LECTURE NOTE

On

WATER SUPPLY AND WASTE WATER ENGINEERING

RAAJDHANI ENGINEERING COLLEGE

DEPARTMENT OF CIVIL ENGINEERING

DIPLOMA 5th SEM

Prepared By- Prof. Achintya Sahoo, Assistant Professor, Dept. of Civil Engineering

Objective:

Equip students with knowledge and skills on water sources selection, collection, water supply equipment and structures design, as well as wastewater collection and disposal.

Learning outcomes:

The student is expected to be able to identify, characterize and quantify different kinds of water sources, estimate water demand, design structures for water intake, treatment, storage, and distribution. He should also be able to identify different sources of wastewater, characterize and quantify the wastewater and design of major sewers.

Content:

1. **GENERALITIES**

I. Basic considerations
   - i. Getting interested in water
   - ii. Water requirements (different water uses)
   - iii. Water cycle

II. Water supply technologies
   - i. Rainwater
   - ii. Surface water
   - iii. Ground water
   - iv. Advantages and disadvantages
   - v. Decision making in selection of a suitable water source

2. DESIGN

I. Population study

  i. Population forecasting techniques

  ii. Factors affecting the population growth

II. Water demand

  i. Factors affecting water demand

  ii. Water consumption

  iii. Design period for a water supply project

III. Components of a water supply project

   i. Intake structures
   ii. Transmission lines
   iii. Water treatment plants
   iv. Pipe network and storage tanks
   v. Pumping station and pumps selection

IV. Water quality and water treatment techniques

   i. Composition of water
   ii. Main pollutants and their effects
   iii. Drinking water quality
   iv. Measurement of water quality
   v. Procedures for water treatment

3. SANITATION

I. Wastewater estimation
II. Wastewater composition
III. Design and construction of sewers

References:

1. W.Viesmsman, Jr., et al (2009). *Water Supply and pollution control, 8th ed*. New Jersey: Pearson Pretence Hall.
2. L.W. Mays, *Water distribution systems Handbook* (New York: McGraw-Hill, 2000).
3. Action contre le faim. Design, sizing, construction and maintenance of a gravity- fed system in rural areas. Updated 2008.
4. Babbitt, H. E. & Doland, J. J. (1955) *Water supply engineering, 5th ed*., New York.
5. Metcalf & Eddy, Inc., Wastewater Engineering: Treatment and Reuse (New York: McGraw-Hill, 2003).
6. American Water Works Association and American Society of Civil Engineers, *Water Treatment plant Design 4th ed.,* (New York, NY: McGraw-Hill, 2005).

1. GENERALITIES

I. Basic considerations

i. Getting interested in water

→ **Water is vital**

All life depends on water, which is the main component of living cells: from less than 40% (certain species of plants) to more than 95% (jellyfishes). The human body contains about 80% of water, i.e. 4/5 of its weight. A human body of 60 kg is therefore made up of 48 kg of water and 12 kg only of other substances. A day without water is sufficient for the brain to become unable to work properly (obsessed by the lack of water) and one can die in 2 or 3 days without water.

→ **Water is essential in any human activity.**

People use water to carry out a large number of activities such as:

- Agriculture (irrigation) and cattle breeding (animals).
- Industry (cooling, dissolution, washing, dilution).
- Electricity production (dams and hydropower plants).
- Cooking (to prepare meals).
- Cleaning (clothes, dishes, cars, floors in the home …).
- To move and to transport equipment (people can use rivers as communication mean).
- Distraction and relaxing (swimming and bathing places).

→ **Human living standard depends on water availability**.

The level of comfort and the development of a population can be measured by the quantity of water this population consumes, per person and per day. Therefore, considering a country where people consume 500 liters of water per person per day, the country where people consume 1000L/p/d can be considered as wealthier. Drought is a main factor of poverty and famine.

→ **Dirty water is lethal**

Dirty water is the cause of more than half of the known diseases, which can be divided in 4 groups, according to their ways of transmission:

- **Water-borne diseases** represent the category of illness caused by drinking water contaminated by human or animal faeces, which contain pathogenic micro organisms. Water-borne diseases include cholera, hepatitis A, diarrhea, typhoid…;

- **Water-washed diseases** are illness developed in conditions where freshwater is scarce and sanitation is poor. Infections are transmitted when too little freshwater is available for washing hands or body. Water-washed diseases include trachoma, leprosy, tuberculosis, whooping cough, tetanus, and diphtheria;

- **Water-based diseases** are caused by aquatic organisms that spend part of their life cycle in the water and another part as parasites of animals. Water-based diseases include guinea worm (dracunculiasis), paragonimiasis, clonorchiasis, and schistosomiasis (bilharzia);

- **Water-related vector diseases** are infections that are transmitted by vectors— insects or other animals capable of transmitting an infection, such as mosquitoes and tsetse flies—that breed and live in or near both polluted and unpolluted water. Water-related vector diseases include malaria, dengue, yellow fever….

ii. Water requirement

✓ **What is the quantity of water needed per capita per day?**

The notion of water quantity needed per capita and per day is subjective because it depends on many factors:

- Climate (air temperature).
- Intensity of physical activity.
- Uses made of water (drinking, cooking, bathing, washing, irrigation…).
- Cultural habits (lifestyle, cooking practices). etc

The average quantity of water used per capita and per day hence varies from a region to another one throughout the world. While an European consumes in average 50 liters per day, an American consumes four times more and inhabitants of developing countries four times less. Guidelines values for water supply were elaborated by international organizations and acknowledged as norms in terms of public health to answer to humanitarian and priority needs. These guidelines are only indicative and must be adapted according to the context (density of population, climate, and national sanitation norms). Standards are also defined by the national ministry in charge of the water and sanitation sector.

Two types of norms are usually indicated:

- The minimal values, under which risks of outbreak spreading are real and the right of populations to live with dignity is not respected.
- The sanitation norms which correspond to priorities used in a stable political situation.

The guidelines values concerning the necessary quantities of water are given in table 1.

✓ **What are the domestic water needs?**

It is the quantity of water considered necessary to cover needs at the level of house (family).

The domestic needs cover mainly the following activities:

- drinking,
- food preparation,
- washing, cleaning,
- personal hygiene,
- toilets

✓ **How much water must be available?**

To cover sufficiently the water needs of a population, it is not enough to provide water in sufficient quantity, it is also necessary to ensure a good access to this water:

- The water points should not be too far.
- The users must be able to transport and store easily the water in their home (if necessary, the distribution of transport and storage containers should be anticipated).
- The waiting time at the water point must be acceptable.

The guidelines values concerning water availability are given in table 1.

Table 1. Guidelines values for water supply (Action contre la faim, 2008)

	Vital minimum	Sanitation standards
WATER QUANTITY STANDARDS		
Domestic needs	7-20 l/pers/d	30-60 l/pers/d
Health centre	10 l/pers/d	
Hospital	50 l/bed/day	50-220 l/bed/d
School	10 l/student/d	15-30 l/student/d
Market	Used by ACF: 10l/pers/day	
Temple/mosque/church	Used by ACF: 5l/visitor/day	
Small size cattle (goats, pigs)	5 l/animal/d	10-20 l/animal/d
Large size cattle (cows)	30-60 l/animal/d	
ACCESSIBILITY/AVAILABILITY		
Maximum distance between users and water point	125 to 250 m	
Maximum number of users per water point	600	150

iii. Water resources

- ✓ **Where does water come from?**

The Earth works as a huge distillation machine, where water evaporates continuously then condenses and falls again on the surface of the globe. This dynamic process is called the water cycle as represented in figure 1.

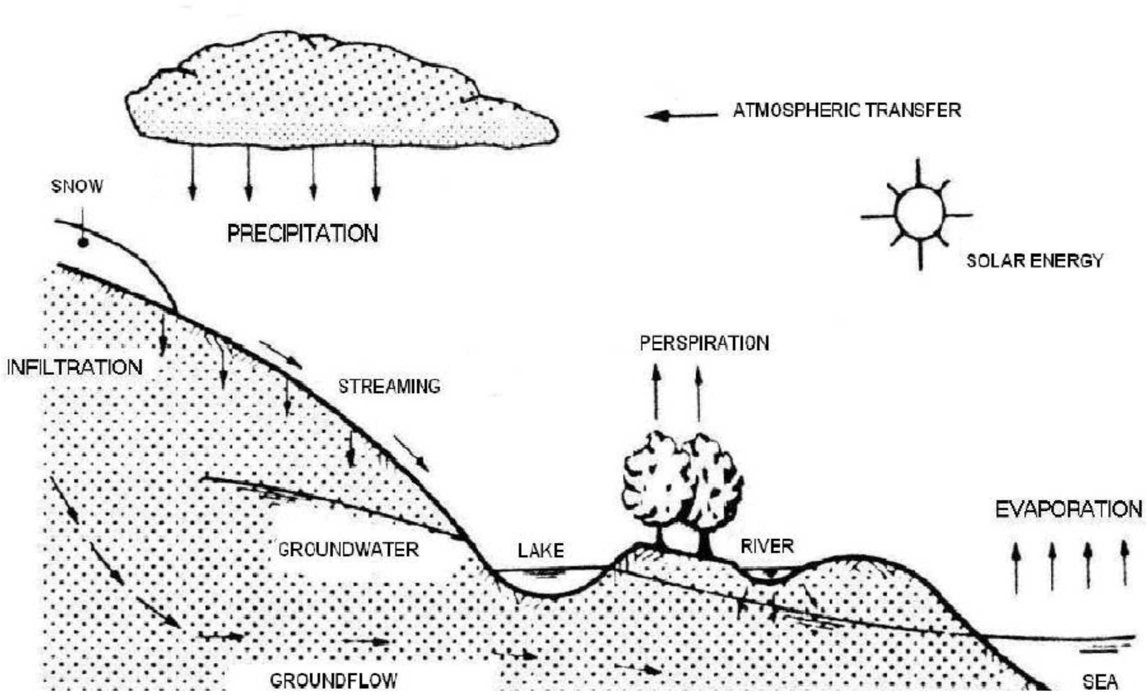


Fig.1: Water cycle

All water which arriving on the surface of the earth falls therefore from the clouds in the form of rain (or of snow in cold countries). Once arrived on earth, water follows the three following ways:

- One part flows rapidly at the surface of the ground.
- One part evaporates (transformed into steam).
- One part infiltrates into the ground.

Water which flows on surface:

By gravity, (gravity being the attraction force exerted by the Earth to all objects) water is attracted towards the lowest points of the ground surface. Following the slopes and natural relief of the ground, water tries to find its way, flowing first in the form of small streams, which merge to form bigger streams. These merge again to form rivers, which flow finally in to the sea.

Water which evaporates:

On the continents, part of rainwater evaporates at the first contact with the ground heated by the sun. This phenomenon is the cause of the white haze formed above the road after a short rain:

this haze is just steam. A part of the surface water and shallow groundwater can also evaporate under the influence of the sun.
However, the most important continental contribution of steam water is represented by the part of rainwater which evaporates in the air through plants: right after having penetrated in the plant's roots, shallow ground water is absorbed; goes up in plants stem and tree trunks until the leaves where it evaporates.

A large part of water also evaporates above the oceans, under the influence of solar energy.
The water that evaporates reaches the atmosphere to form clouds and is subject to the various winds and movements of transfer.

Water, which infiltrates in the ground:
A part of rainwater infiltrates in the ground through a multitude of small interstices (small empty spaces). This way, presenting numerous obstacles to be by-passed, is of course very long and difficult. This water, which had become underground water, keeps on being part of the water cycle: it forms groundwater, which flows slowly below the ground and supply springs, rivers and eventually discharge into seas.

The Earth has four large reservoirs of water: the sea (the oceans), the atmosphere (the sky), groundwater, and surface water (rivers/lakes). Volumes and duration of stay of the water in each of these reservoirs are represented in table 2. However, the local relief, climate or geology makes the water cycles varying in an important way from a region to another.

Table 2: Large natural reservoirs of the planet.

Reservoir	Capacity (%)	Average duration of stay of water
Seas (oceans)	80	3,172 years
Atmosphere	0.3	4 months
Rivers and lakes	0.1	5.6 years
Groundwater	19.6	8,250 years

II. Different Water Supply Technologies

Among the above described water resources, the types of water appropriate for the water supply are:

- Rainwater.
- Surface water: streams, river, lake, pond.
- Groundwater.

i. Rainwater

The use of rainwater is a common technique in many countries.

The rainwater is collected on an equipped surface (generally the roof of a house) and is brought by gutters in a storage reservoir.

This technique of water supply can prove to be interesting in certain areas, either where it rains a lot, either very dry when the water resources are very scarce and any source of water should be used. Because of the great storage volumes required, this technique is used as a temporary resource in complement to others.

In all cases, the main disadvantage of this water resource comes from the quality of water, polluted and charged with sediment during the filling (roof dusty or covered with animal excrements for example), and contaminated after a long period of storage. Moreover, the lack of minerals makes rainwater (often acid) corrosive towards metals (because no limestone is present to neutralize acidity), and if the rainwater is the only source of drinking water for the population, it might be necessary to add minerals to the food (iodine for example…) to compensate their absence in the drinking water.

An example of rainwater collection is given on figure2.

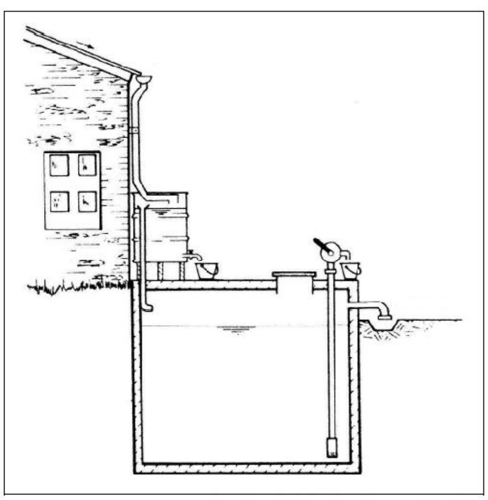


Fig.2: Rainwater collection through the home roof.

ii. Surface Water

Surface water has the advantage to be usually easily accessible (ponds, lakes, rivers); near of the people's living place, but unfortunately this water is extremely vulnerable to pollution (suspended substances, pathogens). Some of the surface water points can dry out in dry season. Surface water sources are varied and it exists numerous ways to use them.

• ***Direct intake from the river***

Simple and quick solution which requires a minimum level of infrastructures: creation of catchment areas along the river according to the different use made of the water (washing, bathing…see figure 3). Upstream of all other activities, a zone will be reserved to fetch water for human consumption.

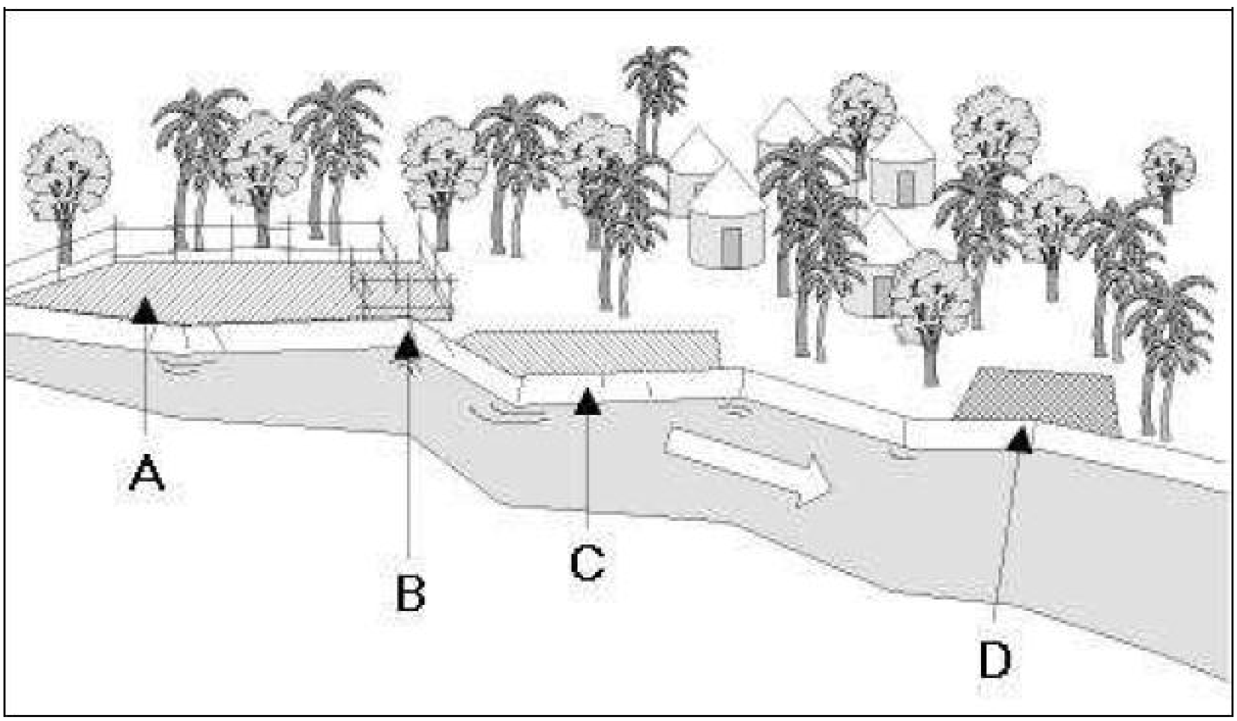


Fig.3: Example of water fetching point along the river

(A: closed perimeter in an upstream part to fetch water for domestic usage, B: protected access to this perimeter (for example chicane access), C: zone for washing and bathing, D: downstream zone reserved for the animals).

• ***Direct river catchments***

Rivers catchments are done through a pipe equipped with an inlet filter connected to a pump. Water is pumped in a reservoir then distributed through a distribution network. A water treatment will always be necessary given all the possible risks of pollution.

The yield which it is possible to pump depends on the type of pump, the population to be served and the river flow (average yield along the year, taken into account also the lowest yield, i.e. dry season).

The catchments should be done upstream of the populations living place, where risks of pollution are lower. It is necessary to clean river banks on a certain distance and a dam/reservoir can be necessary to stabilize a river if this one has an irregular flow.

An example of direct rivers catchment is given in figure 4.

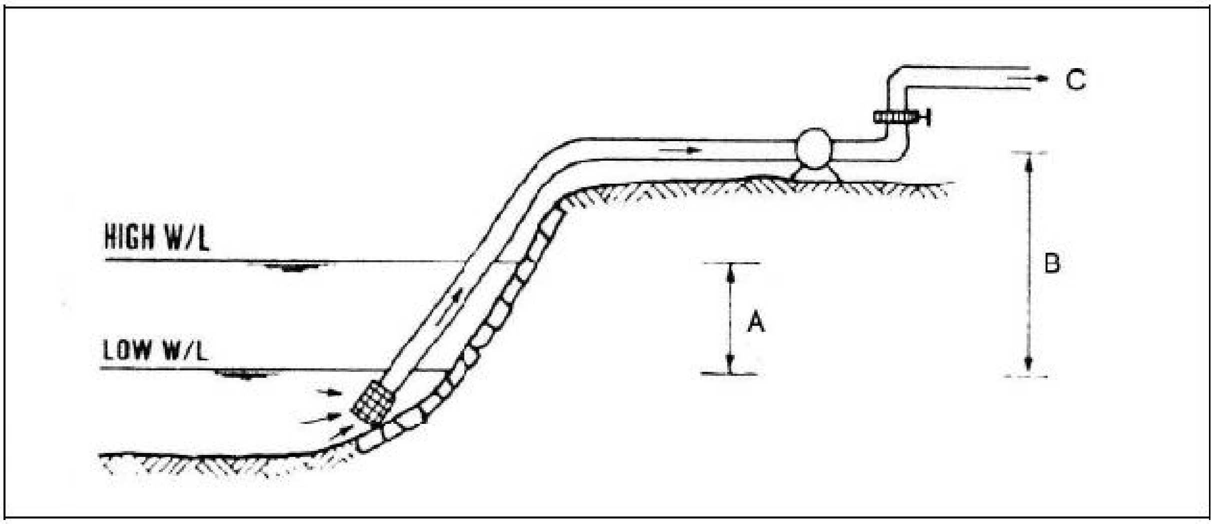


Fig.4: Example of direct river catchments.

A: variation between the highest and the lowest level (W/L=water level) of the river,

B: suction height limited by the type of pump used, **C**: leading to a reservoir / treatment unit.

• ***Indirect river catchments***

It is possible to improve the quality of river water by using as a natural filter the alluvium layer often located under the bedrock: infiltration well, bottom filters, and infiltration galleries or even a borehole drilled in the alluvium. An example of indirect river catchment of is given in the figure 5.

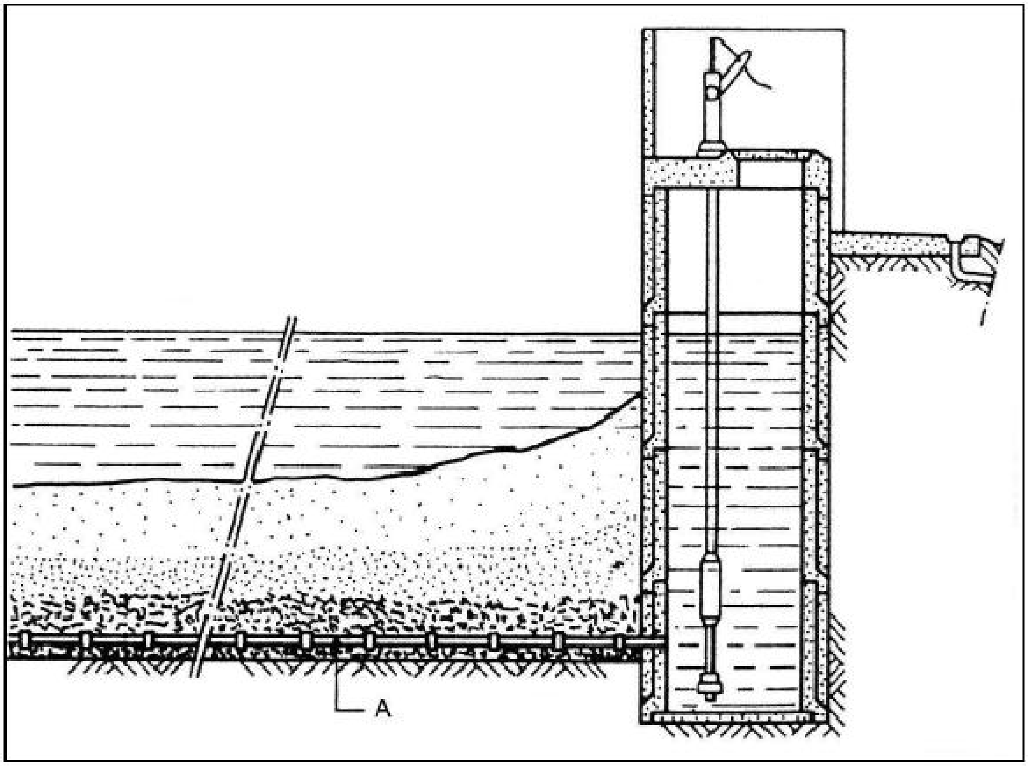


Fig.5: Indirect river catchments with the support of drainpipes (**A**) laid horizontally under the river bed.

• ***Catchments from a lake or from a dam***

When a stagnant water (or still water) is in good conditions, it presents the best resource for surface water. Indeed, if the water duration stay is long enough, harmful substances either settle in the bottom of the lake/pond, or accumulate at the surface, or can also oxidize and then settle down.

The catchment should not be too deep, so that the water caught is clear and oxygenated enough, and should not be too shallow, so that water does not contained too much organic substances (indeed, organic substance often develop near the water surface where more sunlight is available).

Ideally, the depth of the catchment should be between 10 and 20 m. One must take care that the water resource does not contain any wastewater.

iii. Groundwater

Groundwater presents in general a good bacteriological quality, due to their long decantation and auto-purifying properties of the ground. They are therefore often appropriate for consumption: in that case, the treatment will not be necessary. The deeper the catchment is, the

better the quality of water will be. However, they can also present mineral pollutions, emanating from the rocks dissolved by water in the ground (notably iron, sodium chloride, calcium, magnesium and fluorine).
Moreover, rocks present in a crystalline ground can present cracks and can therefore contain groundwater polluted by direct transfer of surface pollutions in the cracks (absence of usual filtration process by the soil). In addition, any groundwater located at a depth lower than 3 m is considered to be surface water.

• ***Spring catchment***

A spring is a natural flow of a groundwater reserve in the open air. It is the easiest way to use groundwater. The objective of a spring catchment is to use at best the spring flow, while protecting it from outside pollutions, especially from faecal origin. Every spring catchment is a particular case: it is therefore not possible to offer an example adapted to all situations.
They can however be classified in two types: reinforced concrete catchments box, or buried drains. An example of spring catchments is given in the figure 6: the catchment is made by the mean of a drain; once collected by the drain, water goes directly to a header tank.

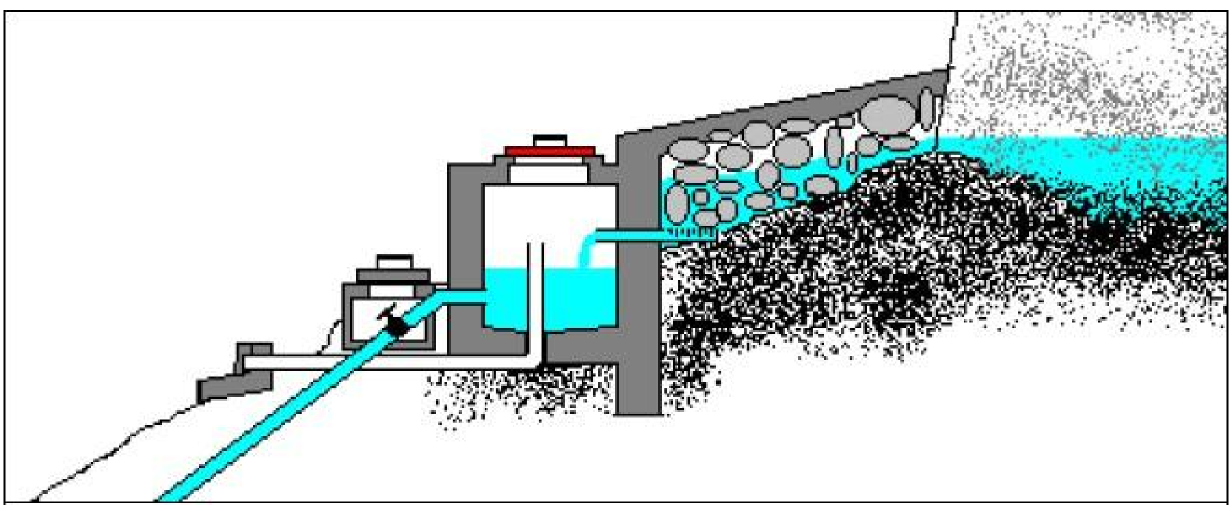


Fig.6: Example of spring catchment

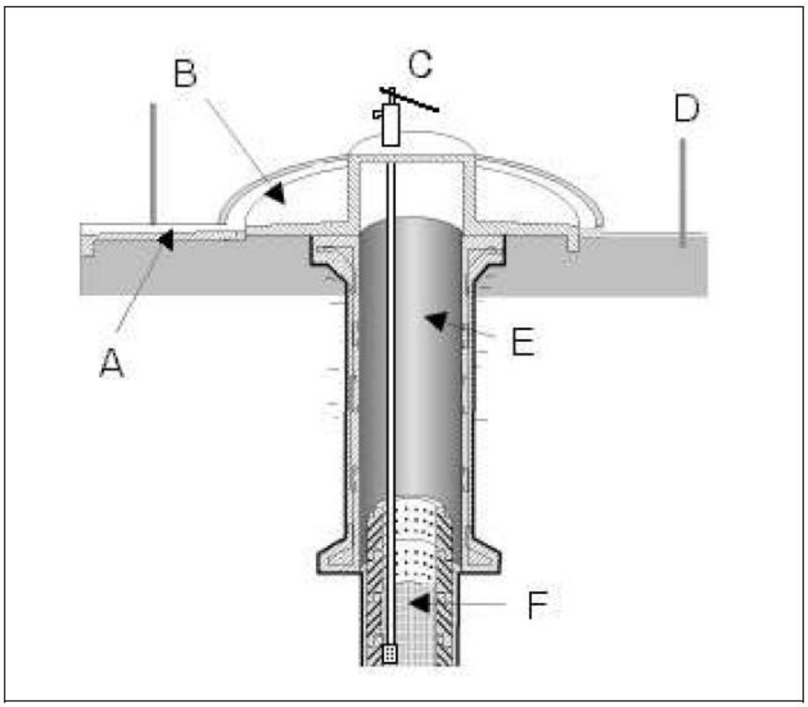
• ***Hand Dug Well***

A well is an infrastructure dug manually, which collects a moderately deep or deep aquifer.
Its depth and diameter vary according to the context. Generally speaking, the diameter of a well is from 1 to 2 meters and its depth does not exceed 25 to 30 meters. After this depth, work is very dangerous and expensive.

The most important parts of a well in terms of water quality are the surface infrastructures, which protects it from of surface water infiltrations and facilitates the water access and use. The surface infrastructures are designed to:

- drain surface water towards the outside perimeter of the well (construction of apron, drainage channel, soakage pit)
- limit the risks of falls of the well users inside the well (well head)
- limit the access of the animals (well fence, well head)
- Limit the pollution of the well water during the fetching operation (cover slab, well head, well door, pumping system). Ensure that the well head is low enough to be accessed by children, and the door can be opened by them also.

An example of properly built well is given in figure 7:



- A: Drainage ditch that prevents stagnant water and allows the surrounding of the well to stay clean.
- B: Apron which avoids the formation of muddy land and prevents the infiltration of water along the well lining. The apron must be built with a sufficient slope so that stagnant water can be easily evacuated.
- C: Pumping system appropriate for all the users (manual or electric pump, pulley, winch…).
- D: Fence which prevents the well access to the animals.
- E: Sealed lining on the first three meters to prevent surface water to infiltrate in the well.
- F: Water catchment area that must be always inside water and clean

• ***Borehole***

A borehole is deep ground water catchment. It consists of a long-narrow tube inserted in the ground by the mean of a drilling machine. Its usual diameter is 10 to 20 cm. The drilling of a borehole requires most of the time the intervention of a drilling machine (expensive!).

However, manual drilling, at low cost, under good conditions, can reach 60 m of depth.

Borehole must be equipped with a pump (powered by hand or not) and with roughly the same surface infrastructure than a hand dug well (reinforced concrete apron, drainage ditch and fence which prevents the access to the animals). A standard example of borehole is given on figure 8.

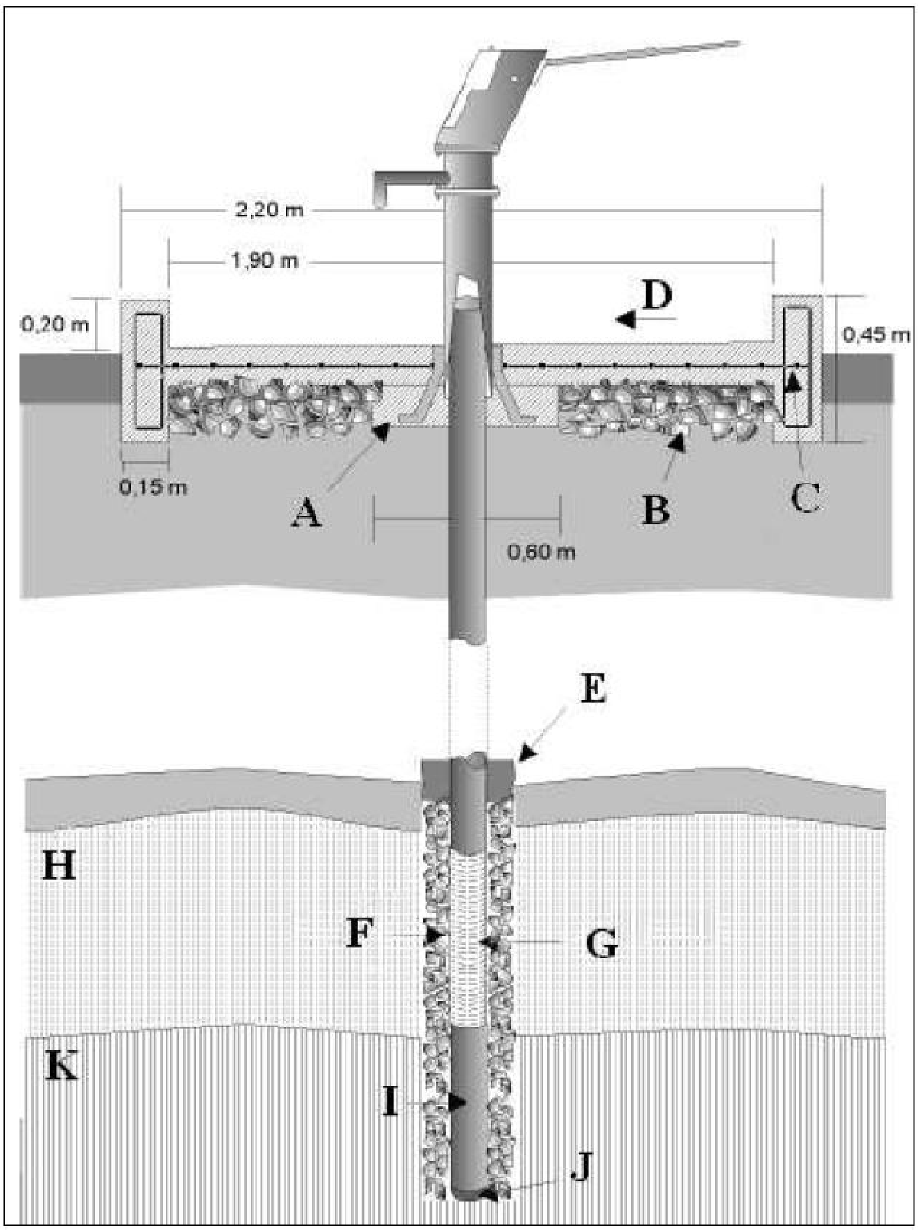


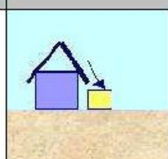
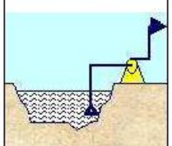
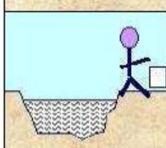
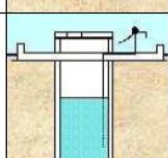
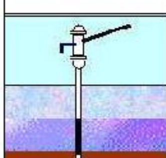
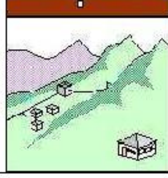
Fig.8: Cross section of an equipped borehole

A: hand pump concrete anchoring, b: stone foundation, C: iron bars, D: slope of 2% to allow drainage, E: cement plug, F: gravel filter, G: filter pipe (screen), H: aquifer, I: PVC sedimentation tube, J: bottom plug, K: substratum = bedrock).

iv. Advantages and disadvantages of various technologies of water supply

Every technology has its advantages and disadvantages. A résumé of features of the various water supply technologies is given in table 3.

Table 3. Characteristics of various water supply technologies

Water resources	Water supply technology		Water supply system	Advantages	Disadvantages
Rainwater		Rainwater collection	Reservoir with tap	- Good bacteriological quality if collected under good conditions.	-Temporary or complementary resource. -Not containing minerals and can be easily polluted
Surface water		River or lake intake	Pumps, reservoir and taps	- Rapid to implement	-Requires generally a treatment unit. -Requires a pump to lift the water.
		Direct river intake	None, or very simple (budket, jerrican)	Easy and quick to implement	- High probability of faecal pollution - Water can be very turbid
		Hand dug wells	Pump (powered by hand or not), pulley, bucket…	- Clear water and good bacteriological quality if well is deep and well protected.	-Can request a lot of work for the digging/construction (especially for deep wells). -Water can be easily contaminated if well badly protected.
Groundwater > 3 meters		Borehole	Pump (powered by hand or not)	- Clear water and good bacteriological quality. - Allows deep water catchments.	- Request a lot of work for the construction -Borehole cost can be very high.
		Spring catchments	Gravity Fed System and tapstand	- Clear water and good bacteriological quality. - Easy to use.	- Flow is not always sufficient to cover population water needs.

v. Suitable water source selection

To choose the most suitable water resources it is necessary to take into account several factors such as:

- <u>Technical Criteria</u>:
  - Operation costs.
  - Complexity of the commissioning and the maintenance (required people?).
  - Spare parts availability on the local market?
  - Use of local or imported materials?
- <u>Environmental Criteria</u>:
  - Water quality.
  - Potential contaminations.
- <u>Community Criteria</u>:
  - Organization capacity and motivation.
  - Financial capacity to pay for the maintenance of the infrastructures.
  - Technical skills availability within the community.
- <u>Institutional Criteria</u>:
  - Design and model validated by the national policy and relevant ministries
  - Capacity of the local authorities to assist the community.
  - Available technical assistance and follow-up.
  - Available budget.

The choice must go for the most durable solution. It is obvious that the maintenance of the installation in the medium and long term plays an important role in the choice of the resource, and particularly the type of installation envisaged which will have to be managed with by the users. It is necessary to avoid the construction of a pumping or treatment station that nobody will be able to operate (because of the high price of the consumable like fuel, spare parts, maintenance, or because the full understanding of the system not possible for the community due to their low education level).

For the drinking water, groundwater resources should be chosen in priority, when they are accessible, as compared to surface water source, river or lake, for quality and cost reasons.

The realization of a gravity fed system based on a spring catchment is an interesting solution, because it allows having a groundwater available with minimal operation costs.

It is thus preferable to choose firstly (see figure 9):

1. Spring catchments with gravity fed network.
2. Deep groundwater (well and boreholes of at least 10 meters deep).
3. Medium-deep groundwater (well and boreholes from 3 to 10 meters of depth).
4. Surface water.

The order of priorities quoted above is to be respected in the majority of the cases. It is necessary however to use one's common sense for the choice of the water resource and it is possible that situation will occur where this priorities order must be reversed. For example, if there is near a village a groundwater rich in arsenic and a river not contaminated by upstream industries, the river should be used as water resource. Indeed, although potentially contaminated by pathogens, the river offers better quality water from a chemical point of view (absence of toxic pollutant).

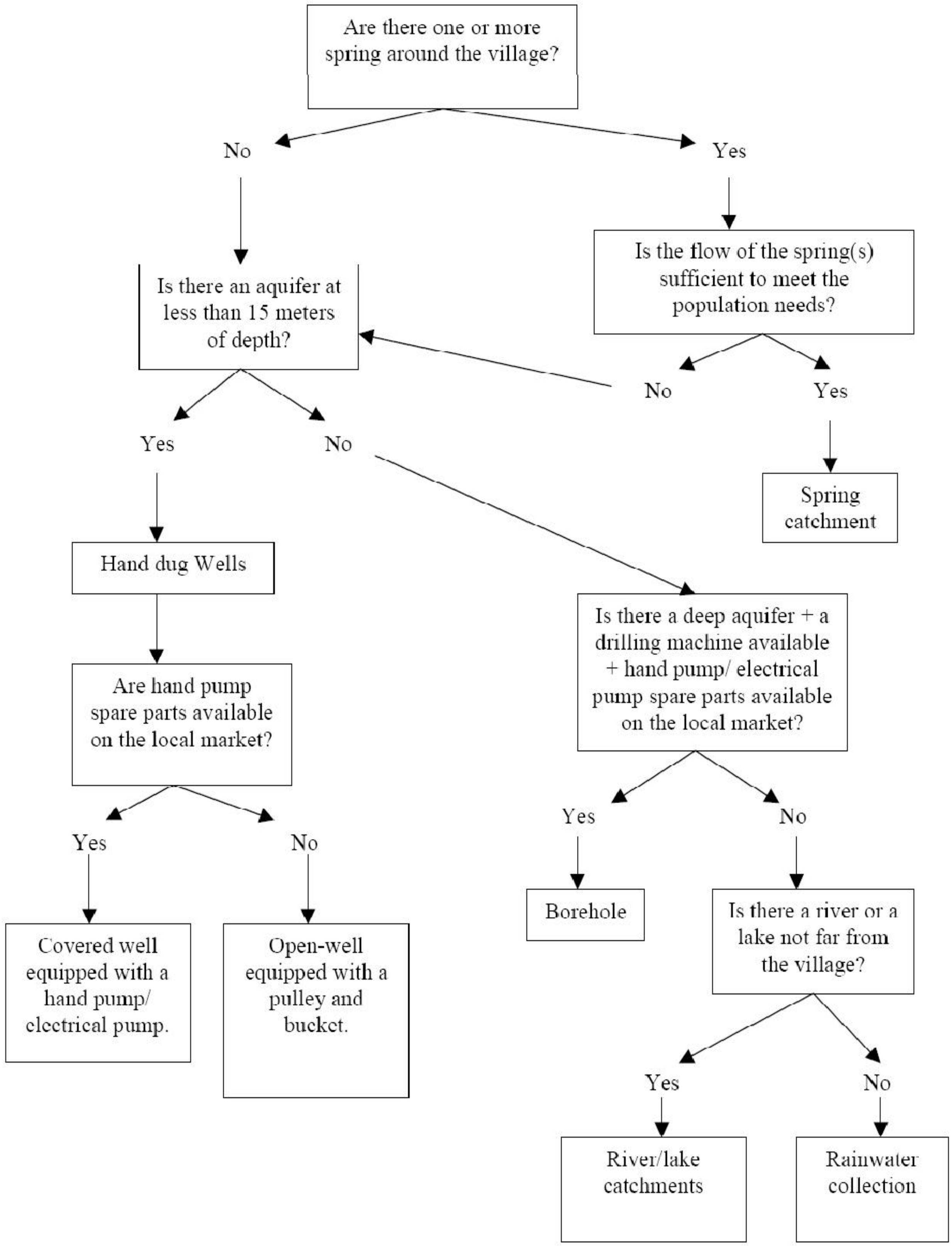


Fig. 9. Decision making flowchart in source selection

2. DESIGN

I. Population study

The water supply problem is one of balancing supply and demand. The geographical and temporal availability of water sources, the quality of these resources, the rates at which they are replenished and depleted, and the demands placed upon them by the water users are determining factors in water management strategies. Estimates of future water uses, uncertain as they might be, are fundamental to efficient and equitable allocation of water supplies. These estimates depend on ability to forecast changes in population, agricultural and industrial activity, ecologic and economic conditions, technology, and social and other related factors.

i. Population forecasting

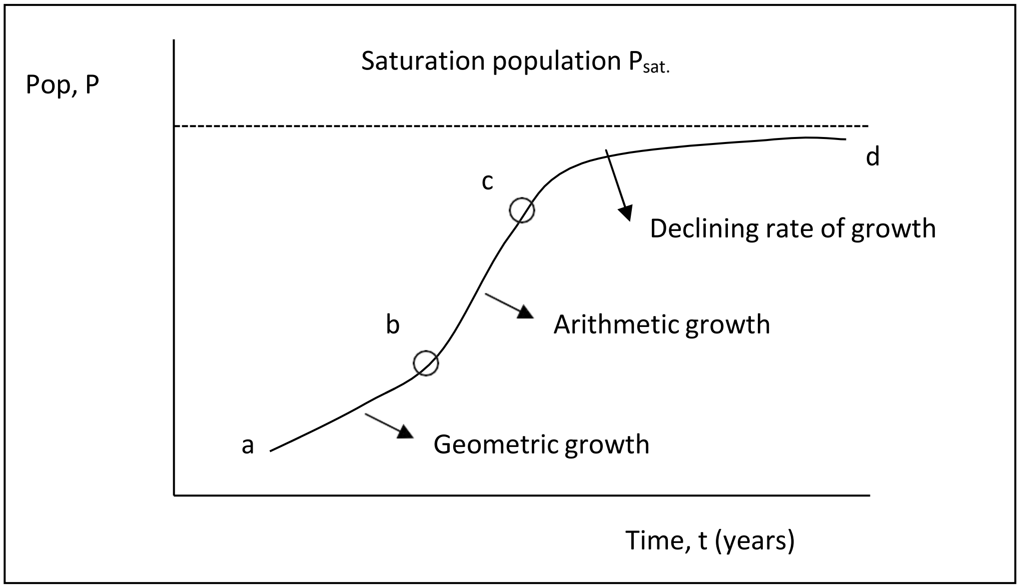
The success of forecast lies in the judgment of the forecaster on the reliability of his assumptions. Most methods pertain to trend analysis, wherein future changes in population are expected to follow the pattern of the past. There are four broad categories of population forecasting techniques: 1) graphical, 2) mathematical, 3) ratio and correlation, and 4) component methods.

Two types of population predictions required are 1) short-term estimates for 1 – 10 years, and 2) long-term 10 – 50 years

We shall consider only the short term technique.

Short term estimates

Graphical and/or mathematical methods are use for short-term estimates which are essentially trend analyses in graphical or mathematical form. The mathematical approach assumes three forms of population growth: geometric, arithmetic growth, and declining rate of growth. Each segment has a separate relation. The historic population data of study area may be plotted and according to the shape of the plot, the relationship of that segment should be used for the population projection.



Graphical Extension Method

This consists of plotting the population of the past years against time, sketching a curve that fits the data, and extending this curve into the future to obtain the projected population. It is better to plot the data by a semi log or logarithmic plot.

Arithmetic Growth Method

His method considers that the same population increase takes place in a given period. Mathematically,

$$\frac{dP}{dt} = K_a$$

By integrating, we obtain

$$\boldsymbol{P_t = P_o + K_a t}$$

where P_t = projected population, P_o = present population, t =period of projection, and

$$K_a = \frac{P_2 - P_1}{\Delta t}$$ where P_1 and P_2 are recorded populations at some Δt interval apart.

Geometric Growth Rate

This considers that the increase in population takes place at a current population given mathematically by

$$\frac{dP}{dt} = K_G P$$

and by integrating we obtain

$$\ln P_t = \ln P_o + K_G t$$

where $$K_G = \frac{\ln P_2 - \ln P_1}{\Delta t}$$

Declining Growth Rate Method

This assumes that the area has a saturation population and the rate of growth becomes less as the population approaches the saturation level. Thus the rate of increase is a function of the population deficit (P_{sat} – P) i.e.

$$\frac{dP}{dt} = K_D \left(P_{sat} - P\right)$$

and upon integration, we have $$p_t = p_{sat} - \left(p_{sat} - p_0\right) e^{-k_D \Delta t}$$

rearranging,

$$K_D = -\frac{1}{\Delta t} \ln \left(\frac{P_{sat} - P_2}{P_{sat} - P_1} \right)$$

Comparative Graphical Method

In this method, the cities having conditions and characteristics similar to the city whose future population is to be estimated are selected. It is then assumed that the city under consideration will develop, as the selected similar cities have developed in the past.

Ratio Method

In this method, the local population and the country's population for the last four to five decades is obtained from the census records. The ratios of the local population to national population are then worked out for these decades. A graph is then plotted between time and these ratios, and extended up to the design period to extrapolate the ratio corresponding to future design year. This ratio is then multiplied by the expected national population at the end of the design period, so as to obtain the required city's future population.

Drawbacks:

1. Depends on accuracy of national population estimate.
2. Does not consider the abnormal or special conditions which can lead to population shifts from one city to another.

II. Assessing Technical Demand for Design Capacity

When designing a water supply system, it is necessary to know how much water and at what quality should be provided. This will vary between countries and it is helpful to measure how much is used from existing improved system elsewhere in the region if local data are not readily available. For non-availability of data, the tables below could be useful.

i. Water Quantity Estimation

The quantity of water required for municipal uses for which the water supply scheme has to be designed requires following data:

1. Water consumption rate *(Per Capita Demand in litres per day per head)*
2. Population to be served.

Quantity = Per capita demand x Population

ii. Water Consumption Rate

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption. The various types of water demands, which a city may have, may be broken into following classes:

<u>Water Consumption for Various Purposes:</u>

	Types of Consumption	Normal Range (lit/capita/day)	Average	%
1	Domestic Consumption	65-300	160	35
2	Industrial and Commercial Demand	45-450	135	30
3	Public Uses including Fire Demand	20-90	45	10
4	Losses and Waste	45-150	62	25

Typical domestic consumption

Type of water system	Average consumption (litres/person/day)	Range (litres / person /day)
Communal water point (e.g. well, handpump or standpost i) round trip walking distance 500 – 1000 m ii) round trip walking distance 250 – 500 m iii) round trip walking distance <250 m	 20 20 25	 10 – 25 15 – 25 15 - 50
Yard tap (i.e. water point outside house but in house compound)	40	20 – 80
Water point inside house i) single tap ii) multiple taps	 50 120	 30 – 80 70 – 250

Allowance is made for water used for livestock watering needs and also water for irrigation needs. However, as much as possible, irrigation water needs and water for livestock are usually not considered as part of domestic water supply.

Fire Fighting Demand:

The per capita fire demand is very less on an average basis but the rate at which the water is required is very large. The rate of fire demand is sometimes treated as a function of population and is worked out from following empirical formulae:

	Authority	Formulae (P in thousand)	Q for 1 lakh Population)
1	American Insurance Association	Q (L/min)=4637 < P (1-0.01 < P)	41760
2	Kuchling's Formula	Q (L/min)=3182 < P	31800
3	Freeman's Formula	Q (L/min)= 1136.5(P/5+10)	35050
4	Ministry of Urban Development Manual Formula	Q (kilo liters/d)=100 < P for P>50000	31623

iii. Factors affecting per capita demand:

a. Size of the city: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.
b. Presence of industries.
c. Climatic conditions.
d. Habits of people and their economic status.
e. Quality of water: If water is aesthetically $ medically safe, the consumption will increase as people will not resort to private wells, etc.
f. Pressure in the distribution system.
g. Efficiency of water works administration: Leaks in water mains and services; and unauthorized use of water can be kept to a minimum by surveys.

h. Cost of water.
i. Policy of metering and charging method: Water tax is charged in two different ways: on the basis of meter reading and on the basis of certain fixed monthly rate.

iv. Fluctuations in Rate of Demand

Average Daily Per Capita Demand

= Quantity Required in 12 Months/ (365 x Population)

If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

- ***Seasonal variation***: The demand peaks during summer. Firebreak outs are generally more in summer, increasing demand. So, there is seasonal variation.
- ***Daily variation*** depends on the activity. People draw out more water on Sundays and Festival days, thus increasing demand on these days.
- ***Hourly variations*** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand. The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases.

Maximum daily demand = 1.8 x average daily demand

Maximum hourly demand of maximum day i.e. Peak demand

= 1.5 x average hourly demand

= 1.5 x Maximum daily demand/24

= 1.5 x (1.8 x average daily demand)/24

= 2.7 x average daily demand/24

= 2.7 x annual average hourly demand

v. Design Periods

A water supply scheme will have a certain design life beyond which it may not be possible to continue without major rehabilitation or new works. The likely increase of population during the design life needs to be considered at the design stage. This quantity should be worked out with due provision for the estimated requirements of the future. The future period for which a provision is made in the water supply scheme is known as the design period.

Design period is estimated based on the following:

- Useful life of the component, considering obsolescence, wear, tears, etc.
- Expandability aspect.
- Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
- Available resources.
- Performance of the system during initial period.
- Availability of materials

The following table gives a rough idea on design life of some components

Components	**years**	**Remarks**
Ground water wells	5	As additional wells are easy
River intakes	40	Difficult expansion
Pipe lines & Distribution system	40	Difficult expansion
Water treatment plant	10-15	Easy expansion Designed for max daily flow
Pumping plant	10	Easy expansion
Storage tanks	10	Easy expansion Designed for max hour demand + fire

III. Design of some components of a water supply system

A **Water supply system** (*water works*) is complex system of structures performing the duty of water supply, that is getting it from the natural source, purifying it transporting and distributing it to the users. The basic components of water supply system include:

- collection works,
- transmission works,
- purification works, and
- distribution works

Collection works are the structures that either collect source of supply continuously adequate in volume or convert an intermittently insufficient source into a continuously adequate supply.
Collection works may be in form of river diversion structures, dam reservoir and intakes, borehole or wells, spring capping, etc.
Conversion in form of *reservoir* is done to ensure adequacy due to seasonal variation or draught.

When the quality of the collected water is not satisfactory, **purification works** are introduced to render it suitable for the purposes;

- Purification may includes:
- ✓ disinfecting contaminated water ;
- ✓ making attractive and palatable esthetically displeasing water;
- ✓ deferrized and demanganized water containing iron and manganese;
- ✓ deactivating corrosive water and
- ✓ Softening hard water

etc.
Transmission works convey the collected water to the water treatment plant and/or purified water to the community,

This part of a system usually demands major investments. Transmission consists of various types of pipes, joints, fittings and connections, that operate together with control equipment.

According to the purpose they serve, pipes can be classified as follows:

Trunk main is a pipe for transport of potable, clean water from the water treatment plant to the distribution area. Depending on the area size, the pipe(s) can have diameters from a few hundred millimetres to a few metres (common range is from 250-1500 *mm*). Some branching of trunk mains is possible but consumer connections are rare.

Secondary main pipes provide the basic structure of the distribution system. They are used to link main distribution pipes with the service reservoirs or/and with the trunk distribution mains. A number of direct connections can be provided as well, especially for large consumers. Common diameters are 150-400 *mm*.

The distribution works distribute water to consumers in adequate volume and at adequate pressure.

Distribution mains carry water from the secondary mains to the smaller consumers. These are in particular pipes laid in the roads and streets of urban areas with the diameters in principle between 80-150 *mm*.

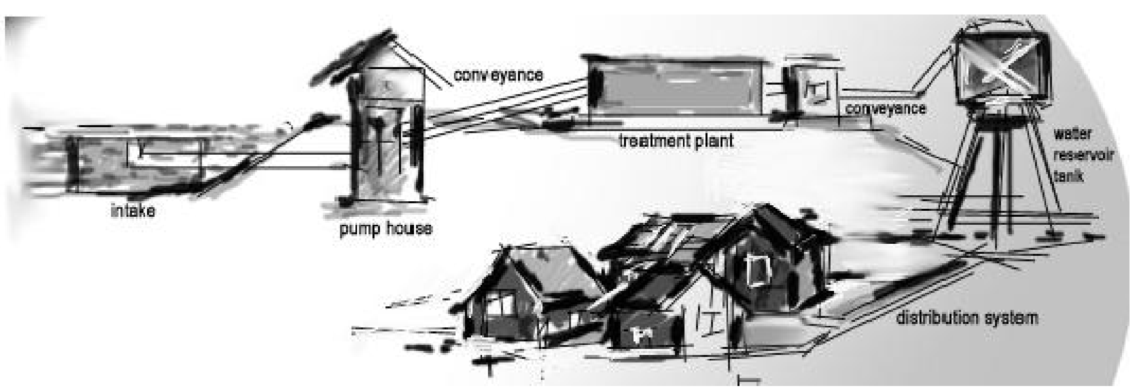
Service pipe directly brings water from the distribution main to either a public standpost, yard connection or to a dwelling. In case of domestic supplies, service pipes are generally less than 25 *mm* in diameter but other consumers may require larger size (up to 80 *mm*).

End of service pipe is the end point of the distribution system. From that point on, two options are possible:

Public connection: the service pipe terminates in one or more taps and water is consumed directly. This can be any type of public standpipe, tap or hydrant.

Private connection: the service pipe terminates at a stopcock of a private domestic installation within a dwelling. This is the point where responsibility of the water supply company usually stops. These can be different types of house or yard connections, as well as connections for non-domestic use.

The figure below shows the layout of different water supply system components;



Sketch showing the components of the water supply system

Collection works are done using intake structures:

Intake Structure

The basic function of the intake structure is to help in safely withdrawing water from the source over predetermined pool levels and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant.

Factors Governing Location of Intake

1. As far as possible, the site should be near the treatment plant so that the cost of conveying water to the city is less.
2. The intake must be located in the purer zone of the source to draw best quality water from the source, thereby reducing load on the treatment plant.
3. The intake must never be located at the downstream or in the vicinity of the point of disposal of wastewater.
4. The site should be such as to permit greater withdrawal of water, if required at a future date.
5. The intake must be located at a place from where it can draw water even during the driest period of the year.
6. The intake site should remain easily accessible during floods and should not get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

Design Considerations

1. Sufficient factor of safety against external forces such as heavy currents, floating materials, submerged bodies, ice pressure, etc.
2. Should have sufficient self weight so that it does not float by upthrust of water.

Types of Intake

Depending on the source of water, the intake works are classified as follows:

Pumping

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower to a higher level and delivers it at high pressure. Pumps are employed in water supply projects at various stages for following purposes:

1. To lift raw water from wells.
2. To deliver treated water to the consumer at desired pressure.
3. To supply pressured water for fire hydrants.
4. To boost up pressure in water mains.
5. To fill elevated overhead water tanks.
6. To back-wash filters.
7. To pump chemical solutions, needed for water treatment.

Types of distribution systems

With respect to the way the water is supplied into the network, the following distribution can be distinguished:

a) Gravity
b) Pumped
c) Combined

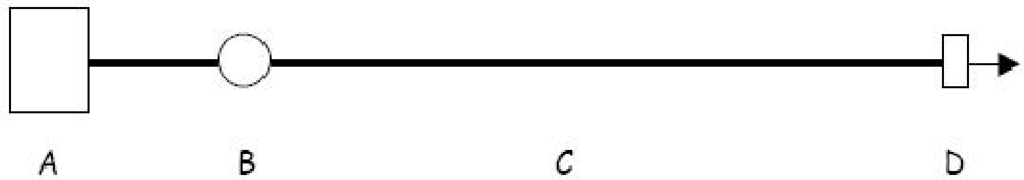
Choice for one of the above alternatives is very much linked to the existing topographic conditions.

Gravity systems make use of existing topography. The source is in this case located at higher elevations than the distribution area itself. The distribution of potable water can take place without pumping and nevertheless under acceptable pressure.

Pumped systems operate without or with limited water storage (water towers) in the distribution system. As the pumping regime has to follow variations in water demand, proper selection of units has to be done in order to optimize the energy consumption. Reserve pumping capacity for irregular situations should also be planned.

Combined distribution systems operate with reservoirs and pumping stations. A considerable storage volume is needed in this case: for balancing the demand variations, and as a buffer used in irregular situations. Such systems usually supply consumers from more than one point. They are common for large distribution areas.

Case of a gravity fed system from a spring



A. Catchment box to protect and to collect water from the spring.

B. Header tank which has several functions:

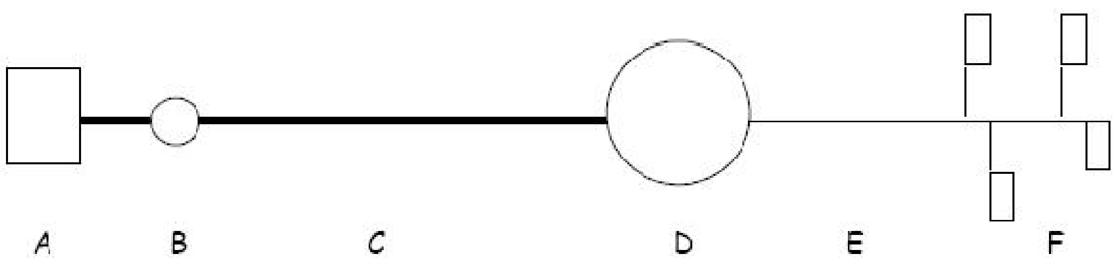
- Avoid an accidental increase of pressure “inside” the spring in case of network blockage. (Indeed, if a spring is put under pressure, there is a risk that it disappears).
- allows the decantation of the suspended substances if present (sand.…) before they enter in the pipe,
- Stabilize the flow coming from the spring.

C. Main pipeline which brings water from the spring to the users.

D. One or more water points, without tap, where the users can come to fetch water and where water run continuously.

This type of system is called “opened” because nothing closes the network and water runs out continuously. But what should be done, if the spring flow is not sufficient to cover the population water needs?

It is then necessary to close the system using taps installed on the tapstands and to store the unused water in a tank. The diagram of such a system, called “closed” system, is as follows:



A. Catchments box (Header tank)

B. Main pipeline

C. A storage tank to store water during periods when the populations demand is low in order to provide a more important flow when the demand increases.

D. Distribution line to supply water to the consumers.

E. Tapstands equipped with taps where the users can come to take water.

When the difference of height between the spring and the tank or the tap is too important, it is necessary to put small tanks, called break pressure tanks, between the two infrastructures in order to prevent the damage of the pipes because of the effect of high pressure.

The Study of a gravity fed system

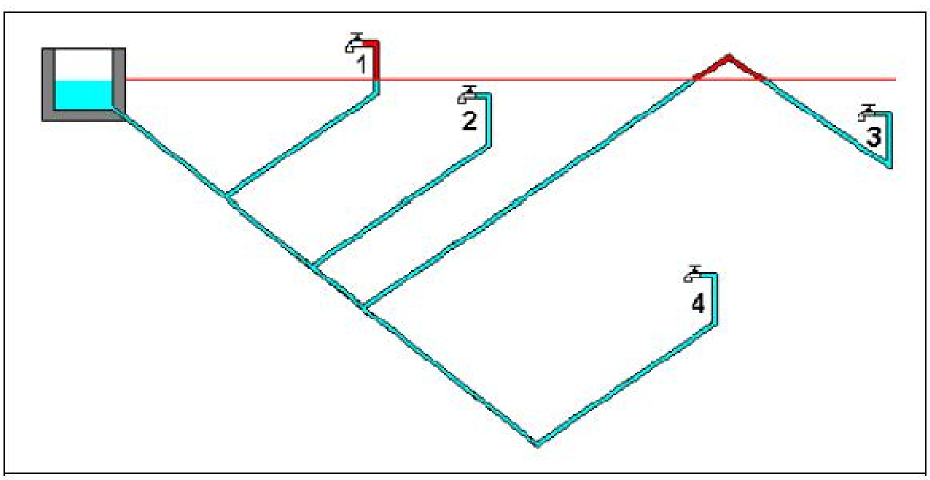
Gravity

A gravity fed system function thanks to the gravity. Gravity is a force which attracts all objects on the earth surface, due to the attraction exerted by the planet's mass. It is this force which makes that all bodies or things always fall at the lowest point (for example, a mango which falls from a tree).

It is thus by gravity that the water stored in tank goes down by its own weight inside the pipes and run out from the taps. But this system works only if the pipes and taps are at a lower level than the water level at the starting point.

To illustrate this fact, let's consider the example of the figure below:

Tap 1: water does not run out because the tap is on a higher level than the water level in the tank; **Tap 2**: water runs out from the tap but with low pressure (i.e. low power) because the tap is close to water level in tank; **Tap 3:** water does not run out because part of the pipes is found in the top of water level in the tank; **Tap 4**: water runs out from the tap with a good pressure.



Pressure

The water pressure is the force which water exerts in the walls of the container it is contained (pipe's walls, reservoir's wall…).

The pressure in a considered point correspond (or is equivalent) to the weight of water column above this point. Knowing that the density of water is 1 g/cm^3, we can easily calculate the water column weight above a given point:

Water column weight = water density x water column height

= 1 g/cm^3 x water column height (cm)

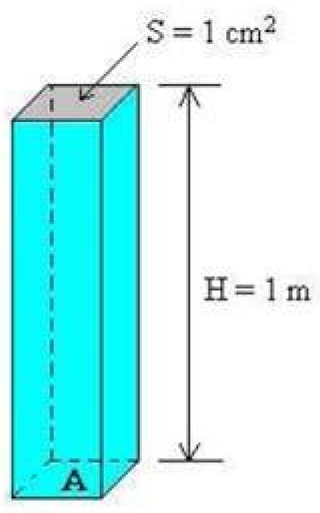
= pressure at the considered point (g/cm^2)

So, we obtain:

Pressure (g/cm^2) = 1 g/cm^3 x water column height (cm)

= water column height (cm)

The pressure which is exerted by water on the bottom of a water column depends only on the height of water column.



The pressure at point A = 1kg for S=1cm^2

Let's take the following example:

The pressure units are the kg/cm^3, the bar or the "metres water gauge":

1 kg/cm^2 = 1 bar = 1 mWG

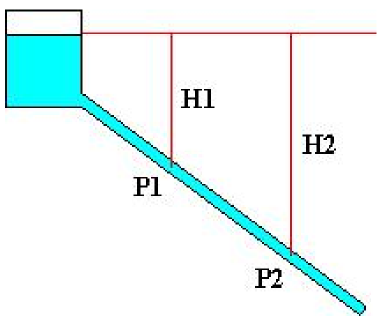
For the hydraulic calculations used for the sizing of a gravity fed system, we always measure the pressure in mWG.

We have to differentiate the static pressure from the dynamic pressure:

- The static pressure is the force exerted by water on the pipes walls when all taps are turned off (water does not circulate in the pipes), for
- The dynamic pressure is the force exerted by water on the pipe walls when 1 or several taps are open (water circulates in the pipeline).

Static pressure

The static pressure corresponds to the water column weight between the highest point of the pipe and the considered point and is thus equal to the difference of height between the highest point of the pipe and the point considered. The highest point of the pipe corresponds to the water's free surface in one of the various infrastructures of the gravity fed system (the spring catchment, the header tank, the break pressure tank or the reservoir).

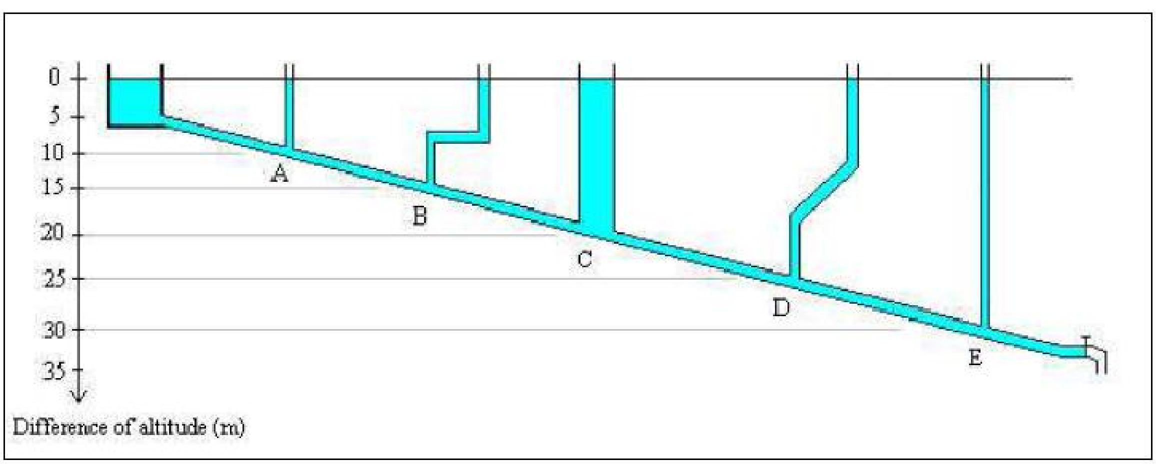


Pstatic (mWG) = H (m)

The pressure exerted by water in the pipe at the point P1 = the height H1 (in meters).

The pressure exerted by water in the pipe at the point P2 = the height H2 (in meters).

If we take the example of the following figure: let us determine the pressure exerted by water in the pipe at the points A, B, C, D and E



- Point A: Pstatic = 10 meters.
- Point B: Pstatic = 15 meters.
- Point C: Pstatic = 20 meters.
- Point D: Pstatic = 25 meters.
- Point E: Pstatic = 30 meters.

For simple projects, the static pressure is the maximum pressure which can exist in the pipes. It allows determining the pressure to which the pipe must resist, as well as the need to install pressure breaking devices to protect the pipe.

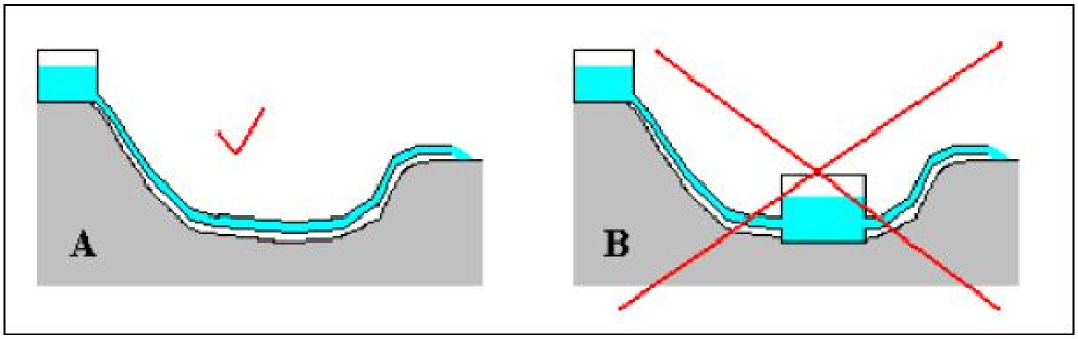
The pipes used for gravity fed systems are resistant to a certain pressure, call Nominal Pressure (NP): if the pressure in the pipe is higher than this NP, there is a risk of rupture.

The range of pipes nominal pressure generally used for the gravity fed system are given in table below:

Pipe type	Nominal Pressure	Maximum Pressure (P_{static})
Plastic pipe (PVC or PE)	NP 6	60 meters
	NP 10	100 meters
	NP 16	160 meters
Galvanized Iron (GI)	NP 16	160 meters
	NP 25	250 meters

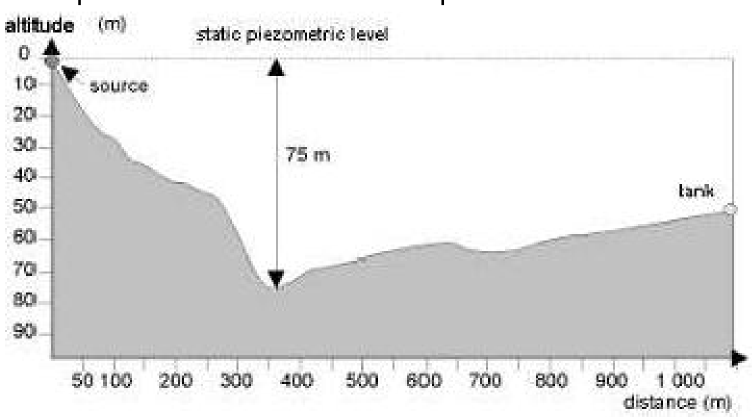
If the pressure imposed by topography is too important for the available pipes' nominal pressure, it is possible to build a break pressure tank which brings back the pressure in the network to the atmospheric pressure. Indeed, each time we have a free surface of water in contact with the atmosphere, the static pressure becomes zero, because it is in equilibrium with the atmospheric pressure.

In a network, the free surfaces are the reservoir, the break pressure tanks, the header tanks and the spring catchments unit. It is thus not possible to have a network where the tank is placed as illustrated on the figure below (case B).



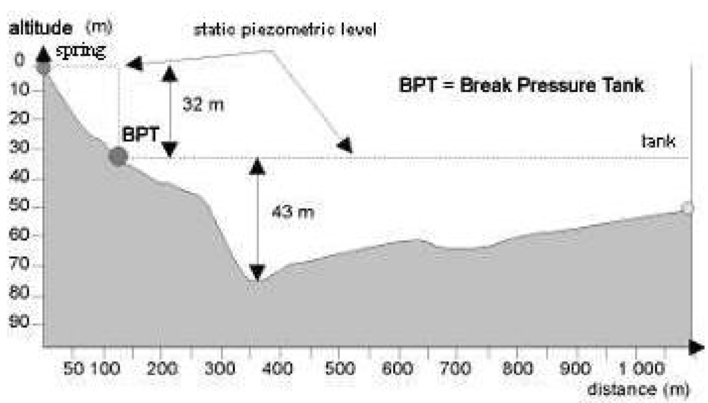
The static pressure becomes null in a tank: the scenario B is not possible, the water will
Not run after the second tank.

In the example of the following figure, the pipes nominal pressure should be NP10. If the available level is only NP6 (i.e. maximum difference of height of 60 meters), it is possible to install a break pressure tank to control the pressure.



Example of topographic survey with maximum height H =75m.

The figure below shows the case in which a break pressure tank is built which allows obtaining maximum static pressures of 32 mWG upstream and 43 mWG downstream.



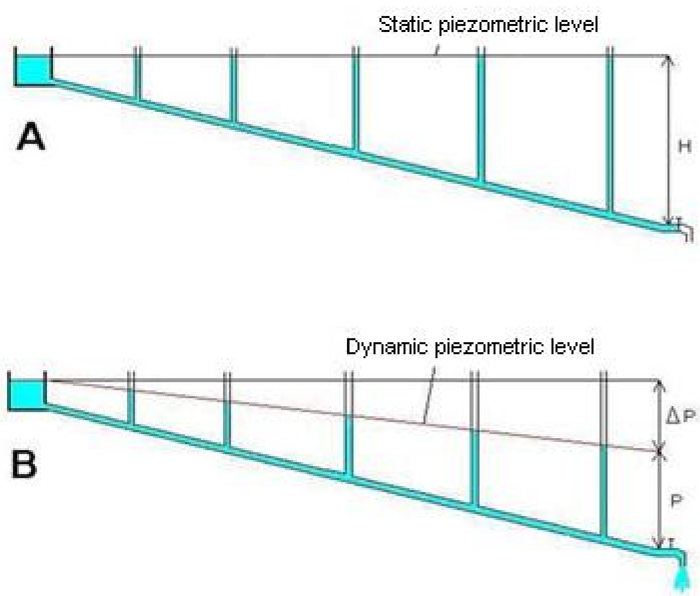
Installation of a break pressure tank to respect the pipe nominal pressure (NP6).

Dynamic pressure

The dynamic pressure is the force which water exerts in pipes when water flows in the pipes, i.e. when the taps are open, and that pipes are full of water. The dynamic pressure is lower than the static pressure because of the fact that when water circulates in pipes, it loses energy. Indeed, pressure losses due to the frictions of water against the pipe's walls can be observed when water circulates in the pipe. These losses of pressure are called **"head losses".**

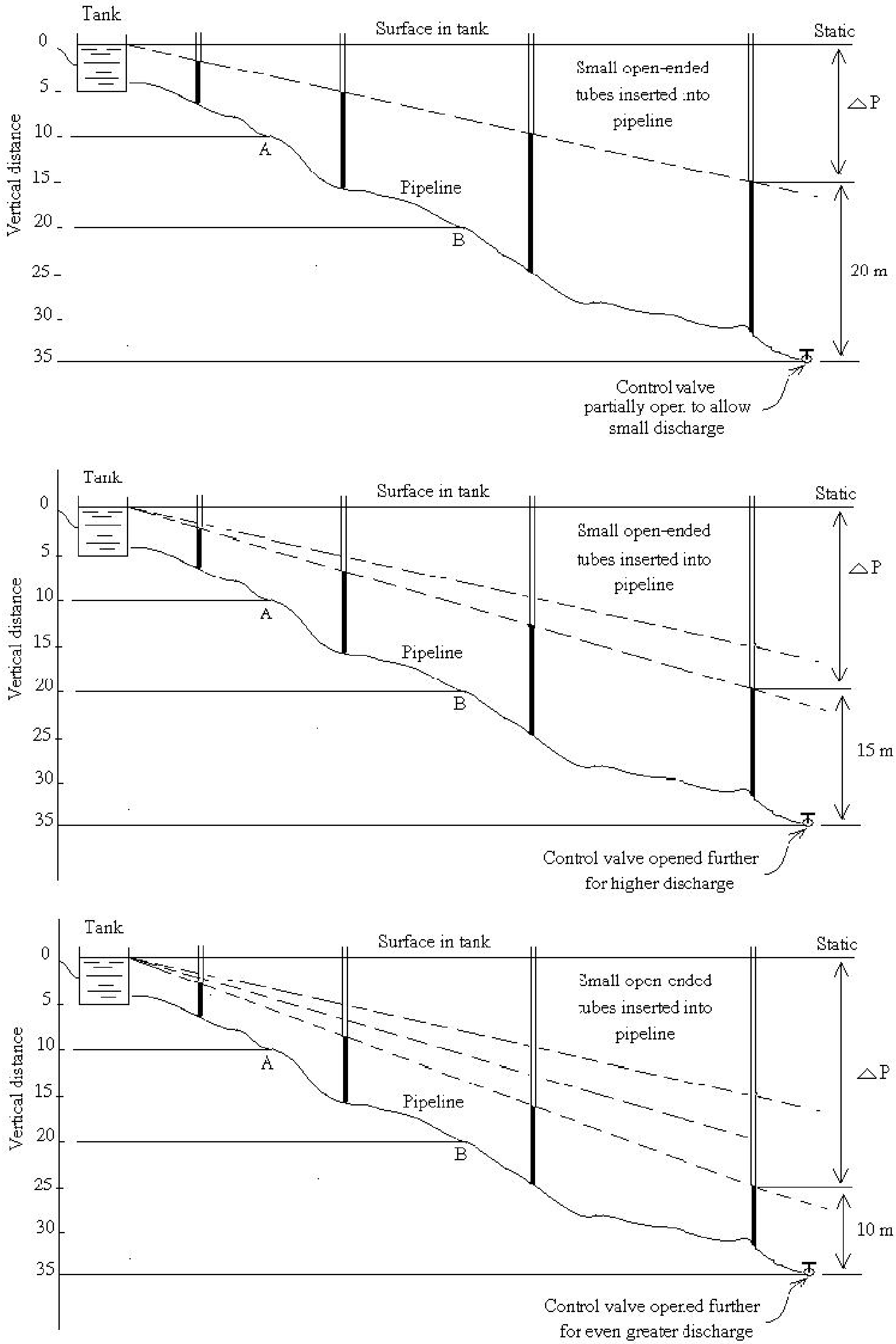
This phenomenon is illustrated on the following figure. The **piezometric line** allows visualizing the evolution of the water pressure all along the pipelines. It corresponds to the level that water would reach in a vertical pipe connected to the pipeline. If we draw the pressure line during the flow, we obtain the dynamic head profile (dynamic piezometric level). Part of the energy of the water is used by the head losses (ΔP) during water transportation. The residual pressure is defined by:

Presidual (mWG) = H (m) - ΔP (m)



Static level (illustrated in A where the taps are closed) and dynamic level (illustrated in B where taps are opened) of a pipeline.

The figure below shows the variation of dynamic piezometric level according to the flow of water in pipes: the more opened is the tap → the larger is the quantity of water circulating in pipe → the more water loses energy → the larger are the head losses → the smaller is the residual pressure.



Variation of the head losses according to the flow of water in pipes.

Calculation of the head losses (∆P)

To facilitate the head losses calculation, we must make the difference between the head losses created by the pipes (= "linear head losses") and those created by the fittings such as elbows, T junction, valves etc. (="secondary head losses")

Linear head losses

The head losses in a pipe, called linear head losses, depend on various factors:

- D_p (m): pipe diameter. For a given flow, the smallest is the pipe diameter, the more important are the head losses.
- Q (m^3/s): the flow of water flowing in the pipe. For a given diameter, the higher is the flow, the more important are the head losses.
- L (m): pipe length. The longer is the pipe, the more pressure is lost through head losses.
- Pipe roughness. The higher pipe roughness is, the more important are the head losses.

The roughness of pipes depends on their quality (materials, manufacture) and age.

From hydraulics Engineering we know some formulas which can be used to calculate the head losses such as the Hazen-Williams, Darcy-Weisbach and Manning.

Secondary head losses

The secondary head losses are losses of pressure occurring when water is passing through the pipelines fittings (elbow, T junction, valve, reducers…). These head losses depend on the shape of the fitting and the flow of water which circulates.

Generally, in the simple networks, the secondary head losses are very small as compared to the linear head losses and thus can be neglected.

Infrastructure positioning and Sizing

1. **Header tank**

The header tank should be located downstream of the spring catchment. The exact location depends on the ground topography around the spring. The static and dynamic piezometric lines of the network generally start from this point. Indeed, by safety measure (in order to make sure that the spring is not put under pressure), the part of the system connecting the spring to the header tank should not be pressurized. For this part, a larger pipe diameter and a great difference of height between the spring and the header tank should be chosen.

Sizing of the header tank

One of the roles of the header tank is to facilitate the decantation of the suspended substances present in water, such as sand. For that, it is necessary that water can remain long enough in the header tank in order to let time for the particles to settle down at the bottom of the tank.

For spring catchment, collected water is generally clear and contains only very small quantity of suspended substances: It is thus only necessary to impose a water retention time sufficient for the sand to settle down.

The effective header tank volume is given by the following formula: V = Q x t

V = tank volume (liters)

Q = water flow (liters/second)

t = water retention time in the header tank (second)

The recommended water retention's time for the **spring catchment** equipped with tank is from **15 to 20 minutes (or 900 to 1,200 seconds**). For the open-type networks, i.e. without tank, the minimum recommended retention time is 60 minutes (or 3,600 seconds).

Example: What is the effective volume of a header tank for a close-type gravity fed system, if the flow of the spring = 580 l/h?

Solution: Q = 580 l/h = 580/3,600 = 0.16 l/s.

V = 0.16 x 1,200 = 192 liters.

Once you have the volume of the required tank, then you can make a proper choice of the size of the tank and its shape (rectangular, circular,..).

2. Storage Tank

The positioning of the storage tank depends on the location and number of downstream tapstand to be supplied by gravity. As a first estimation, we can consider that the head losses in the distribution network (downstream of the tank) are about 1 meter for 100 meters of horizontal distance. We draw a straight line of 1% slope starting from the highest tapstand (by taking account of a residual pressure of 10 mWG at the tapstand). All the points located below this line are not suitable for the positioning of the tank. The selected site must then depend on a head losses calculation.

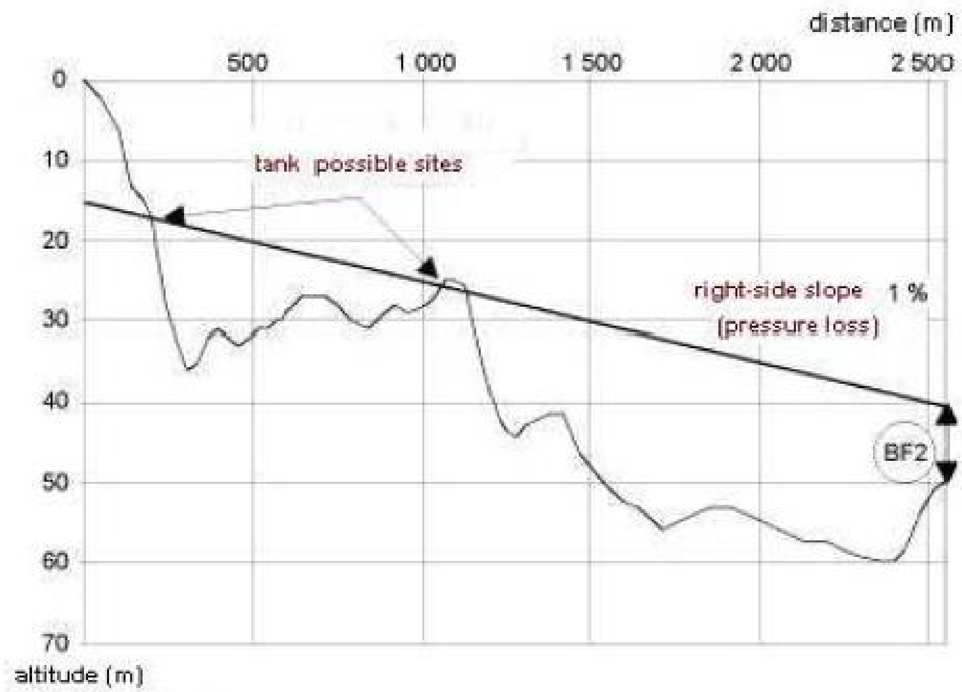
Generally, we should choose to position the tank as near as possible from the village for several reasons:

- Accessibility (in order to facilitate construction and maintenance),

- The overflow of water can be used by the villagers for others uses (agriculture, animals…),
- To limit the length of the distribution network (i.e., the part of the network placed after the tank). Indeed, the distribution network generally requires pipes of larger diameter because it transports larger flows of water.

An example is given on the following figure where two sites are possible to build the storage tank.

The site located upstream is in an apparently steep zone, and the downstream one is located on a relief approximately half way between catchments and the tapstand no. 2. By positioning the tank on the downstream site, it will also play the role of break pressure tank to limit the maximum static pressure in the network to 35 mWG. The second option should then be chosen.



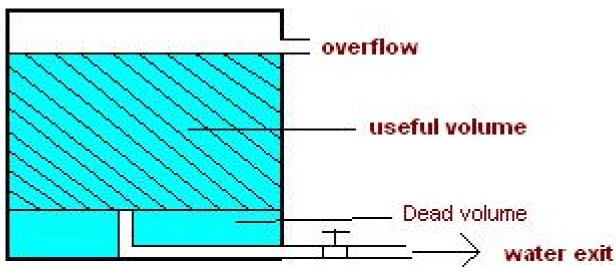
Possible choices for tank location

3. Break pressure tank

The need to install a break pressure tank in the network depends on the study of the static profile (as considered previously). During the topographic survey, the favorable sites for a break pressure tank should be noted. Accessibility problems (transportation of construction material and equipments, maintenance…) must be considered.

Sizing of the storage tank

In order to size the storage tank, it is necessary to calculate the useful volume required to meet the total water demands of the target population. This useful volume corresponds to the effective water volume in the tank, which means the volume below the overflow pipe and above the outlet pipe as shown by the following figure.



For the spring catchments presenting a low flow, it is possible to make a first estimation of the useful volume: the tank volume must correspond to the volume of water produced by the spring in night period, i.e. over 10 hours period.

If we have, for example, a spring flow of 0.44 m^3 / h, a first approximation of the tank volume is 0.44 X 10 = 4.4 m^3.

To make an optimal tank sizing, it is necessary to calculate the population water requirements by sections of time using a consumption rate for each time section.

Table below shows an example of a standard hourly water demand calculation for a certain rural village. However, for each country, the consumption rate has to be determined.

The evolution of the water demand in hourly period for a village (the total daily needs are 10.56m^3/day)

The water consumption characteristics for the population in the village.

HOUR	Consumption m^3 /s	HOUR	Consumption m^3 /s
1	0	13	0.71
2	0	14	0.59
3	0	15	0.83
4	0	16	1.18
5	0	17	1.18
6	0.59	18	1.06
7	1.12	19	1.3
8	0.59	20	0.47
9	0.35	21	0
10	0.47	22	0
11	0.77	23	0
12	0.59	24	0

The method to calculate the necessary useful tank volume is as follows (see the example given on the following table):

1. Calculate the accumulated spring production per hour: (if you are given the daily production, then the hourly production can be obtained by dividing the daily flow of the spring by 24 (column 4)
2. Calculate the accumulated water needs according to the time (column 5).
3. Calculate the difference between the accumulated spring flow per hour and the accumulated hourly water demand (column 6 = column 4 – column 5). When the difference is negative (sign -), the water provided by the spring is insufficient.
4. Check whether you are emptying or filling your tank at every hour (column 6).
5. Calculate the water stock: can be obtained by cumulating the differences calculated previously (column 7).
6. Calculate the necessary quantity of water stored in the tank: can be obtained by adding the reserve of water for each period of time and the highest water stock deficit (column 9 = column 8 + 1.44 m^3). The highest water stock deficit (for this example it is 1.44m^3) this represents the reserve volume of water that should be inside the tank in order to have always a positive stock. Indeed, at 24h, we notice a deficit of volume of -1.44m^3: so we should take this value as the minimal reserve of water. The calculation of this parameter is not useful to size the tank, but it allows visualizing the water volume variation in tank during the day.

7. Calculate the minimal useful tank volume, i.e. minimum volume required to meet the population needs per time: Vmin = maximum water stock – minimum water stock = 16.49 - (- 1.44) = 16.49 + 1.44 = 17.93 m^3. Or in the other terms take the maximum stock + the maximum deficit or (Minimum tank capacity = |max +ve|+ |max -ve|).

But if the tank has this volume, there may be some overflow, so some water will be lost.

We should hence calculate the "recommended tank volume", for which there will be no overflow.

8. Calculate the daily Overflow Volume OF. It can be obtained by withdrawing the total daily needs from the total spring flow if the later is greater than the former. Practically add between 15 and 20 cm to the height of the tank for the over flow volume.
9. Calculate the dead volume. Take 5% of the minimum volume. But practically add between 10 and 15 cm to the height of the tank.
10. Calculate the recommended useful tank volume, i.e. the volume for which the tank will use all the water produced by the spring without producing overflow: for the minimum volume of 17.93 m^3 let us take a circular tank of height h = 3m and radius r = 1.38 m and add 10 cm for dead volume and 15 cm for overflow. So the required volume for our tank becomes V = 19.5 m^3

We can also estimate the volume required for the tank using graphical method, with time in hours on the X axis and Consumption and supply on Y axis. The surface under the supply line will be representing the volume of storage whereas the surface above the supply line is representing the deficit volume.

The volume is then the sum of the maximum storage and the maximum water deficit.

1	2	3	4	5	6	7	8	9
			Cumulated					
Hr	Sup.	Cons.	Sup.	Cons.	S-C	F or E	W Stock	W Stored
1	0.44	0	0.44	0	0.44	F	0.44	1.88
2	0.44	0	0.88	0	0.88	F	1.32	2.76
3	0.44	0	1.32	0	1.32	F	2.64	4.08
4	0.44	0	1.76	0	1.76	F	4.4	5.84
5	0.44	0	2.2	0	2.2	F	6.6	8.04
6	0.44	0.59	2.64	0.59	2.05	E	8.65	10.09
7	0.44	1.12	3.08	1.71	1.37	E	10.02	11.46
8	0.44	0.59	3.52	2.3	1.22	E	11.24	12.68
9	0.44	0.35	3.96	2.65	1.31	F	12.55	13.99
10	0.44	0.47	4.4	3.12	1.28	E	13.83	15.27
11	0.44	0.77	4.84	3.89	0.95	E	14.78	16.22
12	0.44	0.59	5.28	4.48	0.8	E	15.58	17.02
13	0.44	0.71	5.72	5.19	0.53	E	16.11	17.55
14	0.44	0.59	6.16	5.78	0.38	E	16.49	17.93
15	0.44	0.83	6.6	6.61	-0.01	E	16.48	17.92
16	0.44	1.18	7.04	7.79	-0.75	E	15.73	17.17
17	0.44	1.18	7.48	8.97	-1.49	E	14.24	15.68
18	0.44	1.06	7.92	10.03	-2.11	E	12.13	13.57
19	0.44	1.3	8.36	11.33	-2.97	E	9.16	10.6
20	0.44	0.47	8.8	11.8	-3	E	6.16	7.6
21	0.44	0	9.24	11.8	-2.56	F	3.6	5.04
22	0.44	0	9.68	11.8	-2.12	F	1.48	2.92
23	0.44	0	10.12	11.8	-1.68	F	-0.2	1.24
24	0.44	0	10.56	11.8	-1.24	F	-1.44	0.00

Table: Calculation of the hourly water demand and the volume of water inside the reservoir (the flow of the spring is 10.56 m^3/ and the daily needs is 11.8 m^3/day).

4. Choosing and sizing the pipes

Plastic and metal pipe

In general, the use of galvanized-iron pipes (GI) pipe for rural GFS is not recommended: they are very expensive, very heavy, very difficult to maintain, and can get corroded very easily: plastic pipe (PVC or PE, PPR) are preferred. However, for certain parts of the network (above the- ground part, crossing rivers or road, and when pipe are cast in concrete), it is necessary to use GI pipe, more resistant to the shocks and to the sun.

PVC and PE pipe

The recommended pipes for GFS construction in rural are the polyethylene (PE) plastic pipes. PE pipe are flexible, present less joints (thus less risk of leaks), and they come in rolls of 50 or 100m, so they are very easy to install, whereas PVC pipes have to be glued together, which can be a tedious task if the network is long. However, it is usually very difficult to find PE pipes and fitting in the shops in rural areas, they often have to be ordered from large towns. On the opposite, PVC pipes and fittings are available almost everywhere, and are usually quite cheap, so the community can maintain their GFS easily after the handover of the project.

Pressure pipes

Among the plastic pipes (PE or PVC), we can distinguish 2 types: pressure pipes, and no pressure pipes. Only pressure pipes (pipes that can withstand the pressure) should be used for GFS, and water supply in general. Non-pressure pipes are used mainly for sanitation systems, which are usually not under pressure.

There are usually different types of pressure pipes that can withstand different Nominal Pressure (NP). The most adapted pipe's nominal pressure for the GFS construction is NP10 (that can resist to a water pressure of 10 bar = 100m). Higher NP (12, or 16) should be avoided because of their high cost and because of the difficulty to find the fittings.

For example, the Wavin PVC inch standard, there are several types of pressure pipes:

AW class, that can withstand a pressure of 10kg/cm2 = 10 bars = 100m

D class, that can withstand a pressure of 5kg/cm2 = 5 bars = 50m; Please refer to the Wavin brochure for more details

→ **Correspondence in pipes diameters**

Nominal diameters (DN) of galvanized-iron (GI) pipes correspond to the internal diameters, whereas the reference diameters of plastic pipes (PVC and PE) correspond to the external diameters.

It is always the internal diameter that should be considered for the head loss calculation.

Pipes diameters are given in mm or in inch. **One inch is theoretically equal to 25mm**.

Theoretical correspondences between pipes with these two units are given in table below.

PVC pipes / PE		GI pipe	
DN in mm (ext. diameter)	Equivalent diameter in inches	DN in inches (int. diameter)	Equivalent diameter in mm (int/ext)
20	¾"	½"	15 / 21
25	1"	¾"	20 / 27
32	1" ¼	1"	26 / 34
40	1" ½	1" ¼	33 / 42
50	2"	1" ½	40 / 49
63	2" ½	2"	50 / 60
75	3"	2" ½	66 / 76
90	3" ½	3"	80 / 90
110	4" ½	4"	102 / 114

In reality, this correspondence is very rarely respected, and one must refer to the catalogue of the pipe suppliers available locally.

An important thing to consider is that Wavin has two different standards of PVC pipes:

Pipes with diameter measured in mm (metric standard, also called European standard), used mainly by the government; Pipe with diameter measured in inch (Japanese standard), used by private sector, usually available in the shops.

The two standards are unfortunately not compatible. ACF advices to use the PVC "inch standard" pipe for which the fittings can be easily purchased by the villagers.

The two following tables present various examples of Wavin pipes:

The following table shows outside and internal diameter of Wavin polyethylene pipes (PE)

PE pipe PN = 16 bar			PE pipe PN = 10 bar		
Pipe Ext. Diameter (mm)	Wall thickness (mm)	Corresponding internal diameter (mm)	Pipe Ext. Diameter (mm)	Wall thickness (mm)	Corresponding internal diameter (mm)
32	3	26	32	2.3	27.4
40	3.7	32.6	40	2.4	35.2
50	4.6	40.8	50	3	44
63	5.8	51.4	63	3.8	55.4

External and internal diameters of Wavin PVC pipes inch standard (type of pipe generally used by ACF in rural areas)

PVC pipe Wavin AW class (NP=10 kg/cm2 = 10 bar), inch standard			
diameter (inch)	Corresponding external diameter (mm)	Wall thickness (mm)	Internal diameter (mm)
½	22	1.5	19
¾	26	1.8	22.4
1	32	2	28
1 ¼	42	2.3	37.4
1 ½	48	2.3	43.4
2	60	2.3	55.4
2 ½	76	2.6	70.8
3	89	3.1	82.8

Warning:

Plastic pipes are sensitive to heat. In area where farmers practice slash and burn agriculture (which consists in leaving the field not cultivated for a few years, and then burning the field before cultivation to make it more fertile), there are a risk that if the pipe is buried in a field that is burned, it will melt. Attention should be paid either to avoid laying the pipe in a field that is likely to be burned in the future, Identify carefully the location of the pipes (with concrete marking blocks, for example) so the farmers will pay attention not to burn this area (fire should be at least 40m away from the pipe).

Choice of pipes diameters

The choice of pipes' diameters is determined by the study of the dynamic head profile and by the head losses calculation.

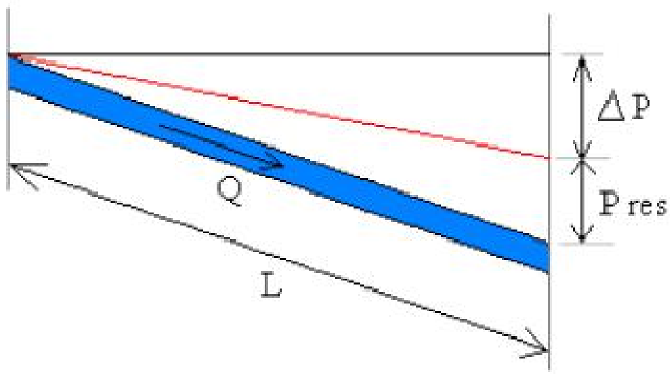
The sizing of the network must **start with the main line,** i.e. the part of the network located between the spring catchment and the storage tank. The main line is sized to allow to the maximum flow of the spring to pass through the pipes. Indeed, it is important to collect and transport the totality of the spring flow to the storage tank, even if the present population needs are lower: that makes possible future network extensions. A compromise must be found when the spring flow is much higher than the populations needs, or fluctuates in an important way during the year.

The distribution network (i.e. the network part between the tank and the tapstand) is sized.

One can start by sizing the main network line, then the secondary lines. We proceed then by successive tries until the selected diameter allows obtaining the required residual pressure.

From a financial point of view, it is always interesting to use small diameter of pipes, because they are cheaper. Consequently, it may happen that two different diameters on the same line are used to optimize the network construction cost.

The step to be followed is as follows: for each network section we know, L, H and Q:



L = pipe length in meters

H = difference of height in meters (= ΔP + Pres)

Q = flow in m^3 /seconds

For the calculation of head losses it is preferable to use the Hazen-Williams Equation.

ΔE is the energy losses caused by: friction between water and the pipe wall

Turbulence developed by obstructions of the flow.

The losses are called friction losses and minor losses, respectively. Both can be expressed in the same format:

$$\Delta E = h_f + h_m = R_f Q^{n_f} + R_m Q^{n_m}$$

where R_f is resistance of a pipe with diameter D, along its length L. R_m can be considered as resistance at the pipe cross-section where obstruction occurs. Exponents n_f and n_m depend on the type of formula applied.

Darcy-Weisbach

$$R_f = \frac{8\lambda L}{\pi^2 g D^5} = \frac{\lambda L}{12.1 D^5} \quad ; \quad n_f = 2$$

Manning

$$R_f = \frac{10.29 N^2 L}{D^{16/3}} \quad ; \quad n_f = 2$$

Hazen-Williams

$$R_f = \frac{10.68 L}{C_{hw}^{1.852} D^{4.87}} \quad ; \quad n_f = 1.852$$

The friction losses $h_f = R_f \times Q^{nf}$ and it is expressed in mWG.

λ, C_{hw} & N are experimentally determined factors which describe the impact of the pipe wall roughness on the friction loss.

The *friction factor,* λ (-) (in some literature *f*) can be calculated from the formula of Colebrook-White:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{2.51}{Re\sqrt{\lambda}} + \frac{k}{3.7D}\right]$$

where:

k	Absolute roughness of the pipe wall (*mm*).
D	Inner diameter of the pipe (*mm*).
Re	Reynolds number (-).

The Hazen-Williams' formula is an empirical formula widely used in practice. It is especially applicable for smooth pipes of medium and large diameters, and pipes that are not attacked by corrosion. Values of C_w for selected materials and diameters are shown in the T_V following table. They are experimentally determined for flow velocity of 9m/s. So, if considered velocity is much different, the C_w value has to be corrected according to table T_C.

Hazen-Williams Factor, C_{hw}

Material	Diameter [mm] 75	150	300	600	1200
Uncoated cast iron	121	125	130	132	134
Coated cast iron	129	133	138	140	141
Uncoated steel	142	145	147	150	150
Coated steel	137	142	145	148	148
Wrought iron	137	143			
Galvanized iron	129	133			
Uncoated AC	142	145	147	150	
Coated AC	147	149	150	152	
Concrete, minimum	69	79	84	90	95
Concrete, maximum	129	133	138	140	141
Prestressed concrete			147	150	150
PVC,brass,lead,copper	147	149	150	152	153
Wavy PVC	142	145	147	150	150
Bitumen/Cement lined	147	149	150	152	153

Table T_V

Correction of C_{hw}

C_{hw}	v < 0.9 m/s Per halving	v > 0.9 m/s Per doubling
<100	+5%	-5%
100-130	+3%	-3%
130-140	+1%	-1%
>140	-1%	+1%

Table T_C

Nota. For simple calculations we will use **C_w = 140** for **GI pipes** and **C_w = 150** for **PVC pipes** and **C_w = 135** for **concrete pipes.**

The **velocity is assumed to be 1m/s** and then corrected after the determination of the diameter. However the **velocity** in the main line should lay between **0.75m/s and 1.5m/s** since low velocities allow suspended materials to settle down and may block the flow and high velocities lead to hydraulic problems. The continuity equation is used to determine the pipe diameter:

$$Q = v\frac{D^2\pi}{4}$$

In the case of high discharge, we can use a number **n** of pipes to carry the required discharge to a certain distance. In that case the continuity equation becomes:

$$Q_{des} = v_p \frac{n\pi\, D_p^{\,2}}{4} \rightarrow \quad D_p = \sqrt{\frac{4\,Q_{des}}{n\,v_p\,\pi}}$$

With: $\boldsymbol{Q_{des}}$ the design discharge (m^3/s), $\boldsymbol{D_p}$ the pipe diameter (m) and $\boldsymbol{V_p}$ the velocity within the pipe (m/s).

- Assume the V_p = 1m/s
- Determine D_p
- Check for $V_p = \dfrac{4\,Q_{des}}{n\,\pi\,D_p^{\,2}}$ if it does not lie within the range, then, change $\boldsymbol{D_p}$**.**
- once an acceptable pipe diameter haven been determined, we will check for the head losses, using the Hazen-William formula, $H_f = 10.68 \left(\dfrac{Q}{150}\right)^{1.852} \left(\dfrac{L}{D_p^{\,4.87}}\right)$
- If the residual pressure is positive, water will go. If not, no flow of water is expected. Retake the design operation.
- For the number of pipes, take **n + 1. 1** is for spare

In the distribution pipe lines, some conditions are necessary to obtain a good water flow. These conditions are as follows:

- The water velocity in pipes must respect a certain interval,
- The residual pressure must always be positive.

It is important to take into account the velocity of the water in pipes. High velocities generate excessive friction and lead to hydraulic problems. On the contrary, low velocities let solid particles contained in the water to sediment in the network low points, and eventually block the flow.

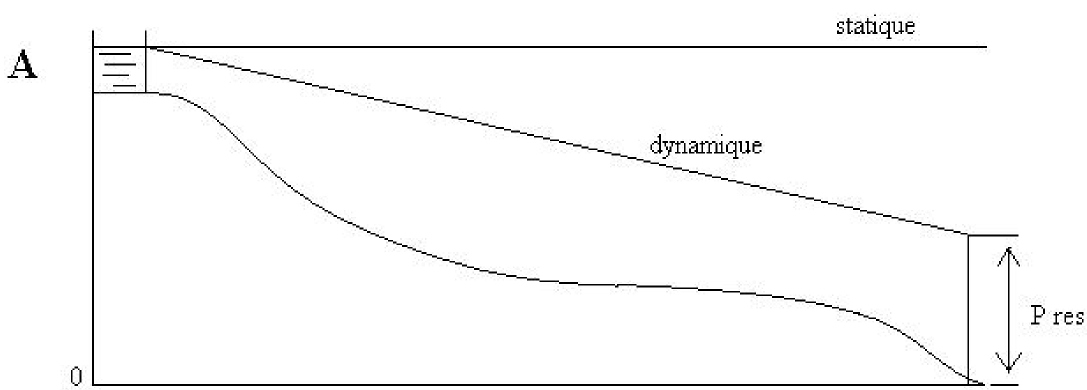
The recommended limits for water velocity in pipes depend on the pipe diameter. Generally, the following limits are recommended for the plastic pipes:

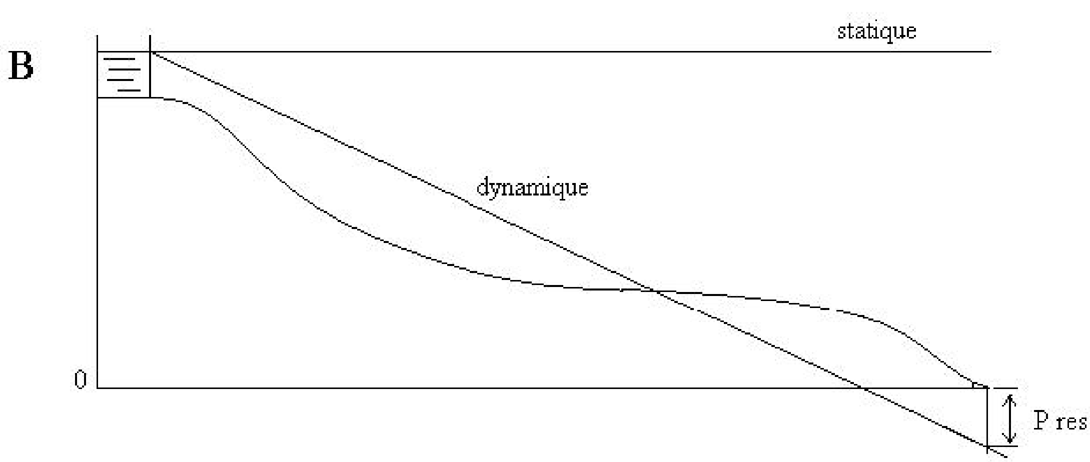
Diameter (mm)	20 to 40	50 / 63	75 / 90
Maximum velocity:	2 m/s	4 m/s	10 m/s
Minimum velocity:	0.3 m/s	1 m/s	3 m/s

For the water to flow properly in a pipe, the residual pressure should always be positive. If we take the example of the following figure:

Case A: The residual pressure is positive. That means that water has enough energy to flow in the pipes and the desired flow reaches the tank or the tapstand.

Case B: The residual pressure is negative. That means that water does not have enough energy to flow properly in the pipes. The water velocity in network will be lower than required and the volume of water delivered will be lower than planned. For the water to flow properly in the network, it is necessary to increase the diameter of the pipes.





To avoid having a negative residual pressure on a part of a network (case A), it is necessary to change the pipes diameter, even if it means to use different pipe diameter along the same line (case B).

Pumping

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower to a higher level and delivers it at high pressure. Pumps are employed in water supply projects at various stages for following purposes:

1. To lift raw water from wells.
2. To deliver treated water to the consumer at desired pressure.
3. To supply pressured water for fire hydrants.
4. To boost up pressure in water mains.
5. To fill elevated overhead water tanks.
6. To back-wash filters.
7. To pump chemical solutions, needed for water treatment.

Classification of Pumps

Based on principle of operation, pumps may be classified as follows:

1. Displacement pumps (reciprocating, rotary)
2. Velocity pumps (centrifugal, turbine and jet pumps)
3. Buoyancy pumps (air lift pumps)
4. Impulse pumps (hydraulic rams)

By installing pumps, a certain amount of energy can be added to the pipe flow. This energy, generated by the pump impeller, is usually expressed as a head of water column (in mwc). The pumping head (pump lift) h_P represents the difference between energy levels at the pump exit i.e. the discharge pipe.

The theoretical power required N (w) for the pump to lift water is $N = \frac{\gamma\, Q\, H}{75}$

where, γ= specific weight of water kg/m^3, Q= discharge of pump, m^3/s; and H= total head against which pump has to work.

H= H_s + H_d + H_f + (losses due to exit, entrance, bends, valves, and so on) where:

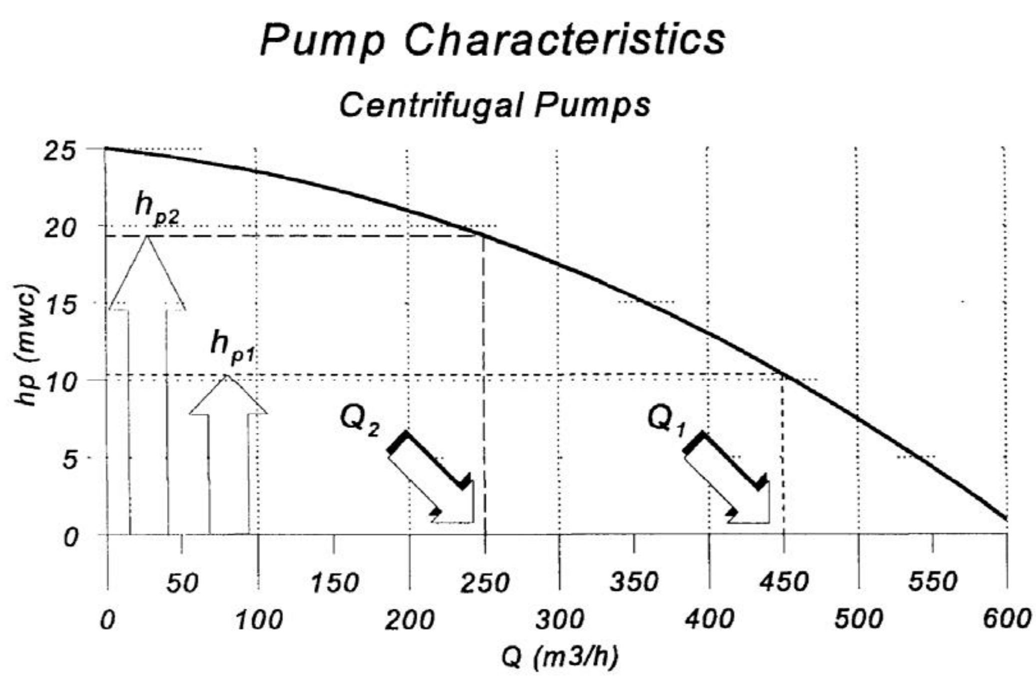
H_s=suction head, H_d = delivery head, and H_f = friction loss.

However, the power to drive the pump will be somewhat higher, due to energy losses in the pump: then, $N = \dfrac{\gamma\, Q\, H}{75\eta_p}$ where η_p is the motor efficiency.

For simple calculations we take η_p = 70%.

Hydraulic performance of pumps is described by the **pump characteristics**. This curve shows relation between the pump discharge and delivered head (figure below). For centrifugal pumps, a very good approximation of pump curve is achieved by the following equation:

$h_p = aQ^2 + bQ + c$ Where factors a, b & c depend on the pump model and flow units.



The pump characteristics are supplied by the manufacturer for each model.

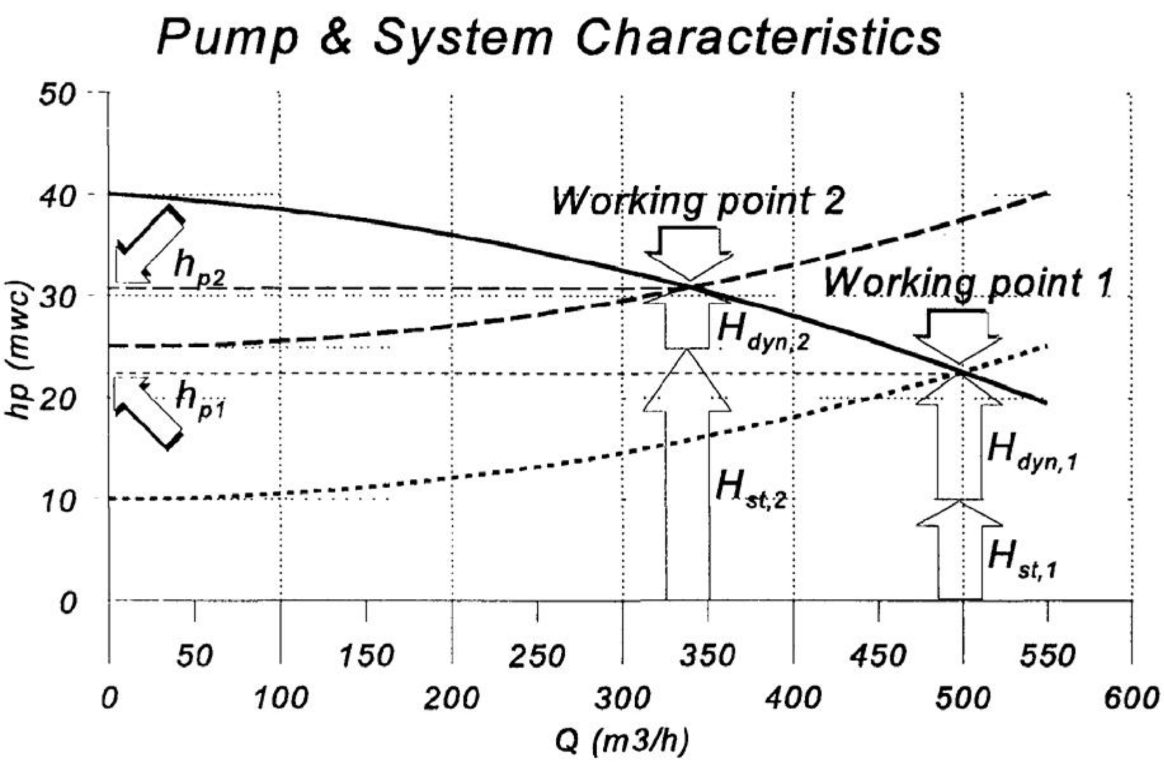
Number of pumps is the n + 2 for standby with n the minimum number of pumps equal to the power required dived by the capacity (power for one pump) for one pump.

Water is by pumps usually transported to higher elevations, and in that case the static head includes elevation difference Δz. Thus:

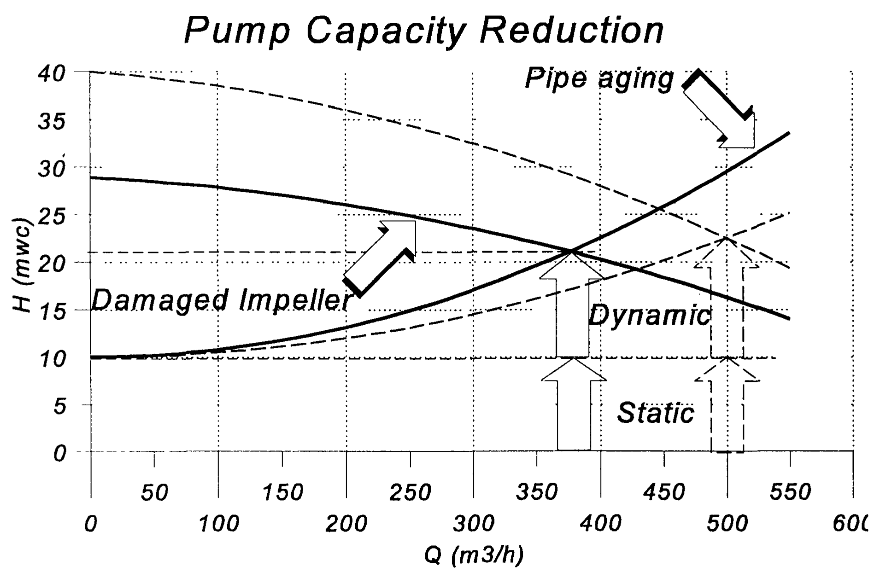
$$h_p = H_{dyn} + H_{st} = \Delta H + \frac{p_{end}}{\rho g} + \Delta Z$$

If water is pumped directly into the system, the demand variations should be followed by the pumping schedule.

Capacity and pressure which can be delivered, are determined from the system and pump characteristics. The intersection of these two curves, the *working point*, shows required pumping head which provides the delivery and static head as indicated on the graph



Once determined, the maximum pumping capacity may vary in time. Aging of the pump impeller, pipe corrosion, increase of leakage, etc. will cause reduction of the maximum flow which can be delivered

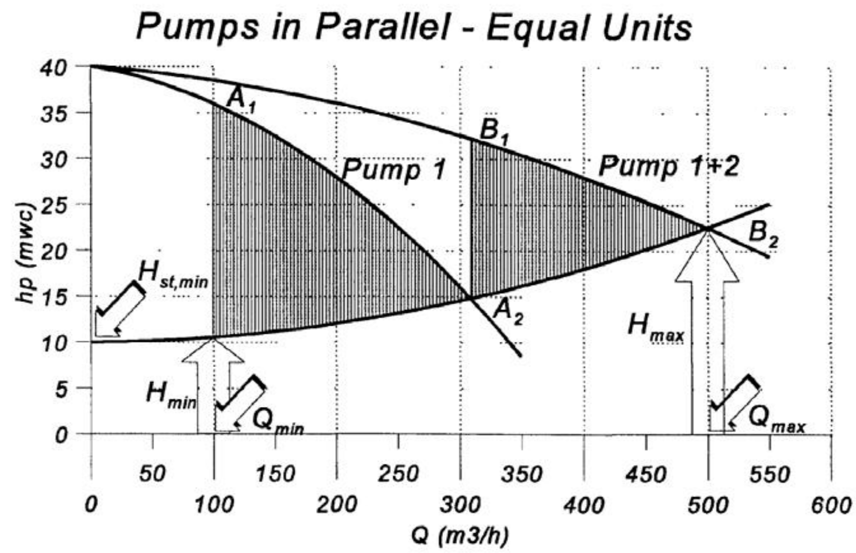


While thinking about number and size of pumps in a pumping station, general intention is to keep the pressure variations in the system at lowest acceptable level, in order to minimise pumping energy. For that reasons, several pumps connected to the same delivery main, can be installed in parallel. The composite pump curve is obtained by adding the pump discharges at the same pumping head, hence for n pumps:

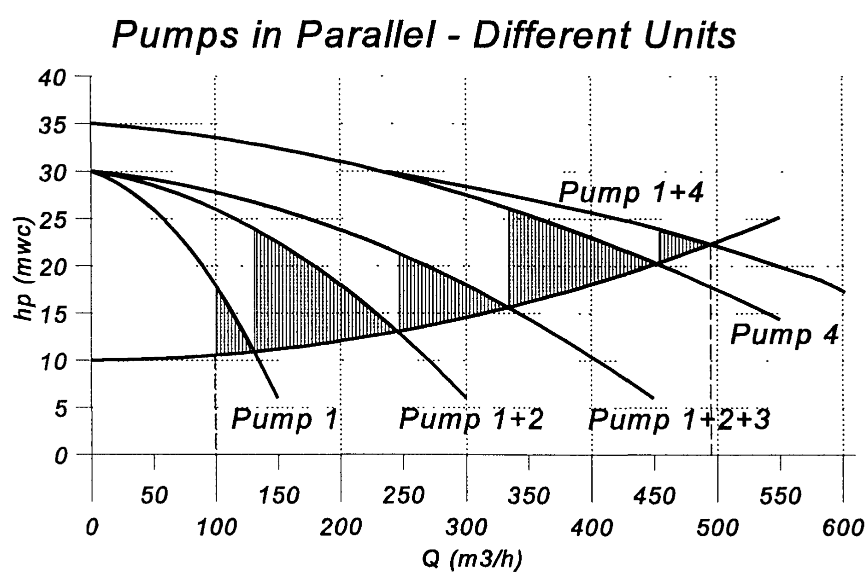
$$h_p = h_1 = h_2 = \ldots = h_n$$

$$Q_p = Q_1 + Q_2 + \ldots + Q_n$$

The following figure shows operation of two equal pump units in parallel arrangement. The system should preferably operate at any point of the line $A_1 - A_2 - B_1 - B_2$, between the minimum and maximum flow.



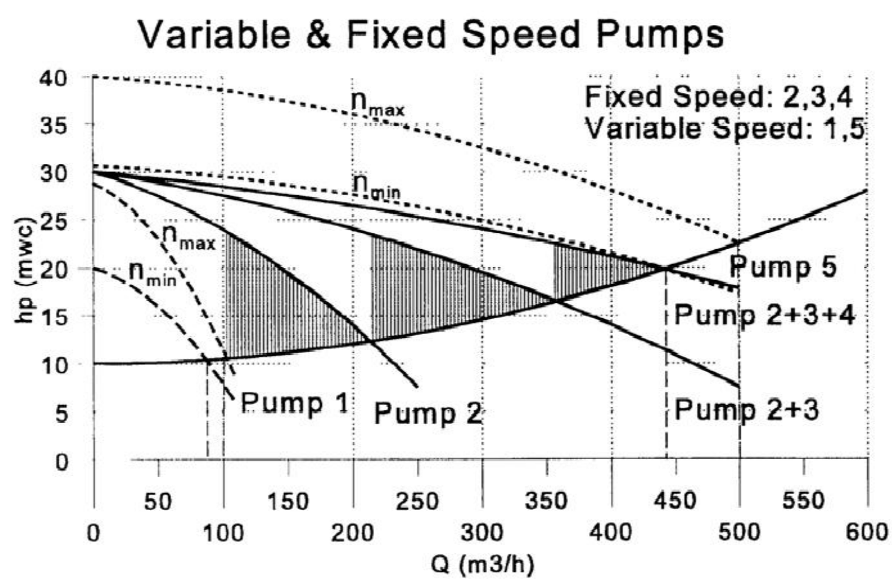
The shaded area from the figure indicates excessive pumping, which is unavoidable when fixed speed pumps are used. A properly selected combination of pump units, should reduce this area. This is often achieved by installing pumps of different capacity



The excessive head is eliminated completely by operation of variable speed pumps. The flow variation is met in this case by adjusting the impeller rotation, keeping the discharge pressure constant (figure below). The pump characteristics diagram will consist of family of curves for various pump frequencies *n (rpm)*. Relation between the pumping heads and flows between any two curves is proportional to the frequencies, in the following way:

$$\frac{Q_2}{Q_1} = \frac{n_2}{n_1} \quad ; \quad \frac{h_{p,2}}{h_{p,1}} = \left(\frac{n_2}{n_1}\right)^2$$

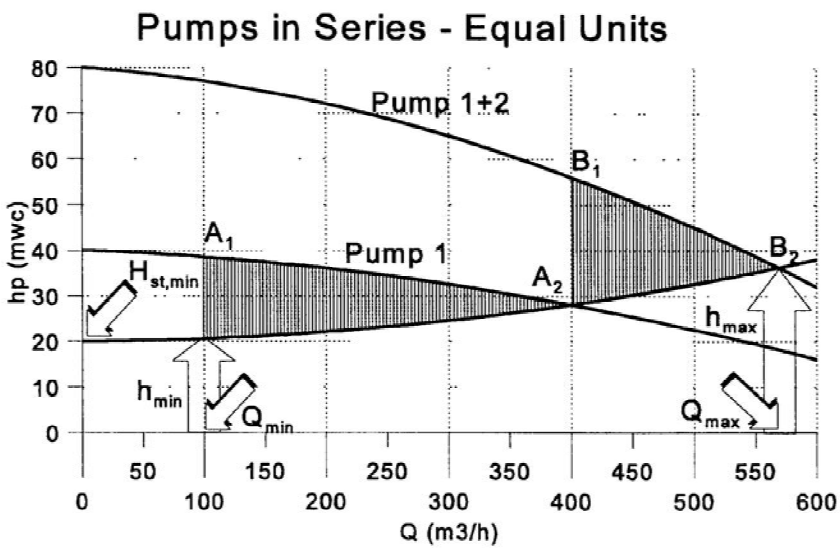
As it is hardly possible to cover wider range of flows with single pump unit, the variable speed pumps are often combined with fixed speed pumps, controlling the peak flows, only.



In case of large pressure variations, pumps have to be installed in series. The total head is equal to the sum of heads for each pump. For *n* equal units:

$$h_p = h_1 + h_2 + \ldots + h_n$$

$$Q_p = Q_1 = Q_2 = \ldots = Q_n$$



Water Distribution Systems

The purpose of distribution system is to deliver water to consumer with appropriate quality, quantity and pressure. Distribution system is used to describe collectively the facilities used to supply water from its source to the point of usage.

Requirements of Good Distribution System

1. Water quality should not get deteriorated in the distribution pipes.
2. It should be capable of supplying water at all the intended places with sufficient pressure head.
3. It should be capable of supplying the requisite amount of water during fire fighting.
4. The layout should be such that no consumer would be without water supply, during the repair of any section of the system.
5. All the distribution pipes should be preferably laid one metre away or above the sewer lines.
6. It should be fairly water-tight as to keep losses due to leakage to the minimum.

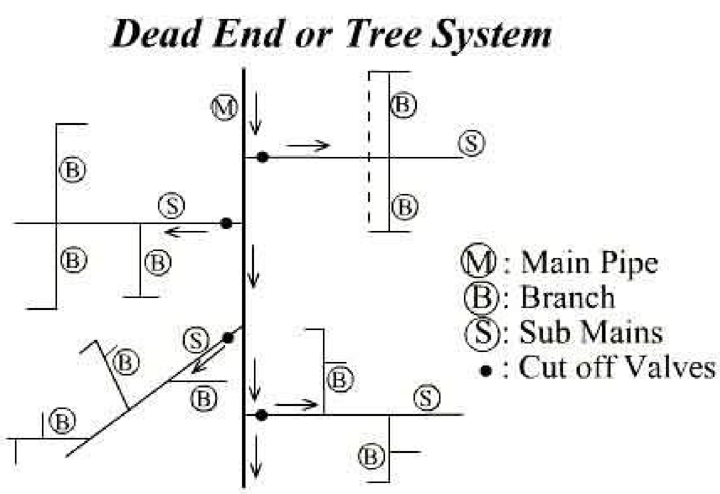
Layouts of Distribution Network

The distribution pipes are generally laid below the road pavements, and as such their layouts generally follow the layouts of roads. There are, in general, four different types of pipe networks; any one of which either singly or in combinations, can be used for a particular place. They are:

Dead End System
Grid Iron System
Ring System
Radial System

Dead End System:

It is suitable for old towns and cities having no definite pattern of roads.



Advantages:

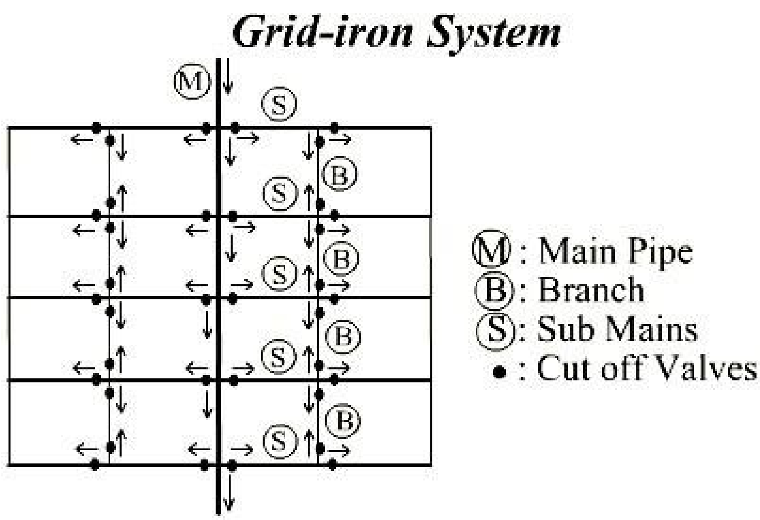
1. Relatively cheap.
2. Determination of discharges and pressure easier due to less number of valves.

Disadvantages

1. Due to many dead ends, stagnation of water occurs in pipes.
2. Less uniformity
3. End users suffer

Grid Iron System:

It is suitable for cities with rectangular layout, where the water mains and branches are laid in rectangles.



Advantages:

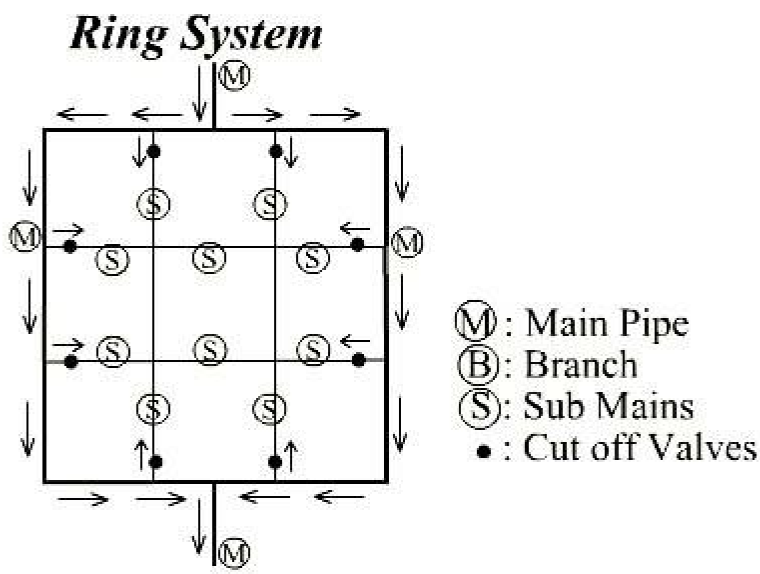
1. Water is kept in good circulation due to the absence of dead ends.
2. In the cases of a breakdown in some section, water is available from some other direction.
3. Uniform pressure
4. Better services and better maintenance

Disadvantages

1. Exact calculation of sizes of pipes is not possible due to provision of valves on all branches.
2. More cost

Ring System:

The supply main is laid all along the peripheral roads and sub mains branch out from the mains. Thus, this system also follows the grid iron system with the flow pattern similar in character to that of dead end system. So, determination of the size of pipes is easy.



Advantages:

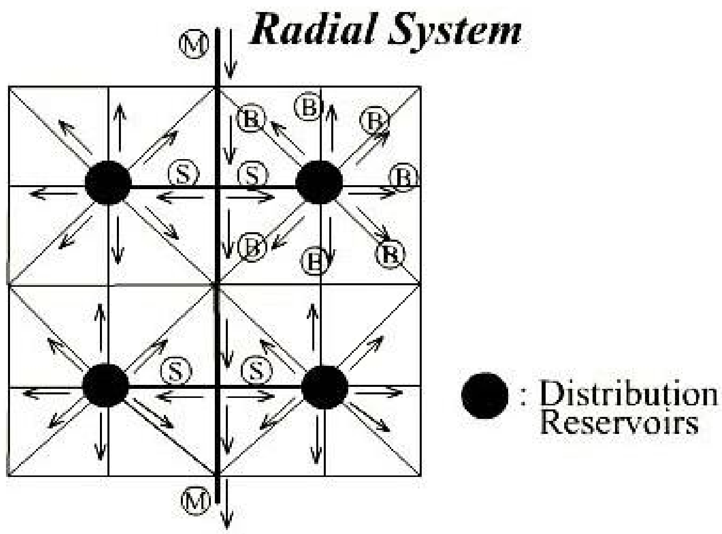
1. Water can be supplied to any point from at least two directions.
2. Uniform pressure
3. Better services and better maintenance

Disadvantages

1. More cost

Radial System:

The area is divided into different zones. The water is pumped into the distribution reservoir kept in the middle of each zone and the supply pipes are laid radially ending towards the periphery.



Advantages:

1. It gives quick service.
2. Calculation of pipe sizes is easy.
3. Uniform pressure and better maintenance

Disadvantages

1. Most expensive

Pipe Network Analysis

Analysis of water distribution system includes determining quantities of flow and head losses in the various pipe lines, and resulting residual pressures. In any pipe network, the following two conditions must be satisfied:

1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Based on these two basic principles, the pipe networks are generally solved by the methods of successive approximation. The widely used method of pipe network analysis is the Hardy-Cross method.

Design of water supply networks

Design discharge

A) Case of Tree or ring networks

$\boldsymbol{Q_{des} = Q_{av\ annual} \times P}$ with p the peak factor

Table of peak factors

population	urban	rural
Up to 50 000	2.25	2.00
50 000 – 100 000	2.00	1.80
100 000 – 500 000	1.4 – 1.6	
1 000 000 and more	1.2 – 1.4	

B) Case of grid Iron networks

Transmission lines: $Q_{des} = Q_{max\ daily} + Q_{fire}$

Main and secondary lines $Q_{des} = Q_{max\ daily} + Q_{fire}$

or $Q_{des} = Q_{max\ hourly}$

Take the greater value

Minor distributors and services connectors: $Q_{max\ daily}$

or $+ Q_{fire}$

Take the greater value

Method of sections

It assumes several sections perpendicular to the direction of flow:

1. Calculate the served area by the section
2. Calculate the design discharge for the portion served by the section
3. Find the number of the pipes serving this area region

4. Assume suitable diameters for these pipes
5. Find the discharge capacity for each pipe (assu ing v= 1m/s)
6. Find the total discharge capacity for all the pipes crossing the section
7. Compare the total capacity with the design discharge (demand)
8. Readjust the pipe diameters (repeat from step 5)
9. Stop when the difference is ≤ 5% of the design discharge

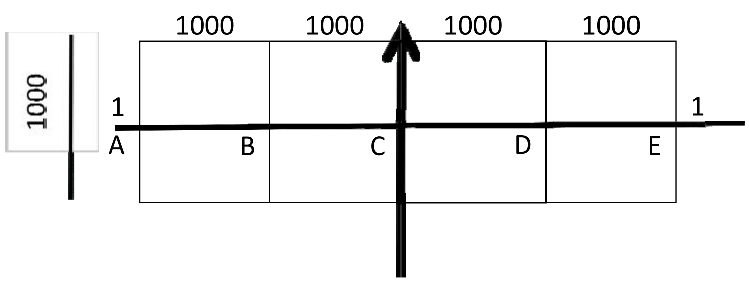
Example: Consider section 1-1 for the network; It is required to calculate the diameters of the pipes A, B, C, D and E. consider the population intensity to be 1000 persons/ hectare and the average annual consumption rate to be 125 l/c/d; Take $Q_{fire} = 0.05$ m^3 /s.

Solution

Population = area x intensity = (1000 x 4000 x 1000) / (100 x 100) = 400 0000

Q_{des} = ((125 x 400 000 x 1.8) / (1000 x 24 x 3600)) + 0.05 = 1.09 m^3 /s.

$Q_{max\ hourly}$ = (125 x 400 000 x 2.7) / (1000x24x3600) = 1.5625 m^3 /s.



		1ST Trial		2ND Trial		3RD Trial		
Section	Pipe	D	Q_{cap}	D	Q_{cap}	D	Q_{cap}	Selected D
	A	0.3	0.071	0.45	0.159			0.45
	B	0.7	0.385	0.7	0.385			0.7
	C	0.7	0.385	0.7	0.385			0.7
	D	0.7	0.385	0.7	0.385			0.7
	E	0.3	0.071	0.45	0.159			0.45
Sum			1.295		1.472			

Check the first trial difference: 1.295 - 1.45 = -0.155

% error = (0.155 x 100)/1.45 = 10.69 % > 5%

Check the first trial difference: 1.472 - 1.45 = 0.022

% error = (0.022 x 100)/1.45 = 1.52 % OK

Hardy-Cross Method

This method consists of assuming a distribution of flow in the network in such a way that the principle of continuity is satisfied at each junction. A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

If Q_a is the assumed flow and Q is the actual flow in the pipe, then the correction δ is given by

$\delta = Q - Q_a$; or $Q = Q_a + \delta$

Now, expressing the head loss (H_L) as

$H_L = K.Q^x$

we have, the head loss in a pipe

$= K.(Q_a + \delta)^x$

$= K.[Q_a^x + x.Q_a^{x-1}\delta + ...\text{negligible terms}]$

$= K.[Q_a^x + x.Q_a^{x-1}\delta]$

Now, around a closed loop, the summation of head losses must be zero.

$\Sigma K.[Q_a^x + x.Q_a^{x-1}\delta] = 0$

or $\Sigma K.Q_a^x = -\Sigma Kx\ Q_a^{x-1}\delta$

Since, δ is the same for all the pipes of the considered loop; it can be taken out of the summation.

$\Sigma K.Q_a^x = -\delta.\ \Sigma Kx\ Q_a^{x-1}$

or $\delta = -\Sigma K.Q_a^x / \Sigma x.KQ_a^{x-1}$

Since δ is given the same sign (direction) in all pipes of the loop, the denominator of the above equation is taken as the absolute sum of the individual items in the summation. Hence,

or $\delta = -\Sigma K.Q_a^x / \Sigma \mid x.KQ_a^{x-1} \mid$

or $\delta = -\Sigma H_L / x.\Sigma \mid H_L/Q_a \mid$

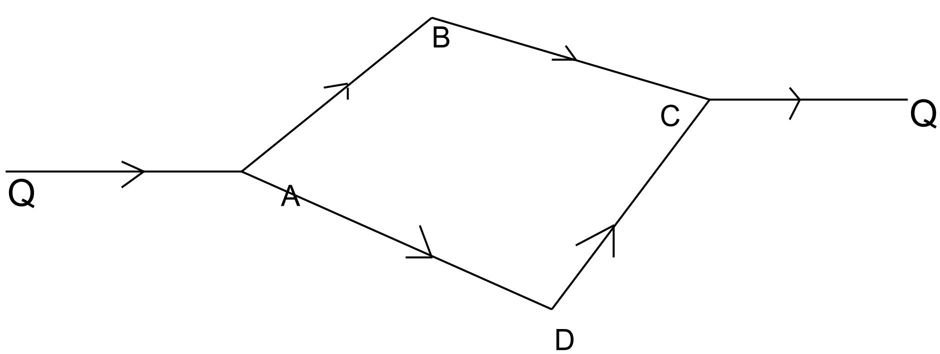
where H_L is the head loss for assumed flow Q_a.

The numerator in the above equation is the algebraic sum of the head losses in the various pipes of the closed loop computed with assumed flow. Since the direction and magnitude of flow in these pipes is already assumed, their respective head losses with due regard to sign can be easily calculated after assuming their diameters. The absolute sum of respective KQ_a^{x-1} or H_L/Q_a is then calculated. Finally the value of δ is found out for each loop, and the assumed flows are corrected. Repeated adjustments are made until the desired accuracy is obtained.

The value of **x** in Hardy- Cross method is assumed to be constant (i.e. 1.85 for Hazen-William's formula, and 2 for Darcy-Weisbach formula).

PRACTICALLY

The method is based on the fact that $H_{f\,A\text{-}B} + H_{f\,B\text{-}C} = H_{f\,A\text{-}D} + H_{f\,D\text{-}C}$



Steps:

- Assume reasonable discharges in each pipe
- Each should be either positive or negative according to the direction of flow
- Get the pipe diameter after assuming the velocity (1 – 1.5 m/s)
- Calculate the new modified discharge in each pipe as $Q_{new} = Q_{old} + \Delta$

$$\Delta = -\frac{\sum H_f}{2\sum \frac{H_f}{Q}}$$

- Repeat until Δ becomes < 3% of the main pipe discharge

Solution is always made in Table form like the following one:

Network Element					1^{st} Trial				2^{nd} Trial				
Loop Nr	**Line Nr**	**Assumed Q (m^3/s)**	**D (m)**	**L (m)**	**H_f**		**H_f/Q**	**Δ**	**Q**	**H_f**		**H_f/Q**	**Δ**
					+	−				+	−		
1	A-B	+											
	B-C	+						Same Value	$Q_{old} + \Delta$				
	A-D	−											
	D-C	−											
					$\sum H_f$		$\sum H_f/Q$			**$\sum H_f$**		$\sum H_f/Q$	