as Table 4.3 for reference purposes.

Table 4.3 : Value of the Hazen-Williams coefficient C_{HW}

Material	C_{HW} factor		
Asbestos Cement	-	140÷150	140
Brass	-	120÷150	-
Black steel (dry systems)	130	100	100
Black steel (wet systems)	130	120	120
Cast iron - New unlined	130	120÷130	100
Cast iron - 10 years old	100	105÷75	-
Cast iron - 15 years old	100	100÷60	-
Cast iron - 20 years old	80	95÷55	-
Cast iron - 30 years old	80	85÷45	-
Cast iron - 50 years old	80	75÷40	-
Cast iron - Bitumen-lined	-	140	-
Cast iron - Cement-lined	140	140	140
Concrete	120	85÷150	140
Copper	-	120÷150	150
Fiberglass pipe	-	150÷160	-
Fire hose (rubber)	-	135	-
Galvanized steel	-	120	120
Lead	-	130÷150	-
Polyethylene	-	150	-
PVC and plastic pipe	150	150	150
Stainless steel	-	150	150
Steel new and unlined	-	140÷150	-
Steel, welded and seamless	130	100	-
Vitrified clays	-	110	-
Wood	120	-	-
Clay, new riveted steel	110	-	

The hydraulic gradients for HDPE pipes can be taken from the tables in the appendices **(Bedjaoui tables (2005)).**

8- The total head loss ΔHta long each section will be given by:

$$\Delta Ht = \Delta HL + \Delta Hs \qquad (4.14)$$

ΔHL: Linear pressure loss (m)

$$\Delta HL = JL \qquad (4.15)$$

ΔHs : Head loss due to the presence of singularities (Elbows, Tees, or any other special fittings), if we cannot calculate it, it is better to estimate it at 10 to 15% of the linear loss depending on the importance special fittings present in the network.

$\Delta Hs = (10 - 15)\% \, \Delta HL$

9- Determination of ground pressures (Ps: Pressures at the nodes):

$$Ps = CP\text{-}C_{TN} \qquad (4.16)$$

CTn : Natural ground level (Start and End of the section)

CP = CPam : Upstream piezometric levels (start of the section)

CP = CPav : Downstream piezometric levels (End of section)

$$CPav = CPam - \Delta Ht \qquad (4.17)$$

Table 4.4: Hydraulic calculation table (case of a dead-end network)

Tronçons	Q (m^3/s)	cotes TN		L (m)	Dcal (m)	DN (m)	v (m/s)	f	J	ΔHt (m)	CP (m)		PS (mce)	
		Am	AV								Am	AV		

Noticed :

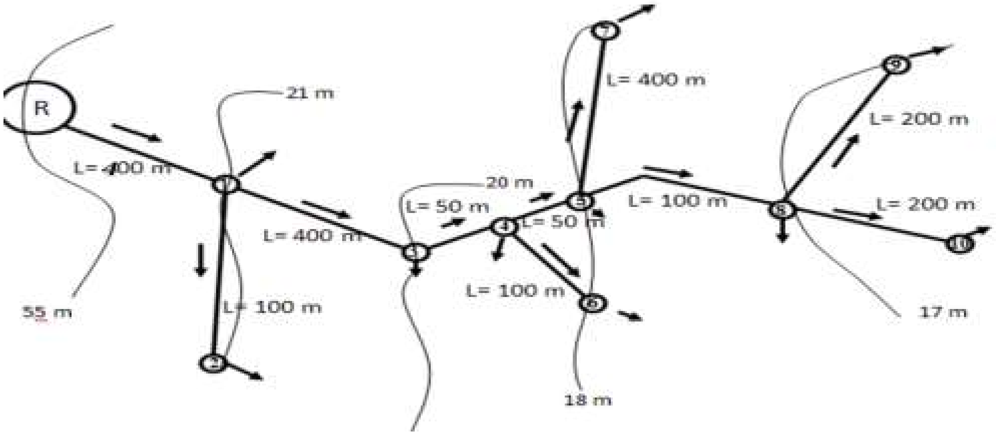
A good calculation is justified if:

$(0{,}5 \leq V \leq 1{,}5)$ (m/s)

$(10 \leq Ps \leq 40)$ (m water column)

4.7.3. Application example

Consider the distribution network shown in the figure below, which supplies an urban area with a flow rate of 40 l/s from a reservoir located at an elevation of 55 m. Calculate this network and equip it with pipes with an absolute roughness of ε = 0.01 mm, see figure below.



1- Normal operating conditions

a) Calculation of section requirements

The flow rates of the sections (needs) will be calculated according to the relation (4.3), that is to say: $Q_{tr} = q_{sp}.Lg_{tr}$ with $q_{sp} = Q_p / \Sigma Lg = 40/2000 = 0.02$ l/s/ml

The obtained results are summarized in table 4.5

Table 4.5: Section flow values

Sections	L(m)	Q section (l/s)
1-R	4 00	8
1-2	100	2
1-3	4 00	8
3-4	50	1
4-5	50	1
4-6	100	2
5-7	400	8
5-8	100	2
8-9	200	4
8-10	200	4
Total	**2000**	**40 l/s**

a) Calculation of road flow rates (sections with road service (see table 4.5).

b) The flow rates at which the pipes will be sized are calculated using the following formula:

$$q = P + 0.55\,Q$$

Table 4.6: Values of road flow rates

Sections	Q tr (l/s)	P (l/s)	Q road (l/s) P+0.55Q
8-10	4	0	2.2
8-9	4	0	2.2
5-8	2	8	9.1
5-7	8	0	4.4
5-6	2	0	1.1
4-5	1	18	18.55
3-4	1	21	21.55
1-3	8	22	26.4
1-2	2	0	1.1
R-1	8	32	36.4
Total	40		

The flow rates mentioned in the previous table will be used to size the distribution network (see table 4.7).

Table 4.7: Network hydraulic calculation table: Normal operating case

Sections	*Q (m3/s)*	*D (m)*	*DN (m)*	*L (m)*	*v (m/s)*	*J*	*Δ Ht (m)*	*CTN*		*CP (m)*		*P.S. (mce)*	
R-1	0.0364	0.2423	0.1808	400	1.42	0.0087	4.80	55	21	55.00	50.20	**0.00**	**29.20**
1-2	0.0011	0.0421	0.0455	100	0.68	0.0123	1.65	21	21	50.20	48.55	**29.20**	**27.55**
1-3	0.0264	0.2064	0.1808	400	1.03	0.0048	2.57	21	20	50.20	47.63	29.20	27.63
3-4	0.02155	0.1864	0.1808	50	0.84	0.0033	0.22	20	19	47.63	47.41	**27.63**	**28.41**
4-5	0.01855	0.1730	0.1808	50	0.72	0.0025	0.16	19	18	47.41	47.25	**28.41**	**29.25**
4-6	0.0011	0.0421	0.0570	100	**0.43**	0.0042	0.52	19	18	47.41	46.89	**28.41**	**28.89**
5-7	0.0044	0.0842	0.0814	400	0.85	0.0089	4.80	18	18	47.25	42.45	**29.25**	**24.45**
5-8	0.0091	0.1212	0.1130	100	0.91	0.0068	0.91	18	17	47.25	46.34	**29.25**	**29.34**
8-9	0.0022	0.0596	0.0570	200	0.86	0.0143	3.94	17	17	46.34	42.40	**29.34**	**25.40**
8-10	0.0022	0.0596	0.0570	200	0.86	0.0143	3.94	17	17	46.34	42.40	**29.34**	**25.40**

CTN: Natural Ground Elevation: mNGA)
C.P: Piezometric Elevation (m)
P.S: Ground Pressure (Nodes) (m water colomn)

- The velocity of 0.43 m/s is acceptable given that the sedimentation velocity of the particles is 0.30 m/s.
- The pressures obtained are between 10 and 40 mce

2- Network calculation: Case of fire

For firefighting purposes, a fire hydrant is provided and located at node 3, which dominates a large area, and the fire flow rate is 17 l/s.

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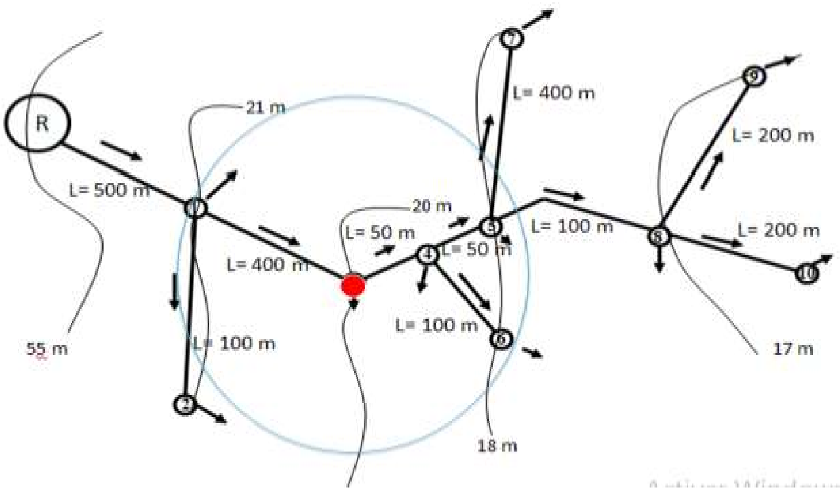


Fire hydrant location (Node 3)

The same calculation steps described for normal operation will be followed in this case. The flow rate used to size the main distribution pipe becomes (Q= 40+17 = 57 l/s).

$$qsp = 57/2000 = 0.02 \text{ l/s/ml}$$

The obtained results are listed in tables 4.8 and 4.9

Table 4.8: En-route flow rates Qr (l/s).

Sections	Q tr (l/s)	P (l/s)	Qtrn (l/s) P+0.55Q
8-10	4	0	2.2
8-9	4	0	2.2
5-8	2	8	9.1
5-7	8	0	4.4
5-6	2	0	1.1
4-5	1	18	18.55
3-4	1	21	21.55
1-3	8	22	26.4 +17
1-2	2	0	1.1
R-1	8	32	36.4 +17
Total	40		

Table 4.9: Hydraulic calculation table: Case of fire

Sections	*Q (m3/s)*	*D (m)*	*DN (m)*	*L (m)*	*v (m/s)*	*J*	*Δ Ht =JL (m)*	*CTN*		*CP (m)*		*P.S. (mce)*	
R-1	0.0534	0.2423	0.1808	400	**2.08**	0.0177	7.77	55	21	55.00	47.23	0.00	26.23
1-2	0.0011	0.0421	0.0455	100	0.68	0.0123	1.35	21	21	47.23	45.88	26.23	24.88
1-3	0.0434	0.2064	0.1808	400	1.69	0.0120	5.30	21	20	47.23	41.93	26.23	21.93
3-4	0.02155	0.1864	0.1808	50	0.84	0.0033	0.18	20	19	41.93	41.75	21.93	22.75
4-5	0.01855	0.1730	0.1808	50	0.72	0.0025	0.14	19	18	41.75	41.61	22.75	23.61
4-6	0.0011	0.0421	0.0570	100	**0.43**	0.0042	0.46	19	18	41.75	41.29	22.75	23.29
5-7	0.0044	0.0842	0.0814	400	0.85	0.0089	3.92	18	18	41.61	37.68	23.61	19.68
5-8	0.0091	0.1212	0.1130	100	0.91	0.0068	0.75	18	17	41.61	40.86	23.61	23.86
8-9	0.0022	0.0596	0.0570	200	0.86	0.0143	3.15	17	17	40.86	37.71	23.86	20.71
8-10	0.0022	0.0596	0.0570	200	0.86	0.0143	3.15	17	17	40.86	37.71	23.86	20.71

Noticed :

For the case of fire, a speed of 2.08 m/s is acceptable although it exceeds 1.5 m/s. the pressures obtained are acceptable

4-7-4 Calculation of a ring network

The ring network is derived from the tree network by connecting the ends of the pipes (usually at the tertiary pipe level), allowing for a return supply. This means that each point in the network can be supplied with water from two or more sides. Small streets are always supplied by branches (Figure 4.7).

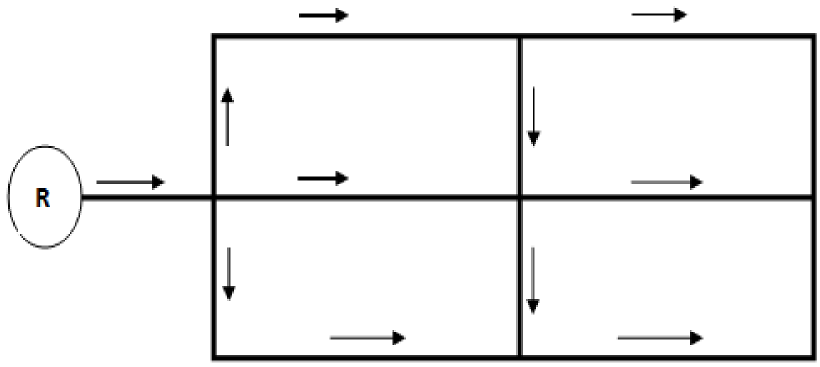


Figure 4.7: Ring network

This type of network offers the following advantages:

- ✓ greater security of supply (if a pipe breaks, it can simply be isolated and all consumers downstream will be supplied by the other pipes).
- ✓ more uniform distribution of pressure and flow throughout the network. However, it is more expensive and more difficult to calculate.

NB: The calculation of a ring network is based on the Hardy-Cross method

a) Principle of the HARDY CROSS method

This method is based on two laws or principles (equivalent to **Kirchoff 's laws** in electricity):

- First law or (law of conservation of mass): At any node in the network, the sum of the flow rates arriving at this node is equal to the sum of the flow rates leaving it (figure 4.8):

$$\sum Q_e = \sum Q_s$$

So, for node A, for example, we have

$$\boldsymbol{Q_A = q_1 + q_6}$$

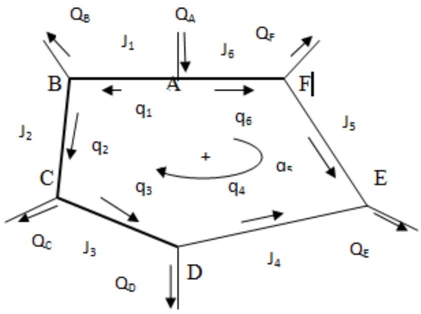


Figure 4.8: Representation of a ring

- Second law or (law of conservation of energy, which states that: Along an oriented and closed path (one ring), the algebraic sum of the head loss is zero.

$$\sum_{i=1}^{n} \Delta Hti = 0 \qquad (4.18)$$

Thus, for the ABCDEF contour, where the positive orientation is given by the clockwise direction and for the flow direction indicated by the arrows.

$$\Delta Ht6 + \Delta Ht5 - \Delta Ht1 - \Delta Ht2 - \Delta Ht3 - \Delta Ht4 = 0$$

The Hardy Cross method consists, first of all, of establishing a provisional distribution of flow rates and a clockwise flow direction throughout the network, while complying with the first law. This initial distribution allows the diameters of the pipes to be chosen, at least provisionally (with velocities between 0.50 and 1.25 m/s), and the corresponding head losses to be calculated.

Usually, the algebraic sum of the head losses cannot be zero in all rings from the outset. Without changing the chosen diameters and without disturbing the first law, the initial assumed distribution of flow rates in the sections must be corrected in order to rectify the pressure losses and verify the second law.

The correction is made by the error Δq made in each ring, which is given by equation (4.18).

Referring to the diagram in Figure 4.9, this error is evaluated as follows:

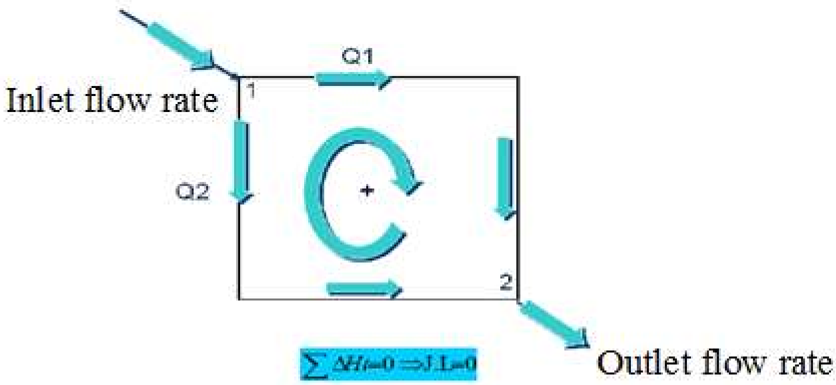


Figure 4.9: Ring

Let Q1 be the flow rate from node 1 to node 2, which is just one error + Δq.

Let Q2 be the flow rate from node 1 to node 2, which is just one error - Δq.

The head loss caused by flow rate Q1 is

$$\Delta H1 = J_1.L_1 = \frac{fv1^2}{2gD_1}L_1 = 8\frac{fQ_1{}^2}{\pi^2 gD_2{}^5}J_2.L_1 = r_1.Q_1{}^2 \tag{4.19}$$

The head loss generated by the flow Q2 is:

$$\Delta H2 = J_2.L_2 = \frac{fv^2}{2gD_2}L_2 = r_2.Q_2{}^2 \tag{4.20}$$

Suppose that the final flow according to direction 1 once equilibrium is reached is

$$Q_1{}' = Q_1 + \Delta q \tag{4.21}$$

Suppose that the final flow according to direction 2 once equilibrium is reached is:

$$Q_2{}' = Q_2 + \Delta_q \tag{4.22}$$

$$\Delta H_1{}' = r_1.Q'_1{}^2 \tag{4.23}$$

The head losses generated by the flow rates Q'_1 and Q'_2 are:

$$\Delta H_1{}' = r_1.Q'_1{}^2 \tag{4.24}$$

$$\Delta H_2{}' = r_1.Q'_2{}^2 \tag{4.25}$$

At equilibrium, the head losses equalize:

$$\Delta H'_1 = r_1.Q'_1{}^2 = \Delta H_2{}' = r_1.Q'_2{}^2 \tag{4.26}$$

Which means :

$$\Delta H'_1 - \Delta H_2{}' = 0 \tag{4.27}$$

Substituting relations (4.21), (4.22), (4.24) and (4.25) into relations (4.27), it follows that:

$$\Delta H'_1 - \Delta H'_2 = r_1.Q'_1{}^2 - r_1.Q'_2{}^2 = 0 \tag{4.28}$$

(4.28) becomes:

$$\Delta H'_1 - \Delta H'_2 = r_1.(\mathrm{Q1} + \Delta \mathrm{q})^2 - r_2.(\mathrm{Q2} + \Delta \mathrm{q})^2 = 0 \tag{4.29}$$

After development we obtain:

$$r_1(Q_1 + \Delta q)^2 - r_2(Q_2 - \Delta q)^2 = 0 \Rightarrow$$
$$r_1 Q_1{}^2 + 2r_1\,\Delta q.Q_1 + r_1\,\Delta q^2 - r_2 Q_2{}^2 + 2r_2\,\Delta q.Q_2 - r_2\,\Delta q^2 = 0$$

$\Delta H_1 = r1.Q1^2$ $\quad \Delta H_2 = r2.Q2^2$ $\quad \Delta q^2 \approx 0$

As :

$$r_1 Q_1^{\ 2} + 2r_1\ \Delta q.Q_1 + r1\ \Delta q^2 - r_2 Q_2^{\ 2} + 2r_2\ \Delta q.Q_2 - r2\ \Delta q^2 =$$
$$\Delta H_1 + 2r_1\ \Delta q.Q_1 - \Delta H_2 + 2r_2\ \Delta q.Q_2 = 0$$
$$\Rightarrow\ \Delta H_1 - \Delta H_2 + 2\ \Delta q(r_1.Q_1 + r_2\ .Q_2) = 0$$
$$\Delta q\ = - \frac{\Delta H_1 - \Delta H_2}{2\ (r_1.Q_1 + r_2\ .Q_2)} = - \frac{\Delta H_1 - \Delta H_2}{2\ (\frac{\Delta H_1}{Q_1} + \frac{\Delta H_2}{Q_2})} = - \frac{\sum \Delta H}{2 \sum \frac{\Delta H}{Q}}$$

Therefore, the relation giving the error made during the distribution of flow rates is given by the relation (4.30) and known as the **Fair's relationship.**

$$\Delta q = - \frac{\sum \Delta H_i}{2\ \sum \frac{\Delta H_i}{Qi}} \tag{4.30}$$

Each mesh is calculated separately, the corrections made to the flow rates are of two types:

- Correction specific to the mesh considered with the sign Δ q.
- Correction specific to the adjacent mesh with the sign of Δ q

b) Steps for calculating a ring network

The recommended steps for calculating a ring network are:

- Draw the distribution network on a site plan;

- Choose rings such that the minimum area of a mesh is 1 ha;

- The minimum diameter in a ring is 80 mm;

- Calculate the requirements for each section;

- Calculate the node flow rates;

- The flow rates representing the requirements (domestic and equipment) will be set at each node;

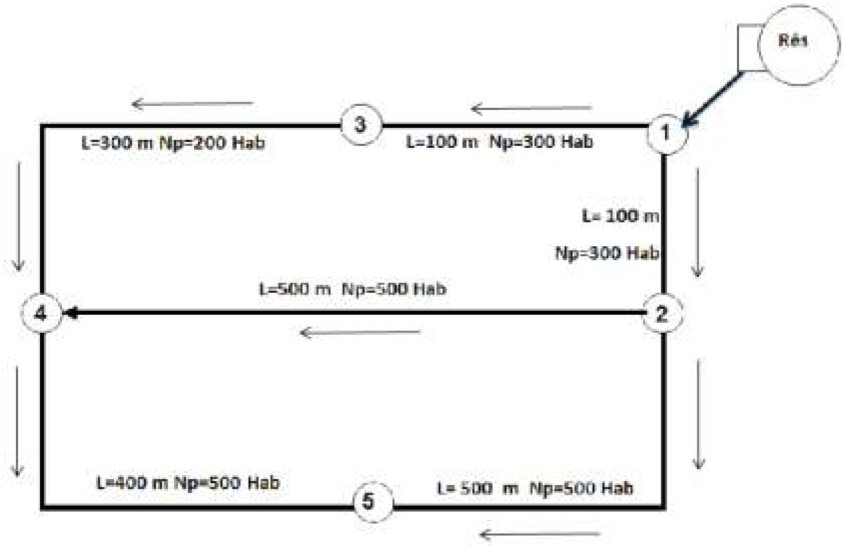
- Choose an arbitrary distribution of flow rates along the sections.

- Calculate the diameters of the sections based on the distributed flow rates.

- Develop a hydraulic calculation table based on the Hardy-cross method, using software or programmes such as LOOP, FASTER, HYDRAU EAU, Epanet, Water CAD, Porteau, Worteau, etc.

4.7.5. Application example

Design a ring distribution network supplying an urban area from a reservoir with a peak flow of 30 l/s (see figure). We give : $\varepsilon = 0.1$ mm and $\nu = 10^{-6}$ m²/s.



The network data is shown in the table below: (Table 4.9)

Table 4.9 : Network Data

Sections	Population	L(m)
R-1	500	500
1-2	300	100
1-3	400	400
3-4	300	200
2-4	500	500
2-5	500	500
5-4	500	400
Total	3000	2600

1- Calculation of section requirements _

$$Q_{tr} = q_{sp}.Np_{tr} \qquad q_{sp} = \frac{Qp}{Np} = \frac{15}{3000} = 0.005 \ l/\text{s/hab}$$

The requirements of the sections (l/s) are shown in the following table 4.10

Table 4.10: Sections requirements:

Sections	Population	Qtr (l/s)
R-1	500	2.5
1- 2	300	1.5
1- 3	400	2
2- 4	300	1.5
3- 4	500	2.5
2- 5	500	2.5
4- 5	500	2.5
Total	3000	15

2- Evaluation of node flow rates

The requirement for a node is equal to the sum of half the flow rates of the sections connected to that node, as shown in the table below.

Table 4.11: Assessment of nodal flow rates

Knots	Sections Attached to Nodes	Needs of the l/s node	Qn (l/s)
1	R-1; 1-2, 1-3	2.5 +(1.5+2)/2=4.25	4.25
2	1-2, 2-4, 2-5	(1.5+2.5+2.5)/2=3.25	3.25
3	1-3; 3-4	(2+1.5)/2=1.75	1.75
4	3-4, 4-5; 4-2	(1.5+2.5+2.5)/2=3.25	3.25
5	5-2; 5-4	(2.5+2.5)/2=2.5	2.5
Total			15

According to the diagram below, the flow rates of the first distribution are proposed:

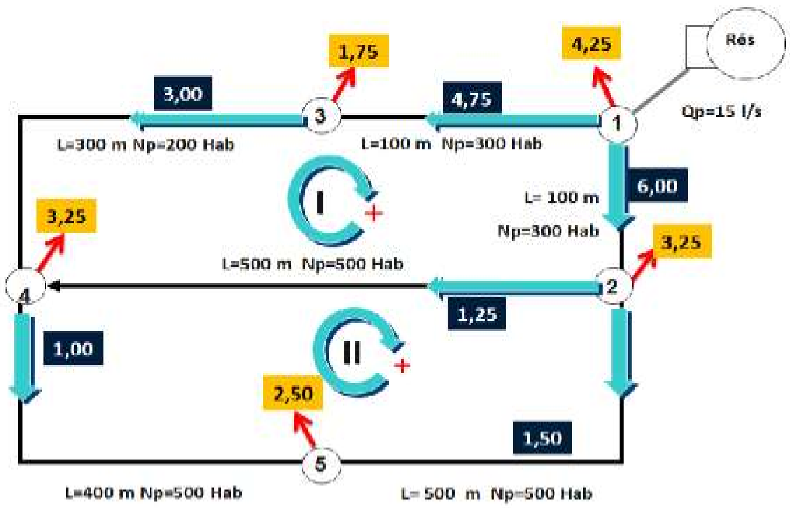


Figure: 4.10: First distribution proposed

flow rates will be used to calculate the mesh network in accordance with the table 4.12 below:

Table 4.12: Hydraulic calculation table for the ring network using the Hardy-cross method: Normal operating case (first iteration)

M,P	MY	Sections		Sense listen-ment	L(m)	Q (l/s)	D (mm)	V (m/s)	J	Δ Ht (m)	ΔH/Q	CMP	CMA	Q horn (l/s)
		R	1		500	15	0.1446	0.91	0.00608	3.04	202.79	0.00	0.0000	
I	2	1	2	1	300	6	0.1446	0.37	0.00109	0.33	54.51	1.3958	0.0000	7,396
		1	3	-1	400	4.75	0.0814	0.91	0.01244	-4.98	1047.44	1.3958	0.0000	-3.354
		3	4	-1	300	3	0.0814	0.58	0.00523	-1.57	522.97	1.3958	0.0000	-1.604
		2	4	1	500	1.25	0.0814	0.24	0.00104	0.52	415.89	1.3958	0.0829	2,729
	Sum									**-5.70**	**2040.81**	ΔΘ1 =	1.39	l/s
II	1	2	4	-1	500	1.25	0.0678	0.35	0.00256	-1.28	1023.47	-0.0829	-1.3958	-2,729
		2	5	1	500	1.5	0.057	0.59	0.00851	4.26	2837.41	-0.0829	0.00	1,417
	3	5	4	-1	500	1	0.057	0.39	0.00401	-2.00	2004.16	-0.0829	0.00	-1.083
	Sum									**0.97**	**5865.03**	ΔΘ2 =	-0.0829	l/s

Hydraulic calculation table for the mesh network using the Hardy-cross method: Normal operating case (Fourth iteration)

M,P	MY	Sections		Sense flow	L(m)	Q(l/s)	D(mm)	V (m/s)	J	Δ Ht (m)	ΔH/Q	CMP	CMA	Qcor (l/s)
		R	1		500	15	0.1446	0.91	0.00608	3.04	202.79	0.00	0.0000	
I	2	1	2	1	300	7,621	0.1446	0.46	0.00170	0.51	66.91	0.0289	0.0000	7,650
		1	3	-1	400	3,129	0.0814	0.60	0.00566	-2.26	723.43	0.0289	0.0000	-3,100
		3	4	-1	300	1,379	0.0814	0.27	0.00124	-0.37	270.53	0.0289	0.0000	-1,350
		2	4	1	500	2,613	0.0814	0.50	0.00404	2.02	773.30	0.0289	-0.0358	2,607
	Sum									**-0.11**	**1834.17**	ΔΘ1 =	0.0289	l/s
II	1	2	4	-1	500	2,613	0.0678	0.72	0.01010	-5.05	1932.24	0.0358	-0.0289	-2,607
		2	5	1	500	1,757	0.057	0.69	0.01146	5.73	3259.16	0.0358	0.00	1,793
		5	4	-1	500	0.743	0.057	0.29	0.00232	-1.16	1563.27	0.0358	0.00	-0.707
	Sum									**-0.48**	**6754.67**	ΔΘ2 =	0.0358	l/s

Noticed:

It is not necessary to continue the calculations for an approximation because the errors made in the initial distribution of flow rates are almost negligible.

Similarly, as with the tree network, the ring network must be checked for fire conditions. This involves recalculating the network, using the same diameters, adding the fire flow rate (17 l/s) at the sensitive points of the network. It is then necessary to check that the velocities in all sections are less than 2.5 m/s and that the pressures in all nodes are greater than 10 meters. If these conditions are not met, the diameters of certain sections must be modified and the calculation restarted from the beginning (during peak hours, then another check during peak hours + fires).

4.8.Conclusion

In real-life situation, there are three types of networks: (tree, ring, and mixed)

Generally, peak flow is used to calculate distribution networks. A good calculation leads to flow velocities between 0.5 and 1.5 m/s and pressures in the network between 10 and 40 eter water column. The pipes currently used are made of PVC, HDPE, and sometimes with GRP.

In summary, a drinking water distribution network ring or tree) is a water distribution system that uses underground pipes in the form of meshes or branches to supply different areas. ring networks are commonly used due to their advantages, such as the flexibility, lightness and non-corrosiveness of plastic pipes. The design of a ring drinking water distribution network is a complex process that takes into account several criteria and uses advanced design methods to ensure an adequate and reliable supply of drinking water for users.

Chapter 5.
Water Supply

5.1 Introduction

Water supply refers to all the techniques used to transport water from its source to its place of consumption. Water can be transported through pipes or aqueducts.

The water supply system consists of:

- From the source (river, body of water, aquifer), from which the water is pumped;
- The transport network (canal, pipes);
- Storage (tanks, water tower);
- Finally, the distribution network, which brings water to consumers (tap, fountain, etc.)

5.2 Study of the route

“Obligatory” paths

- “Mandatory” routes are imposed by the need to follow the route of the road network or road shoulders.

- Otherwise, limit passage through private land (to minimize expropriation)

Intermediate transport

Example: Pumping station - Reservoir

- Once the location of the reservoir(s) has been determined (based on the altitude of the areas to be served)

- The route to be adopted must:

- Be as short as possible to reduce initial installation costs

- Avoid multiple costly or fragile structures (crossings of rivers, canals or major roads, etc.)

- Avoid crossing private property requiring expropriation.

- Follow public roads, which offer the following advantages:

* Earthworks and pipe supply often less expensive
* Easy access to manholes containing fittings and pipes for repairs.

5-3 Types of water supply

There are two types of water supply

1- Gravity feed, where water flows at high pressure due to differences in hydraulic levels: the source is higher than the point of consumption and therefore moves by means of gravity, hence the name. This is the principle behind water towers (Figure 5.1).

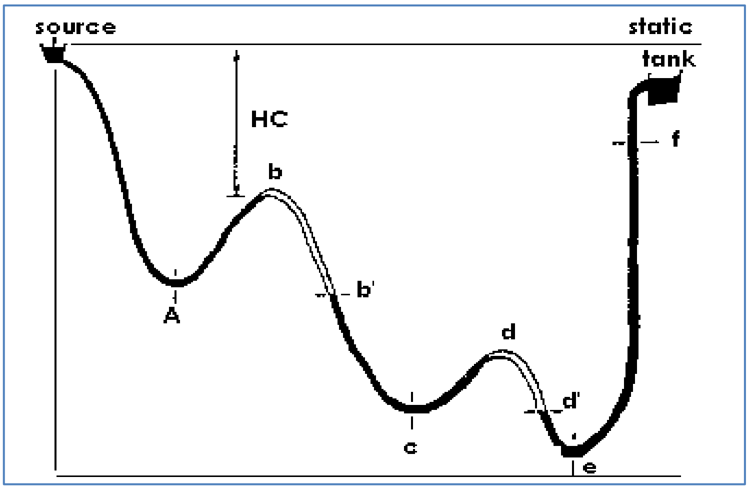


Figure 5.1: Gravity feed

2- Pressure supply where pressure is applied to the network and water is transported using pumps inside pumping stations (Figures 5.2 and 5.3).

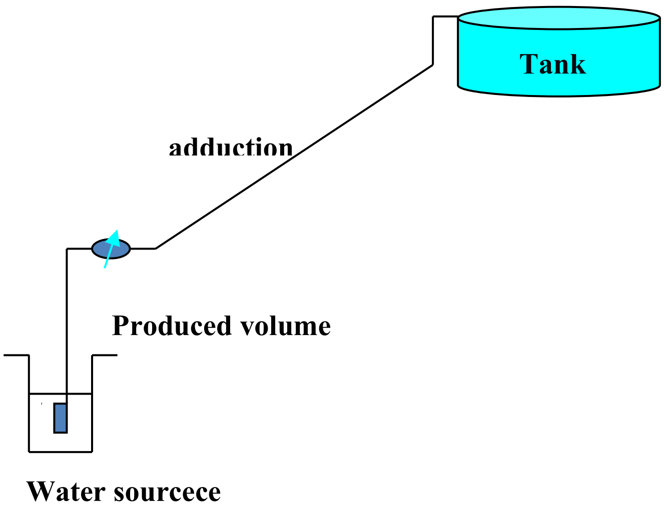


Figure 5.2: Pressure supply: (source a borehole)

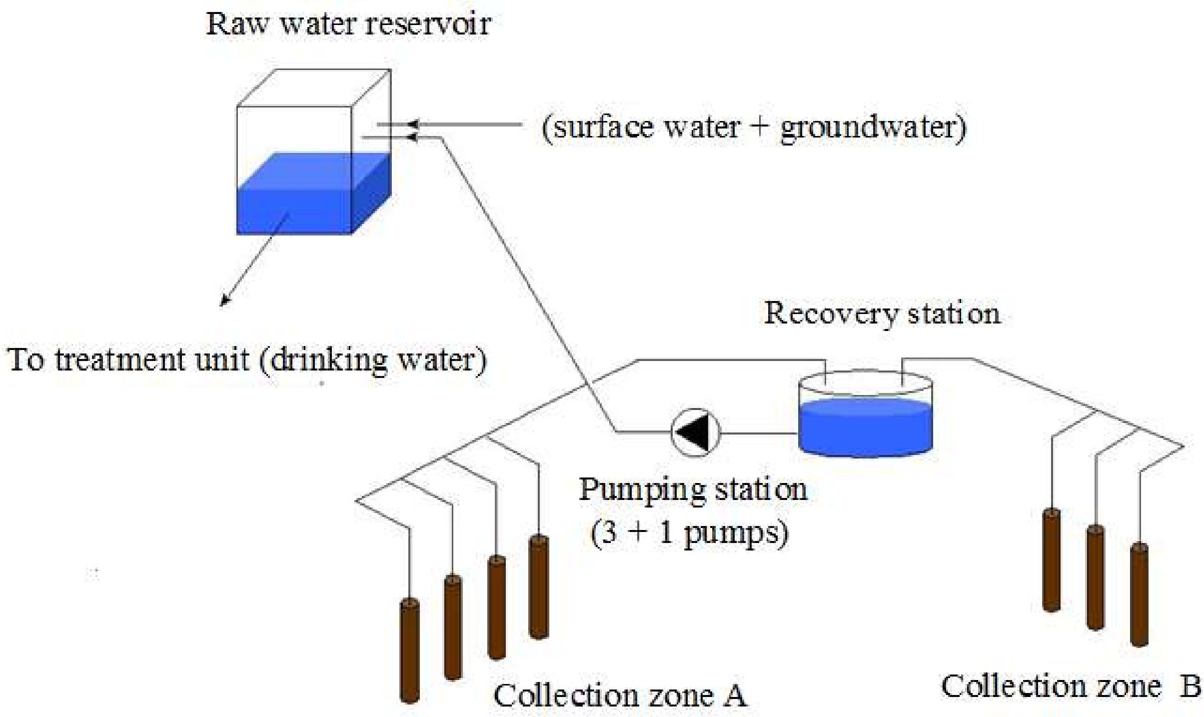


Figure 5.3: pressure supply (water collection field)

Pressure supply requires the presence of one or more pumps.

5.4 Calculation of gravity feed system

The diameter of gravity feed pipes must be determined based on the available load, the difference between the water level at the source and that at Laval, and the required water flow. It is necessary to check that the average velocity of the water in the pipe remains acceptable, i.e. between 0.50 m/s and 1.5 m/s. This is because a velocity of less than 0.5 m/s promotes deposits in the pipe, which can sometimes be difficult to remove, and air has difficulty reaching the high points. On the other hand, high velocities can create operational difficulties: the water hammer increases, cavitation and noise are possible, and there is a greater risk of water leakage.

5.4.1 Reminders of hydraulic concepts

Remember that the hydraulic head (m) in any section of a pipe is defined by (Figure 5.4):

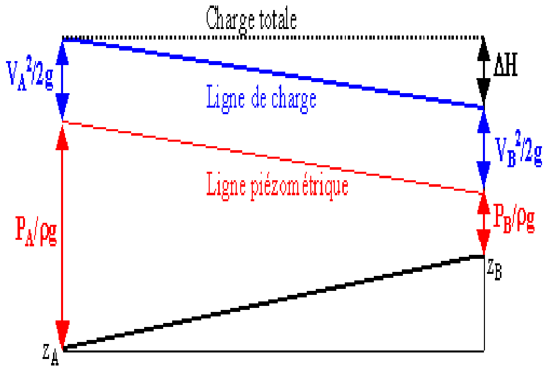


Figure 5.4: Bernoulli's theorem between two sections

The total head is written according to the relation (5.1à) by:

$$H = \alpha \frac{V^2}{2g} + \frac{P}{\rho g} + Z \tag{5.1}$$

In which :

V: Average velocity of the water in the pipe equal to flow/section (m/s);

P: Average pressure in the pipe (Pa)

g : Acceleration of gravity (g = 9.81 m/s);

Z: Average elevation of the pipe (m);

ρ : Density of water (ρ = 1000 kg/m 3);

α : Coefficient due to the non-homogeneity of velocities in the section equal to 1.05, we will take it, in the following, equal to 1 .

Let H1 be the hydraulic head in section S1 and H2 in section S2, Bernoulli's theorem, for a real fluid, allows us to write:

$$H1 = H2 + \Delta H \tag{5.2 }$$

Or :

Δ H : Represents the total head loss between section S1 and S2. The head loss is of two types:

1- Linear head loss: Distributed over the entire length of the pipe due to viscous and turbulent friction against the pipe walls.

a) According to Darcy formula (1875):

The linear head loss is expressed by:

$$\Delta H_L = JL = \frac{fv^2}{2gD}L = 8\frac{fQ^2}{\pi^2 gD^5}L \tag{5.3}$$

With :

J: Hydraulic gradient;

L: Length of the pipe (m).

f : Friction coefficient. This coefficient is given as a function of the Reynolds number. It is expressed by several formula, such as:

The relationship of **Achour et al (2002)**

$$\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3{,}7D} + \frac{4{,}5}{Re}Log\frac{Re}{6{,}97}\right) \tag{5.4}$$

The Swamme-Jaain Relationship **(1976)**

$$\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3{,}7D} + \frac{5{,}5}{Re^{0{,}9}}\right) \tag{5.5}$$

In which :

D: Diameter of the section (m);

ε : Absolute roughness of the internal wall of the pipe (m);

Re: Reynolds number given by:

$$Re = \frac{4Q}{\pi D\vartheta} \tag{5.6}$$

Q: Flow conveyed by each section (m^3/s);

ν : Kinematic viscosity of water ($\nu = 10^{-6}\ m/s^2$)

b) According to Hazen-Williams (1906).

The hydraulic gradient can be calculated by *Hazen-Williams' relation* (1906) as follow:

$$J = 10{,}675\left(\frac{Q}{CHW}\right)^{1{,}852}\frac{1}{D^{4{,}75}} \tag{5.7}$$

C_{HW} : *Hazen-Williams* coefficient that we choose from tables (See table 4.5 of chapter 4).

From the relationship (5.7), the linear head loss can be expressed as follow:

$$\Delta H_L = JL = 10{,}675\left(\frac{Q}{CHW}\right)^{1{,}852}\frac{1}{D^{4{,}75}}L = KQ^{1{,}852} \tag{5.8}$$

Noticed

For a quick calculation of the friction coefficient *f,* the Moody-Stanton diagram **(1944)** is recommended, noting that the result is not as accurate as the formulas mentioned above (figure 5.5).

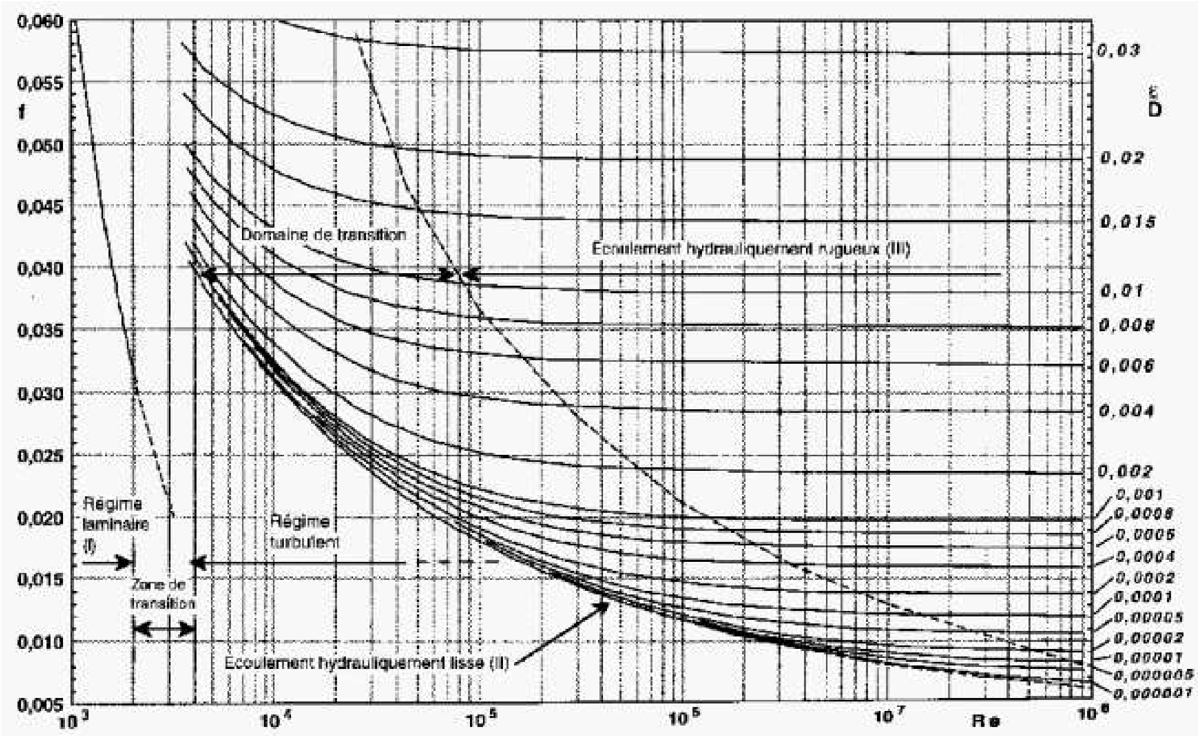


Figure 5.5 : **Moody-Stanton's diagram (1944)**

2- Singular or local head losses

They are due to the various singularities that can be placed along the pipeline. The singularities encountered on the pipes are generally changes in the section of the pipe (widening, narrowing, comma diaphragm, etc.) or changes in the direction of flow (elbows, branches, taps, valves, etc.). These singularities behave like "short structures" and cause local head losses.

The local head loss noted ΔH_S caused by these singularities are expressed as follow:

$$\Delta H_S = k\frac{v^2}{2g} \tag{5.9}$$

Or :
k is a coefficient which depends on the shape and dimensions of the singularity.

The values of this coefficient for the most encountered singularities are presented in these documents (General Hydraulics Manual, **A. Lancastre** or General and Applied Hydraulics by **M.Carlier).**

5.4.2 2 **Hydraulic characteristics of a pressurized pipe**

In practice, most industrial pipes operate under rough turbulent flow conditions, where the expression for the friction factor f becomes independent of the Reynolds number.

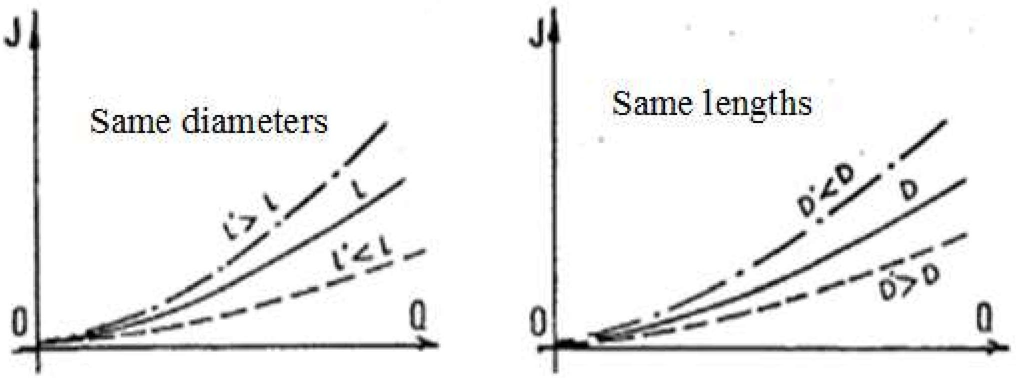
$$\frac{1}{\sqrt{f}} = -2Log\left(\frac{\varepsilon}{3{,}7D}\right) \tag{5.10}$$

The unit gradient (L=1m) is written in the form:

$$J = \left(\frac{\Delta H}{L}\right) = 8\frac{fQ^2}{\pi^2 g D^5}\frac{1}{L} = rQ^2 \tag{5.11}$$

r : Pipe resistance

The curve J as a function of Q^2 therefore provides the characteristic of this pipe as indicated by (figure 5.6):



F

Figure 5.6 : Characteristic curve of a pipe

a) Series pipes

The series pipes are traversed by the same flow rate. The total head loss is the sum of the linear and singular head losses.

Q1=Q2=Q3……… = Qn (5.12)

Δ Ht = Δ H1+H2 + Δ H3+……. (5.13)

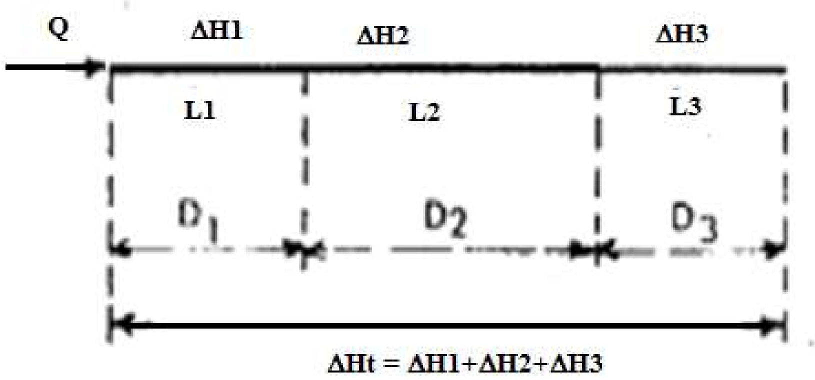


Figure 5.7: Assembly of series pipes

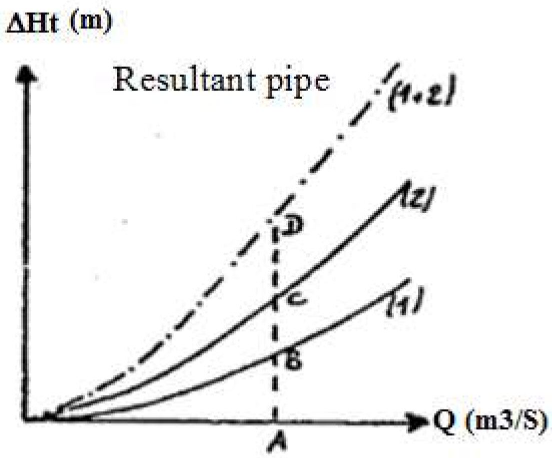


Figure 5.8 : **Characteristic curves of an equivalent pipe of two pipes in series.**

b) Parallel pipes

The parallel pipes (Figure 5.9) have the same head loss, but the total flow crosses all the pipes and the sum of the flow rates is written as follw:

Qt = Q1+Q2+Q3……… .+ Qn (5.14)

Δ Ht = Δ H1= ΔH2 = Δ H3=……. Δ Hn (5.15)

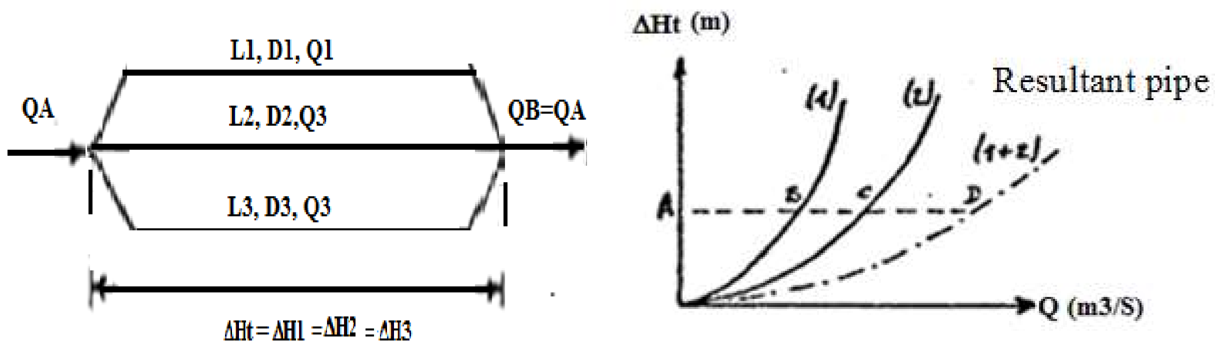
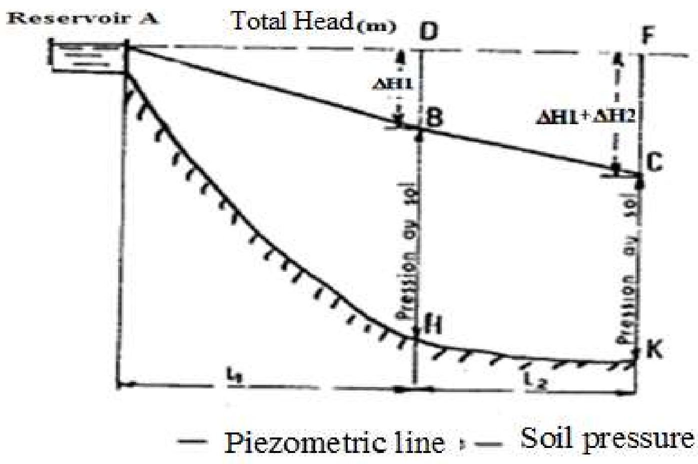


Figure 5.9: Assembly of pipes in parallel

c) Pipe leading from a tank



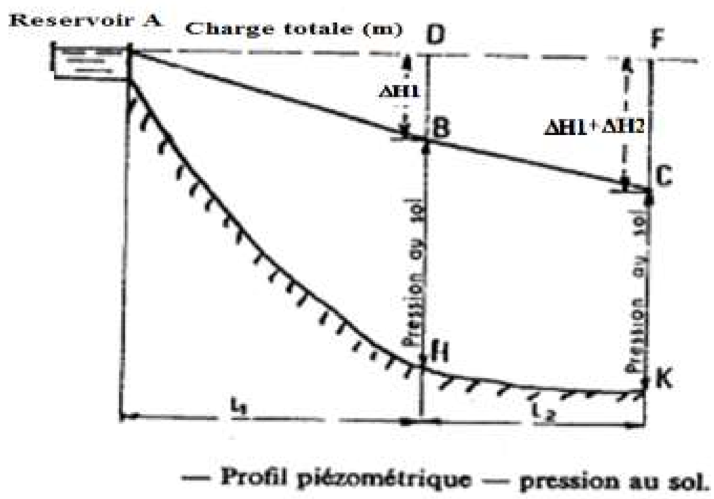


Figure 5.10: Pipe leading from a tank

Generally in this case (Figure 5.10) it is required to determine either the maximum flow rate from the tank if the diameter is reduced or to determine the diameter if the flow rate is fixed.

The maximum flow at point K and which comes from tank A is obtained for the greatest head loss, that is to say for: $\Delta H = \Delta H_1 + \Delta H_2 = FK$

5.4.3. Application example 1

Using Moody's diagram, calculate the friction loss in a welded steel pipe with a diameter of 900 mm, an length of 500 m, traversed by a flow of 2.3 m ^{3}/s, given ν=10 ^{6} m ^{2}/s.

Solution

Average water velocity is :$v = \frac{4Q}{\pi D^2} = 3.62\ m/s$

ε / D =0.000617

$Re = \frac{4Q}{\pi D \vartheta} = 2560490{,}59 = 2{,}6 \,.\, 10^6$

From Moody’s diagram: the coefficient of friction is f =0.0175

The hydraulic gradient will be: J= Δ H/L = 0.01298

The total head loss (if the singular losses are neglected) will be according to the relation (5.2):

$$\Delta H_L = JL = \frac{f v^2}{2gD} L = 8 \frac{f Q^2}{\pi^2 g D^5} L = 6{,}49\ m$$

Knowing the following hydraulic parameters of the pipe: the roughness of the pipe ε , the volume flow Q, the gradient of the head loss J, the kinematic viscosity of the water ν , it is possible to determine the diameter D of the supply pipe by referring to the relationships of the rough reference model method established by Achour-Bedjaoui (2002) and (2006):

5.4.4. Application Example 2

Let's take the data from application 1 and apply the relationships of the rough reference model method to determine the diameter of the gravity-feed pipe (D=900 mm).

The data are: the roughness of the pipe ε =0.6 mm, the volume flow Q =2.3 m ^{3} /s, the gradient of the pressure loss J=0.0038, the kinematic viscosity of the water ν =10 $^{-6}$ m ^{2} /s, it is possible to determine the diameter D of this pipe by referring to the following relationships:

Solution

Diameter of the reference pipe:

$$\bar{D} = (2\pi^2)^{-1/5} \left(\frac{Q^2}{gJ}\right)^{1L5} = 1{,}155\ m$$

Reynolds number of the reference pipe:

$$\bar{R} = 4\sqrt{2}\ \frac{\sqrt{gJD^3}}{\vartheta} = 2523489{,}537$$

Actual pipe diameter will be:

$$D = 1{,}35 \left[-Log(\frac{\varepsilon}{4{,}75\bar{D}} + \frac{8{,}5}{\bar{R}})\right]^{2/5} \bar{D} = 0{,}904\ m$$

Or the diameter is calculated from the Reynolds number R:

$$R = \frac{\bar{R}}{1{,}35}\left[-2Log(\frac{\varepsilon}{4{,}75\bar{D}} + \frac{8{,}5}{\bar{R}})\right]^{2/5} = 3239845{,}513$$

So: $D = \frac{4Q}{\pi R \vartheta} = 0{,}904\ m$

5.5. Calculation of a pressure supply system

In a pressure pipeline, the catchment (water source) is located at a lower level than the distribution reservoir. The catchment or treated water is lifted by a pumping station in this pressure pipeline (Figure 5.11).

The diameter of the pressure pipe is calculated either by applying empirical relationships or after an economic study. In the context of pump pipes, the economic diameter is determined by taking into account several factors such as: the required flow rate, the distance to be covered, the characteristics of the fluid being transported, the roughness of the pipe, head losses and operating costs. To find the economic diameter, a comprehensive economic analysis is required, comparing investment costs (initial costs related to the purchase and operation of the pipeline) and operating costs (costs associated with the energy required to move the fluid and friction losses). The objective is therefore to find the diameter that minimises the sum of investment and operating costs over the expected lifetime of the system.

The concept of economic diameter represents the diameter that minimizes the total cost of the pipeline over its expected service life.

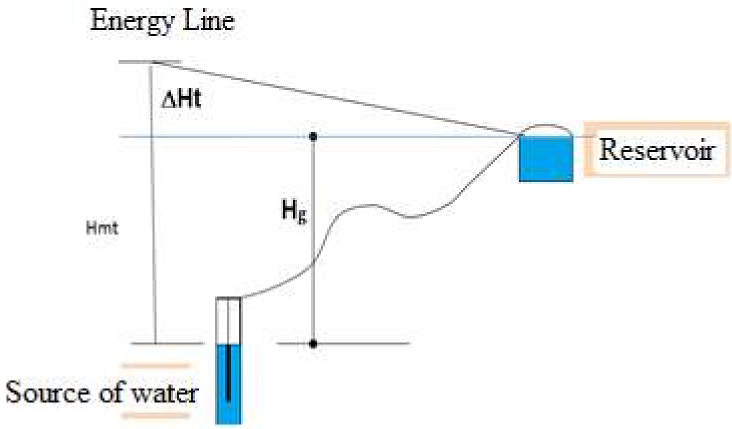


Figure 5.11: Pressure supply system

5.5.1. Calculation of economic diameter

5.5.1.1. Use of empirical formula (direct formula)

The main relationships used are:

Bonnin (1875): $D_{eco} = \sqrt{Q}$

BRESSE (1883): $D_{eco} = 1{,}5\sqrt{Q}$

Munier (1961): $D = (1 + 0{,}02h)\sqrt{Q}$

Square (1973): $D_{eco} = A\ Q^{0{,}46}$

Table 5.1: Values of coefficient A proposed by Carré 1973

Pumping hours	Tension	HAS
20	Low tension	1.3
08	Low tension	1.12
20	High tension	1.12
08	High tension	1

Bedjaoui-Achour (2005): $D_{eco} = 1{,}27\sqrt{Q}$

D , Krier (2011): $D_{eco} = 0{,}89\ Q^{0{,}486}$

Bacharou and al. (2012): $D_{eco} = 0{,}19(\frac{\alpha^4 t_p}{0{,}41.e.\rho_m.P_u+P_2})^{0{,}11} \, Q_{hmoy}{}^{0{,}44}$

With:

D_{eco}- Economic diameter of the discharge pipe (m);

e - Pipe thickness (m);

ρ_m - Density of the pipe material (Kg/m ^{3});

Pu - Unit price per linear meter of pipe;

P2 - Price depending on the diameter of the pipe;

Q_{hm} - Average hourly flow (m ^{3} /s);

tp - Pumping time (h);

$\alpha = K^3/(3K-2)$, where: K is a coefficient expressing the variation in consumption.

5.5.1.2 Economic study

From an economic side, the discharge pipe and the pumping station are linked. To raise a flow rate Q to a given height Hg, it is possible to use a pipe of any diameter; all that is required is to vary the power of the pumping station. In fact, the smaller the diameter, the greater the pressure loss and the greater the power supplied by the pump; therefore, there is an economical diameter for the discharge pipe, resulting from a compromise between the following two contradictory trends (see Figure 5.12):

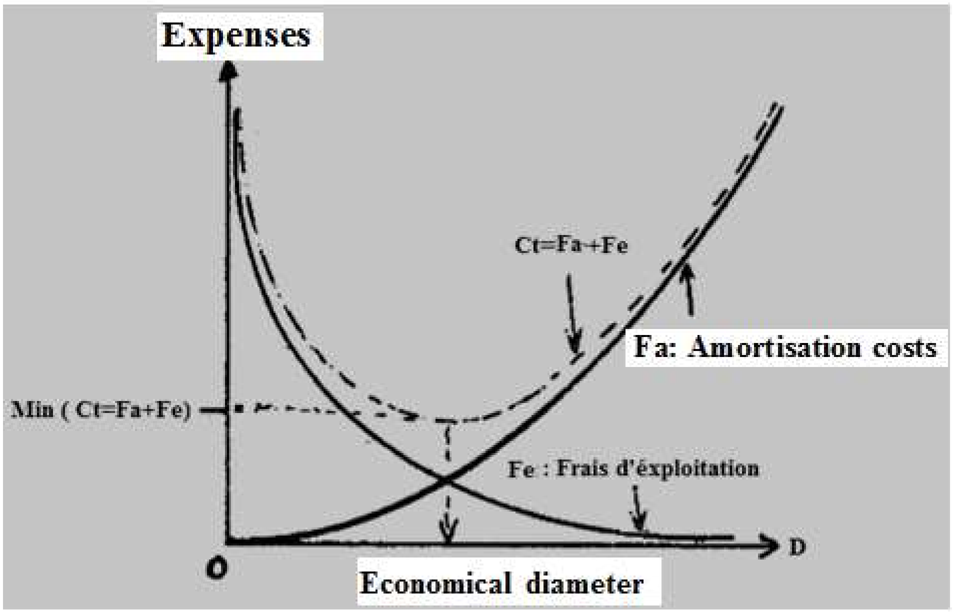


Figure 5.12: Economic diameter calculated following a compromise between depreciation costs and operating costs

The updated costs of purchasing and installing the pipe (Finv) increase as the pipe diameter increases. On the other hand, the operating costs of the pumping station (Fe) decrease as the diameter increases, due to the reduction in head loss. If a large diameter D is chosen, Fa is large and Fe is small. Conversely, if a small diameter is chosen, Fa is smaller but Fe is larger. The most economical or optimal diameter is then given by the minimum total expenses in parentheses Fa + Fe ($_{\text{Updated}}$).

$$D_{eco} = Min\ (Ct) = Min\ (Fe+Fa) \qquad (5.16)$$

a) Depreciation costs

The depreciation costs are given by:

$$Fa = F_{inv} \cdot a \qquad (5.17)$$

With :

Finv : Investment fees (DA)

a : Annuity (costs to be repaid annually to the Bank with an interest rate of i% over 30 years of repayment). The annuity, expressed by:

$$a = i + \frac{i}{(1+i)^n} \qquad (5.18)$$

i: Interest rate from 8 to 14%;

n : Number of years or duration of the project (n=30 years).

b) Operating costs

Operating costs are given by:

$$Fe= Ee \qquad (5.19)$$

E: Quality of energy consumed annually by the pump (KW/year);

$$E= 365.P.tp \qquad (5.20)$$

P: Pump power (KW);

$$P = \rho \frac{QHmt}{\eta} \qquad (5.21)$$

tp : Number of pumping hours (h);

η : Pump efficiency %;

Q: Discharge flow rate (m3/s);

Hmt : Total manometric height (m);

$$H_{mt} = {}_{Hg} + \Delta H\tau \qquad (5.22)$$

ΔH_{t} _: Total head loss along the pipe (m)

$$\Delta H_t = \Delta H_L + \Delta H_s \qquad (5.23)$$

ΔH_L : Linear head losses (m)

$$\Delta H_L = JL \qquad (5.24)$$

Or :

J is the unit gradient or hydraulic gradient which is expressed according to relations (5.3) or (5.7).

e : Price per KWh.

5.5.2. Application Example

Calculate the diameter of a discharge pipe based on the following data: Q = 97 l/ s, t= 24/24(h); Hg =90 m; L=1250 m, ε =0.1 mm, pump efficiency 70% and ν =0.000001 m ^{2} /s. (see Figure 5.13).

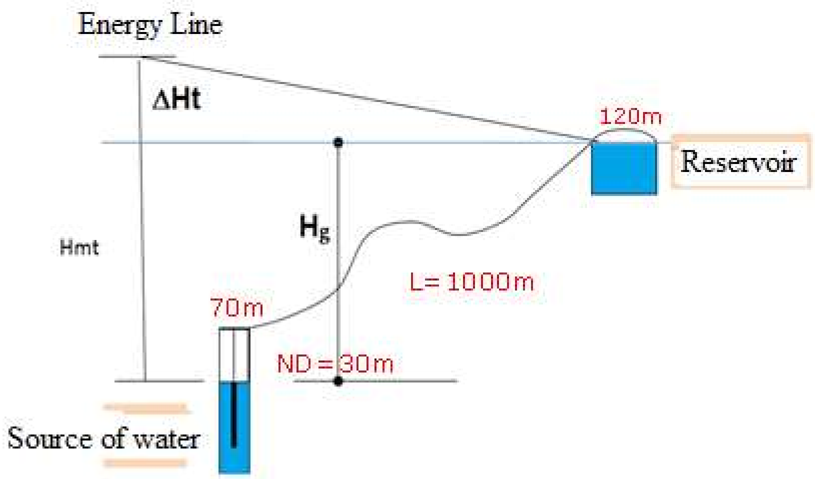


Figure 5.13: Adduction pipeline

Solution

The diameter of the discharge pipe is calculated either by application of empirical relationships or after an economic study.

According to Bonnin and Bresse, the diameter of this pipe varies from 300 mm to 500 mm. A series of diameters (250, 300, 350, 400 and 500) are selected to begin the economic study, which involves calculating the total cost for each diameter (depreciation costs plus operating costs). The economic diameter will be the diameter with the lowest cost, thus striking a balance between operating and depreciation costs. Table 5.2 summarises the results of the depreciation cost calculations.

Table 5.2: Calculation of depreciation costs.

D (mm)	Unit price (DA/ml)	L (m)	Finv (DA)	Fa (DA)
250	1082.4	1250	1353000	1.69E+09
300	2182.3	1250	2727875	242780.9
350	3182.3	1250	3977875	354030.9
400	3697.5	1250	4621875	411346.9
500	5614.02	1250	7017525	624559.7

The results of calculating operating costs are shown in Table 5.3

Table 5.3: Calculation of operating costs for: Hg= 90 m, η = 70%, e= 3DA/ Kw

D (mm)	*f*	D	J	ΔHt (m)	Hmt (m)	P (Kw)	E (KW/year)	Fe (DA)
250	0.0176	3E+05	0.014	17.52	107.5	146	1280334	3841002.4
300	0.0169	4E+05	0.0054	6,762	96.76	132	1152267	3456800.87
350	0.0172	3E+05	0.0025	3,184	93.18	127	1109653	3328960.26
400	0.0167	3E+05	0.0013	1,587	91.59	125	1090634	3271902.99
500	0.0168	2E+05	0.0004	0.522	90.52	123	1077950	3233849.65

According to table 5.4, the minimum cost is obtained for the diameter 400 mm, then the economic diameter is indeed 400 mm.

Table 5.4: Total cost for each diameter (Ct= Fa+Fe).

D (mm)	Fa (DA)	Fe (DA)	Ct= Fa+Fe (DA)
250	120417	2039494.84	2159911.84
300	242780.875	1670567.97	1913348.84
350	354030.875	1539853.43	**1893884.3**
400	411346.875	1485670.21	1897017.08
500	624559.725	1447610.93	2072170.66

So the diameter having a minimum total cost is the diameter 350 mm which we retain as the diameter for the discharge pipe.

Summary for calculating the discharge diameter according to direct formulas is shown in table 5.5.

Table 5.5: The discharge pipe diameter calculation using direct formulas

DN (m)	Calculated diameter (m)	Authors
350	0.311	**Bonnie**
500	0.467	**Bresse**
500	0.460	**Munier**
450	0.444	**Square 1973**
400	0.395	**Achour-Bedjaoui**

5.6- Protection of pipes against water hammer

Water hammer is a phenomenon of overpressure that occurs when there is a sudden change in the velocity of a liquid, as a result of the rapid closing/opening of a valve or tap, or the starting/stopping of a pump. Its consequences can lead to the rupture of the pipe (supply or distribution, etc.), see Figure 5.14.

Figure 5.14: Expansion joint destroyed by water hammer

This overpressure can be significant; it often results in a characteristic noise, and can cause the pipe to rupture in large installations due to the quantity of water in motion. This problem can be solved with the installation of a water hammer arrestor.

The water hammer is an oscillatory phenomenon involving pressure surges and depressions, caused by the following:

- The instantaneous closing of a valve located at the end of a supply pipe;
- The sudden stopping of a pump supplying a discharge pipe;

Water hammer can reach several times the operating pressure of the pipe and is likely to cause the pipe to rupture. Its effects must therefore be limited for reasons of economy and safety in the water supply.

A wave is then generated in the pipe, propagating at the speed of sound, the value of which depends on the compressibility of the water and the elasticity of the pipe.

Allievi gives the following value (m/s) for the speed of the wave:

$$a = \frac{9900}{\sqrt{48{,}3 + k\frac{D}{e}}} \tag{5.25}$$

In which :

a : velocity of the wave (m/s);

D: Diameter of the discharge pipe (m);

e : Thickness of the pipe wall (m);

K: Coefficient depends on pipe material: K=0.5 for Steel, K=1 for cast iron, K=5 for lead and Concrete and K=0.5 for Reinforced Concrete

Water hammer occurs in distribution network, supply pipe,… etc.

a) Water hammer in a gravity-feed (Case: After rapid closing of the valve)

Figure 5.15 presents the case of a main distribution pipe, which ends at its end with a closing valve.

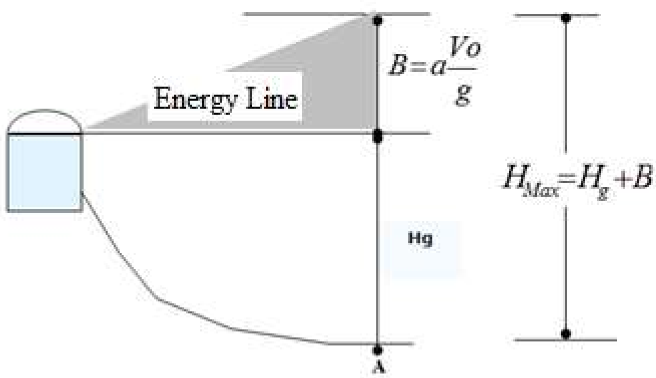


Figure 5.15: Main distribution line

As soon as the valve closes, a maximum overpressure (Hmax) occurs at the valve. The value of (Hmax) is expressed as follow:

$$H_{Max} = Hg + B = Hg + \frac{aV0}{g} \tag{5.26}$$

Or :

Hg: Geometric height (m)

B: Value of the water hammer(m.c.e)

a : velocity of the wave (m/s), which can reach 1000 m/s depending on the characteristics of the pipe;

v0: Average water velocity (m/s).

b) Water hammer in pressure supply pipe (After starting a pump)

When a pump is started, a depression therefore develops (a minimum overpressure) at the level of the pump and the pumping station, its value is expressed as follow (Figure 5.16):

$$H_{Min} = Hg - B = Hg - \frac{aV0}{g} \tag{5.27}$$

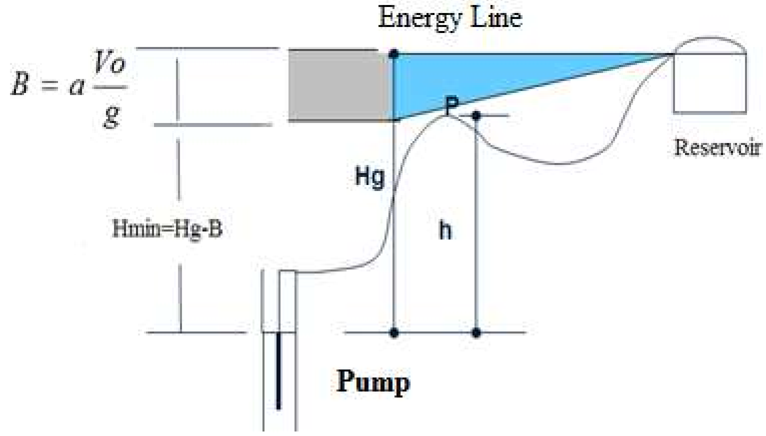


Figure 5.16: Case of pressure supply pipe

This minimum overpressure must be greater than zero, otherwise it is referred to as negative pressure, in which case the pipe and pump must be protected (using a water hammer arrestor). There is no risk for point P. For this type of profile, the pipe must be completely protected against the effects of negative pressure (Figure 5.17).

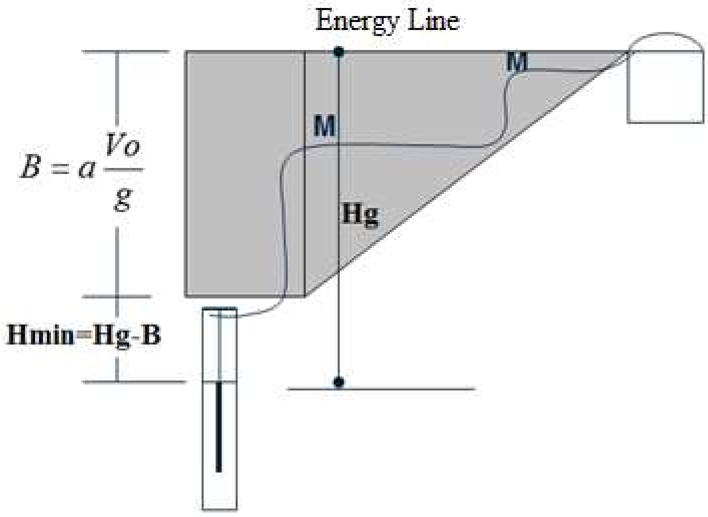


Figure 5.17: Particular profile 1

For this type of profile, the pipe must be completely protected against the effects of water hammer.

Noticed :

For steel pipes, it is necessary to check that the relationship (5.28) is established.

$$e(mm) > 8D(m) \tag{5.28}$$

e : Thickness of the pipe wall (mm);

D: diameter of the discharge pipe (m).

For the profile in Figure 5.18 it is necessary to ensure that:

$$h - Hmin = H - (Hg - B) > 8(m) \tag{5.29}$$

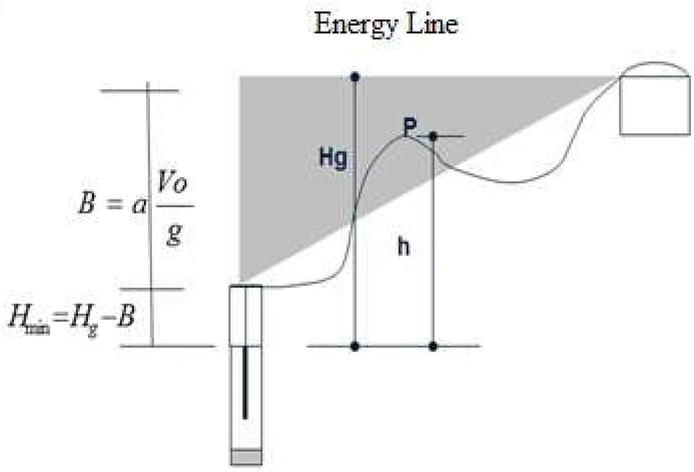


Figure 5.18: Particular profile 2

5.7 Anti-water hammer devices (or water hammer arrestor)

Among the best known anti-rams:

1- Air tanks;

2- Balance chimneys;

3- Flywheels.

5.7.1. Air tanks

A compressed air tank is a closed tank whose upper part contains pressurised air and whose lower part contains a certain volume of water. Thus, when the pumps stop (for example), the tank decompresses and supplies water to the pipe, reducing the pressure drop caused by water hammer. When the direction of flow is reversed, the air in the tank is compressed, allowing a volume of water to be stored (Figure 5.19).

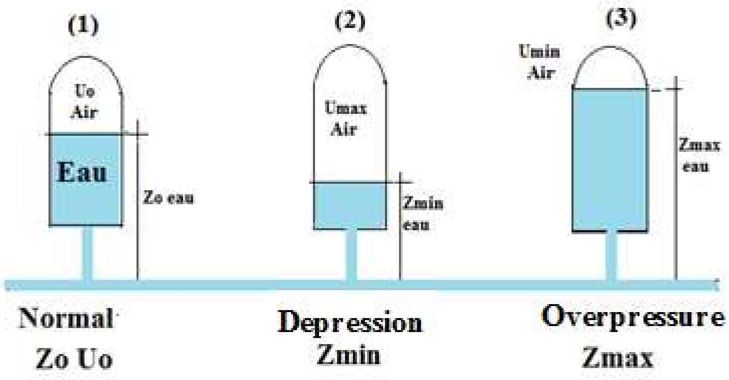


Figure 5.19: Operating phases of an air tank

The air tank operates as follow:

(1) - Normal pump operation, characterized by a volume of air (Uo) and a normal pressure (Zo)

(2) – When the pump starts up, this is characterized by a maximum volume of air in the tank (Umax) and a minimum pressure which can be a depression (Zmin)

(3)- Case of a sudden stop of the pump, this is characterized by a minimum volume of air in the tank (Umin) and a high water pressure (Zmax)

To calculate an air tank it is necessary to use **chart or abacus** of **A. Vibert** (Figure 5.20) in order to determine the different volumes of air (Uo , Umax and Umin).

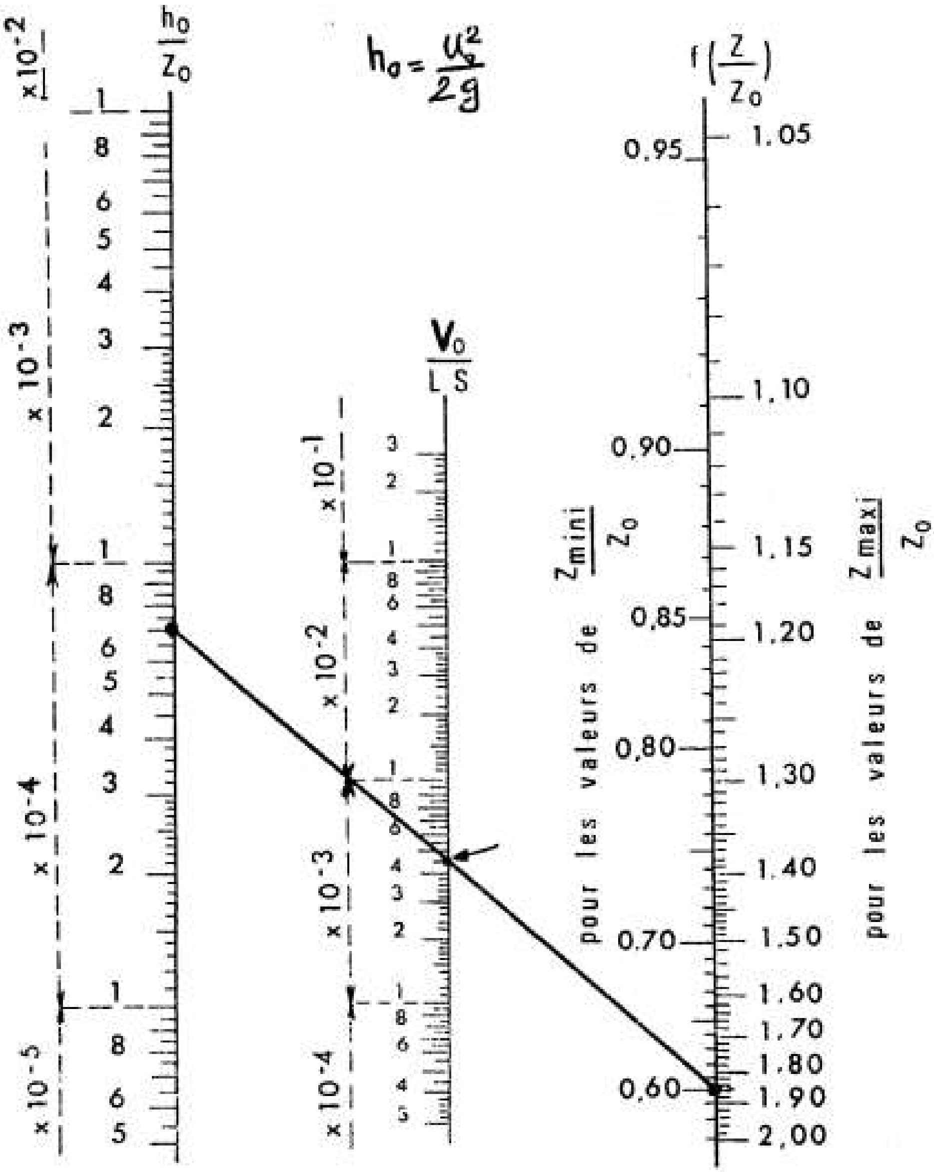


Figure 5.20: Abacus of Vibert (Dupont, 1979)

The relation (5.30) is used to calculate the volume of air in the air tank during normal operation of the installation.

$$Uo = \frac{Vo^2}{2gZo}\frac{LS}{f(\frac{Z}{Zo})} \tag{5.30}$$

Uo : Air volume (m^3);

Vo: Average velocity of the water in the discharge pipe (m/s);

L: Length of the discharge pipe (m);

S: section of the discharge pipe (m^2);

f (Z/ Zo): function determined from the Viber's abacus and expressed as follow:

$$f\left(\frac{Z}{Zo}\right) = \left[\frac{Zo}{Zmin} - 1 - Log(\frac{Zo}{Zmin})\right] \tag{5.31}$$

- Calculate the height ho:

$$ho = \frac{Vo^2}{2g} \tag{5.32}$$

- The intersection of the line between ho and $f\,(\frac{Z}{Zo})$ makes it possible to determine the value of Uo /LS and Zmin / Zo .

$$\frac{Uo}{LS} = \frac{ho}{Zo}\frac{1}{f\left(\frac{Z}{Zo}\right)} \tag{5.33}$$

Checking for water hammer occurrence

To check for the occurrence of water hammer and see if it is necessary to protect the pipe or the pump, follow these steps:

- **Calculate** the velocity (a) of the wave according to the Allievi relation **(** 5.25):

$$a = \frac{9900}{\sqrt{48{,}3+e\frac{D}{e}}} \tag{5.34}$$

- **Calculate** the value of water hammer according to the **Michaud's relationship:**

$$B = \frac{aVo}{g} \tag{5.35}$$

- **Calculate** the maximum overpressure (Hmax) at the pump during a sudden stop of the latter:

$$H_{Max} = Hg + B = Hg + \frac{aV0}{g} \tag{5.36}$$

- **Calculate** the underpressure or minimum pressure (Hmin) and check that it is not negative (vacuum) when starting the pump.

$$H_{Min} = Hg - B = Hg - \frac{aV0}{g} \tag{5.36}$$

5.7.2. Application example

Check for water hammer for the following data: Q=0.031m3/s, D=0.200 m, e=0.01mm, L = 1200 m, Vo=1 m/s, Hg = 60 m, K =0.5 for cast iron pipe.

Solution

To check for water hammer and see if it is necessary to protect the pipe or the pump , follow these steps:

- Calculation of a wave velocity:

$$a = \frac{9900}{\sqrt{48{,}3 + K\frac{D}{e}}} = \frac{9900}{\sqrt{48{,}3 + 0{,}5\frac{0{,}2}{0{,}01}}} = 1200\ m/S$$

- Calculation of the value of water hammer B:

$$B = \frac{aVo}{g} = \frac{1200.1}{9{,}81} = 122\ m.c.e$$

- Calculation of the maximum overpressure (Hmax):

$$H_{Max} = Hg + B = 60 + 122 = 182\ m.c.e = 18{,}2\ bars$$

The cast iron pipe can withstand this excess pressure because the nominal pressure of the pipe is 40 bar

- Calculation of underpressure (Hmin):

$$H_{Min} = Hg - B = 60 - 122 = -62\ m.c.e\ < 0$$

As Hmin is negative, the pipe is subject to negative pressure, which poses a risk to the pump's operation. It must therefore be protected using an air tank

If we want to limit the pressure in the pipe to 12 bars, that is to say 120 mce instead of 182 mce , then:

Zo = 60+10 = 70 m

Zmax = 120+10 = 130 m

Zo / Zmax = 70/130 = 1.85

ho=$Vo^2/2g=1^2/2.9.81=0.051$

ho/ Zo =0.051/70=0.0007

From the diagram of **Vibert** and for Zo / Zmax =1.85 and ho/ Zo =0.0007 we obtain:

The alignments 1.85 read on the Zmax / Zo scale and 0.0007 read on the ho/ Zo scale give on the abacus:

Uo /LS=0.0045 which implies that Uo =0.0045/ LS, and as LS = 39 m^2 , therefore: Uo =0.171m^3 =171 Liters

And Zmin / Zo =0.6

If we assume that Uo.Zo = Umax.Zmin we derive that Umax = Uo / Zmin / Zo = 0.171/0.60= 0.285 m ^{3} = 285 Liters.

The maximum air volume is: 285 Liters

To ensure that there is still water left when the air reaches its maximum volume, we take the total capacity of the tank: Vres=1.3 Umaks, therefore: the recommended air tank is a 370-litre tank.

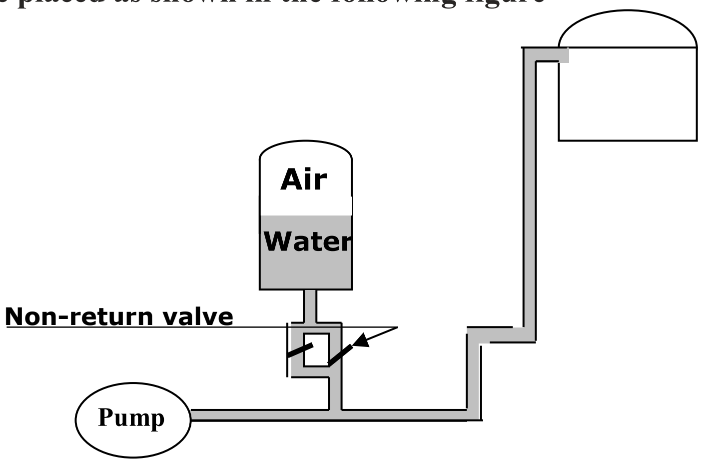
Calculation of Zmin

Zmin / Zo =0.60 $\longrightarrow$ **Zmin =Zo.0.6 = 70.0.6 = 42 mce**

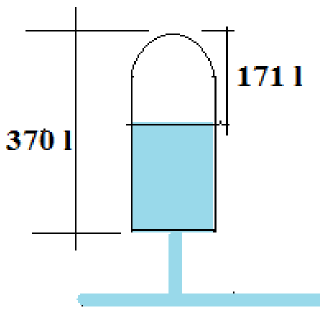
The pressure becomes:

Hmin = Zo-Zmin = 70-42 = 28 mce

The air tank will be placed as shown in the following figure



Air Tank Location



5.7.3 Balance chimneys

There are other water hammer arrestors, such as balance chimneys. and flywheels as shown in Figure (5.21).

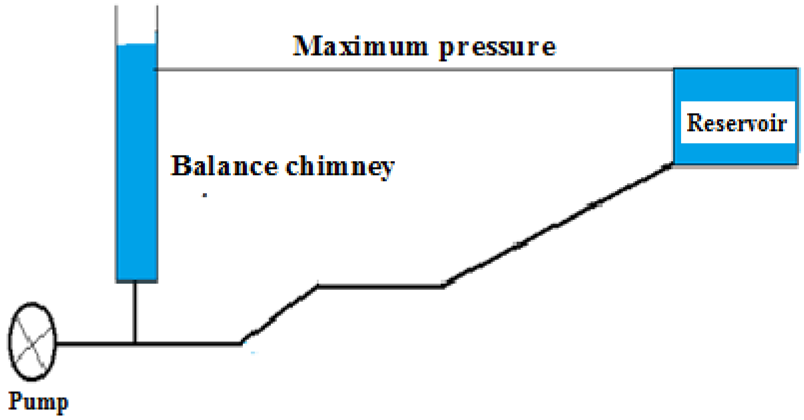


Figure 5.21: location of balance chimney

When topographical conditions and geometric heights allow, it may be possible to introduce a balancing chimney, consisting of a reservoir in contact with the free surface. This device reduces the effects of water hammer. However, another mass oscillation phenomenon, of a completely different nature, occurs between the chimney and the reservoir (Figure 5.22).

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Figure 5.22: A balance chimney

5.7.4. Flywheels .

The use of a flywheel mounted on the shaft of the electric pump unit can increase the downtime. Economically, this solution is only viable for discharge pipes of a few hundred meters: larger pipes would require hand wheels of exaggerated size or would involve excessive current demands during start-up phases (Figure 5.23).

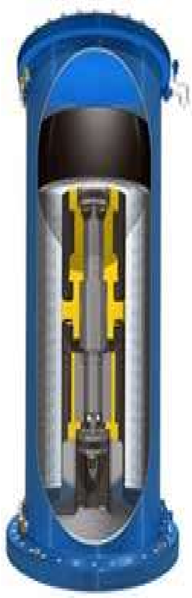

Figure 5.23: flywheel

5.7.5. Evacuation of air pockets

When filling the line, the air evacuates through the two openings of the suction cup until the rising water raises the balls which thus block the air inlets.
The ball periodically opens and closes the drain holes during service.
The float of the large evacuation hole cannot descend and thus remains closed, its weight being less than the pressure.
When the pipe empties, the two floats fall and allow air to enter through the two holes which have become free.

Figure 5.24: Air valve

5.8. Conclusion

In this chapter, a very important work in the drinking water supply system has been treated, that of water conveyance. There are two types of adduction (gravity and pressure).

The calculation of pressure supply pipes is based on a technical-economic study. In fact, we do not know the conditions under which the empirical formulas proposed for the calculation of the diameter in the case of delivery by discharge were developed, but certainly this diameter must be the subject of a technical-economical study where we choose the diameter having the least cost.

Even if the technical-economic calculation shows a diameter with the least cost, it is still necessary to check the average flow speed. If this speed is high then to avoid the effects of water hammer we choose a diameter greater than that obtained by the calculations.

Sometimes the economic diameter is not available on the national market so it is better to take a diameter greater than the diameter found after economic study

Sometimes the annual power is low compared to the investment costs so the economic study notes a relatively small diameter which generates a significant average flow speed hence the possibility of water hammer appearing in the case of a sudden closing of the Valves or during a sudden stopping of the pumps and to avoid the problems which can be caused by this phenomenon it is recommended to take a larger diameter to reduce the speed of the flow.

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