

Gravity distribution systems

A SYSTEM DESIGN AND CONSTRUCTION

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The need for water can be expressed in terms of quantity and quality, but also in terms of accessibility. There are many examples of water points that are under-used because they are too remote from dwellings, or because consumption is restricted because of the long and arduous journey to collect water. It is therefore essential that populations have easy access to water points.

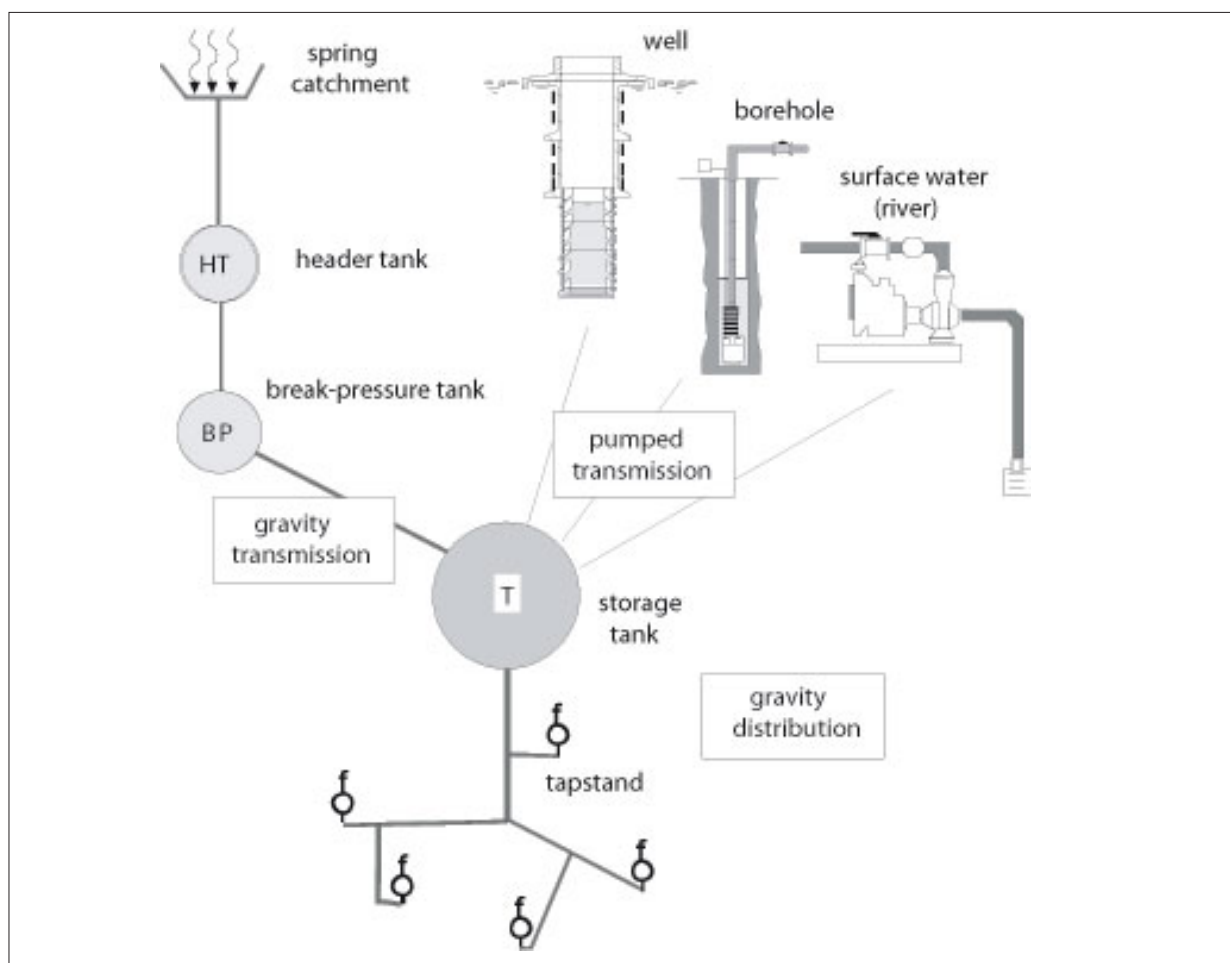


Figure 11.1: Distribution system: various possible schemes.

This chapter presents techniques used by ACF for the design and construction of gravity systems for the transmission and distribution of water (Figure 11.1). Transmission of water by pumping is covered in Chapter 9.

The example of a branched system in the village of Aloua, fed by a spring catchment, illustrates stages of this work, and an example of system design in Laos is given Chapter 11B.

1 Feasibility study

The design of a gravity system is a relatively long piece of work that is difficult to do during a project identification mission. A feasibility study therefore attempts to define the main technical and budgetary aspects of the project. If the findings of this survey indicate that the project is feasible, a more complete survey is undertaken.

1.1 Drawing up the distribution plan

Needs are identified during an assessment visit (see Chapter 2). From this investigation, a layout plan of the site is created, on which the recorded needs and distribution zones are marked. These zones are defined according to social criteria (origins of populations, differences between districts, presence of health structures, religious factors etc.) and technical criteria (accessibility, topography etc.). An example is given in Figure 11.2.

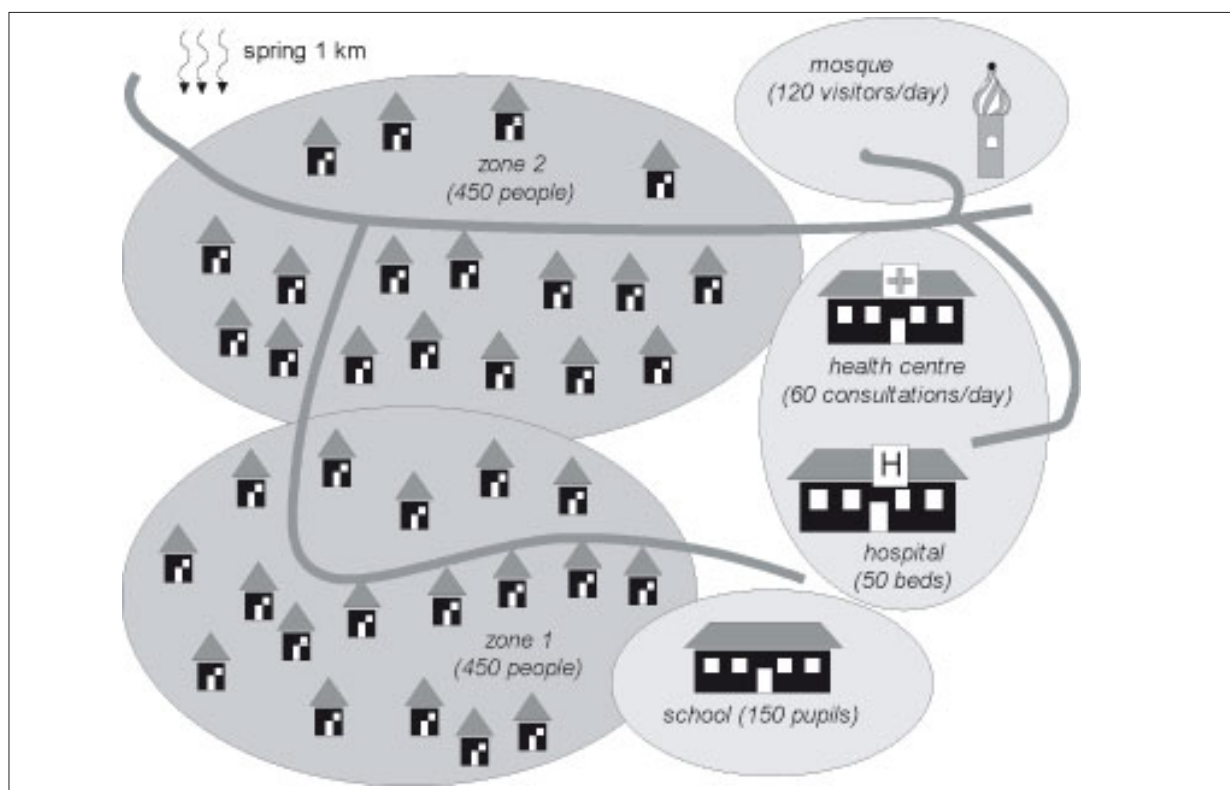


Figure 11.2: Layout plan delimiting distribution zones: Aloua village.

The water needs of the community are identified for every zone. They are expressed as daily volume, then as average flow for the chosen distribution period: for example, in a district of 450 inhabitants whose needs are 20 litres per person per day, the daily volume required is: $450 \times 20 = 9\,000$ l. If daily consumption is considered to be spread over 10 hours, the supply flow for the zone must be $9\,000/10 = 900$ l/h. This is known as the useful flow. Precise definition of the system's period of use during the day is necessary because it has a marked effect on useful flow. In the previous example, if the chosen supply period is 5 h instead of 10, the useful flow is doubled (1 800 l/h).

Broadly speaking, if it is assumed that the system is used continuously for 10 hours a day, consumption peaks, and therefore the useful flow, tend to level out. Depending on the cultural context, it is possible to reduce this period of use to 8 or 6 hours or, better, to estimate demand per hour. It is essentially the habits and daily rhythm of the population that accurately defines the useful flow required.

For public distribution via tapstands, the number of taps is given by the useful flow and access to water points, from the figures shown in Table 11.I.

In the previous example, the useful flow of 900 l/h (or 0.25 l/s) can be distributed by $0.25/0.2 = 1.25$, or two Talbot type taps. With a population of 450, it is preferable to install $450/150 = 3$ taps to limit waiting time at the water point. The three taps are installed on one or more tapstands depending on the area of the zone (maximum distance between two tapstands is 250 m).

Table 11.I: Working criteria for tapstands.

Flow of a standard 3/4" tap (under 10 m of pressure)	0.2 to 0.3 l/s
Flow of a Talbot tap (under 5 m of pressure)	0.1 to 0.2 l/s
Maximum number of people per tap	150 people
Maximum distance between 2 tapstands	250 m

Table11.II: Calculation of needs for Aloua village. January 1995, annual growth rate: 2%.

	Number	Number 10 years	Unit needs (l/pers/day)	Total needs (m ³ /day)	Number of taps at 0.25 l/s
Health centre	50 consultations	60	20	1.2	1
Hospital	40 beds	48	50	2.4	1
School	150 pupils	180	10	1.8	1
Religious centre	100 visitors	120	10	1.2	1
Inhabitants	900 inhabitants	1 080	40	43.2	6
Total needs, water (m ³ /day):	49.8				
Need in tapstands, 1 tap:	4				
Need in tapstands, 2 taps:	0				
Need in tapstands, 3 taps:	2				

The design of the system must be aimed at a minimum life of 10 years. It is necessary to take account the normal population growth rate, plus the possibility of additional population growth due, for example, to the arrival of water or a road (Table11.II).

When the number of tapstands and taps has been determined, information on maximum flows in each distribution branch (maximum flow = number of taps served x unit flow of taps, see Figure 11.3) is marked on the outline layout. In the Aloua example, a unit flow of 0.25 l/s per tap was chosen. The layout plan completed in this way is an essential tool for determining diameters and lengths of pipe, used in parallel with topographic profiles. Symbols used are shown in Figure 11.11.

The maximum instantaneous flows calculated in this way ensure the distribution of a daily volume greater than needed (number of taps rounded up in every zone). In the example of Aloua, daily

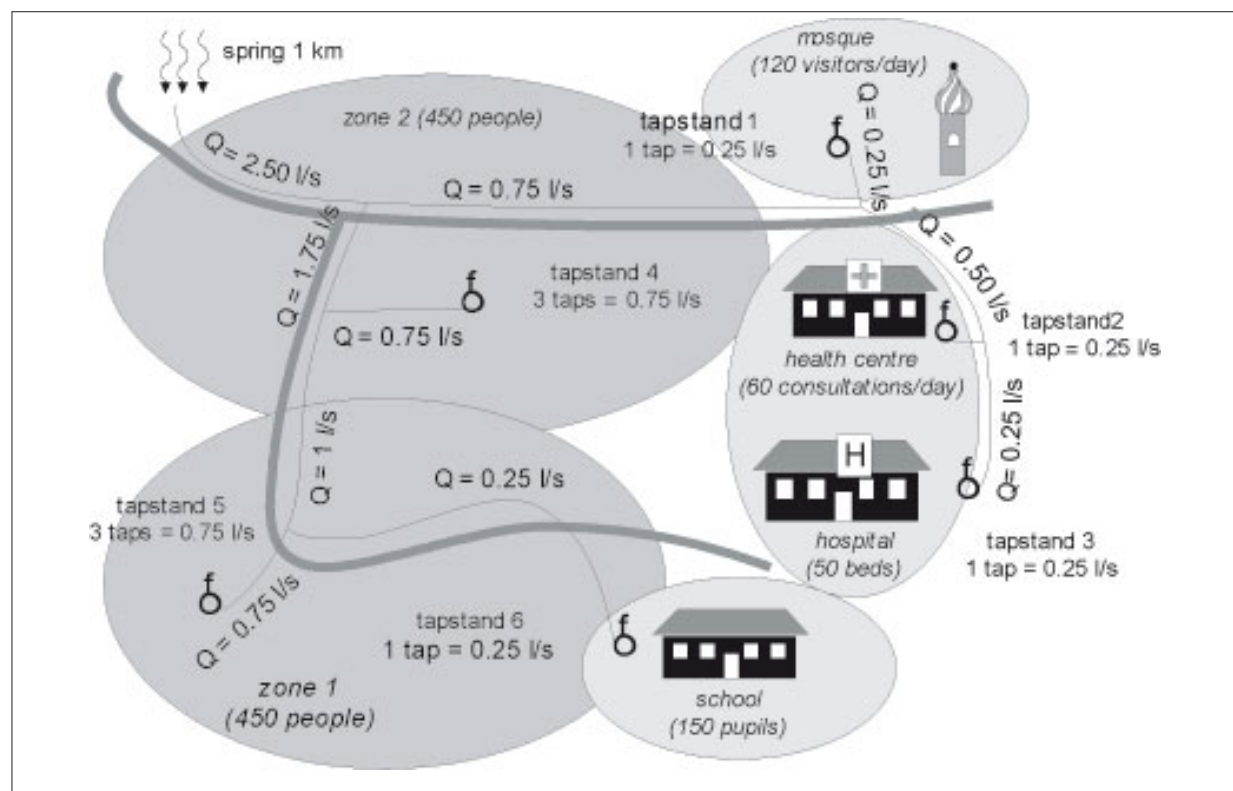


Figure 11.3: Marking out lines, flows and numbers of taps.

needs were estimated at 49.8 m³, whereas the sum of the maximum flows is $(0.75 \times 2) + (0.25 \times 4) = 2.5$ l/s, or 9 000 l/h over a six-hour period, or 54 m³/day. In practice, the flow passing through the system does not correspond to the maximum flow at certain hours in the day, when all taps will be open. The rest of the time, flow will be lower and the daily volume distributed will be closer to that calculated on the basis of need.

1.2 Rapid topographic survey

The objective is to determine whether the difference in level between the various points in the system is sufficient to permit flow of water by gravity.

A rapid estimate can be made simply by using an altimeter. A reading of the altitude of the characteristic points on the line of the pipe is carried out, covering high and low points, areas served etc. Distance at ground level is measured in number of paces.

Since the altimeter is directly linked to atmospheric pressure, altitude measurements must be made at fixed points (fixed altitude) to compare with the altitude measured by the operator as they move from point to point.

A rapid summary card is shown in Table 11.III.

Table 11.III: Altimeter summary card.

Grid for the mobile operator on the pipe route					Grid for the stationary assistant		
Hour / minute	Distance over ground (m)	Distance (cumulative) (m)	Altitude (m)	Comments		Hour / minute	Altitude (m)

1.3 Technical verification

Two conditions must be met in order to verify the feasibility of the system: the available water quantity at the source must be sufficient to cover needs, and the topography must permit gravity distribution. When needs are defined for every zone, it is simple to verify the feasibility of the system in terms of quantity: water flow over 24 hours must be greater than, or equal to, the daily needs of the community.

To permit gravity distribution, the slope between the highest point of the system (reservoir or spring catchment) and the furthest point in the system must be greater than 1%. If a 1% slope line is drawn from the highest point, no part of the system must be situated above it.

If these two conditions are met, it is basically technically possible to construct a gravity distribution system that will cover the needs of the population. However, this must then be confirmed by a detailed survey.

2 Detailed survey

2.1 Topographic survey

The topographic survey is an essential part of the design method for gravity systems. This survey of differences in level enables energy balances to be drawn up, and flows, velocities and head-losses to be calculated for all points in the system.

2.1.1 CHOICE OF PIPE ROUTE

It is possible to establish a theoretical pipe route to avoid difficult passages using a map or aerial photograph, but the definitive route can only be established by visiting the area along with someone who knows the locality well. In defining the route, the following problems need to be taken into account:

- minimising the number of difficult crossings: roads, gullies etc.;
- avoiding steep slopes: difficulty of pipe anchorage;
- avoiding rocky zones: difficulty of trench digging;
- looking for accessible zones: alongside existing paths, for example;
- considering problems of land ownership and authorisation;
- being aware of community problems such as constraints linked to the occupation of land (gardens, houses etc.).

2.1.2 TOPOGRAPHIC SURVEY METHODOLOGY

The rapid survey made during the preliminary assessment must be confirmed and detailed by an accurate survey. The easiest on-site technique consists of using a level. The use of a theodolite is

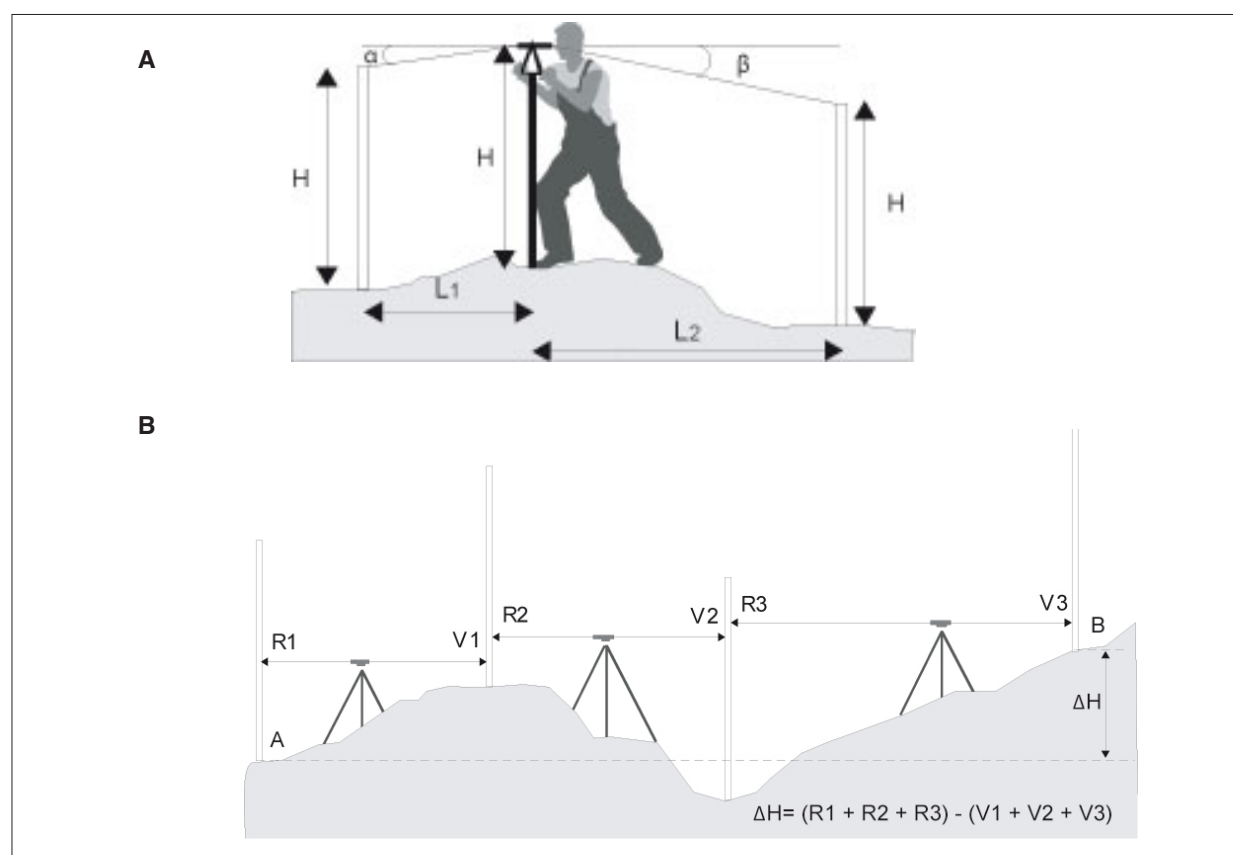


Figure 11.4: Topographic survey.

A: with Abney level or clinometer. B: with dumpy level.

recommended, because the measurement precision is very good. However, an Abney level or a clinometer may be used in relatively inaccessible regions.

The principle of the Abney level and the clinometer involves measuring the angle between a given point and the horizontal. If the distance on the ground between this point and the measuring device is known, it is easy to calculate the difference in level (from the distance on the ground multiplied by the sine of the angle in degrees).

Two readings per point are taken, one reading forward, and one reading back to the previous point to confirm the measurement. The distance on the ground is measured using a tape measure.

The Abney level or clinometer (Figure 11.4A) must be placed on a rule to assure stability. Readings are taken at the top of wooden rules held by assistants. The three rules must be exactly the same length.

The theodolite (Figure 11.4B) can achieve very high accuracy and can also measure horizontal distances with the rangefinder of the telescope. These distances correspond to the horizontal distance between two points, and not to the distance on the ground (the one that is useful to know because it represents the length of pipe needed). The distance on the ground is measured with a tape measure, or calculated from the angle of sight and the horizontal distance read on the telescope.

The reading is made by sighting rules held vertically by assistants.

2.1.3 DRAWING UP THE TOPOGRAPHIC PROFILE

The topographic summaries are represented as a profile that illustrates the energy balances of the water in the pipes. It is also useful to make every profile correspond to a plan to illustrate important points and junctions with other pipelines (Figure 11.5).

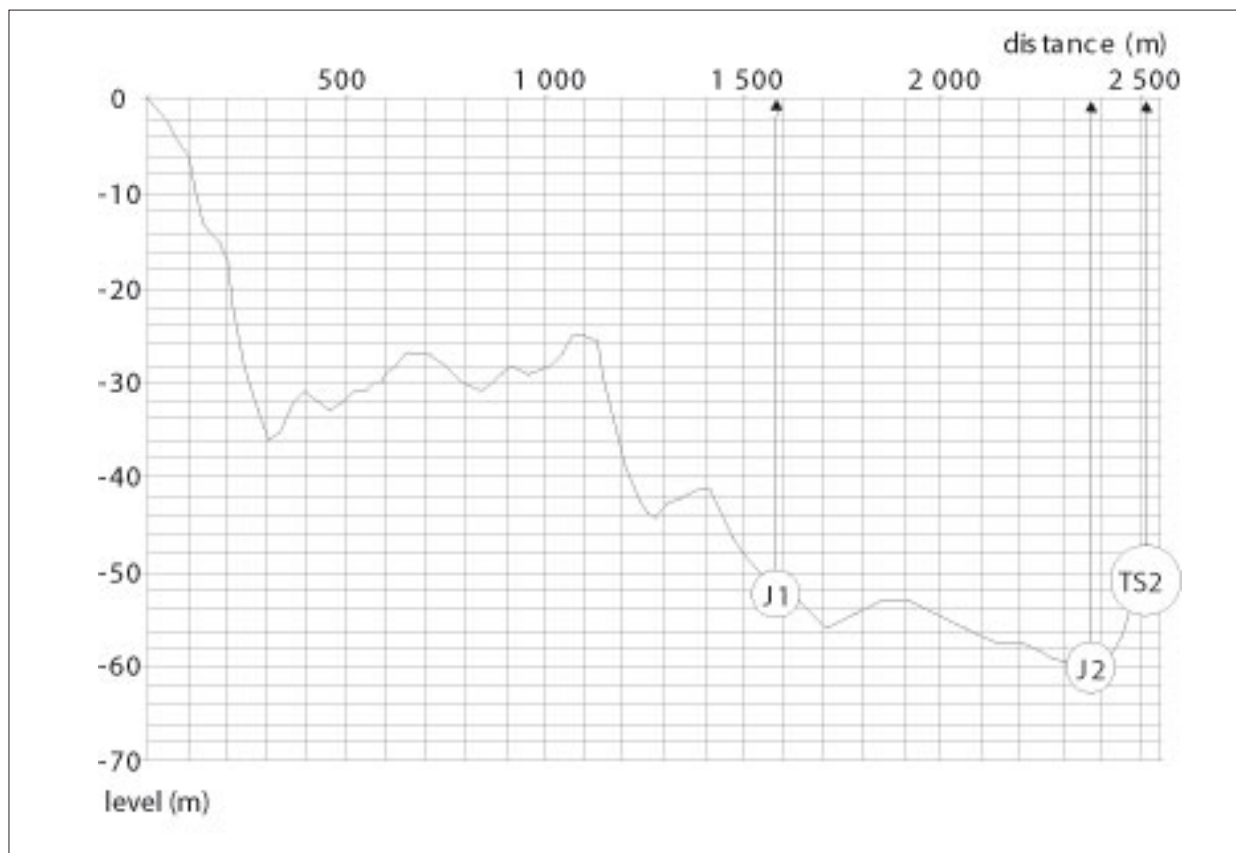


Figure 11.5A: Topographic profile (from catchment/J1/J2/Ts2, Aloua).

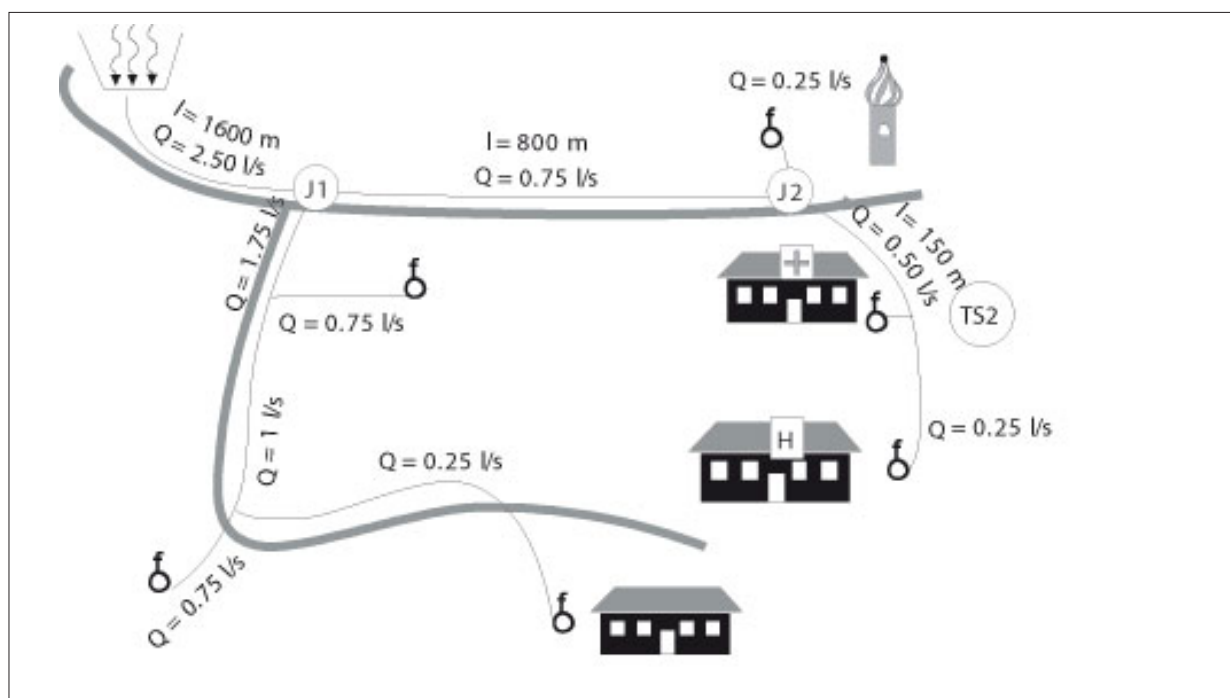


Figure 11.5B: Layout plan (from catchment/J1/J2/Ts2, Aloua).

2.2 Hydraulic design

2.2.1 ENERGY PROFILES

2.2.1.1 Static profile

The static-head profile represents the water energy at zero flow (closed taps).

The pressure in a closed pipe under load ($Q = 0$) can be regarded as being equal to the weight of the water column between its upper point and the point under consideration (see Annex 6). This pressure, known as the static pressure, is the driving force of the system: in other words, it is difference in level between the various points of the system that causes the flow of water from the tank (or the spring catchment) to the tapstands.

The static pressure is expressed in mWG (metres water gauge), so that:

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

where H is the difference in level between the upper point and the point under consideration. In Figure 11.6, the high point is the free water surface of the tank.

In simple systems, the static pressure is the maximum pressure that can exist inside the pipes. It therefore determines the pressure the pipes must be able to resist, and whether it will be necessary to use pressure-limiting devices.

2.2.1.2 Class of pipe pressure and header tanks

The pipes used will withstand a given pressure, known as the nominal pressure (NP): if the pressure in the pipe is greater than this, there is a risk of rupture. The NP is generally expressed in bars. The usual pressure classes are given in Table 11.IV.

From the static-head profile it is possible to define the pipe class needed. In Figure 11.7, the static level at the lowest point is 75 mWG, or 7.5 bar. A pipe of 6 bars nominal pressure (NP 6) would therefore be inadequate, so it is necessary to use NP 10 bar pipe.

Table 11.IV: Classes of pressure of distribution pipes.

PE (polyethylene)	PVC (polyvinyl chloride)	GI (galvanised steel)
NP 6 NP 10 NP 12.5	NP 6 NP 10	NP 16 NP 25

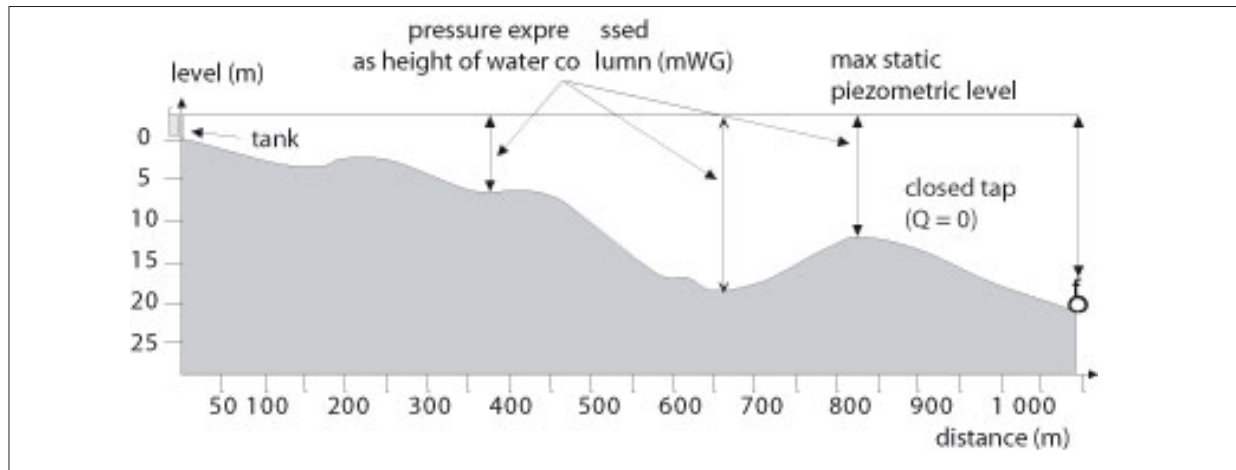


Figure 11.6: Static pressure in a pipe system.

If the pressure imposed by the topography is too great for the class of pipe available, it is possible to construct a break-pressure tank that returns the pressure in the system to atmospheric. Whenever there is a free surface in contact with the atmosphere, the static pressure becomes zero, because the pressure on this surface is equal to the air pressure*.

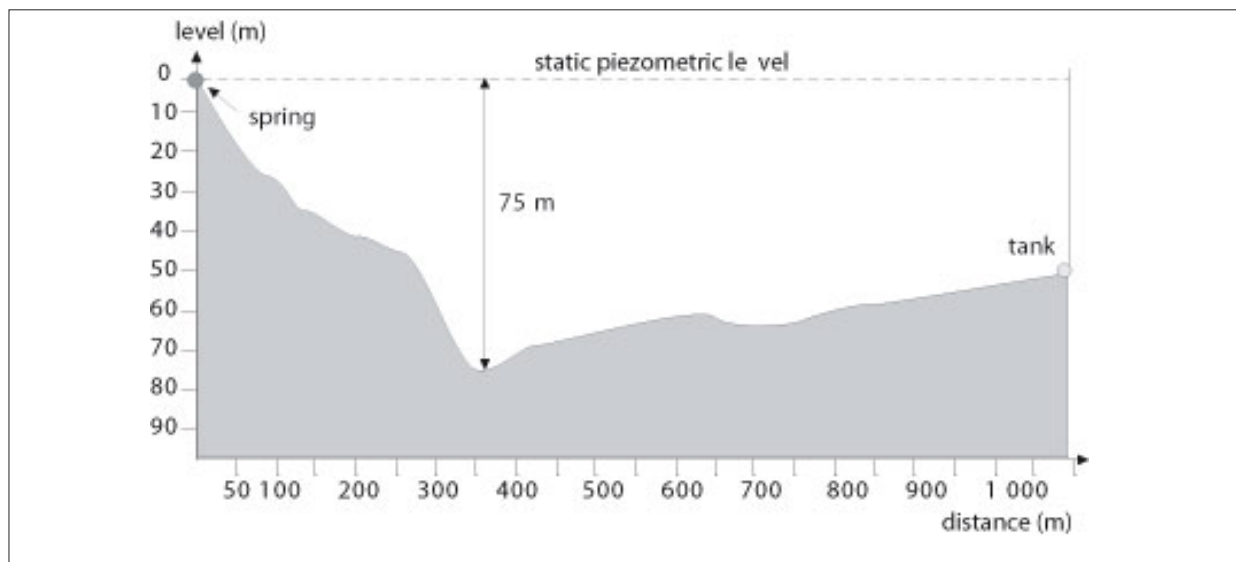


Figure 11.7: Static profile and pressure class of pipes.

* Here we are using relative pressure, i.e. related to the atmospheric pressure that acts on the whole system. Zero pressure means therefore that the actual pressure is $P_{\text{atmospheric}}$.

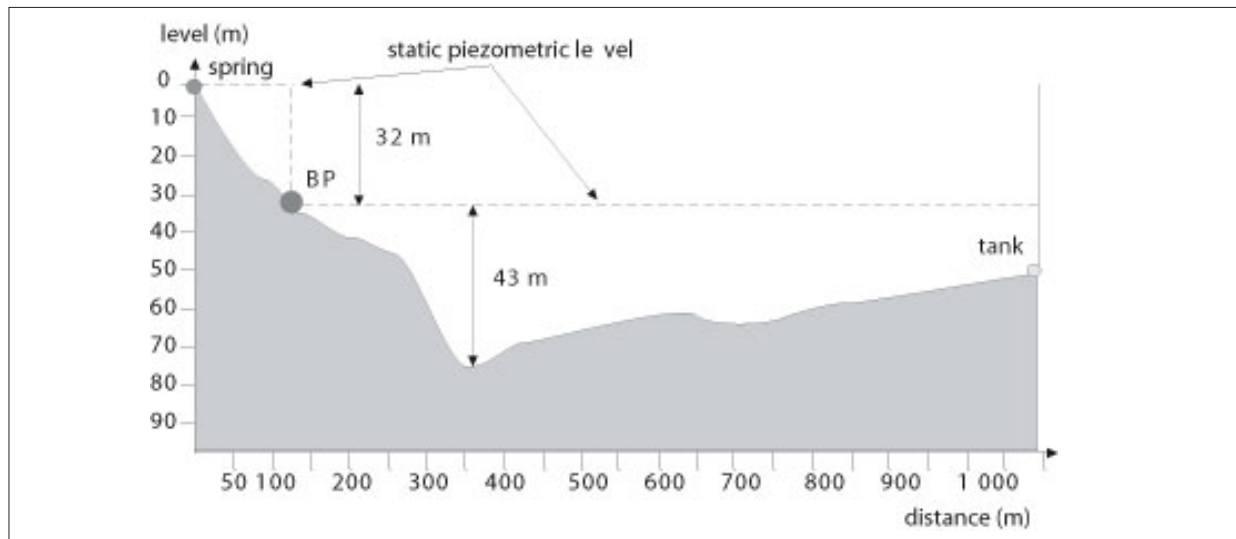


Figure 11.8: Installation of a break-pressure tank to stay within a pipe class ($P_{\text{static}} < 60$ mWG).

In a distribution system, the free surfaces are reservoirs, header tanks, break-pressure tanks, and catchment tanks. In the example in Figure 11.7, the pipe class had to be NP 10. However, if there were only NP 6-class pipes available, it would be possible to install a break-pressure tank to control the pressure. Figure 11.8 shows the installation of a break-pressure tank to obtain maximum static pressures of 32 mWG upstream and 43 mWG downstream.

2.2.1.3 Dynamic profile

When water moves in pipes, pressure losses due to water friction occur. These pressure losses are referred to as 'head-losses' (see Annex 12).

If the pressure-drop line during flow is drawn, the dynamic-head profile is obtained (Figure 11.9).

Part of the total pressure is therefore taken up in head-losses (ΔP) during the transmission of water. The residual pressure is then defined by:

$$P_{\text{residual}} = H - \Delta P$$

where P_{residual} is the residual pressure at the point under consideration (mWG), H is the difference in level between the upper point and the point under consideration (m), and ΔP is the sum of head-losses (m).

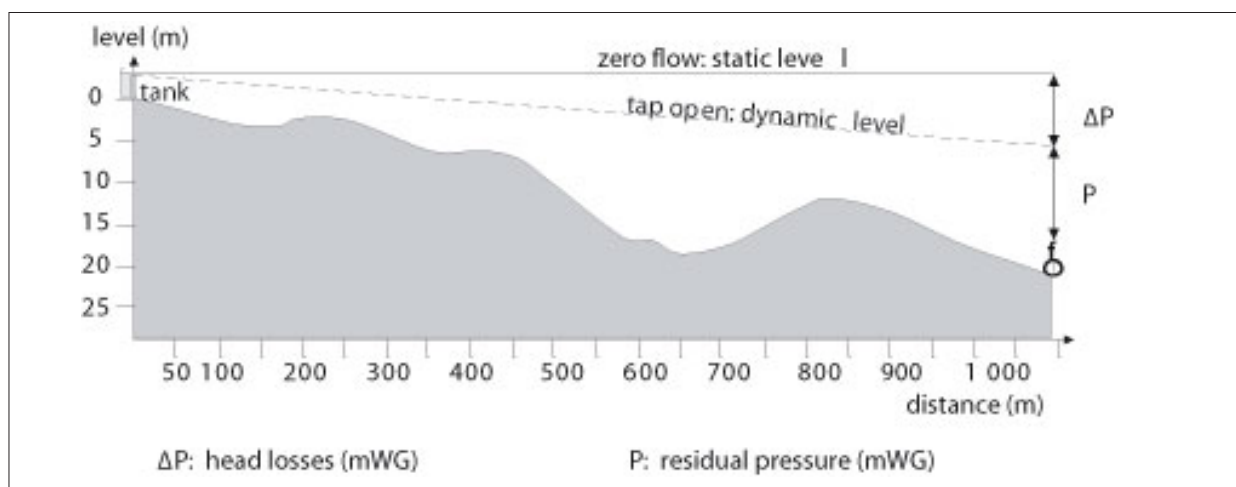


Figure 11.9: Dynamic state and head losses.

2.2.2 CALCULATION OF HEAD-LOSSES

To facilitate the calculation of head-losses, the losses created in the pipes and those induced by components (elbows, tees, valves etc.) are considered separately.

2.2.2.1 Linear head-losses

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

- pipe diameter: for a given flow, the smaller the pipe the greater the losses;
- flow in the pipe: for a given diameter, head-losses increase with flow;
- pipe length;
- pipe roughness: the greater the roughness, the greater the head-losses (all things being equal). The roughness of pipes depends on their quality (materials, manufacture) and age.

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

The numerical calculation of linear head-losses is usually carried out by using empirical relationships, tables or design charts (see Annex 12).

Formulas

There are several formulas for calculating the linear head-losses in a pipe. The most accurate (Darcy and Colebrook-White for example) need iterative calculation and are complicated to use in the field. Other methods were developed, such as the Hazen Williams formula*, which is relatively simple to use:

$$h_F = \frac{10.9 L Q^{1.85}}{C^{1.85} D^{4.87}}$$

where h_F : head-loss (m), L : pipe length (m), Q : flow (m^3/s), C : Hazen Williams coefficient ($C = 150$ for PVC and PE; $C = 130$ for GI), D : internal pipe diameter (m).

Use of design charts

The design chart (or nomogram) shown in Figure 11.10 represents graphically the relationship between pipe diameter, flow velocity, and linear head-losses, for a given roughness. Knowing two parameters (diameter and flow), head-losses can be deduced.

Take for example a flow of 1 l/s in a polyethylene pipe of DN 50 (50 mm outside diameter, 40.8 mm internal diameter). According to the design chart, the head-loss coefficient is 1.7%. If the head-losses are to be reduced, the line is pivoted (as shown by the arrows) around the selected flow to obtain the new pipe diameter; for a head-loss of 1% with the same flow (1 l/s) a pipe of DN 63 (internal diameter = 51.4 mm) should be used, see Table 11.V.

When using design charts, it is important to check the following points:

- the chart must correspond to the type of pipe used (PE, PVC, GI, cast iron etc.). It is essential to verify its validity when calculating head-losses using suitable formulas (see Annex 12);
- the velocity in a pipe must be close to 1 m/s. Higher velocities generate excessive friction, which may lead to hydraulic problems. On the other hand, lower velocities can allow silt deposits at points in the system if the water is turbid.

* Formula valid for $V < 3 \text{ m/s}$ and $Re > 4\,000$, where V = velocity of the flow (m/s) and Re = Reynolds number (ratio between inertial forces and viscous forces, used to determine whether flow in the pipe will be laminar or turbulent).

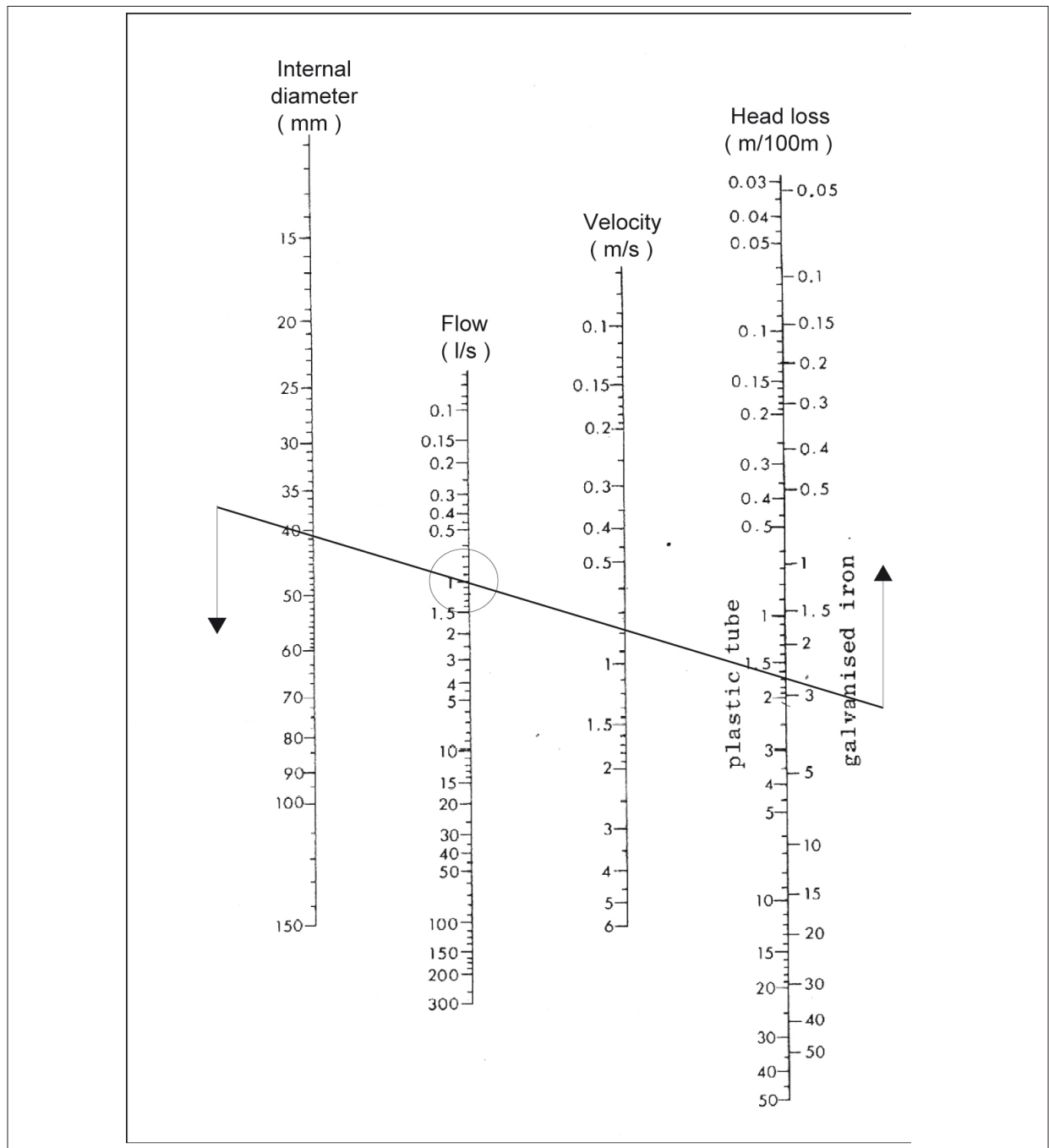


Figure 11.10: Design chart for calculating linear head-losses.

Table 11.V: Example of use of a linear head-loss chart.

Flow (l/s)	Internal diameter (mm)	Velocity (m/s)	Head-loss coefficient (%)	Head losses ΔP per km (mWG)
1	40.8 (DN 50)	0.8	1.7	17
1	51.4 (DN 63)	0.5	0.6	6

2.2.2.2 Secondary head-losses

In addition to the linear head-losses in pipes, there are also losses of pressure due to flow regulation devices (valves), and to passage through fittings such as elbows, tees, reducers etc. These pressure losses are known as ‘minor’ or ‘secondary’ head-losses and can be evaluated in the same way as linear losses.

In simple systems, secondary head-losses are generally small compared with linear head-losses. They are sometimes assumed to be about 10% of the linear head-losses. However, they can also be calculated using characteristic coefficients for each component. As for pipes, there are design charts or tables. Components are generally assigned equivalent straight lengths of pipe. For example, a 90° elbow in DN 50 corresponds, from the point of view of head-losses, to a length of pipe of 1.5 m of the same diameter. This allows simple calculation of the head-losses in the system.

Table 11.VI gives the equivalent lengths of pipe (in terms of friction head-loss) for common PVC or PE fittings. For GI pipes, these values must be multiplied by 0.64. When two values are given under the same category, they correspond to a range of equivalent lengths given by different parameters. Broadly speaking, screwed fittings create higher head-losses than jointed or cemented fittings.

Table 11.VI: Equivalent length (m) for plastic components (Lencastre).

Diameter of components (DN in mm)	12.5	25	50	75	100
Globe valve	1.83 3.61	4.78 9.17	12 23	20.37 39.98	30.3 58.38
Gate valve	0.014 0.055	0.055 1.66	0.11 0.41	0.19 0.75	0.3 1.11
Y-valve	1.08	2.75	6.92	11.75	17.51
Horizontal non-return valve	0.8	0.055 2.11	0.14 5.28	0.22 9	0.36 13.42
Foot valve	5.42	13.76	34.75	58.93	87.57
Screwed reducer	0.72	0.055 1.83	0.11 4.61	0.19 7.83	0.3 11.6
90° normal elbow	0.22 0.33	0.55 0.84	1.39 2.08	2.36 3.53	3.5 5.25
45° elbow	0.08 0.11	0.19 0.27	0.5 0.69	0.86 1.33	1.27 1.75
Normal tee	0.3 0.44	0.77 1.19	1.97 3	3.33 5.08	4.94 7.58

2.3 System design

The documents necessary for pipe-system design are the topographic profiles of the system as a whole, and the plan with the maximum flows for each section of the system.

Documents produced will be head profiles for the whole system, the detailed plan of the system, the summary table of profiles with all features of sections of the system, and the location of valves. The symbols usually used for structures and fittings are given in Figure 11.11.

2.3.1 DESIGN OF THE STORAGE TANK

In the case of gravity supplies from spring catchments, a storage tank is not always necessary. If the useful flow (calculated on maximum hourly demand) is lower than the minimum yield of the spring in one hour, a storage tank serves no useful purpose. This option also has the advantage of not requiring the installation of taps on water-collection points. If a storage tank is needed, however, it must be carefully designed (see Chapter 10).

tapstand (several taps)	
tapstand (one tap)	
header tank	
sedimentation tank	
break pressure tank	
water tank n °1	
spring catchment	
protected pipe	
hanging and protected pipe	
anchor	
pump	
water meter	
ball valve	
stop cock	
adjustable maximum stop cock	
gate valve	
air valve or air vent	
well	
borehole	

Figure 11.11: Symbols for structures and fittings.

2.3.2 LOCATION OF STRUCTURES

2.3.2.1 Header tank

A header tank is essential in the case of gravity transmission from a spring (Figure 11.1). It isolates the spring catchment from the system hydraulically, and therefore avoids any possibility of accidental back pressure on the spring in the event of problems in the system. It also helps to even out the flow from the spring.

Site conditions and the difficulties of transporting materials often define its exact location, just downstream of the catchment. The hydraulic profile of the system begins from this point.

2.3.2.2 Storage tank

The location of the tank is dictated by the necessity to be able to serve the tapstands situated downstream using gravity. As a first approximation, it is assumed that the head-losses in the part of the system below the tank are the order of 1%. A straight line of 1% slope is therefore drawn starting from the highest tapstand (taking account of a minimum residual pressure of 10 mWG). Points situated below this line are unsuitable locations for the tank (Figure 11.12).

The site chosen must then be checked by calculation of head-losses. In all cases, problems of accessibility (transport of construction materials and equipment, maintenance etc.) and land ownership must be considered. The emptying time of a tank can be calculated (Box 11.1).

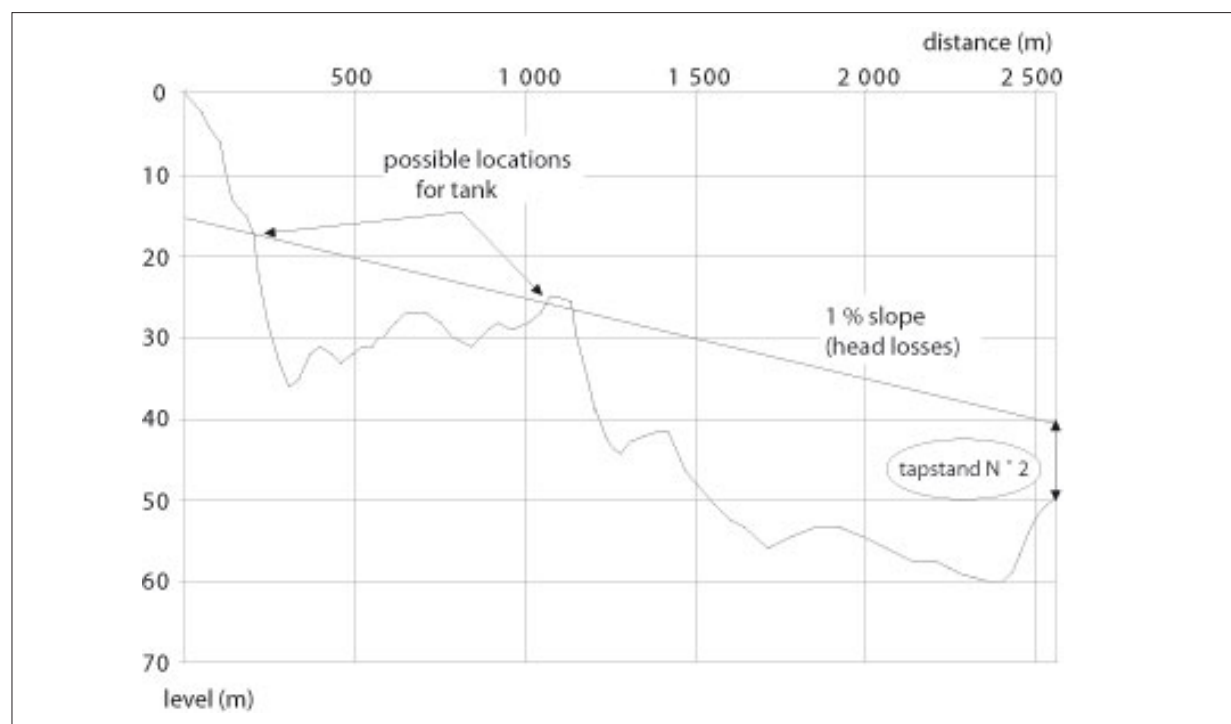


Figure 11.12: Selection of tank location (example of Aloua system).

Two locations are possible. The upstream site is in an apparently steep area, while the lower site is on a raised area about halfway between the catchment and tapstand No. 2. Placing the tank on the lower site means it can act as a break-pressure tank to limit the maximum static pressure in the system to 35 mWG.

2.3.2.3 Break-pressure tank

The necessity for and location of any break-pressure tanks are determined by the static profile survey (see Section 2.2.1).

2.3.3 CHOICE OF TYPES, DIAMETERS AND LENGTHS OF PIPE

2.3.3.1 Types of pipe

Generally, polyethylene pipes of up to 3" are favoured. Above this diameter, pipes become difficult to handle (less flexible, heavy, requiring welded joints). PVC is easy to use and available in most countries. However, galvanised steel (GI) pipes should be used for all unburied sections, gully crossings, and sections enclosed in masonry. See Section 3.3 for more detail on pipes.

Box 11.1

Emptying time for a storage tank.

Tank with outlet

The emptying time of a tank with a side outlet is given by:

$$t = \frac{2S (\sqrt{h_1} - \sqrt{h_2})}{k s \sqrt{2g}}$$

where t is the emptying time (s), S the area of the tank (cm^2), s the area of the outlet (cm^2), g acceleration due to gravity (981 cm/s^2), h_1 the initial height (cm) of water above the outlet, h_2 the final height (cm) of water above the outlet ($h_2 = 0$ if emptying is total), and k is a coefficient of contraction (according to the outlet, $0.5 < k < 1$). For a small thin-wall outlet, $k = 0.62$; for an inserted cylindrical fitting, $k = 0.5$; for an external cylindrical fitting, $k = 0.82$.

Tank connected to an open pipe – maximum gravity flow

When the residual pressure of a pipe discharging freely to atmosphere is zero, the flow passing through the pipe is the maximum gravity flow.

In such conditions, head losses J at the end of the pipeline correspond to the total difference in level h . Knowing the distance between the two points, l , it is simple to calculate the flow Q , or to read it from a chart (Figure 1).

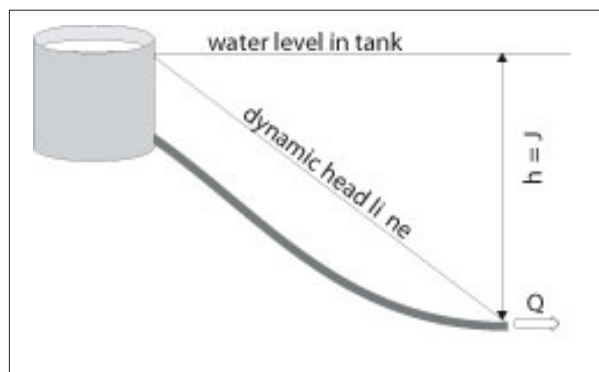


Figure 1: Maximum gravity flow.

2.3.3.2 Pipe diameters

Pipe diameters are determined by the energy-profile survey and head-loss calculations.

The recommended velocity limits in pipes are maximum 3m/s and minimum 0.5 m/s.

The residual pressures in the system must be in accordance with the following rules:

- 2 to 10 mWG for entries to tanks, header tanks etc.;
- 5 to 15 mWG for taps (5 mWG for the Talbot tap);
- 10 mWG minimum in the main distribution lines, to prevent the infiltration of water from outside the pipe. The working pressure of the system guarantees the quality of water distributed.

2.3.3.3 Gravity transmission (spring-to-tank line)

The part of the system feeding the tank from a spring catchment (Figure 11.1) is designed to allow the maximum flow of the spring to pass through the pipes. It is important to be able to capture and transport the total flow of the spring to the tank, even if the current needs of the population are lower than this, as it allows for the possibility of future extensions to the system. If the flow from the spring is very much greater than the needs of the population, or if it fluctuates considerably during the year, a suitable compromise must be determined.

2.3.3.4 Gravity distribution (tank-to-tapstand lines)

Branched-system design begins with the main line, followed by secondary lines. It then proceeds successively until the diameter chosen gives the required residual pressure.

In the example in Figure 11.13, P_{residual} is given by the difference in level between point B at ground level, and that of the piezometric profile vertically above this point. The diameter required is DN 50, because:

- for a DN 50 pipe, P_{residual} is positive and equal to 14 mWG;
- for a DN 40 pipe, P_{residual} is also positive and equal to 4.1 mWG;
- for a DN 32 pipe, P_{residual} is negative and equal to –41 mWG.

Figure 11.14 shows a high point where the pressure must be kept above 10 mWG.

It is best to use small-diameter pipes as they are less costly, and two different diameters may be used in the same line to optimise costs (Figure 11.15).

Head profiles of different lines, accompanied by an overall summary diagram, bring all this information together. The example of the line serving tapstand No. 2 of the Aloua system is illustrated in Table 11.VII and Figure 11.16.

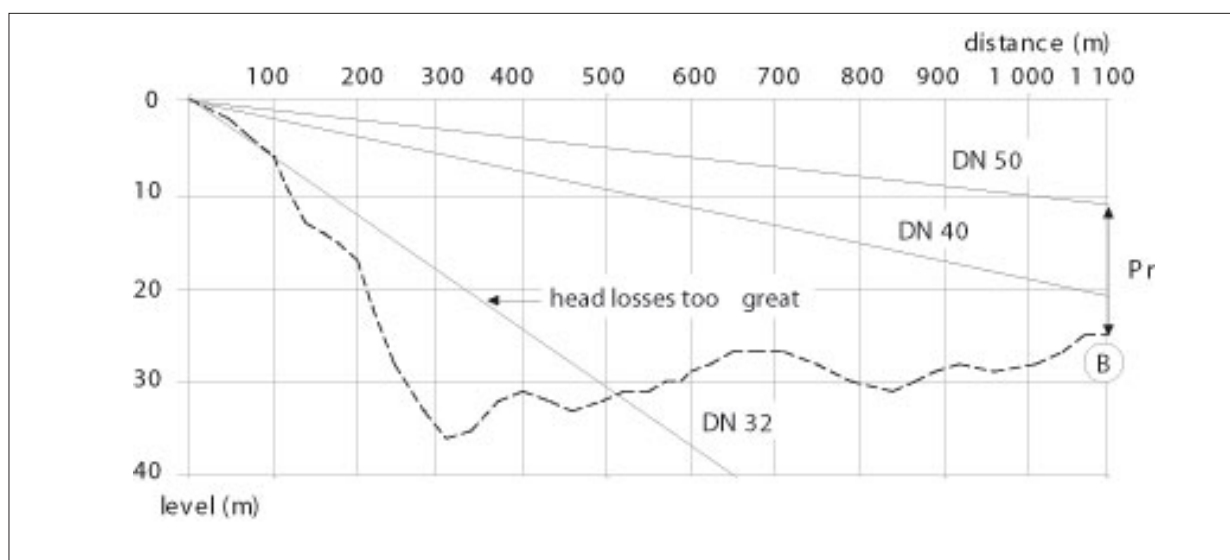


Figure 11.13: Head losses for different pipe diameters for the first part of the Aloua system.

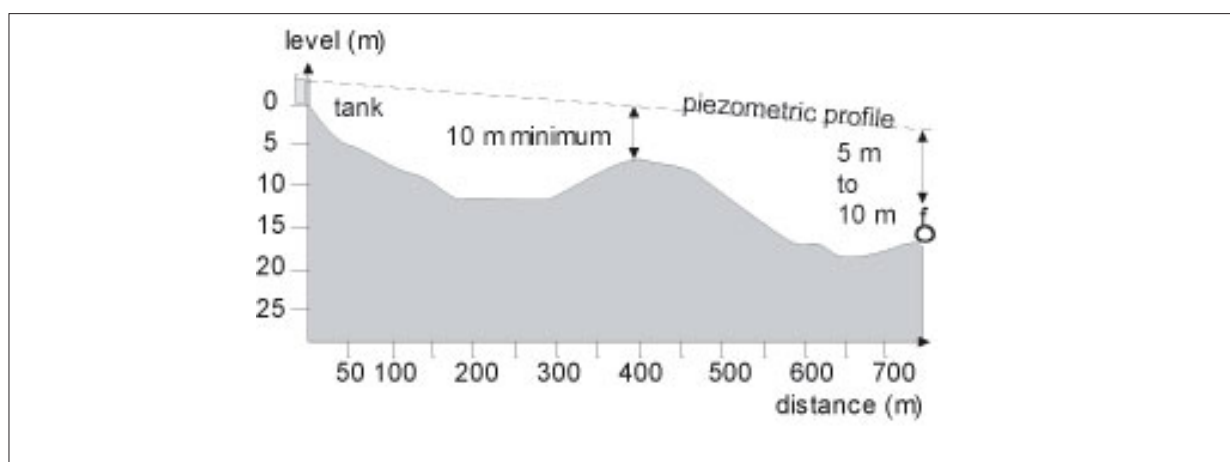


Figure 11.14: Passage over a high point.

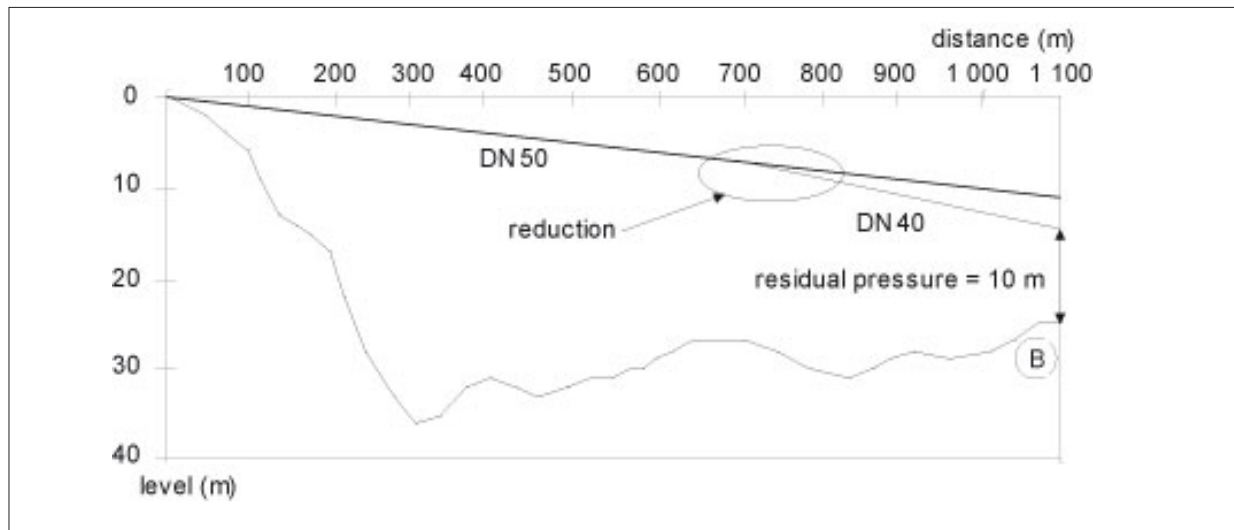


Figure 11.15: Combination of two pipe diameters to obtain a residual pressure of 10 mWG at Point B (example from first part of Aloua system).

Table 11.VII: Summary of the main line serving Tapstand No 2 of the Aloua system.
Ps, static pressure. Ph, dynamic pressure.

Point station (1)	Point station (2)	Flow (l/s)	OD (mm)	Length L (m)	ΔP (A %)	ΔP (m)	P_h (1) (mWG)	Survey level (2) (m)	P_h res (2)
Spring	Reduction	0.7	50	330	1	3.3	0	36	32.7
Reduction	Tank	0.7	40	770	1.9	14.7	32.7	25	7.7
Tank	J1	2.5	63	500	1.4	7	0	52	20
J1	J2	0.75	50	800	0.87	7	20	60	21
J2	TS2	0.5	40	150	0.7	1	21	49.85	10.25

Spring - tank

The flow taken into account for the design is the maximum flow provided by the spring. A combination of two pipe diameters is used for this section.

• Spring - Reduction

$$\begin{aligned} \text{Pressure at reduction} &= P_s \text{ reduction} - \Delta P & \Delta P &= A (\%) \times L(m) \\ P_h \text{ reduction} &= 36 - 3.3 & &= 32.7 \text{ mWG} & \Delta P &= 1 \times 330 = 3.3 \text{ mWG} \end{aligned}$$

• Reduction - Tank

$$\begin{aligned} \text{Pressure at tank} &= P_h \text{ reduction} + (P_s \text{ tank} - P_s \text{ reduction}) - \Delta P & \Delta P &= A (\%) \times L(m) \\ P_h \text{ tank} &= 32.7 + (25 - 36) - 14.7 & &= 7 \text{ mWG} & \Delta P &= 1.9 \times 770 = 14.7 \text{ mWG} \end{aligned}$$

Tank – Tapstand 2

As the tank has a free surface, the pressure at its surface is zero (relative pressure).

• Tank - J1

$$\begin{aligned} \text{Pressure at J1} &= P_h \text{ tank} + (P_s \text{ J1} - P_s \text{ tank}) - \Delta P & \Delta P &= A (\%) \times L(m) \\ P_h \text{ J1} &= 0 + (52 - 25) - 7 & &= 20 \text{ mWG} & \Delta P &= 1.4 \times 500 = 7 \text{ m} \end{aligned}$$

• J1-J2

$$\begin{aligned} \text{Pressure at J2} &= P_h \text{ J1} + (P_s \text{ J2} - P_s \text{ J1}) - \Delta P & \Delta P &= A (\%) \times L(m) \\ P_h \text{ J} &= 20 + (60 - 52) - 7 & &= 21 \text{ mWG} & \Delta P &= 0.87 \times 800 = 7 \text{ m} \end{aligned}$$

• J2-Tapstand 2

$$\begin{aligned} \text{Residual pressure at tapstand 2} &= P_h \text{ J2} + (P_s \text{ TS2} - P_s \text{ J2}) - \Delta P & \Delta P &= A (\%) \times L(m) \\ P_h \text{ tapstand 2} &= 21 + (49.85 - 60) - 1 & &= 10.25 \text{ mWG} & \Delta P &= 0.7 \times 150 = 1 \text{ m} \end{aligned}$$

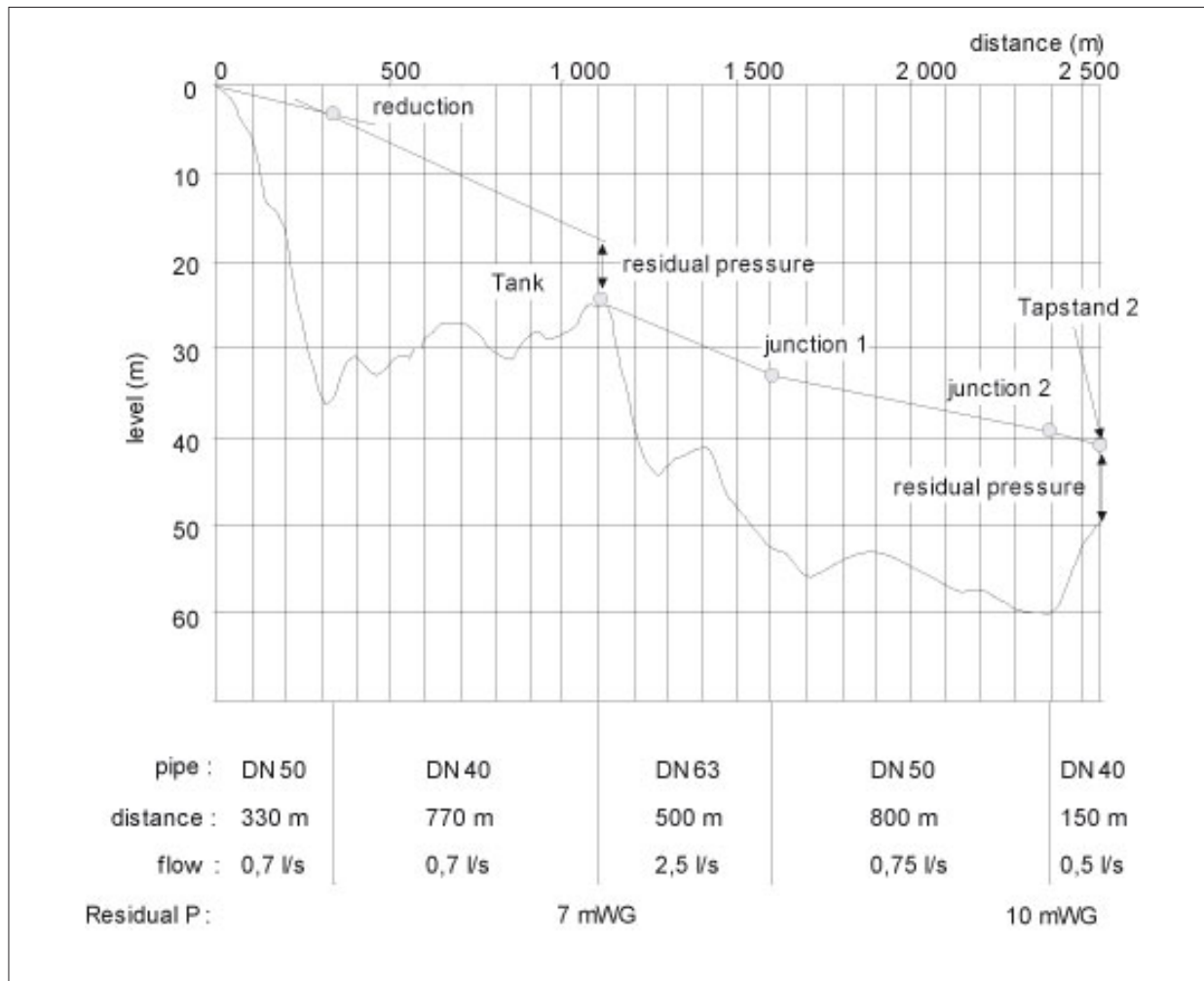


Figure 11.16: Head profile of line serving tapstand No. 2 of the Aloua system.

2.3.4 SYSTEM VALVE PLAN

A certain number of valves must be installed in the system. They have different functions and must all be represented in a specific plan.

2.3.4.1 Regulating valves

To ensure that the system functions properly, it is absolutely necessary to maintain flows established during design. For this reason, regulating valves are installed at inlets to every tapstand as well as at inlets to storage tanks and header tanks.

2.3.4.2 Stop valves

Stop valves (such as ball valves) are installed at every major junction to allow isolation of the various branches of the system. This may be necessary in the case of leaks, or for routine maintenance work. Similarly, drain valves are fitted to all storage tanks, header tanks, and break-pressure tanks (Figure 11.17).

These valves are either “normally open” (NO) or “normally closed” (NC), depending on their function: a isolation valve is NO, while a drain valve is NC. They are all located in valve chambers which both protect them and allow easy access.

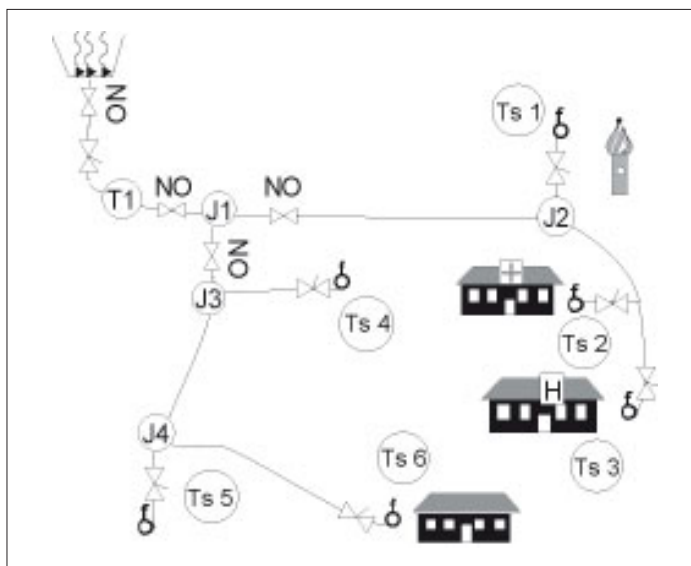


Figure 11.17: Minimum valve plan for the Aloua system (no drains or air bleeds).

3 Construction of the system

The construction of a gravity system involves carrying out a number of different tasks and installing pipework. Recommendations given in this section are based on ACF experience on various projects in Africa and in Asia. Details of construction and civil engineering design are given in Annex 14.

3.1 Header tanks and break-pressure tanks

Tanks are always equipped with:

- a water inlet with a regulator valve (piston type);
- a water outlet with a stop valve (ball type) and air vent;
- a drain;
- an overflow;
- an inspection trap.

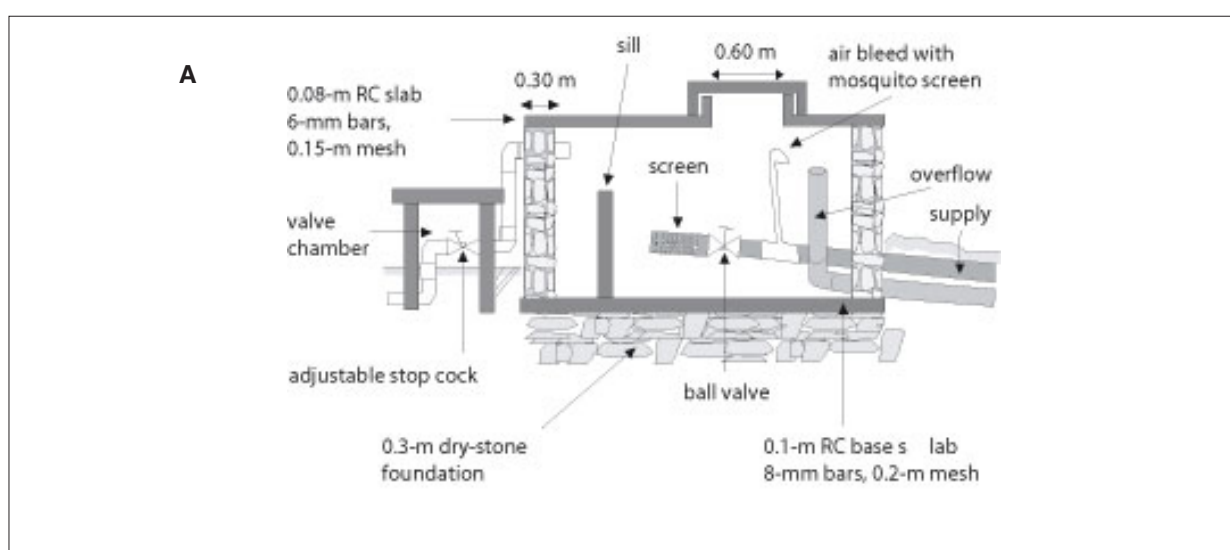


Figure 11.18: Masonry break-pressure tank. A: section.

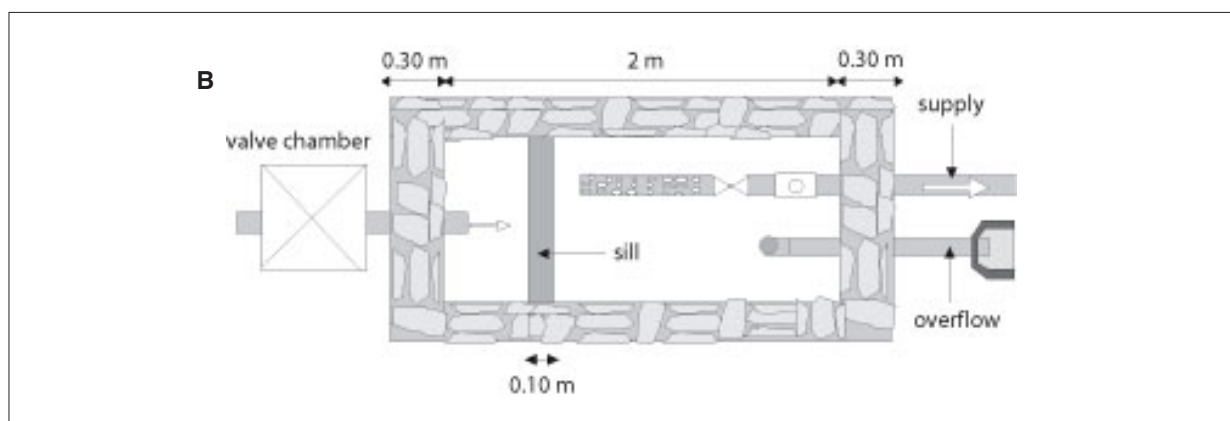


Figure 11.18: Masonry break-pressure tank. B: plan.

The construction can be in masonry or in reinforced concrete (Figures 11.18 & 11.19), the volume is 2 to 3 m³, and of rectangular or cylindrical shape (prefabricated rings are used).

The installation of a sill in break-pressure tanks is recommended to diffuse the dynamic pressure and facilitate good aeration of the water. Also, the depth of the water layer over the sill gives a measure of incoming flow (see Chapter 3).

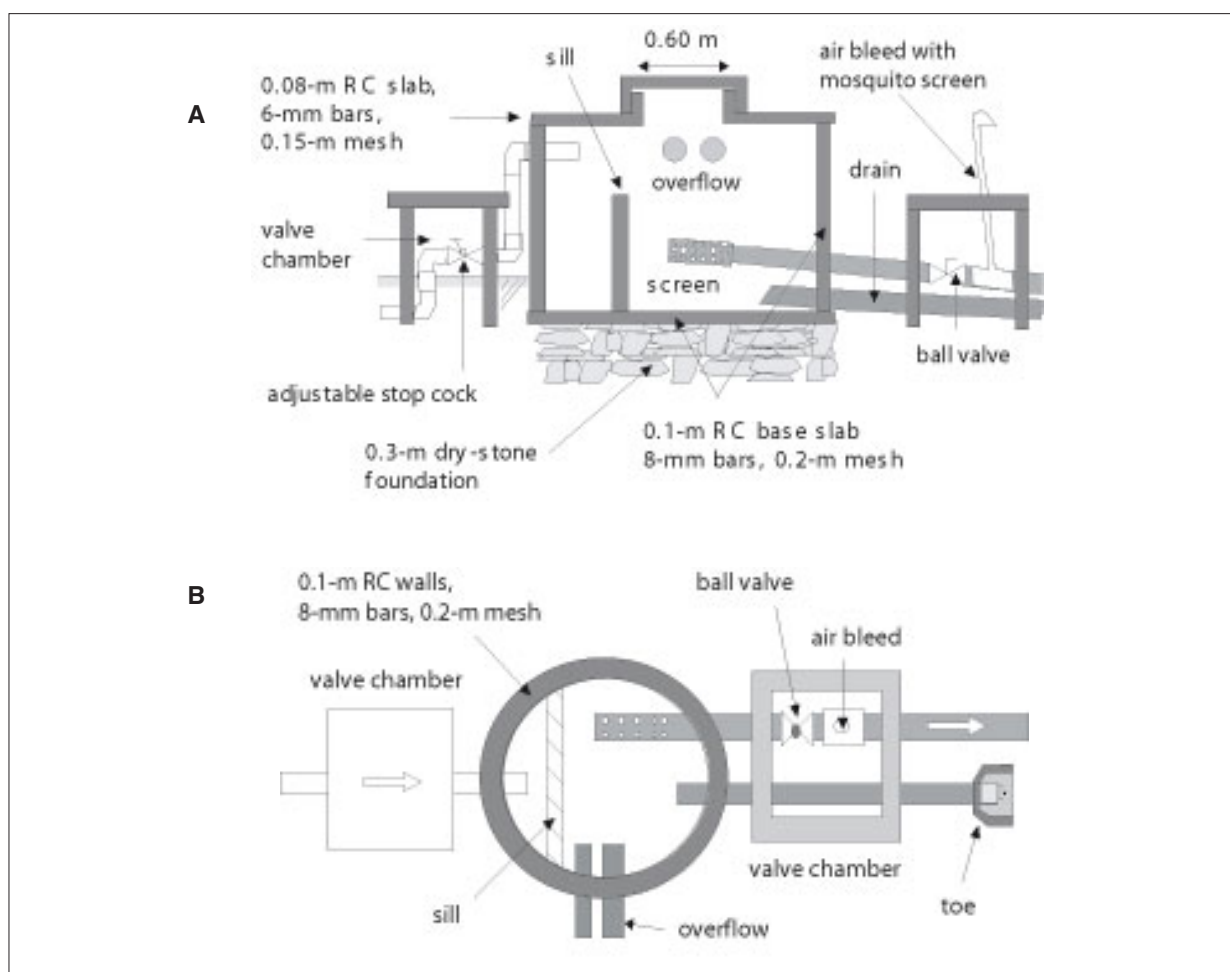


Figure 11.19: Reinforced concrete break-pressure tank.

A: section. B: plan.

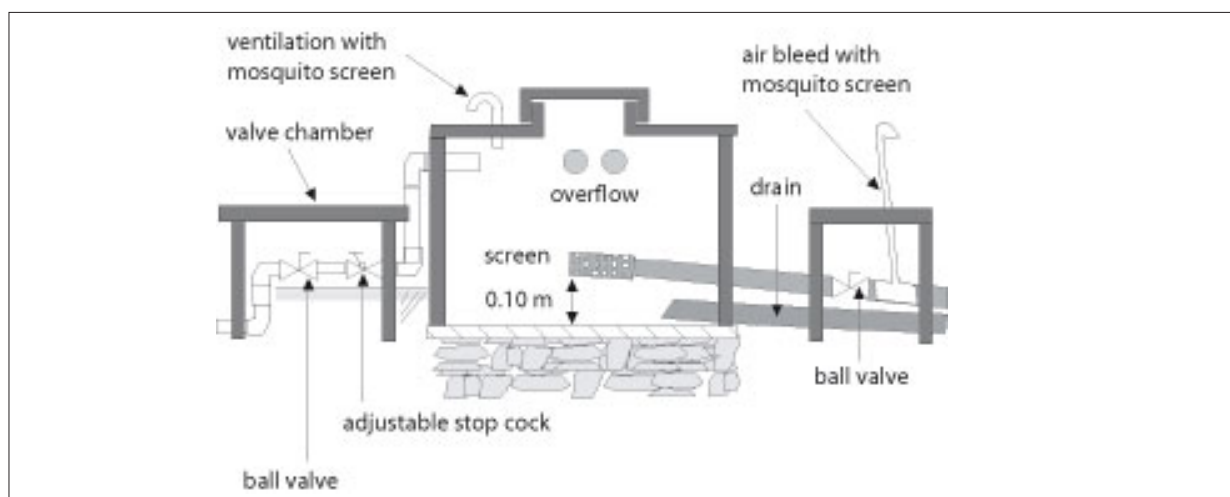


Figure 11.20: Storage tank.

3.2 Storage tank

A storage tank (Figure 11.20) requires:

- a water inlet with adjustable stop cock;
- a water outlet with ball valve and air vent;
- a drain;
- an overflow;
- an inspection trap.

The construction of tanks in reinforced concrete and in masonry is described in Annex 14.

3.3 Pipes

3.3.1 PIPES AND COMPONENTS

3.3.1.1 Equivalent diameters

Nominal diameters of pipe are designated by DN sizes under the ISO system. DN sizes for galvanised steel (GI) pipes refer to the internal diameter (ID), whereas for rigid plastic pipes (PVC and PE) they refer to the outside diameter (OD) (Table 11.VIII).

Table 11.VIII: Equivalent pipe diameters.

PVC/PE pipes		GI pipes	
OD in mm	equivalent diameter OD in inches	ID in inches	equivalent diameter ID/OD in mm
16		3/8"	12/17
20	3/4"	1/2"	15/21
25	1"	3/4"	20/27
32	1"1/4	1"	26/34
40	1"1/2	1"1/4	33/42
50	2"	1"1/2	40/49
63	2"1/2	2"	50/60
75	3"	2"1/2	66/76
90	3"1/2	3"	80/90
110	4"1/2	4"	102/114

3.3.1.2 Pressure classes

Nominal pressure (NP) is usually expressed in bar (NP 6, NP 10, NP 12.5 etc.). Some countries, such as Kenya and India, have their own standards of pressure resistance (see Table 11.IX).

The service pressure is different from the nominal pressure under particular conditions (elevated water temperature, corrosive water etc.). For most programmes, only nominal pressure is used.

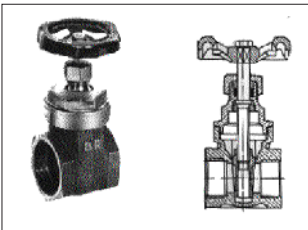
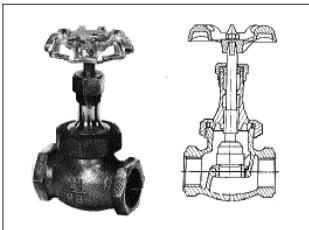
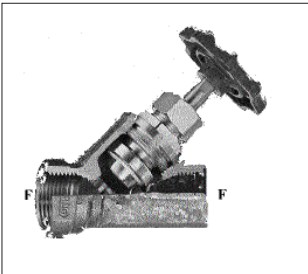
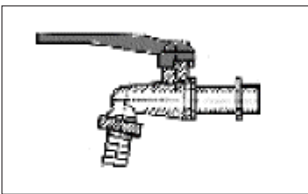
Table 11.IX: Nominal pressure equivalents (NP).

PE pipe NP (bar)	Equivalent class	PVC pipe NP (bar)	Equivalent class	GI pipe NP (bar)
NP 6	Class B or III	NP 6	Class B or III	NP 16
NP 10	Class C or IV	NP 10	Class C or IV	NP 25
NP 12.5	Class D	NP 16	Class E	

3.3.1.3 Valves and taps

The various models of valves and taps are shown in Table 11.X.

Table 11.X: Models of valves and taps.

Model	Name and use	Model
	<i>Gate valve</i> opening/closing possible flow regulation, but stop cock preferable	
	<i>Adjustable stop cock</i> regulation of flow with adjustable maximum opening (concealed stop screw) possible opening/closing, but gate or ball valve preferred	
	<i>Float valve</i> automatic opening/closing according to water level (storage tank, header tank) level control	
	<i>Ball tap</i> public distribution if Talbot not available (with fluted mouthpiece for attaching a hose pipe)	

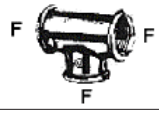
3.3.1.4 GI pipes and fittings

GI pipes are generally available in 6-m lengths. Their properties and the various fittings available are listed in Tables 11.XI and 11.XII.

Table 11.XI: Relationship between internal diameter and weight per metre for galvanised steel pipes.

Diameter (inches)	3/4"	1"	1"1/2	2"	2"1/2	3"
Weight (kg) per m	1.6	2.4	3.6	5	6.5	8.4

Table 11.XII: Fittings for GI pipes.

Model	Name and use	Model
	<i>Socket</i> joining 2 pipes	
	<i>Union</i> joining 2 pipes, disassembly possible	
	<i>45° elbow</i> joining 2 pipes at 45°	
	<i>Reducer</i> female-female reduction	
	<i>Reducer</i> male-female reduction	
	<i>Cap</i> female pipe end	

GI pipes are joined using a screwed socket, also in galvanised steel. Each pipe is threaded at both ends and normally delivered with a socket. Sealing is ensured with Teflon tape for small diameters, and with hemp and jointing compound for diameters above 1"1/2.

Special tools are needed when working with these pipes:

- stock and die for threading pipes (standard die cuts from 1/2" to 2"1/2 with three sets of dies);
- pipe-cutter;
- plumber's vice.

3.3.1.5 PVC pipes and fittings

PVC pipes and fittings are generally available in 6-m lengths, packed in bundles (Figure 11.21). There are various types of joint for PVC pipes (Table 11.XIV):

- rubber joints (Figure 11.22A), generally used for larger diameters (Table 11.XV). It is recommended to use soapy water when connecting pipes with rubber joints in order to ensure that the joint slips fully home;
- glued joints (Figure 11.22B), generally used for small diameters, Table 11.XIII);
- sockets (using cut pipes – Figure 11.22C).

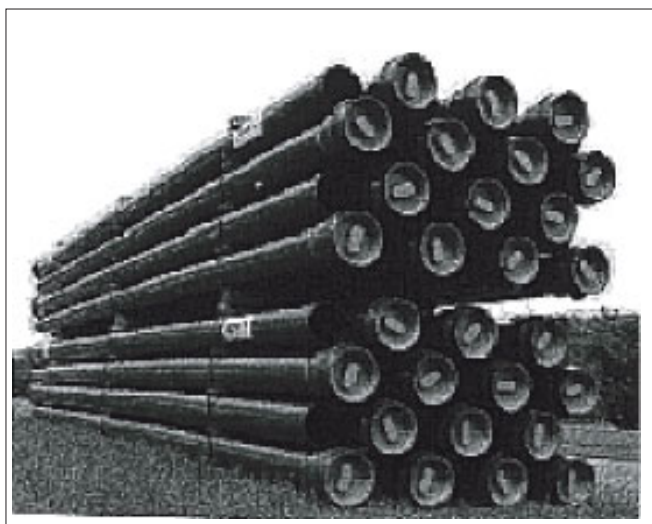


Figure 11.21: PVC pipes with glued joints.

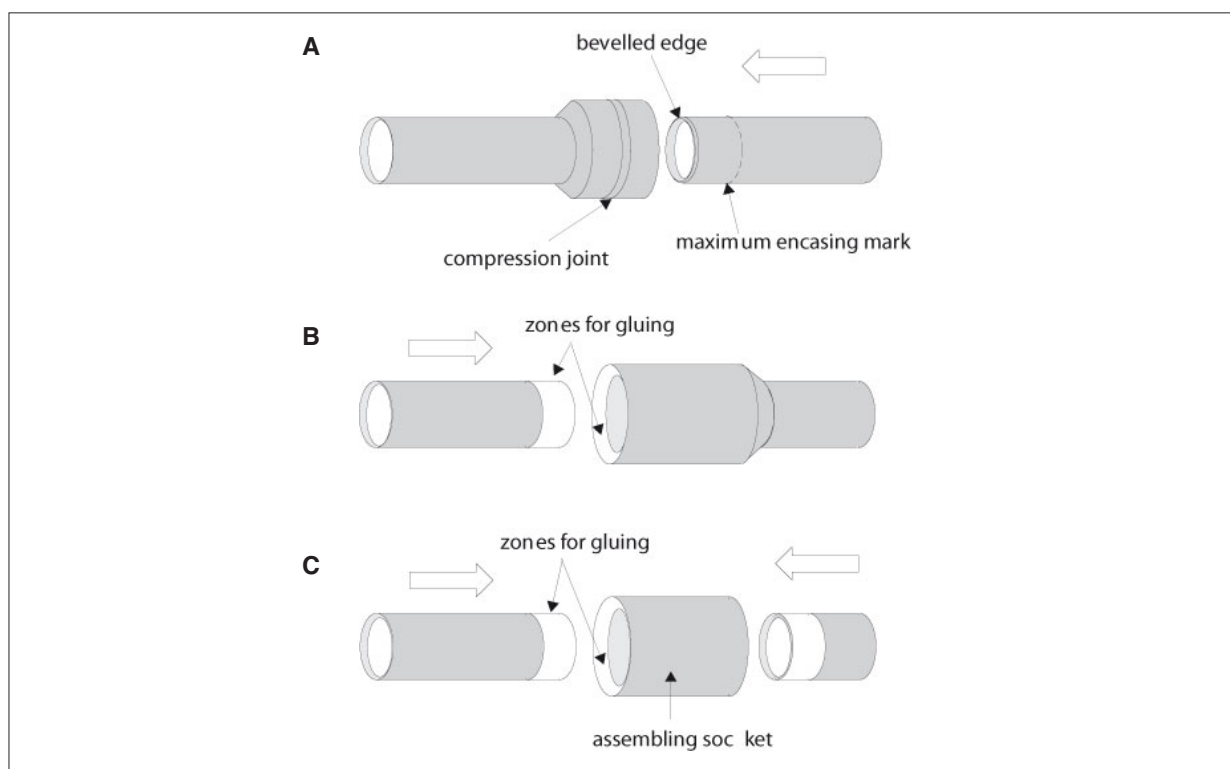


Figure 11.22: PVC pipe joints.

A: rubber joint. B: glued joint. C: socket joint.

Table 11.XIII: PVC pipe with glued joints, NP 10.

Outside diameter (mm)	Internal diameter (mm)	Weight (kg) per m
32	29.2	0.3
40	36.4	0.5
50	45.2	0.8
63	57	1.2
75	69	1.6
90	84	1.8

Table 11.XIV: Fittings for PVC pipes.

Model	Name and use	Model
	<i>Socket</i> joining 2 pipes	
	<i>Union</i> joining 2 pipes, disassembly possible	
	<i>90° elbow</i> joining 2 pipes at 90°	
	<i>45° elbow</i> joining 2 pipes at 45°	
	<i>90° tee</i> joining 3 pipes of the same diameter	
	<i>Reducer</i> joining 2 pipes of different diameters	
	<i>Male adaptor</i> joining PVC-GI, PVC-valves etc.	
	<i>Female adaptor</i> joining PVC-GI, PVC-nipples etc.	
	<i>Hose socket</i> joining PVC-flexible pipes, with hose clamp	
	<i>Plug</i>	

Table 11.XV: PVC pipes with rubber joints.

Outside diameter (mm)	Internal diameter (mm)
63	57
75	69
90	83
110	101.4
125	116.2
140	130.2

3.3.1.6 Polyethylene pipes and fittings

Polyethylene (PE) pipes usually come in 50 or 100-m coils (Figure 11.23), and are available in different qualities: high, medium, and low density (Table 11.XVI). High-density pipe is generally used for drinking water. However, it is more rigid, and medium density PE pipe can be used for connections to tapstands or tanks.



Figure 11.23: PE pipes.

Table 11.XVI: Characteristics of high density and low density PE pipes.

Outside diameter (mm)	Internal diameter (mm)	Weight (kg) per metre
<i>High density, NP 10</i>		
32	26	0.3
40	32.6	0.45
50	40.8	0.7
63	51.4	1.0
75	61.4	1.5
90	73.6	2.2
<i>Low density, NP 10</i>		
25	16.6	0.3
32	21.2	0.45
40	26.6	0.7
50	33.4	1.0
63	42	1.7

Compression couplings are used for connecting PE pipes up to 2½ in diameter (Table 11.XVII & Figure 11.24). For larger diameters, it is preferable to use heat welding.

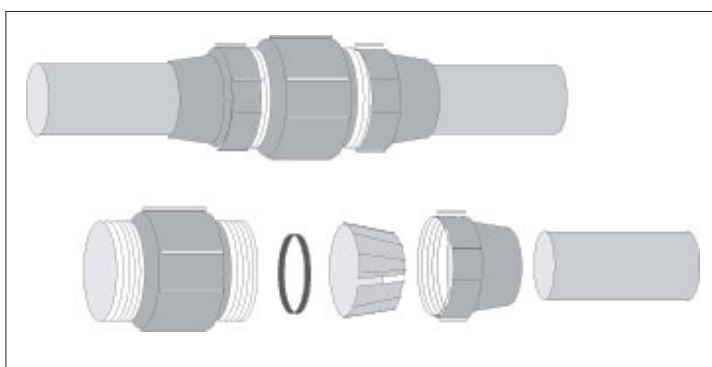
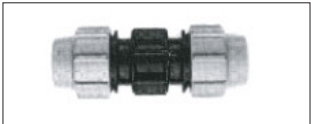
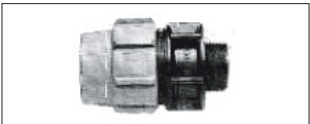
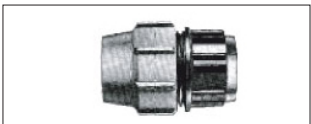


Figure 11.24: PE compression coupling.

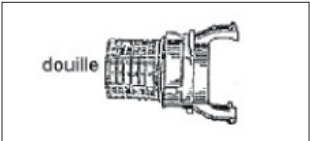
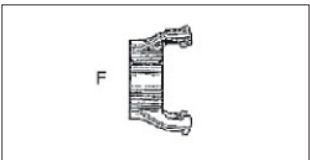
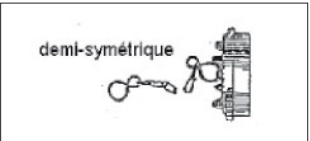
Table 11.XVII: Fittings for PE pipes.

Model	Name and use	Model
	<i>Coupling</i> joining 2 pipes	
	<i>Union</i> joining 2 pipes, disassembly possible	
	<i>90° elbow</i> joining 2 pipes at 90°	
	<i>45° elbow</i> joining 2 pipes at 45°	
	<i>90° tee</i> joining 3 pipes of the same diameter	
	<i>Reducer</i> joining 2 pipes of different diameters	
	<i>Male adaptor</i> joining PE-GI, PE-valves, etc	
	<i>Female adaptor</i> joining PE-GI, PE-nipples, etc	
	<i>Plug</i>	

3.3.1.7 Quick-fit connections

These fittings (Table 11.XVIII) are mainly used in emergency systems, for connections to water tankers, tapstands etc.

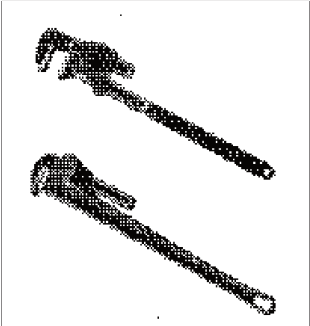
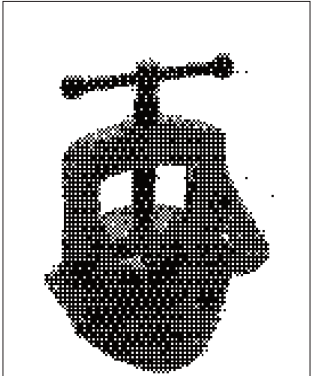
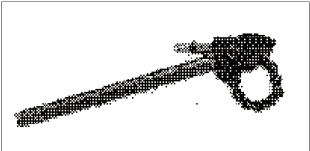
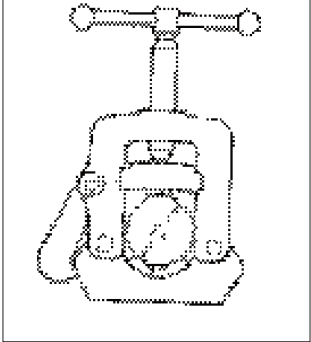
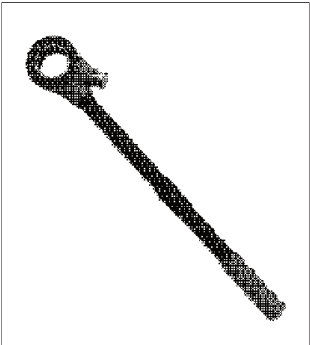
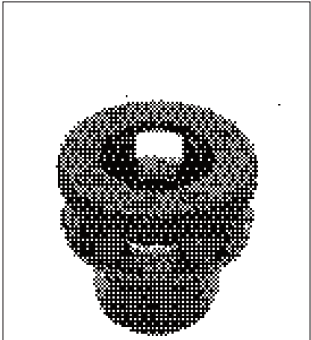
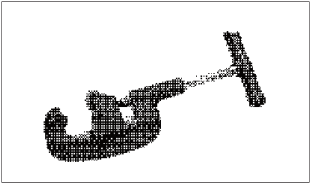
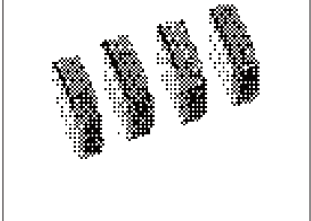
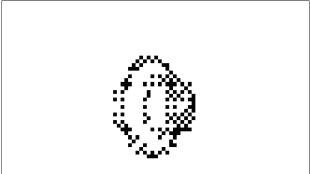
Table 11.XVIII: Quick-fit pipe components.

Model	Name and use	Model
	<i>Quick hose coupling</i> connection to flexible hose with locking collar	
	<i>Quick hose coupling</i> connection to female threaded components (valves etc.)	
	<i>Quick hose coupling</i> connecting to GI pipe or male threaded component (nipple etc.)	
	<i>Wrench</i> for tightening quick-fit joints	
	<i>Dust plug with locking ring</i> plug for semi-symmetrical quick connections	

3.3.1.8 Standard tools

These tools are shown in Table 11.XIX.

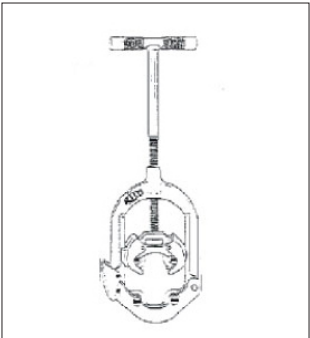
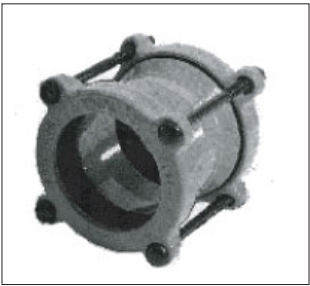
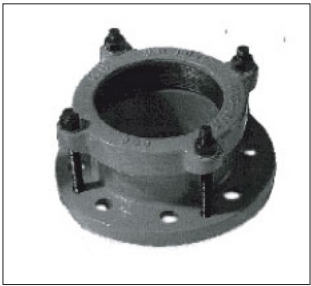
Table 11.XIX: Standard tools for GI pipes.

Model	Name and use	Model
	<i>Stillson pipe wrench</i> <i>Heavy-duty pipe wrench</i>	
	<i>Chain pipe wrench</i>	
	<i>Manual thread cutter with arm</i>	
	<i>GI pipe cutter</i>	
	<i>Pipe cutter wheel</i>	

3.3.1.9 Accessories for repair and connection of cast-iron pipe

These accessories are shown in Table 11.XX.

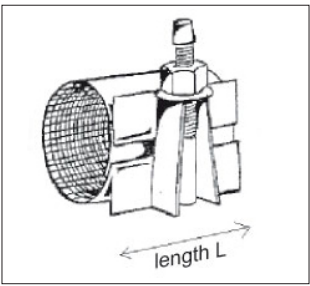
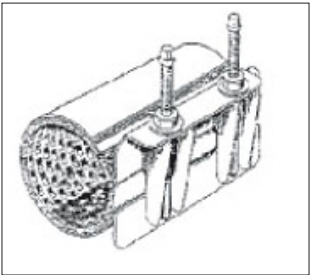
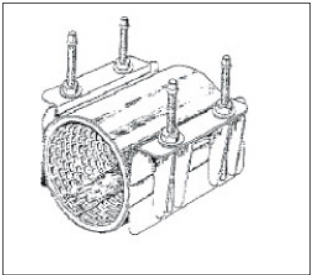
Table 11.XX: Accessories for repair and connection of cast-iron pipe.

Model	Name and use	Model
	<i>Pipe cutter</i> This type of pipe cutter can be used for GI pipes. There are also less expensive two-wheel models which are very convenient for work above ground. For PVC and PE pipes, a metal saw is adequate. There is a 4-wheel model for work with reduced clearance, for instance, in a pipe trench (90°)	
	<i>Repair clamp</i> high-tolerance joint (seals joint with locking pins) for connecting pipes of the same outside diameter, whatever their material or thickness	
	<i>Adaptor</i> same type for adaptation on flange; with a flange washer	

3.3.1.10 Equipment for pipe repair

This equipment is shown in Table 11.XXI.

Table 11.XXI: Equipment for pipe repair.

Model	Name and use	Model
	<i>Repair clamp</i> single model for rapid repair of all type of pipe: asbestos cement, cast-iron, GI, PVC, length 100 mm	
	<i>Repair clamp</i> model identical to the previous one, 200 and 250 mm in length	
	<i>Repair clamp</i> double model; length 200 to 300 mm; 400 mm for 200 - 400 mm dia. pipe; 500 mm for 350 - 500 mm dia. pipe	

3.3.1.11 Equipment for pipes under pressure

This equipment (Table 11.XXII & Figure 11.25) enables work to be carried out on a pipe under pressure without interrupting distribution. It is used particularly when connecting tapstands or private connections.

Table 11.XXII: Equipment for pipes under pressure.

Model	Name and use	Model
	<i>Tapping saddle clamp</i> all types of pipe (PE, PVC, cast-iron, GI, asbestos, cement), delivered with washer	
	<i>Connector tap</i> male or female outlet, gas-fitting thread	
	<i>Tapping saddle clamp</i> for PVC and PE, ductile cast-iron, GI, asbestos and cement	
	<i>Hole cutter</i> equipment for piercing pipe and fitting saddle clamp and connector tap	

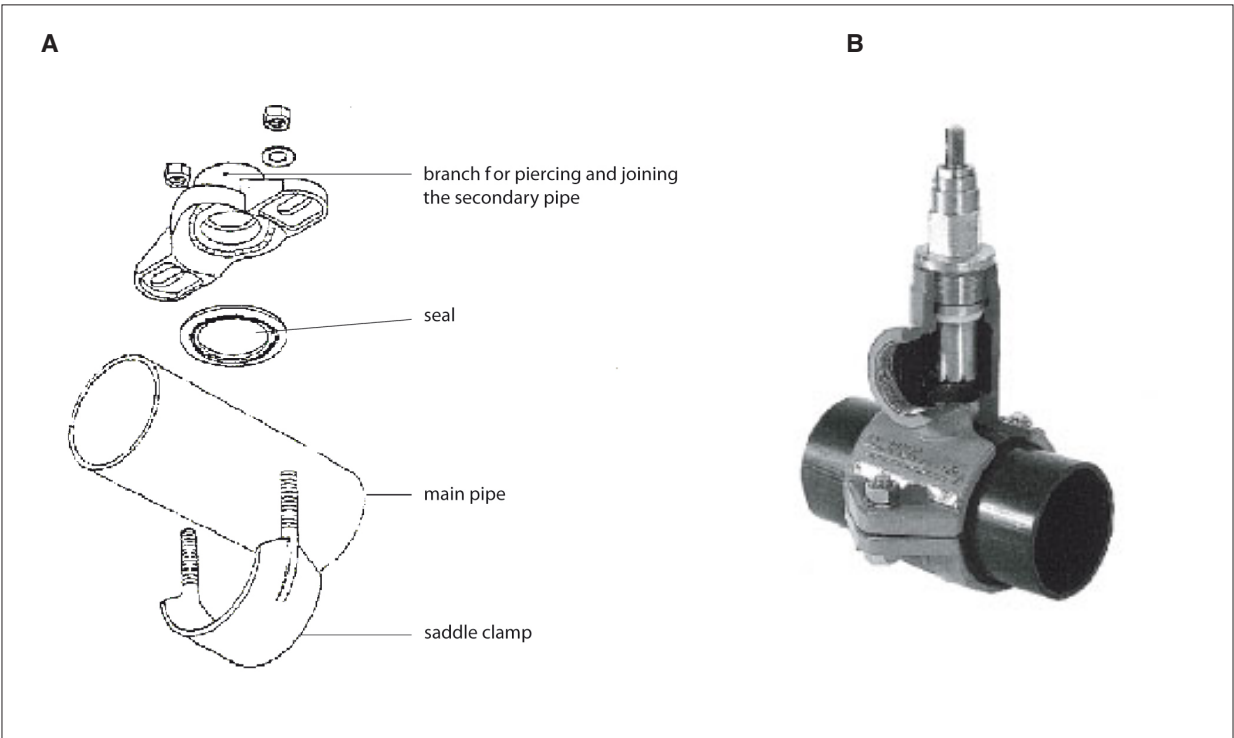


Figure 11.25: Tapping saddle clamp.
A: details. B: with integral cutting valve.

3.3.2 INSTALLING PIPES

3.3.2.1 Trench

- Minimum depth: 0.8 m (width = tool width).
- Remove stones and roots that could damage the pipe.
- Dig the trench just before laying the pipe to avoid all risk of collapse or erosion by flood-water in the event of rain. If this is not possible, place earth dams inside the trench as often as the slope requires.

3.3.2.2 Pipe laying

- If the bottom of the excavation is not flat, or if the earth could damage pipes (e.g. weathered rock), place a layer of sand, or sieved earth if sand is not available, on the bottom of the trench.
- Lay the pipes, joining them carefully (pay attention to problems of pipe expansion, especially when using long polyethylene pipes in high temperatures).

3.3.2.3 Backfilling

- Use the excavated earth for backfilling (remove anything that could damage the pipe).
- After laying the pipe, fill to 30 cm, then compact correctly. If possible, pressurise the system and check for leaks (Figure 11.26A).
- Finish off filling and compaction (Figure 11.26B).

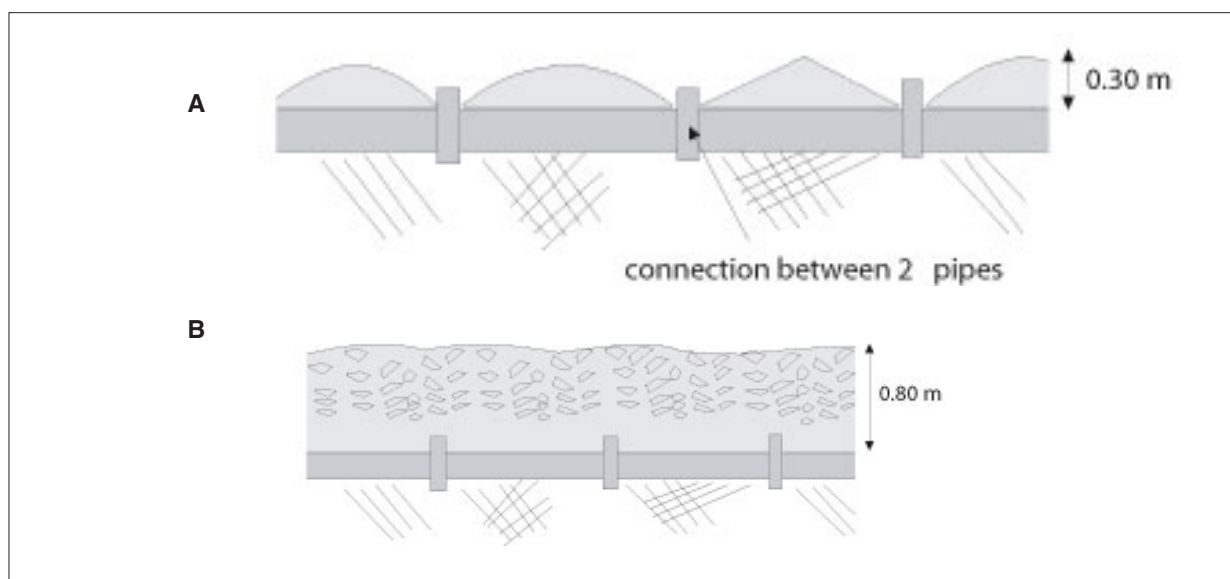


Figure 11.26: Backfilling the pipe trench.

A: initial backfilling to test the pipes (pressurise system). B: final backfilling after pipe test.

3.3.2.4 Passage above ground

When it is not possible to bury the pipe deeper than 30 cm, several other solutions are available (Figure 11.27A):

- use GI pipes;
- put a protective sleeve around the pipe;
- protect plastic pipe with a concrete (10 cm) or masonry covering which may be above ground if necessary.

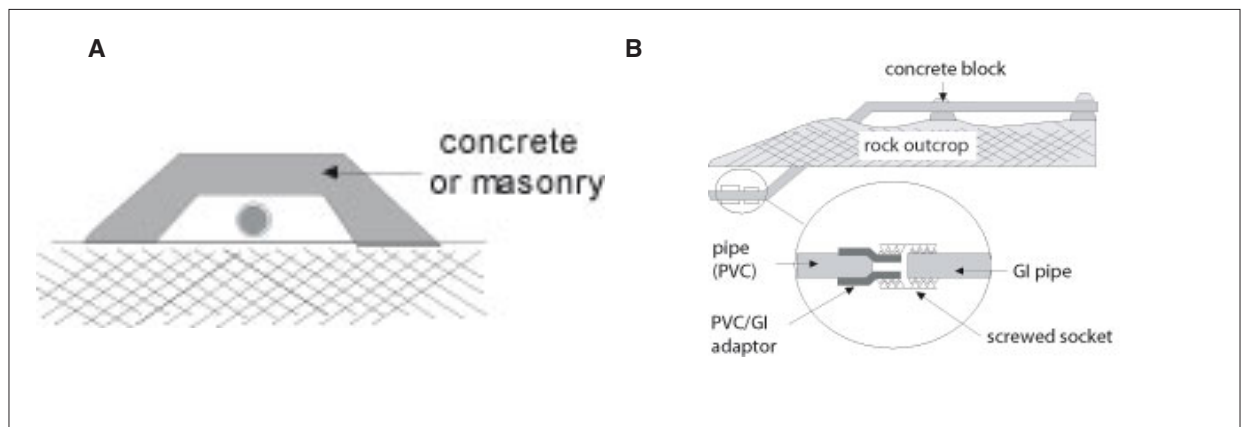


Figure 11.27: Passage above ground.
A: masonry protection for pipe. B: use of GI pipe.

In the case of crossings over gullies, rivers etc.:

- if the span is less than 5 m, the use of GI pipe is recommended. These pipes will easily bear their own weight over a length of 5 m. If the line is in plastic pipe, a GI pipe may be used as a sleeve (anchor the extremities of GI pipe in concrete blocks – Figure 11.28A);
- if the span is greater than 5 m, it is essential to support the pipe with cable.

Table 11.XXIII: Diameter of pipe and maximum span with a 8 mm diameter cable.

Pipe diameter	1"	2"	3"
Maximum span	170 m	70 m	40 m

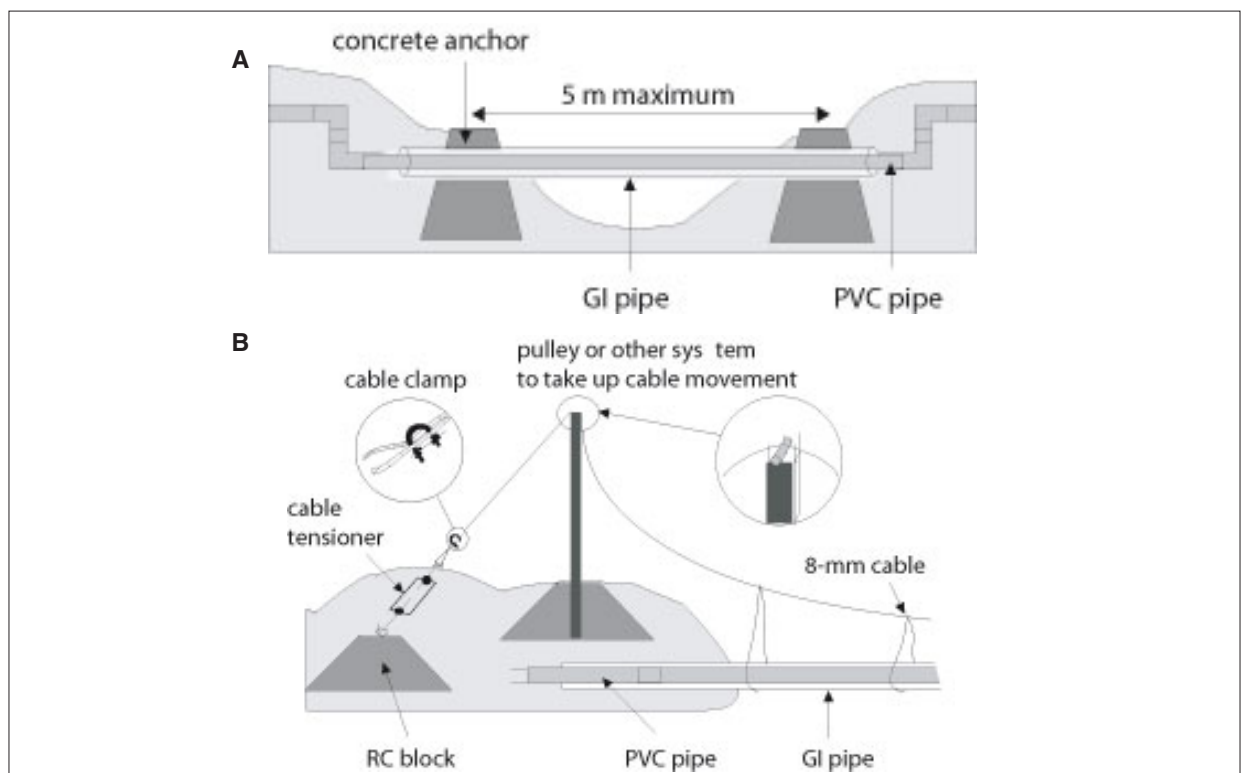


Figure 11.28: Crossing over ravines or rivers.
A: use of a GI sleeve. B: suspended pipe.

The following aspects should be given special attention:

- the cable must be well anchored;
- the PVC pipes must pass inside the GI pipes;
- pipes must be suspended from the cable every 70 cm.

The diameter of the cable is selected according to the length of the span and the diameter of the pipe (Table 11.XXIII & Figure 11.28B).

3.3.2.5 Crossing under a road

- Bury pipe at least 1 m deep.
- Use GI pipe, a GI sleeve, or a concrete conduit.

Sleeve diameters are chosen according to diameters of pipes (Table 11.XXIV).

Table 11.XXIV: Respective diameters of pipes and sleeves.

Plastic pipe	40 mm	50 mm	60 mm	75 mm
GI sleeve	2"	2"1/2	3"	4"

3.3.2.6 Thrust blocks

Thrust blocks (Figure 11.29) are essential to support elbows, tees and valves.

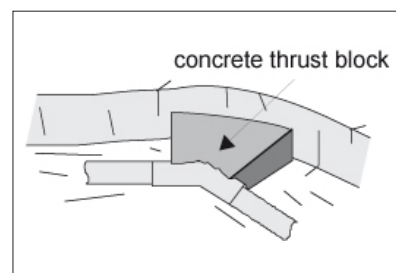


Figure 11.29: Thrust block.

3.3.2.7 Anchors

On very steep slopes (or where the terrain is unstable), the pipe must be anchored (Figure 11.30) to prevent movement when it is filled with water. It is important to use as many anchors as necessary.

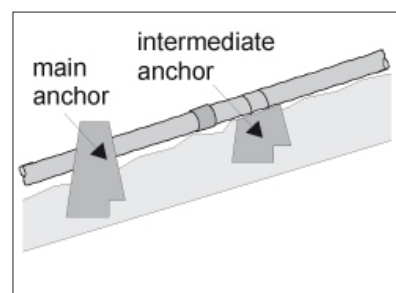


Figure 11.30: Pipe anchors.

3.3.2.8 Location markers

To indicate the position of the pipeline, location markers are placed in concrete or masonry every 250 m, and at specific points (branches, changes of direction etc.). These indicators should be marked with the type of pipe (material and diameter) and direction of flow, plus a reference number (Figure 11.31).

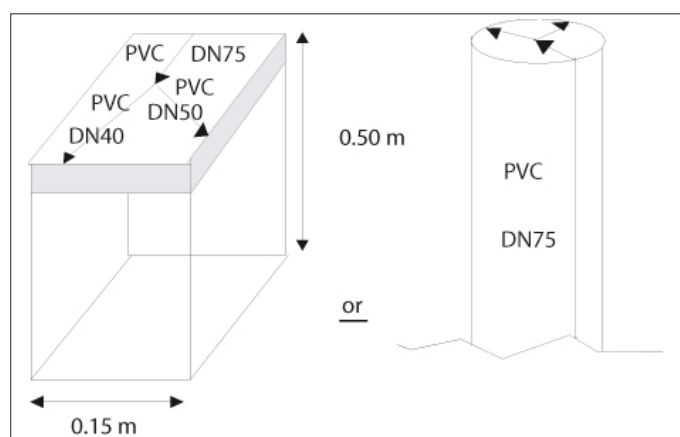


Figure 11.31: Pipeline markers.

3.3.2.9 Drain valves

Ball valves are installed at low points in the line, so as to be able to drain pipes and remove deposits of silt (Figure 11.32).

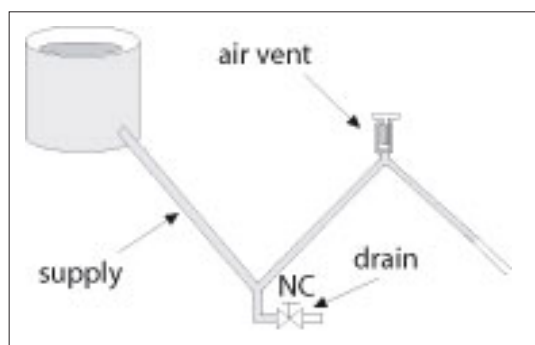


Figure 11.32: Drain and air vent.

3.3.2.10 Air vents

Ball valves are installed at high points in the line, in order to vent accumulated air. These valves are open at the time the system is filled with water, and are then regularly opened when the system is checked.

Air valves, installed at high points, allow automatic removal of air trapped in the pipe, and facilitate maintenance.

3.3.2.11 Valve chambers

All valves, along the line as well as at tapstands, tanks etc., must be protected in valve chambers, and should be:

- fitted with screw-down cover;
- provided with a drain for infiltration water;
- large enough to allow easy disassembly and replacement of the valve.

All valve handles must be removed and retained only by the person responsible for the system, to avoid unauthorised operation.

All valves must be fitted between two unions to permit disassembly without cutting the pipe.

3.4 Tapstands

Tapstands must allow easy and hygienic distribution of water for users of the system.

3.4.1 FITTINGS

It is generally considered that 3/4" taps provide a flow of 0.25 l/s at 10 mWG. Talbot taps cannot reach these flows, and only produce about 0.1 to 0.15 l/s at 5 mWG. In addition, they function correctly only under low pressure, around 1 to 8 mWG. Above 8 mWG they become very difficult to open. However, they have the advantages of closing automatically and being rugged. Their use is therefore recommended, but their correct working range must be borne in mind.

All tapstands must be fitted with a regulating valve (adjustable stop cock) or, better, a pressure regulator. This device enables the service pressure to be pre-set, and therefore ensures optimal operation of the taps. If pressure regulators are installed (strongly recommended), the tapstand must also be fitted with a stop cock or other valve that normally remains open.

3.4.2 CONSTRUCTION

Figures 11.33 and 11.34 show two types of tapstand, in reinforced concrete or masonry, with 1 or 4 taps: details of the structure are given in Annex 14. Many designs are possible and they must be adapted to the cultural context. As it is a meeting place, the tapstand is a social facility that deserves special attention in both design and structure. Users must be involved in its development, as this is

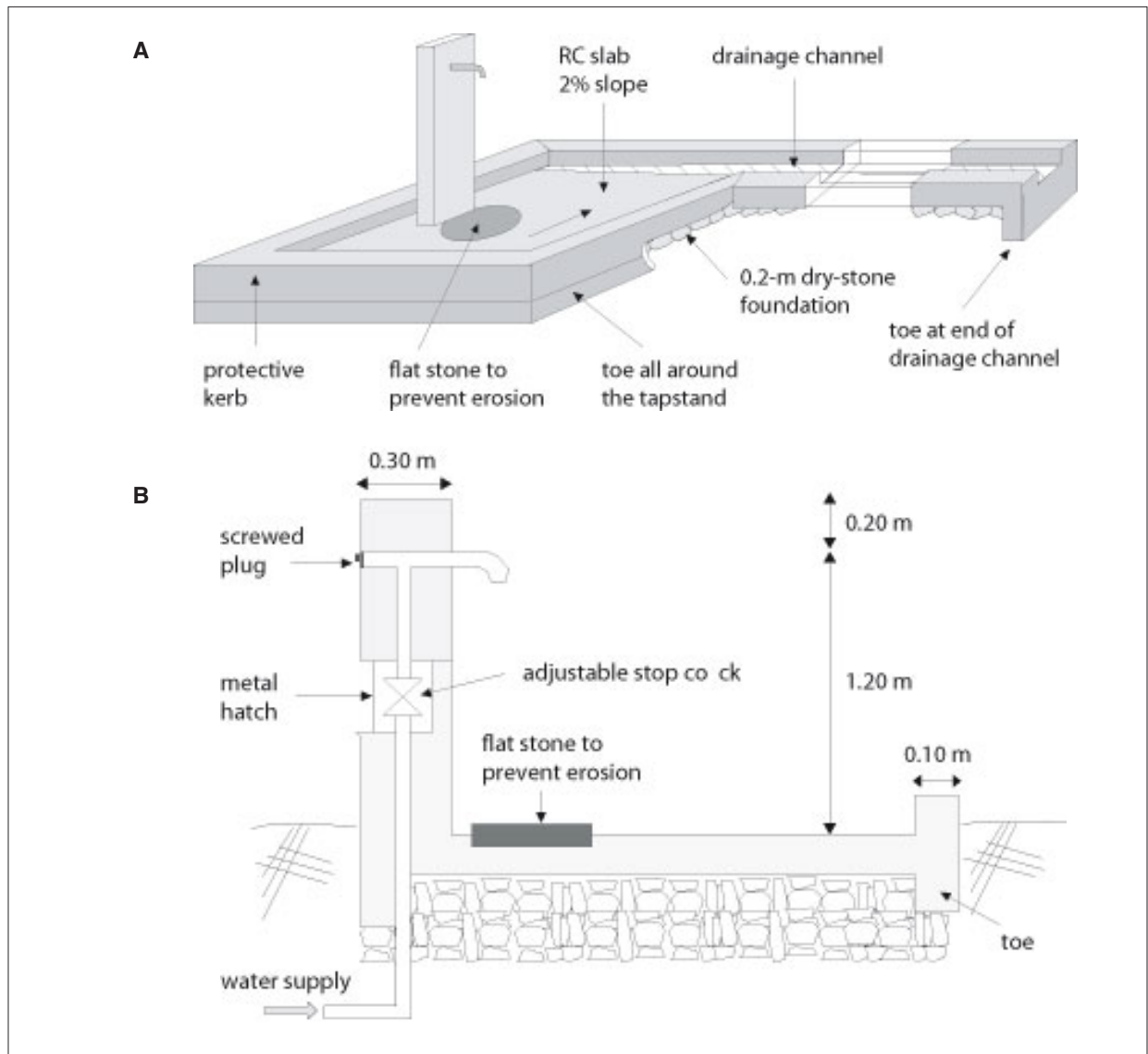


Figure 11.33: Reinforced-concrete tapstand with one tap.

A: general view. B: section.

the only guarantee of basing its design on local preferences for water-drawing (shape of containers, height of taps, washing habits etc.). An example of a tapstand design developed with a local community in Ethiopia is given Chapter 10B.

It is important to pay attention to the following technical points:

- set a flat stone where water flows onto the ground, to avoid rapid erosion of the concrete slab;
- create a definite slope in the slab toward the drainage channel (about 2%);
- make the drainage channel an integral part of the slab design, make it as long as necessary, and finish it with a toe;
- if necessary, install a barrier to prevent access to the tapstand by animals;
- ensure the slab is raised at least 10 cm above ground level to prevent ingress of surface water.

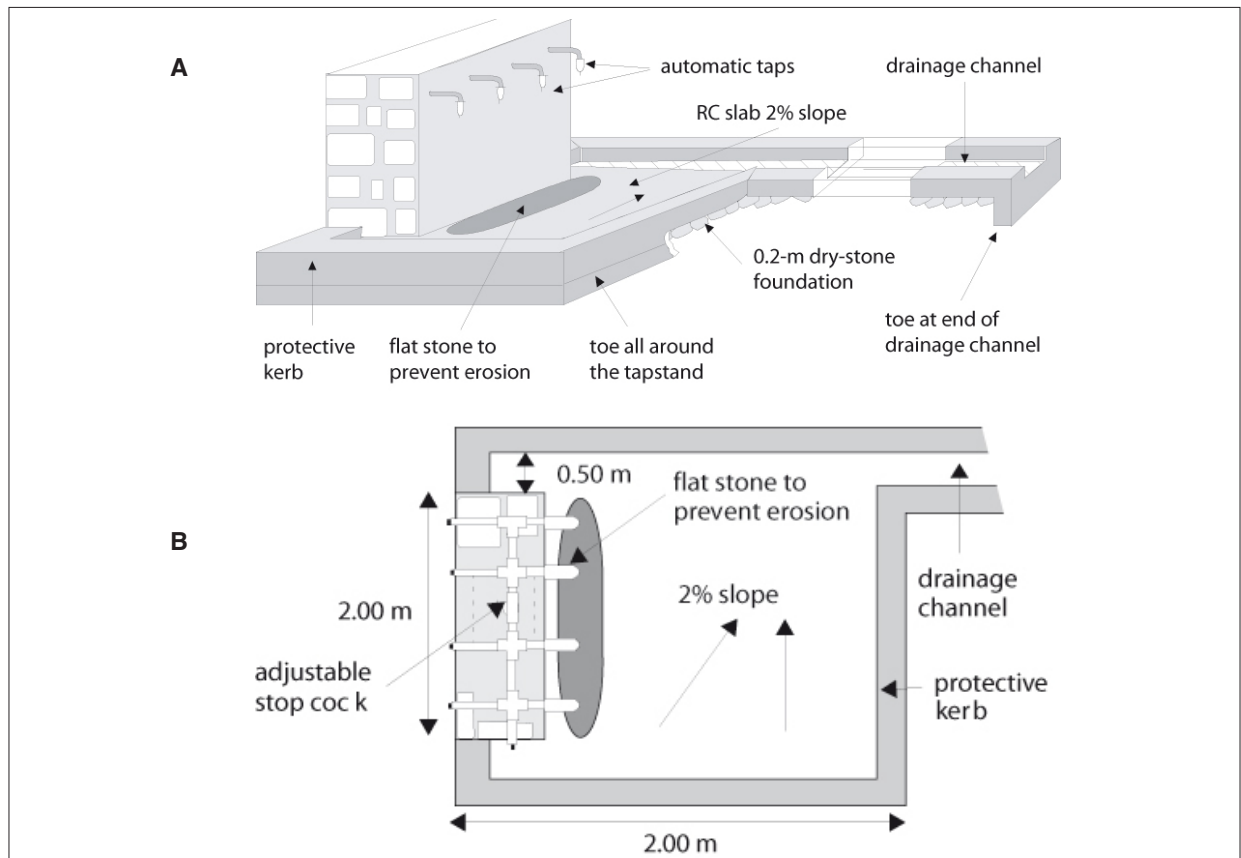


Figure 11.34: Masonry tapstand with 4 taps.
A: general view. B: plan.

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1 Gravity-flow water systems on spring catchments

Projects must be planned and implemented in collaboration with the community and its leaders. It is important to take into consideration the special characteristics of each community and to plan the development of the project with the local population.

1.1 Planning

The time necessary for the construction of a gravity-flow system depends on numerous factors (Table 11.XXV): participation of the community, length and complexity of the system, weather conditions (important above all for digging the trench) etc.

Work planning for the Aloua system shown in Chapter 11A is given in Table 11.XXVI.

Table 11.XXV: Example of time required for the main works in the construction of a gravity-flow water system.

Spring catchment	5 weeks – 10 persons
Constructing storage tank	1 month – 8 persons
Constructing tapstand	1 to 2 weeks – 5 persons
Digging trench (80 cm)	5 m/person/day
Installing pipes	50 m/person/day
Backfilling trench	8 m/person/day

Table 11.XXVI: Timetable for the implementation of the Aloua system.

Month	Activities
1	Field survey (resources, demand) on site, contact with the community Plan of the village Topography Design calculations
2	Purchase of materials and equipment (pipes, cement, moulds etc.) Recruitment of labourers Planning community participation programme
3	Spring catchment Construction of particular structures (header / sedimentation tanks, break-pressure tanks etc.) Implementation of community participation programme
4	Construction of storage tank Continuation of community participation programme
5	Construction of 3 tapstands Continuation of community participation programme
6	Construction of 4 tapstands Start of trench-digging, installation of pipes, partial backfilling Continuation of community participation programme
7	Completion of construction of tapstands Completion of pipe installation, backfilling trench Continuation of community participation programme
8	Filling pipes with water Repairing leaks Completion of site (fencing around tapstands etc.) Evaluation, new proposals

1.2 Human and financial resources

The skilled jobs for the construction of a gravity-flow system are pipe-laying and civil engineering, which may or may not be sub-contracted. When working directly, two teams are used, one for plumbing (connection and installation of the pipes) and the other for civil engineering (Table 11.XXVII). Community members can be involved in some of the tasks, particularly:

- clearing the pipe routes and transport of materials along the line;
- digging the trenches;
- managing wastewater from tapstands and erecting fences or hedges around the spring and tapstands.

Table 11.XXVII: Personnel required for the construction of a distribution system.

<i>Management</i>	
1 water technician	Design, planning, management of the teams
1 logistician	Supplies to the site, follow-up of material and vehicles
1 foreman	Overseeing plumbing and brickwork teams
<i>Masonry team</i>	
1 mason foreman	In charge
2 assistant masons	Preparation, concrete work
<i>Plumbing team</i>	
1 plumber foreman	In charge of installation and connection of pipes
2 assistant plumbers	Laying and connection of pipes
<i>Labourers</i>	
Depending on demand	Trenches, building, plumbing

Table 11.XXVIII shows estimates of the cost of materials and equipment in 1997.

Table 11.XXVIII: Average cost (in euros) of materials and equipment purchased in France in 1997.

<i>Piping</i>					
GI (per 6 m)	PVC, NP 10 (per m)		HDPE, NP10 (per m)		
3/4"	18.3 €	32 mm	0.5 €	32 mm	1.5 €
1"	25.2 €	40 mm	0.8 €	40 mm	2.3 €
2"	50.3 €	50 mm	1.2 €	50 mm	3.4 €
3"	86.9 €	75 mm	2.4 €	75 mm	7.6 €
		90 mm	3.2 €	90 mm	10.7 €
		110 mm	4.6 €		
<i>Fittings</i>					
GI		PVC		PE	
2" socket	7.6 €	2" socket	2.0 €	2" socket	13.0 €
2" nipple	4.6 €	2" union	6.9 €	2" union	15.2 €
2" union	14.5 €	90° 2" elbow	3.4 €	90° 2" elbow	17.5 €
90° 2" elbow	5.9 €	90° 2" tee	4.0 €	90° 2" tee	19.1 €
90° 2" tee	8.8 €	2-1" reducing socket	2.1 €	2-1" reducing socket	13.0 €
2-1" reducing socket	6.4 €				
<i>Miscellaneous</i>					
Tapping saddle clamp	50		15.2 €		
Repair clamp (100 mm)			7.6 €		
Repair clamp (200 mm)			61 €		
High tolerance connection DN 40 to 300			25.9 to 99.1 €		
<i>Equipment</i>					
		Quantity		Price	
Diesel pump and accessories		1		1 829.3 €	
Rapid land survey kit		1		609.8 €	
Proportional dosing device (1 to 20 m ³ /h)		1		1 524.4 €	
Analysis kits:					
– bacteriological analysis + consumables		1		2 134.4 €	
– aluminium analysis		1		101.2 €	
Small tools and equipment		1		762.2 €	
Aluminium sulphate (50 kg)		10		762.2 €	
HTH (kg)		10		152.4 €	

1.3 Example of the Ban Houn system

The following example shows a gravity distribution system from a spring catchment in Laos, for the village of Ban Houn (ACF, 1998). Figures 11.35 and 11.36 show plans for the tanks built for this system (Box 1).

When villages are inaccessible, especially for the transport of materials such as cement and reinforcing steel, it is possible to use pre-fabricated polyethylene tanks. They can be buried and protected by a fenced-off area on the surface (Figure 11.37).

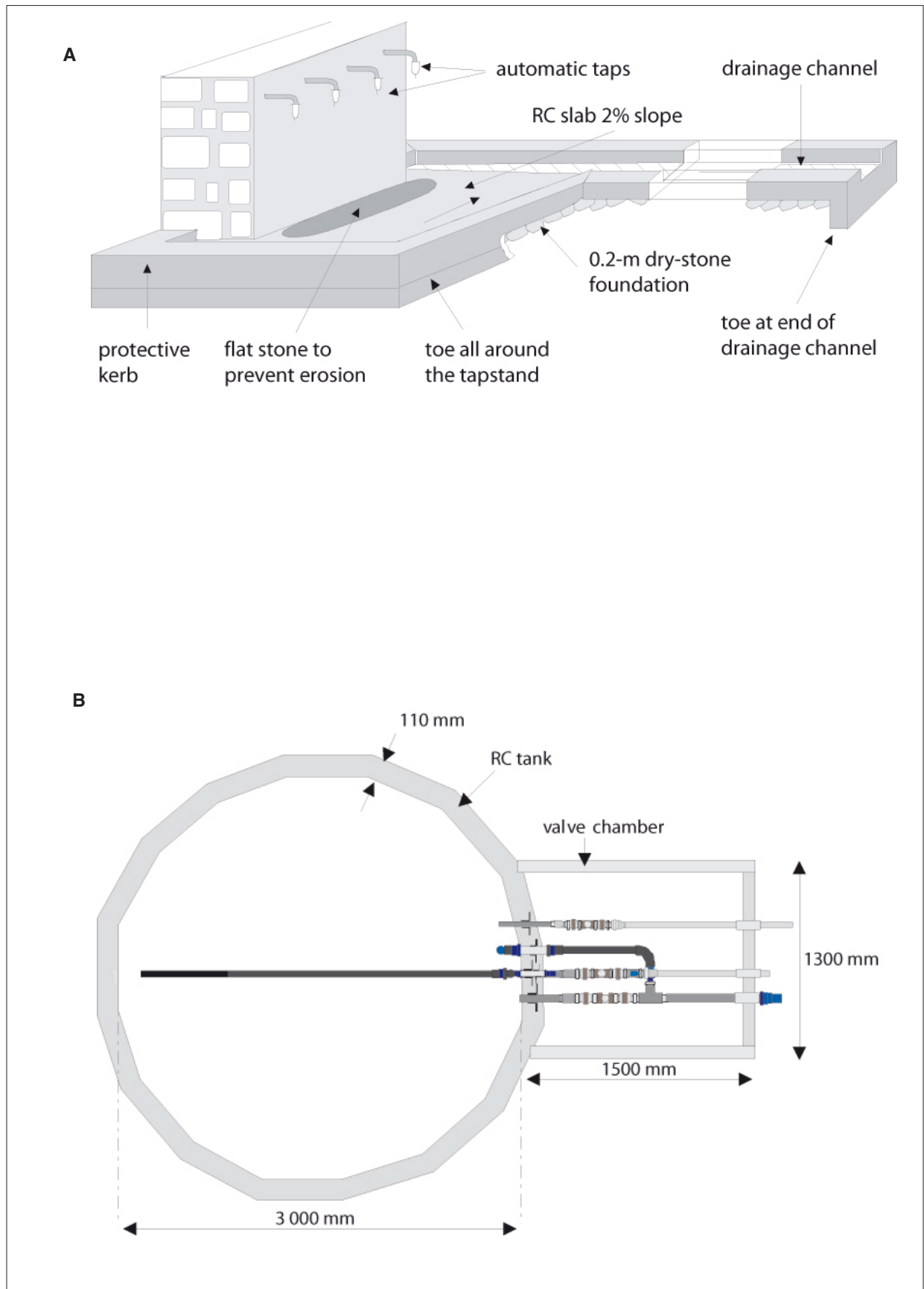


Figure 11.35: Reinforced concrete tank (ACF, Laos, 1998).

A: section. B: plan.

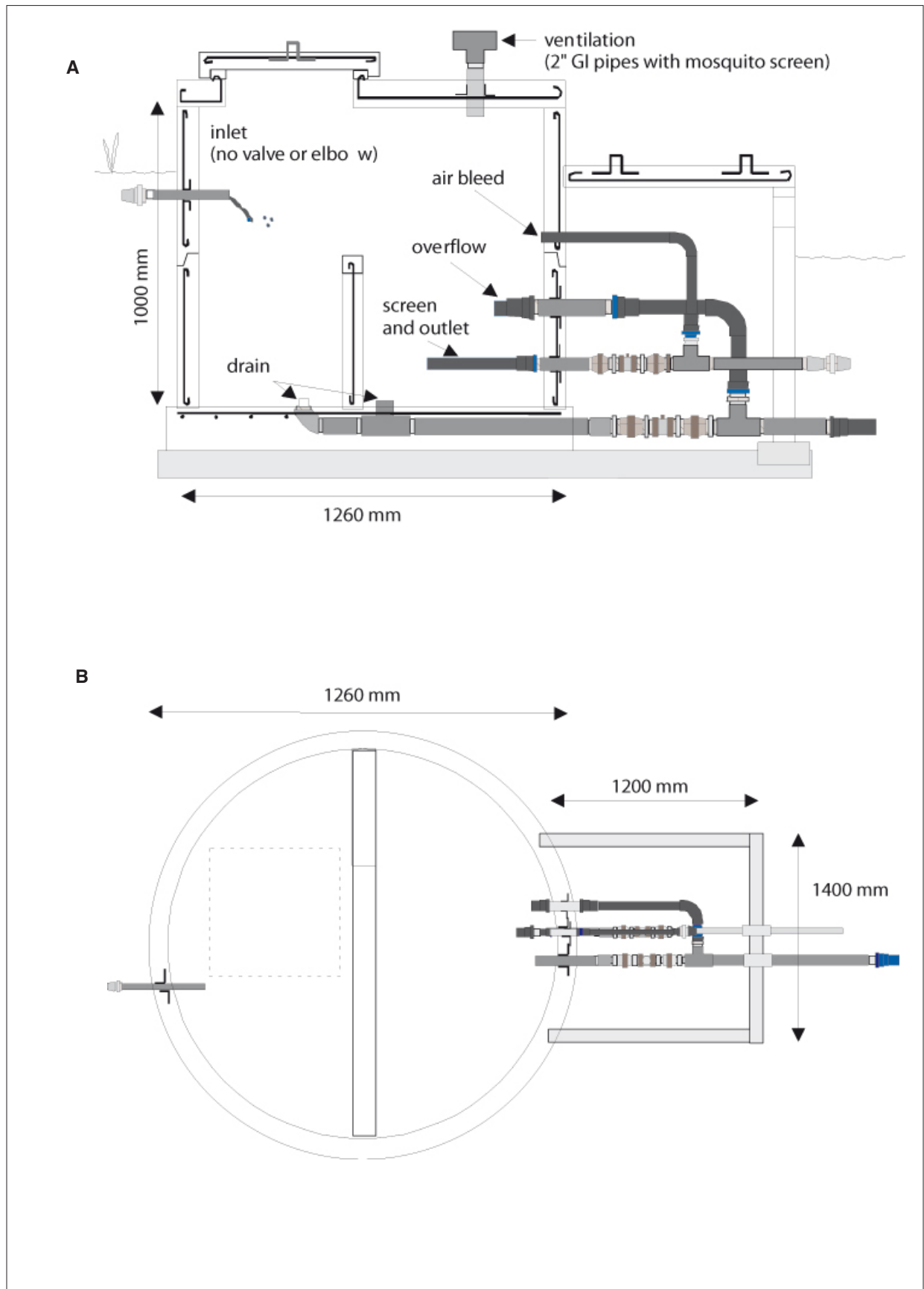


Figure 11.36: Header tank.
A: section. B: plan.

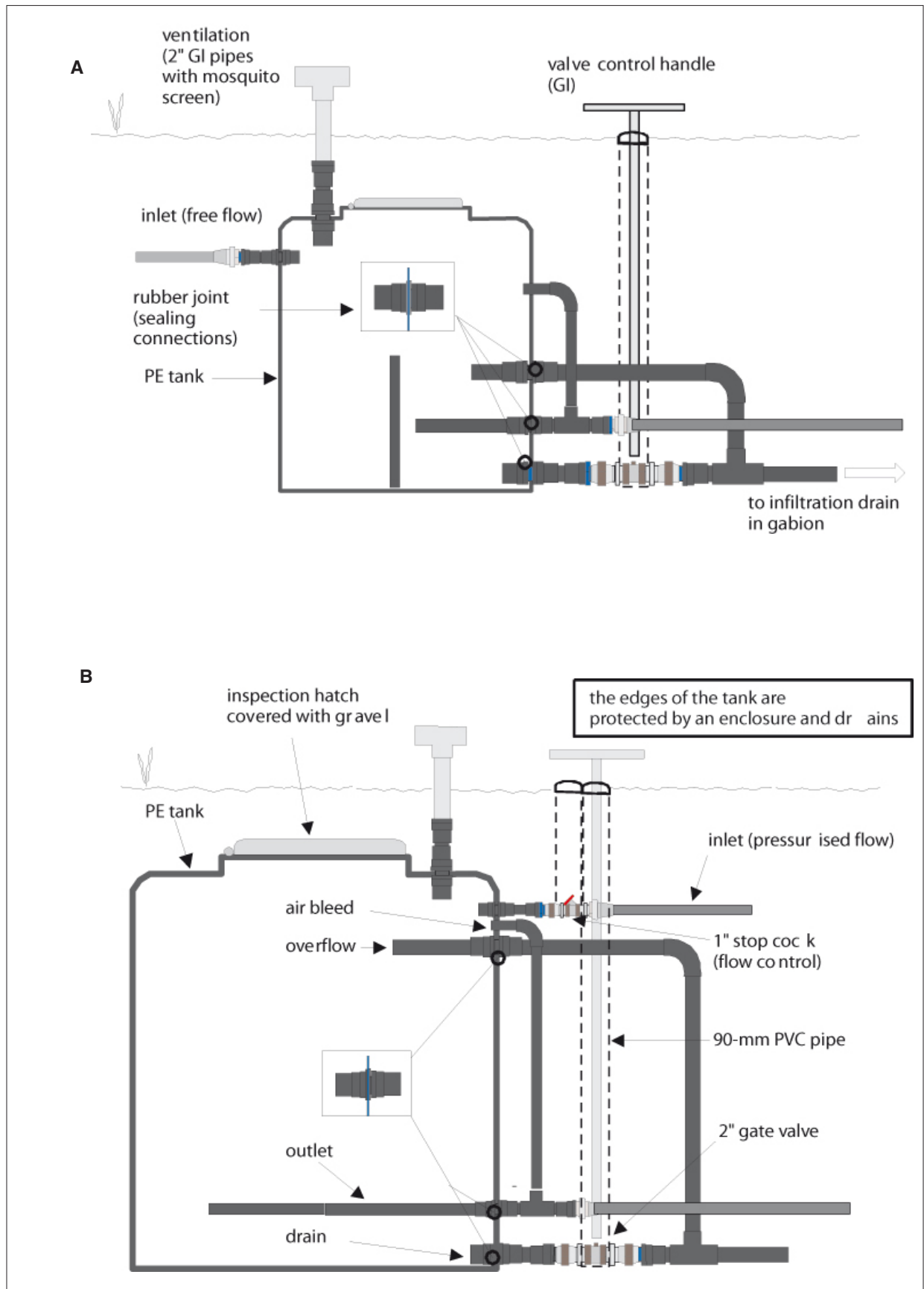


Figure 11.37: Polyethylene tank.
A: header. B: tank.

Box 11.2

Design of the Ban Houn system.

Site: Ban Houn

Date: 22/07/98

Population growth rate: 3%

Analysis of the situation

	Current population	Population in 10 years	Demand l/person/day	Daily demand (m ³ /day)	Number of taps
Population	123	159.9	40	6.396	2
Health centre		0	10	0	0
Hospital		0	50	0	0
Market		0	10	0	0
Temple		0	10	0	0
School		0	0	0	0

Daily demand: 6.40 m³/day. Number of taps: 2. Spring yield (dry season): 0.075 l/s.

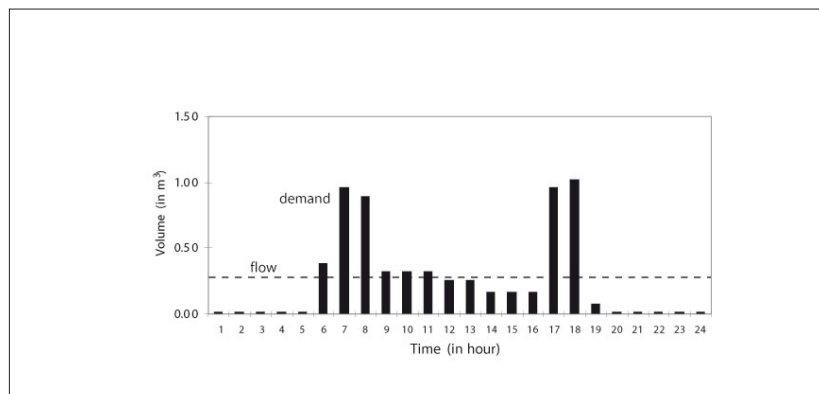


Figure 1: Comparison of hourly demand and spring flow.
Spring flow (l/h) < hourly demand → storage is required (tank).

Volume of tank

Flow of spring (dry season): 0.27 m³/h. Total demand: 6.40 m³/day.

Time of day	Coefficient of consumption (%) during the period	Demand during the period (m ³)	Volume of water produced by the spring during the period (m ³)	Difference (m ³)	Negative stock	Positive stock	Accumulated stock (m ³)
1	0.2	0.01	0.27	0.26	0.00	0.26	0.26
2	0.2	0.01	0.27	0.26	0.00	0.26	0.51
3	0.2	0.01	0.27	0.26	0.00	0.26	0.77
4	0.2	0.01	0.27	0.26	0.00	0.26	1.03
5	0.2	0.01	0.27	0.26	0.00	0.26	1.29
6	6.0	0.38	0.27	- 0.11	- 0.11	0.00	1.17
7	15.0	0.96	0.27	- 0.69	- 0.69	0.00	0.48
8	14.0	0.90	0.27	- 0.63	- 0.63	0.00	- 0.14
9	5.0	0.32	0.27	- 0.05	- 0.05	0.00	- 0.19
10	5.0	0.32	0.27	- 0.05	- 0.05	0.00	- 0.24
11	5.0	0.32	0.27	- 0.05	- 0.05	0.00	- 0.29
12	4.0	0.26	0.27	0.01	0.00	0.01	- 0.28
13	4.0	0.26	0.27	0.01	0.00	0.01	- 0.26
14	2.6	0.17	0.27	0.10	0.00	0.10	- 0.16
15	2.6	0.17	0.27	0.10	0.00	0.10	- 0.06
16	2.6	0.17	0.27	0.10	0.00	0.10	0.05
17	15.0	0.96	0.27	- 0.69	- 0.69	0.00	- 0.64
18	16.0	1.02	0.27	- 0.75	- 0.75	0.00	- 1.40
19	1.2	0.08	0.27	0.19	0.00	0.19	- 1.20
20	0.2	0.01	0.27	0.26	0.00	0.26	- 0.94
21	0.2	0.01	0.27	0.26	0.00	0.26	- 0.69
22	0.2	0.01	0.27	0.26	0.00	0.26	- 0.43
23	0.2	0.01	0.27	0.26	0.00	0.26	- 0.17
24	0.2	0.01	0.27	0.260	0.00	0.26	0.08
TOTAL	100.0	6.40	6.48	0.08	- 3.02	3.10	

Minimum tank volume: 2.68 m³. Optimum tank volume): 2.77 m³. Volume selected: 3.50 m³.

Topographic survey, Abney level

Province: Luang Namtha

District: Nale

Site: Ban Houn

Date: 19/07/98

Station	Distance to ground between stations	Distance to ground cumulative	Vertical angle (decimal degrees)	Vertical angle (degrees & minutes)	Vertical distance (m)	Elevation (m)	Comments
0		0.0				626.0	Catchment
1	19.8	19.8	- 6.67	- 6° 40'	- 2.30	623.7	Header tank (HT)
2	29.6	49.4	- 6.17	- 6° 10'	- 3.18	620.5	
3	22.0	71.4	- 8.17	- 8° 10'	- 3.13	617.4	
4	17.8	89.2	- 7.17	- 7° 10'	- 2.22	615.2	
5	22.5	111.7	- 7.17	- 7° 10'	- 2.81	612.4	
6	22.3	134.0	- 7.00	- 7° 00'	- 2.72	609.7	
7	20.4	154.4	- 0.67	- 0° 40'	- 0.24	609.4	
8	28.1	182.5	- 3.33	- 3° 20'	- 1.63	607.8	
9	23.0	205.5	- 2.67	- 2° 40'	- 1.07	606.7	
10	19.4	224.9	- 1.67	- 1° 40'	- 0.56	606.1	
11	23.6	248.5	- 3.33	- 3° 20'	- 1.37	604.8	
12	24.7	273.5	- 13.17	- 13° 10'	- 5.63	599.1	
13	25.1	298.3	- 19.17	- 19° 10'	- 8.24	590.9	
14	25.3	323.6	- 19.33	- 19° 20'	- 8.38	582.5	Break-pressure tank (BPT)
15	19.8	343.4	- 20.17	- 20° 10'	- 6.83	575.1	
16	21.3	364.7	- 18.67	- 18° 40'	- 6.82	568.9	
17	9.1	373.8	- 15.17	- 15° 10'	- 2.38	566.5	
18	22.8	396.6	- 8.67	- 8° 40'	- 3.44	563.1	
19	27.0	423.6	- 18.00	- 18° 00'	- 8.34	554.7	
20	27.8	454.4	- 19.67	- 19° 40'	- 9.36	545.4	
21	30.0	481.4	- 10.33	- 10° 20'	- 5.38	540.0	
22	22.2	503.6	- 8.33	- 8° 20'	- 3.22	536.8	
23	22.3	525.9	- 8.00	- 8° 00'	- 3.10	533.7	Storage tank Junction
24	20.2	546.1	- 7.33	- 7° 20'	- 2.58	531.1	
25	25.9	572.0	- 11.17	- 11° 10'	- 5.02	526.1	
26	22.0	594.0	- 18.33	- 18° 20'	- 6.92	519.2	
27	29.3	623.3	- 20.83	- 20° 50'	- 10.42	508.7	
28	28.0	651.3	- 18.67	- 18° 40'	- 8.96	499.8	
29	17.4	668.7	- 19.50	- 19° 30'	- 5.81	494.0	
30	17.5	686.2	- 22.67	- 22° 40'	- 6.74	487.2	
31	24.2	710.4	- 21.67	- 21° 40'	- 8.93	478.3	
32	20.5	730.9	- 19.83	- 19° 50'	- 6.96	471.3	
33	21.2	752.1	- 21.17	- 21° 10'	- 7.65	463.7	
34	30.0	782.1	- 18.33	- 18° 20'	- 9.44	454.2	
35	30.0	812.1	- 1.17	- 1° 10'	- 0.61	453.6	Tapstand 1 (T1) Storage tank Tapstand 2 (T2)
36	8.0	820.1	- 2.67	- 2° 40'	- 0.37	453.3	
37	30.0	850.1	- 2.67	- 2° 40'	- 1.40	451.9	
38	30.0	880.1	- 4.83	- 4° 50'	- 2.53	449.3	
39	6.7	886.8	- 0.83	- 0° 50'	- 0.10	449.2	
40	30.0	916.8	- 1.00	1° 00'	0.52	449.8	
41	11.2	928.0	- 0.50	- 0° 30'	- 0.10	449.7	
42	8.0	68.0	0.00		- 0.37	453.3	
43	30.0	98.0	0.00		- 4.20	449.1	
44	30.0	128.0	0.00		- 3.10	446.0	

Head-losses

Station 1	Station 2	Flow (l/s)	Pipe length (m)	Pipe dia (mm)	Friction coefficient (%)	Speed (m/s)	Head- losses (m)	Station altitude 1 (h1)	Station altitude 2 (h2)	Drop (h1 – h2) (m)	Static head 1 Ph1 (m)	Residual head 2 Ph1+h-P (m)	Comments
0	2	0.075	49.4	33/40				626.0	620.5	not under pressure			CAPT - HT
2	22	0.075	454.2	26/32	0.15	0.15	0.68	620.5	536.8	83.7	0.0	83.1	HT - BPT
22	36	0.075	316.5	26/32	0.15	0.15	0.47	536.8	453.3	83.5	0.0	83.04	BPT- STO
36	37	0.5	30.0	33/40	1.3		0.39	453.3	451.9	1.4	0.0	1.01	STO - J
37	41	0.25	77.9	26/32	1.3	0.5	1.01	451.9	449.7	2.2	1.01	2.19	J - T1
43	44	0.25	30.0	26/32	1.3	0.5	0.39	449.1	446.0	3.1	1.01	3.72	J - T2
PE 32 = 878.6 m			PE 40 = 79.4 m			TOTAL = 958 m							

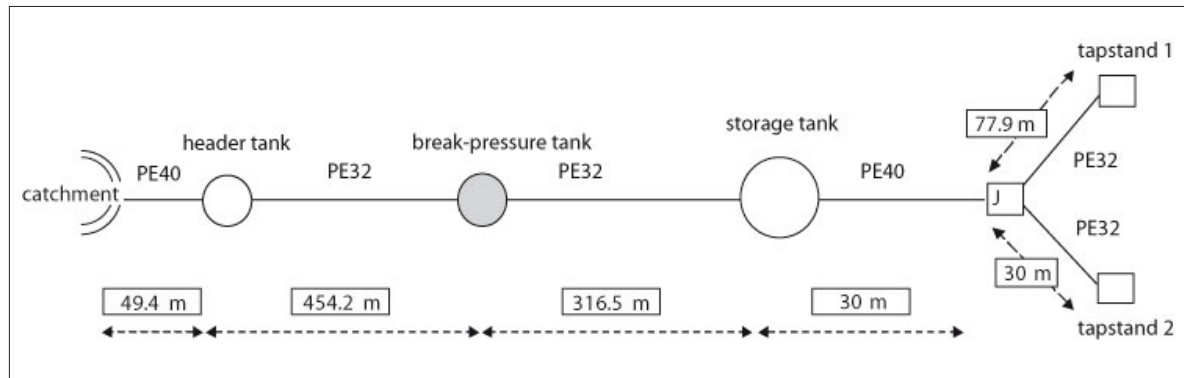


Figure 2: The gravity system.

Head profile

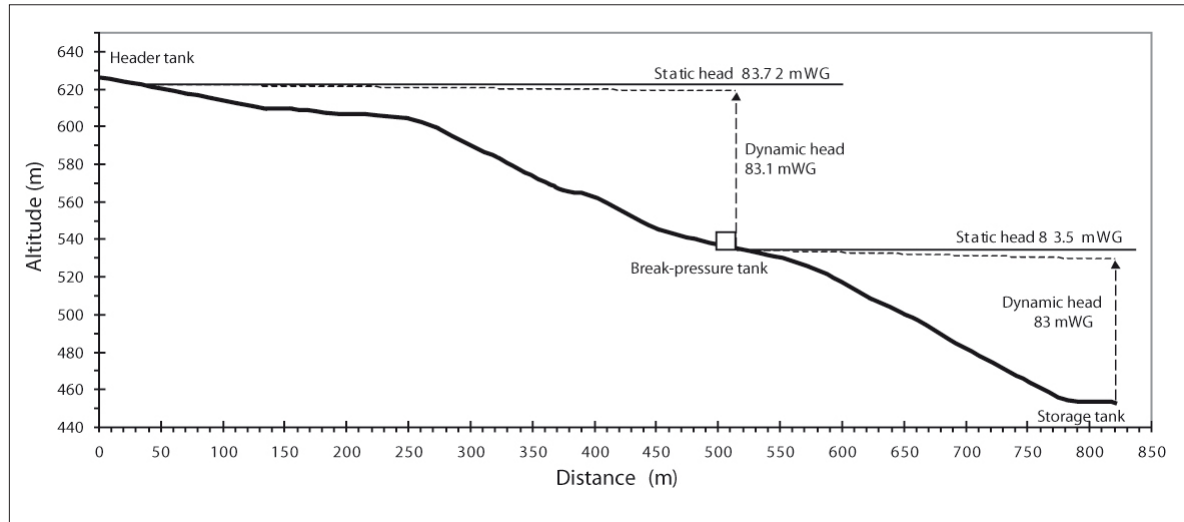


Figure 3: Main supply line.

	Spring	Header tank	Break-pressure tank	Storage tank
Altitude (m)	626.0	620.52	536.8	453.3
Pipe dia (mm)		33/40	26/32	26/32
Length (m)	0	49.4	503.6	820.1
Flow (l/s)		0.075	0.075	0.075
Residual head (mWG)	0	0.0	83.1	83.0

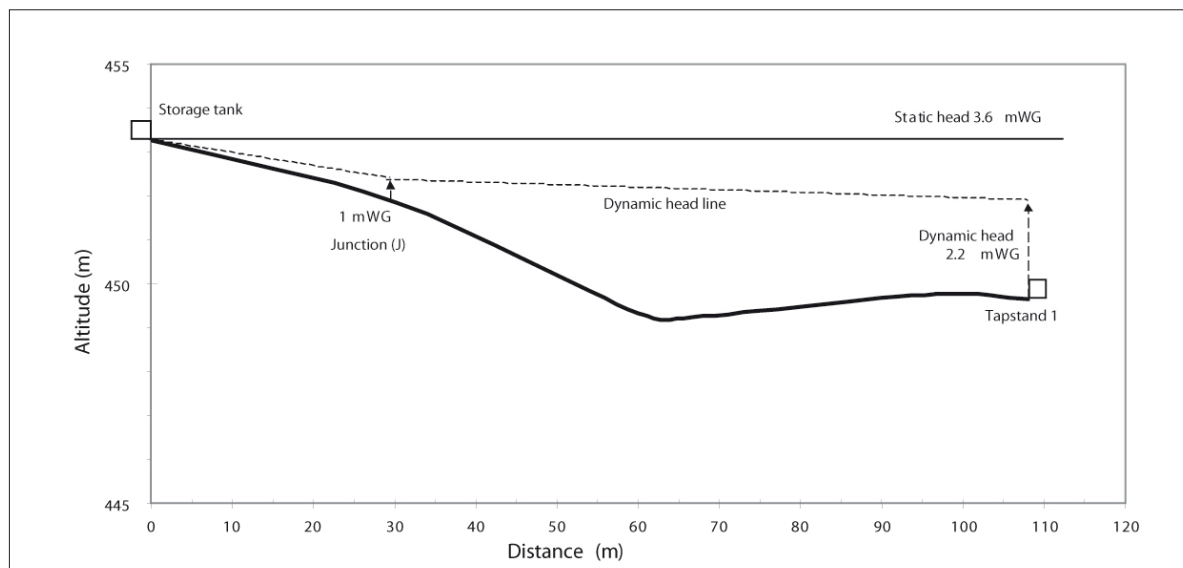


Figure 4: Storage tank to tapstand 1.

	Storage tank		Junction		Tapstand 1
Altitude (m)	453.3		451.9		449.7
Pipe dia (mm)		33/40		26/32	
Length (m)	0		30		107.9
Flow (l/s)		0.5		0.25	
Residual head (mWG)	0.0		1.0		2.2

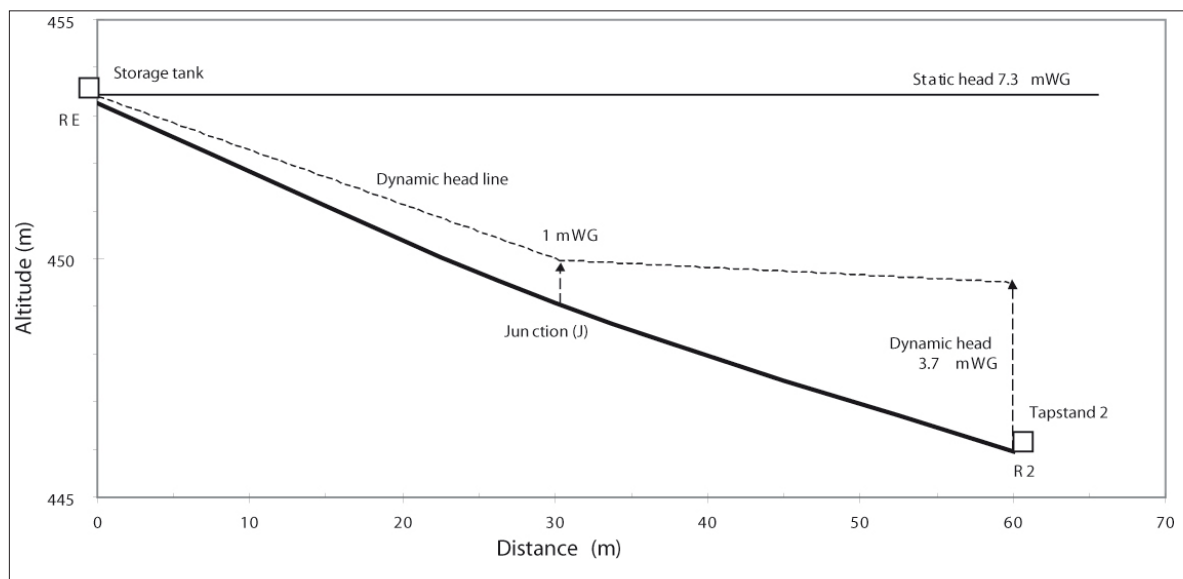


Figure 5: Storage tank to tapstand 2.

	Storage tank		Junction		Tapstand 2
Altitude (m)	453.3		451.9		446.0
Pipe dia (mm)		33/40		26/32	
Length (m)	0		30		60
Flow (l/s)		0.5		0.25	
Residual head (mWG)	0.0		1.0		3.7

Quantities

HT		HT/ VC		HT REINFORCEMENT	
H	1 m	H	0.9 m		
Exterior dia	1.3 m	I	0.8 m	Side	
Wall thickness	0.07 m	Wall thickness	0.08 m	6-mm bar	
Cover thickness	0.08 m	Cover thickness	0.08 m	Mesh H	0.12 m
Slab thickness	0.15 m	Slab thickness	0.1 m	Mesh V	0.12 m
Depth of the foundations	0.15 m	Depth of foundations	0.1 m	Bar H	17.90 m
Wall centre circumference	2.15 m			Bar V	17.90 m
External area	1.33 m ²	Side-wall area	2.16 m ²	Cover	
Internal area	1.06 m ²	Exterior area	0.64 m ²	6-mm bar	
External volume	1.33 m ³			Mesh	0.12 m
Internal volume	1.06 m ³			Bar	22.10 m
Side-wall volume	0.27 m ³	Side-wall volume	0.17 m ³		
Roof volume	0.11 m ³	Roof volume	0.051 m ³	Slab	
Slab volume	0.20 m ³	Slab volume	0.064 m ³	6-mm bar	
Internal-wall volume	0.078 m ³			Mesh	0.12 m
Total 350 kg/m ³	0.65 m ³	Total 350 kg/m ³	0.29 m ³	Bar	22.10 m
Foundation volume	0.20 m ³	Foundation volume	0.064 m ³	VC sides	
Total 200 kg/m ³	0.20 m ³	Total 200 kg/m ³	0.06 m ³	6-mm bar	
				Mesh H	0.12 m
				Mesh V	0.12 m
				Bar V	18.00 m
				Cover + Slab VC	
				6-mm bar	
				Mesh	0.12 m
				Bar	10.70 m

TANK		Theoretical demand (m ³)		2.68
		Actual demand (m ³)		3.50
		Tank W (m)		2.00
		Tank L (m)		2.00
		Demand H (m)		0.88
		Sedimentation H (m)		0.25
		Aeration H (m)		0.25
		Total H (m)		1.38
Height of tank (m)	1.38	HT		
Height of water (m)	1.13	HW		
Internal width of tank (m)	2.00	LA	Width VC	1.2 LA2
Internal length of tank (m)	2.00	LO	Length VC	1.2 LO2
Volume of tank (m ³)	4.50			
Perimeter (m)	8.24			
Maximum bending moment (kg. m)				
$M_{\max} = p \times H^3 / 6$	237.3047			
Minimum side-wall thickness		REINFORCEMENT BAR CALCULATION		
$e = 2 \times \text{rac2} (M_{\max}/100)$	3.08	Choice	12 cm	
			0.12 m	
		Binding wire weight	0.025 kg/m	
Section of vertical bar for 1 m of side-wall		Section of horizontal bar for 1 m of side-wall		
$A_{\text{barV}} = M_{\max} / (T_{\text{bar}} \times z)$	5.33 cm ²	$A_{\text{barH}} = 1/3 A_{\text{barV}}$	1.78 cm ²	
$z = 7/8 \times e$				
T_{bar}	1 650 kg/cm ²			
Bar V available dia 10 mm		Bar H available dia 8 mm		
Section V	0.79 cm ²	Section H	0.50 cm ²	
Number of bars V	7.0	Number of bars H	4.0	
Mesh V	14.3 cm	Mesh H	25.0 cm	
Choice	12 cm	Choice	15 cm	

SIDES		Quantity of bar V	10 mm	Quantity of bar H	8 mm
		Length	137.50 m	Length	110 m
		Weight	81.13 kg	Weight	41.80 kg
		Binding wire	70 m		
		Weight	1.7 kg		
SLAB		L	W		
	Mesh	15 cm	15 cm		
	Bar dia	8 mm	8 mm		
	Length	26.67 m	26.67 m		
	Weight	10.05 kg	10.05 kg		
	Binding wire		25 m		
	Weight		0.6 kg		
COVER		L	W		
	Mesh	15 cm	15 cm		
	Bar dia	8 mm	8 mm		
	Length	26.67 m	26.67 m		
	Weight	10.05 kg	10.05 kg		
	Binding wire		25 m		
	Weight		0.6 kg		
SLAB, VALVE CHAMBER		L	W		
	Mesh	10 cm	10 cm		
	Bar dia	6 mm	6 mm		
	Length	14.4 m	14.4 m		
	Weight	3.05 kg	3.05 kg		
	Binding wire		3 m		
	Weight		0.1 kg		
COVER, VALVE CHAMBER		L	W		
	Mesh	10 cm	10 cm		
	Bar dia	8 mm	8 mm		
	Length	14.4 m	14.4 m		
	Weight	5.43 kg	5.43 kg		
	Binding wire		24 m		
	Weight		0.6 kg		
Volume		Cement (kg)	Cement (bags)	Losses (%)	T o t a l
(bags)					
1.4	Sides	490	9.8	10	11
0.7	Slab 1	236	4.7	10	6
0.7	Foundation slab	135	2.7	10	3
0.4	Cover 1	126	2.5	10	3
0.1	Slab 2	50	1.0	10	2
0.1	Foundation slab	29	0.6	10	1
0.1	Cover 2	40	0.8	10	1
			Total		27 bags
					1.35 t
SPRING					
Low wall					
	Length	5 m	Mesh	15 cm	
	Height	0.6 m	Bar dia	8 mm	
	Thickness	150 mm			
	Cement	350 kg/m ³			
	Area	3.00 m ²	Bar length	46 m	Calculated
	Volume	0.45 m ³	Bar weight	18 kg	10%
	Cement	157.5 kg	Binding wire	17 m	0.33 t
	Sand	191 l	Binding wire weight	0.4 kg	53 kg
					57.8 kg
					1.2 kg
					1.3 kg

<i>Slab</i>	Length	3 m	Mesh	15 cm
	Width	2 m	Bar dia	8 mm
	Thickness	80 mm		
	Cement	350 kg/m ³		
	Area	6.00 m ²	Bar length	85 m
	Volume	0.43 m ³	Bar weight	34 kg
	Cement	168 kg	Binding wire	30 m
	Sand	203 l	Binding wire weight	0.8 kg
	Gravel	407 l		
TAPSTAND				
<i>Slab</i>	Length	2.5 m	Mesh	10 cm
	Width	2 m	Bar dia	6 mm
	Thickness	100 mm		
	Cement	350 kg/m ³		
	Area	5.00 m ²	Bar length	105 m
	Volume	0.50 m ³	Bar weight	24 kg
	Cement	175 kg	Binding wire	55 m
	Sand	212 l	Binding wire weight	0.4 kg
	Gravel	424 l		
<i>Kerb</i>	Length	9 m	Mesh	10 cm
	Width	0.1 m	dia mm bar	6 mm
	Thickness	100 mm		
	cement	350 kg/m ³		
	Area	0.90 m ²	Bar length	27 m
	Volume	0.09 m ³	Bar weight	6 kg
	Cement	31.5 kg	Binding wire	18 m
	Sand	38.12 l	Binding wire weight	0.1 kg
	Gravel	76.25 l		
<i>Tap post</i>	Length	1.35 m	Mesh	10 cm
	Width	0.15 m	Bar dia	6 mm
	Thickness	150 mm		
	Cement	350 kg/m ³		
	Area	0.20 m ²	Bar length	6 m
	Volume	0.03 m ³	Bar weight	1 kg
	Cement	10.631 kg	Binding wire	4 m
	Sand	12.87 l	Binding wire weight	0.03 kg
	Gravel	25.73 l		
<i>Valve chamber</i>	Length	2.6 m	Mesh	10 cm
	Width	0.5 m	Bar dia	6 mm
	Thickness	80 mm		
	Cement	350 kg/m ³		
	Area	1.30 m ²	Bar length	29 m
	Volume	0.10 m ³	Bar weight	7 kg
	Cement	36.40 kg	Binding wire	16 m
	Sand	44.05 l	Binding wire weight	0.11 kg
	Gravel	88.11 l		
<i>Slab base</i>	Length	2.7 m	Mesh	10 cm
	Width	2.2 m	Bar dia	6 mm
	Thickness	100 mm		
	Cement	200 kg/m ³		
	Area	5.94 m ²	Bar length	124 m
	Volume	0.59 m ³	Bar weight	28 kg
	Cement	118.80 kg	Binding wire	64 m
	Sand	274.46 l	Binding wire weight	0.45 kg
	Gravel	548.92 l		

Total for a tapstand

Cement	0.372 t
Cement	8 bags
6-mm bar	145 m
6-mm bar	30.5 kg
Binding wire	0.5 kg

Summary of materials and cost of gravity distribution system

Site: Ban Houn

Date: 22nd July 1998

Material and fittings	U	Unit cost (US\$)	Spring	HT	System components			Tapstand	Total required	Total cost (US\$)
					BPT	Pipeline	Storage tank			
REINFORCEMENT										
Bar dia10	kg	0.65	11				115		126	81.90
Bar dia 6	kg	0.65		25	20		5	35	85	55.25
Bar dia 8	kg	0.65	8				55		63	40.95
1 mm binding wire	kg	1.89							10	18.90
CONSTRUCTION										
Bricks	each	0.04					400		400	16.00
Cement	t	95	0.3	000	000		001	0.6	003	251.75
Bar dia 8	kg	0.5							4	2.00
Bar dia 6	kg	0.5							2	1.00
Cement additive	kg								10	
VALVES										
1/2" ball valve	each	8.82							0	128.58
3/4" Talbot tap	each	16						2	2	32.00
3/4" stop cock	each	44						2	2	88.00
1" adjustable stop cock	each	39			1				1	39.00
1" 1/2 stop cock	each	14.29		1	1		1		3	42.87
Joint for valves	each	1							20	20.00
PVC										
1" 1/2 socket	each	0.26					1		1	0.26
1" 1/2 F adaptor	each	2	1	1	1		1		4	8.00
2" F adaptor	each	6	3	2	2		1		8	48.00
2" M adaptor	each	0.43		2	2		1		5	2.15
90° 2" elbow	each	4.76	2	1	1				4	19.04
1" 1/2 PVC pipe	m	0.63	1.5	1.5	1.5		2.0		6.5	4.10
2" PVC pipe	m	1	12.0	16.0	16.0		12.0		56.0	56.00
PVC Glue	Tin								1	
HDPE										
90° 32 x 32 x 32 tee	each	4.9				1			1	4.90
32 x 32 compression connection	each	2.84				9			9	25.56
40 x 32 reducer	each	4.29				1			1	4.29
HDPE pipe DN 32 NP 10	m	0.36				920			920	331.20
HDPE pipe DN 40 NP 10	m	0.56				80			80	44.80
32 x 1" female adapter	each	1.72					1		1	1.72
32 x 1" male adapter	each	1.7		1	2			2	5	8.50
40 x 1" 1/2 male adapter	each	3	1	1					2	6.00
G.I.										
G.I. 3/4" elbow	each	0.33		1	1		3	12	17	5.61
G.I. 1" elbow	each	1.2			2		2		4	4.80
G.I. 1" 1/2 elbow	each	2.4		2					2	4.80
G.I. 2" elbow	each	1.79		1	1		3		5	8.95
G.I. 3/4" nipple	each	0.6						8	8	4.80
G.I. 1" nipple	each	0.8			4		4		8	6.40
G.I. 1" 1/2 nipple	each	1.5		3	3		3		9	13.50
G.I. 2" nipple	each	2.2		3	3		3		9	19.80
G.I. 3/4" pipe	m	1.92		1.0	1.0		3.5	6.0	11.5	22.08
G.I. 1" pipe	m	2.72			1.0		2.0		3.0	8.16
G.I. 1" 1/2 pipe	m	3.76	0.5	1.5	0.5		0.5		3.0	11.28
G.I. 2" pipe	m	4.7	1.0	2.0	2.0		3.5		8.5	39.95
G.I. 1"-3/4" F-F reducer	each	0.85						2	2	1.70
G.I. 1" 1/2-3/4" M-F reducer	each	1.28		1	1		1		3	3.84
G.I. 1" 1/2 socket	each	2.15	1						1	2.15
G.I. 1" 1/2 tee	each	3.24		1	1		1		3	9.72
G.I. 2" tee	each	4.72		2	1		1		4	18.88
G.I. 3/4" union	each	2					1	4	5	10.00
G.I. 1" union	each	3.22			2		1		3	9.66
G.I. 1" 1/2 union	each	3.96		2	2		2		6	23.76
G.I. 2" union	each	6.8		2	2		2		6	40.80
TOTAL COST									1 653.36	

2 Emergency distribution systems from boreholes

2.1 Implementation

On productive boreholes, with an exploitation flow of several m³/h, and in certain contexts (emergency situations, densely populated or urban areas etc.), a submersible pump may be installed to supply a series of tapstands, instead of using several boreholes equipped with handpumps. Implementation is quicker, and management of one motorised system is simpler than regular maintenance of a series of handpumps. Also, a simple system of water chlorination can be installed at the distribution tank. This is much simpler and more flexible than a treatment system at every handpump or in every house, and is particularly important in regions where cholera is endemic and risks of epidemics are high. Finally, if a management system is established, these mini-systems, often installed under emergency conditions, can be kept and improved to supply an area which is insufficiently supplied by the permanent system for the local population.

The mini-system consists of:

- a 103-113 mm or 112-125 mm borehole (or it is possible to use a 167-180 mm borehole to install a 6" submersible pump for producing large flows);
- a pumping station, equipped with a 4" submersible pump and a generator;
- a storage tank, placed at sufficient height to supply the system by gravity;
- a system for disinfecting the water using chlorine (HTH);
- primary and secondary water-supply lines, supplying the tapstands;
- emergency tapstands with simple installation (to minimise mud).

Obviously, the choice of submersible pump depends on the flow and head required (see Chapter 6), but it is possible to use a submersible pump with standard equipment, linked to a generator if necessary. The storage tank may be temporary or permanent, depending on the context. Emergency tanks made of corrugated sheets and liners, with standard capacities of 45, 70 and 95 m³, are very quick to assemble, and are suitable for a semi-permanent installation (several years). However, they have to be mounted on the ground. Where a water tower is required, Braithwaite-type tanks consisting of galvanised steel panels, which can be permanently mounted on top of a metal tower, are preferable. The dimensions and volumes of tanks offered by this supplier range from several dozens to several hundreds of cubic metres.

For simplicity, 2" piping is regarded as standard for small-scale emergency installations (installation of emergency tapstands, pump outlet, standard connections). In more complex systems, which may continue working over several years, it is essential to design the whole system correctly (see Chapter 11A). The pipes are buried, except in extreme emergencies where this is done in a second phase.

The system of water chlorination is either a Dosatron-type dosing pump proportional to the flow, or a simple drip-feed system installed in the tank. (Note: chlorine is consumed by the metal in metallic tanks, see Chapter 12.)

2.2 Human and financial resources

A list of the personnel necessary for the implementation of an emergency mini-system from a borehole is shown in Table 11.XIX.

The cost of a mini-system from a borehole (excluding the borehole itself) supplying two water supply lines of 250 m, and 6 tapstands with 4 taps each, providing the daily supply for 3 200 people is shown in Table 11.XX. The storage tank is a 24 m³ Braithwaite tank.

Water is distributed in two periods of 2 h/day, one in the morning and the other in the afternoon. The flow per tap is fixed between 0.15 and 0.2 l/s. The tank is refilled in 4 h, during the evening for the following morning, and from 11:00 to 15:00 for the afternoon distribution.

Table 11.XIX: Personnel required for the installation of an emergency mini-system on a borehole.

Team	Indicative monthly salary (US\$)
1 electrician (installing and repairing pumps)	150-200
1 site foreman	150-200
1 person responsible for the station (chosen from within the community)	40-60
2 plumbers (for the duration of the work)	100-150
12 labourers (for the duration of the work)	40-80
2 masons (for the duration of the work)	100-150
1 driver	60-80

Table 11.XX: Cost of a mini-system on a borehole producing 6 m³/h, which provides 48 m³/day and covers an emergency demand of 15 l/person/day.

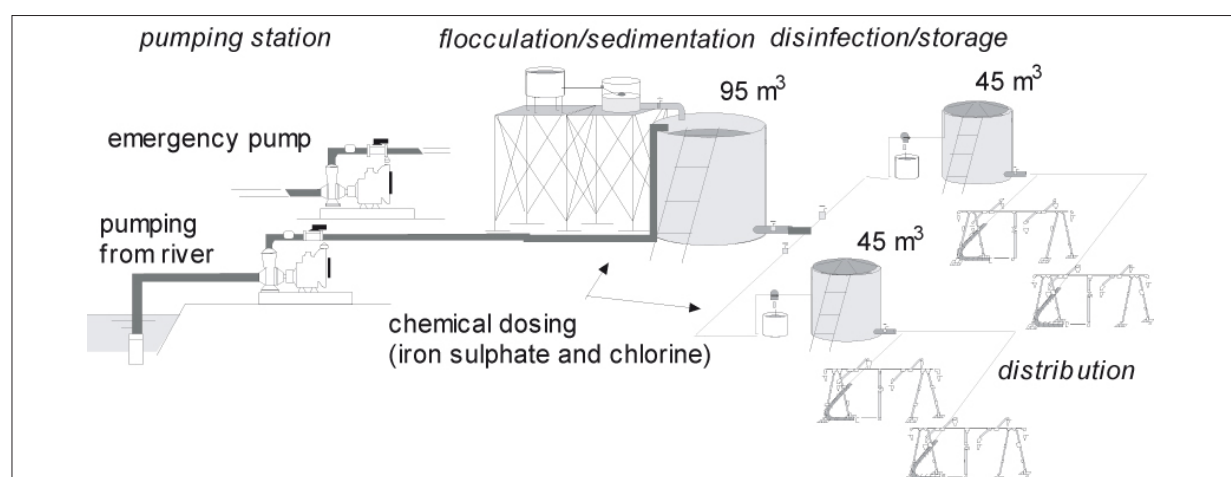
	Unit cost (€)	Quantities	Total cost (€)
Pumping kit	5 000	1	5 000
24 m ³ Braithwaite storage tank	5 000	1	5 000
Tapstands	270	6	1 620
Dosatron dosing system	1 100	1	1 100
2" water-supply line (m)	6	500	3 000
Distribution accessories kit	2	1 500	3 000
HTH stock (kg)	4	100	400
Bacteriological analysis kit	2 500	1	2 500
TOTAL			21 620

3 Emergency systems from rivers

3.1 Implementation

These emergency water-supply systems involve production and treatment stations using a surface-water source (lake or river). The water is distributed over a branched system via emergency tapstands.

The water-treatment system uses simple technology, and generally consists of flocculation-sedimentation followed by chlorine-based disinfection (Figure 11.38).

**Figure 11.38: General plan of an emergency water-supply system.**

3.1.1 PUMPING

Water is pumped directly from the river with a surface motorised pump (simple and cheap), or with a submersible electric dewatering pump. The pump strainer is kept at 1 m below the water surface by a float, or is installed in a pumping well (see Chapter 12). The water intake is located as far upstream as possible in relation to inhabited areas, and given maximum protection from faecal pollution.

3.1.2 FLOCCULATION-SEDIMENTATION

This operation (see Chapter 12) is carried out in the standard manner in rigid tanks, by injecting a flocculant such as aluminium sulphate. Determining the correct dose requires the preparation of a mother solution at 5% and some calibration (jar test) depending on the turbidity of the untreated water.

3.1.3 DISINFECTION AND STORAGE

Once sedimented, the water goes to a tank, where it is disinfected and stored before being distributed via a gravity system.

The tanks used (Oxfam type – see Section 3.1.5) consist of an assembly of corrugated sheets screwed together, with a rubber lining inside. The disinfection-storage tanks must be covered.

The volumes generally used are 45, 70 and 95 m³.

3.1.4 MOBILE TREATMENT UNIT

The system consists of a 95 m³ flocculation/sedimentation tank, plus two 45 m³ tanks for chlorination and storage, for a daily production of 80 or 160 m³ with two sedimentation cycles. The exact measurement of decantation time enables the number of daily refill-distribution cycles to be determined (test of preliminary decantation time – see Chapter 12). To increase the volume of water treated, simply add one or more modules (a sedimentation tank and two storage tanks) to the treatment chain.

3.1.5 CONSTRUCTING AN OXFAM-TYPE RIGID TANK

These tanks have a range of capacities (Table 11.XXI). The example given involves the installation of an Even-Products 45 m³ tank, the construction time for which is half a day with a team of six people (Table 11.XXII).

3.1.6 FILTRATION-FLOCCULATION

Filters working under pressure are used, with a 40 micron pore-size, composed of active carbon and a filter medium (gravel, fine sand etc.). The pumped water goes directly under pressure through the filters after the injection of a flocculant, the principle being to trap the flocs in the filters*. The necessary contact time for coagulation occurs in the pipes.

Table 11.XXI: Characteristics of OXFAM-type rigid tanks

Capacity (l)	Height (m)	Number of rows of sheets	Diameter (m)
11,000	2.3	2	2.5
45,000	1.5	2	6.4
70,000	2.3	3	6.4
95,000	3.0	4	6.4

* Experiments and studies under development.

Table 11.XXII: Installation of an Even-Products 45 m³ tank.

Clearing site of soil and all coarse particles
Marking 2 concentric circles (radiuses of 3.2 and 3.5 m)
Digging a channel of 5 cm of depth between these circles
Laying the tank in the centre of the circles
Assembling the first row of sheets and sitting them in the channel
Consolidation of the base (interior and exterior) with soil or sand
Mounting the second row of sheets
Applying the protection tape and the PVC capping
Installing the tank liner (the bottom of the tank is the bare ground)
Fixing the liner to the top of the second layer of sheets with clips
Cutting a hole in the liner at the outlet hole, and fitting the flanges
Installing the central pole and supporting ropes connecting this pole to the tank sides
Fitting the tank roof, held tight with elastic strainers

The greatest disadvantage of this treatment system is the rapid blocking of the filters when the water is too turbid (> 100 NTU). It is then necessary to carry out frequent back-washings, which consume a great of water: the flow falls radically from 10 to 5 m³/h.

One solution consists of locating this filter downstream of a standard in-tank flocculation-sedimentation system (e.g. Bô, ACF Sierra Leone, 1996). The variations in the turbidity of the water leaving the sedimentation tank can therefore be prevented, and the 'residual flocs' can be trapped in the filter.

This system is particularly useful in the case of poor flocculation/sedimentation resulting from sudden variations in the turbidity of the raw water (following storms etc.), and also to shorten the duration of the water treatment allowing two sedimentation cycles per day, thus doubling the production of treated water.

3.2 Example of the Aswha system

Aswha is located in Southern Sudan, some 40 km from the border with Uganda: a system was installed there in 1993 to provide water for a camp of 5 000 displaced people (Figure 11.38).

3.2.1 PUMPING STATION AND DISTRIBUTION SYSTEM

Situated on the banks of the Ashwa river, the pumping station has an Atlanta pump driven by a 2-cylinder Lister-Petter diesel engine. Daily fuel consumption is 15 l for 4 h pumping. The suction head is 2 to 7 m depending on the level of the river. The 15-m suction pipe has a pump strainer and a foot-valve. The buried delivery pipe has a total length of 700 m and consists of 3" PVC pipes, with push-fit joints.

The total difference in height between the pump and the treatment station (Table 11.XXIII) is 35 m. Residual pressure in the pipe is 20 mWG. Flow is 30 m³/h. Head-losses are estimated at 25 mWG.

Table 11.XXIII: Treatment unit.

Two 95 m ³ tanks for sedimentation
Three 45 m ³ tanks for chlorination-storage
Foundations of the tanks: 30 cm stone base and concrete slab

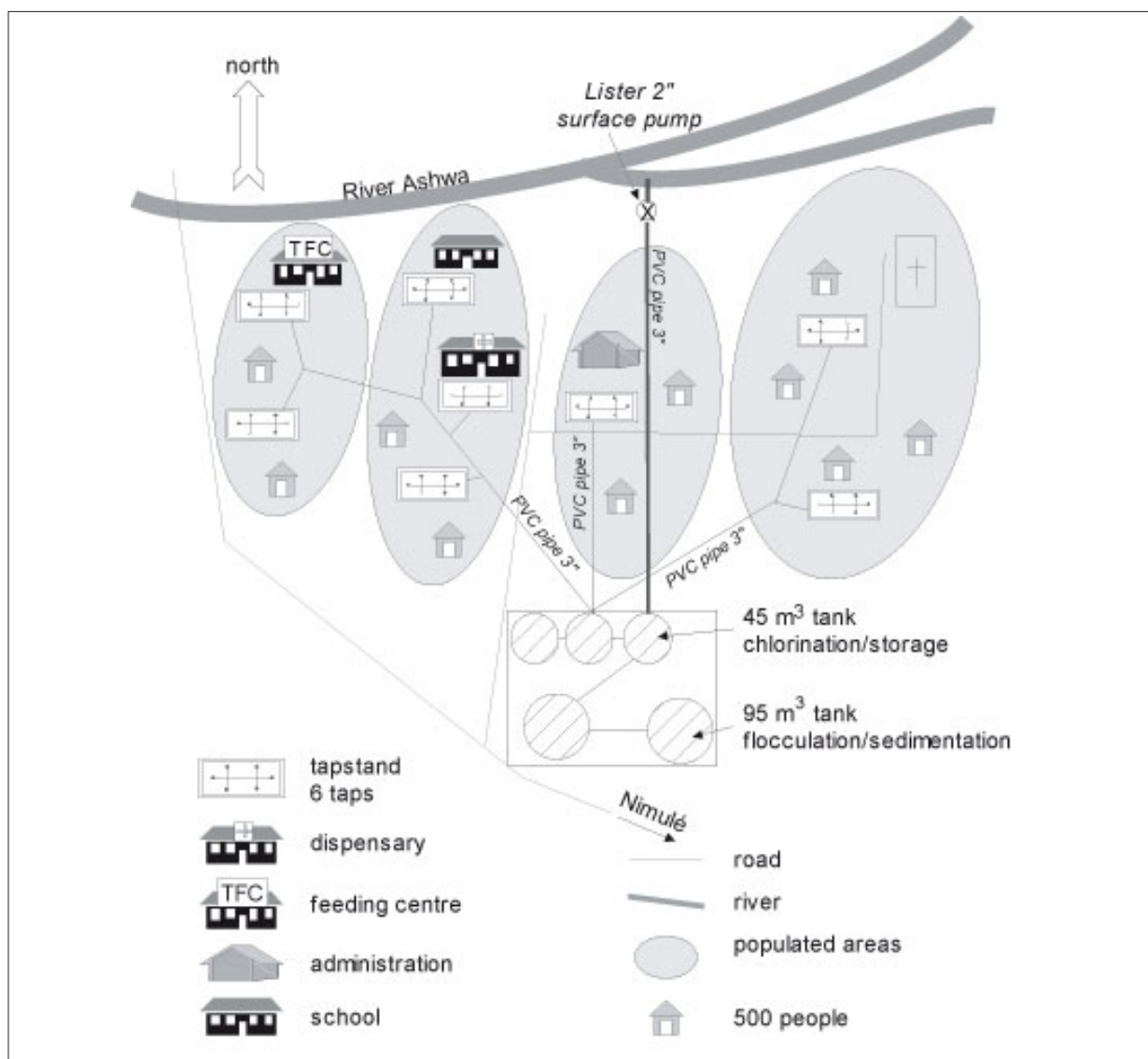


Figure 11.39: Emergency water supply system installed at Ashwa (Southern Sudan, 1993).

3.2.2 OPERATION

Daily production is 120 m³ of treated water. Tanks are refilled in a single four-hour pumping operation in the afternoon.

The flocculation additive is mixed during the simultaneous refilling of the two 95 m³ tanks. After a night of sedimentation, the tanks are then emptied into the chlorination-storage tanks (duration: 3 h). Chlorine solution is added to the first tank in the morning. The treated water is then distributed throughout the gravity system in the afternoon.

Table 11.XXIV: Consumables.

Aluminium sulphate	8 kg/day
Calcium hydroxide [Ca(OH) ₂]	3 kg/day
HTH at 65 %	360 g/day
Phenol red (for checking pH)	12 tablets/week
DPD 1 (free residual chlorine)	12 tablets/week

Table 11.XXV: Treatment results.
T= storage tank.

Parameters	Before treatment	After treatment	In the system
Turbidity (NTU)	80	< 5	
pH	7.8	6.8	7
Residual free chlorine (mg/l)	0	T1: 0.6 – T3: 0.3	0.25
Number of faecal coliforms/100 ml	> 50		0
Average flow per tap (l/s)			0.2

Consumption of treatment products is shown in Table 11.XXIV, and the treatment results in Table 11.XXV.

3.2.3 HUMAN RESOURCES AND STATION MANAGEMENT

Daily planning of the various treatment phases (refill-flocculation-sedimentation-disinfection) and distribution times is carried out by the station personnel (Table 11.XXVI). Daily updated registers record the volume of water treated, the doses of chemicals, depending on turbidity, free residual chlorine and pH before and after flocculation, and the aluminium levels after treatment. Regular bacteriological analyses are used to check water quality before and after treatment.

Table 11.XXVI: Personnel required for a pumping station.

1 head of station, in charge of chemical dosing, analysis and good operations
1 technician, in charge of the maintenance of pumps and equipment
2 unskilled labourers
Guards (day and night)

3.2.4 EQUIPMENT COSTS

The financial resources required for a system such as the one at Ashwa are shown in Table 11.XXVII.

Table 11.XXVII: Cost (€) of the equipment needed for an Ashwa-type coagulation/flocculation treatment module.

	Unit cost	Quantities	Total cost (€)
Diesel pump (30 m ³ /h at 45 m)	2 500	1	2 500
95 m ³ rigid tank (sedimentation, without roof)	5 350	1	5 350
45 m ³ rigid tank (chlorination and storage, with roof)	4 300	3	12 900
Tapstands	270	8	2 160
Dosing system, dosing pump	1 100	1	1 100
3" PVC water supply line (m) with fittings	6	700	4 200
3" PVC distribution line (m) with fittings	6	1 500	9 000
Chlorine powder (kg)	4	500	2 000
Aluminium sulphate(kg)	1	250	250
Bacteriological analysis kit	2 000	1	2 000
TOTAL			41 460