



MEMOIRE DE FIN D'ETUDES

Pour l'obtention du diplôme d'Ingénieur d'Etat en Hydraulique

Option: ALIMENTATION EN EAU POTABLE

THEME :

**STUDY OF THE DRINKING WATER SUPPLY TO THE
VILLAGES (ELDRIAT,BOURACHED) IN HAMMAM
DALAA MUNICIPALITY (S. M'SILA) FROM A BUFFER
TANK (2000 M3)**

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Session Novembre 2024



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**ETUDE DE L'ALIMENTATION EN EAU POTABLE DES
VILLAGES (ELDRIAT,BOURACHED) DE LA COMMUNE
DE HAMMAM DALAA (W.M'SILA) A PARTIR D'UN
RESERVOIR TAMPON (2000 M3)**

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Acknolegement :

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SAMAH OLAYA

Dedication :

To the soul of my bereaved, to my pure angle ,Mum Cherifa may God have mercy on her

To my back bone in life, my dear parents Rabeh and Salima

To my mother Zouina, the pillar of our family, to my little aunts Huda and Louisa

To my illuminating stars that had and will always shine leading me to,my dears Rebaiha and Fouzia

To my pearls,my sistres Sarah ,Romaissa, Djouhaina,Soundos, Loudjaine and Djilane Roudaina

To my little prince Taha Iyass

To my grandmothers Elkhamsa and Zahra

To my dear uncles Rizki, Taher and Kassi and their children (Malek, Iyad, Rimas, Ibrahim and Khalil Elrahemane)

To my beautiful roses ,my friends Sabrin ,Romaissa ,Mounia , houda and Lara

To all of my familly and to all those who belive in me

SAMAH OLAYA

ملخص

تتطلب أنظمة الإمداد بمياه الشرب شبكات أنابيب ذات أبعاد صحيحة لضمان التشغيل المستمر تتعلق دراستنا بربط منطقتي بوراشد والدريعات الواقعتين في حمام الضلعة لولاية المسيلة ، من خلال تصميم نظام إمداد مياه الشرب لتلبية احتياجات هاتين البلديتين من الخزان المؤقت الموجود ونظام نقل مختلط من الخزان المؤقت إلى برج توزيع المياه الموجود .كما ذكرنا أيضا بعض طرق حماية أنابيب التوصيل وعلى وجه الخصوص ظاهرة المطرقة المائية وأخيرا ناقشنا منهجية تنفيذ المشروع و الموارد المختلفة المستخدمة لتنفيذه.

الكلمات المفتاحية : أنظمة الإمداد بمياه الشرب ، نظام مختلط ، الخزان المؤقت ، برج توزيع المياه ، المطرقة المائية
بلدية حمام الضلعة (المسيلة)

Résumé :

Les systèmes d'alimentation en eau potable nécessitent des réseaux de conduites correctement dimensionnés afin de permettre un fonctionnement continu de ces derniers.

Notre étude porte sur le raccordement des zones de Bourached et Driat ,située dans la commune de Hammam Dalaa wilaya de M'sila, par concevoir un système d'alimentation en eau potable pour répondre aux besoins des ces deux villes à partir le reservoir tampon existant et un système d'adduction mixte du réservoir tampon vers le château d'eau de distribution existant .

Nous avons abordé aussi des moyens de protection des conduites de refoulement, en particulier, contre un phénomène transitoire dit : Coup de bélier.En fin dans notre travail, nous abordé la méthodologie de mise en œuvre du projet etles différents moyens déployés pour son exécution.

Mots clés : systèmes d'alimentation en eau potable, système d'adduction mixte ,réservoir tampon , château d'eau de distrubution ,Commune de Hammam dalaa (M'slia)

Abstract :

Drinking water supply systems require correctly dimensioned pipe networks in order to ensure continuous operation.

Our study concerns the connection of the Bourached and Driat areas, located in the municipality of Hammam Dalaa in the province of M'sila, by designing a drinking water supply system to meet the needs of these two villages from the existing buffer tank and a mixed supply system from the buffer tank to the existing distribution water tower.

We also looked at ways of protecting the delivery pipes, in particular against a transient phenomenon known as water hammer. Finally, we discuss the methodology for implementing the project and the various resources deployed for its execution.

Key words: drinking water supply systems, mixed supply system, buffer tank, distribution water tower, munisipilaty of Hammam dalaa (M'sila)

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Abbreviation :

WRD : Water Resources Department .

UTM :Universal Transversre Mercator .

NAHR: National Agency for Hydraulic Resources .

WS: Weather Station .

PRA: Paraseismic Rules Algerian .

NCAREE: National Centre for Applied Research in Earthquake Engineering .

NADT: National Agency for Dams and Transfers .

UDP: Urban Development Plan .

SDS: special delegation status .

NWP: National Water Plan .

DWS: Drinking Water Supply .

HDPE: High Density Polyethylene .

TMH :Total Manometric Head .

ND: Nominal Diameter .

GE: Ground elevation .

IE : Invert elevation .

General introduction :

The development of the water sector is an essential expense for the development of a region in all areas, since the water sector is linked to all branches of the economy. Water is a vital element and the main means of public hygiene, but if it is poorly treated or polluted, it becomes a dangerous agent for the spread of epidemics, which is why this resource is so useful in most areas of human activity.

The lack of drinking water has become a daily concern for Algerians, and this is the background to our study, which involves investigating the drinking water supply to the municipality of Hammam dala, in the wilaya of M'sila, from a buffer reservoir (with a capacity of 2000 m³) to comfortably meet demand in 2054. Our study comprises eight chapters:

We begin in the first chapter with a presentation of the study area (the geographical, topographical, seismic, climatological, hydraulic and urban situation). In the second chapter, we proceed to estimate the population and the needs of each category of the agglomeration in order to calculate the water requirements for 2024 and 2054, with the aim of drawing up a balance sheet that meets current and long-term needs. In the third chapter we presented the main characteristics of the existing buffer reservoir and we also verified the capacity of the existing cylindrical water tower, and the fourth chapter will be devoted to the sizing of the adduction network .

Then in the next chapter, the projected pumping stations and the choice of pumps, The sixth chapter we will be interested in the sizing of the distribution network pipes taking into account the technical-economic factors . In chapter seven, we will study the laying of pipes and the protection system needed to prevent water hammer, and we will conclude our work with a study of the organisation of site work, taking into account safety during its execution, and draw up an estimate and quantity of the project for effective planning, which will be the eighth and final chapter of our project.

CHAPTER I:

Presentation of the study area

Introduction:

In order to meet the water needs of the two villages of Dreat and Bourached in the municipality of Hammam Dalaa, it is necessary to present the project site as well as the context of implementation. Therefore, it is important to know the site and its limitations, namely its geographical location, topography, climate, and current water status. This chapter will serve as the basis for the rest of this work.

I.1.Presentation of the study area :**I.1.1.Geographical location:**

The province of M'sila, within its current boundaries, occupies a privileged position in the central part of northern Algeria as a whole. It is part of the high plateau region of the centre and covers an area of 18,175 km² (*Annuaire statistique 2022 ,M'sila*). It is located at 35°40' north latitude and 04°30' east longitude, at an altitude of approximately 441m.

The state of M'sila is located in the semi-arid zone where rainfall is less than 250 mm/year. It is bordered:

In the northeast by Bordj Bou-Argeridj and Setif,

To the northwest by Medea and Bouira,

To the east by the state of Batna,

To the west by the state of Djelfa, and to the southeast by the state of Biskra

The municipality of Hammam Dalaa is geographically located between the Tell Atlas range in the north and the Hodna mountain range to the east. The geographical coordinates of Hammam Dalaa are 35°55'41" North, 4°22'28" East, and between the longitudinal lines (4°56'- 5°33') and latitude (22°- 34°). It is located in the northwest of the province of M'sila, 30 km away. It covers an area of 341 km² (*Annuaire statistique 2022, M'sila*). It is bordered:

To the north: the state of Bordj Bou Argeridj

To the southeast: district of M'sila

To the south: the district of Ouled Madhi

And to the west: the district of Ouanough

The villages of Dreat and Bourached are located north of the district of Hammam Dalaï, approximately 22 km away. at a high altitude in the mountains, on the southern flank of Djebel Maâdid. They are bordered to the north by the canton of Ogribissa and to the east by Douar .

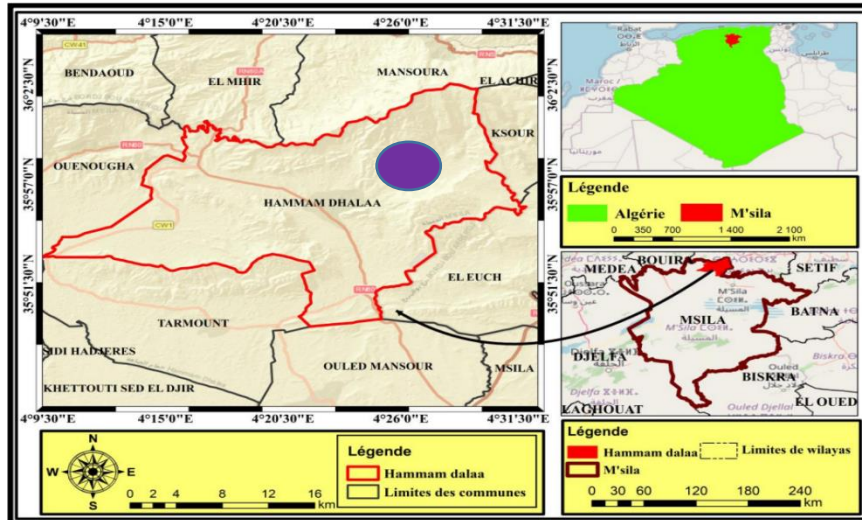


Figure I.1: Map representing the districts of Hammam dalaï municipality.

Source : (DHW) of M'sila ,2023 .

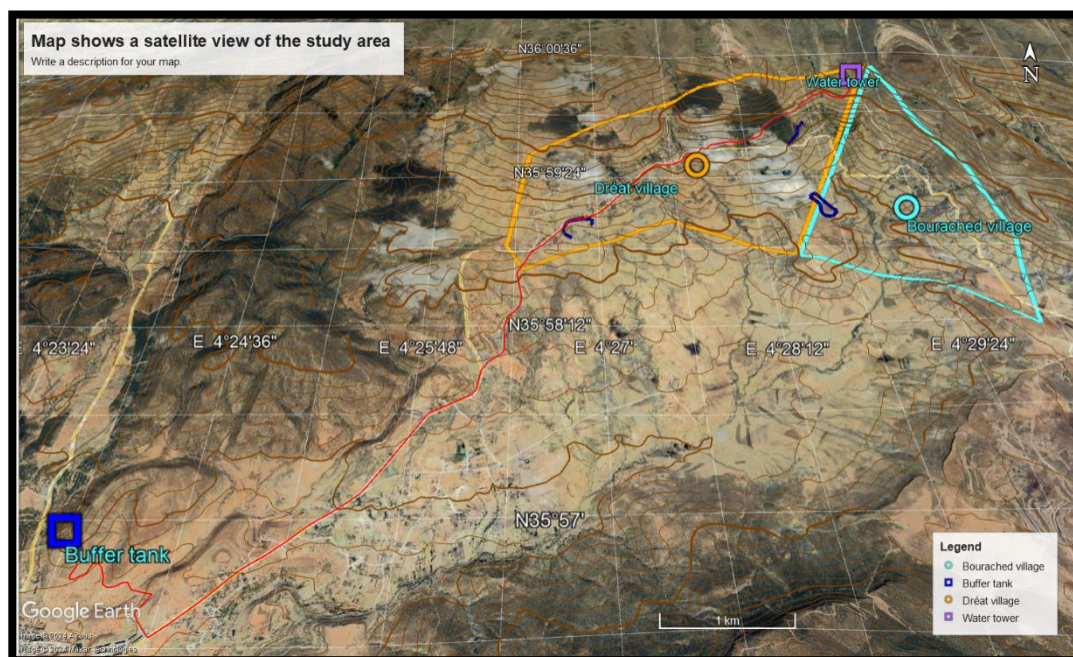


Figure I.2: Satellite view of the study area .

Source:(Google Earth 29-04-2024 /11:14 pm) .

I.1.2.Topographical Situation:

The coordinates of the Bourached and dreat , (GPS), are :

Bourached village

X= 4°29'46.55" E

Y= 35°59'15.70" N

Z=1129 m

Dreat village

X= 4°27'48.02" E

Y=35°59'13.35" N

Z=1318 m

I.1.2.1.Relief:

The territory of the province of M'sila constitutes a pivotal and transitional zone between the two major mountain ranges, the Tell Atlas and the Saharan Atlas.

The geographical configuration is as follows:

A mountainous area on either side of Chott El Hodna.

A central area mainly composed of plains and high plateaus.

An area of salt flats and depressions with Chott El Hodna in its centre.

An area of wind-blown sand dunes.

I.1.2.2.Geology:

The geology of the Chott El Hodna basin is quite complex due to its position at the contact between the Tell Atlas and the Saharan Atlas.

The facies of the different formations show frequent variations, and the superposition of several tectonic styles is observed.

Contrary to the marine series, the continental deposits of the Quaternary do not form continuous levels, but conglomeratic, sandstone, or sandy lenses.

The most important characteristic is the predominant influence of cyclic climatic variations on the nature of the deposits, which are related to their geological origin.

I.2.Climate Situation :

M'sila Located in a semiarid zone, our study area is subject to a very severe and complex environment.

climate regime. It serves as a transition zone between areas influenced by the Mediterranean climate and those influenced by the continental subtropical desert climate.

The study of climatic factors in the area concerned is based on data from the M'sila code station (60467) for the period 2001-2021, which has similar characteristics to those of the study region.

Table I.1: Characteristics of M'sila station :

STATION	Longitude	Latitude	Elevation
M'sila code 60467	4,50°E	35,67°N	441 m

I.2.1. Rainfull:

The annual rainfall ranges between 175 and 250 mm, with a very significant irregularity in its distribution.

Table I.2: Average monthly rainfull (2021).

Month	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Annual
P(mm)	13.76	24.44	26.77	22.09	16.94	18.68	18.41	25.20	29.55	14.18	5.06	6.51	221.59

Source:(NAHR) 2021

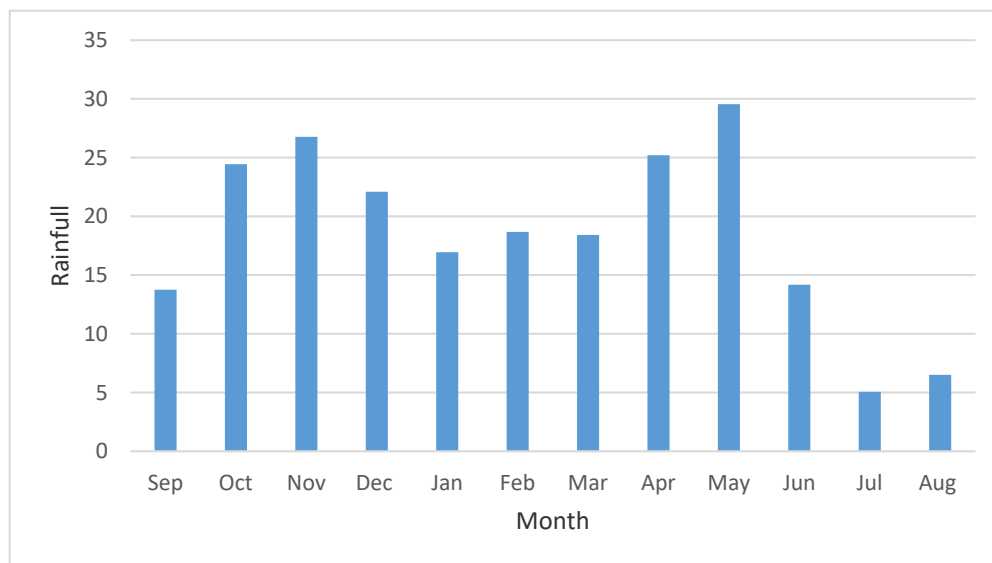


Figure I.3: Average monthly rainfull (M'sila station ,2021).

I.2.2.Relative Humidity :

The relative humidity in M'sila reaches its maximum in January with 93% and its minimum in July with 41%.

Table I.3:Monthly averages of relative humidity(%)in M'sila for the Period (2019-2021).

Month	jan	fab	mar	apr	mai	jun	jul	aug	sep	oct	nov	dec
RelativeHumidity (%)	93	59.6	53.6	53.3	50.1	47.8	41	49.2	57.3	63.9	69.5	71.3

Source:(NAHR) 2021

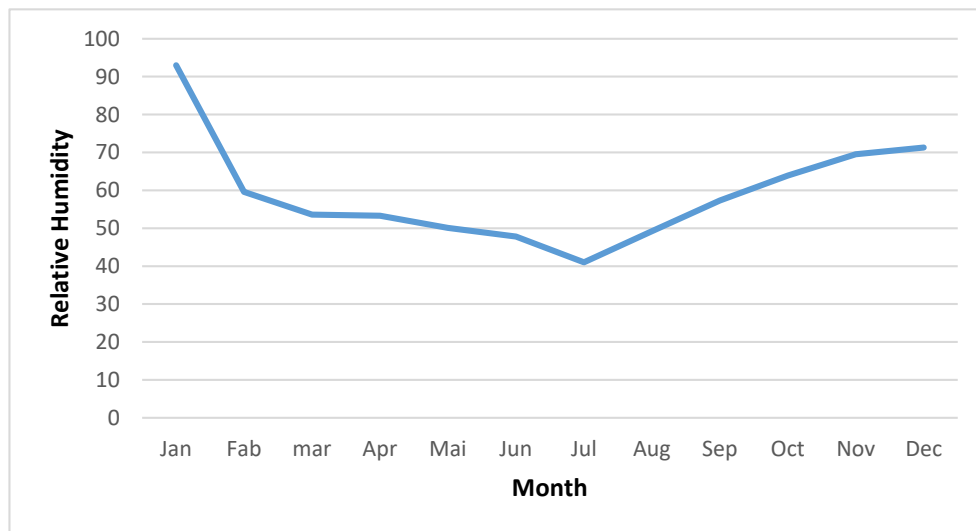


Figure I.4:Monthly averages of relative humidity (%) (M'sila station,2021).

I.2.3. Temperature :

The average monthly temperatures recorded at the station in the M'sila region are shown in the table .

Table I.4: Average monthly temperatures at the M'sila weather station. (2021).

The average annual temperature is 18.55° C. Winter temperatures are around 3.4°C and rise to almost 38.2°C in the summer .

Mois	J	F	M	A	M	J	J	A	S	O	N	D
Max T C°	12.7	14.2	19.1	23.4	28.3	34.4	38.2	37	30.8	25.2	17.3	13.2
Min T C°	3.4	3.8	7.2	10.7	14.8	19.8	23.3	23	18.9	14.4	8.2	4.4
Avg T C°	7.7	8.8	13.1	17.1	21.8	27.5	31.2	30.2	24.8	19.6	12.4	8.4

Source: (NAHR) 2021

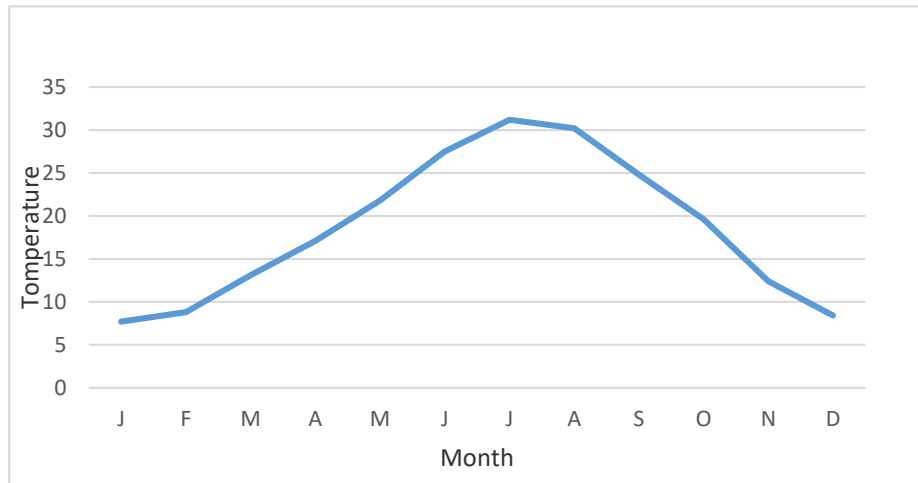


Figure I.5: Average monthly temperatures (M'sila station, 2021).

I.2.4 .Winds :

The region is generally dominated by westerly and northwest winds in the winter season, which are laden with rain and snow due to their source. Another type of prevailing wind is the southwesterly wind, as well as the Sirocco winds in the summer season, which are laden with dust and hot.

Table I.5 : Average monthly wind speed (m/s) at the M'sila station (2021).

Month	J	F	M	A	M	J	J	A	S	O	N	D
Wind speed (m/s)	3.81	4.64	5.0	4.82	4.81	4.55	4	3.73	3.72	3.55	4.0	3.64

Source: (NAHR) 2021

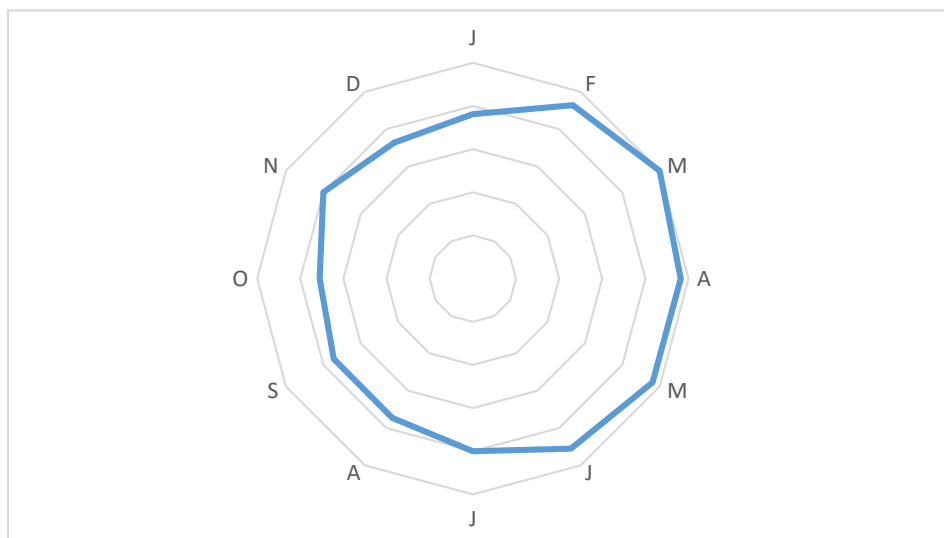


Figure I.6: Average monthly wind speed (M'sila station,2021)

According to Table I.5 the minimum wind speed is recorded during the months of September and December, reaching 3.55 m/s . The maximum wind speed is recorded in June and reaches 4.82 m/s . The wind blows in different directions.

Winds Dominated by westerly and northwest winds in winter , southwesterly and Sirocco winds in summer

I.3. Insolation :

Insolation duration characterizes the time interval, which varies with the season, between sunrise and sunset. In the region of M'sila, it varies irregularly throughout the year, reaching its maximum in July with an average daily duration of 11.12 hours and decreasing to its minimum in December, where the duration is 5.58 hours per day. The daily average is 8.5 hours.

Table I.6:The average monthly and annual duration of sunshine 2005 to 2021

Month	J	F	M	A	M	J	J	A	S	O	N	D
Insolation(h/d)	6.06	7.97	8.25	8.96	10.09	10.68	11.12	10.04	8.81	8.01	6.54	5.58

Source : NAHR of M'sila2021.

I.4. Seismicity :

According to the Algerian seismic laws (PRA 1993 Version 2003), the national territory is divided into five seismic zones , (See seismic zoning map below)

- Zone 0: Negligible seismicity .
- Zone 1: Low seismicity .
- Zone 2 and 3: Medium seismicity .
- Zone 4: High seismicity .

The province of M'SILA is located in zone one (1), which is an area of low seismicity. M'SILA does not experience major earthquakes. However, The province contains a group of faults in the eastern and central regions.

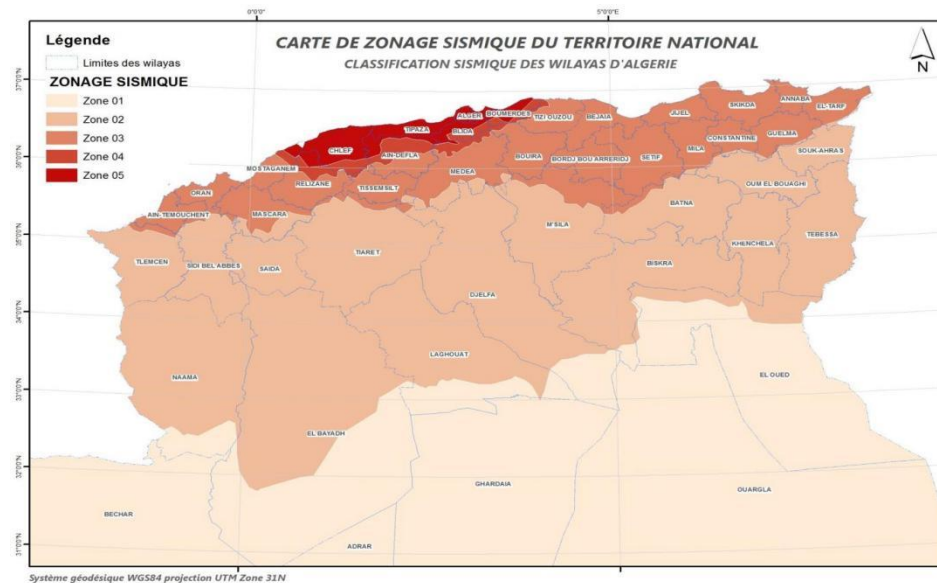


Figure I.7: Seismic Zoning Map of the National Territory

Source : (NCAREE, 2017)

I.5. Urban Planning Situation :

The villages of Dreat and Bourached are characterised by scattered rural dwellings with agricultural purpose, where buildings do not exceed one floor above ground level (R+1).

I.6. Hydraulic situation:

The villages of Dreat and Bourached are located to the north of the main town of Hammam Dalaa. They cover an area of 974 ha. The two towns will be supplied with drinking water from an existing buffer reservoir with a volume of 2,000 m³ and a ground level of 884 m. It is located in Elhourane and is supplied from the Tilesdit dam by a treatment station.

According to the information we have received, the reservoir is in good condition (according to the WRD).

- **Tilesdit Dam :**

The TILSEDIT dam is an embankment dam located in the Eddous valley , at the outlet of a catchment area of 843 km², with a regularizable annual contribution of 83hm³. The TILESDIT dam is located in the middle of northern Algeria, 18 km east of the town (village) of Bouira and 4 km from the national road linking Algiers and Constantine. The TILESDIT dam is located in an agricultural region.

The capacity of this dam regulates the flow of the Eddous valley to supply drinking water to the population of the wilaya of Bouira (695,583 inhabitants), as well as the Sidi Khaled and El Hachimia industrial zones. The reservoir is also used to irrigate the El-Esnam plateau (2,200 ha) and the Sahel valley (3,400 ha). Some of the surrounding wilayas, such as Bordj Bou Arreridj and M'Sila, are also served.

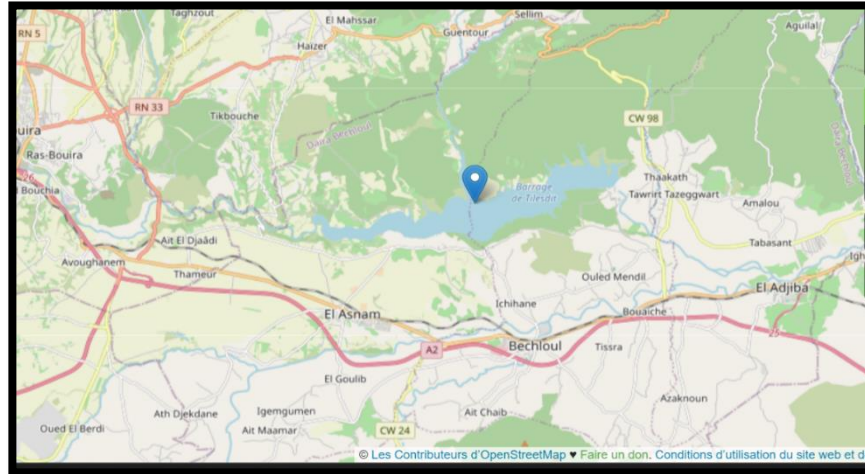


Figure I.8: TELSDIT dam situation.

Source : (Open Street Map , 30-04-2024 /18:06 pm)

Table I.7: Main Characteristics of the dam .

Dam type	Height (m)	Capacity (m3) million	Spade length (m)	Within in Crete (m)	Watercourses	Geographical coordinates
Embankment	65	167 million	425	10	Eddous valley	X= 4° 03' 15" E Y= 36° 21' 10" N

Source : National Agency for Dams and Transfers (NADT)

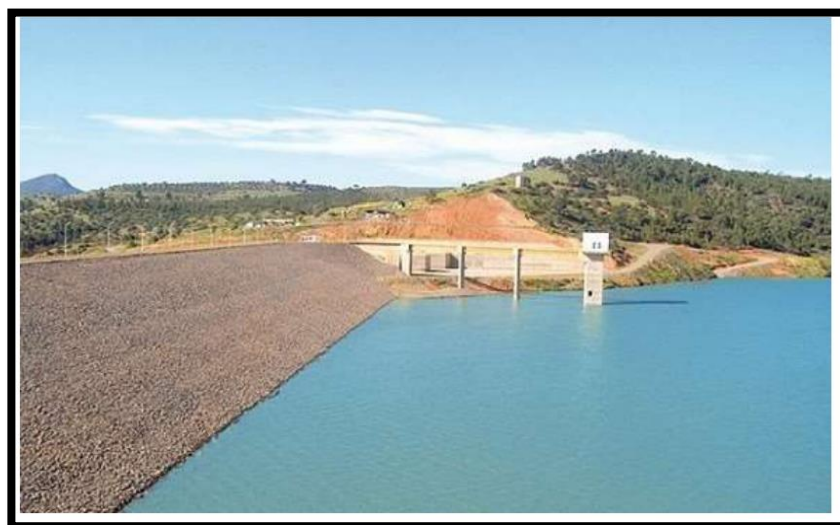


Figure I.9: View of the Tilesdit dam .

Source: National Agency for Dams and Transfers (NADT)

The study area lacks a drinking water supply network. The inhabitants obtain water from a few wells that generally run dry in the summer. It also lacks a sewerage network, with all households relying on septic tanks or discharged wastewater into nearby ravines (chaaba).

Conclusion:

In this chapter, we have represented the topography, climate, geology and hydraulic situation of the study area.

The M'sila region, located in northern central Algeria, is characterised by limestone and clay geological structures, with shrub and herbaceous vegetation. The municipality of Hammam Dalaa , located in the south of the province of M'sila, is characterised by geological structures consisting mainly of sedimentary formations and mountains. The vegetation consists mainly of steppes and scrubland. The water sources come from the Jebel massifs (mountains) and the groundwater in the catchment area, but the quaternary alluvium offers few water resources. The semi-arid Mediterranean climate has average annual temperatures of 15 to 20°C and concentrated rainfall of 300 to 400 mm. The average relative humidity is 60%, with annual evaporation of 1,500 to 2,000 mm.

This different information represents the basic data for the development of our work, which is the dimensioning of the supply and distribution network to supply the two villages of Dreat and Bourached.

CHAPTER II:
Water Needs Assessment

Introduction:

This chapter will address the assessment of the water needs of the two villages of Dreat and Bourached by the year 2054. For this, it is imperative to take into account the increase in consumption, which is mainly linked to population growth, industrialization, and the improvement of the general living environment.

Before the start of any project, it is essential to carry out a census of all categories of consumers, which will be the subject of this chapter.

The information provided was supplied by the municipality of Hammam dala (Census Department).

II.1: Population Evaluation :

The demographic evolution in Algeria follows the law of geometric increases given by the relationship of compound interests that takes into account births, the growth rate, and the mortality rate, namely:

$$P_n = P_0 (1 + \tau)^n \quad (II.1)$$

Where:

τ : Growth rate considered in %

n : the number of years separating the reference year and the projection year.

P_n : population estimate at the study horizon.

P_0 : Population corresponding to the reference year.

According to information provided by the UDP (Urban Development Plan) , the demographic growth rate of the study area (the villages of Dreat and Bourached) is estimated at 2.5% .

Table II.1: Population Evaluation .

Village	Growth rate (%)	Actuale population (2024)	Short-term Population (2029)	Medium-term Population (2039)	Long term Population (2054)
Dreat	2.5	3439	891	4981	7214
Bourached	2.5	1100	1245	1593	2307

This figure shows population trends in the villages of Bourached and Dreat

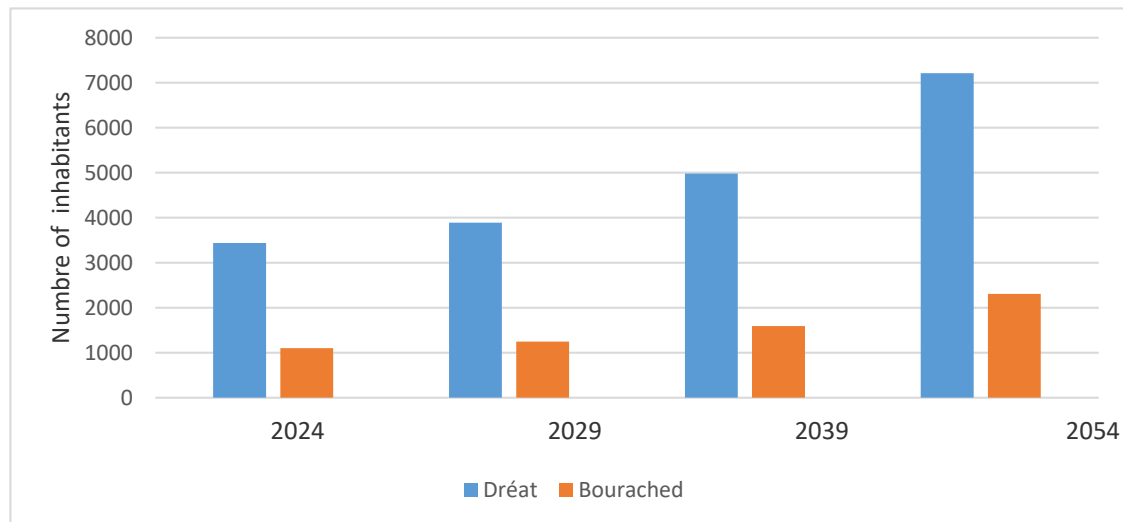


Figure II.1: Bourached and Dreat population trend graph (2024-2054) .

II.2.Categories needs :

In any study of a potable water supply system, various categories of needs are encountered:

- Domestic needs .
- Public needs .
- Sanitary needs .
- Industrial needs .
- School needs .
- Business Needs .
- Sociocultural and Sports Needs .

In the context of our study, no industry is located in the region, so the estimation of needs is based on the water endowment.

II.3. Estimating Needs:

II.3.1. Choice of the Unit Consumption Standard:

The quantity of water required for supplying an urban area is typically assessed based on the type of consumer, in liters per inhabitant per 24 hours, per square meter of plant surface, per cubic meter, per ton of productivity, per head of livestock, per vehicle, etc. This quantity of water is referred to as the consumption standard. It depends on certain criteria, with the main ones being:

- The standard of living of the population .
- The number of inhabitants.

- The urban development of the city.
- The availability of resources.
- The population's habits .

The estimated drinking water requirements given include domestic needs as well as all other types of consumption .

Table II.2: Typology of Agglomerations / Population Strata .

Typology of Agglomerations/ Population Strata	
Designation	Characteristic
Metropolises with a special delegation status (SDS)	Algiers ,Oran ,Constantine, Annaba
Metropolitan areas.	Cities with more than 300,000 inhabitants
The "superior" urban	100,000 < population < 300,000 inhabitants
The urban	20,000 < population < 100,000 inhabitants
Semi-urban	5,000 < population < 20,000 inhabitants
Semi-rural	3,000 < population < 5,000 inhabitants
Agglomerated Rural	600 (100 dwelling units) < pop. < 3,000 inhabitants
Sparse rural	Population of less than 600 inhabitants (100 housing units)

Source : (NWP,2010). The population of Bourached is 1,100, ranging from 600 to 3,000. The town is therefore of the agglomerated rural type .

Dreat has a population of 3,439, ranging from 3,000 to 5,000. The village is therefore of the semi-rural type.

The method is to calculate the gross endowment from the net endowment and its yield using the trend hypothesis .

The table below shows the Proposed Endowments for the MRE Workshop on April 26, 2009.

Table II.3: Proposed allocations from the MRE workshop on 26 April 2009

Domestic Unit Endowments and

"Other Equivalent Uses" in		Trend Hypothesis				
l/day/capita						
HORIZON		2010	2015	2020	2025	2030
Metropolitan areas (SPE)		100	105	110	115	120
Metropolitan areas		90	95	100	110	120
Upper (higher) urban		85	85	90	100	110

Urban	80	85	90	100	110
Semi-urban	80	80	85	90	100
Semi-rural	75	80	85	90	100
Clustered (agglomerated)rural	70	75	80	85	90
AVERAGE ALLOCATION	82	85	90	98	107
Sparse	60	60	60	60	60

Source :(NWP,2010).

According to the trend assumption for two agglomerations, one agglomerated rural (Bourached) and the other semi-rural (Dreat):

- In 2030, net assets are estimated at 90 l/d/capita for the agglomerated rural agglomeration (Bourached)
- In 2030, net assets are estimated at 100 l/d/capita for the semi-rural agglomeration (Dreat)

To determine the net assets in 2054 for these two agglomerations, we can draw the trend curves for their respective evolution.

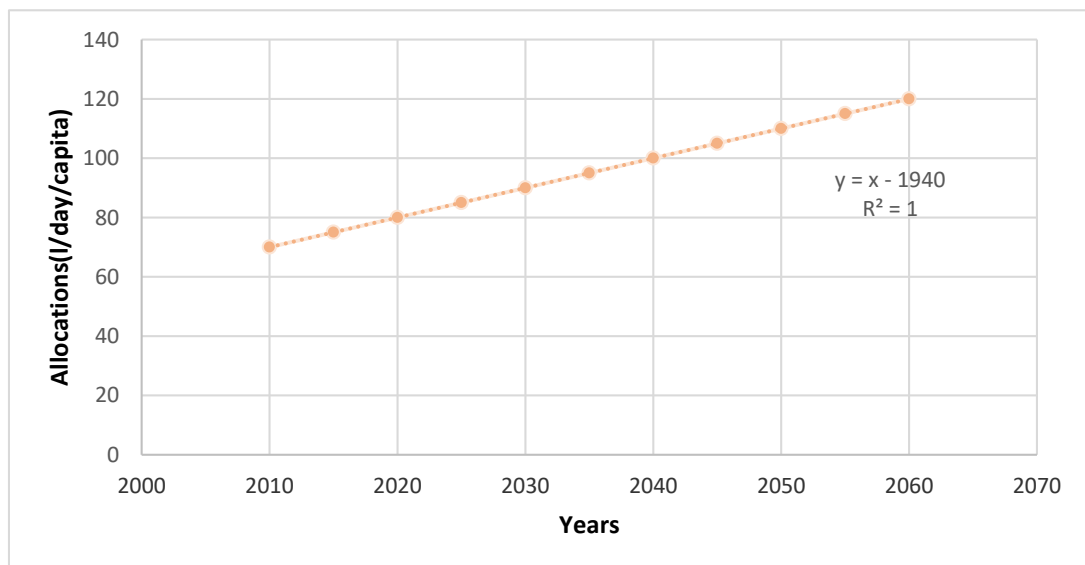


Figure II.2: Trend curve in net allocations for the village of Bourached .

So the net endowment (allocations) by 2054 is 114 l/day/capita .

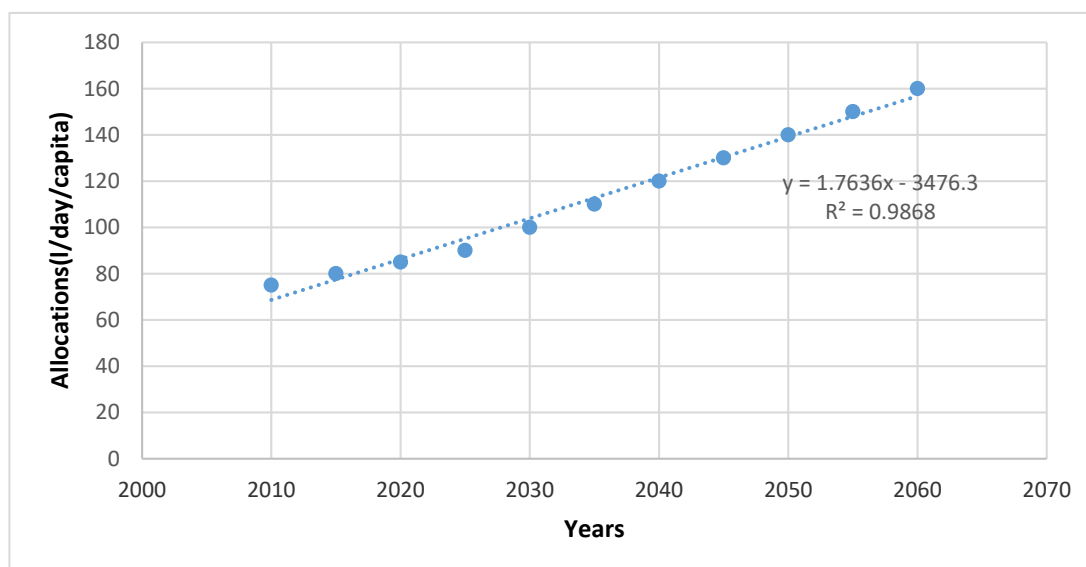


Figure II.3: Trend curve in net allocations for the Village of Dreat .

So the net endowment (allocations) by 2054 is 150 l/day/capita .

II.3.2.Network Yields:

This infrastructure efficiency factor for hydraulic systems represents the loss between the raw resource mobilised upstream and the consumer "at the tap," and must take into account losses in the distribution network as well as losses at treatment and conveyance facilities.

II.3.3.Distribution Yield:

Table II.4: Trend Hypothesis for the Evolution of Distribution Yields .

Yield evolution (%)	Trend hypothesis				
	Supply management		Demand management		
HORIZON	2010	2015	2020	2025	2030
Metropolises	55	60	65	70	75
Higher urban	55	60	65	70	75
Urban	55	55	60	65	70
Semi urban	55	55	60	65	70
Semi rural	55	55	60	65	70
Agglomerated rural	55	55	60	65	70
Agglomerations - wilaya	50	55	60	65	70
administrative centre	55	70	80	85	80

Source: (NWP,2010).

We have :

$$\text{Gross endowments} = \text{Net endowments} * 1/\eta \quad (\text{II.2})$$

Table II.5: Net and gross endowments for the various timeframes .

Village	Timeframe	Actuale 2024	Long term 2054
Bourached	Net endowments (l/day/capita)	84	114
	η	64	80
	K2	1 / 64	1/ 80
	Gross endowments (l/day/capita)	131	143
	Net endowments (l/day/capita)	89	150
Dreat	η	64	80
	K2	1 / 64	1/ 80
	Net endowments (l/d/capita)	139	160

II.3.4 . Determination of average daily consumption :

To determine average daily consumption, the different categories of consumer are taken into account. It is given by the following formula :

$$Q_{ave,d} = (A_i \cdot N_i) / 1000 \text{ (m}^3/\text{d)} \quad (\text{II.3})$$

With :

Q ave,d: Average daily consumption in (m³/d).

A_i: Daily allocation in (l/d/capita).

N_i: Number of consumers.

II.4. Calculation of water needs:

II.4.1. Domestic needs:

Table II.6. Water needs for the different timeframes .

Village	Timeframe	Numbre of inhabitants	Average allocation (l/d/ Capita)	Average daily consumption (m ³ /d)
Bourached	2024	1100	131	144,1
	2054	2307	143	330
Dreat	2024	3439	139	478
	2054	7214	160	1154,2

II.4.2 . Equipment needs:

To estimate average daily consumption, an allocation can be assigned to each piece of equipment.

II.4.2.1. School needs:

Table II.7. Water needs for school facilities .

Village	Designation	Quantity	Pupils number	Allocation (l/d/unit)	Average daily consumption (m3/d)
Bouraced	Primary school	1	150	20	3
	Primary school	2	570	20	11.4
Dreat	M.H.SC	1	580	20	10.56
	Total (m3/d)	21.96			

Source: (Hammam dala municipality,2023).

II.4.2.2. Commercial needs :

Table II.8. Determination of commercial needs .

The table below shows the commercial needs for both of the villages

Village	Designation	Quantity	Allocation (l/d/ unit)	Average daily Consumption (m3/d)
Bourached	/	/	/	/
	Cafe	2	600	1.2
Dreat	Bakery	1	1000	1
	Total(m3/d)	2.2		

Source: (Hammam dala municipality,2023).

II.4.2.3. Cultural and sports needs:

Table II.9: Water needs for cultural and sports facilities.

Village	Equipment type	Numbre	Uses number	Allocation (l/d/unit)	Average daily consumption (m3/d)
Bourached	Mosque	1	800 loyal	20	16
	Stadium	1	30 player	15	1,5
	Total (m3/d)	17.5			
Dréat	Mosque	4	3234 loyal	20	64,68
	Stadium	1	30 player	15	1,5
	Total (m3/d)	66.18			

Source: (Hammam dalaa municipality,2023).

II.4.2.4.Health needs:

Table II.10: Assessment (evaluation) of health needs .

Village	Equipment	Unit	Numbre	Allocation (l/d/unit)	Q ave d (m3/d)
Bourached	Treatment room	Patient	50	20	1
Dreat	Treatment room	Patient	50	20	1

Source: (Hammam dalaa municipality,2023).

II.5. Total equipment water needs:

For equipment needs, we are forecasting an overall increase of 30 %.The total water needs for the two villages are shown in the table below.

Table II.11: Total equipment water needs (net and gross) .

Village	Timeframe	Actuale 2024	Long term 2054
	Growth rate	/	30 %
Bourached	Net equipment needs	21,5	28
	K2	1/64	1/80
	Gross equipment needs	33,59	35
	Growth rate	/	30%
Dreat	Net equipment needs	91,34	118,74
	K2	1/64	1/80
	Gross equipment needs	142,72	148,43

So the average daily flow for the two towns will be :

Table II.12: The average daily flow for the two villages .

Village	Timeframe	Designation	Average daily flow (m3/d)	Total (m3/d)
Bourached	2024	Domestic	144,1	177,69
		Equipments	33,59	
	2054	Domestic	330	365
		Equipments	35	
Dreat	2024	Domestic	478	569,34
		Equipments	91,34	
	2054	Domestic	1154,2	1302,63
		Equipments	148,43	

II.6. Study of Variation in Consumption:

Within an agglomeration, the water called for consumption varies over time. This variation can be:

- **Hourly** : Depending on the consumption pattern of the population.
- **Daily** : Depending on the day of the week when consumption is most significant.
- **Monthly** : Depending on the size of the city.
- **Annual** : Depending on the standard of living of the agglomeration.

To take into account the irregularity of consumption, we must consider a number of irregularity coefficients.

II.6.1. Maximum Irregularity Coefficient ($K_{max,d}$):

It is defined as the ratio between the maximum daily consumption ($Q_{max,d}$) and the average daily consumption ($Q_{ave,d}$).

$$K_{max,d} = Q_{max,d} / Q_{ave,d} \quad (II.4)$$

The value of K_{max} varies between 1.1 and 1.3 (for our case, we take $K_{max,d} = 1.2$).

Therefore, the maximum daily consumption will be as follows :

II.6.1.1. Maximum Daily Consumption ($Q_{max,d}$) :

This flow represents the maximum water consumption on the busiest day of the year. It is obtained using the following relationship :

$$Q_{max,d} = K_{max,d} \times Q_{ave,d} \quad (II.5)$$

Therefore: $Q_{max,d} = 1.2 \times Q_{ave,d}$

II.6.2. Minimum Irregularity Coefficient ($K_{min,d}$):

This coefficient tells us how many times lower the minimum consumption is than the average daily consumption. This coefficient is given by the following formula:

$$K_{min,d} = Q_{min,d} / Q_{moy,d} \quad (II.6)$$

This coefficient is used to determine the minimum daily flow, varies from 0.7 to 0.9 (In our case, we take $K_{min,d} = 0.8$). The minimum daily flow will therefore be as follow:

II.6.2.1 Minimum daily consumption ($Q_{min,d}$) :

This flow represents the minimum water consumption on the lightest day of the year. It is obtained using the following relationship :

$$Q_{min,d} = K_{min,d} \times Q_{moy,d} \quad (II.7)$$

Therefore: $Q_{min,d} = 0.8 \times Q_{ave,d}$

Table II.13: Maximum and minimum daily consumption.

Village	Timeframe	Q ave,d (m3/d)	Kmaxd	Qmax (m3/d)	Kmind	Qmin d (m3/d)
Bourached	2024	177.69	1.2	213.23	0.8	142.15
	2054	365	1.2	438	0.8	292
	2024	569.34	1.2	683.21	0.8	546.57
Dreat	2054	1302.63	1.2	1563.16	0.8	1042.10

II.6.3 Variation in hourly consumption :

1) Maximum hourly irregularity coefficient ($K_{max,h}$):

This coefficient represents the increase in hourly consumption during the day. This is known as the peak coefficient. It takes into account:

- Population growth .
- The degree of comfort and the working regime of industry
- Sanitary development and also the layout of buildings.

The following formula is used to calculate it:

$$K_{max, h} = \alpha_{max} \times \beta_{max} \quad (\text{II.8})$$

2) Minimum hourly irregularity coefficient ($K_{min,h}$):

This is given by the minimum hourly irregularity coefficient $K_{min,h}$ which is used to determine the minimum hourly flow rate allowing for under consumption

The following formula is used:

$$K_{min, h} = \alpha_{min} \times \beta_{min} \quad (\text{II.9})$$

Such as:

α : Coefficient that depends on the level of comforts of local conditions and the level of development.

- $\alpha_{max} = 1,2 \div 1,4$.
- $\alpha_{min} = 0,4 \div 0,6$.

β : Coefficient depending on the number of inhabitants [population].

Table II.14: Values of β_{max} .

Population	<1000	1500	2500	4000	6000	10000	20000	30000	50000	100000
β_{max}	02	0,8	1,6	1,5	1,4	1,3	1,2	1,15	1,13	1,1

β min	0,1	0,1	0,1	0,2	0,25	0,4	0,5	0,6	0,63	0,7
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Source: (DWS course,2023).

II.6.3.1 Calculating the minimum ($K_{min,h}$) and maximum ($K_{max,h}$) hourly irregularity coefficients:

Based on the population of each village, we obtain :

Table II.15: The minimum ($K_{min,h}$) and maximum ($K_{max,h}$) hourly irregularity coefficients:

Village	Timeframe	α max	β_{max}	$K_{max,h}$	α min	β min	$K_{min,h}$
Bourached	2024	1,3	0,8	1,04	0,6	0,1	0,06
	2054	1,2	1,6	1,92	0,6	0,1	0,06
Dreat	2024	1,3	1,5	1,95	0,6	0,2	0,12
	2054	1,2	1,3	1,56	0,6	0,4	0,24

II.6.4. Average hourly flow rate:

The average hourly flow rate is given by the following relationship:

$$Q_{ave, h} = Q_{max} d / 24 \text{ (m}^3/\text{h)} \quad (\text{II.10})$$

Where:

$Q_{ave, h}$: average hourly flow rate in m³/h .

$Q_{max,h}$: maximum daily flow rate in m³/d

II.6.4.1.Maximum hourly flow rate :

This flow rate plays a very important role in the various distribution network calculations, and is determined by the following relationship:

$$Q_{max, h} = K_{max, h} \times Q_{ave, h} \quad (\text{II.11})$$

II.6.4.2.Minimum hourly flow rate:

$$Q_{min, h} = K_{min, h} \times Q_{ave, h} \quad (\text{II.12})$$

Table II.16:Calculation of hourly, minimum and maximum flow rates.

Village	Timeframe	$Q_{ave,h}$ (m ³ /h)	$K_{max, h}$	$Q_{max,h}$ (m ³ /h)	$K_{min, h}$	$Q_{min, h}$
Bourached	2024	8,88	1,04	9,24	0,06	0,53
	2054	18,25	1,92	35,04	0,06	1,1
Dréat	2024	28,47	1,95	55,52	0,12	3,42

2054 65,13 1,56 101,60 0,24 15,63

II.6.5. Evaluation of hourly consumption as a function of the number of inhabitants :

The variation in hourly flow rates is expressed as a percentage (%) of the maximum daily flow rate for the agglomeration .We need to establish the hourly distribution of the maximum daily flow rate for our villages, with hourly consumption coefficients determined according to their populations.

With :

$$Q_h = (P\% \times Q_{max\ d}) / 100 \quad (II.15)$$

Such that:

Q h: Hourly flow rate required [m³/h] .

P%: Hourly percentage .

Q maxj: Maximum daily flow rate .

II.6.6. Determination of maximum hourly flow Q max,h(peak Qp) :

✓ **Timeframe 2024 :**

For the village of Bourached , the number of inhabitants is 1100 .

So :

Table II.17: Hourly consumption in the town of Bourached .

Hours	Agglomeration consumption		Cumulative consumption	
	Q_{maxd}=213.23 (m³/d)			
	P %	Q _h [m ³ /h]	P % cumulative	Q _h cumulative
0-1	1	2,13	1	2,13
1-2	1	2,13	2	4,26
2-3	1	2,13	3	6,39
3-4	1	2,13	4	8,52
4-5	2	4,26	6	12,78
5-6	3	6,40	9	19,18
6-7	5	10,66	14	29,84
7-8	6,5	13,86	20,5	43,7
8-9	6,5	13,86	27	57,56
9-10	5,5	11,73	32,5	69,29
10-11	4,5	9,60	37	78,89
11-12	5,5	11,73	42,5	90,62
12-13	7	14,93	49,5	105,55

13-14	7	14,93	56,5	120,48
14-15	5,5	11,73	62	132,21
15-16	4,5	9,60	66,5	141,81
16-17	5	10,66	71,5	152,47
17-18	6,5	13,86	78	166,33
18-19	6,5	13,86	84,5	180,19
19-20	5	10,66	89,5	190,85
20-21	4,5	9,60	94	200,45
21-22	3	6,40	97	206,85
22-23	2	4,26	99	211,11
23-24	1	2,13	100	213,24
Total	100	213,24		

The table above shows that the peak consumption hours are from (12 to 14am), with an estimated maximum hourly flow of 14.93 [m³/h] and minimum consumption is estimated at 2.13 m³/h of the hours (0 to 4am) and (23 to 24 pm).

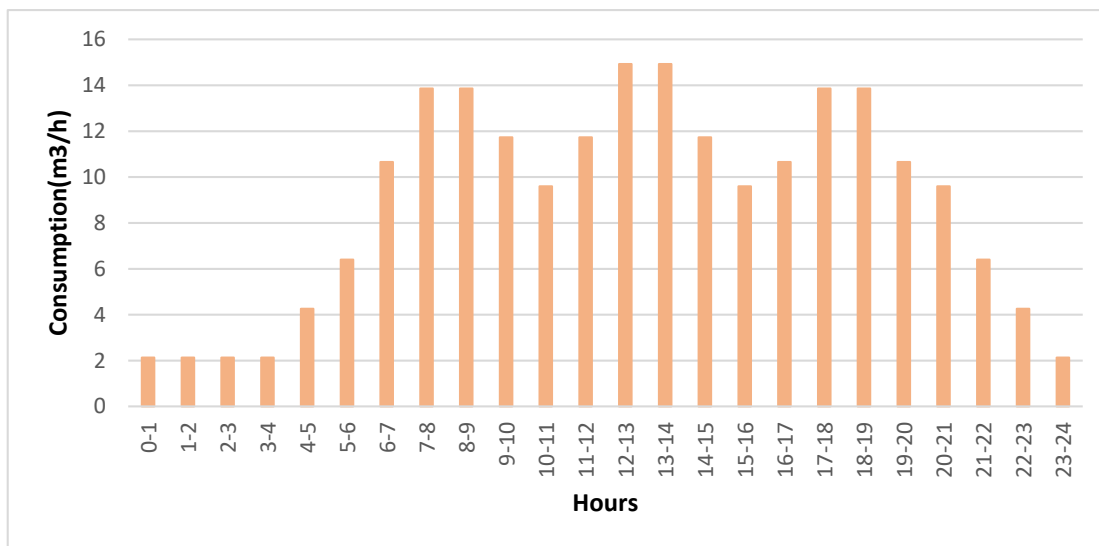


Figure II.4: Variation in hourly flow during the peak day for Bourached village (2024)

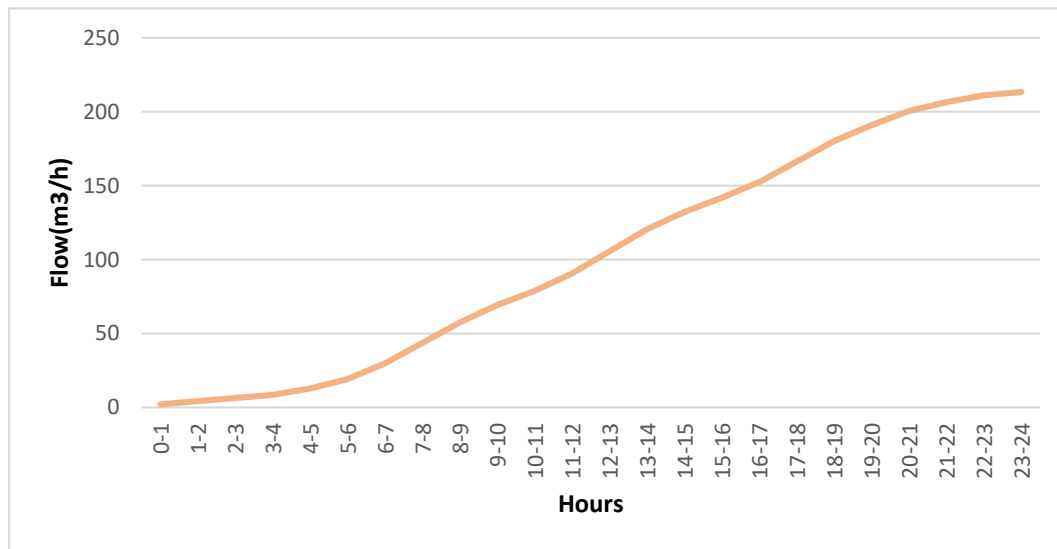


Figure II.5: Cumulative hourly flows for Bourached village (2024) .

According to the graph of the variation in hourly flow, $Q_{maxh} = 14.93 \text{ m}^3/\text{h}$.

For the village of Dreat , the number of inhabitants is 3439 .

So :

Table II.18: Hourly consumption in the village of Dreat .

Hours	Agglomeration consumption $Q_{maxd}=683,21 \text{ (m}^3/\text{d)}$		Cumulative consumption	
	P %	Qh [m³/h]	P% cumulative	Qh cumulative
0-1	1	6,83	1	6,83
1-2	1	6,83	2	13,66
2-3	1	6,83	3	20,49
3-4	1	6,83	4	27,32
4-5	2	13,66	6	40,98
5-6	3	20,50	9	61,48
6-7	5	34,16	14	95,64
7-8	6,5	44,41	20,5	140,05
8-9	6,5	44,41	27	184,46
9-10	5,5	37,58	32,5	222,04
10-11	4,5	30,74	37	252,78
11-12	5,5	37,58	42,5	290,36
12-13	7	47,82	49,5	338,18
13-14	7	47,82	56,5	386
14-15	5,5	37,58	62	423,58
15-16	4,5	30,74	66,5	454,32
16-17	5	34,16	71,5	488,48
17-18	6,5	44,41	78	532,89
18-19	6,5	44,41	84,5	577,3

19-20	5	34,16	89,5	611,46
20-21	4,5	30,74	94	642,2
21-22	3	20,50	97	662,7
22-23	2	13,66	99	676,36
23-24	1	6,83	100	683,19
Total	100	683,19		

The table above shows that the peak consumption hours are from (12 to 14am) , with an estimated maximum hourly flow of 47.82 [m³/h] and minimum consumption is estimated at 6.83 m³/h of the hours (0 to 4am) and (23 to 24pm) .

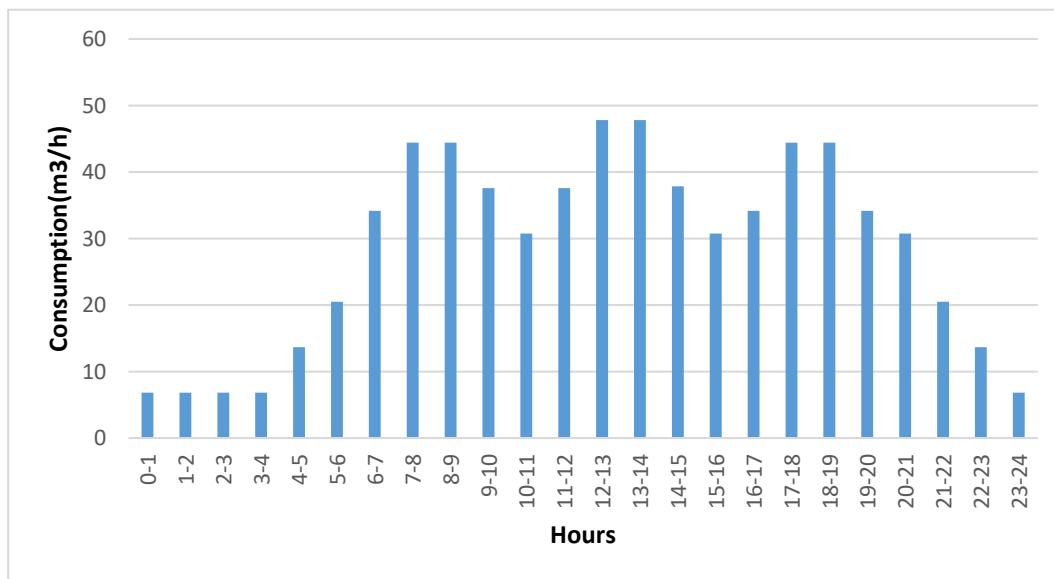


Figure II.6: Variation in hourly flow during the peak day for Dreat village (2024) .

The figure below shows the hourly flow variation in the Dreat village in 2054

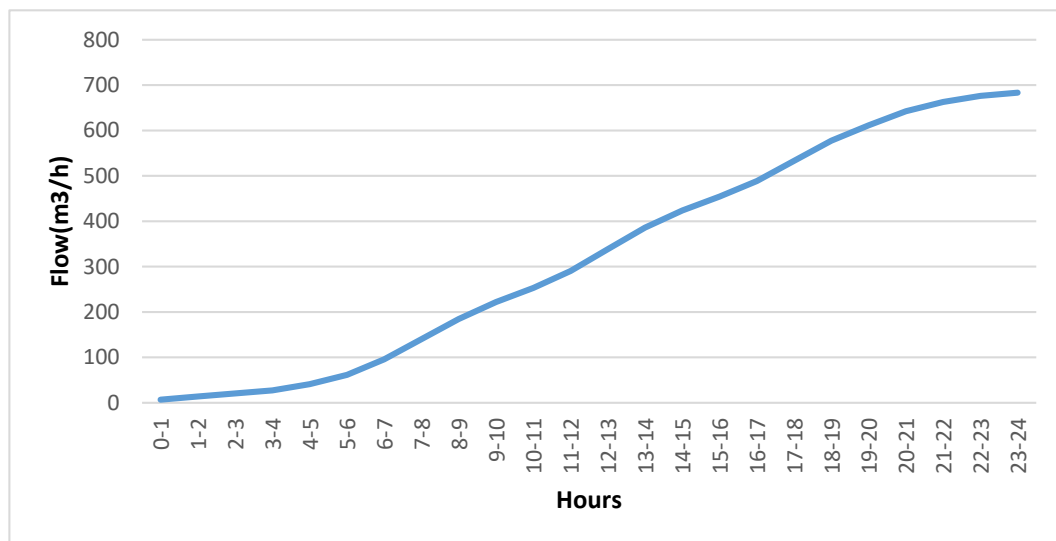


Figure II.7: Cumulative hourly flows for Dreat village (2024) .

According to the graph of the variation in hourly flow, $Q_{maxh} = 47.82 \text{ m}^3/\text{h}$.

✓ **Timeframe 2054 :**

For the village of Bourached , the number of inhabitants is 2307.

So :

Table II.19: Hourly consumption in the village of Bourached .

Hours	Agglomeration consumption $Q_{maxd} = 438 \text{ (m}^3/\text{d)}$		Cumulative consumption	
	P %	Qh [m ³ /h]	P% cumulative	Qh cumulative
0-1	1	4,38	1	4,38
1-2	1	4,38	2	8,76
2-3	1	4,38	3	13,14
3-4	1	4,38	4	17,52
4-5	2	8,76	6	26,28
5-6	3	13,14	9	39,42
6-7	5	21,9	14	61,32
7-8	6,5	28,47	20,5	89,79
8-9	6,5	28,47	27	118,26
9-10	5,5	24,09	32,5	142,35
10-11	4,5	19,71	37	162,06
11-12	5,5	24,09	42,5	186,15
12-13	7	30,66	49,5	216,81
13-14	7	30,66	56,5	247,47
14-15	5,5	24,09	62	271,56
15-16	4,5	19,71	66,5	291,27
16-17	5	21,9	71,5	313,17
17-18	6,5	28,47	78	341,64
18-19	6,5	28,47	84,5	370,11
19-20	5	21,9	89,5	392,01
20-21	4,5	19,71	94	411,72
21-22	3	13,14	97	424,86
22-23	2	8,76	99	433,62
23-24	1	4,38	100	438
Total	100	437,88		

The table above shows that the peak consumption hours are from (12 to 14am) , with an estimated maximum hourly flow of 30.66 [m³/h] and minimum consumption is estimated at 4.38 m³/h of the hours (0 to 4am) and (23 to 24pm).

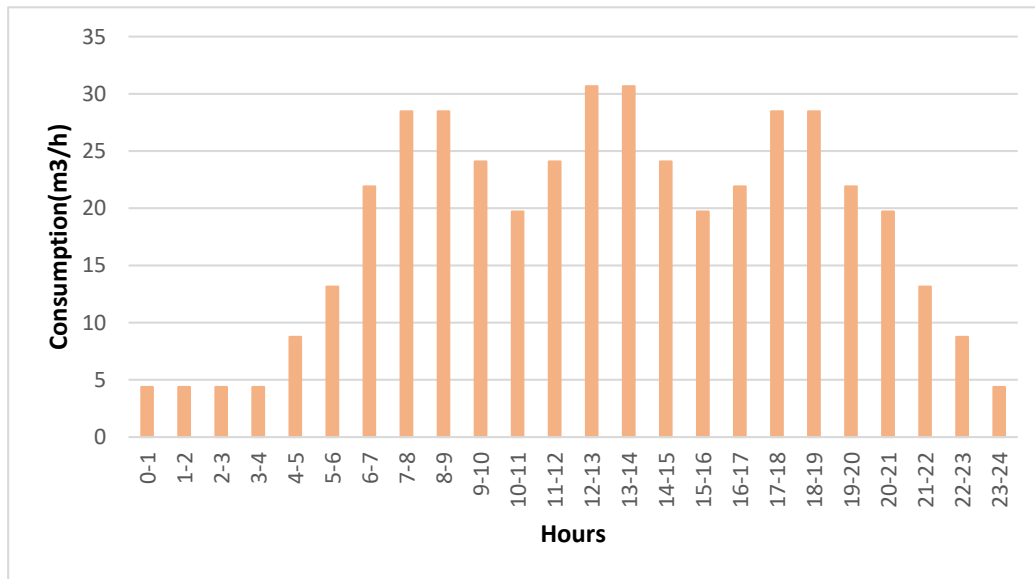


Figure II.8: Variation in hourly flow during the peak day for Bourached village (2054) .

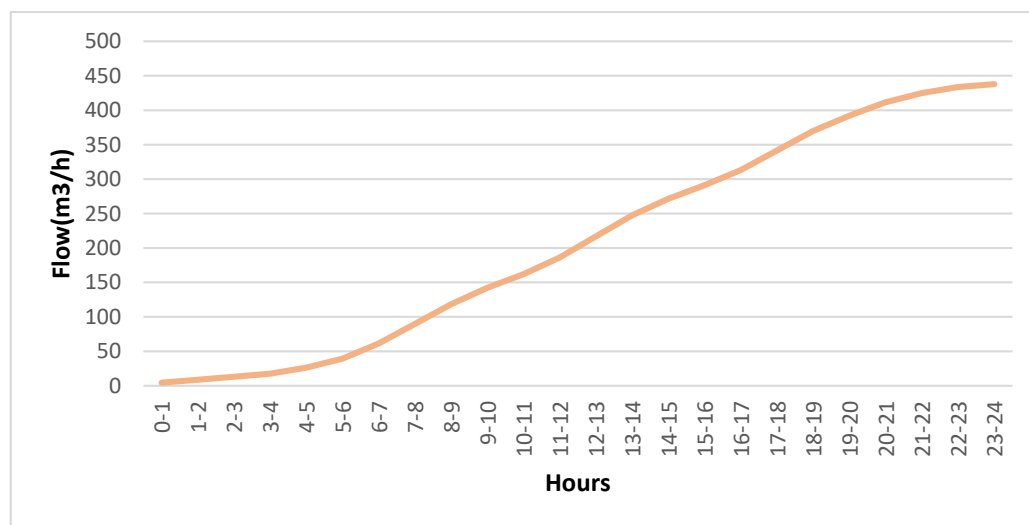


Figure II.9: Cumulative hourly flows for Bourached village (2054).

According to the graph of the variation in hourly flow, $Q_{maxh} = 30.66 \text{ m}^3/\text{h}$.

For the village of Dreat , the number of inhabitants is 7214.

So :

Table II.20: Hourly consumption in the village of Dreat .

Hours	Agglomeration consumption Q _{maxd} =1563,16 (m ³ /d)		Cumulative consumption	
	P %	Q h [m ³ /h]	P% cumulative	Q h cumulative
0-1	1	15,63	1	15,63
1-2	1	15,63	2	31,26
2-3	1	15,63	3	46,89
3-4	1	15,63	4	62,52
4-5	2	31,26	6	93,78
5-6	3	46,89	9	140,67
6-7	5	78,16	14	218,83
7-8	6,5	101,61	20,5	320,44
8-9	6,5	101,61	27	422,05
9-10	5,5	85,97	32,5	508,02
10-11	4,5	70,34	37	578,36
11-12	5,5	85,97	42,5	664,33
12-13	7	109,42	49,5	773,75
13-14	7	109,42	56,5	883,17
14-15	5,5	85,97	62	969,14
15-16	4,5	70,34	66,5	1039,48
16-17	5	78,16	71,5	1117,64
17-18	6,5	101,61	78	1219,25
18-19	6,5	101,61	84,5	1320,86
19-20	5	78,16	89,5	1399,02
20-21	4,5	70,34	94	1469,36
21-22	3	46,89	97	1516,25
22-23	2	31,26	99	1547,51
23-24	1	15,63	100	1563,14
Total	100	/	/	/

The table above shows that the peak consumption hours are from (12 to 14am) , with an estimated maximum hourly flow of 109.42 [m³/h] and minimum consumption is estimated at 15.63 m³/h of the hours (0 to 4am) and (23 to 24pm) .

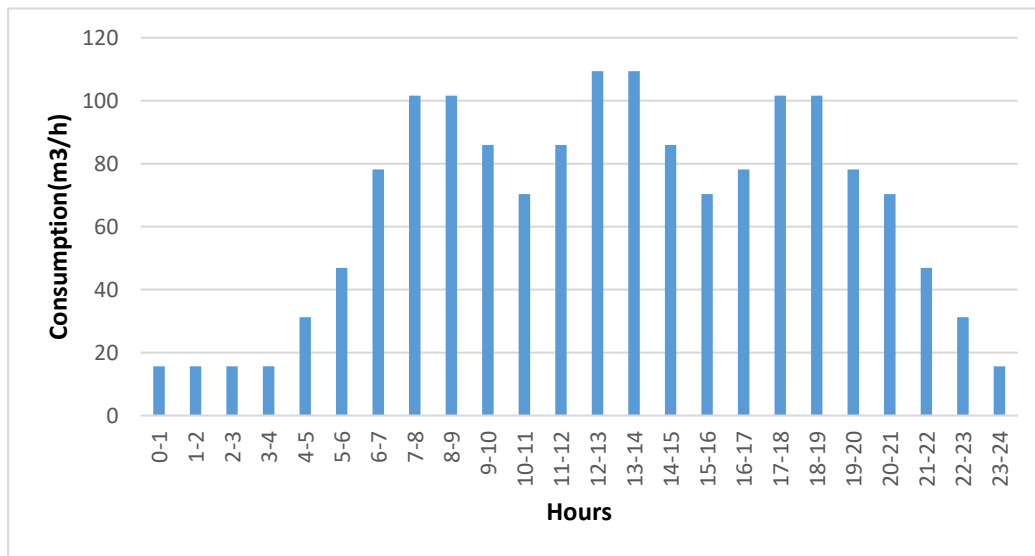


Figure II.10: Cumulative hourly flows for Dreat village (2054) .

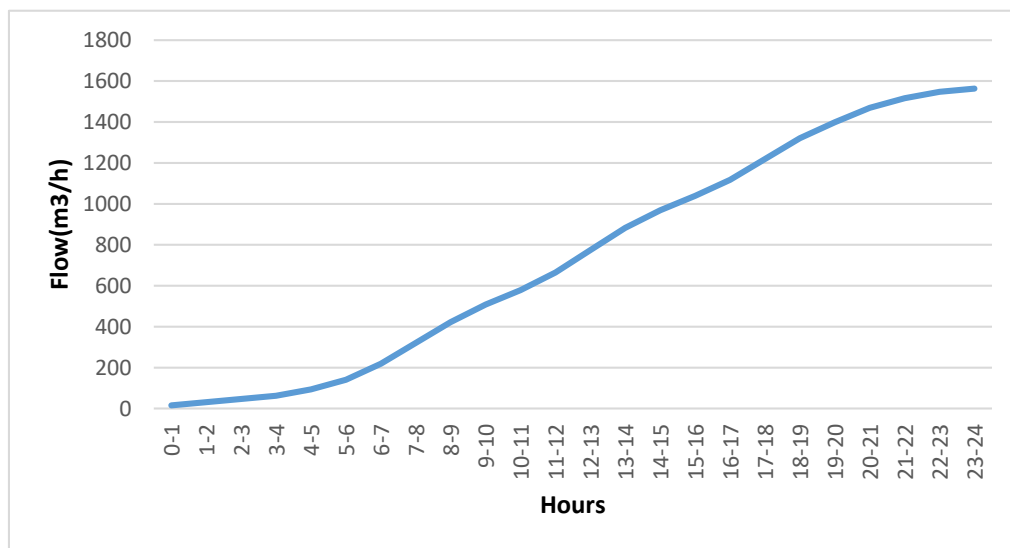


Figure II.11: Cumulative hourly flows for Dreat village (2054) .

According to the graph of the variation in hourly flow, $Q_{maxh} = 109.42 \text{ m}^3/\text{h}$.

Conclusion :

This chapter has enabled us to identify, calculate and estimate the various water needs (domestic, public, taking into account leaks and wastage), of the two villages for the current

(2024) and projected (2054) timeframes. We have then calculated the maximum daily flows and the peak flows for the villages, we found :

- (Q max d= 213.23 m³/d (2024) , Q max d= 438 m³/d (2054)) for Bourached village.
- (Q max d= 683.21 m³/d (2024) , Q max d= 1563.16 m³/d (2054) for Dreat village.

In order to transfer this quantity to the consumers, it is essential to size the supply works (the supply structures), which is the objective of the next chapter .

CHAPTRE III:

Study of Tanks

CHAPTRE III: STADY OF TANKS

Introduction :

In all water supply networks, tanks are needed. These are hydraulic structures that generally serve to store the water that is found after the adduction network and before the water distribution network .The purpose of this chapter is to determine the necessary storage volumes and verify them in such a way as to ensure the optimal functioning of the networks.

III.1. Tanks Roles :

Among the known roles of a reservoir in a water supply system are:

- Serving as a reserve to ensure the maximum flow rate required during peak hours.
- Ensuring continuity of distribution during pump shutdown.
- Connecting the supply flow rate to the distribution flow rate.
- In the case of staged distribution, it acts as a pressure equalizer.
- To play the role of a relay.
- To ensure fire reserve.
- To regulate the operation of the pump.
- Regulate pressure and flow in the distribution network .
- Establish a reserve for unforeseen events (ruptures, pump failures, repairs, network extension...).

III.2.Tanks Classification :

Tanks can be classified as follows:

1. Classification based on construction material :

- Masonry tanks .
- Reinforced concrete reservoirs, ordinary or pre-stressed .
- Metallic reservoirs .

2. Classification based on location :

- Buried tanks .
- Semi-buried tanks .
- Elevated tanks .
- Tanks on .

3. Classification based on usage :

- Main accumulation and storage tank
- Terminal tank
- Equilibrium tank
- tank buffer
- Safety net
- Load breaker

4. Classification according to geometric shape :

- **Cylindrical tank:** Large circular tanks can be made of reinforced concrete. In most cases, we do not achieve significant savings compared to rectangular tanks. The advantage is stability of excavation slopes and reduced risk of cracking, as well as construction material savings.
- **Rectangular tank:** In general, tanks are rectangular or polygonal (if the work must adapt to the shape of the plot or to the terrain conditions). These shapes allow for static construction without surprises and adaptable, solid construction, as well as later expansion without major difficulties.

III.3.Tank Location :

The location of the tank is subject to the condition of ensuring sufficient pressure to subscribers at peak flow. They must be placed at a level higher than that of the agglomeration they serve. The altitude of the tanks, specifically the sill, must be at a level higher than the highest piezometric height required on the network.

The placement of the reservoir is influenced by various factors, such as:

- The terrain model.
- The availability of suitable land at an affordable price.
- Potential height restrictions.
- The nature of the soil (foundation).
- The size of the tanks.

III.4.Operation Principle :

The principle of operation of water regulation (demand and supply) is based on the following points:

- The installations and accessories for water supply that allow for significant regulation.

- The tank allows for storing differences in water flow during periods of low consumption. However, during peak consumption hours, the deficit will flow from the reservoir to the distribution network.

III.5. Equipment of the reservoir :

Reservoirs must be equipped with:

III.5.1. Inlet or Feed Line :

This line, whether of the pumping or gravity type, should preferably enter the tank through a submerged siphon (at the top of the tank) or from the bottom, always opposite to the outlet line to induce mixing. This inlet allows for water renewal through mixing by creating disturbances and a rolling flow.

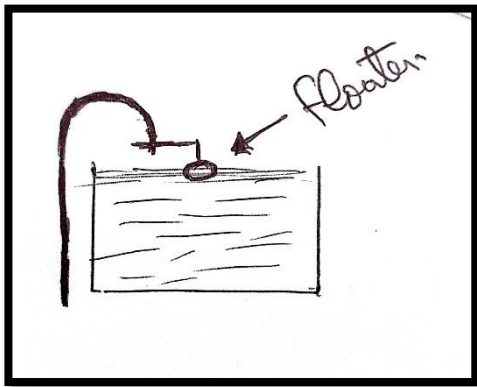


Figure III.1: Free Fall Arrival

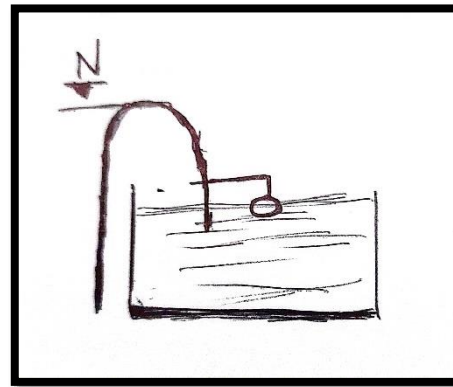


Figure III.2: drowned arrival

III.5.2. Outlet or Distribution Line :

This line is positioned opposite to the inlet line a few centimeters above the base (15 to 20 cm) to prevent the entry of suspended matter. The end is equipped with a curved strainer to prevent the vortex phenomenon (air penetration into the line). This line is fitted with an over speed valve for quick closure in case of a break in this line.

III.5.3. Drain Line :

It enables draining the tank for cleaning or repairs. It is equipped with a valve tap and is usually connected to the overflow line. The valve tap should be cleaned after each draining to prevent sand deposits.

III.5.4. Overflow Line :

This line allows excess water entering the tank to be discharged in case a pump does not stop. If the tank is compartmentalized, each compartment must have an overflow line. These lines

should converge in the control chamber to form a hydraulic seal, preventing the entry of any foreign bodies

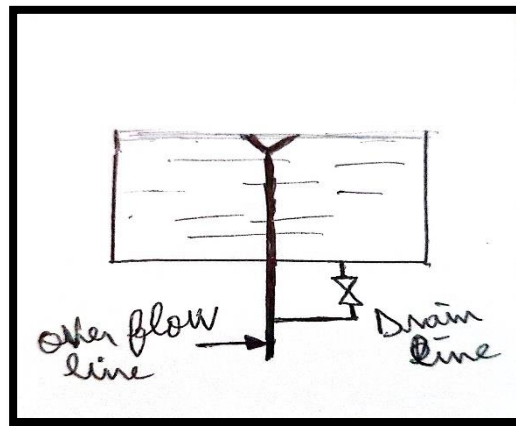


Figure III.3: Drain and Overflow line

III.5.5. Sectioning Valves :

Sectioning valves are placed at the entrance and exit of each compartment to isolate it during cleaning operations

III.5.6. By-pass Piping:

A section of piping connects the inlet pipe and the outlet pipe in the case of a single tank. This piping functions when the tank is isolated for maintenance.

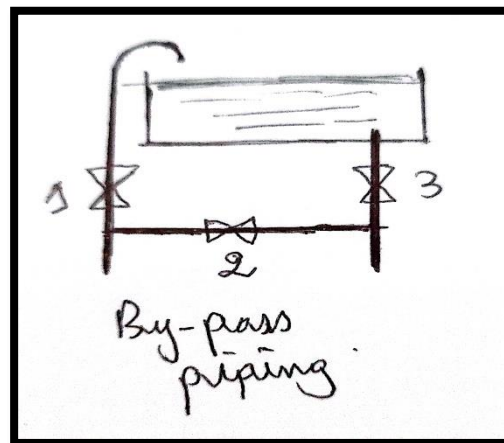


Figure III.4: By-pass Piping

III.5.7. Fire Demand Materialization System:

This is a special arrangement that preserves the fire volume at the tank level and allows the flow to be interrupted once the fire reserve level is reached. We have two systems:

III.5.7.1. Two-Intake System:

This two-intake system is rarely used because the safety reserve is not properly renewed.

III.5.7.2. Siphon System

This system has the advantage of constantly renewing the fire reserve.

III.6. Hygiene and Safety:

For hygiene and safety reasons, the tanks must:

- Place the drinking water tanks away from potential sources of pollution .
- Install physical barriers such as fences to restrict unauthorized access to the tanks .
- Conduct regular surveillance of the tanks .
- Carry out maintenance and cleaning operations in accordance with sanitary standards .
- Be ventilated through the grill openings .

III.7. Choice of the tank :

It will of course be a matter of species for each case, however, whenever possible. It will be preferable to use the buried, semi-buried or at most elevated above the ground tank with a slightly buried slab.

The choice of the tank depends on:

- **Topographic conditions :**
The topography of the site, such as the slope and the available space, will influence the choice of the tank type (buried, semi-buried or elevated).
- **Geotechnical conditions :**
The geotechnical characteristics of the soil, such as the bearing capacity, the water table level, and the risk of flooding, will also impact the choice of the tank type.
- **Hydraulic conditions.**
- **Economic conditions.**

III.8. Determination of tank Capacity:

- Tank capacity should be estimated taking into account variations in flow both at the inlet and outlet : this includes the operation mode of upstream structures and the demand variation.
- Typically, capacity is calculated to meet daily flow variations in consumption, considering the day of highest consumption and the water reserve allocated for firefighting purposes.

III.9. Calculation Principle :

There are two methods that can be used:

- The analytical method .
- The graphic method .

III.9.1 The analytical method :

The analytical method consists of calculating the residue in the tank.

The useful volume of the tank will be :

$$Vu = (a\% * Q_{max.j}) / 100 \quad (III.1)$$

With :

Vu: The useful volume (m3).

a%: Percentage of maximum volume to be stored (%).

Q_{max.j}: Maximum daily consumption (m3/d).

The total volume is determined by adding the fire volume to the useful volume:

$$VT = Vu + Vinc \quad (III.2)$$

With :

VT: Total tank capacity (m3).

Vinc: Estimated fire volume of 120 m3.

III.9.2. The graphical method :

The graphical method takes into account the total consumption curve (integral) deduced from the hourly variation coefficients of consumption, and the curve of pumped flow rate as a function of pumping time. The capacity is deduced from the extremes of the cumulative consumption versus inflow.

$$Vu = \Delta V1 + \Delta V2 \quad (III.3)$$

With the total volume will be :

$$V_t = V_u + V_{inc}$$

(III.4)

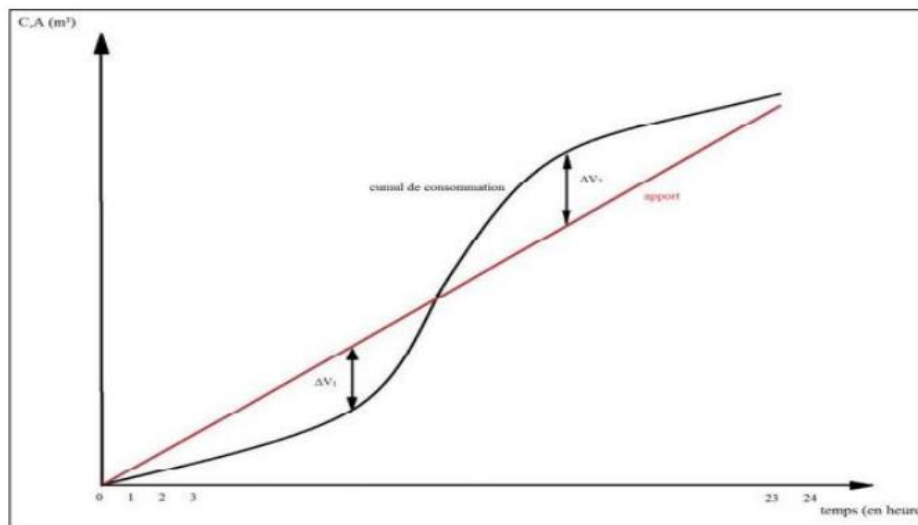


Figure III.5: Total consumption curve, Input curve .

III.9.3. Checking the capacity of the existing water tower :

In our study, we have an existing cylindrical water tower with a capacity of 1000 m³ intended to distribute water to the two villages concerned by the study, but this water tower is not in service because it is waiting to be connected to the existing reservoir. We will therefore check whether or not its capacity is sufficient to cover the population's water needs.

III.9.3.1. Calculating the inflow rate (Q_{app}) :

Water is supplied from the spring to the reservoir over a period of 20 hours.

flow will be calculated as follows:

$$Q_{app} = Q_{max}.d / 20\text{hours}$$

Where:

Q_{app} : is the inflow into the existed reservoir (m³/d).

III.9.3.2. Calculating the useful volume of the tank :

The following table summarises the analytical method for calculating the useful volume :

Table III.1: Calculation of reservoir capacity using the analytical method.

Hours	Backflow	Water consumption (%)	Departure	Arrival	Residue
0-1		1		1	1
1-2		1		1	2
2-3		1		1	3
3-4		1		1	4
4-5	5	2	3		1

5-6	5	3	2		-1
6-7	5	5	0		-1
7-8	5	6,5		1.5	0.5
8-9	5	6,5		1.5	2
9-10	5	5,5		0.5	2.5
10-11	5	4,5	0.5		2
11-12	5	5,5		0.5	2.5
12-13	0	7		2	4.5
13-14	0	7		2	6.5
14-15	0	5,5		0.5	7
15-16	0	4,5	0.5		6.5
16-17	5	5	0		6.5
17-18	5	6,5		1.5	8
18-19	5	6,5		1.5	9.5
19-20	5	5	0		9.5
20-21	5	4,5	0.5		9
21-22	5	3	2		7
22-23	5	2	3		4
23-24	5	1	4	0	0

According to the table: the maximum residue in the tank is 10,5%. And we have

$$Q_{maxj} = 3275,42 \text{ m}^3/j$$

Therefore:

$$\text{Residue} = |9.5| + |-1|, \text{Residue} = 10.5$$

$$Vu = (10,5 \times 2001.16) / 100, \quad Vu = 210.12 \text{ m}^3$$

$$VT = Vu + V_{fire}, \quad = 210.12 + 120$$

$$VT = 330,12 \text{ m}^3.$$

The total volume of the tank is normalized to be :

$$VT = 400 \text{ m}^3.$$

III.9.3.3. Reservoir sizing :

A tank with a circular cross-section will be used, the main dimensions of which will be determined from the following relationship from the following relationship :

$$V = V = ((\pi D^2) / 4) \cdot H \quad (\text{III.6})$$

Therefore:

$$D = \sqrt{((4 \cdot V) / (\pi \cdot H))} \quad (\text{III.7})$$

Where:

V: Tank volume (m³).

D: Tank diameter (m)

H: Optimum useful water height in the tank (m). The value of the height "H" can vary between 3 and 6 m, and can reach 7 to 8 m in large structures

III.9.3.4 Calculating the cross-sectional area of the tank :

$H = [3 \text{ to } 6] \text{ m}$, We take $H = 3 \text{ m}$

$$VT = S * H$$

$$S = VT / H \quad , \quad S = 400/3 \quad , \quad S = 133.33 \text{ m}^2 .$$

Total tank height :

$$H = h + R \quad (III.8)$$

h: Height of water (m)

R: Freeboard (m), varies from 0.25 to 1 m

III.9.3.5. Calculating the tank diameter :

$$S = \pi * D^2 / 4$$

$$D = \sqrt{(4 * S / \pi)} = \sqrt{(4 * 133.33 / \pi)}$$

$$D = 13.03 \text{ m}$$

III.9.3.6. The height of the fire tank :

$$H_{inc} = V_{inc} / S \quad (III.9)$$

$$H_{inc} = 120 / 133.33$$

$$H_{inc} = 0.9 \text{ m} .$$

III.9.3.7. Tank shape :

The cylindrical section is generally adopted for tanks with a capacity greater than 250 m³, which means that this shape can be chosen for the reservoir .

The following table summarises the characteristics of the reservoir

Table III.2: Characteristics of the reservoir

Tank	Type	V	D	S	h	R	H
Planned							
tank	On floor	400	13.03	133.33	2.75	0.25	3

III.9.3.8. Interpretation of results :

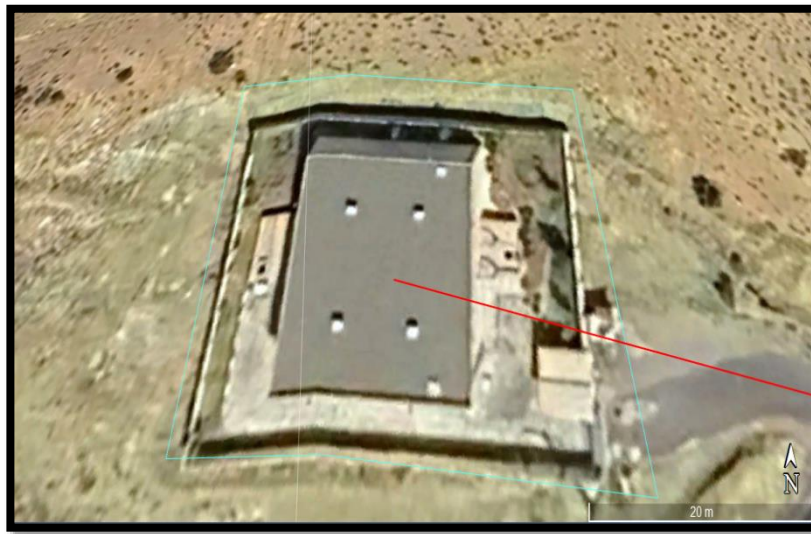
Based on the results, we conclude that the capacity of the existing water tower 1000 m³ is greater than the calculated capacity 400 m³, which means that the existing water tower is sufficient to cover the needs of the population .

Table III.3: Characteristic of the water tower.

Reservoir	Type	GE (m)	IE(m)	V(m)	D (m)	S (m2)	H(m)
Water tower	On floor	1279.4	1282	1000	18	250	4

Table III.4: Characteristic of the existing buffer tank .

Reservoir	Type	CTN (m)	V (m)	D (m)	S (m2)	H (m)
Buffer tank	On floor	884	2000	24.5	470.92	4.5

**Figure III.6: Satellite view of the existed rectangular buffer tank with a capacity of 2000 m3.**

Source : (Google Earth 17-06-2024 /01:15 am)

III.10. Some recommendations on reservoirs :

- After any work likely to contaminate the water contained in the tanks, and in any case at least once a year, the tanks are emptied, cleaned and disinfected.
- Tanks must be protected against any pollution from outside sources and against significant temperature rises.
- They must be easy to access and installed in such a way that they can be checked for leaks at any time, at all times.

Conclusion :

In this chapter, we have presented the main characteristics of the existing buffer tank and verified the capacity of the existing cylindrical water tower which located at a ground elevation of 1279.4 m to guarantee the water distribution of our two villages. The results obtained show that it is the capacity of 1000 m^3 is sufficient to cover the needs of the agglomerations , After having checked the capacity of the water tower , it would be imperative to dimension the pipes conveying the water, which will be the subject of the next chapter.

CHAPTER IV :
Convenience study

Introduction:

Water conveyance involves the transportation of water from a collection or storage point to another location. Depending on the terrain of the area in question, the design of the conveyance system can be achieved through pumping or gravity flow. In this chapter, we will assess the size of the adduction network that we propose for the drinking water supply of our study area.

IV.1. Type of supply:

According to their operation, water supplies can be classified into three groups :

IV.1.1. Gravity-driven supply :

Adduction where the flow occurs by gravity. This type of adduction is encountered when the source is located at an elevation higher than the piezometric level of the delivery structure. The flow can be under pressure or on a free surface. (absence of significant counter slope).

IV.1.2 Supply by pumping:

Conveyance where the source is located at a lower level than the piezometric level of the intake structure, and the water flows under pressure using hydraulic machines.

IV.1.3. Mixed supply:

It is an supply where the pressure pipeline is transformed into a gravity pipeline or vice versa. The transition between the two types of pipelines is ensured by a tank called a buffer tank. And for these three cases, the topography factor comes into play. The latter forces us to place certain accessories:

- Air valves at the high points of the route for air evacuation.
- Drain valves at the low points.
- Section valves to avoid complete draining of the pipeline.

IV.2. Supply and types of flow:

There are two types of flow :

IV.2.1. Free Surface Flow :

Free surface flow that is a type of flow encountered only in the case of gravity-fed conduits. It can be open-air or in covered channels. This type of flow is dependent on the slope.

IV.2.2. Pressurized Flow :

This is a type of flow encountered in the case of pressurized gravity-fed conduits and pumped conduits.

IV.3. Layout selection (trace):

IV.3.1. Selection criteria :

In the choice of route alignment, certain conditions must be respected, which are as follows:

- The alignment must be as short as possible from an economic point of view and as direct as possible.
- Choose a stable terrain along the alignment.
- Choose an accessible site
- Avoid reverse slopes, which can lead to high points and make it more or less difficult to evacuate air pockets.
- Avoid too rugged or rocky terrain, crossing of Oueds (seasonal rivers), roads and railways.
- Take into account the longitudinal profile, with widely open bends to avoid significant butting.

IV.3.2. Choice of pipe type :

The choice of different pipes is made on the basis of the following:

- Diameter .
- Supported pressures .
- Installation conditions .
- Coating .
- Market availability .
- Unit price .
- Type of soil .

Don't forget to factor in the cost of transport, which remains an important parameter.

IV.4. Selecting the pipe material :

The choice of material for buried pipes depends on their technical and economic reliability.

This technical feasibility depends on a number of requirements such as :

- Resistance chemical attack, pressure and resistance to moving loads .
- Adaptation laying sites .
- water tightness .
- Facility maintenance and commissioning .

In our case, we opt for HDPE (High Density Polyethylene) pipes, which better meet the aforementioned requirements and which are also highly resistant to corrosion and ground movement.

IV.4.1. Pipes of HDPE :**IV.4.1.1 Advantages :**

- Good resistance to internal, external and microbiological corrosion and scaling, thanks to the chemical inertness of the pipe, which prevents the dissolution and abrasion of metals even if the water is not perfectly balanced.
- Market availability.
- Easy to fit (good flexibility). This means they adapt well to the terrain, they can be for small diameters.
- Reliability of connections (reduced risk of leakage), which improves the network Water tightness.
- As the pipe is self-butting, there is virtually no need for a thrust block.
- Good hydraulic properties (very low roughness coefficient).
- Durability proven by experience and the ageing test theoretically 50 years at a Temperature of 20°C.

IV.4.1.2 Disadvantages :

- Nécessite une grande technicité pour la jonction.

IV.5. Pipe Selection Parameters :

The choice of pipe diameter is the result of a compromise between operating costs and investment costs. The optimum will therefore be to choose the diameter that gives the minimum cost price for the whole installation in operation. However, the flow velocity must be between 0.4 and 2 m/s.

The parameters involved in this study are :

- Flow rate .
- Diameter .
- Velocity .
- Head loss .

IV.6. Hydraulic calculation :**IV.6.1. Description of the pipeline route :**

The starting point of the supply system is the 2000 m³ buffer tank, fed by the Tilesdit dam pumping station, which provides the necessary head to feed the tank located at a ground level of 884 m. The supply pipe leaving the tank will go down by gravity to a suction tank at the first pumping station at a height of 864 m , then the pipe from the pumping station will go north and

up until it reaches a suction tank at the second pumping station at a height of 1072 m From there , a pipe will fill the existing water tower with a capacity of 1000 m³ , located at a height of 1279.4 m.

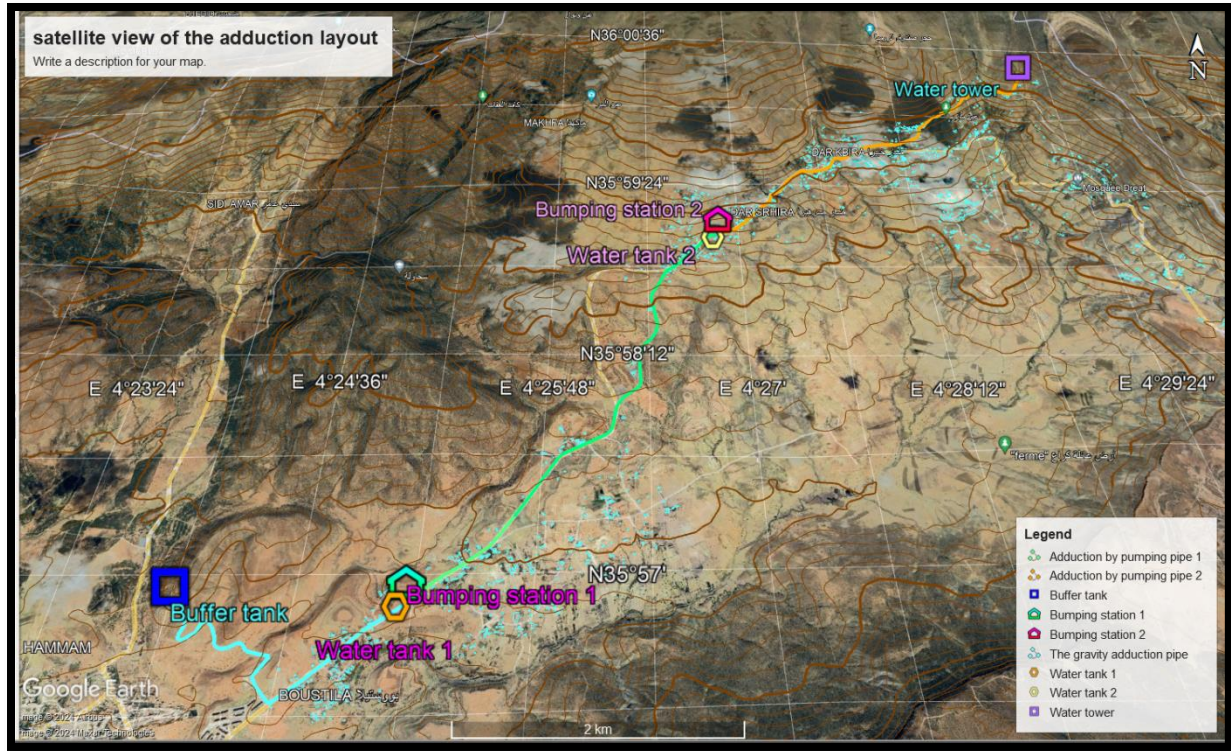


Figure.IV.1: Satellite view of the supply layout

(Source :Google Earth 20-06-2024 /10:34 pm)

IV.6.2.Calculation method :

IV.6.2.1.Gravity case :

The most commonly used formula for calculating the head loss for a flow in a pipe is :

$$\Delta H_t = \frac{K' * L_e * Q^\beta}{D a v e^m} \quad (IV.1)$$

ΔH_t : Total pressure drop (m) .

K' : Coefficient of friction .

L_e : Equivalent pipe length (m) .

$$L_e = L_g + L_{eq} \quad (IV.2)$$

L_g : Geometric length of pipe (m) ;

L_{eq} : Equivalent length of singular head losses (m) .

In the case of pipes, singular head losses are estimated at 10% of linear head losses.

losses.

$$\Delta H_t = 1,10 * \Delta H_{lin}$$

Therefore:

$$Le = 1,10 * Lg \quad (IV.3)$$

ΔH : Total pressure drop (m) .

ΔH_{lin} : Linear pressure drop (m) .

Q : Flow conveyed by the pipe (m³/s) .

D_{av} : Calculated advantageous pipe diameter (m) .

B : Exponent taking into account the flow regime .

m : Exponent taking into account the type of material .

Depending on the type of material, the coefficients K' , m and β are given in Table IV.1.

Table IV.1: Coefficients K' , m , β for different types of pipe.

Pipe	K'	m	β
Steel and cast iron	0,00179 - 0,001735	5,1 - 5,3	1,9 - 2
Plastic	0,001052	4,772	1,77

We then calculate the diameter of the gravity pipe (advantageous diameter) :

$$D_{ave} = \sqrt[m]{\frac{K' * Le * Q^B}{\Delta H_t}} \quad (IV.4)$$

In our case, we take: $K'=0.001052$, $B = 1.77$, $m = 4.772$.

IV.6.2.2 Discharge case :

- **Estimation of the discharge pipe diameter :**

We propose to make a first approximation of our diameter using one of the following two expressions following expressions :

$$Dec = \sqrt{Q} \quad (\text{Relation of Bonin}) \quad (IV.5)$$

$$Dec = 1,5 * \sqrt{Q} \quad (\text{Relation of Bresse}) \quad (IV.6)$$

With :

Q : flow conveyed by the pipe (m³/s).

Dec : economic diameter of the pipe (m) ;

- **Total manometric height (head) :**

The total head will be calculated as follows:

$$TMH = Hg + \sum \Delta h \quad (IV.7)$$

TMH : Difference in level between suction and discharge planes (m) ;

$\Sigma \Delta h$: Sum of linear and singular head losses (m)

In the case of adductions, Singular head losses are estimated at 10% of linear head losses.

$$\Sigma \Delta h = 1,10 * \Delta H_{lin} \quad (IV.8)$$

ΔH_{lin} : Linear head (pressure) losses (m).

$$\Delta H_{lin} = \frac{8 * \lambda * L * Q^2}{\pi^2 * g * D^5} \quad (\text{Darcy-Weisbakh formula}) \quad (IV.9)$$

g : Acceleration due to gravity (m/s²) ;

Q : Flow rate to be delivered (m³/s) ;

L : Geometric length of pipe (m);

λ : Coefficient of friction, which is a function of the roughness of the internal wall of the pipe and the flow regime :

$$\lambda = (1,14 - 0,86 \ln(\frac{\varepsilon}{D})) - 2 \quad (IV.10)$$

$$\frac{1}{\sqrt{\lambda}} = -2 * \text{Log} \left(\frac{\varepsilon}{3,75 * D} + \frac{2,51}{Re * \sqrt{\lambda}} \right) \quad (IV.11)$$

Re : Reynolds number ;

$$Re = (V * D) / \nu \quad (IV.12)$$

With :

V : flow velocity in the pipe (m/s) .

D : pipe diameter (m) .

ν : kinematic viscosity of water (m²/s).

ε : Absolute roughness of the pipe .

Note:

In the calculations introducing the coefficient, the latter is estimated by the Nikuradze formula

- **Pumping power**

The power absorbed by the pump is calculated using the following formula:

$$P = \frac{\rho * g * Q * TMH}{\eta} \quad (IV.13)$$

P : Power absorbed by the pump (kw).

g : Acceleration due to gravity (m/s²) .

TMH : Total manometric head (m) .

Q : Flow rate to be discharged (m³/s) .

η : Pumping station efficiency, varies (65%-90%) , An efficiency of 80% is used.

ρ : Water density (1 kg/l)

- **Energy consumed by pumping**

The energy consumed by the pumping station over the course of a year is the product of the pumping power and the number of pumping hours per year. and the number of pumping hours per year.

$$E = P * t * 365 \quad (IV.14)$$

E: Energy consumed during a year (KWh) ;

P: Pump power (KW) ;

t: Pumping time per day (h)

- **Annual energy expenditure:**

Annual energy expenditure is the product of the unit price per KWh and the energy consumed per year.

$$K = \delta * E \quad (IV.15)$$

K: Annual energy expenditure (DA) ;

E: Energy consumed per year (KW) ;

δ : The price of KWh, varies by time of day $\delta = 4,17$ (according to Sonelgaz).

- **Depreciation costs :**

Depreciation costs are given by the following relationship :

$$F = A * L * Price \ u \quad (IV.16)$$

With :

A: Annual amortisation.

L: Length of the discharge pipe (m) .

Pr: Price of the pipe (DA)

Tel that :

$$A = \frac{i}{(1+i)^n - 1} + i \quad (IV.17)$$

With :

i: Annuity rate; $i = 8\%$;

n: Depreciation period of the pipe, $n = 30$ years.

Note : $A = 0.089$.

IV.6.3.Sizing the gravity supply system :

To determine the advantageous diameter of the gravity-fed pipe, bearing in mind that the buffer reservoir is located at a height of 884 m, the calculation process is as follows :

- Calculation of the available load between the buffer reservoir and the first pumping station .
- Assume that the head loss is equal to the available load.
- The advantageous diameter is determined and standardised.
- Check whether the speed is acceptable

The flow velocity in the gravity pipe is determined using the continuity equation.

$$Q = S * V \quad (IV.18)$$

$$Q = V * \frac{\pi * D^2}{4} \quad \text{ainsi : } V = \frac{4 * Q}{\pi * D^2}$$

With :

Q: Maximum daily flow (Q = 2001.16 m³/d) ;

S: Pipe cross-section (m²) ;

V: Flow velocity (m/s);

D: Pipe diameter (m).

The results are shown in the table below:

Table IV.2: Sizing results for the gravity section :

Material	D(mm)	L(m)	V(m/s)	ΔH_d (m)	GE(m)
PEHD	160	3203.2	0.18	30	884

IV.6.4.Sizing of the backflow supply:

The choice of an economical pipe diameter is a compromise between capital expenditure and operating costs. In fact, by increasing the diameter, we increase the former and very quickly reduce pressure losses, and subsequently the expenditure of energy to circulate the water.

Pump stations are designed to deliver the operating flow to the reservoirs. The stations operate 20 hours a day.

IV.6.4.1 Pumped flow :

The pumped flow is calculated as follows:

$$Q_p = \frac{Q_{max,d}}{tf} \quad (IV.19)$$

Q_{max,d}: Maximum daily flow (m³/d) .

Q_{max,d} = 2001,16 (m³/d)

tf: Station operating time per day (**tf** = 20 h).

With $Q_p = 0,027794 \text{ m}^3/\text{s}$.

IV.6.4.2 The economic diameter :

Bonin $Dec = 0.166 \text{ m}$.

Bresse $Dec = 0.250 \text{ m}$.

The technical calculation of the economic diameter will be for the range of diameters between 150 and 315 mm.

IV.6.4.3. Calculation of operating costs :

Operating costs are calculated on the basis of the electrical energy consumed during pumping, for the for the range of diameters obtained. It is understood that the total manometric head varies according to diameter.

- The discharge section 1 (pump station 1_ pumping station 2) :

The following table shows the variation in TMH as a function of diameter :

Table IV.3: Variation in TMH as a function of diameter PS1-PS2 :

Diameter (mm)	L(m)	Hg(m)	Vitesse(m/s)	ϵ/D_{int}	λ	ΔH total (m)	TMH (m)	$V^2/2g$
160	5039	208	2.620513548	0.000625	0.01784971	298.3158632	506.3158632	0.350004651
200	5039	208	1.67828392	0.0005	0.01696845	93.0861329	301.086133	0.143559475
250	5039	208	1.0729191	0.0004	0.01615088	28.9529539	236.952954	0.058672548
315	5039	208	0.6759065	0.00031746	0.01536487	8.67610302	216.676103	0.023284893
400	5039	208	0.41899365	0.00025	0.01461121	2.49622678	210.496227	0.008947792

The operating costs for the different diameters are shown in the following table.

Table IV.4: Variation in operating costs as a function of diameter PS1-PS2 :

Diameter (mm)	TMH (m)	Power (KW)	Energy consumed (KWh)	Annual energy Expenditure
160	506.3158632	172.539725	1511447.991	6302738.122
200	301.086133	102.6025893	898798.6822	3747990.505
250	236.952954	80.74761325	707349.0921	2949645.714
315	216.676103	73.83777192	646818.882	2697234.738
400	210.496227	71.73182537	628370.7902	2620306.195

- The discharge section 2 (pumping station 2_ water tower) :

The following table shows the variation in TMH as a function of diameter :

Table IV.5: Variation in TMH as a function of diameter PS2-Water tower:

Diameter (mm)	L(m)	Hg(m)	Vitesse(m/s)	ϵ/D_{int}	λ	ΔH total (m)	TMH (m)	$V^2/2g$
160	4781	207	2.62051354	0.000625	0.01784971	283.041901	490.041901	0.350004651
200	4781	207	1.67828392	0.0005	0.01696845	88.3200638	295.320064	0.143559475
250	4781	207	1.0729191	0.0004	0.01615088	27.4705443	234.470544	0.058672548
315	4781	207	0.6759065	0.00031746	0.01536487	8.23188103	215.231881	0.023284893

400	4781	207	0.41899365	0.00025	0.01461121	2.36841839	209.368418	0.008947792
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The operating costs for the different diameters are shown in the following table.

Table IV.6: Variation in operating costs as a function of diameter PS2-Water tower :

Diameter (mm)	TMH (m)	Power(KW)	Energy Consumed(KWh)	Annual energy expenditure
160	490.0419016	167.018	1463077.68	6101033.926
200	295.3200638	100.6521433	881712.7464	3676742.152
250	234.4705443	79.9131712	700039.378	2919164.206
315	215.231881	73.35617448	642600.0842	2679642.351
400	209.3684184	71.35776613	625094.0302	2606642.106

IV.6.4.4. Calculation of depreciation costs :

- **The discharge section 1 (pump station 1_ pumping station 2) :**

The capital expenditure corresponding to the different diameters is shown in the table below.

Table IV.7: Variation in depreciation costs as a function of diameter PS1-PS2 :

Diameter	Price per ml of pipe(DA)	Pipe length(m)	Total cost of the pipe(DA)	A	Depreciation costs
160	2750.54	5039	13859971	0.089	1233537.4
200	3173.48	5039	15991166	0.089	1423213
250	4940.17	5039	24893517	0.089	2215523
315	7870.28	5039	39658341	0.089	3529592.3
400	12683.91	5039	63914222	0.089	5688365.8

Source : (des prix :<http://www.algerie.prix>

construction.info/espaces_urbains/Reseaux_divers/Distribution_d_eau/Eau_potable/Tube_en_polyethylene.html)

- **The discharge section 2 (pumping station 2_ water tower) :**

The capital expenditure corresponding to the different diameters is shown in the table below.

Table IV.8: Variation in depreciation costs as a function of diameter PS2-Water tower :

Diameter (mm)	Price per ml of pipe(DA)	Pipe length(m)	Total cost of the pipe(DA)	A	Depreciation costs
160	2750.54	4781	13150331	0.089	1170379.5
200	2666.79	4781	12749923	0.089	1350344.301
250	4151.4	4781	19847843	0.089	2102086.797
315	6613.68	4781	31620004	0.089	3348874.973
400	10658.75	4781	50959484	0.089	5397117.86

Source : (des prix :<http://www.algerie.prix>

construction.info/espaces_urbains/Reseaux_divers/Distribution_d_eau/Eau_potable/Tube_en_polyethylene.html)

IV.6.4.5. Costs balance :

- The discharge section 1 (pump station 1_ pumping station 2) :

The balance of costs is shown in table below.

Table IV.9: Costs Balance (PS1-PS2) :

Diameter (mm)	Total (Million DA)
160	7.536275546
200	5.171204254
250	5.165168694
315	6.22682708
400	8.308671997

The graphical interpretation of the results of the balance sheet is shown in Figure below.

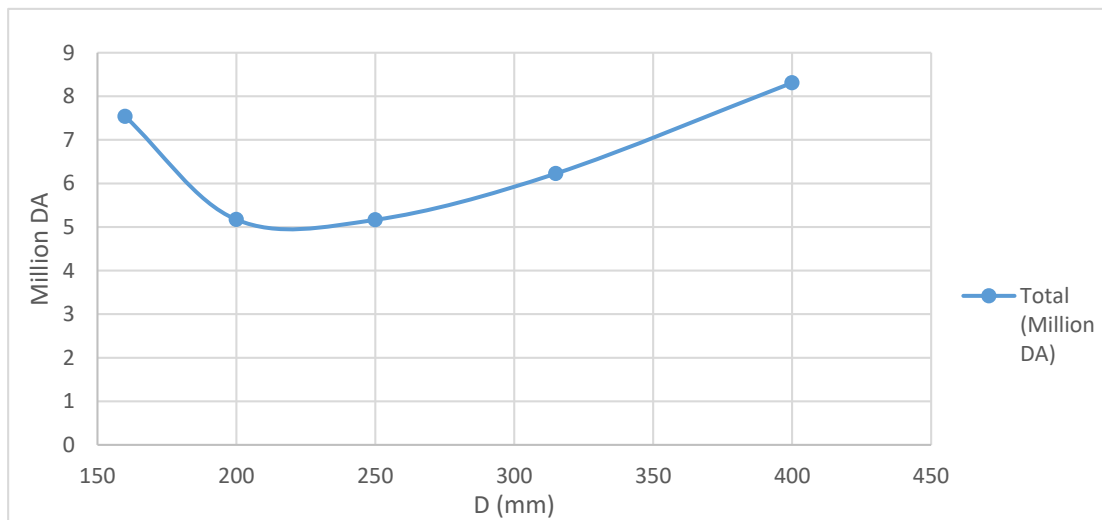


Figure IV.2: Economic diameter curve PS1-PS2.

Depending on the costs combination , we opt for a **ND 250** diameter with a velocity of **1,07 m/s**.

- The discharge section 2 (pumping station 2_ water tower) :

The balance of costs is shown in table below.

Table IV.10: Costs Balance (PS2-Water tower) :

Diameter (mm)	Total
160	7.271413426
200	5.027086452
250	5.021251006
315	6.028517351
400	8.003760006

The graphical interpretation of the results of the balance sheet is shown in Figure below.

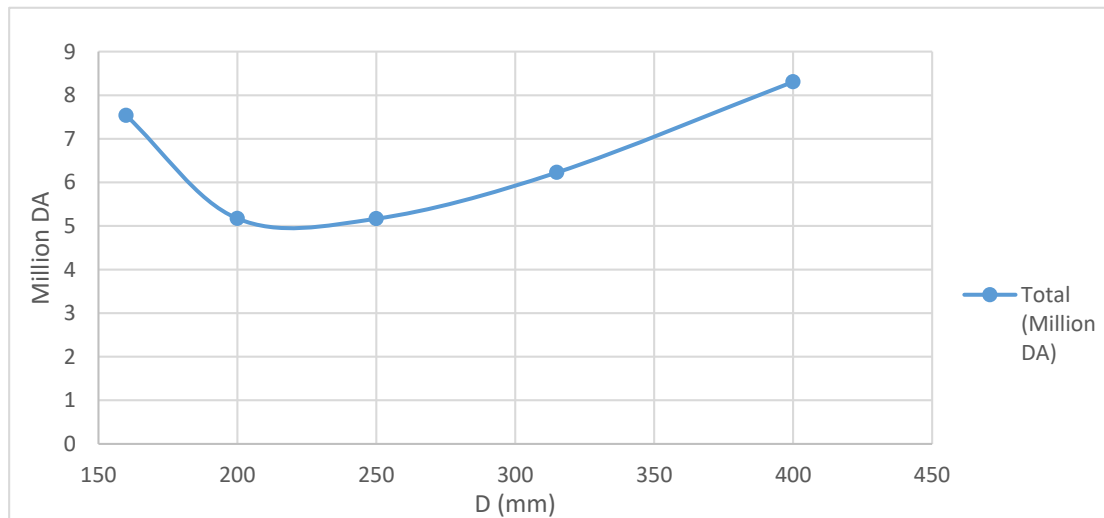


Figure IV.3 : Economic diameter curve PS2-Water tower.

Depending on the costs combination , we are opting for a **ND 250** diameter with a velocity of **1.07 m/s**.

Conclusion :

In this chapter, we conducted a water supply study for our study area. This led us to project a mixed type of conveyance with:

- A gravity section which will transport the water from the buffer reservoir to a water tank at the first pumping station located at a height of 884 m with a diameter of 160 mm.
- A backflow section that will transport the water from the first supplying pipe

pumping station to a suction water tank of the second pumping station located at a height of 1072 m with a diameter of 250 mm, the sourcing pipe from the latter to an existing water tower located at a height of 1279.4 m with a diameter of 250 mm.

CHAPTER V :
Pumping station study

Introduction:

The pumping station is a fundamental element upstream of any backflow supply to raise the water to higher altitudes when the topography does not allow gravity flow. This is achieved by means of pumps and other specialised equipment.

The aim of this chapter is to design the two planned pumping stations delivering water to the existing water tower by selecting the various types of equipment (hydro mechanical, energy and auxiliary...).

V.1.The location of the pumping station :

The choice of location for a pumping station is important to ensure efficient and reliable operation of the pumping system. The following criteria are important to consider when selecting the location of a pumping station:

- Avoid areas prone to flooding, landslides or other natural hazards (stable terrain).
- The station must be positioned in such a way as to guarantee sufficient elevation to convey the water to its final destination, taking into account the topography of the land
- The location must be accessible to a reliable power supply.
- Ensure the safety of operators and people living near the station by implementing appropriate safety measures.

V.2. Choice of pump type :

The criteria for choosing the type of pump are:

- Ensuring the flow rate called for F_{app} and the head .
- Best efficiency .
- Reduced size .
- Highest speed of rotation .
- Minimum power consumption .
- Minimum net suction load (NPSH)_r required .
- Being mass-produced .

V.3.Choosing the number of pumps :

The criteria for choosing the number of pumps are :

- Minimum number of pumps n .
- Best efficiency .

- Minimum net suction head (NPSH)_r .
- Minimum number of stages .
- Minimum power consumption

V.4. Pump coupling :

There are two types of pump coupling:

- **Series coupling:** is used to increase the head of the pumps.
- **Parallel coupling:** is used to increase the flow delivered by the pumps.

V.5. Pumping study:

The pumping study will enable us to choose the ideal pump for our supply system to ensure the required flow rate and head.

V.5.1.The discharge section 1 (pumping station 1_ pumping station 2) :

The pump at the first station will deliver a flow rate of F_p to the water tank at the second pumping station. The flow rate is 0.027794 m³/s and the TMH is 236.95 m.

V.5.1.1.Variants for choosing the type of pump (PS1) :

The Caprari software was used to select the optimum number and type of pump, and the results are shown in the following table.

Table V.1: Pump selection optionsPS1.

Pumps Number	F_p (l/s)	TMH (m)	Efficiency (%)	P_{shaf} (kw)	N (tr/min)	N_{psh r} (m)
1	31,9	246	76,6	102	2900	3,52
2	13.9	237	71.8	45.4	2900	2.36
3	8.91	235	69	30.5	2900	2
4	6.95	237	56.1	28.8	2900	1.79

According to this table, we have :

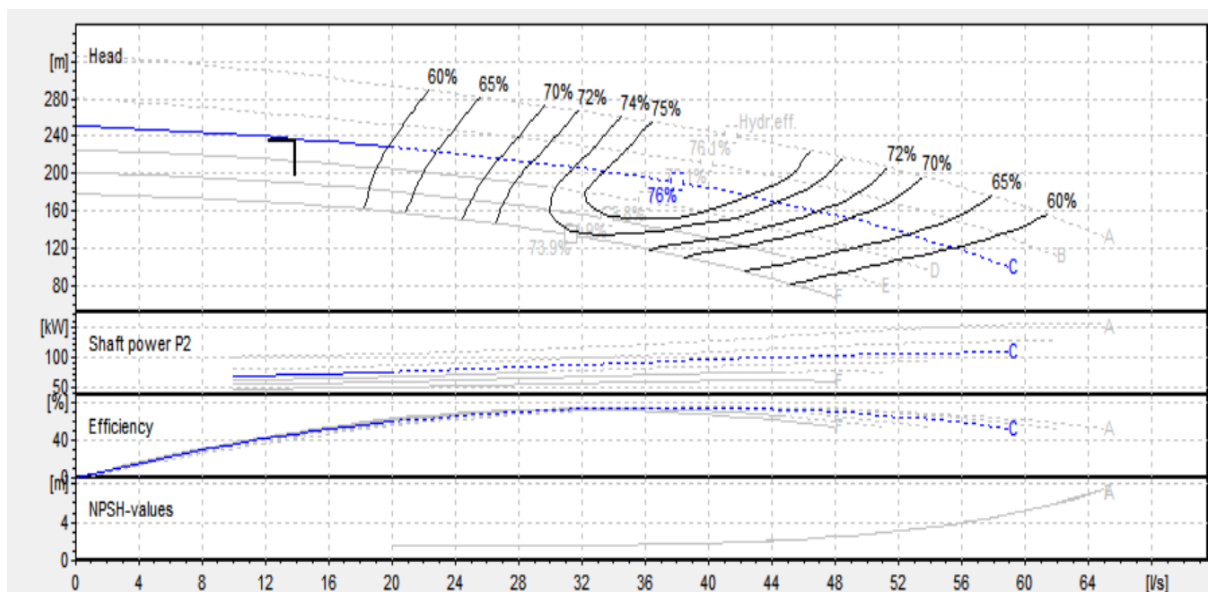
- Variant (1) with one pump has a sufficient flow, sufficient TMH ,maximum efficiency, maximum power consumption and maximum (NPSH_r).
- Variant (2) with two pumps has a sufficient flow and sufficient TMH .
- Variant (3) with three pumps has insufficient flow rate and sufficient TMH .

- Variant (4) with four pumps has a sufficient flow rate, a sufficient TMH , minimum power consumption and minimum NPSHr .

We therefore choose variant (2) with two horizontal multistage pumps for high head and a back-up pump of the same type.

V.5.1.2.Characteristic curves for pumps (PS1) :

- Height versus flow curve: $H = f(Q)$;
- Efficiency curve as a function of flow rate: $\eta = f(Q)$.
- Power curve as a function of flow rate: $P = f(Q)$.
- Net suction load as a function of flow rate:(NPSH)_r= $f(Q)$.



Figures V.1: Shows the characteristic curves of the chosen pump PM 125 .

V.5.2.The discharge section 2 (Pumping station 2_ Water tower) :

The pump at the first station will deliver a flow rate of F_p to the water tower .The flow rate is 0.027794 m³/s and the TMH is 234.47 m.

V.5.2.1.Variants for choosing the type of pump (PS2) :

The Caprari software was used to select the optimum number and type of pump, and the results are shown in the following table.

Table V.2: Pump selection options PS2.

Pumps Number	Fp (l/s)	TMH (m)	Efficiency (%)	Pshaf (kw)	N (tr/min)	Npsh r (m)
1	29.2	237	76.6	90.8	2900	3.37
2	13.9	236	71.9	45.7	2900	2.39
3	9.07	233	69.3	30.7	2900	2.01
4	5.9	227	67.9	19.5	2900	1.95

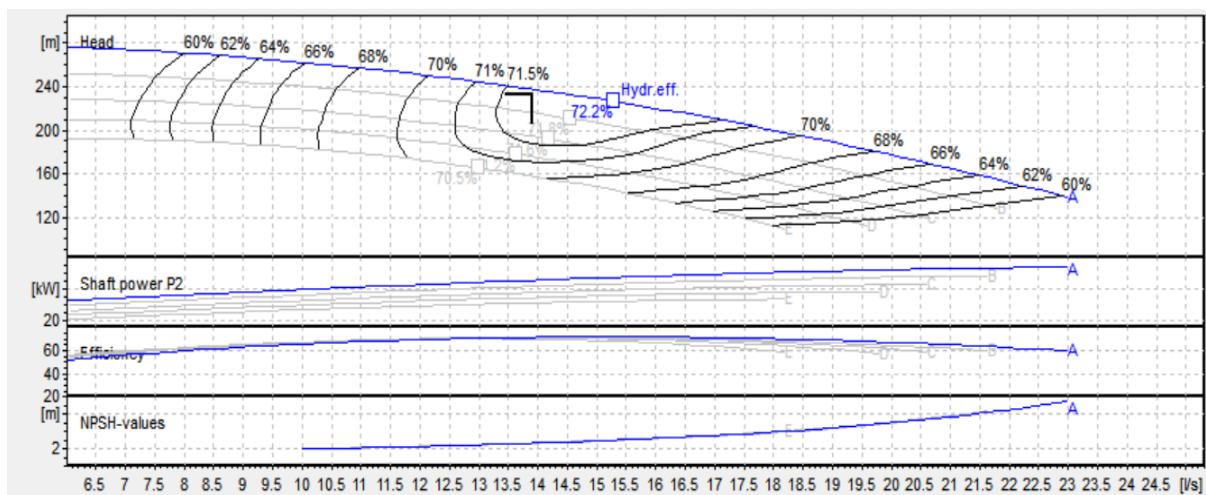
According to this table, we have :

- Variant (1) with one pump has sufficient flow rate , sufficient TMH , minimum NPSHr and maximum efficiency .
- Variant (2) with two pumps has a sufficient flow rate and sufficient TMH .
- Variant (3) with three pumps has insufficient flow rate, and a sufficient TMH .
- Variant (4) with four pumps has a sufficient flow rate, a sufficient TMH and minimum power consumption .

We therefore choose variant (2) with two horizontal multistage pumps and a back-up pump of the same type.

V.5.2.2.Characteristic curves for pumps (PS2) :

- Height versus flow curve: $H = f(Q)$.
- Efficiency curve as a function of flow rate: $\eta = f(Q)$.
- Power curve as a function of flow rate: $P = f(Q)$.
- Net suction load as a function of flow rate: $(NPSH)r = f(Q)$.



Figures V.2: Shows the characteristic curves of the chosen pump HMU50-2 .

V.6.Pump operating points :

The duty point of a pump is the intersection between the characteristic curve of the installed and operating pump [H-Q] and the characteristic curve of the pipe delivering a flow from the operating pump [Hc-Q].

- **Pipe characteristic curve:**

The pipe characteristic curve is a parabola whose equation is as follows:

$$H_c = H_g + \Delta h_t \quad (V.1)$$

With:

H_g: Difference in level between the suction and discharge planes (m) .

Δh_t: Sum of linear and singular head losses (m).

In the case of pipes, singular head losses are estimated at 15% of linear head losses .

$$\sum \Delta h = 1.15 * \Delta h_{lin} \quad (V.2)$$

$$\Delta H_{lin} = \frac{8 \lambda L Q^2}{\pi^2 g D^5} \quad (\text{DARCY-WEISBAKH formula}) \quad (V.3)$$

g: Acceleration due to gravity (m/s²).

Q: Flow rate to be delivered (m³/s) .

L: Geometric length of the pipe (m) .

λ: Coefficient of friction which depends on the roughness of the internal wall of the pipe and the flow regime,

$$\lambda = (1.14 - 0.86 \ln(\frac{\epsilon}{D}))^{-2} \quad (\text{NIKURADZE formula}) \quad (V.4)$$

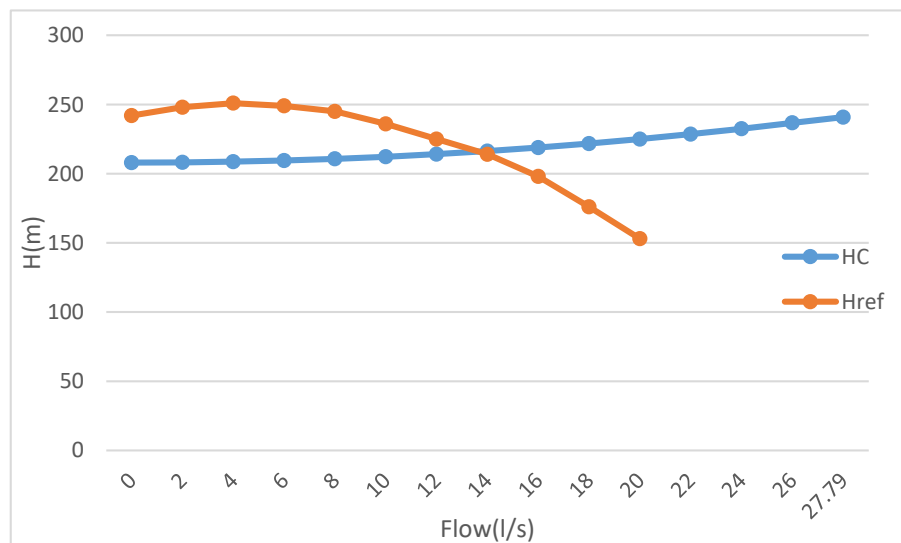
λ: Coefficient of friction which is a function of the roughness of the internal wall of the flow regime pipe .

V.6.1. The discharge section 1 (pumping station 1_ pumping station 2) :

The characteristic curve of the discharge pipe is given in the following tables:

Table V.3: Characteristic curve of the discharge pipe (PS1_PS2) :

Q (l/s)	R	ΔH (m)	H_c (m)
0	4723.56	0	208,00
2	4723.56	0.17	208.17
4	4723.56	0.68	208.68
6	4723.56	1.53	209.53
8	4723.56	2.72	210.72
10	4723.56	4.25	212.25
12	4723.56	6.12	214.12
14	4723.56	8.33	216.33
16	4723.56	10.88	218.88
18	4723.56	13.77	221.77
20	4723.56	17.00	225.00
22	4723.56	20.57	228.57
24	4723.56	24.48	232.48
26	4723.56	28.33	236.73
27,79	4723.56	32.83	240.83



Figures V.3: Operating point for PS1 .

Table V.4 : peak operating characteristics PS1 :

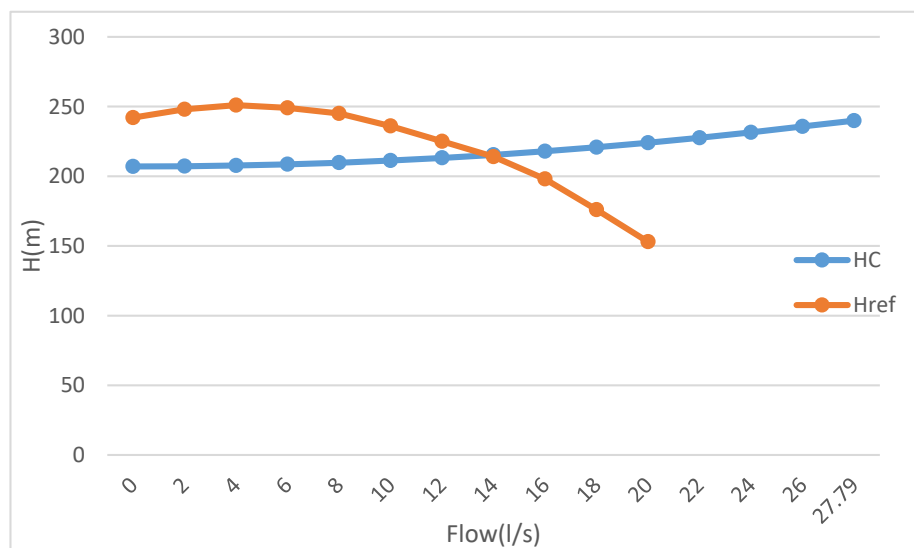
Qop (l/s)	Hop(m)	Pop (kw)	Effeciency op (%)	NPSHr op (m)
13.9	237	45.4	70.2	2.85

V.6.2.The discharge section 2 (Pumping station 2_ Water tower) :

The characteristic curve of the discharge pipe is given in the following tables:

Table V.5: Characteristic curve of the discharge pipe (PS2_Water tower) :

Q (l/s)	R	ΔH (m)	H_c (m)
0	4723.56	0	207
2	4723.56	0.17	207.17
4	4723.56	0.68	207.68
6	4723.56	1.53	208.53
8	4723.56	2.72	209.72
10	4723.56	4.25	211.25
12	4723.56	6.12	213.12
14	4723.56	8.33	215.33
16	4723.56	10.88	217.88
18	4723.56	13.77	220.77
20	4723.56	17.00	224.00
22	4723.56	20.57	227.57
24	4723.56	24.48	231.48
26	4723.56	28.33	235.73
27,79	4723.56	32.83	239.83



Figures V.4: Operating point for PS2 .

Table V.6: Peak operating characteristics PS2 :

Qop (l/s)	H op (m)	Pop (kw)	efficiency op (%)	NPSHr op (m)
14	236	45.7	71.9	2,39

As **Qapp** included in (at **0.9*Qop -1.1Qop**) ,then the pump is well adapted to the operating conditions of the network

V.7.Determining the volumes of the projected water tanks :

To calculate the useful volume of the projected water tank, we use the following formula:

$$VU = \frac{Q_p * (\frac{1}{n})}{(4 * (N - 1))} \quad (V.10)$$

Where:

Q_p: Pumped flow rate (m³/h) .

n: Number of starts per hour .

N: Number of identical pumps N= 3 pumps .

Table V.7: Variation in the number of starts per hour as a function of pump power.

This table shows how the number of starts per hour changes depending on the pump's power rating

Number of start-ups	4	6	8	10
Pump power	< 30 KW	> 15 KW	< 8 KW	< 4 KW

Source : Polycopy Mme MOKRANE , 2024

According to the table, we can assume 6 starts per hour for the two water tanks. For the total volume, and to be on the safe side, we increase the calculated volume by 30% to obtain :

Table V.8: The number of start- ups and the volume of water.

Tanks	Pump Power (KW)	Number of start-ups	Volume (m ³)	Increased Volume (m ³)
Water tank 1	136.2	6	2.1	2.75
Water tank 2	137.1	6	2.1	2.75

V.7.1.Sizing water tanks :

We will take a water tanks with a square cross-section, the main dimensions of which will be determined from the following relationship:

$$V = S * H \quad (V.5)$$

Where:

$$S = a^2$$

V: Volume of the water tank (m3).

S: Cross-section of the water tank (m).

H: Height of water in the water tank (m) .

a: Length of the side of the square (m) .

V.6.1.1 Calculating the cross-section of the water tanks :

We take a water height inside the tank of 1.5 m.

$$\mathbf{H= 1\ m\ .}$$

$$\mathbf{V=S*H}$$

$$\mathbf{S=V/H}$$

$$\mathbf{S=2.75/1.5}$$

$$\mathbf{S= 1.83\ m^2}$$

V.7.1.2 Calculating the length of the side of the section :

Now let's determine the side of this section :

$$\mathbf{S = a^2}$$

$$\mathbf{a = \sqrt{s}}$$

$$\mathbf{a = \sqrt{1.83}}$$

$$\mathbf{a = 1.35\ m}$$

The following table summarises the characteristics of the planned water tanks

Table V.9: Characteristics of the planned water tanks.

Water tank	Type	Volume (m3)	Section (m2)	Height (m)
Planned	On ground	2.75	1.83	1.35
Planned	On ground	2.75	1.83	1.35

Conclusion:

In this chapter, we have proposed a variant for each pumping station as follows:

- A variant with two horizontal multistage pumps and a back-up pump of the same type for the first pumping station to ensure the delivery of the required flow to the second pumping station

- A variant with two horizontal multistage pumps and a back-up pump of the same type for the second pumping station to ensure the delivery of the required flow to the water tower

Also we determined the capacity of the square section water tanks with a capacity of 2.75 m^3 for the two pumping stations .

CHAPTER VI :

Sizing of the distribution network .

VI :SIZING OF THR DISTRIBUTION NETWORK**Introduction :**

Once we have assessed the needs of our conurbation, we need to size the distribution network so that it can meet demand in terms of flow and pressure at all times

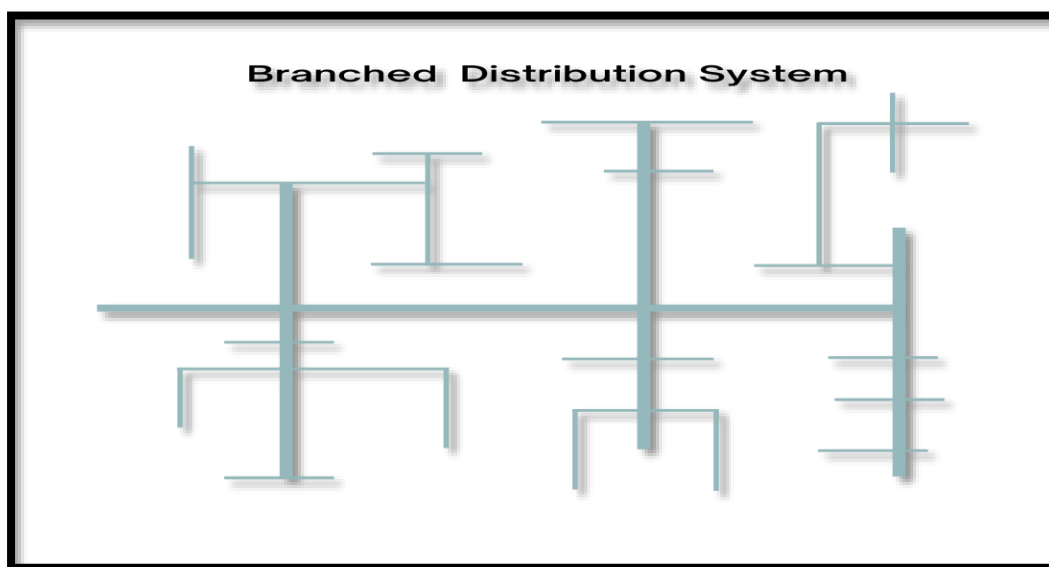
To do this, we need an optimum diameter for all sections of the network.

VI.1. Types of networks :

Depending on the links between the various distribution trunks, there are generally three types of network: branched networks, meshed networks and mixed network.

VI.1.1.Branched (ramified) network :

The branched network, in which the service conditions do not include any return feed, has the advantage of being economical, but it lacks safety and flexibility in the event of a break. An accident on the main line deprives subscribers downstream.



FigureVI.1: Diagram of a branched network

VI.1.2. Meshed (grid) network :

The meshed network, on the other hand, allows a return feed and therefore avoids the disadvantage of a branched network. A simple to isolate the damaged section. It is of course more expensive to build, but because of the safety it provides, it should be preferred to the branched preferred to the branched network

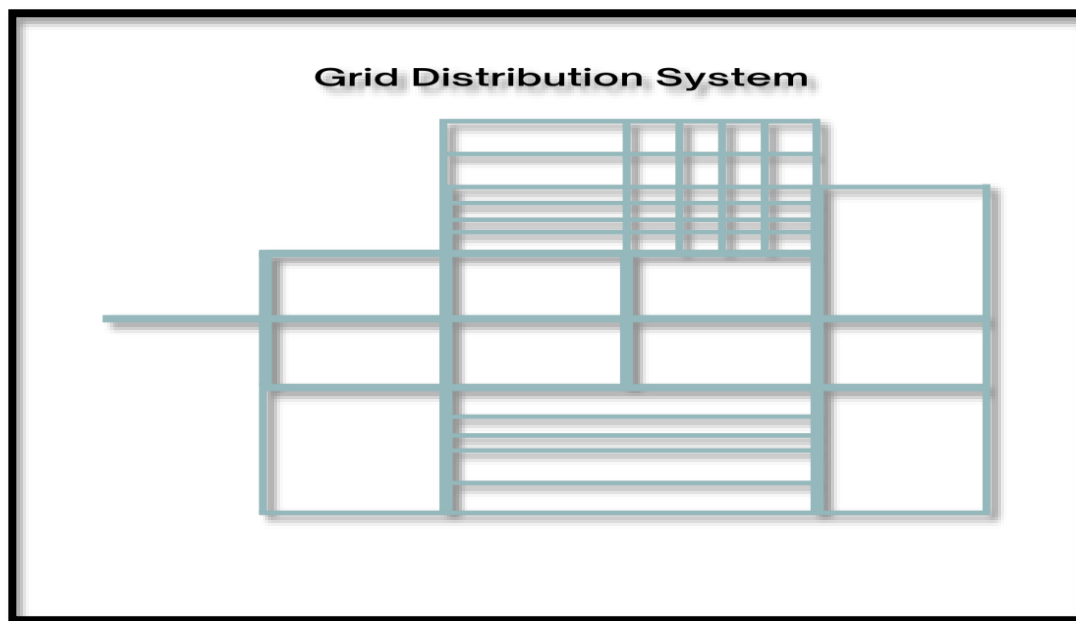
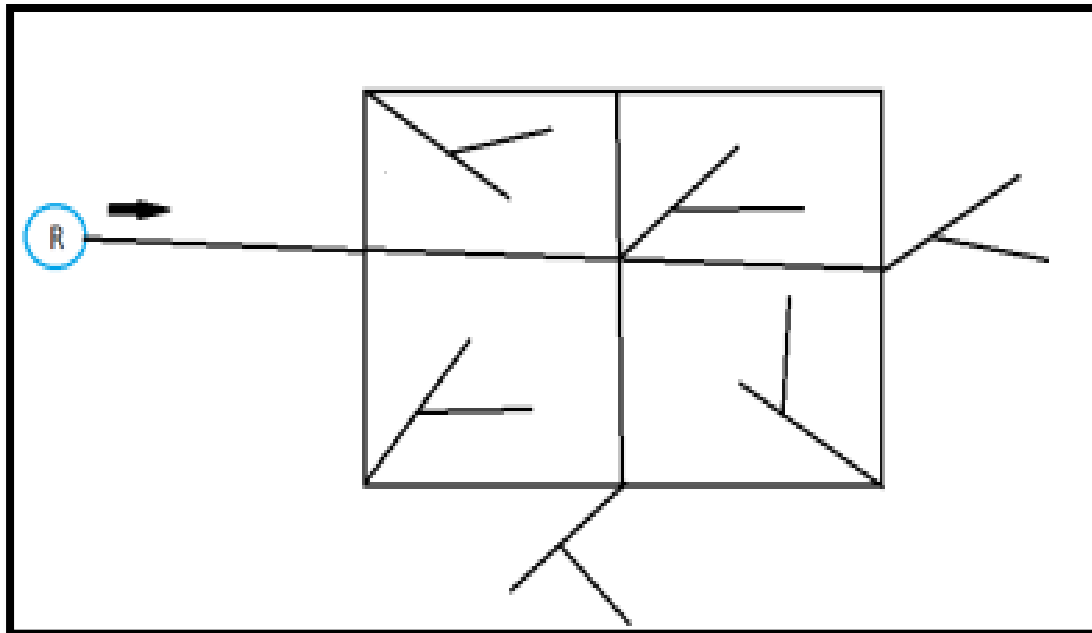


Figure VI.2 : Schematic diagram of a meshed network

VI.1.3.Mixte network :

It's a network made up of several pipes, some of which are meshed and others branched off depending on the configuration of the site and the pressure zones.



FigureVI.3: Schematic diagram of a mixed network .

VI.2. Choice of network types:

When designing a distribution network, a number of factors must be taken into account, which can influence the network:

- The location of neighbourhoods .
- the location of consumers .
- the terrain .
- The need to provide a flexible and regular service .

VI.3. Principle for laying out a network :

To lay out a network, there are a number of conditions that must be met:

- First of all, we need to identify the major consumers, (concentrated flow) .
- Identify the districts with a high population density .
- Determine the main route (direction) for distribution to these consumers .
- Following this direction, lay out the main pipes in parallel, these main pipes must be well distributed to ensure good water distribution .
- To supply the interior of the districts, these main pipes linked together by secondary pipes to form loops (mesh).

VI.4 Choosing the type of material:

In order to choose the right type of material, the following parameters must be taken into account:

- The diameter .
- The operating pressure supported by the material .
- The laying conditions .
- The price of the pipe .
- The service life of the material .
- The availability of the material on the market.

The materials to be used include : steel, ductile cast iron, PVC (polyvinyl chloride) and PE (polyethylene).

VI.4.1.HDPE pipes :

VI.4.1.1.Advantages :

- Good resistance to internal, external and microbiological corrosion and scaling, thanks to the chemical inertness of the pipe, which prevents the dissolution and abrasion of metals even if the water is not perfectly balanced.
- Market availability .

- Easy to install (good flexibility), as they are available in coils or drums up to Ø160, so they adapt well to the terrain; they can be wound in coils for small diameters.
- Reliable connections (reduced risk of leakage), which improves the network's water tightness.
- As the pipe is self-butting, there are virtually no solid stops..
- Good hydraulic properties (very low roughness coefficient)
- Durability proven by experience and ageing test theoretically 50 years at a temperature of 20°C.

VI.4.1.2 Disadvantages :

- The joint requires a high level of technical expertise.

VI.5 Network parameters :

VI.5.1 Flow rate :

The pipes must withstand the highest instantaneous flows as well as the peak flow and the peak flow plus fire.

VI.5.2 The choice of diameter :

Once you have calculated the theoretical diameters, you should refer to the catalogues where you will find the standard and commercial diameters.

VI.5.3.Velocity :

Knowing that high flow velocities of over 2 m/s favour the degradation of the internal walls of the pipe, and that low velocities of less than 0.5 m/s favour the formation of deposits in the pipes. It is therefore necessary for the flow velocity in the pipes to be between 0.5 and 2 m/s, as far as possible. However, the case of a fire is an exception, as it is possible to exceed 2 m/s as the velocity does not last long (as the time required to fight and control the fire is set at 2 hours at a rate of 17 l/s).

VI.5.4.The pressure :

It is necessary to check the ground pressures at each node, so the maximum ground pressure must not exceed 6 bar or fall below 1 bar.

VI.6. Hydraulic calculation of the branched network :

Flow rates in a branch network are determined as follows:

- The length of each section of the branch network is determined.

- Calculate the en-route flows during the hours under consideration (peak hour and fire hour).
- The specific flow rate is determined by considering the en-route flow rates.
- Knowing the specific flow rate, we determine the flow rate assumed to be concentrated at the nodes.

VI.6.1.Calculation of flow rates :

In this chapter, we look at the case of peak hour and fire peak hour.

✓ The specific flow :

The specific flow rate is calculated as a function of the linear distance as follows:

$$Q_{sp} = Q_{maxh} / (\sum L_i) \quad (VI.1)$$

Where:

Q_{sp} = The specific flow rate (l/s/lm) .

Q_{maxh} = The maximum hourly flow rate (peak) (l/s) .

$\sum L_i$ = The sum of the lengths of the sections with a consumption (lm) .

✓ The en-route flow rate :

The en-route flow rate is calculated as follows:

$$Q_{ri} = Q_{sp} * L_i \quad (VI.2)$$

Where:

Q_{ri} = the en-route flow rate of section i (l/s) .

Q_{sp} = the specific flow rate (l/s/lm) .

L_i : Length of section i (lm) .

✓ The flow at the node :

The flow at the node is calculated as follows:

$$Q_{ni} = (\sum F_{ri}) / 2 + F_{conc} \quad (VI.3)$$

With:

Q_{ni} : Flow at the node (l/s) .

$\sum Q_r$: The sum of the en-route flows of the sections adjacent to node i (l/s) .

Q_{conc} : Concentrated flow (l/s) .

VI.6.1.1. Peak case :

The following table summarises the calculation of flow rates:

Table VI.1: Calculation of specific flow rate, en route flow rate and nodal flow rate :

Knots	Sections	Lengh	Q specific	Q route	Q nodal
1	1_2	1328	0.0031433	4.17430586	5.82218226
	1_3	999.5	0.0031433	3.14173096	
	1_4	1377	0.0031433	4.32832769	
2	2_1	1328	0.0031433	4.17430586	2.08715293
3	3_1	999.5	0.0031433	3.14173096	1.57086548
4	4_1	1377	0.0031433	4.32832769	5.19336457
	4_5	700.4	0.0031433	2.20156915	
	4_6	1227	0.0031433	3.8568323	
5	5_4	700.4	0.0031433	2.20156915	1.10078457
6	6_4	1227	0.0031433	3.8568323	7.8367249
	6_7	3078	0.0031433	9.67508543	
	6_8	681.3	0.0031433	2.14153207	
7	7_6	3078	0.0031433	9.67508543	4.83754271
8	8_6	681.3	0.0031433	2.14153207	3.75326048
	8_9	595.8	0.0031433	1.87277969	
	8_10	1111	0.0031433	3.4922092	
9	9_8	595.8	0.0031433	1.87277969	0.93638985
10	10_8	1111	0.0031433	3.4922092	3.75891842
	10_11	667.2	0.0031433	2.0972115	
	10_12	613.5	0.0031433	1.92841615	
11	11_10	667.2	0.0031433	2.0972115	1.04860575
12	12_10	613.5	0.0031433	1.92841615	0.96420808

VI.6.1.2. Peak case + Fire:

The only thing we have to ensure is that the fire flow given by the reservoir (17 l/s) is at the most unfavourable point, which is node 11 so:

$$Q_n(11) = 18.05 \text{ l/s}$$

VI.6.2. EPANET software :**VI.6.2.1. Presentation :**

Pressurised networks, consisting of pipes, nodes (pipe junctions), pumps, valves, tanks and reservoirs, are analysed over long periods. EPANET calculates various parameters, such as the flow rate in each pipe, the pressure at each node, the water level in the tanks, and the concentration of chemical substances at different points in the network. This analysis is carried out in a simulation divided into several stages. The software is also capable of determining residence times and tracing the origin of the water.

The aim of EPANET is to provide a better understanding of the flow and use of water in distribution systems. It can be applied to a variety of distribution system analyses. EPANET

provides an integrated environment for editing network data, running hydraulic and quality simulations, and displaying results in a variety of formats, such as colour-coded maps, tables and graphs.

VI.6.2.2 Hydraulic modelling :

The main factors in the modelling are :

✓ Demand nodes:

These are the points in the network where the arcs meet. They are water entry or exit points and may also have no flow. The minimum input data required for demand demand nodes are :

- The altitude above a certain reference plane (usually sea level) .
- Water demand (flow taken from the network)

The results calculated at the demand nodes, at each time interval of a simulation are :

- The hydraulic head (or piezometric height): internal energy per specific weight of fluid or the sum of altitude and pressure head.

✓ The pipes :

These are arcs that carry water from one point in the network to another. EPANET assumes that all pipes are full at all times. The water flows from the end with the highest hydraulic load (altitude + pressure) to the end with the lowest hydraulic load. The basic data for pipes are :

- The initial and final node .
- The diameter .
- Length .
- Roughness coefficient (to determine head loss) .

Calculated pipe values include:

- Flow rate .
- Flow velocity .
- Head loss .

The head loss or hydraulic head lost due to friction between the water and the pipe walls can be calculated using one of these three formula:

- Hazen-Williams formula .
- Darcy-Weisbach formula.

- Chezy-Manning formula .

VI.6.3 Distribution network :

Once we have drawn up our distribution network according to the ground plan, we need to lay out our supply tank. We carry out a study of the network for one variant

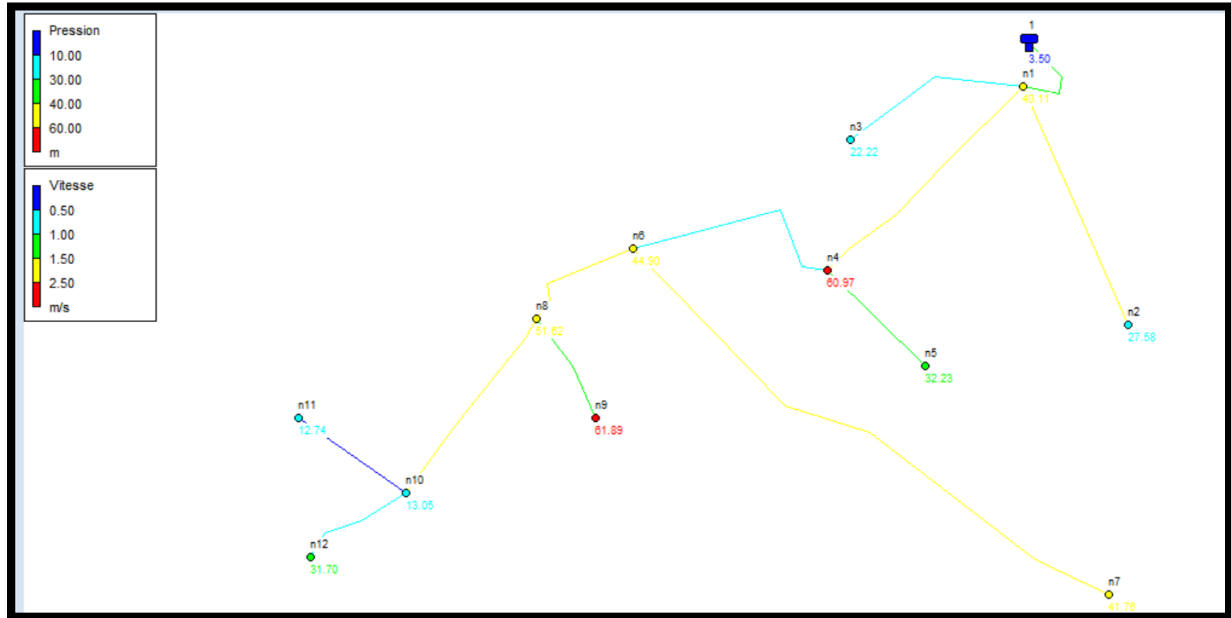


Figure VI.4 :Diagram of the network simulated on EPANET (peak case)

Table VI.2: Condition of pipes (peak case).

Pipe	Diameter (mm)	Lengh (m)	Flow (l/s)	Velocity (m/s)	Head loss gradient (wcm/km)
Pipe 1	250	506	38.50	1.17	6.36
Pipe 11	160	1377	29.02	2.16	25.96
Pipe 12	250	1227	22.73	0.69	2.33
Pipe 4	110	681.3	10.45	1.64	33.70
Pipe 9	75	1111	5.76	2.05	26.53
Pipe 10	75	613.5	0.96	0.78	22.99
Pipe 7	63	999.5	1.57	0.80	17.90
Pipe 2	50	1328	2.09	1.70	15.31
Pipe 8	40	704	1.10	1.40	23.86
Pipe 3	75	3078	4.84	1.58	12.33
Pipe 5	40	595.8	0.94	1.20	19.49
Pipe 6	75	667.2	1.05	0.37	3.46

Table VI.3: Node status (peak case).

Junction	Elevation (m)	Demand (l/s)	Hydraulic Grid (m)	Pressure (wcm)
Junction n1	1239.39	5.82	1282.99	40.11
Junction n2	1117.36	2.09	1144.94	27.58
Junction n6	1182.19	7.84	1227.10	44.90
Junction n7	1024.27	4.84	1066.04	41.76
Junction n8	1152.51	3.75	1204.14	51.62
Junction n9	1100.84	0.94	1162.74	61.89
Junction n10	1094.96	3.75	1108.01	13.05
Junction n11	1092.96	1.05	1105.70	12.74
Junction n3	1239.36	1.57	1261.59	22.22
Junction n4	1168.99	5.19	1229.96	60.97
Junction n5	1131.99	1.10	1164.22	32.23
Junction n12	1062.20	0.96	1093.91	31.70
Reservoir 1	1282.02	-38.90	1282.69	3.50

• **Case (Peak + fire):**

In this case, it is assumed that there is a concentrated fire flow (8 l/s) at the most unfavourable point in our network (in our case this is node 11).

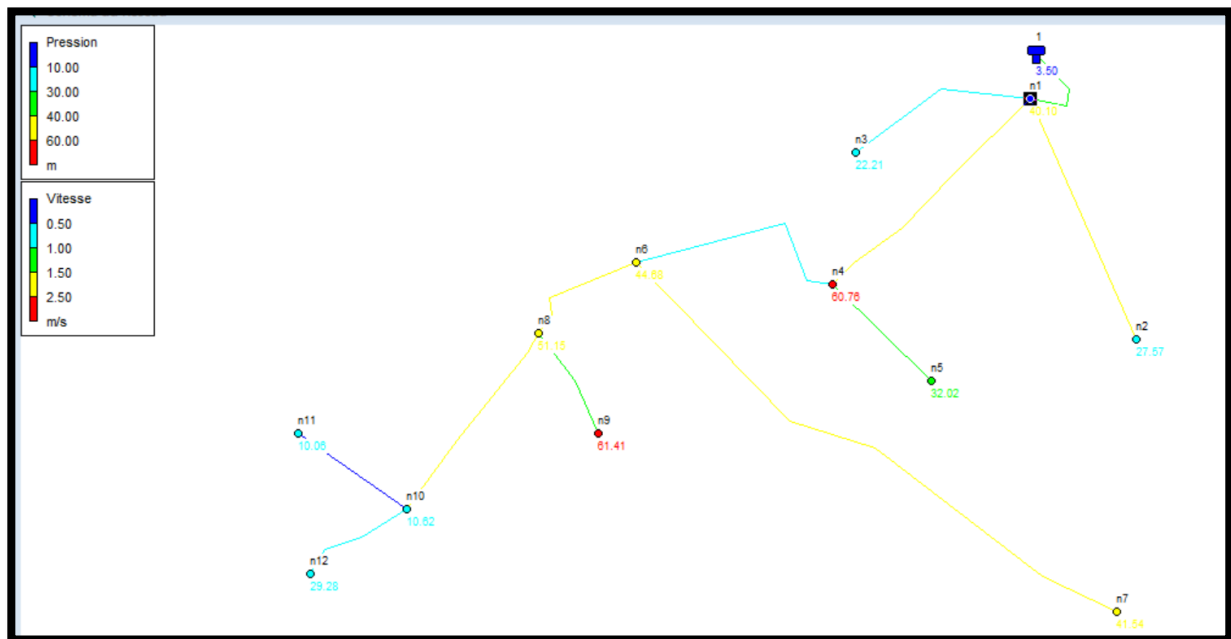


Figure VI.5: Diagram of the network simulated on EPANET (peak case +fire)

Table VI.4: Condition of pipes (peak case+fire).

Pipe	Diameter (mm)	Lengh (m)	Flow (l/s)	Velocity (m/s)	Head loss gradient (wcm/km)
Pipe 1	250	506	46.50	1.17	6.37
Pipe 11	160	1377	29.43	2.16	16.10

CHAPTER VI:**SIZING OF THE DISTRIBUTION NETWORK**

Pipe 12	250	1227	23.14	0.69	2.34
Pipe 4	110	681.3	10.46	1.65	24.08
Pipe 9	75	1111	5.77	2.07	18.28
Pipe 10	75	613.5	0.96	0.78	22.97
Pipe 7	63	999.5	1.57	0.80	17.90
Pipe 2	50	1328	2.09	1.70	10.31
Pipe 8	40	704	1.10	1.40	13.86
Pipe 3	75	3078	4.84	1.58	12.33
Pipe 5	40	595.8	0.94	1.20	27.39
Pipe 6	75	667.2	1.06	0.40	3.38

Table VI.5: Node status (peak case + fire).

Junction	Elevation (m)	Demand (l/s)	Hydraulic Grid (m)	Pressure (wcm)
Junction n1	1239.39	5.82	1281.88	40.10
Junction n2	1117.36	2.09	1144.93	27.57
Junction n6	1182.19	7.84	1226.88	44.68
Junction n7	1024.27	4.84	1065.82	41.54
Junction n8	1152.51	3.75	1203.66	51.15
Junction n9	1100.84	0.94	1162.26	61.41
Junction n10	1094.96	3.75	1105.58	10.62
Junction n11	1092.96	18.05	1103.03	10.06
Junction n3	1239.36	1.57	1261.58	22.21
Junction n4	1168.99	5.19	1229.75	60.76
Junction n5	1131.99	1.10	1164.02	32.02
Junction n12	1062.20	0.96	1091.48	29.28
Reservoir 1	1282.02	-46.50	1285.62	3.50

Conclusion:

In this chapter we have done :

- A suitable layout for the distribution network
- Calculation of different flow rates (specific, en route, nodal)
- Determined the various pipe diameters on the basis of the flow rates.

Using EPANET software, we were able to size our distribution network and see how it would function in the most unfavourable cases.

CHAPTER VII :
**Pipes installation and water
hammer protection**

CHAPTER VII : Pipes installation and water hammer protection**Introduction :**

Pipes used to transport water under pressure are made up of straight cylindrical pipes and special fittings (elbows, flanges, reducers).

The main materials used to manufacture pipes are: cast iron and ductile iron, steel, plastics, reinforced or unreinforced concrete and pre-stressed concrete. In our study, we opted for HDPE pipes.

In this chapter we will explain the different types of installation depending on the location and the obstacles that may be encountered, as well as the different accessories placed in the network.

VII.1 Installation of pipes :**VII.1.1. Principle of pipe installation :**

The choice of pipe-laying method depends on the topography of the terrain and the different types of pipe encountered :

- Installation of pipes in ordinary ground .
- Installation of pipes in soft ground .
- Installation of pipes in marshy ground .
- Installation of pipes in galleries .
- Crossing a river .
- Installation of pipes without opening a trench.

VII.1.1.1 Pipes installation on ordinary land :

The pipe is laid in a trench at least 60 cm wide, the depth of which must be sufficient to accommodate the laying bed, the pipe and the thickness of the backfill (minimum 80 cm). The bottom of the trench is covered with a bed of sand 15 to 20 cm thick, suitably levelled. Before the trench is dug, the pipes are sorted to remove any that have been damaged or cracked. The pipes are then lowered into the bed either manually or mechanically. In the case of joint welding, this task should preferably be carried out at the bottom of the trench, wedging the pipe in place either with earth stops or with sections of wooden plank laid out in the direction of the width of the trench. The trench is then backfilled up to 0.30 m above the pipe, in successive layers, watered and well compacted with stone-purged earth. The backfill is completed with rubble.

Ordinary pipe laying can be done in two main ways:

- Buried pipe laying
- Overhead laying

The following diagram shows the laying of a pipeline in ordinary ground .

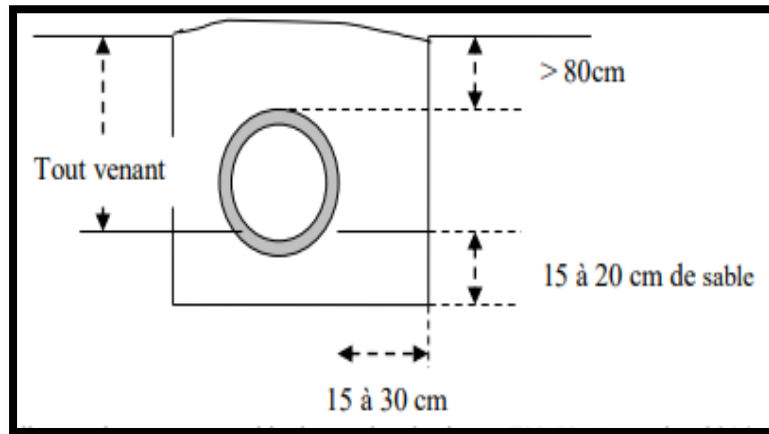


Figure VII.1: Ordinary installation (Source: Cours alimentation en eau potable,2024)

VII.1.1.2. Pipes installation in soft terrain:

To avoid any subsequent movement, the pipe should be laid on a reinforced concrete footing with a sand bed in between. The footing may or may not be continuous, depending on the nature of the soil.

If the pipe is laid on cleats, these must be placed close to the joints and supported by piles driven into the ground.

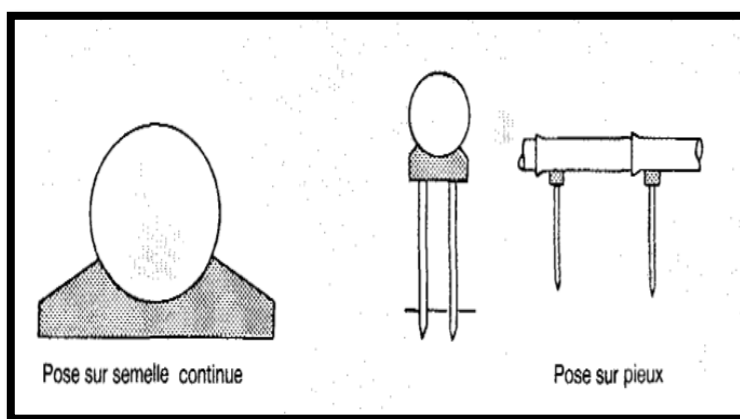


Figure VII.2: Pipes installation in soft terrain. (Source: Alimentation en eau potable,2024)

VII.1.1.3.Pipes installation in marshy ground :

Installation in marshy ground refers to the laying of pipes in areas of marshy or waterlogged ground. In this case, laying pipes is more complex than in drier environments, and the method involves installing a concrete bed under which a water drainage system is integrated, such as drainage gravel and a drainage pipe, as well as devices such as drainage water stops, etc.

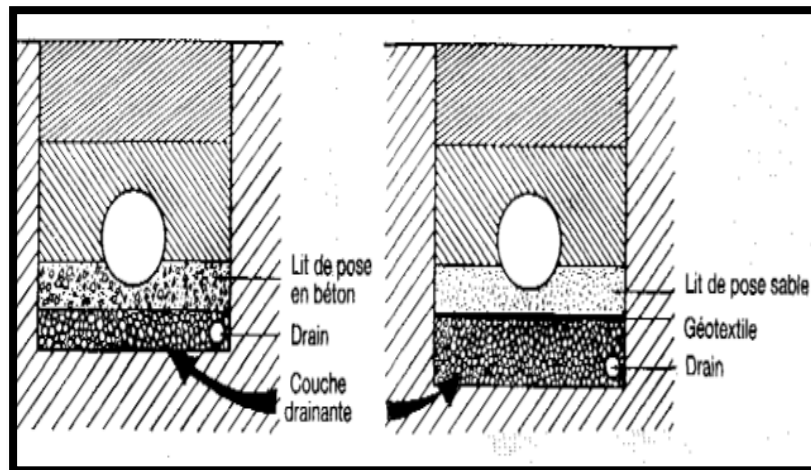


Figure VII.3: Pipes installation in marshy ground .

VII.1.1.4.Pipes installation in galleries:

A canal is provided to prevent and evacuate stagnant water in the gallery and discharge into the neighbouring sewer. Small-diameter pipes can be laid in a larger-diameter sleeve diameter sheath resting on concrete cleats. The gate valves are placed on both sides of the road.

VII.1.1.5.Crossing a river :

- Laying a pipeline across a river requires certain reinforcement work, depending on the condition of the river crossing and the size of the pipeline.
- The existence of a road bridge will also serve as a support for the pipe, where it will be attached to the deck .
- The pipe can follow the riverbed, laid on special structures (e.g. cleats) and covered with alluvial soil. It will be covered with alluvial materials to protect it from impacts (e.g. from shipping).
- Steel pipes are coated with a rust inhibitor in several layers.

VII.1.1.6.Pipes installation without opening a trench :

Underground pipes can be laid without a continuous trench, by sinking or drilling into the ground, in order to reduce the effect of disturbance on the surrounding environment and rehabilitation costs.

It should be noted that, for diameters greater than 800mm, sinking is adopted as the technique. and for smaller diameters, drilling is used.

- **Directional drilling:**

This method uses a directional drill to bore a tunnel below the surface of the ground, The pipe is then pulled through the tunnel, in two stages

- Pilot hole drilling .
- Simultaneous boring and pulling .

- **Installation by jacking :**

A sleeve driven by jacks and supported by a perforating transverse anchor is used.

VII.1.1.7.Installation on a slope :

If the installation site is on a slope, the pipe must be anchored in a concrete bed to ensure Stability

Knowing that the sliding force taken up by the massif is :

$$F = (\sin\alpha - \tan \varphi \cos \alpha) [daN]$$

P: is the weight of the section of pipe between two anchor blocks, in daN

α :is the angle of inclination of the pipe

φ : is the angle of friction between the pipe and the ground

$$\text{If: } (\sin\alpha - \tan \varphi \cos \alpha) > 0$$

So mooring is essential to prevent the pipe from slipping.

VII.2. Pipe stabilisation :

Water pipes are generally connected by joints. If these joints are not firmly fixed, except in the case of flanged joints and welds, there is a risk that the pressure of the water will cause the joints to come apart. In these cases, concrete blocks or thrust blocks are built in to prevent the joints from dislodging .

VII.3. Type of pipe :

The agglomeration's distribution network is made up of a main pipe and smaller diameter pipes known as secondary pipes (branch pipes). network is branched.

The network is made up of HDPE pipes with diameters ranging from Ø 40 mm to Ø 250 mm.

VI.3.1.HDPE pipes :

HDPE pipe or high density polyethylene pipe is a synthetic material pipe which, because of its qualities, will ensure the routing of drinking water for any type of installation, domestic or agricultural.

It is easily recognised by its black colour and blue stripes.



Figure VII.4: HDPE. pipe.

VII.3.2.HDPE pipe characteristics :

- Corrosion-resistant .
- Efficient, high-performance seals .
- Trenchless installation .
- Respectful of nature .

VII.4.Pipe assembly techniques :

Plastic pipe assembly techniques are mainly electro-fusion and butt welding.

VII.4.1. Electro fusion technique :

This reliable technique is very well known. tubes using a sleeve or an electro weldable deflection socket comprising a heating mat a heating mat consisting of a metal winding which, by means of the joule effect the energy required to generate fusion and inter diffusion of the macromolecules of the polyethylene.

VII.4.1.1.Main stages of Electro fusion technique :

✓ Phase 1: Positioning the sleeve :

Position the sleeve and align it with the scraped and cleaned tubes.

✓ Phase 2: Melting of the material :

The live coils heat up and the material around the tube around the tube begins to melt.

✓ **Phase 3: Increase in diameter**

The heated tube increases in diameter and the molten material, limited by the cold zones, reaches the necessary pressure required.

✓ **Phase 4: Complete junction**

The welding machine interrupts the flow of energy; once the cooling time has elapsed, the joint is finished.

VII.4.2. Butt welding :

Butt welding is a technique for joining thermoplastics by melting the ends of two tubular elements using a heated plate, called a mirror. The process involves melting the material at the surface to be welded, bringing the molten parts into contact to ensure intimate mixing and allowing the resulting assembly to cool. The welds are characterised by the appearance of a bead, or weld bead, at the level of the welding plane which results from the lateral ejection of the molten material formed at the end of the tubes. Its shape is used to give an initial visual indication of the quality of the weld.

VII.4.2.1.Welding conditions :

Butt welding is an effective technique provided that the conditions under which the welding is carried out. The parameters that can be controlled and modified by the welder are :

- The temperature of the heating mirror.
- The pressure applied to the tubes during each stage.
- The duration of the various welding stages (preparation, heating, cooling).

VII.4.2.2.Main stages of butt welding :

Butt welding can be divided into four phases:

✓ **Phase 1: Levelling :**

The entire surface of the tube is in contact with the mirror. The two tubes to be welded are pressed against the mirror pressure .

The duration of this stage is very short (a few seconds), the time it takes for a thin layer of polymer to melt. A weld bead then begins to form.

✓ **Phase 2: Heating :**

A layer of molten material expands when the tubes come into contact with the mirror at a low pressure of around 0.01 MPa. The heating is completed when a sufficient thickness of molten material for a good quality weld.

✓ **Phase 3: Mirror removal :**

The tubes are moved back, and the mirror is removed once the two tubes. This is a very short phase, during which the flow and cooling of the molten polymer with the ambient air must be limited

✓ **Phase 4: Welding and cooling :**

The two tubes are held together at a certain pressure until the weld solidifies. The material that flows out gives the final shape of the bead.

VII.5.Accessories and special parts for water supply :

Various accessories are installed along a pipe to :

- Ensure proper flow .
- Insulate pipes .
- Change the direction of pipes .
- Remove flow.
- Adjust pressures .
- Measure flow rates .
- Empty a pipe .
- Allow air to enter a pipe .
- Connect pipes and change their diameter .

VII.5.1. Accessories :

VII.5.1.1.Valves :

These are sectioning devices used to separate and isolate the different parts of the supply pipe during repairs to one of them, and they are also used to adjust the flow rate.

A valve consists of the following components:

- ✓ A casing (comprising a body and a bonnet) which forms the interface between the fluid and the external environment. interface, usually in steel or cast iron.
- ✓ An obturator: which forms the upstream fluid-downstream fluid interface (plug, membrane, gate).
- ✓ A pipe connection system: Connections can be

- Welded.
- Bolted (flanged).
- Screwed.
- ✓ A system for operating the obturator which forms the valve-operator interface (hand wheel, lever, actuator, etc. + stem);
- ✓ A system that provides a dynamic seal to the outside (seal, press, gaskets, etc.).

There are wedge gate valves and butterfly valves.

1.Gate valves:

The gate valve is a shut-off device that must be completely open or closed. It takes up a lot of space compared with a butterfly valve. From a certain , by-passes should be installed to balance the pressures on both sides of the valve. on both sides of the gate when it is to be operated from the closed position .



Figure VII.5: Gate valve .

Source : (https://www.google.com/search?q=vanne+%C3%A0+opercule&sca_esv=33efa)

2.Butterfly valves:

Can be used for both blocking and regulating flows. They take up little space, are lightweight, have a low operating torque and create low head losses at full opening.

- Above a certain diameter (200 to 300 mm), gate valves of both types are generally motorised. Butterfly valves are more commonly used for larger diameters (350 and over).

- The distance between the isolation valves must be chosen so that the length of pipelines taken out of service does not exceed 200 to 250 m where pipelines cross; for main pipelines this length may be as much as 300 to 350 m

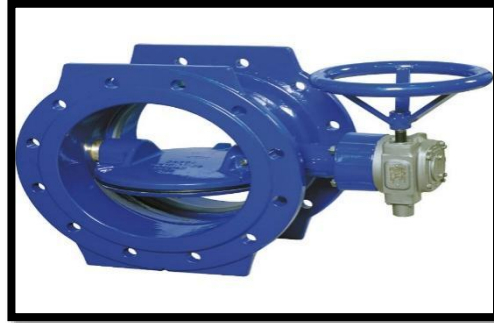


Figure VII.6: Butterfly valve .

Source : (https://www.google.com/search?sca_esv=33efa13a2b517c28&sca_upv=1&q)

3. High level valves :

They are placed at the inlet of a high reservoir, when the water in this reservoir reaches its maximum level. The valve closes under the effect of the water pressure to prevent the reservoir from overflowing.

4. Discharge valves :

- Valve located at low points for draining pipes. This discharge is carried out to the neighbouring sewer or into a roadside ditch (outside the village). This tap will be placed inside a manhole and must be easily accessible.
- Small gate valves, connection valves and shut-off valves must be placed under keyholes above ground level; large-diameter taps are placed in chambers under manholes in installations, they are in manoeuvring chambers (pumping stations and structures such as reservoirs).

VII.5.1.2. Suction (vacuum) cups :

The suction cups are installed at high points to evacuate the air accumulated following a degassing of dissolved oxygen. They are installed in manholes that can be inspected and their correct operation must be checked periodically.



Figure VII.7: Suction cups .

Source : (https://www.google.com/search?q=ventouse+de+d%C3%A9gazage&scas_es)

VII.5.1.3. Fire hydrants and fire poles :

Their diameter is standardised at 80, 100 or 150 mm. They must be supplied by pipes of the same diameter, which must ensure a minimum flow rate of 17 L/s at 1 bar pressure. The configuration of the network (fire reserve) must allow the fire brigade to have access to 120 m³ . The area served by each fire hydrant depends on the flow required to fight the fire.

To protect against freezing, fire hydrants must be kept empty of water.



Figure VII.8: Fire hydrants and fire poles .

Source : (<https://www.google.com/search?q=.+Fire+hydrants+and+fire+poles+&scasv>)

VII.5.1.4.Flow and pressure measurement :

1. Flow measurement: flow is measured using a flow meter with depressurising elements:

- The diaphragm .
- Venturi .
- The nozzle
- Ultrasonic flowmeter: an ultrasonic transmitter emits sound waves. the time taken for the signal to travel the distance L gives us the speed of the fluid.

2. Pressure measurement: Pressure is measured using pressure gauges.

- Bellows pressure gauge.
- Capsule pressure gauge.
- Diaphragm pressure gauge.

VII.5.2. Special connection pieces:**VII.5.2.1.Bends (elbows):**

They are particularly useful for meshed and branched networks where the pipe changes direction.



Figure VII.9: Flanged elbows .

Source : (https://www.google.com/search?q=les+coudes+%C3%A0+bride&sca_esv=33e)

VII.5.2.2.Tees:

They are used to withdraw a flow from a pipe or to add an additional flow. They are also found in meshed and branched networks and in supply pipes in the event of tapping.



Figure VII.10:Tee .

Source : (https://www.google.com/search?q=tees+en+aep&sca_esv=33efa13a2b517c28&s)

VII.5.2.3. Sleeves :

They are found mainly on accessory fittings (gate valves, some joints)



Figure VII.11: Polypropylene sleeves

Source : (<https://www.google.com/search?q=Manchons+en+polypropyl>)

VII.5.2.4.Junction crosses :



In our network, they are used to cross pipes running perpendicular directions.

Figure VII.12: Cast iron cross with flanges .

Source : (https://www.google.com/search?q=+Croix+de+fonte+%C3%A0+brides&sca_esv)

VII.6.Protection against water hammer :

VII.6.1.Definition :

Water hammer is an oscillatory phenomenon caused by a variation in the flow regime. It is a pressure wave, positive or negative, caused by a variation in the hydraulic regime, and propagating in the medium formed by the water and the pipe containing it. This wave is characterised by its speed of propagation or celerity. It reflects off obstacles, particularly the ends of the pipe.

VII.6.2.Causes of water hammer:

There are several possible causes of water hammer. The main ones are :

- Sudden stoppage by disconnection of one or more electric pump units .
- Instantaneous closing or opening of a valve, particularly in the case of a gravity flow pipe .

- Simultaneous or separate start-up of one or more electric pump units.
- Filling or emptying a water supply system .
- Changing the angular speed of a pump.
- Variation of a body of water connected to a network under load.
- Interruption of the power supply to a pumping station, which is the most common cause of water hammer .
- Vibration of control accessories such as valves .

VII.6.3.Risks due to water hammer:

In practice, these risks are very significant :

- **Risk of high pressure:**

Overpressure due to water hammer can be generated in addition to the initial pressure. If the sum of these pressures exceeds the maximum permissible pressure in the pipe, this can lead to cracking of the pipe and dislocation of the points.

- **Risk of low pressure:**

This pressure can appear either after a strong overpressure, or following an instantaneous stop of a pump if this pressure falls below 10 w.c.m ; it reproduces a cavitation pocket

- **Pipe fatigue:**

Under transient conditions, the alternation of over pressure and under pressure that is an inevitable consequence of the phenomenon causes fatigue in the pipe material, even if the amplitude is low.

VII.6.4.Water hammer protection:

In order to ensure that the installations have a long service life and are always in good working order, water hammer protection devices are installed:

VII.6.4.1.At the pumping station :

- Air tank.
- The fly wheel

VII.6.4.2.At the pipes :

- Relief valve.
- Suction cup.

- Balancing flue.

VII.6.5. Water hammer study:

This study consists of determining the over pressure and under pressure values in the discharge pipes and ensuring that these do not exceed the permissible operating pressure value.

VI.6.5.1. Calculation of wave velocity:

There are several formulae for calculating the velocity of the water hammer wave in pressurised pipes. For our calculations we use the following usual formula:

$$C = \frac{\sqrt{\frac{K}{\rho}}}{\sqrt{1 + \frac{K.D}{E.e}}} \quad (\text{IV.19})$$

With :

C : Wave velocity.

ρ: Density of the water.

D: Pipe diameter (m).

E: YOUNG modulus of the pipe. HDPE: $1.2 \cdot 10^9$ Pa.

e: Pipe thickness (m).

K: Coefficient of elasticity of water: $K=2.16 \cdot 10^9$ Pa.

Table VII.1: Wave Velocity values for the two discharge pipes

Pipe	L(m)	D(mm)	e(m)	C(m/s)
PS1_PS2	5039	200	34,2	447.68
PS2_Water tank	4781	200	34,2	447.68

VII.6.5.2. Calculating the maximum value of overpressure and negative pressure in discharge pipes :

The maximum values for overpressure and negative pressure are given by the following relationship :

$$b = \pm CV0/g \quad (\text{IV.20})$$

Cas de surpression :

$$H_s = H_0 + b \quad (\text{IV.21})$$

Cas de dépression :

$$H_d = H_0 - b \quad (\text{IV.22})$$

Table VII.2: Maximum values of overpressure and negative pressure for the discharge pipe.

Pipe	b(m)	TMH(m)	Hd(m)	Hs(m)
PS1_PS2	48,82	236,95	169,18	266,82
PS2_Water tank	48,82	234,47	168,18	265,82

VII.6.5.3. Analysis of the results :

For our supply, we opted for HDPE pipes, PN25, i.e. they can withstand pressures of up to 25 bars (250 m)

- In the event of excess pressure, the delivery pipes cannot withstand the pressure, so we need to provide protection
- In the event of negative pressure: there is no risk of negative pressure in the delivery pipe.

VII.6.6. Protection of the discharge pipe against water hammer :

VII.6.6.1 Flywheel :

A flywheel is a mechanical device used to store kinetic energy and release it when required. It is particularly useful for regulating variations in speed or energy in mechanical systems. In a pumping station, it is often used to mitigate the adverse effects of a sudden pump stop, such as water hammer.

VII.6.6.2 Pressure reduction mechanism :

When a pump stops suddenly or when a valve closes too quickly, there is a sharp increase in pressure due to the inertia of the water, which continues to move through the pipes. This creates pressure waves that can generate dangerous excess pressure.

The flywheel works as follows:

- **Kinetic energy storage:** While the pump is operating normally, the flywheel rotates and accumulates kinetic energy.

- **Energy release:** When water hammer is likely to occur, the flywheel releases the stored energy to help slow the pump speed variation and maintain a more stable flow.
- **Progressive energy dissipation:** The flywheel gradually slows down the system, reducing the intensity and speed of pressure variations, thereby reducing hydraulic shock.

VII.6.6.3 Advantages of the flywheel :

- 1. water hammer mitigation:** reduces sudden pressure variations in pipes, minimising the risk of overpressure or under pressure, and protects equipment against hydraulic shock.
- 2. Reliability and simplicity:** A simple, reliable mechanical device with little risk of failure.
- 3. Reduced maintenance costs:** Low maintenance and protection of equipment, reducing repair and replacement costs.
- 4. Energy efficiency:** Stores energy during pump operation and releases it during flow variations, improving overall efficiency.
- 5. Extended equipment life:** Reduces wear and tear on pumps, pipes and valves by stabilising pressure fluctuations.
- 6. Improved safety:** Maintains stable pressure, reducing the risk of burst pipes and leaks, and ensuring the safety of operators.
- 7. Reduced vibration:** Improves system balance by reducing vibration due to speed variations.
- 8. Low installation and running costs:** Less expensive to install and maintain than other complex systems.
- 9. No external power supply required:** Operates without an external power source, making it reliable and cost-effective in environments with no available power supply.

In the context of this study, the risk of water hammer, mainly due to a rapid variation in pressure in the discharge pipe as a result of sudden stops or starts of the pump. The pipe is made of HDPE PN25, and the risk of rupture is high in the event of pressure loss. The solution must therefore provide effective protection against these fluctuations to guarantee the safety and longevity of the equipment

The flywheel is the ideal choice for our case, as it reduces the risk of suppression in the PN25 HDPE discharge pipe, which cannot withstand large variations in pressure. With a suppression head (HS) close to the total manometric head (HMT), the flywheel helps to attenuate pressure fluctuations and avoid water hammer, thus protecting the pipe from potential damage. It's a cost-effective, reliable and easy-to-maintain solution that ensures system stability and extends equipment life by reducing overpressure and under pressure.

Conclusion:

In this chapter, we have presented the techniques and works required for the laying of pipes and we have also discussed the accessories used to connect the pipes and which must be correctly selected to ensure the smooth running of the network.

In this way, we have explained the phenomenon of water hammer and seen the means of protection against the transient regime.

For the scheme of our adduction , flywheel will be used to combat water hammer to our two pumping stations (PS) .

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CHAPTER VIII : Quantity and cost estimates

Introduction:

The sizing study for a drinking water supply is never complete until it has been properly implemented. To ensure that our project runs under the best possible conditions, it is essential to organise the worksite. This involves planning, coordinating and implementing all the resources of all the resources required to complete the work, with the emphasis on optimising resources and deadlines.

VIII.1. Definition of site organisation :

The organisation of a building site consists of providing all the necessary resources in terms of human resources, equipment and materials, and requires and imposes a work rhythm for the proper use of these resources with the aim of achieving: speed, quality and economy.

For example, cement, sand, gravel and steel are used to make reinforced concrete bodies, and the selling price is agreed before the product is manufactured. The correct evaluation of prices and the resources to be used is difficult because it is based on assumptions that will be confirmed during and at the end of the project. This constraint obliges the company to check the forecasts regularly during the execution of the project.

VIII.2. Execution the work :

The main operations involved in laying the pipes are as follows:

- Checking and handling the pipes.
- Layout of trench routes .
- Extraction of the trench .
- Laying the bed .
- Laying pipes .
- Pipe assembly.
- Backfilling trenches .
- Carry out tightness tests on pipes and joints .
- Compacting .
- Network disinfection .

VIII.2.1. Checking and handling pipes:

Checks are carried out for prefabricated pipes on :

- Quantities .
- Appearance and integrity check .

- Marking in the event of defects

VIII.2.2.Layout of trench routes :

VIII.2.2.1.Marking out :

The axis of the trench is marked out on the ground using stakes aligned in a straight line 50 metres apart. This work is carried out by measuring the distances on the plan from fixed markers or milestones. In this way, the direction of the axes and their extremities are clearly determined.

VIII.2.2.2. Levelling the installation platform :

Levelling consists of measuring the difference in altitude between two or more points located on a uniform slope. In the presence of obstacles on the ground, levelling is carried out by path. By means of a simple calculation, the height of each point and the depth of the trench at each location can be determined.

VIII.2.3. Extraction of the trench :

It is divided into two parts :

VIII.2.3.1.Stripping (Removal)the topsoil or tar layer:

Before starting to excavate the trenches, the first operation to be carried out is topsoil stripping. This operation consists of removing the surface layer of the soil (vegetation and organic matter).

The volume of the layer to be stripped is calculated using the following formula:

$$V_{str} = L * b * e \quad (VIII.1)$$

With :

V_{ev}: volume of stripped earth in (m³) .

L: total length of trenches (m) .

b: width of topsoil in (m) .

e: layer thickness in (m), e = 10 cm .

VIII.2.3.2.Trenches and excavations :

Excavation work is carried out mechanically from downstream to upstream. Its width must be the outside diameter of the pipe, with a 30 cm gap between the walls of the trench and the pipe.

The depth of the trench must be greater than its width, and should be between 80cm and 120cm in order to :

- protect the pipe from being crushed by loads and overloads .
- Protect the pipe from temperature variations .

- Protect the pipe from freezing .

Excavation requires a number of parameters to be determined, such as:

- The depth of the trench (H tr) .
- The width of the trench (b) .

✓ **Trench Depth Htr :**

The depth of the trench depends on the diameter of the pipe, the rolling (external) loads and the temperature. It is determined by the following relationship:

$$\mathbf{Htr = D+h+e} \quad \textbf{(VIII.2)}$$

Where :

Htr: Trench depth (m) .

D: Pipe diameter (m) .

h: Height of the upper generatrix of the pipe,(h=1m is assumed) .

e: Thickness of the bed .(e=0.15 m is assumed)

✓ **Trench width b :**

The width of the trench will be determined according to the diameter of the pipe, with a margin of 30 cm on each side. The formula is as follows

$$\mathbf{b=D +0.6} \quad \textbf{(VIII.3)}$$

With :

b: Trench width in (m).

D: Pipe diameter in (m).

✓ **Trench Section St :**

The cross-sections of the tracheae are trapezoidal, given by the following relationship:

$$\mathbf{St = B * H} \quad \textbf{(VIII.4)}$$

Where:

H : Depth of the trench (m) .

B: Width of trench (m) .

St: Cross-sectional area of trench.

VIII.2.4.Laying the bed :

The pipes must be laid on a bed 10 cm to 20 cm thick, generally consisting of well-levelled sand. But if the ground is not very consistent, the bottom of the trench will be consolidated, and each pipe will be laid on a reinforced concrete footing with a sand bed in between.

VIII.2.5.Laying the pipe :

Once the bed has been laid, the pipe is laid in the following stages following operations:

- Before the pipe is laid, the trench bed must be cleaned .
- The pipe must be laid with care
- The alignment of the pipes must be checked regularly and they must be wedged to align them .
- Every time there is a work stoppage, the pipes must be closed off with a temporary plug to prevent the ingress of dirt. to prevent the ingress of foreign bodies.

VIII.2.6.Pipe assembly :

Circular pipes are assembled either by socket joints or by welding.

VIII.2.7.Leak test on joints and pipes :

Once the pipe has been laid at the bottom of the trench, a tightness test is carried out on sections of maximum maximum length of 500 m using a test pump. This test should last approximately 30 minutes and the pressure variation must not exceed 0.2 bar.

VIII.2.8.Compacting :

Once the backfill has been made, it must be compacted, which consists of spreading the earth to ensure protect the pipe .

VIII.2.9.Network disinfection :

When pipes are laid, dust can be introduced into the pipes, and to eliminate the existence of these foreign bodies in the pipe, it is essential to disinfect the network before it is put into service.

VIII.3. Choice of equipment :

To have better output and better profitability of the work, the choice of machines is necessary, because each operation has a machine that suits it.

VIII.3.1.For the Stripping of topsoil :

For stripping topsoil, the machine used in areas that are easy to strip is the grader. In difficult terrain, however, a bulldozer is used.



FigureVIII.1:Bulldozer

Source: (https://www.google.com/search?sca_v=d0b2727f85f62fcc&sca_upv=1&q=bulldoz)

VIII.3.2.For Excavating trenches :

An excavator equipped with a retrofit suitable for all types of terrain is used for this operation. This machine is used on a large scale thanks to the good performance and quality of the work it can provide.



FigureVIII.2: An excavator equipped with a retrofit suitable .

Source:(<https://www.google.com/search?q=une+pelle+%C3%A9quip%C3%A9e+en+retro>)

VIII.3.3.Back filling trenches :

The machine best suited to this operation is the loader. This machine is used to backfill excavations, and the trench after the pipe has been laid, and if the work is not very extensive, a backloader is used



FigureVIII.3: A back loader .

Source : (https://www.google.com/search?q=+le+r%C3%A9tro+chargeur&sca_esv=d0b)

VIII.3.4.For Compacting :

A smooth roller compactor is used for compacting the backfill in narrow areas.



FigureVIII.4: A smooth roller compactor .

Source: (https://www.google.com/search?sca_esv=b45802d3195ccd8a&sca_upv=1&q=com)

VIII.4.Calculation of the volume of work :

VIII.4.1.Laying pipes (network + adduction) :

VIII.4.1.1.Volume of stripping of topsoil or tar :

According to formula (VIII.1), the volume of the layer to be stripped is :

Table VIII.1: Calculation of the topsoil or tar volume :

Network	Pipe	Diameter (mm)	Lengh (m)	B(m)	e(m)	Vstri (m3)
Adduction	B.T_PS1	160	2912	0,76	0,1	221.312
	PS1_PS2	200	5039	0.8	0,1	403.12
	PS2_WT	200	4781	0.8	0,1	382.48
Distribution	Pipe 1	250	506	0,85	0,1	43.01
	Pipe 11	160	1377	0,76	0,1	104.652
	Pipe 12	250	1227	0,85	0,1	104.295
	Pipe 4	110	681,3	0,71	0,1	48.3723
	Pipe 9	75	1111	0,675	0,1	74.9925
	Pipe 10	75	613,5	0,675	0,1	41.41125
	Pipe 7	63	999,5	0,663	0,1	66.26685
	Pipe 2	50	1328	0,65	0,1	86.32
	Pipe 8	40	704	0,64	0,1	45.056
	Pipe 3	75	3078	0,675	0,1	207.765
	Pipe 5	40	595,8	0,64	0,1	38.1312
	Pipe 6	75	667,2	0,675	0,1	45.036

So the total volume of topsoil stripped is **1912.2201m3**.

VIII.4.1.2. Volume of trench excavation :

The volume of excavated soil is given by the following formula:

$$Vd = L \times B \times H \quad \text{(VIII.5)}$$

Vd: volume of trench excavation (m^3)

L: total length of trench (m).

B: width of trench (m).

H: depth of trench (m).

Table VIII.2: Calculation of trench excavation volume :

Network	Pipe	diameter	Lengh (m)	B(m)	Htr(m)	Vexc (m3)
Adduction	B.T_PS1	160	2912	0,76	1,31	2899.1872
	PS1_PS2	200	5039	0,8	1,35	5442.12
	PS2_WT	200	4781	0,8	1,35	5163.48
	Pipe 1	250	506	0,85	1,40	602.14

Distribution	Pipe 11	160	1377	0,76	1,31	1370.9412
	Pipe 12	250	1227	0,85	1,40	1460.13
	Pipe 4	110	681,3	0,71	1,26	609.49098
	Pipe 9	75	1111	0,675	1,23	918.65813
	Pipe 10	75	613,5	0,675	1,23	507.28781
	Pipe 7	63	999,5	0,663	1,21	803.81689
	Pipe 2	50	1328	0,65	1,20	1056.5568
	Pipe 8	40	704	0,64	1,19	536.1664
	Pipe 3	75	3078	0,675	1,23	2545.1213
	Pipe 5	40	595,8	0,64	1,19	453.76128
	Pipe 6	75	667,2	0,675	1,23	551.691

So the total volume of trench excavation is **24920, 549 m3** .

VIII.4.1.3.Backfill volume :

The volume of backfill is equal to the difference between the volume of excavated soil, less the volume occupied by the pipe, the volume of the sand bed, and the volume of coating calculated as follows:

$$\mathbf{V_{backfill} = V_{excavation} - V_s - V_p} \quad \textbf{(VIII.6)}$$

Where :

V excavation: Volume of excavation (m3).

Vs: Volume of sand bed (m3) .

Vc: Volume of pipe (m3) .

VIII.4.1.3.1.Volume of the sand bed :

The sand bed, 15 cm thick along the trench, will have a volume that can be calculated as follows can be calculated as follows:

$$\mathbf{V_s = e_s \times b \times L} \quad \textbf{(VIII.7)}$$

Where :

Vs : Volume of sand bed (m3).

es: thickness of sand (m). es =15 cm

b: width of trench (m).

L: Trench length (m).

VIII.4.1.3.2.Pipe volume :

Once the trench has been excavated and the sand bed placed, the pipe must be laid. Knowing its cross-section is crucial for determining the volume of backfill, which can be calculated using the following relationship:

$$V_p = \frac{L * \pi * D^2}{4} \quad (\text{VIII.8})$$

With :

V_p: Volume of pipe (m3).

L: Length of trench (m).

D: pipe diameter (m).

Table VIII.3: Calculation of backfill volume :

Network	Pipe	Diameter	Leng h (m)	B(m)	Htr (m)	Vexc (m3)	Vs (m3)	Vp (m3)	Vback(m3)
Adduction	B.T_PS1	160	2912	0,76	1,31	2899,1872	331.968	58,549234	2508.67
	PS1_PS2	200	5039	0,8	1,35	5442.12	604.68	158.2246	4679.2154
	PS2_WT	200	4781	0,8	1,35	5163.48	573.72	150.1234	4439.6366
Distribution	Pipe 1	250	506	0,85	1,4	602,14	64.515	24,838217	512.78678
	Pipe 11	160	1377	0,76	1,31	1370,9412	156.978	27,686228	1186.277
	Pipe 12	250	1227	0,85	1,4	1460,13	156.4425	60,230222	1243.4573
	Pipe 4	110	681,3	0,71	1,26	609,49098	72.55845	6,4746104	530.45792
	Pipe 9	75	1111	0,675	1,23	918,65813	112.48875	4,9082476	801.26113
	Pipe 10	75	613,5	0,675	1,23	507,28781	62.116875	2,71036	442.46058
	Pipe 7	63	999,5	0,663	1,21	803,81689	99.400275	3,1156867	701.30093
	Pipe 2	50	1328	0,65	1,20	1056,5568	129.48	2,6075219	924.46928
	Pipe 8	40	704	0,64	1,19	536,1664	67.584	0,8846725	467.69773
	Pipe 3	75	3078	0,675	1,23	2545,1213	311.6475	13,598187	2219.8756
	Pipe 5	40	595,8	0,64	1,19	453,76128	57.1968	0,7487044	395.81578
	Pipe 6	75	667,2	0,675	1,23	551,691	67.554	2,9475993	481.1894

So the total volume of backfill is **21534,571 m3** .

VIII.4.2.Calculation of the water tank :

We have projected two square section water tanks on the ground and we have their characteristics:

Table VIII.4: Characteristics of the planned water tanks.

Volume (m3)	Section (m2)	Height (m)	Thickness of invert a (m)	Thickness walls b (m)
2.75	1.83	1.35	0.2	0.2
2.75	1.83	1.35	0.2	0.2

VIII.4.2.1.The external length of the side of the square base :

The external length of the side of the square base is given by the following formula:

$$A_{ext} = A_{ins} + 2b \quad (VIII.9)$$

With :

A_{ins}: Inside length of the side of the square base.

b: Wall thickness.

With : $A_{ext} : 1.35 + (2 \times 0.2) = 1.75 \text{ m}$

VIII.4.2.2.Stripping volume :

The stripping volume is given by the following formula:

es: Stripping thickness (e=0.1m).

$$V_{stripping} = S \text{ water tank} * e \text{ stripping} \quad (VIII.10)$$

VIII.4.2.3.Volume of excavation :

The volume of excavation is given by the following formula:

$$V_{excavation} = S_{tank} * h \quad (VIII.11)$$

Where:

S: Surface area of the water tank.

h: Height of earthwork.

VIII.4.2.4.The volume of clean concrete :

The volume of clean concrete is given by the following formula:

$$V_{cc} = S_{tank} * e_{cc} \quad (VIII.12)$$

Where:

S: surface area of the tarpaulin.

e_{cc}: thickness of clean concrete (e cc =0.1 m)

VIII.4.2.5.Volume of reinforced concrete :

The volume of reinforced concrete is given by the following formula:

$$V_{rc} = S_{tank} * a * 2 \quad (VIII.13)$$

With:

S: Surface area of the tank.

a: Invert thickness.

VIII.4.2.6. The volume of reinforced concrete in the walls :

The volume of reinforced concrete in the walls is given by the following formula:

$$V_{rcw} = V_{ext} - V_{int} \quad (VIII.14)$$

With :

: The external volume: $V_{ext} = S_{ext} * H$

V_{int} : The internal volume: $V_{int} = S_{int} * H$

H : The height of the walls .

This table summarises all the calculations for the tanks :

Table VIII.5: Calculation of different volumes for the two projected water tank.

Volume of Stripping (m3)	Volume of excavation (m3)	Volume of clean Concrete (m3)	Volume of reinforced Concrete (m3)	Volume of reinforced concrete walls (m3)
0.183	2.47	0.183	0.732	1.75
0.183	2.47	0.183	0.732	1.75

VIII.4.3. Work on pumping stations :

In our project we have two pumping stations and each station contains four pumps as mentioned previously in the Pumping stations study chapter, the dimensions of the pumping station building chosen are :

- Length of the station: 17 m
- Station width: 9 m
- Height of the station : 8 m
- Posts: $b*b = 0.40*0.40$
- Beams : $b*b = 0.40*0.50$

VIII.4.3.1. Determining the volume of work on the pumping stations :

VIII.4.3.1.1. Pumping station 1 :

The volume of work at the two pumping station is given by the following relationship:

1. Calculating the stripping volume :

$$V_{stri} = S_{stri} * h_{stri} \text{ et } S_{stri} = L' * l' \quad (VIII.15)$$

With:

h_{dec}: The thickness of the surface layer to be stripped is 10 cm.

$$L' = L + 0.55 + 0.55, L' = 17 + 0.55 + 0.55 = 17.1\text{m}, L' = 18.1\text{m}$$

$$l' = l + 0.55 + 0.55, l' = 9 + 0.55 + 0.55 = 10.1\text{m}, l' = 10.1\text{m}.$$

$$S_{\text{stri}} = 18.1 \times 10.1 = 182.81\text{m}^2, S_{\text{stri}} = 182.81\text{m}^2$$

$$\text{So : } V_{\text{stri}} = 18.28\text{m}^3$$

2. Calculating the volume of soil excavation :

The pumping station building is above ground. The volume of excavated soil is equal to the volume of soil excavated from the excavations.

The excavation has a volume of $1.5 \times 1.5 \times 1.5 \text{ m}^3$

The volume to be excavated for one excavation is 3.375 m^3 .

The number of excavations is 12. The volume resulting from the excavation of all the trenches is 40.5m^3

3. Determining the volume of concrete

- **Volume of clean concrete dosed 150 kg/m^3 (under soles):**

(thickness of our clean concrete = 10cm)

$$V_{\text{us}} = (1.5) \times (1.5) \times (0.1) \times 12 = 2.7\text{m}^3, V_{\text{bp}} = 2.7\text{m}^3$$

- **Volume of reinforced concrete dosed 350 kg/m^3 (soles):**

$$V_{\text{so}} = (1.5) \times (1.5) \times (0.3) \times 12 = 8.1\text{m}^3, V_{\text{bas}} = 8.1\text{m}^3.$$

- **Volume of Reinforced concrete dosed 350 kg/m^3 (half posts):**

$$V_{\text{hp}} = (0.4) \times (0.4) \times (0.6) \times 12 = 1.152\text{m}^3, V_{\text{dp}} = 1.152\text{m}^3$$

- **Volume of Reinforced concrete dosed 350 kg/m^3 (poles) :**

$$V_{\text{P}} = (0.4) \times (0.4) \times 8 \times 12 = 16.13\text{m}^3, V_{\text{P}} = 16.13\text{m}^3$$

- **Volume of clean concrete below the belt :**

$$V_{\text{belt}} = (14.4 \times 3 + 7.8 \times 4) \times 0.1 \times 0.4 = 2.976\text{m}^3, V_{\text{belt}} = 2.976 \text{ m}^3$$

- **Volume of Reinforced concrete dosed 350 kg/m^3 (stringers) :**

$$V_{\text{stringers}} = (14.4 \times 3 + 7.8 \times 4 + 12 \times 0.4) \times 0.4 \times 0.4 = 12.67\text{m}^3, V = 12.67\text{m}^3$$

- **Volume of concrete in the lower screed:**

$$V_{\text{low screed}} = (4.8 \times 1.95 \times 6) \times 0.07 = 3.93\text{m}^3, V_{\text{low screed}} = 3.93\text{m}^3$$

- **Volume of concrete in the top slab (upper):**

$$V_{\text{slab}} = (16) \times (9) \times (0.07) = 10.08\text{m}^3, V_{\text{top slab}} = 10.08\text{m}^3.$$

➤ **Girder (beam)volume:**

$$V_{\text{beam}} = 0.2 \times V_{\text{slab}}$$

$$V_{\text{beam}} = 0.21 \times 0.08 = 2.02 \text{ m}^3, V_{\text{beam}} = 2.02 \text{ m}^3$$

➤ **The number of warps:**

$$N_{\text{warps}} = (4.8 \times 1.95 \times 6) / (0.45 \times 0.20) = 630 \text{ wraps}.$$

➤ **Slab volume:**

$$V_{\text{slab}} = V_{\text{low screed}}, V_{\text{slab}} = 3.93 \text{ m}^3$$

4. Determining the masonry :

For the pumping station, the exterior and interior walls will be built with bricks measuring 30 x 20 x 10 cm, leaving an area lamp 10 cm thick. The total masonry surface area is the difference between the total wall surface area and the surface of the walls. So the total external and internal surface area to be covered is :

Hence the number of bricks required is : $762.7 / (0.3 \times 0.2) = 12750 \text{ bricks}$.

5. Determining the volume of backfill :• **Volume of excavation backfill:**

$$V_{\text{eb}} = 12 \times [(1.5 \times 1.5 \times 1.1) - (1.1 \times 0.4 \times 0.4)] = 27.19 \text{ m}^3, V_{\text{eb}} = 27.19 \text{ m}^3$$

• **Volume of backfill for the boxes:**

$$V_{\text{bb}} = (4.8 \times 1.95 \times 6) \times (0.4 + 0.4 - 0.07) = 50 \text{ m}^3, V_{\text{bb}} = 50 \text{ m}^3.$$

VIII .4.3.1.2. Pumping station 2 :

Pumping station 2 has the same dimensions as pumping station 1, so the different working volumes are the same as pumping station 1.

VIII.5. Planning the work :

Before starting work on the site, it is necessary to carry out planning, which consists of a theoretical study that constantly seeks the best way of using labour and other resources .

The aim of planning is to ensure that all the work gets done:

- In the correct order (correct succession of network operations) .
- Without delay .
- As economical as possible .

VIII.5.1 Planning work using the Gantt chart method :

The tasks involved in building the network (distribution + supply) and their duration are listed in the table below :

Table VIII.6: Completion times for network operations (distribution + supply) :

Rating	Transaction	Duration (days)
A	Site preparation	20
B	Stripping topsoil	52
C	Trenching	62
D	Laying the sand bed	35
E	Laying pipes	46
F	Fitting and connecting pipes	30
G	Tightness tests	28
H	Backfilling the trenches	46
I	Levelling and compacting	25

VIII.5.2. Job scheduling using the graph method (Gantt chart) :

Graph theory is used to solve all scheduling and planning problems, in particular the Gantt diagram.

Les opérations qui précèdent et succèdent chaque opération dans le tableau suivant :

Table VIII.7: Tasks preceding and following each network operation (distribution + supply).

Operation	Preceding operation	Following operation
A	---	B
B	A	C
C	B	D
D	C	E
E	D	F
F	E	G
G	F	H
H	G	I
I	H	---

Determining lead times using the table method :

Table VIII.8: Determination of delays for the network (distribution + supply).

Operation	CT	DL ESD	ECD	DLL SDAL	CDAL	MT
A	20	0	20	0	20	0
B	52	20	72	20	72	0
C	62	72	134	72	134	0
D	35	134	169	134	169	0
E	46	169	215	169	215	0
F	30	215	245	215	245	0

G	28	245	273	245	273	0
H	46	273	319	273	319	0
I	25	319	344	319	344	0

with

CT: Completion time.

ESD: Earliest start date.

ECD: Earliest completion date.

SDAL: Start date at the latest.

CDAL: Completion date at the latest

TM: Total margin.

- The critical path of the project is: A-B-C-D-E-F-J-H-I ,With a completion time of 344 working days .

VIII.6.Project evaluation :

This evaluation consists of determining the quantities of all the operations carried out in the field to carry out the project, then multiplying them by the corresponding unit price, and finally finding the total cost of the project.

VIII.6.1. Estimated cost of laying pipes and structures :

The following table shows the estimated cost of laying pipes and structures .

TableVIII.9: Estimated and quantities quote of laying pipes and structures.

ESTIMATE OF QUANTITIES AND COSTS					
N°	Description	Unit	Quantity	Price unit(DA)	Total amount (DA)
1	Stripping of the topsoil	M3	1912.2201	300	573666.03
2	Excavation of ordinary land	M3	24920.549	350	8722192.15
3	Sand bed	M3	2868.33015	1800	5162994.27
4	Compacted backfill	M3	21534.571	300	6460371.3
5	Warning grid	M3	12732	1000	12732000
6	HDPE pipes PN16 .PN25				
Ø	160		2912	2989,25	8704696
Ø	200		9910	3173.48	31449186.8

Ø	250		1733	4940,17	8561314.61
Ø	160		1377	2989,25	4116197.25
Ø	110		681,3	1244,45	847843.785
Ø	75		5469,7	452,65	2475859.705
Ø	63		999,5	324,87	324707.565
Ø	50		1328	204,82	272000.96
Ø	40		595,8	131,5	78347.7
Total.					90481378.1

VIII.6.2.Estimated costs for the pumping stations :

The following table shows the estimated costs for the pumping stations

Table VIII.10: Estimated and quantities quote for the various works at the two pumping stations :

N°	Designation of work or equipment used	Unit	Quantity	Price unit (DA)	Total amount (DA)
1	Stripping of the topsoil		18.1	250	4525
2	Excavation		40.5	400	16200
3	Backfill from excavations		27.19	450	12235.5
4	Backfill of pits		50	450	22500
5	Cleaning concrete dosed 150 kg/m ³ (under sole)		5.68	8000	45440
6	Reinforced concrete dosed 350 kg/m ³ (soles)		8.1	216	1749,6
7	Reinforced concrete dosed 350 kg/m ³ (half posts)		1.152	34.56	39,813
8	Reinforced concrete dosed 350 kg/m ³ (poles)		16.13	43.20	696,816
9	Reinforced concrete dosed 350 kg/m ³ (stringers)		12.67	40000	506800
10	Reinforced concrete dosed 350 kg/ m ³ (lower screed)		3.93	40000	31440
11	Reinforced concrete dosed 350 kg/ m ³ below the belt		2.976	40000	119040

12	Reinforced concrete dosed 350 kg/ m ³ (upper slab)		10.08	40000	432000
13	Girder volume		2.02	40000	80800
14	Reinforced concrete dosed 350 kg/ m ³ (lower screed)		3.93	40000	157200
15	Number of casting pieces		630	350	220500
16	Number of bricks	Unit	12750	300	3.825.000
17	Hydraulic equipment for pumping stations	Unit	2	2516938.5 3	5033877.06
Total : Ten million five hundred ten thousand forty-three Algerian dinars and seventy-nine centimes.					10510043.79

VIII.6.3.Estimated costs for the planned water tanks :

The following table shows the estimated costs for the pumping stations

Table VIII.11: Estimates and quantities quote for the various works at the planned water tanks

N°	Designation of work or equipment used	Unit	Quantity	Price unit (DA)	Total amount (DA)
1	Striping of topsoil		0.183	500	91.5
2	Excavation		2.47	450	1111.5
3	Clean concrete	M3	0.183	13000	2379
4	Reinforced concrete		0.732	55000	40260
5	Reinforced Concrete walls		1.75	55000	96250
6	Number of water tanks		2	132772.5	265545
Total : Four hundred five thousand six hundred thirty-seven Algerian dinars and zero centimes.					405637

VIII.6.4.Estimated and quantities quote of the project :

Estimate of the cost of our project.

Table VIII.12: Overall estimated and quantities of the project .

Type of charges	Costs in DA
Supply and distribution networks	90481378.1
Pumping stations	10510043.79
Water tanks	405637
Total ET	101397059
VAT 19 %	19265441.19
Total ATI	120662500.1

The total cost of the project is 120662500.1 DA

Quotation in words :

One hundred twenty million six hundred sixty-two thousand five hundred Algerian dinars and ten centimes.

VIII.7.Safety at work :

Safety at work is one of the main conditions for development, and in some cases can become a binding obligation. can become a binding obligation in certain cases.

The main objective of work safety is to reduce the frequency and severity of accidents on construction sites. severity of accidents on building sites, which is why the hydraulic sector covers a wide range of possible During the execution of a transfer project, the various phases of work are carried out, such as work is carried out, such as

- Excavation and earthworks (pipe laying, installation of storage tanks, pumping station, storage tanks, pumping stations, etc.).
- Construction work (civil engineering), such as concreting, reinforcement and other phases of the work. the installation of storage tanks and pumping stations.

VIII.7.1.Causes of accidents at work :

Accidents at work in dangerous conditions and dangerous actions are caused by two factors:

VIII.7.1.1.Human factors :

- Lack of control and negligence .
- Fatigue of workers, supervisors and managers .
- Congestion during the various phases of the works.
- Errors poor judgement or reasoning .

- Follow unsuitable pace of work.

VIII.7.1.2.Equipment factors :

- Tools, machinery and work equipment .
- Nature materials used .
- Lack in carrying out the work .
- Les mechanical and electrical installations.

VIII.7.1.3.Hazardous conditions :

- Installations unprotected .
- Installations poorly protected .
- Tools, equipment and machines in poor condition .
- Protection no individual protection .
- Defects in design and construction .
- Defective materials .
- Irrational storage.
- Poor layout .
- Defective lighting .
- Factors unsuitable atmospheric conditions .
- Conditions unfavourable weather .

VIII.7.1.4.Dangerous actions :

- intervene without due care on machines in motion .
- intervene without due care on pressurised or live installations .
- Act without warning or authorisation .
- Neutralize safety devices .
- Do not using personal protective equipment .
- Wrong using tools or machinery .
- Importance during storage operations .
- Adopting an unsafe position .
- Working at an inappropriate altitude .
- Working at an unsuitable pace .
- To banter or quarrel .

VIII.7.2.Preventive measures to avoid the causes of accidents :

VIII.7.2.1. Personal protection :

In order to provide better protection against hazards during the exercise of certain occupations, it is the use of personal protective equipment (helmets, gloves, shoes, safety glasses, shoes, protective glasses, etc.)

VIII.7.2.2. Collective protection :

The site manager or contractor must highlight the following points in this position the following:

- Application strict compliance with safety regulations .
- Affection staff at the controls of the key points of a modern installation. installation.

a) Lifting equipment :

The crane, pipe layer and other machines with their precision and various handling constitute a work area where safety does not allow the slightest negligence, so the technician responsible for :

- Affecter counter staff.
- Carry out periodic checks on equipment in accordance with the manufacturer's instructions.
- Establish a safety zone around lifting equipment and, in particular, avoid parking under a lifted load.

b) Electrical equipment :

To avoid the risks posed by electrical equipment, do-it-yourself work should be avoided at all costs, as an electrical line or installation should only be installed by qualified electricians.

Conclusion:

In this chapter, we have drawn up an estimate for the supply pipe, the distribution network, the pumping stations and the water tanks. The total amount of project is 120662500.1 DA (One hundred twenty million six hundred sixty-two thousand five hundred Algerian dinars and ten centimes.) ,and will be completed in 344 days. Finally, we have identified the appropriate equipment for each task. and some advice on maintaining safety at work.

General conclusion :

In the course of this work, we believe that we have clearly identified the different phases of the drinking water supply project for the towns of ELDREAT and BOURACHED , from the 2000 m³ buffer reservoir .

First of all, we carried out a descriptive summary of these localities and collected the essential data on our study area, located in Hammam dala in the province of M' sila and estimated the various water requirements for 2054, when the population will reach 9521 inhabitants with a long-term requirement of 3275.42 m³/d.

During the study of the distribution network, we opted for a branched network as a distribution method, fed by a water tower. distribution system fed by an above-ground water tower and HDPE PN 16 pipes.

The behaviour of the network in question was simulated using EPANET software for two cases two cases: peak case and peak +fire case.

For the storage facilities, we checked that the existing water tower had sufficient volume to meet the estimated water requirements. As a result, after checking the capacity of the existing water tower, we proposed the construction of 2.75m³ capacity water tanks for pumping stations to ensure the supply of water to our conurbation by backflow. for our conurbation.

We studied a mixed supply system with protection against water hammer.

The gravity section has a diameter of 160 mm in HDPE PN 25 between the buffer reservoir (existing) and the first pumping station (PS1), and a diameter of 200 mm in HDPE PN 25 for the two delivery sections, between the first and second pumping stations and the second pumping station and the existing distribution water tower.

We completed our work by quoting the pipe-laying methods and calculating the estimated cost and quantity of the project, which is estimated at 120662500.1 DA (One hundred twenty million six hundred sixty-two thousand five hundred Algerian dinars and ten centimes.).

On the other hand, we have established a work schedule, which is necessary for the smooth running of the project, the duration of which is estimated at 344 working days.

General conclusion

This study has enabled us to put into practice all the knowledge we acquired during our Hydraulics training course, and we also hope that this dissertation will be useful to future students in the field of drinking water supply.

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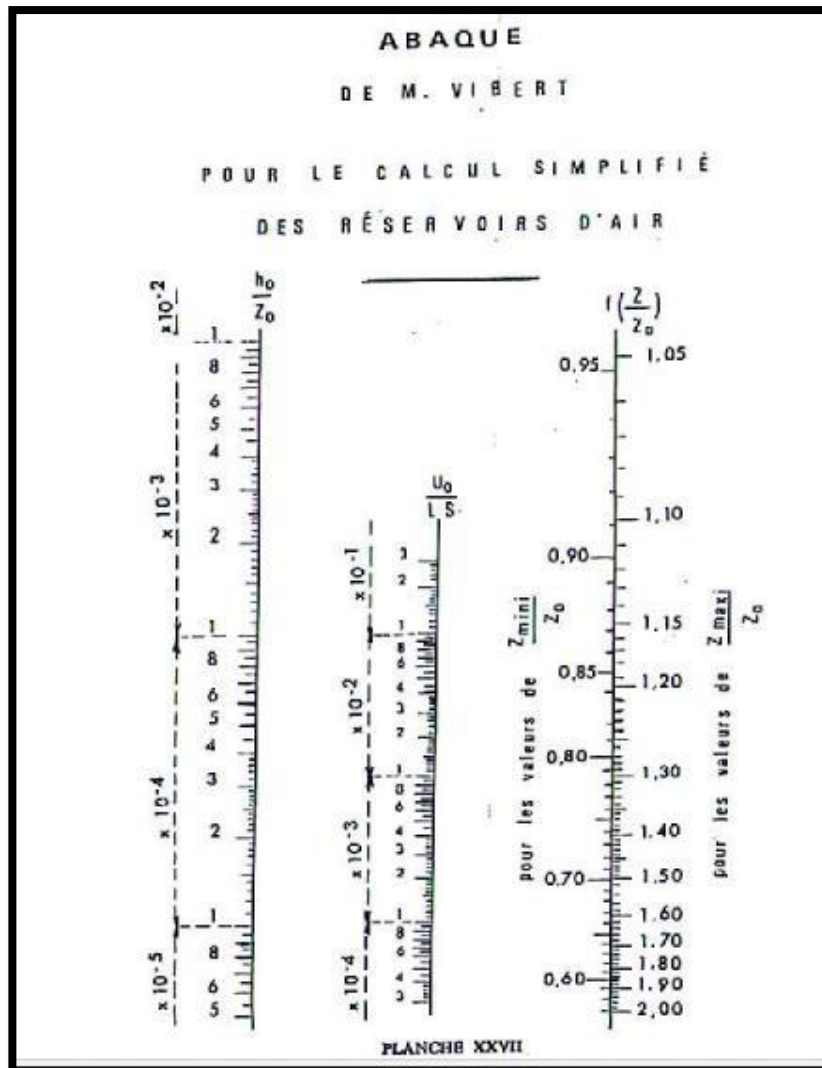
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Site :

HDPE PN 16 pipe price : (http://www.algerie.prixconstruction.info/espaces_urbains/Reseaux_divers/Distribution_d_eau/Eau_potable/Tube_en_polyethylene.html)

ANNEX :

VIBERT abacus:



References

Repair of hourly flow rates as a function of the number of inhabitants :

Hours	Number of inhabitants				
	Less than 10,000	From 10001 to 50000	From 50001 to 100000	More than 100,000	Rural conurbation
0-1	1	1,5	3	3,35	0,75
1-2	1	1,5	3,2	3,25	0,75
2-3	1	1,5	2,5	3,3	1
3-4	1	1,5	2,6	3,2	1
4-5	2	2,5	3,5	3,25	3
5-6	3	3,5	4,1	3,4	5,5
6-7	5	4,5	4,5	3,85	5,5
7-8	6,5	5,5	4,9	4,45	5,5
8-9	6,5	6,25	4,9	5,2	3,5
9-10	5,5	6,25	5,6	5,05	3,5
10-11	4,5	6,25	4,8	4,85	6
11-12	5,5	6,25	4,7	4,6	8,5
12-13	7	5	4,4	4,6	8,5
13-14	7	5	4,1	4,55	6
14-15	5,5	5,5	4,2	4,75	5
15-16	4,5	6	4,4	4,7	5
16-17	5	6	4,3	4,65	3,5
17-18	6,5	5,5	4,1	4,35	3,5
18-19	6,5	5	4,5	4,4	6
19-20	5	4,5	4,5	4,3	6
20-21	4,5	4	4,5	4,3	6
21-22	3	3	4,8	4,2	3
22-23	2	2	4,6	3,75	2
23-24	1	1,5	3,3	3,7	1
Total	100	100	100	100	100

Source :(Ouvrage d'AEP D'ABRAMOV).

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