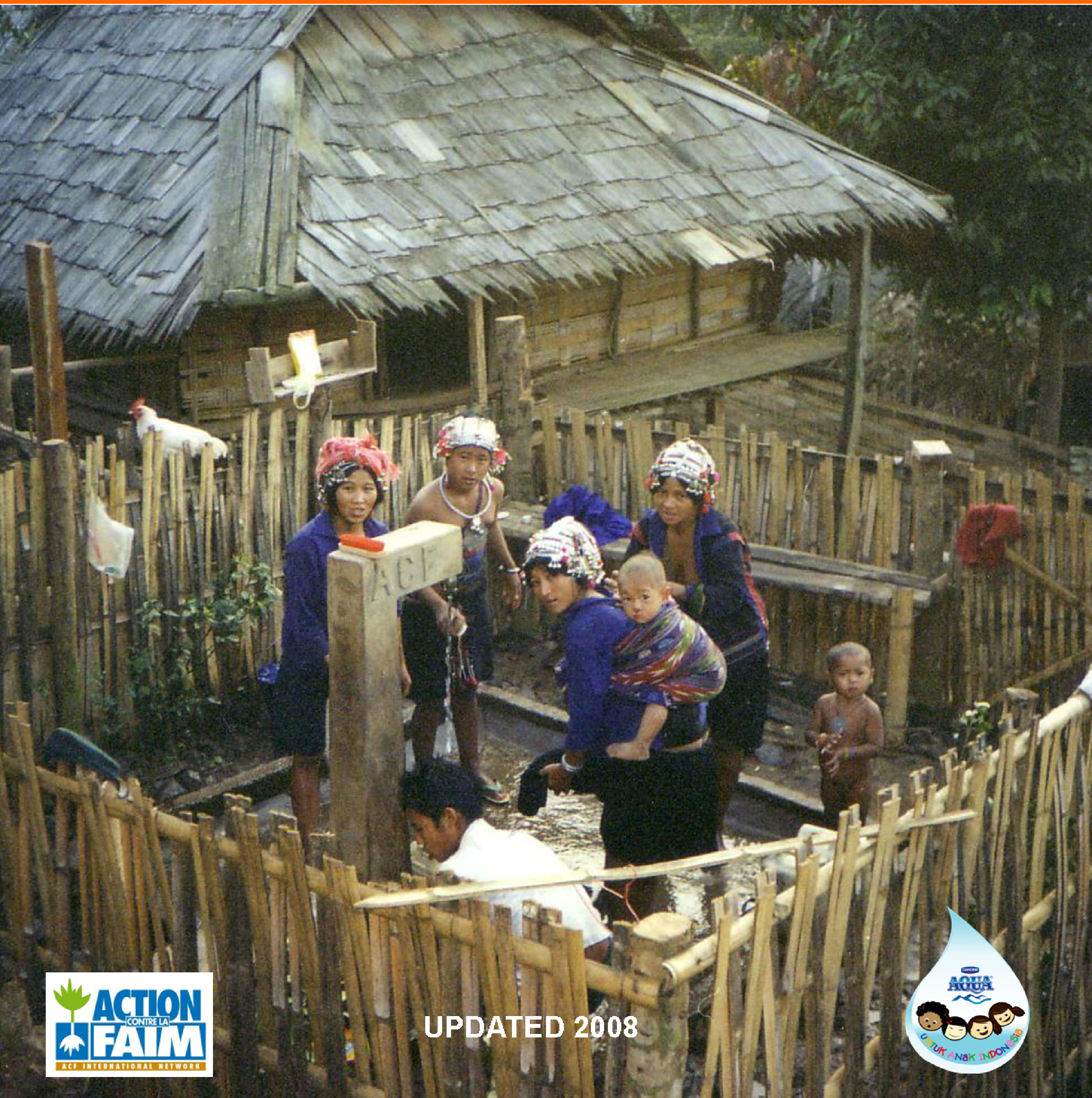
# DESIGN, SIZING, CONSTRUCTION AND MAINTENANCE OF GRAVITY-FED SYSTEM IN RURAL AREAS

## MODULE 2: PRINCIPLES AND SIZING OF A GRAVITY FED SYSTEM

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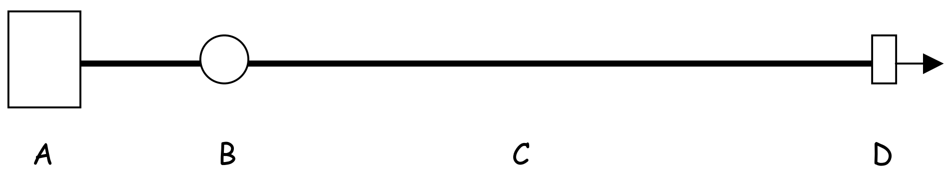
# I. INTRODUCTION

In order to size a gravity fed system, it is important and necessary that a technician knows and understands how gravity fed systems work.
In this module, a first section (chapter II) is devoted to the functioning of gravity fed system, giving definitions of concept such as gravity, pressure and head loss. A brief description of the various infrastructures and their use is also given in this section.
A second section (chapter III) is devoted to the sizing and designs, i.e. how to size and where to place the various infrastructures of a gravity fed system.
Calculation exercises are given at the end of the module (paragraph III.7) in order to put into practice the theory developed in this module (exercise corrections are given in appendix).

# II. PRINCIPLE OF A GRAVITY FED SYSTEM

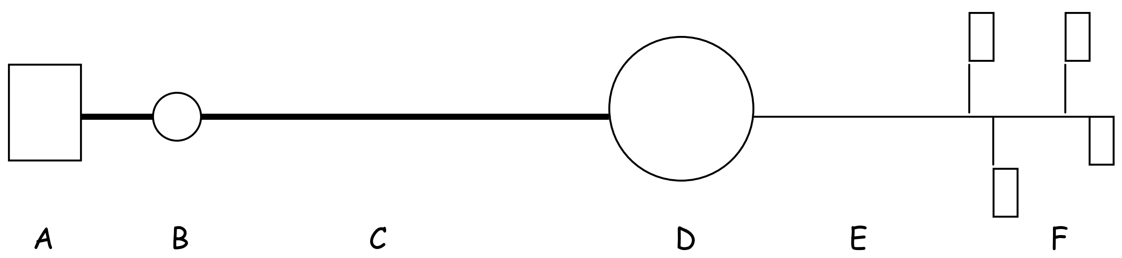
## II.1. WHAT ARE THE MAIN ELEMENTS OF A GRAVITY-FED SYSTEM AND THEIR FUNCTION?

A simple spring catchment includes the following elements:




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.

## II.2. HOW DOES A GRAVITY FED SYSTEM WORK?

### II.2.1. 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 drawing of picture 1:
- 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.

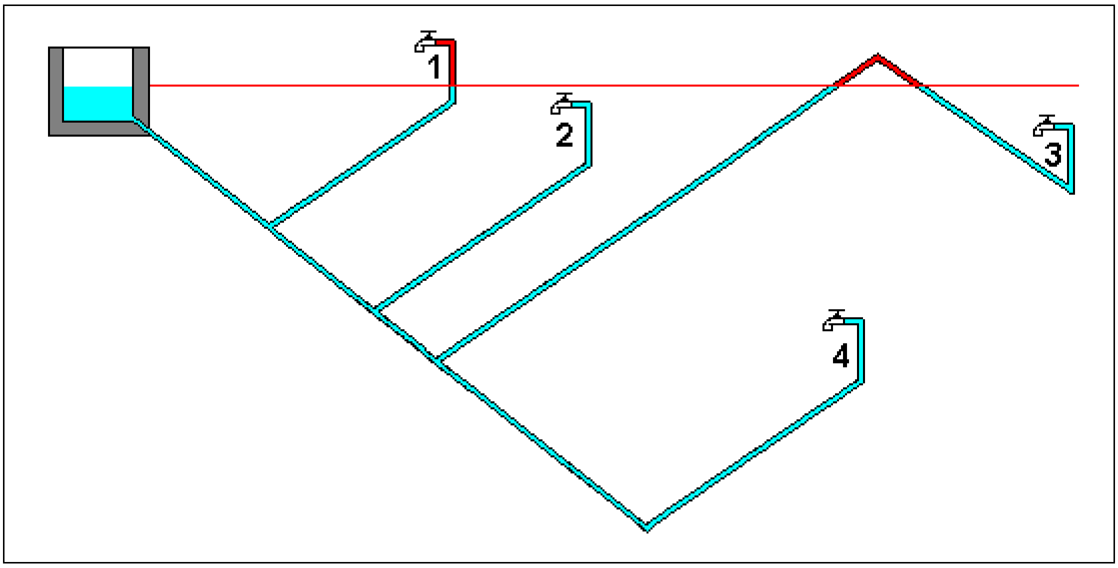


Fig.1: Illustration of the gravity force

### II.2.2. Pressure

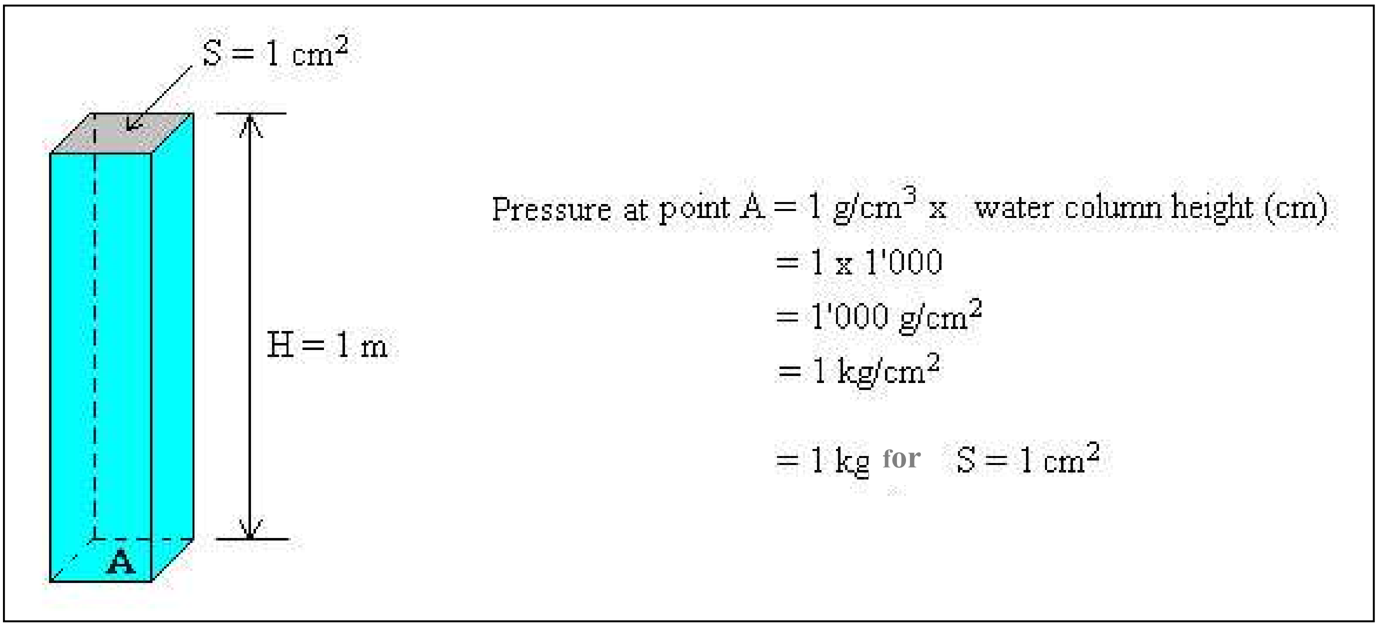
The water pressure is the force which water exerts in the walls of the container it is contained (pipe's walls, reservoir's wall…).
The pressure in a considered point correspond (or is equivalent) to the weight of water column above this point. Knowing that the density of water is 1 g/cm3, we can easily 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.




Let's take the following example:

The pressure units are the kg/cm$^2$, the bar or the "metres water gauge":
1 kg/cm$^2$ = 1 bar = 1 mWG

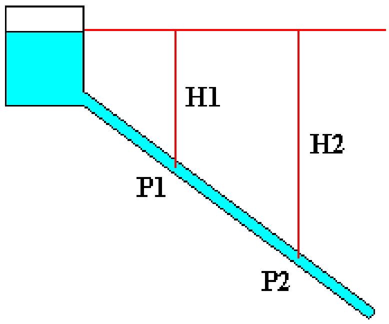
For the hydraulic calculations used for the sizing of a gravity fed system, we always measure the pressure in mWG.
We have to differentiate the **static pressure** from the **dynamic pressure**:

- 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),

- 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).




$$P_{static}\ (\text{mWG}) = H\ (\text{m})$$

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 picture 2: what is the pressure exerted by water in the pipe at the points A, B, C, D and E?

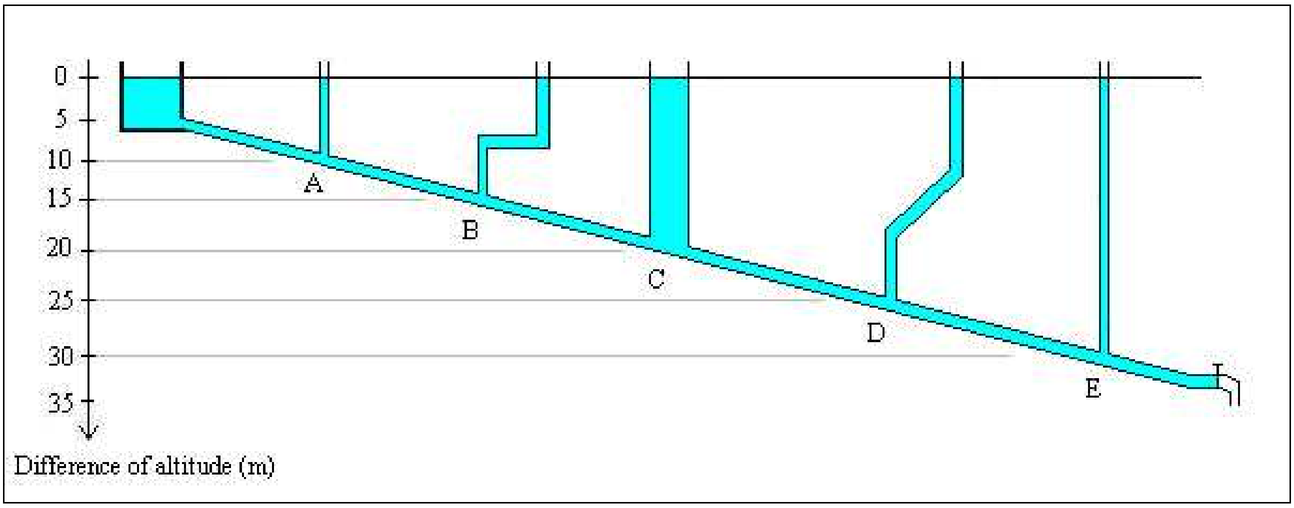



Fig.2: Static pressure.

Answer: - Point A: $P_{static}$ = 10 meters.
- Point B: $P_{static}$ = 15 meters.
- Point C: $P_{static}$ = 20 meters.
- Point D: $P_{static}$ = 25 meters.
- Point E: $P_{static}$ = 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 1.

Table 1: Pipes pressure level NP

| 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 picture 3 (case B).

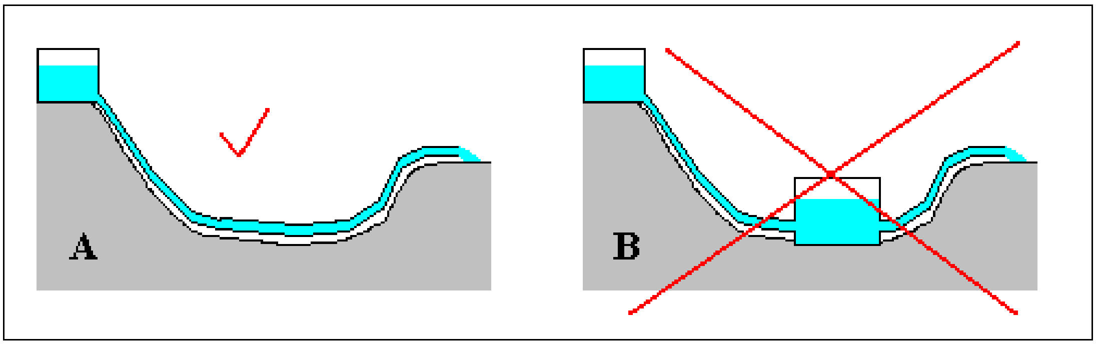



Fig.3: The static pressure becomes null in a tank: the scenario B is not possible, the water will not

run after the second tank.

In the example of picture 4, 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. Picture 5 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.

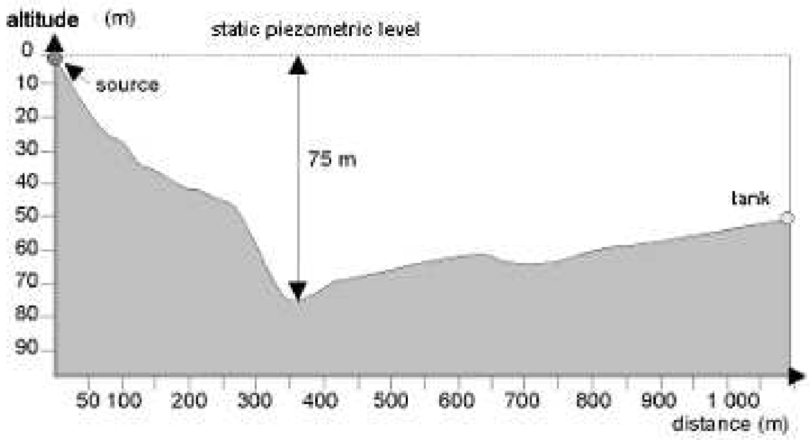



Fig.4: Example of topographic survey with maximum height H =75m.

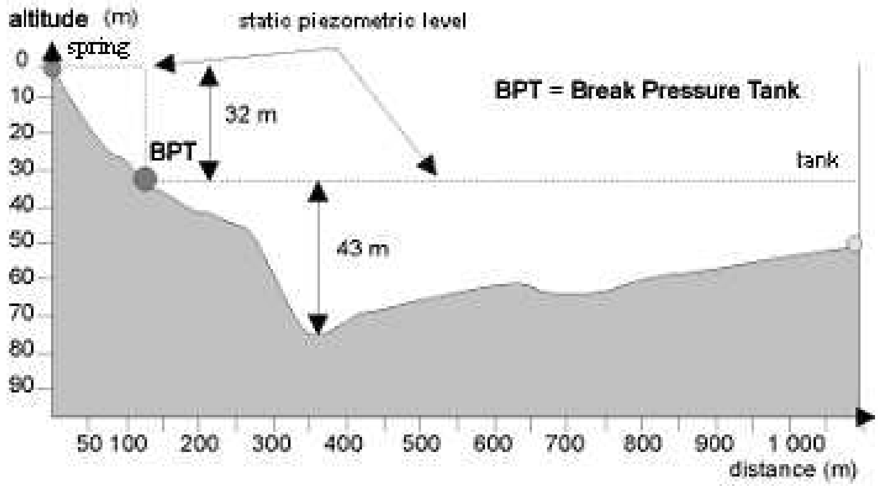



Fig.5: Installation of a break pressure tank to respect the pipe nominal pressure (NP6).

→ **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 picture 6. 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:

$$P_{residual}\ (\text{mWG}) = H\ (\text{m}) - \Delta P\ (\text{m})$$

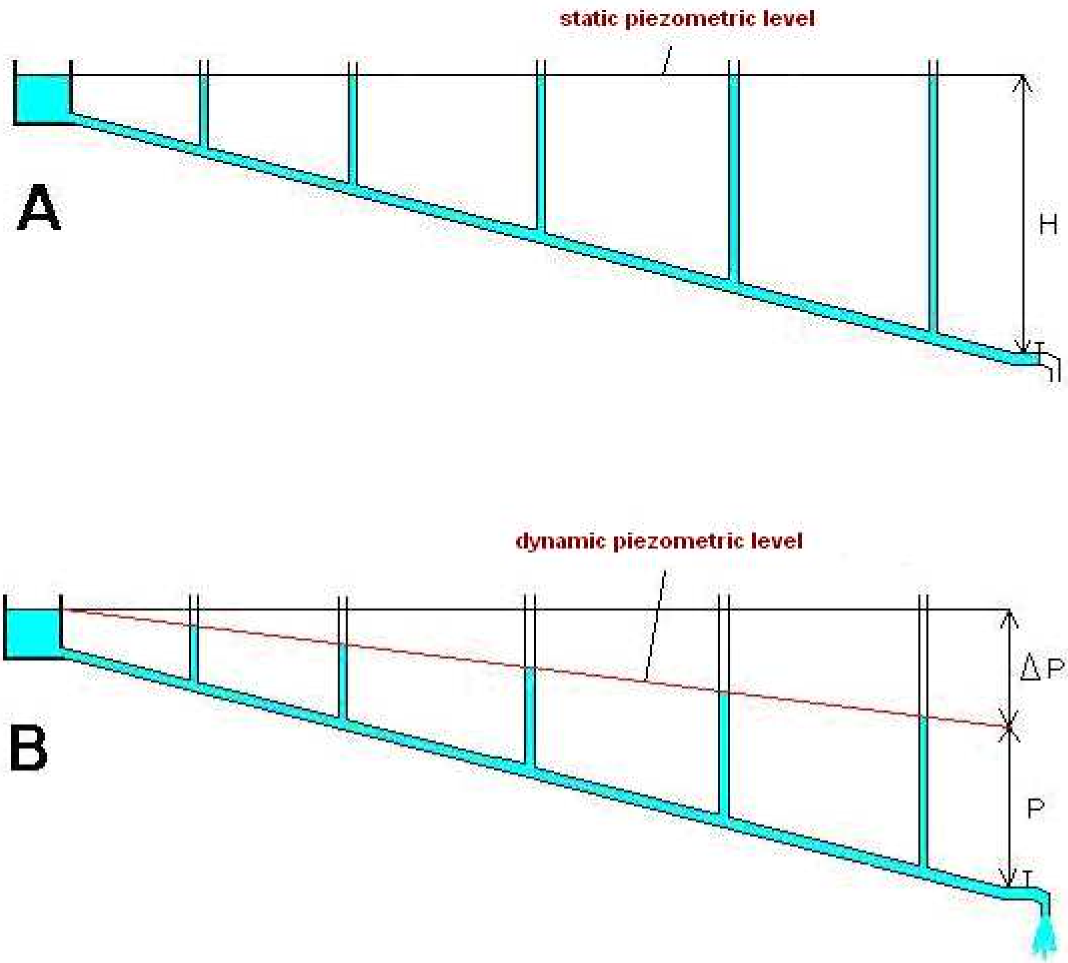



Fig.6: Static level (illustrated in A where the taps are closed) and dynamic level (illustrated in B where taps are opened) of a pipeline.

Figure 7 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.

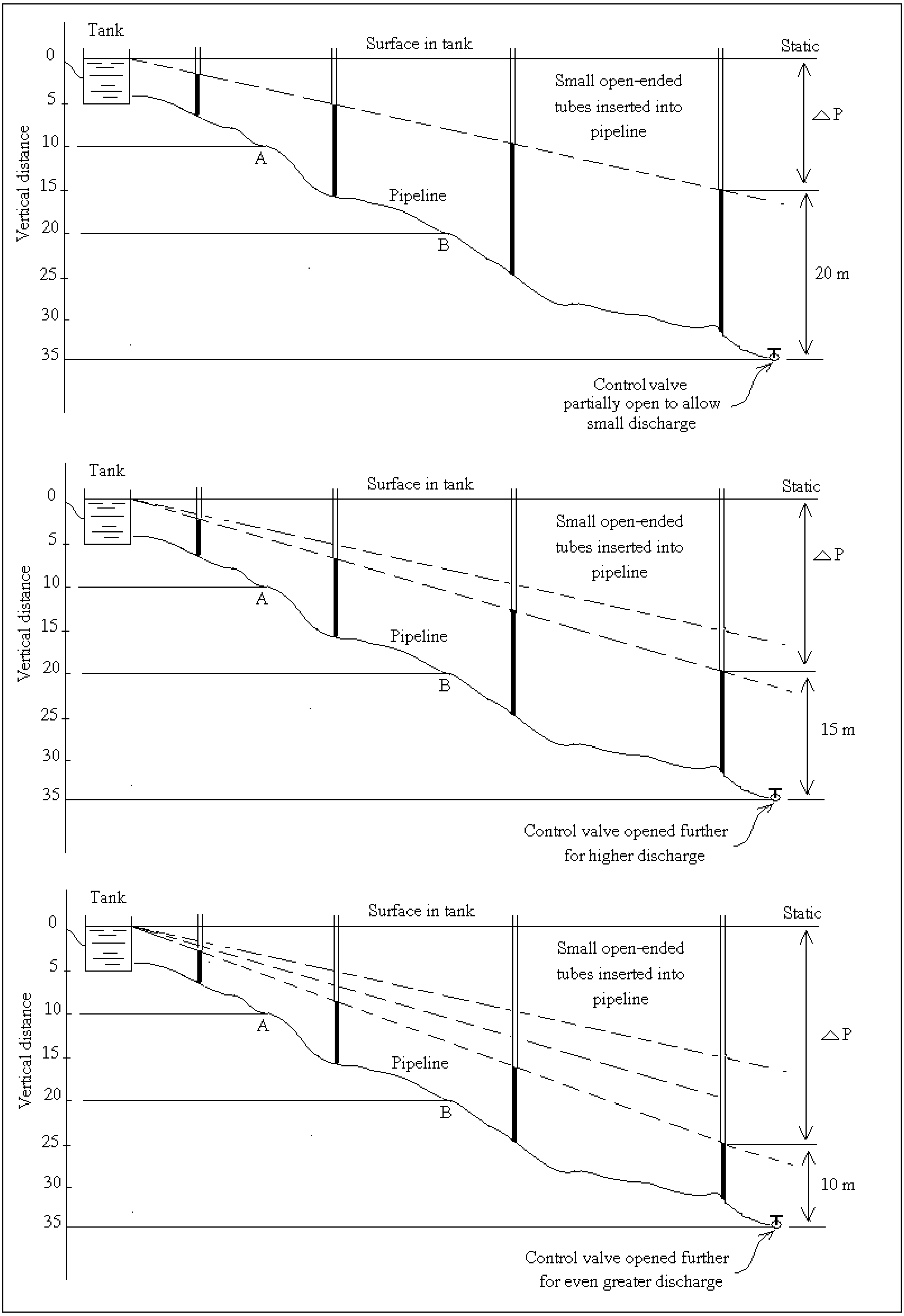



Fig.7: Variation of the head losses according to the flow of water in pipes.

## II.2.2. How to calculate the head losses (ΔP)?

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 (mm): pipe diameter. For a given flow, the smallest is the pipe diameter, the more important are the head losses.
- Q (l/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.

Linear head losses are generally expressed in metres of head loss per 100 m of pipe. A head loss coefficient of 1% therefore corresponds to a loss of 1 mWG of pressure for each 100 m of pipe length.

The numerical calculation of the linear head losses in % (f) is done using a nomograph, which represents on a graphic form the relations between the internal pipe diameters (d), the water flow in pipes (q) and the linear head losses (f), for a given roughness.

The nomograph in appendix 2 allows determining the linear head losses for the plastic pipes (on the left part of the graph) and for the GI pipes (on the right part of the graph).
To calculate the linear head losses (f), it is necessary to use the <u>internal diameter</u> of the pipes. For the GI pipes, the diameters given by the pipe manufacturer correspond to the internal diameters, whereas for the plastic pipes (PVC and PE) the diameters given by the pipe manufacturer correspond to the external diameters. Generally, PVC or PE pipe's internal diameter should be calculated using the thickness of the pipe walls, measured or given by the manufacturer catalogue. An example of external and internal diameters of Wavin PVC pipes (main brand of plastic pipe available in Indonesia) is given in table 6.

The head losses in mWG (ΔP ) created in a length of pipe L can be obtained with the following formula:

$$\Delta P = L \times f / 100$$

→ **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.

# III. SIZING AND DESIGN OF GRAVITY FED SYSTEM

## III.1. INFRASTRUCTURES POSITIONING

### III.1.1. 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.

### III.1.2. 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 picture 8 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.

### III.1.3. 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.

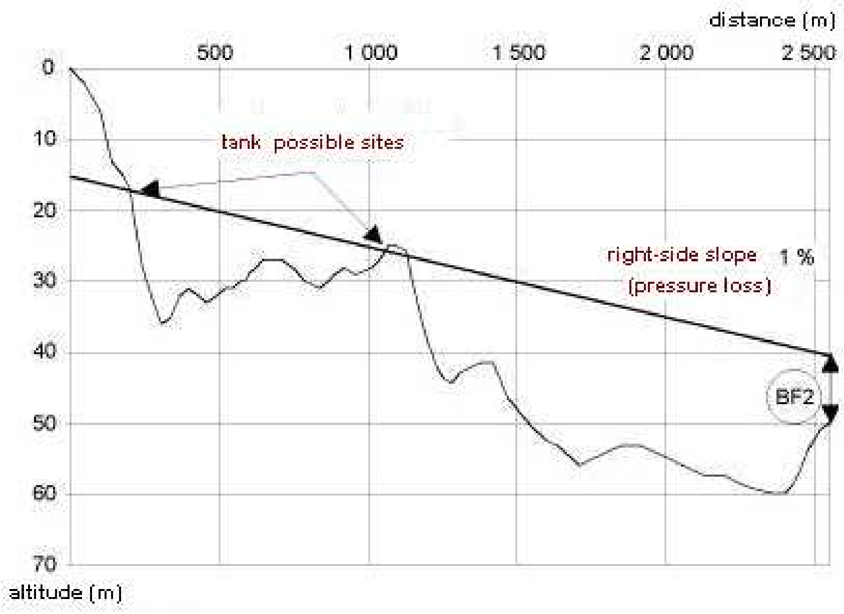



Fig.8: Possible choices for tank location

## III.2. 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: $V = Q \times t$

V = tank volume (litres)

Q = water flow (litres/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 litres.

## III.3. 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 (see picture 9).

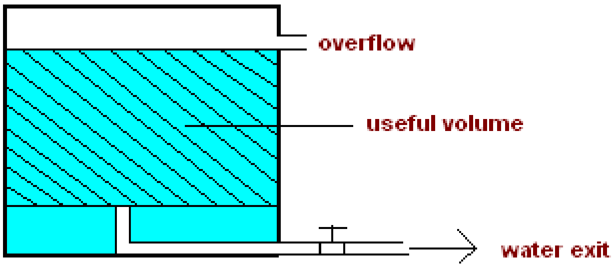



Fig.9: Useful tank volume.

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 0.58 m3 / h, a first approximation of the tank volume is 0.58 X 10 = 5,8 m3.

To make an optimal tank sizing, it is necessary to calculate the population water requirements by sections of time using a consumption coefficient for each time section. The needs for the given time duration are obtained by multiplying the daily needs (calculated for 10 years) by the consumption coefficient of the given time section.
Table 2 shows an example of a standard hourly water demand calculation for a South-East Asian village. However, for each country, the coefficient should be adapted. The chart corresponding to table 3 is given in picture 10.

Table 2: The evolution of the water demand in hourly period for a village (the total daily needs are 11,83m3/day)

| Hours (h) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Consomption coefficient (%) | 0 | 0 | 0 | 0 | 0 | 5 | 9.5 | 5 | 3 | 4 | 6.5 | 5 | 6 | 5 | 7 | 10 | 10 | 9 | 11 | 4 | 0 | 0 | 0 | 0 |
| Hourly need ($m^3$) | 0 | 0 | 0 | 0 | 0 | 0.6 | 1.1 | 0.6 | 0.4 | 0.5 | 0.8 | 0.6 | 0.7 | 0.6 | 0.8 | 1.2 | 1.2 | 1.1 | 1.3 | 0.5 | 0 | 0 | 0 | 0 |

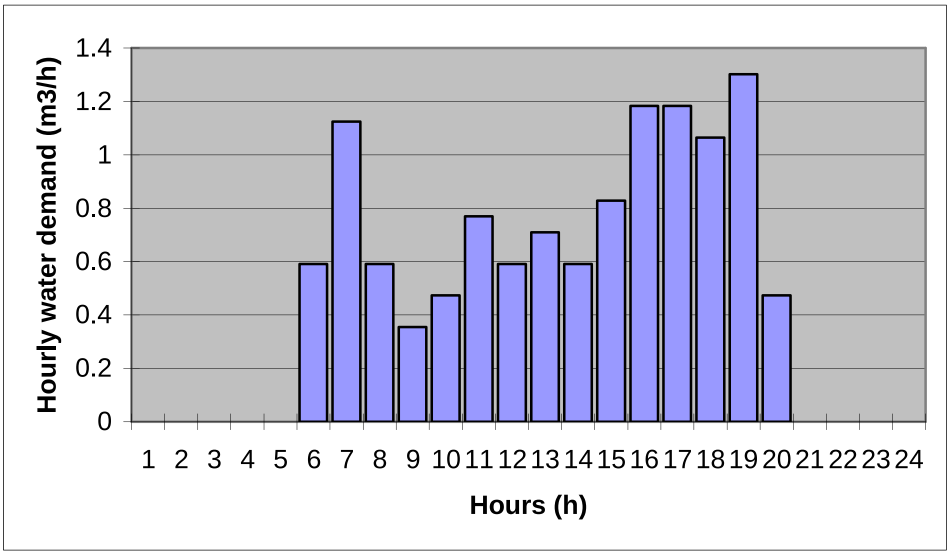



Fig.10: Chart of hourly water demand, as calculated in table 2.

The method to calculate the necessary useful tank volume is as follows (see the example given on table 3):

1. Calculate the water needs according to the time (column 3).
2. Calculate the spring flow per hour: can be obtained by dividing the daily flow of the spring by 24 (column 4)
3. Calculate the difference between the spring flow per hour and the hourly water demand (column 5 = column 4 – column 3). When the difference is negative (sign -), the water provided by the spring is insufficient.
4. Calculate the water stock: can be obtained by cumulating the differences calculated previously (column 6).
5. 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 7 = column 6 + 0.41 $m^3$). The highest water stock deficit (for this example it is 0.41$m^3$) represents the reserve volume of water that should be inside the tank in order to have always a positive stock. Indeed, at 19h, we notice a deficit of volume of -0.41$m^3$: 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.
6. 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 = 2.88 - (- 0.41) = 2.88 + 0.41 = 3.28 $m^3$.

But if the tank has this volume, there will be some overflow, so some water will be lost. We should hence calculate the "recommended tank volume", for which their will be no overflow

7. Calculate the daily Overflow Volume OF. It can be obtained by withdrawing the daily needs from spring flow during 24 hours (OF = 13.82 – 11.83 = 1.99 $m^3$).
8. 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:

V = Vmin + OF = 3.29 + 1.99 = 5.3 $m^3$.

Table 3: Calculation of the hourly water demand and the volume of water inside the reservoir (the flow of the spring is 13.82 $m^3$/day and the daily needs is 11.83 $m^3$/day).

| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|
| Period (h) | Consumption coefficient (%) | Hourly water demand ($m^3$) | Source production ($m^3$) | Difference ($m^3$) | Water Stock ($m^3$) | Necessary quantity of water stored in the tank ($m^3$) |
| 1 | 0 | 0 | 0.58 | 0.58 | 0.58 | 0.99 |
| 2 | 0 | 0 | 0.58 | 0.58 | 1.15 | 1.56 |
| 3 | 0 | 0 | 0.58 | 0.58 | 1.73 | 2.14 |
| 4 | 0 | 0 | 0.58 | 0.58 | 2.30 | 2.72 |
| 5 | 0 | 0 | 0.58 | 0.58 | 2.88 | 3.29 |
| 6 | 5.0 | 0.59 | 0.58 | -0.02 | 2.86 | 3.28 |
| 7 | 9.5 | 1.12 | 0.58 | -0.55 | 2.32 | 2.73 |
| 8 | 5.0 | 0.59 | 0.58 | -0.02 | 2.30 | 2.71 |
| 9 | 3.0 | 0.35 | 0.58 | 0.22 | 2.52 | 2.94 |
| 10 | 4.0 | 0.47 | 0.58 | 0.10 | 2.63 | 3.04 |
| 11 | 6.5 | 0.77 | 0.58 | -0.19 | 2.43 | 2.84 |
| 12 | 5.0 | 0.59 | 0.58 | -0.02 | 2.42 | 2.83 |
| 13 | 6.0 | 0.71 | 0.58 | -0.13 | 2.28 | 2.70 |
| 14 | 5.0 | 0.59 | 0.58 | -0.02 | 2.27 | 2.68 |
| 15 | 7.0 | 0.83 | 0.58 | -0.25 | 2.02 | 2.43 |
| 16 | 10.0 | 1.18 | 0.58 | -0.61 | 1.41 | 1.82 |
| 17 | 10.0 | 1.18 | 0.58 | -0.61 | 0.80 | 1.21 |
| 18 | 9.0 | 1.06 | 0.58 | -0.49 | 0.31 | 0.73 |
| 19 | 11.0 | 1.30 | 0.58 | -0.73 | -0.41 | 0.00 |
| 20 | 4.0 | 0.47 | 0.58 | 0.10 | -0.31 | 0.10 |
| 21 | 0 | 0 | 0.58 | 0.58 | 0.27 | 0.68 |
| 22 | 0 | 0 | 0.58 | 0.58 | 0.84 | 1.25 |
| 23 | 0 | 0 | 0.58 | 0.58 | 1.42 | 1.83 |
| 24 | 0 | 0 | 0.58 | 0.58 | 1.99 | 2.41 |
| TOTAL | **100 %** | **11,83 $m^3$** | **13,82 $m^3$** | **1,99 $m^3$** | | |

For the example illustrated on table 3, we obtain a recommended useful volume for the tank of 5.3 $m^3$. We remember that the first estimation of the useful tank volume that we made gave a volume of 5.83 $m^3$ (= 0.58$m^3$ x 10hours).

For Vtank = Vmin, the water volume visualization in tank during the day is given in picture 11. We note that we consider that the tank does not start empty, but with the volume of the previous day. So the volume at the first hour is not 0.99 $m^3$ as indicated in the Table 3, but 2.41 $m^3$, the last volume of the previous day.

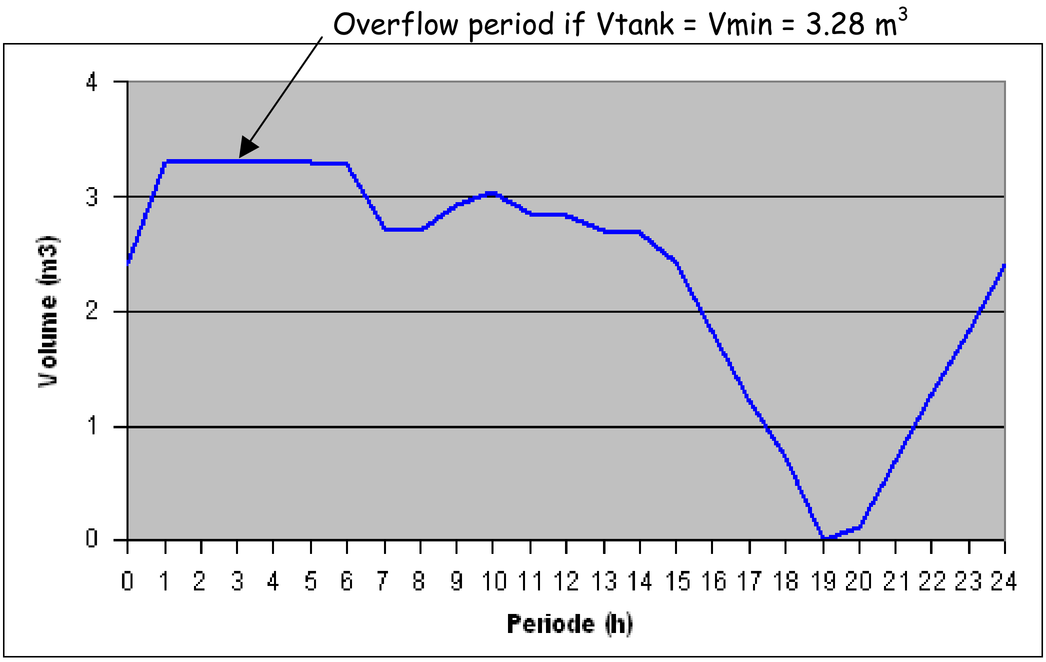



Fig.11: Water volume visualization in tank during the day.

## III.4. CHOSING AND SIZING THE PIPES

Plastic or 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) 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. In Indonesia, in NTT province, the main brand of GI pipe available is "Bakrie". They can have 2 qualities: BSP (used mainly for oil and gas), and GSP (used for water). GSP pipe can have several classes: "TT", "SIO" and "BOSS". The best one, used by ACF, is the TT class. The other types can get corroded very easily.

PVC or 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 like Jakarta or Surabaya. 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 hand over. ACF thus advices to use PVC pipes for the construction of GFS in rural areas of Indonesia.

Pressure pipes

Among the plastic pipes (PE or PVC), we can distinguish 2 types: pressure pipes, and non-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 the Wavin PVC inch standard, there are several type 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 headloss 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 4.

Table 4: Correspondence of pipes diameter

| 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. In Indonesia, the main supplier of plastic pipes is the company Wavin. They produced both PVC and PE pipe.

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:

Table 5: 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 |

Table 6: 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 |

For example, for a 1 ¼ " pipe, external diameter is 42mm
Internal diameter = 42 – thickness of the two walls = 42 – (2.3*2) = 37.4mm

The most careful attention should be paid when purchasing PVC-GI adaptors: on the GI pipe side, the second diameter given (for example 1" / 1") corresponds to the internal diameter of the GI pipe, while the first one correspond to the external diameter of PVC pipe

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.

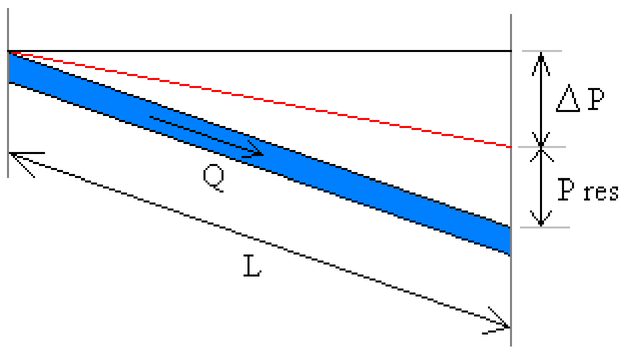
The sizing of the network must start with the main line, i.e. the part of the network located between the spring catchment and the storage tank. The main line is sized to allow to the maximum flow of the spring to pass through the pipes. Indeed, it is important to collect and transport the totality of 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:

*1. 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 litres/seconds

2. *We choose a diameter for our pipe and we calculate the head loss ΔP and the corresponding residual pressure.*

Example: H = 36m; Q = 1 l/s; L = 330m

We choose a DN 50 pipe (internal diameter = 40.8 mm)
Head loss in % = 1.7 m/100m = 1.7% (given by the nomograph)
Head loss in meters = 1.7 x L / 100 = 5.6m
Residual pressure = H – 5.6 = 30.4m

*3. We check that the conditions necessary to obtain a good water flow are respected. These conditions are as follows:*

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

→ **the water velocity in pipes must respect a certain interval**

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 depends on the pipe diameter. Generally, we can recommend the following limits 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 |

The water velocity in pipes can be calculated using the nomograph given in appendix, or by using the following formula:

$$V = 10^3 \times Q / A \text{ and } A = 3,14 \times d^2 / 4 \quad \rightarrow \quad V = 10^3 \times 4 \times Q / (3.14 \times d^2)$$

With: V = water velocity in m/s,
A = pipe section in $mm^2$,
Q = flow in l/s,
d = internal pipe diameter in mm.

→ **the residual pressure must always be positive**

For the water to flow properly in a pipe, the residual pressure should always be positive. If we take the example of picture 12:

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.

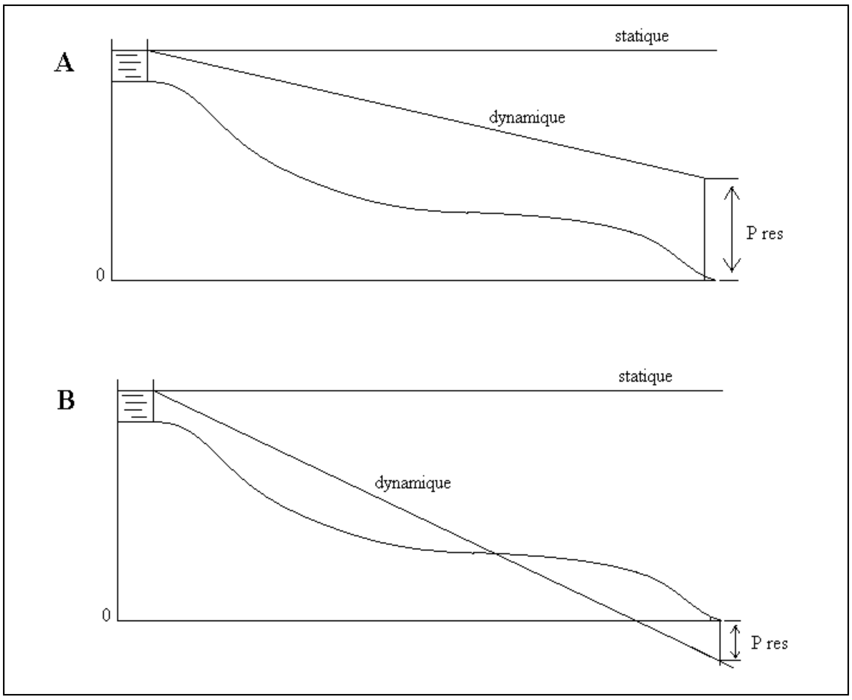



Fig.12: Positive residual pressure (A) and negative (B).

The network residual pressures must also comply with the following rules:

- A minimum of 10 mWG at the level of the inlet of the storage tank and break pressure tank. This minimum can be reduced to 5 mWG for the short networks,
- From 5 to 15 mWG at the level of the taps (5 to 20 for the tapstand equipped with a stop cock valve).

It is possible, by drawing the piezometric lines, to notice that a part of the pipeline present negative residual pressure (see picture 13A). A negative pressure in a section of a network causes problems of air in the network, and facilitates the infiltrations of polluted water inside the pipeline in case of leak. When the pipeline crosses section of high relief, it is recommended to keep a residual pressure higher than 10 mWG.

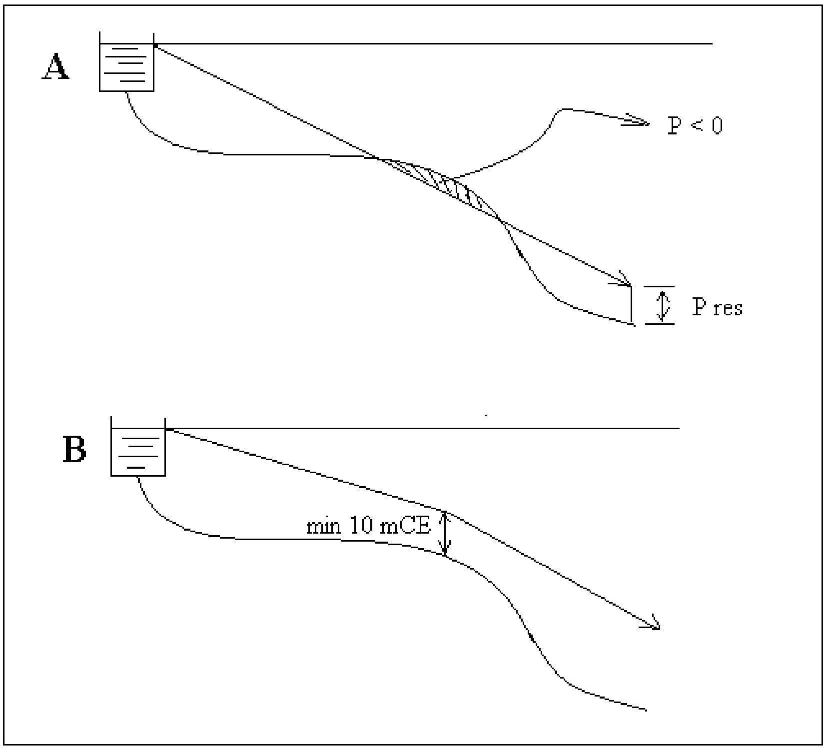



Fig.13: To avoid having a negative residual pressure on a part of a network (case A), it is necessary to change the pipes diameter, even if it means to use different pipe diameter along the same line (case B).

→ **Example of calculation (refer to the topographic graph on the next pages)**

If we take the example of a gravity fed system where the flow is of 0.25 l/s. The available pipes are Wavin PE 32, PE 40 and PE 50, all of NP 16 bars[1].

Section from spring to header tank:
The distance between the spring and the header tank is 30 meters. It is very important to ensure a good and regular water flow in this section in order to avoid putting the spring under pressure. Therefore, a large diameter of pipe should be chosen to avoid any blocking in the pipes (air plug, deposits, etc) and to ensure the good flow of water. The diameter used by ACF for this type of section is always equal to or higher than PE 50. If we calculate the residual pressure and the Total head for this diameter (40.8 mm as internal diameter):
Q = 0.25, L = 30 m and H = 100 – 96.34 = 3.66 m.
f = 0.14% and $\Delta P$ = 0.14 x 30 / 100 = 0.04 m
→ $P_{res} = H + P_{res}(\text{spring}) - \Delta P = 3.66 + 0 - 0.04 = 3.62$ m.

From the header tank to the storage tank:
On this section, we observe the presence of a high point (C1) and of a low point (L1). The topographic profile is called "U". It will thus be necessary to place a wash-out valve at the low point L1 and an air-bleeding valve at the high point C1.
To be sure not to have any problems at the critical point C1, we can start by calculating the residual pressure at this point.

[1] NP 10 would be enough for the given range of pressure, but we consider here that only pipe of NP 16 bars are available

- *Section from the header tank to the point C1.*

  The static level is at 96.34 meters (= altitude of header tank) and the lowest point is at 39.72 meters (= altitude of L1). The maximum static pressure in the pipe is 96.3 – 39.7 = 56.6 meters. We can thus use without problems the NP 16 pipes.

  Q = 0.25, L = Distance (spring to C1) – Distance (spring to header tank) = 669.4 – 30 = 639.4 m and H = (altitude header tank) – altitude (C1) = 96.3 – 89.3 = 7 m.

- If we take a pipe of 26 mm of internal diameter (PE 32):

  f = 1.35% and $\Delta P$ = 1.35 x 639.4 / 100 = 8.63 m

  $\rightarrow P_{res}$ = H + $P_{res}$ (header tank) - $\Delta P$ = 7 +0 – 8.63 = -1.63 m.

  Remark: $P_{res}$ (header tank) = 0 because in the tank the water is at atmospheric pressure

  The residual pressure is negative, i.e. water cannot flow and will not arrive in the storage tank. It is then necessary to choose a larger diameter of pipe.

- If we take a pipe of 32.6 mm of internal diameter (PE 40):

  f = 0.42% and $\Delta P$ = 0.42 x 639.4 / 100 = 2.69 m

  $\rightarrow P_{res}$ = H + $P_{res}$ (header tank) - $\Delta P$ = 7 + 0 – 2.69 = 4.31 m.

  The residual pressure is positive but low. By taking into account the possible mistake made during the topographic survey or the network construction, it is risky to take a diameter with such a low calculated residual pressure (the real residual pressure might be close to 0). It is thus preferable to choose a larger diameter of pipe.

- If we take a pipe of 40.8 mm of internal diameter (PE 50):

  f = 0.19% and $\Delta P$ = 0.19 x 639.4 / 100 = 0.9 m

  $\rightarrow$ Pres = H + $P_{res}$ (header tank) - $\Delta P$ = 7 + 0 – 0.9 = 6.1 m.

  The residual pressure is positive but always lower than the residual pressure recommended for the network's high point (i.e. 10 mWG). However, the water velocity in the pipe is limited (0.19 m/s). By taking a larger diameter pipe, we would increase the residual pressure at the C1 point but at the same time we would decrease the water velocity in pipes. It is thus better to remain with a water velocity of 0.19 m/s. The air-valve and wash-out valve installation at the points L1 and C1 is compulsory to avoid problems of flow in this section.

  **$\rightarrow$ the most suitable diameter for this section is thus PE 50 NP16**

- *Section from C1 to the storage tank*

This section has a steady downward slope, it should not be a problem regarding the flow of water in the pipes.

The static level is 96.34 meters (= header tank) and the lowest point is 21.72 meters (= storage tank). The maximum static pressure in the pipes is 96.34 – 21.72 = 74.62 meters. We can thus use without problems NP 16 pipes.

Q = 0.25 , L = 1254.2 – 669.4 = 584.8 m and H = 89.3 – 21.7 = 67.6 m.

- If we take 26 mm of internal diameter (PE 32):

$f = 1.35\%$ and $\Delta P = 1.35 \times 584.8 / 100 = 7.9$ m.

$P_{res} = H + PC1 - \Delta P = 67.6 + 6.1 - 7.9 = 65.8$ m.

The residual pressure is positive and the water velocity is 0.47 m/s.

→ **the chosen diameter is thus PE 32**

The head losses profile is given on the following page. Three profiles are illustrated there:

- PE32 pipes all along the layout,
- PE40 pipes (section from header tank to C1) then PE 32 NP 16 pipe (section from C1-to storage tank),
- PE 50 pipe (section header tank to C1) then PE 32 pipe (section C1 to storage tank)

First try: PE32 pipes all along the layout. This is represented by the lower pink line on the graph. We can see that the line pass below C1, so water will not reach C1

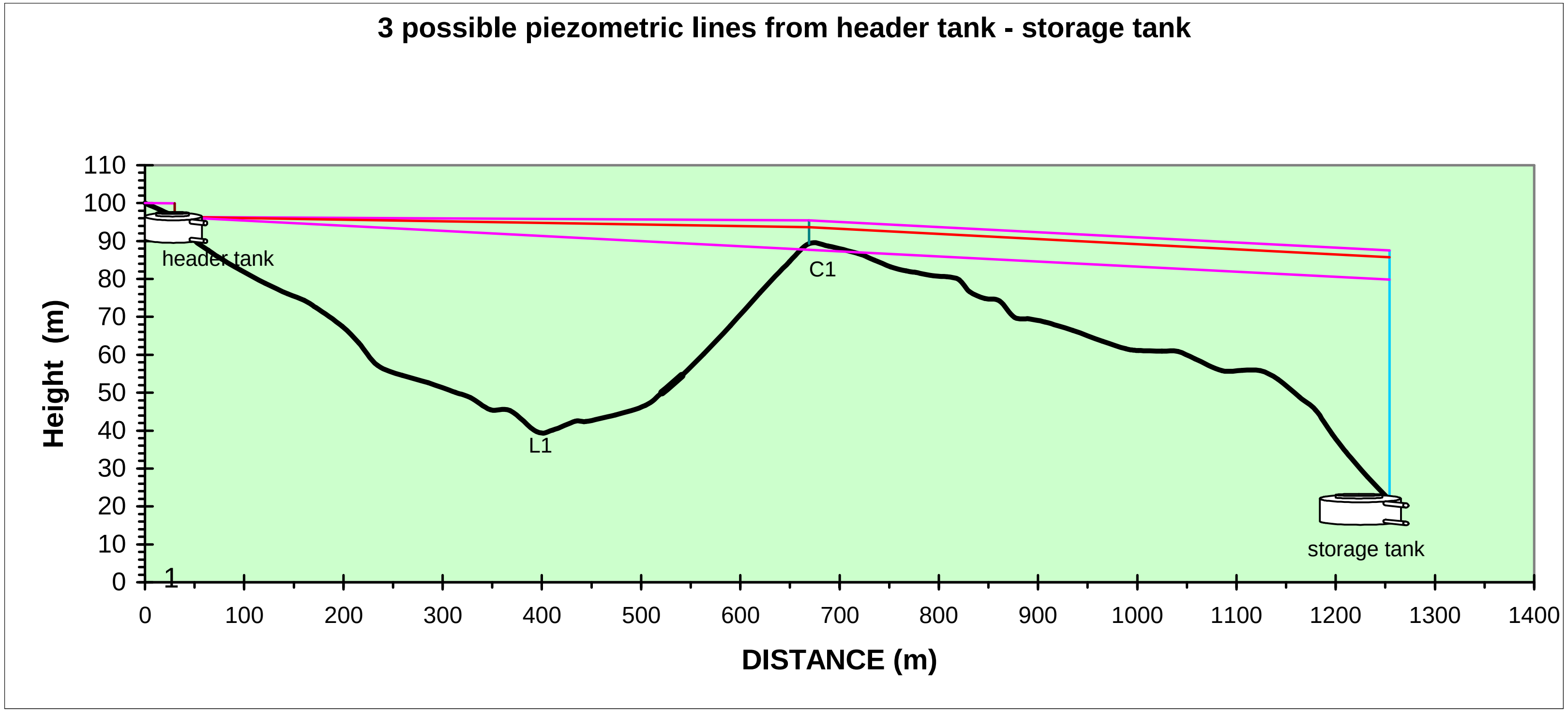
| From | To | Dist 1 (m) | Dist 2 (m) | Alti 1 (m) | Alti 2 (m) | Dist. between 1 & 2 = L (m) | Difference of altitude between 1 & 2 = H (m) | Flow (l/s) | Internal Diam. (mm) | Friction Factor % | Head loss (m) | P1 (m) | P2 (m) | Velocity (m/s) | Diameter of pipe (NP 10) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spring | Header T | 0.00 | 30.00 | 100 | 96.3 | 30 | 3.66 | 0.25 | 40.8 | 0.14 | 0.04 | 0.00 | 3.62 | 0.19 | PE 50 |
| Header T | C1 | 30.00 | 669.40 | 96.3 | 89.3 | 639.4 | 7 | 0.25 | 26 | 1.35 | 8.63 | 0.00 | -1.63 | 0.47 | PE 32 |

Second try: PE 40 pipes (section from header tank to C1) then PE 32 NP 16 pipe (section from C1-to storage tank). This is represented by the middle orange line on the graph. The line pass above C1, but not high enough

| From | To | Dist 1 (m) | Dist 2 (m) | Alti 1 (m) | Alti 2 (m) | Dist. between 1 & 2 = L (m) | Difference of altitude between 1 & 2 = H (m) | Flow (l/s) | Internal Diam. (mm) | Friction Factor % | Head loss (m) | P1 (m) | P2 (m) | Velocity (m/s) | Diameter of pipe (NP 10) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spring | Header T | 0.00 | 30.00 | 100 | 96.3 | 30 | 3.66 | 0.25 | 40.8 | 0.14 | 0.04 | 0.00 | 3.62 | 0.19 | PE 50 |
| Header T | C1 | 30.00 | 669.40 | 96.3 | 89.3 | 639.4 | 7 | 0.25 | 32.6 | 0.42 | 2.69 | 0.00 | 4.31 | 0.30 | PE 40 |

Third try: PE 50 pipe (section header tank to C1) then PE 32 pipe (section C1 to storage tank). This is represented by the top pink line. This is the solution chosen

| From | To | Dist 1 (m) | Dist 2 (m) | Alti 1 (m) | Alti 2 (m) | Dist. between 1 & 2 = L (m) | Difference of altitude between 1 & 2 = H (m) | Flow (l/s) | Internal Diam. (mm) | Friction Factor % | Head loss (m) | P1 (m) | P2 (m) | Velocity (m/s) | Diameter of pipe (NP 10) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spring | Header T | 0.00 | 30.00 | 100 | 96.3 | 30 | 3.66 | 0.25 | 40.8 | 0.14 | 0.04 | 0.00 | 3.62 | 0.19 | PE 50 |
| Header T | C1 | 30.00 | 669.40 | 96.3 | 89.3 | 639.4 | 7 | 0.25 | 40.8 | 0.19 | 2.69 | 0.00 | 6.10 | 0.19 | PE 50 |
| C1 | Storage T | 669.40 | 1254.20 | 89.3 | 21.7 | 1254.2 | 67.6 | 0.25 | 26 | 1.35 | 7.90 | 6.10 | 65.80 | 0.47 | PE 32 |


3 possible piezometric lines from header tank - storage tank
Height (m)
DISTANCE (m)
header tank
storage tank
L1
C1
0
10
20
30
40
50
60
70
80
90
100
110
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1

## III.5. PLANNING AND DESIGNING THE VALVE NETWORK

A certain number of valves must be installed on the network. They have various roles, among which can be distinguish two main type of valves:

- the control valves
- the open/close gatevalves type

The valves should always be located in valve boxes which, while protecting them, can be easily accessible.

### III.5.1. Regulation valves

To guarantee the good network operation, it is necessary to respect the flows established during the sizing of the network. For that, control valves should be installed at the entry of each tapstand, storage tank and break pressure tank.

→ *Adjustable stock cock valve*

These control valves give the possibility to regulate the flow by adjusting their maximum opening. Their use is necessary when the spring flow is higher than the desired flow in pipe: we prevent that a more important flow than the one required enter in the pipe, which could lead to formation of air plug.

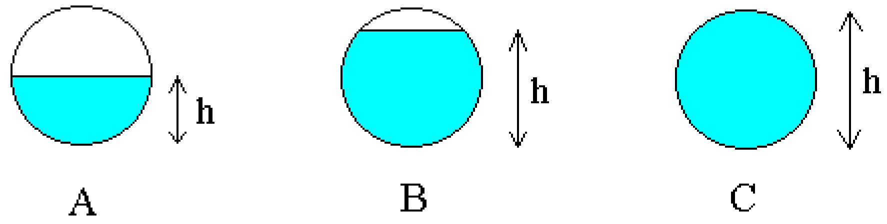
### III.5.2. The open/close valves type

Open/close valves type should be installed to isolate the various sections of the network from each other. Their installation gives the possibility to stop the supply in certain zones in case of leak or maintenance work. These valves must be installed at each important junctions. In the same way, open/close wash-out valves are installed on all the storage tanks, break pressure tank, header tank and lower points of the network.
These valves remain most of the time opened or closed according to their role: the valves whose purpose is to isolate a part of the network in case of maintenance remains normally open, while wash-out valve remains normally closed.

## III.6. HOW TO AVOID THE USUAL PROBLEMS PRESENTED BY GRAVITY FED SYSTEM?

### III.6.1. How to avoid the sometimes violent shocks in pipes?

If large diameter pipes are chosen, it can happen that the pipes are not completely full of water during the flow. Water can hence presents a free surface inside the pipes in some sections (free surface flow), while in other sections water will filled completely the pipe (=pressure flow).
It should be known that transition between a free surface flow and a pressurized flow is problematic:




A: Free surface flow. If the flow increases, **(h)** increases and friction surfaces of water against the pipe walls increase too.

B: Starting from a certain limit, if the flow continues to increase, **h** increases slower than the frictions on pipe walls. The frictions limit the water flow and the flow which can pass through the pipe decreases, causing shocks upstream and downstream. The pressure in the pipe is null.

C: If we continue to increase the flow, the top wall of the pipe is reached → the pressure increases in the pipe and allows increasing the flow again.

A free surface flow in certain network sections and a pressure flow in others thus cause sometimes violent shocks in pipes. It is then important to size as properly as possible the pipes (and avoid over-sizing pipes diameters) and to control the flow if necessary.

### III.6.2. How to avoid having air in pipes?

Water closed in a pipe, even if under pressure, produces air. All along its way, water passes by various pressure conditions that causes degasification. The formed bubbles move together with the water, but at the same time they are pushed to the top because of their low density (the air bubbles are lighter than water and always go upwards). The imprisoned air creates an air plug, forming a barrier to the water flow. This phenomenon especially occurs at the network first filling and commissioning.

Generally, there is air plug formation if:
- the pipe layout forms a reversed "U" profile,
- the highest point of the pipe is at least 10 meters lower than the static piezometric level (see picture 14).

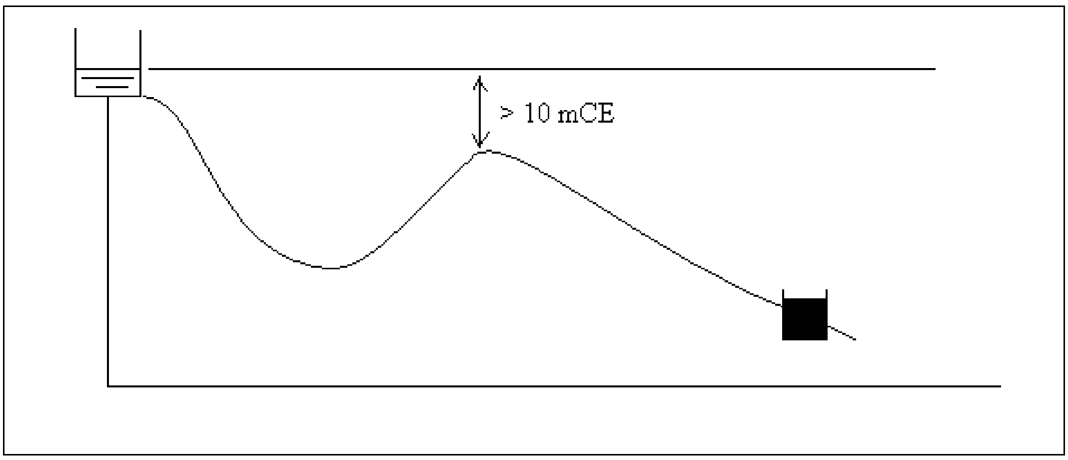



Fig.14: Profile possibly leading to the formation of air plug.

To prevent the formation of air plug in the pipes, it is necessary to:

- Avoid the high points when that is possible, even if it means to increase the total length of the pipeline.
- Avoid mixed operations, free surface/ pressure flow, which are often producing air.
- Locate the high points and install a degasification device: air-bleeding valve, break pressure tank or storage tank.
- In the case of a very uniform slope section, accentuate the high points, to avoid flat layouts where high and low points do not appear clearly. For the flat profiles, accentuate the slope located downstream of the air valve (see picture 15).
- Size properly the pipes:
  - Choose diameters which minimize the head losses between the spring and the first location where the formation of air plug are possible.
  - Use larger diameters pipe at the high point of the network (to facilitate the evacuation of the air bubbles) and smaller diameters at the lower points of critical sections such as section B-C and D-E of picture 16 (to increase the velocity of water and avoid sedimentation)

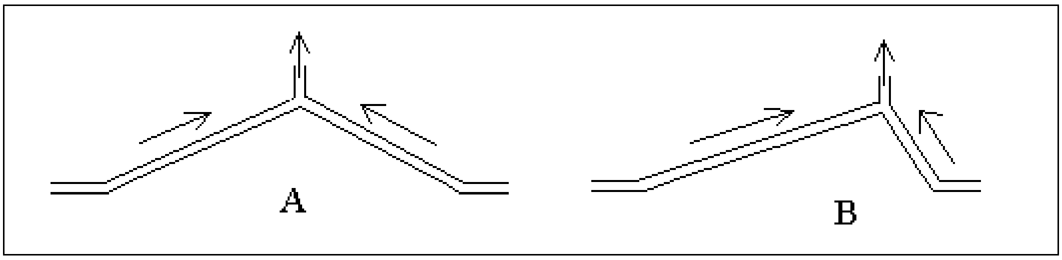



Fig.15: Profile to be avoided (A) and profile to implement (B) for an air-bleeding valve installation: in the A case, the coming up (= evacuation) of the air bubbles located downstream of the air-bleeding valve is difficult; while for the case B, the coming up of the air bubbles is facilitated.

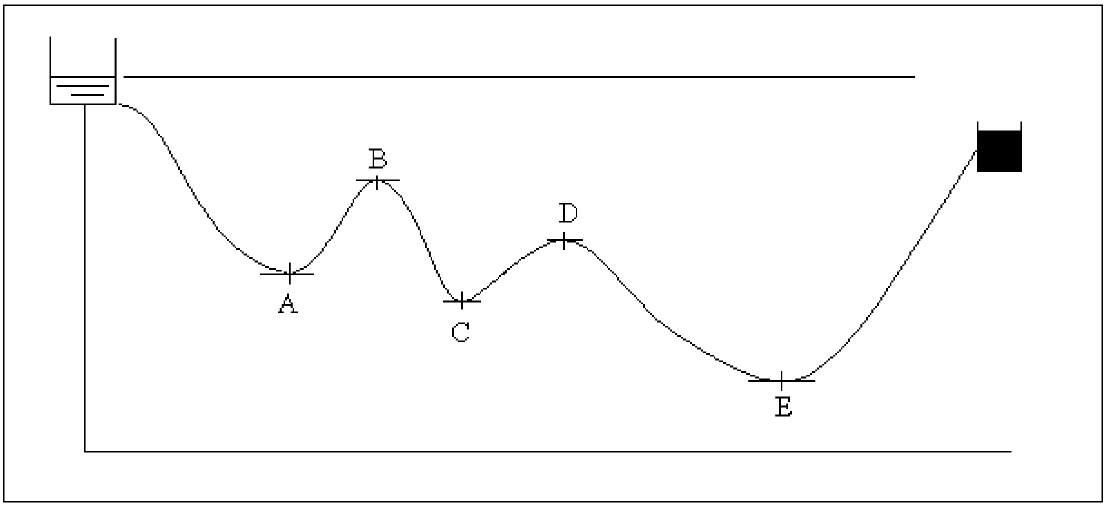



Fig.16: Example of profile with air plug formation (points B and D) and sedimentation in pipe (points A, C and E).

→ **air-bleeding**

To avoid the problem of flow blockage in pipes due to the creation of air plugs, air-bleeding devices can be installed at the network highest points (points B and D of picture 16). These devices allow the evacuation of the air accumulates at this point.

Manual air draining:
We install ball valve at the highest point of the pipeline (see picture 17). These valves must be opened at the time of network filling/commissioning, then opened regularly at each network inspection.

Automatic air-vents:
Specific valves allow evacuating automatically the air accumulated in pipes (see picture 18). Located at the highest points, they facilitate the maintenance. However, they are usually not used for GFS in rural area, because of the impossibility for the villagers to repair or maintain them

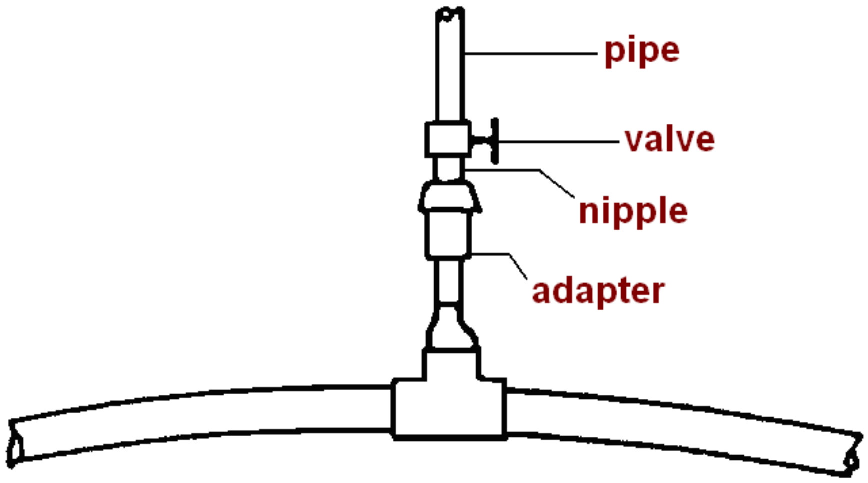



Fig.17: Air-bleeding valve (ball valve can be used).

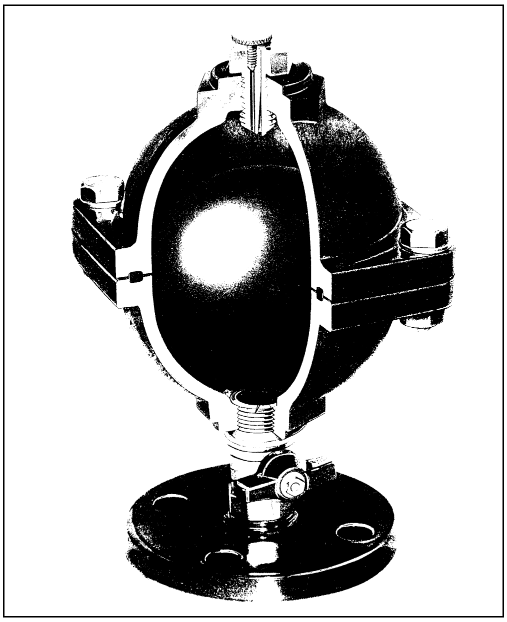


Fig. 18: Automatic air-vents

### III.6.3. How to avoid the deposits in pipes?

Water collected from the spring can contain silt and sands, especially for the springs where the flow varies a lot. We often observe a sedimentation of these solid materials on the lowest points of the pipeline. The lowest points of the profiles in "U" are the most prone to this phenomenon (points A, C and E of figure 16). Sediments in pipe lead to an increase of headlosses and a reduction of the flow of the network.

To avoid this problem, especially in the case of long pipelines, three precautions must be taken:

- Install a header tank at the beginning of the network where the largest particles will sedimentate (sands…),
- Size correctly the network by respecting the minimal velocity of 0.3 m/s,
- Install large diameter wash-out valves (ball valves can be used) in the network's lowest points, to be able to drain the pipes and to evacuate the possible deposits (see picture 19)

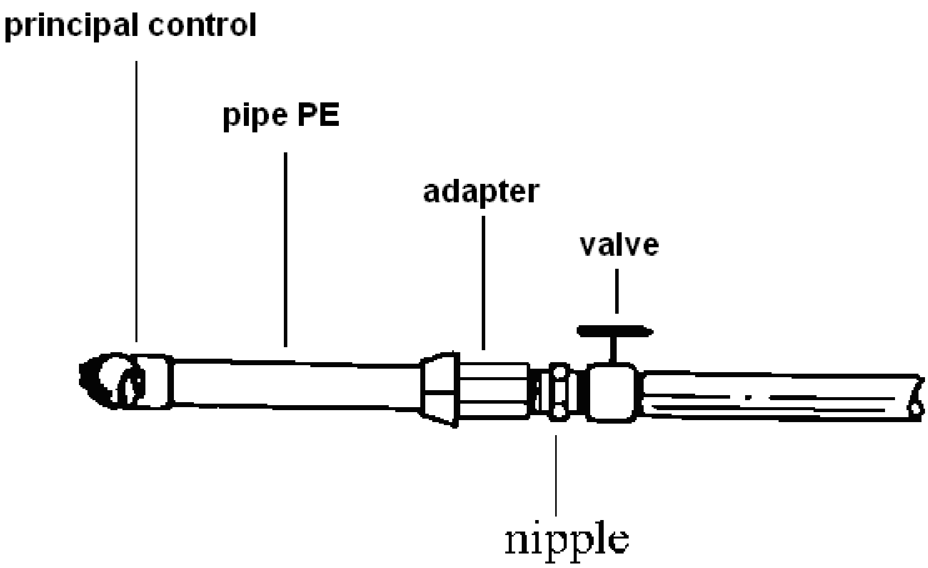



Fig.19: wash-out valve (ball valve)

### III.6.4. How to avoid water hammers?

Water hammers are overpressure and depressions waves caused by the abrupt change of water flow in a pipe.
Let's consider the example of picture 20. To block abruptly the water flow at point (A) by the mean of a valve generates overpressure waves (B) and depression waves (C) which are propagated in the pipe while slowly decreasing in their intensity.

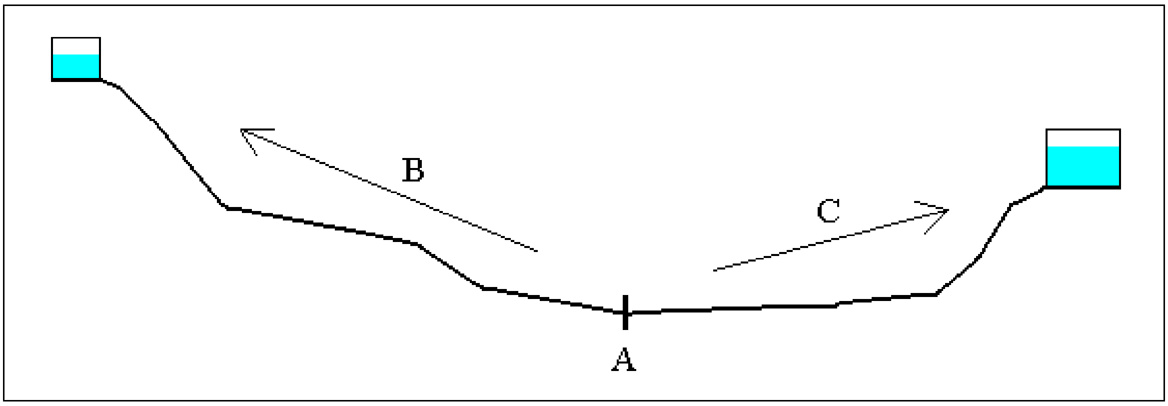



Fig.20: Water hammer illustration caused by the abrupt valve closing placed at point A

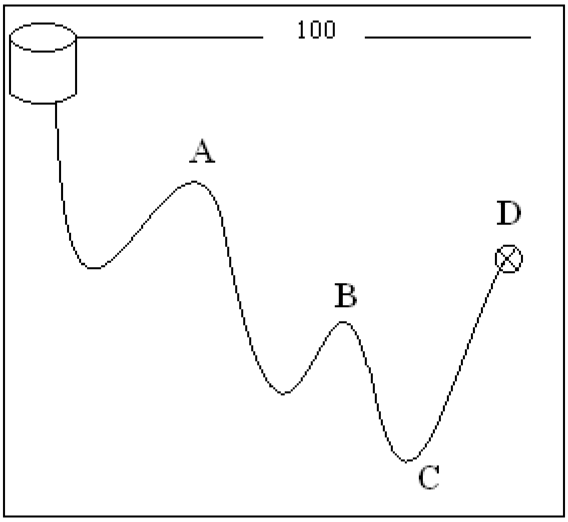
It is strongly recommended to avoid water hammers which can cause extremely important damages, sometimes long time after the action. For the network in rural areas, installation of anti-water hammer equipment is expensive and hardly justified. However, preventive measures can be taken:

- Avoid using simple plug as wash-out valve
- Sensitize the beneficiaries and technicians responsible of the gravity fed system maintenance to the risks of water hammers related to abrupt operations: valves should always be opened or closed very slowly.

## III.7. CALCULATION EXERCISES

### III.7.1. Exercise 1

The pipe is filled with water and the point D is closed. What is the pressure at the points A, B, C and D?




Altitude of A is 53 m.
Altitude of B is 28 m.
Altitude of C is 15 m.
Altitude of D is 38 m.

### III.7.2. Exercise 2

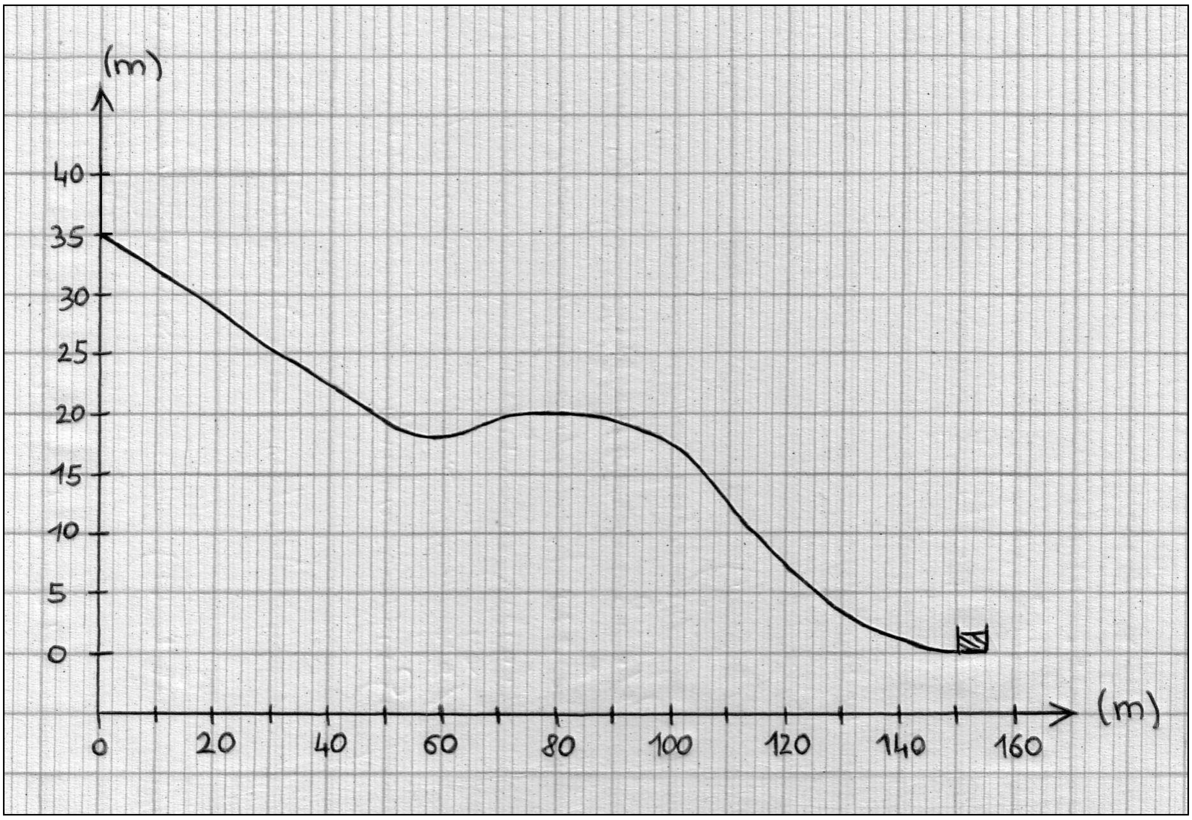
Calculate the head losses for a section going from the header tank to the storage tank. The pipes length is 1320m and the flow 0.75 l/s. The pipes are PE 32 NP 16.

### III.7.3. Exercise 3

A storage tank is located at 50 meters height and the header tank is at 135 meters height. The distance between the two is 920 meters and the water flow is 0.45 l/s. The pipes used are PE 32 NP 16. What is the residual pressure at the level of the storage tank?

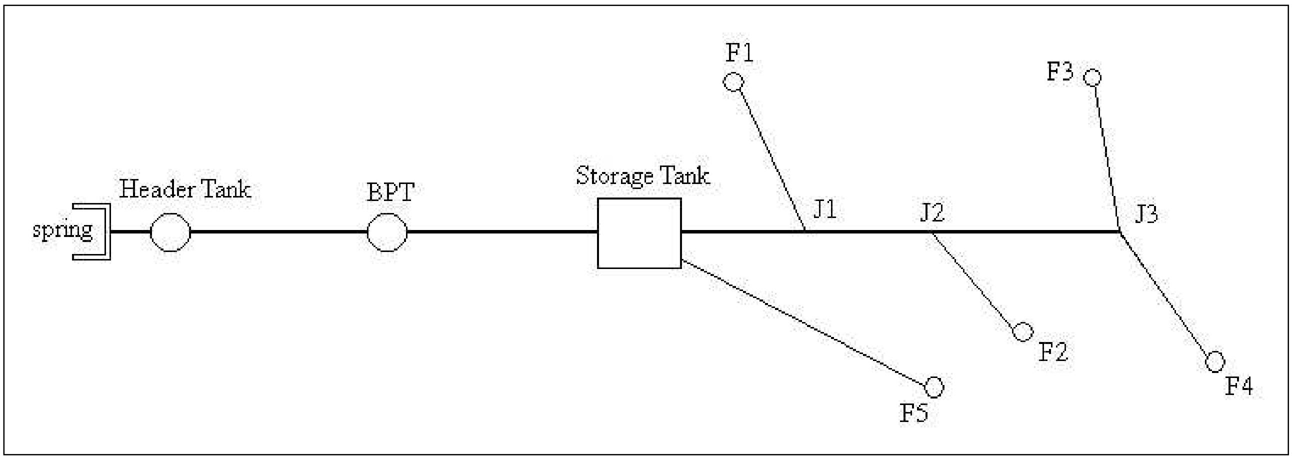
### III.7.4. Exercise 4

Determine the pipe size and draw the dynamic piezometric line of the sections given below (header tank to storage tank). The available pipe diameters are 32, 40, 50 and 63 NP 16. The flow is 1.2 l/s.



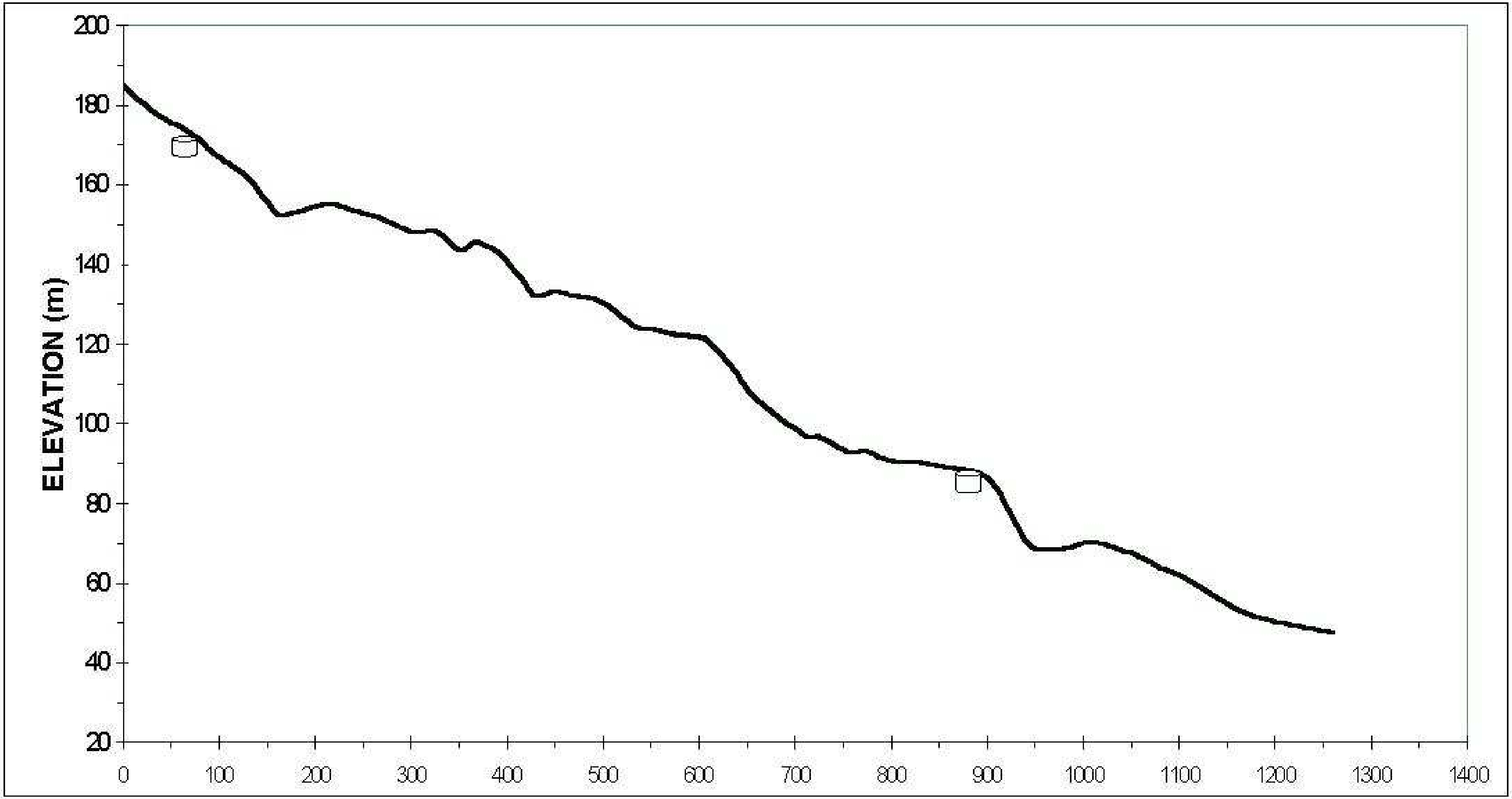

## III.7.4. Exercise 5

Determine the pipes diameter and draw the dynamic piezometric line of the entire network.
The available pipes diameters are 32, 40, 50 and 63, NP 16.
The network layout is as follows:

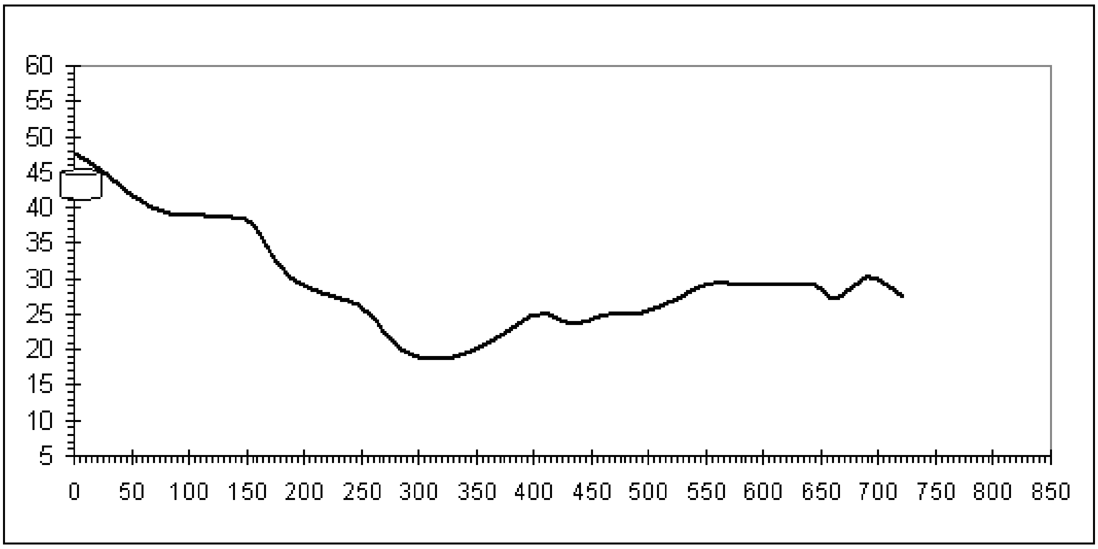



The topographic data all along the pipeline is given in the following pages.

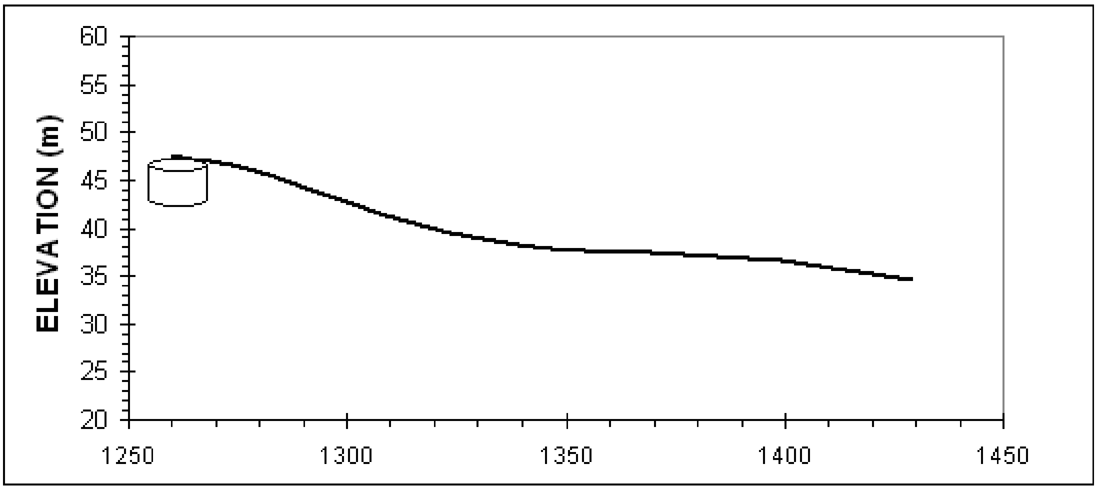
Topographic profile from the spring catchments to the storage tank:



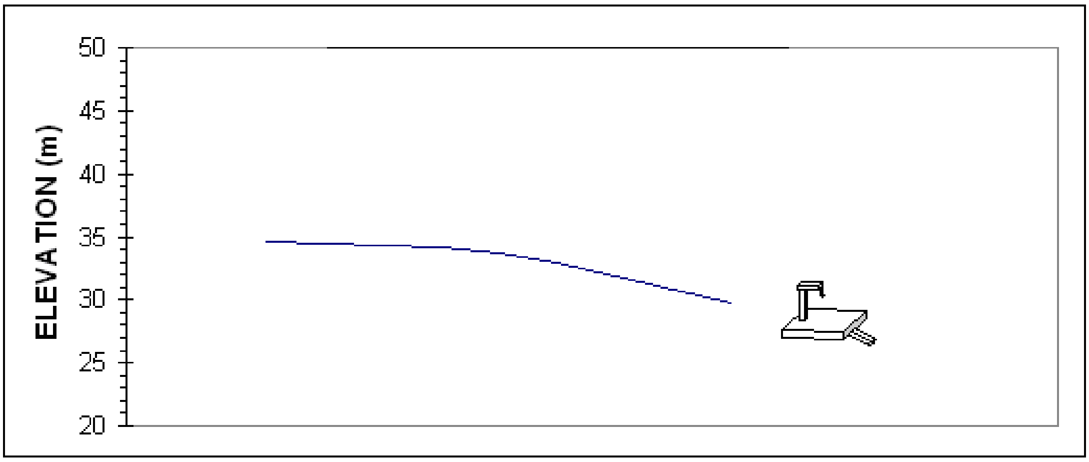
Topographic profile from the storage tank to the tapstand F5:



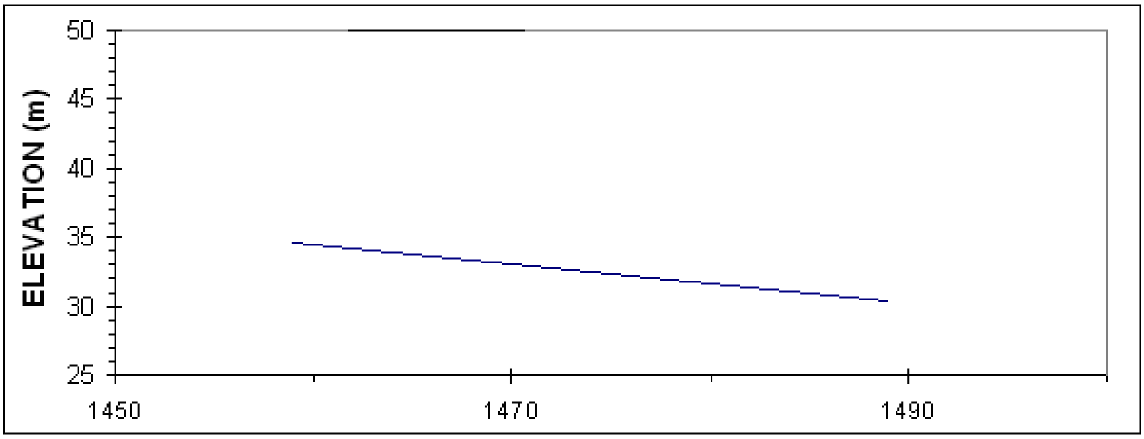

Topographic profile from the storage tank to the junction J1:



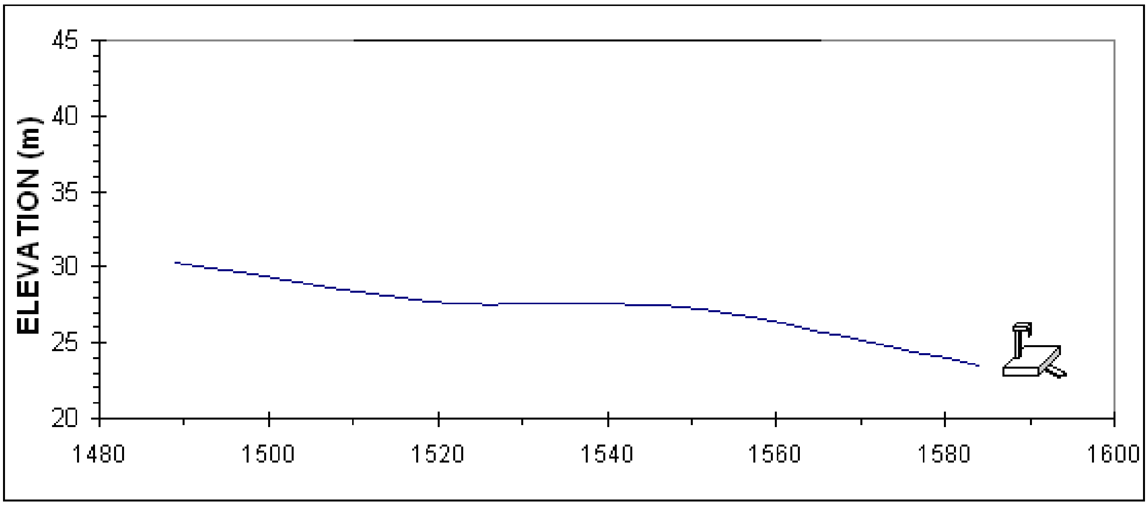
Topographic profile from the junction J1 to the tapstand F2:



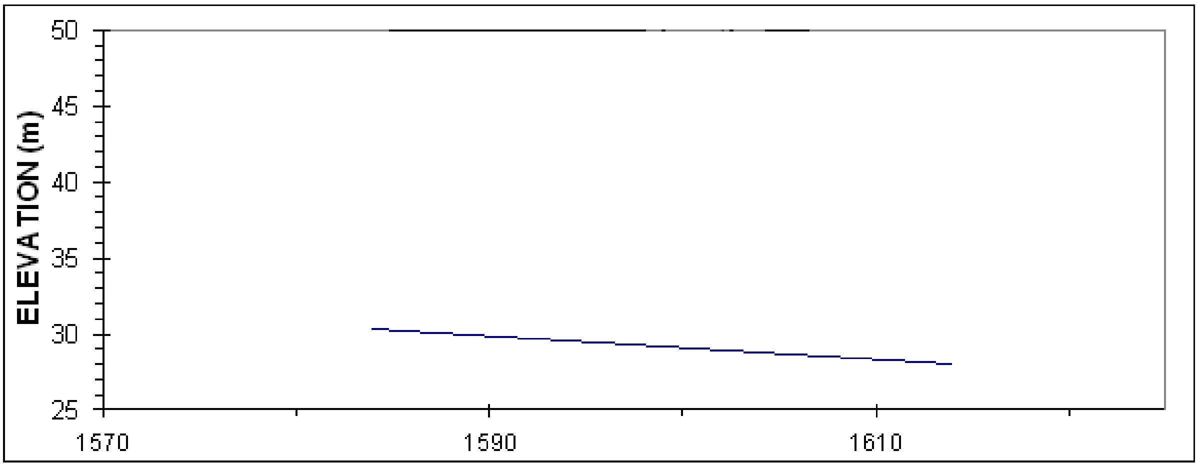

Topographic profile from the junction J1 to the junction J2:



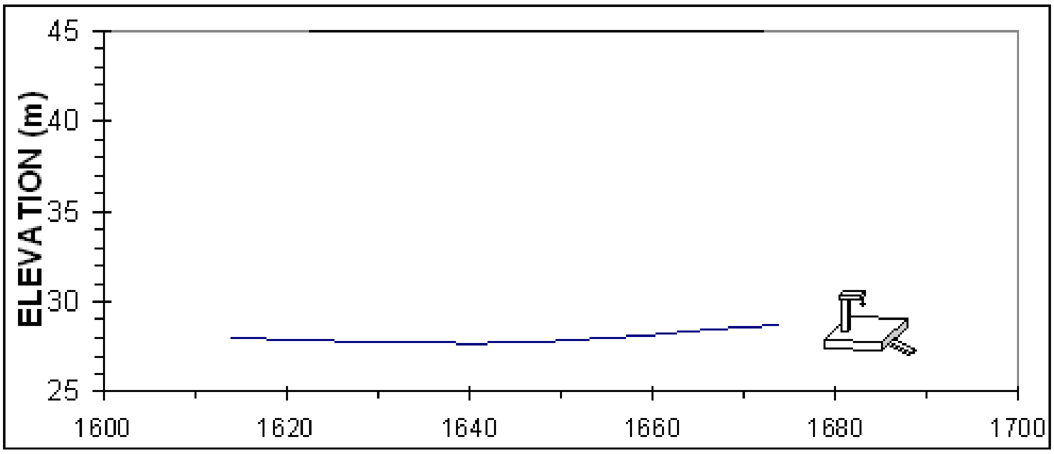

Topographic profile from the junction J2  to the tapstand F2:



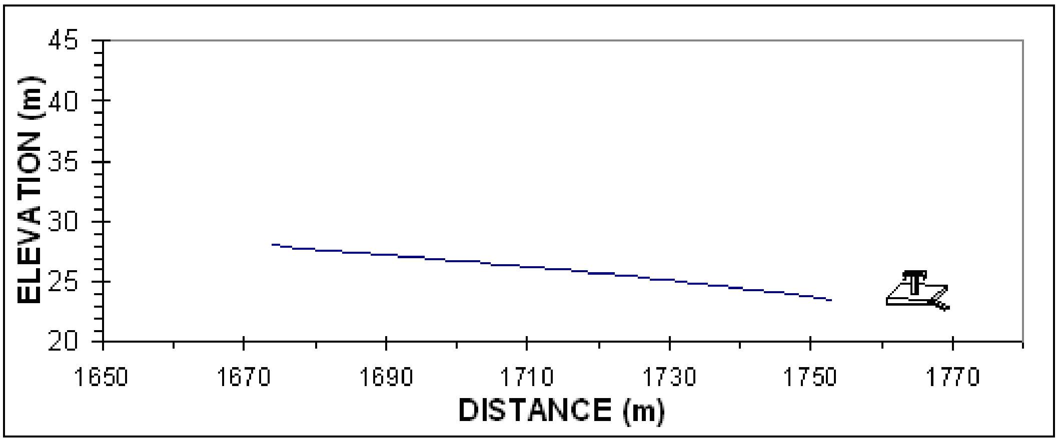

Topographic profile from the J2 junction to the junction J3:




Topographic profile from the junction J3 to the tapstand F3:



Topographic profile from the junction J3 to the tapstand F4:




Make the calculation for the following sections:

1. From catchment to the header tank. The distance is 67 meters and the flow is 0.45 l/s. The altitude of the catchments is 185 meters and the altitude of the header tank is 174 meters.

2. From header tank to the break pressure tank. The flow is 0.45 l/s, the distance is 813 meters, the altitude of the break pressure tank is 89 meters.

3. From break pressure tank to the storage tank. The flow is 0.45 l/s, the distance is 380 meters and the tank storage altitude is 48 meters.

4. From the storage tank to the tapstands:
   - 4.1. From storage tank to the tapstand F5. The flow is 0.2 l/s, the distance is 722 meters and the tapstand altitude is 28 meters.
   - 4.2. From storage tank to the junction J1. The flow is 1.25 l/s, the distance is 169 meters and the junction altitude is 35 meters.
   - 4.3. From junction J1 to the tapstand F1. The flow is 0.25 l/s, the distance is 30 meters and the stand-pipe height is 30 meters.
   - 4.4. From junction J1 to the junction J2. The flow is 1 l/s, the distance is 30 meters and the J2 junction altitude is 30 meters.
   - 4.5. From junction J2 to the tapstand F2. The flow is 0.25 l/s, the distance is 95 meters and the stand-pipe altitude is 24 meters.
   - 4.6. From junction J2 to the junction J3. The flow is 0.75 l/s, the distance is 30 meters and the J3 junction altitude is 28 meters.
   - 4.7. From junction J3 to the tapstand F3. The flow is 0.25 l/s, the distance is 60 meters and tapstand altitude is 29 meters.
   - 4.8. From junction J3 on the tapstand F4. The debit is 0.5 l/s, the distance is 79 meters and the tapstand altitude is 24 meters.

# IV. APPENDIXES

**Appendix 1: Solution of the exercises**

**Appendix 2: Nomograph PE and GI pipes**

## APPENDIX 1:
## Solutions of the exercises

**Exercise 1:**

- Pressure at point A = 100 – 53 = 47 meters.
- Pressure at point B = 100 – 28 = 72 meters.
- Pressure at point C = 100 – 15 = 85 meters.
- Pressure at point D = 100 – 38 = 62 meters.

**Exercise 2:**

Q = 0.75 l/s and d = 32mm (= 26 mm internal).
f = 9, 4%
$\Delta P = L \times f / 100 = 1320 \times 9.4 / 100 = 124.08$ meters.

**Exercise 3:**

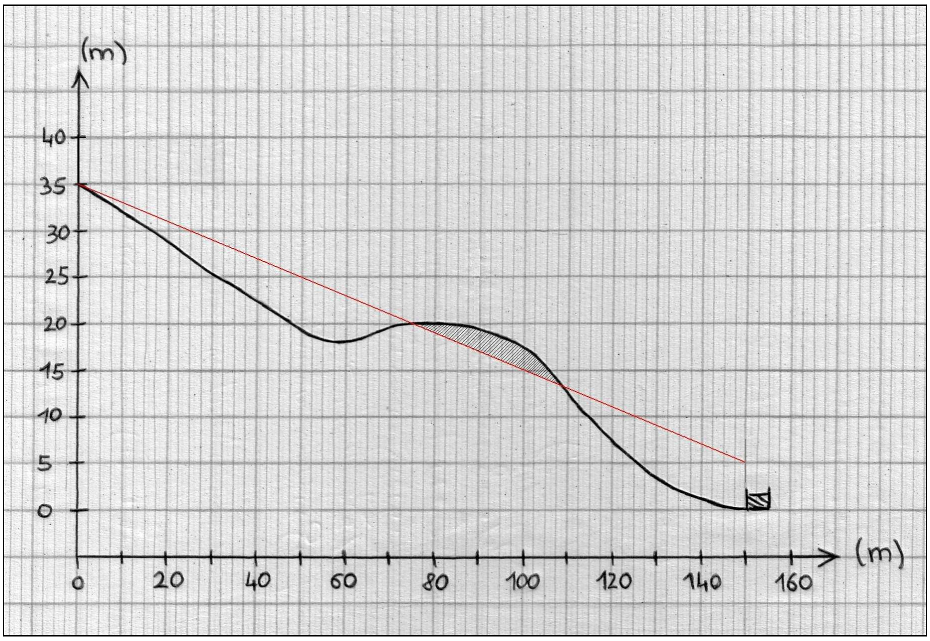
Q = 0, 45 l/s and d = 32mm (= 26mm internal).
f = 3.9%
$\Delta P = L \times f / 100 = 920 \times 3.9 / 100 = 35.88$ meters.
H = 135 – 50 = 85 meters.
$P_{res} = H - \Delta P = 85 - 35.88 = 49.12$ meters.

**Exercise 4:**

Q = 1.2 l/s, L = 150 m and H = 35 m.

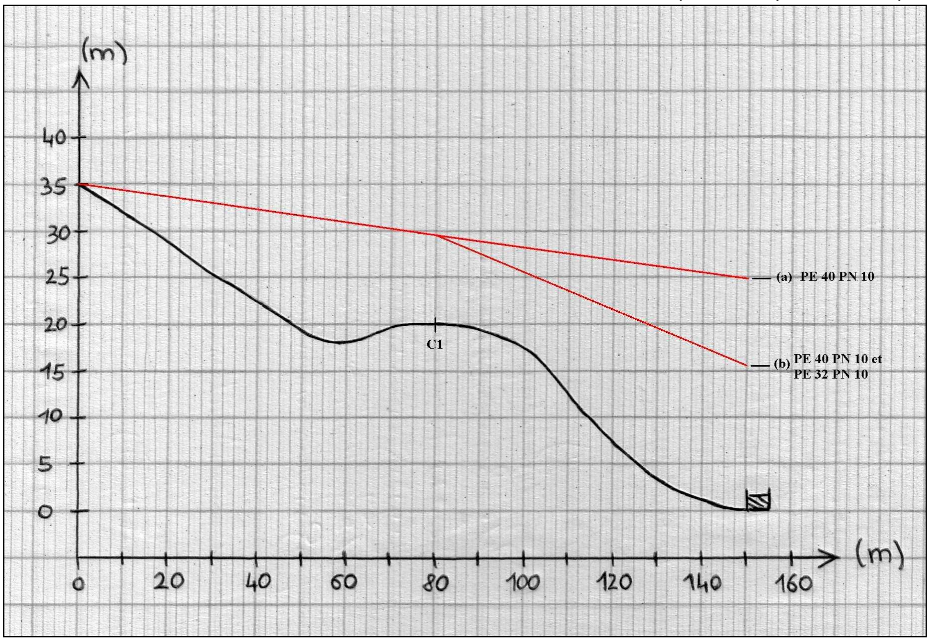
- If we take PE 32 (26 mm of internal diameter):
  F = 20% and $\Delta P = 20 \times 150 / 100 = 30$ m $\rightarrow P_{res} = H - \Delta P = 35 - 30 = 5$ m.
  The residual pressure at the tank level is acceptable although slightly low.
  Velocity is 2.4 m/s.

If we see the dynamic profile layout:




Network part has negative pressure → the pipe PE 32 is not acceptable.

- If we take PE 40 (32.6 mm of internal diameter):
  f = 7% and $\Delta P = 7 \times 150 / 100 = 10.5$ m → $P_{res} = H - \Delta P = 35 - 10.5 = 24.5$ m.
  The residual pressure of tank level is acceptable (even if quite high) and velocity is 1.45 m/s, which is within the limits recommended. If we see the dynamic profile layout (line a):



We observe that the pressures in pipes are always positive. The residual pressure at the high point C1 = 15 – (7 X 80/100) = 15 – 5.6 = 9.4 m. The residual pressure to the C1 point is close to 10 mWG: the limit advised for relief passages is thus respected.
We can choose pipes level PE 40 for the whole pipeline, but to optimize the network cost and to decrease the $P_{res}$ to the tank entry, we can also choose a smaller diameter pipe on the C1-tank section.

- If we take PE 32 (26 mm of internal diameter) for the C1-tank section:
  L = 150 – 80 = 70 m, H = 20 – 0 = 20 m and Pres at point C1 = 9.4 m.
  f = 20% and $\Delta P = 20 \times 70 / 100 = 14$ m → $P_{res} = H + P_{res}C1 - \Delta P = 20 + 9.4 - 14 = 15.4$ m.

  The residual pressure on the tank level is acceptable and velocity is 2.3 m/s, which is within the limits recommended. The dynamic profile (line b) shows that the pressure in pipes is always positive.

**Exercise 5:**

1. Q = 0.45 l/s, L = 67 m.
   H = 185 – 174 = 11 m.
   - If we take 26mm of internal diameter (PE 32):
   f = 3.75% and $\Delta P = 3.75 \times 67 / 100 = 2.5$ m → $P_{res} = H - \Delta P = 11 - 2.5 = 8.5$ m.

A pipe diameter of PE 32 could be appropriate. However, for the section between the spring and the break pressure tank, we prefer to choose a large diameter pipe in order to be sure to avoid putting the spring under pressure. We thus choose PE 50.
If we calculate the residual pressure and the Total head for this diameter (40.8mm of internal diameter):

f = 0.4% and $\Delta P = 0.4 \times 67 / 100 = 0.3$ m → $P_{res} = H - \Delta P = 11 - 0.3 = 10.7$ m.
Total head is = Altitude + $P_{res}$ = 174 + 10.7 = 184.7 m.

2. Q = 0.45 l/s, L = 813 m.
   H = 174 – 89 = 85 m.

   - If we take 26mm of internal diameter:
   f = 3.75% and $\Delta P = 3.75 \times 813 / 100 = 30.5$ m → $P_{res} = H - \Delta P = 85 - 30.5 = 54.5$ m.

The piezometric layout shows that the pipes pressures are always positive and velocity limits are respected (V = 0.9 m/s). We thus choose pipe levels of: **PE 32.**
Total head is = Altitude + $P_{res}$ = 89 + 54.5 = 143.5 m.

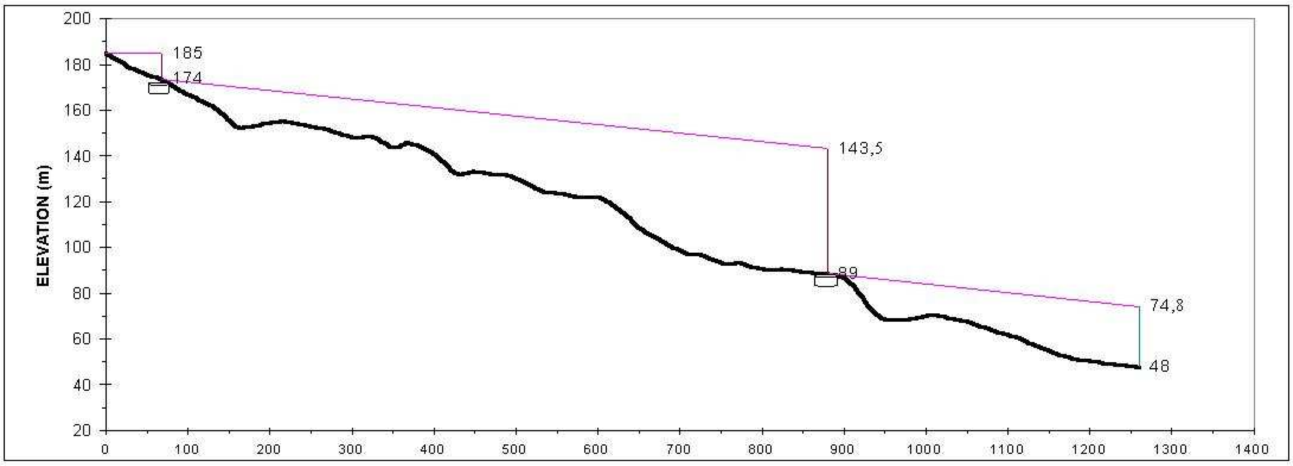
3. Q = 0.45 l/s, L = 380 m.
   H = 89 – 48 = 41 m.

   - If we take 26 mm of internal diameter:

$f$ = 3.75% and $\Delta P$ = 3.75 x 380 / 100 = 14.25 m → $P_{res}$ = H - $\Delta P$ = 41 – 14.25 = 26.8 m.

The piezometric layout shows that the pipes pressures are always positive and velocity limits are respected (V = 0.9 m/s). We thus choose pipe levels of: **PE 32.**
Total head is = 48 + 26.8 = 74.8 m.

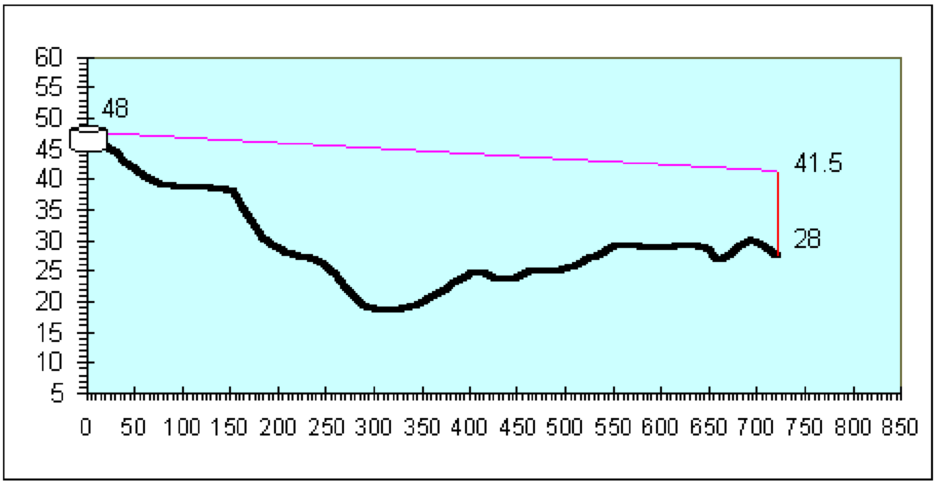
The head losses profile for section going to the tank spring is given to the following page.




4.1. Q = 0.2 l/s, L = 722 m.
H = 48 – 28 = 20 m.

- If we take 26 mm of internal diameter:
$f$ = 0.9% and $\Delta P$ = 0.9 x 722 / 100 = 6.5 m → $P_{res}$ = H - $\Delta P$ = 20 – 6.5 = 13.5 m.

The piezometric layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 0.4 m/s). We thus choose like pipe levels of: **PE 32.**
Total head = Altitude + $P_{res}$ = 28 + 13.5 = 41.5 m.




4.2. Q = 1.25 l/s, L = 169 m.

H = 48 – 35 = 13 m.

- If we take 26 mm of internal diameter:

  f = 22.5% and $\Delta P$ = 22.5 x 169 / 100 = 38.0 m → $P_{res}$ = H - $\Delta P$ = 13 – = -25 m.

  The residual pressure is negative → pipe of PE is not acceptable.

- If we take 32.6 mm of internal diameter:

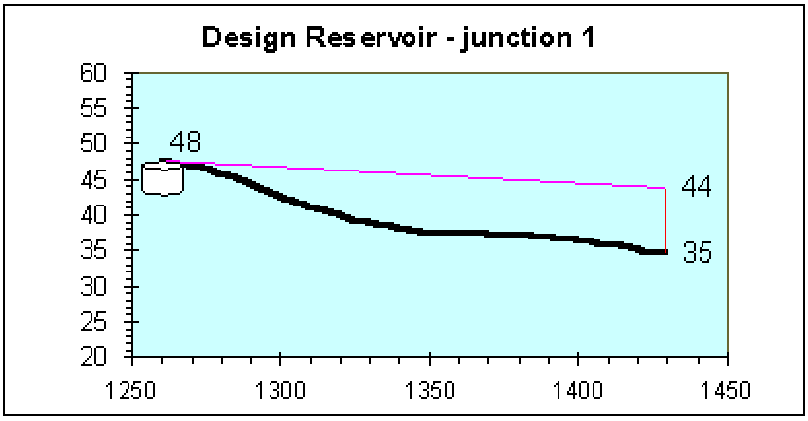
  f = 7.5% and $\Delta P$ = 7.5 x 169 / 100 = 12.7 m → $P_{res}$ = H - $\Delta P$ = 13 – 12.7 = 0.3 m.

  The residual pressure is too low → take 40.8 mm of internal diameter:

  f = 2.5% and $\Delta P$ = 2.5 x 169 / 100 = 4.2 m → $P_{res}$ = H - $\Delta P$ = 13 – 4.2 = 8.8 m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 0.95 m/s). We thus choose pipe levels of: **PE 50**.

Total head is = 35 + 8.8 = 43.8 m.



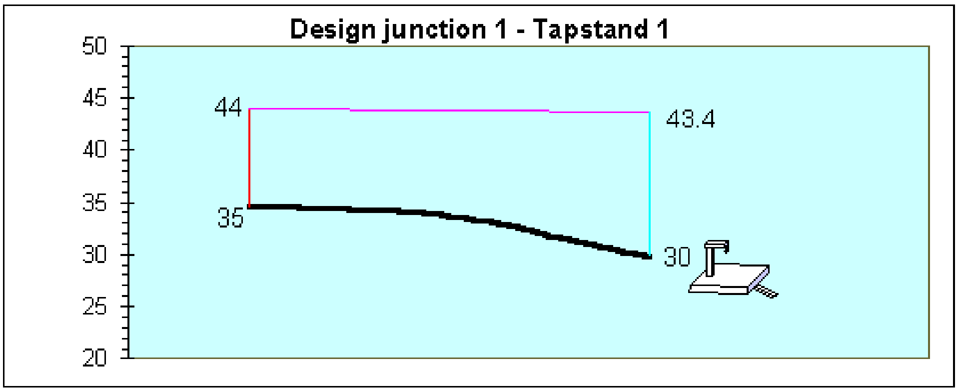

4.3. Q = 0.25 l/s, L = 30 m.

H = 35 – 30 = 5 m.

- If we take 26 mm of internal diameter:

f = 1.3% and $\Delta P$ = 1.3 x 30 / 100 = 0.4 m → $P_{res}$ = H + $P_{res}$J1- $\Delta P$ = 5 + 8.8 – 0.4 = 13.4 m.

The piezometric layout shows that the pressure in the pipe is always positive and the velocity limits are respected (V = 0.5 m/s). We thus choose pipe levels of: **PE 32.**
Total head = 30 + 13.4 = 43.4 m.



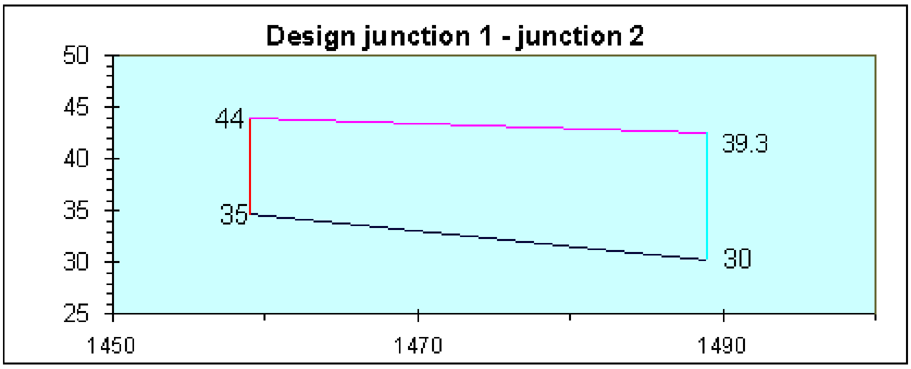

4.4. Q = 1 l/s, L = 30 m.

H = 35 – 30 = 5 m.

- If we take 26 mm of internal diameter:

f = 15% and $\Delta P = 15 \times 30 / 100 = 4.5$ m $\rightarrow P_{res} = H + P_{res}J1 - \Delta P = 5 + 8.8 - 4.5 = 9.3$ m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 1.95 m/s). We thus choose pipe levels of: **PE 32.**
Total head = 30 + 9.3 = 39.3 m.



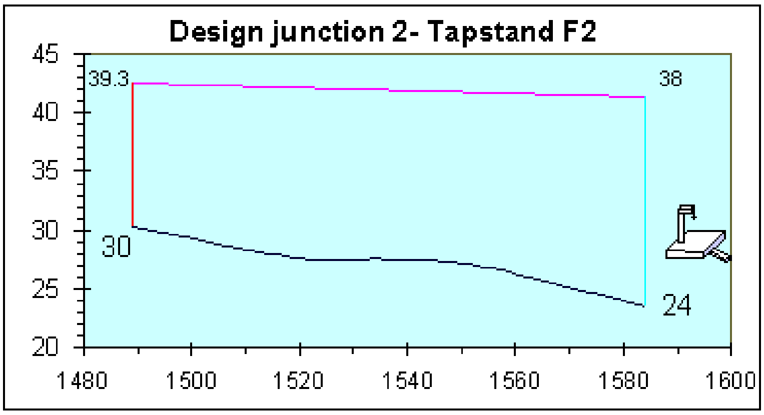

4.5. Q = 0.25 l/s, L = 95 m.

H = 30 – 24 = 6 m.

- If we take 26 mm of internal diameter:

f = 1.3% and $\Delta P = 1.3 \times 95 / 100 = 1.2$ m $\rightarrow P_{res} = H + P_{res}J2 - \Delta P = 6 + 9.3 - 1.2 = 14.1$ m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 0.5 m/s). We thus choose pipe level of: **PE 32.**
Total head = 24 + 14.1 = 38.1 m.




4.6. Q = 0.75 l/s, L = 30 m.

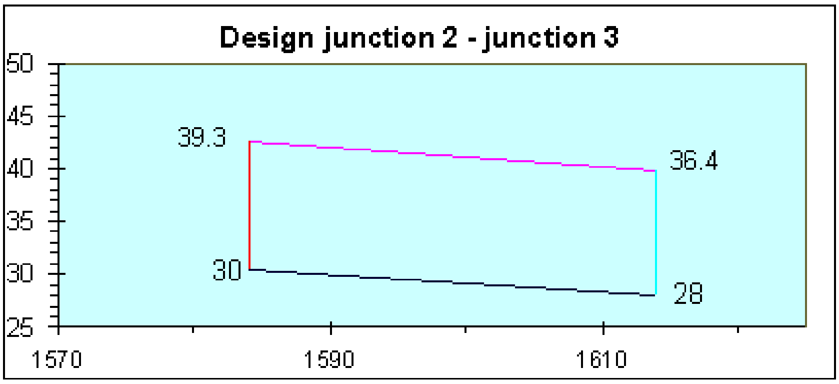
H = 30 – 28 = 2 m.

- If we take 26 mm of internal diameter:

f = 9.5% and $\Delta P$ = 9.5 x 30 / 100 = 2.9 m → $P_{res}$ = H + $P_{res}$J2 - $\Delta P$ = 2 + 9.3 – 2.9 = 8.4 m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 1.5 m/s). We thus choose pipe level of: **PE 32.**

Total head = 28 + 8.4 = 36.4 m.




4.7. Q = 0.25 l/s, L = 60 m.

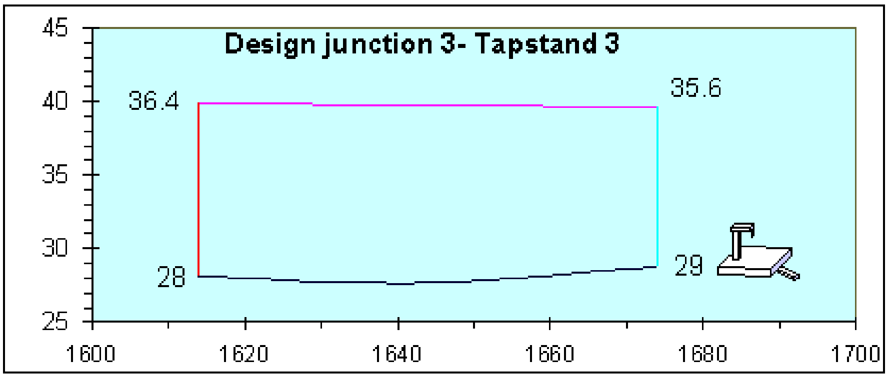
H = 28 – 29 = -1 m.

- If we take 26 mm of internal diameter:

f = 1,3% and $\Delta P$ = 1,3 x 60 / 100 = 0,8 m → $P_{res}$ = H + $P_{res}$J3 - $\Delta P$ = -1 + 8,4 – 0,8 = 6,6 m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 0.5 m/s). We thus choose pipe level of: **PE 32.**

Total head = 29 + 6.6 = 35.6 m.




4.8. Q = 0.5 l/s, L = 79 m.

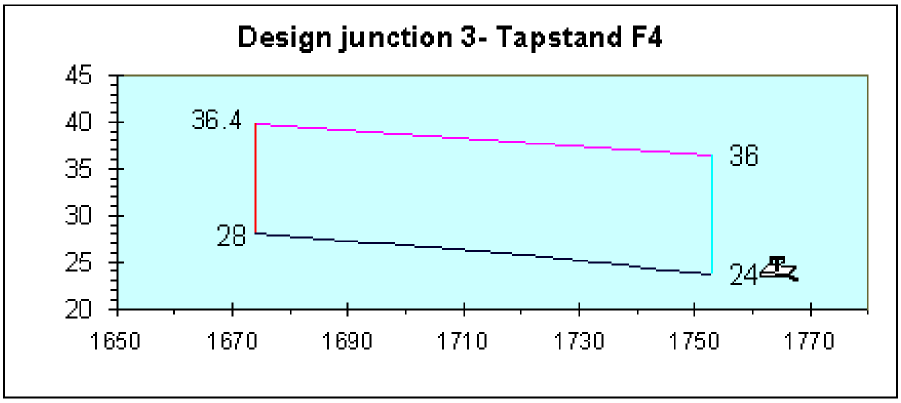
H = 28 – 24 = 4 m.

- If we take 26 mm of internal diameter:

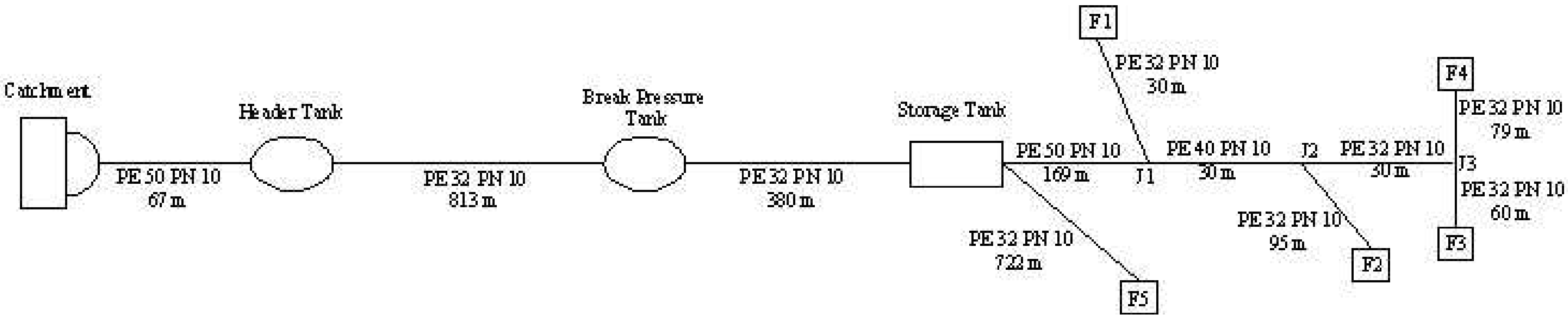
f = 4.3% and $\Delta P$ = 4.3 x 79 / 100 = 3.4 m → $P_{res}$ = H + $P_{res}$J3 - $\Delta P$ = 4 + 8.4 – 3.4 = 9.0 m.

The layout shows that the pipes pressures are always positive and the velocity limits are respected (V = 0.95 m/s). We thus chose pipe level of: **PE 32.**

Total head = 24 + 9 = 33 m.



| Station 1 | Station 2 | Flow (l/s) | Distance (m) | Type of pipe | Head losses (m) | Total Head at station 1 | Station 1 altitude (m) | Total Head at station 2 (m) | Station 2 altitude | Residual pressure at station 1 (m) | Residual pressure at station 2 (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Intake | Header T | 0.45 | 67 | PE 50 PN 10 | 0.25 | 185 | 185 | 185 | 174 | 0 | 11 |
| Header T | Break Press. T | 0.45 | 813 | PE 32 PN 10 | 30.49 | 174 | 174 | 143 | 89 | 0 | 54.7 |
| Break Press. T | Storage T | 0.45 | 380 | PE 32 PN 10 | 13.3 | 89 | 89 | 75 | 48 | 0 | 27.6 |
| Storage T | Tapstand F5 | 0.2 | 722 | PE 32 PN 10 | 6 | 48 | 48 | 41 | 28 | 0 | 13.8 |
| Storage T | Junction J1 | 1.25 | 169 | PE 50 PN 10 | 3.72 | 48 | 48 | 44 | 35 | 0 | 9.3 |
| Junction J1 | Tapstand F1 | 0.25 | 30 | PE 32 PN 10 | 0.39 | 44 | 35 | 44 | 30 | 9.3 | 13.8 |
| Junction J1 | Junction J2 | 1 | 30 | PE 40 PN 10 | 1.44 | 44 | 35 | 43 | 30 | 9.3 | 12.2 |
| Junction J2 | Tapstand F2 | 0.25 | 95 | PE 32 PN 10 | 1.24 | 43 | 30 | 41 | 24 | 12.2 | 17.7 |
| Junction J2 | Junction J3 | 0.75 | 30 | PE 32 PN 10 | 2.7 | 43 | 30 | 40 | 28 | 12.2 | 11.8 |
| Junction J3 | Tapstand F3 | 0.25 | 60 | PE 32 PN 10 | 0.24 | 40 | 28 | 40 | 29 | 11.8 | 10.8 |
| Junction J3 | Tapstand F4 | 0.5 | 79 | PE 32 PN 10 | 3.48 | 40 | 28 | 36 | 24 | 11.8 | 12.8 |



## APPENDIX 2: Nomograph PE and GI pipes