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THE RURAL WATER SUPPLY AND ENVIRONMENTAL PROGRAMME, AMHARA REGION

TRAINING MANUAL for STUDY and DESIGN

Of

GRAVITY WATER SUPPLY SYSTEMS

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PREFACE

The Rural Water Supply and Environment Programme in Amhara Region (RWSEP) is a regional rural development programme supported by the Government of Ethiopia (GoE) and Finland (GoF) since 1994. Phase I (1994–1998) of the Programme focused on capacity building at regional level, Phase II (1998–2002) shifted the focus to zone and woreda levels and Phase III (2003 - 2006) is continuing in the decentralization trend

In its 3 Phases porgramme period, RWSEP has been financing small scale water supply schemes mainly hand dug wells and spring developments on spot. Although great number of people benefited from these water points, there are communities located in the programme area which can be supplied by gravity springs having considerable length of pipeline and even there are relatively better living standard rural village people which are willing and can afford to pay for water from deep well sources.

The mid term review conducted for the third phase of the programme in January 2005 has given a green light that RWSEP can finance model gravity and pumped water supply systems if the communities are willing and able to sustain the systems.

As it is a new technology type to be financed by RWSEP, in order for the Woreda Water Resources Development Team Experts to be familiar with the design principles of gravity water supply systems, this design manual is prepared for training the experts.

Of course as most of the Woreda Water Resources Development Team (WWRDT) experts do not have the appropriate academic background and experience in the design of such schemes, what is done in this manual is to highlight the basic principles to be used in the design and those experts at least in the first batch of designs can speak related language of design with the engineers which may be assigned for the designs from the Water Resources Development Bureau or relevant local consultants. After participating in some number of such designs with the experienced engineers, the Woreda WRDT experts can handle other designs of related nature with minimum assistance.

Finally, efforts have been made to organize this manual for quick reference, and to present it is a manner that allows professionals of both engineering and non-engineering backgrounds to readily understand and use it.

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1.1 Introduction

A major problem in providing adequate amounts of clean water to communities is that safe water is often available only at considerable distances from people's houses. Often many hours are needed to carry the water to the home. This is hard work and only small amounts of water are carried.

Water can be transported from its source to village by pipeline if a source of water is sufficiently above where the water is to be used. If the water flows through the pipeline using only gravity, the network of pipes is called **gravity - flow water system**.

The major components of gravity - flow water system are:
1. An elevated source of water (located at an elevation higher than the village), particularly a spring, or clean river, or stream. Where disinfection is necessary, simple methods should be used. In all cases the source should be protected to prevent contamination.
2. A sedimentation tank (if necessary) near the source which allows suspended solids to settle out of the water.
3. The main pipeline which transports the water to where it is distributed. Within limits, this can follow land counters and may even go up and over small hills.
4. District reservoirs that may be required to store water overnight for peak use during the day time.
5. A break pressure tank (if necessary) to prevent excessive pressure from bursting the pipes and taps/faucets.
6. Networks of smaller pipes that distributes the water to the public fountains/standpipes.
7. Public fountains/standpipes where the water is made available to the people.
8. Washout valves and air release valves.
9. Valve boxes
10. Pipe supporting structures when crossing rivers and gullies.

For the typical gravity flow water system see Figure 1.1

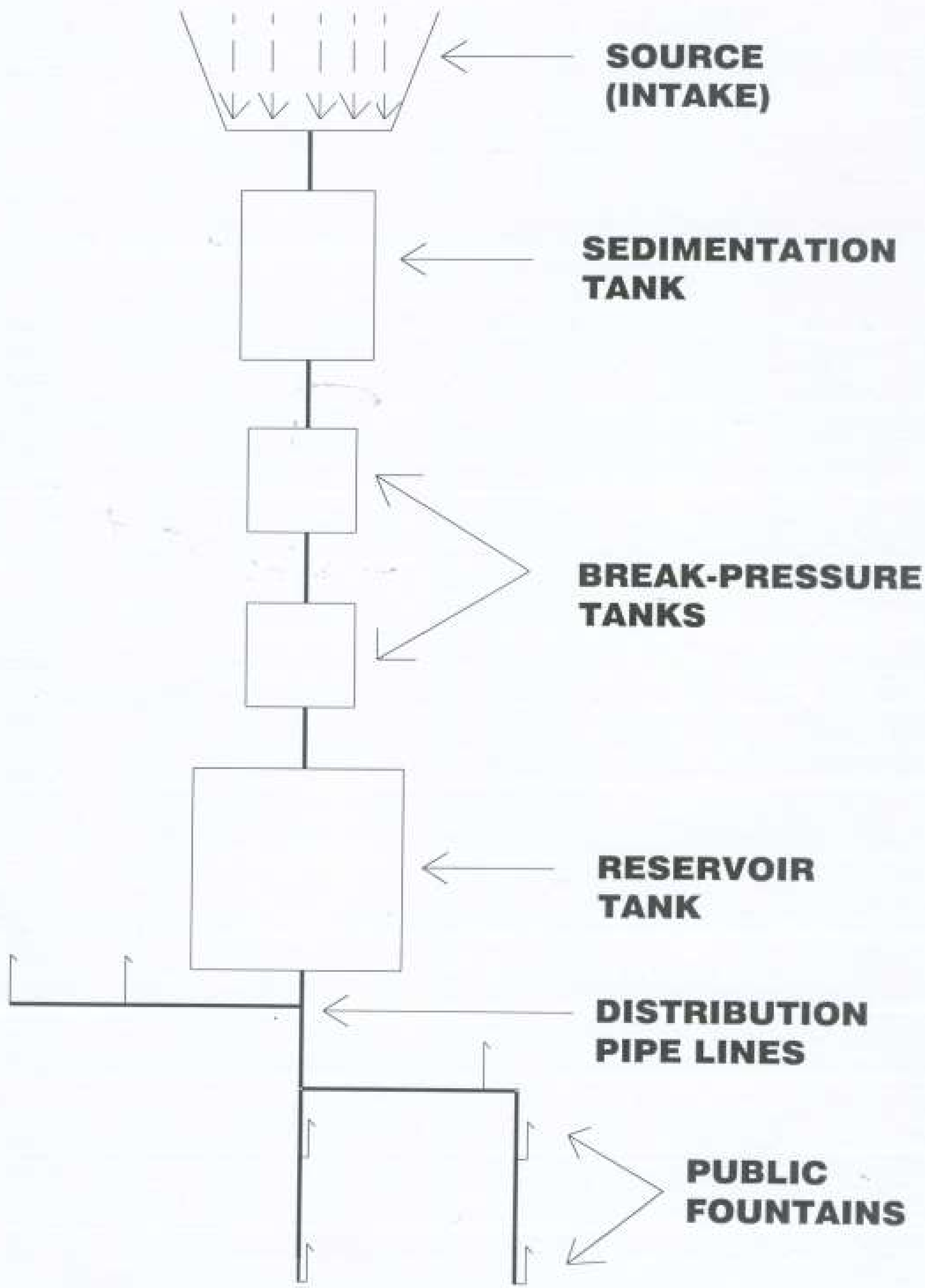


Fig 1.1 Typical Gravity Water Supply System

1.2 Advantages and Disadvantages of a Gravity Flow System

Advantages,

- A gravity - flow water system requires no energy to operate as the water is propelled by gravity alone. No pumps are needed and there are very few moving parts. Therefore, maintenance is simple and is required only infrequently.

- Water is delivered when required, close to the user's home. If the source permits, enough water can be delivered to meet all of the needs of the household. A better quality of water is obtained than in the village's traditional water source.

- Gravity flow water systems can be built by the village people themselves, if trained project supervisor is provided.

- Systems of different sizes can be built for different members of people. Small systems can serve single households and large ones will serve many villages or towns.

Disadvantages,

- Usually water quality depends on the quality of the water of the source, but if sufficiently clean water is not available, additional treatment facilities may have to be built at additional cost.

- Available sources of water may not provide adequate amounts of water throughout the year. Systems should not be built to handle greater flows than water is available. More abundant sources at a greater distance should be considered.

- Water rights cause problems in some areas as villages near the source may object to having "their" water piped to villages below them. Pipelines may be damaged in local disputes.

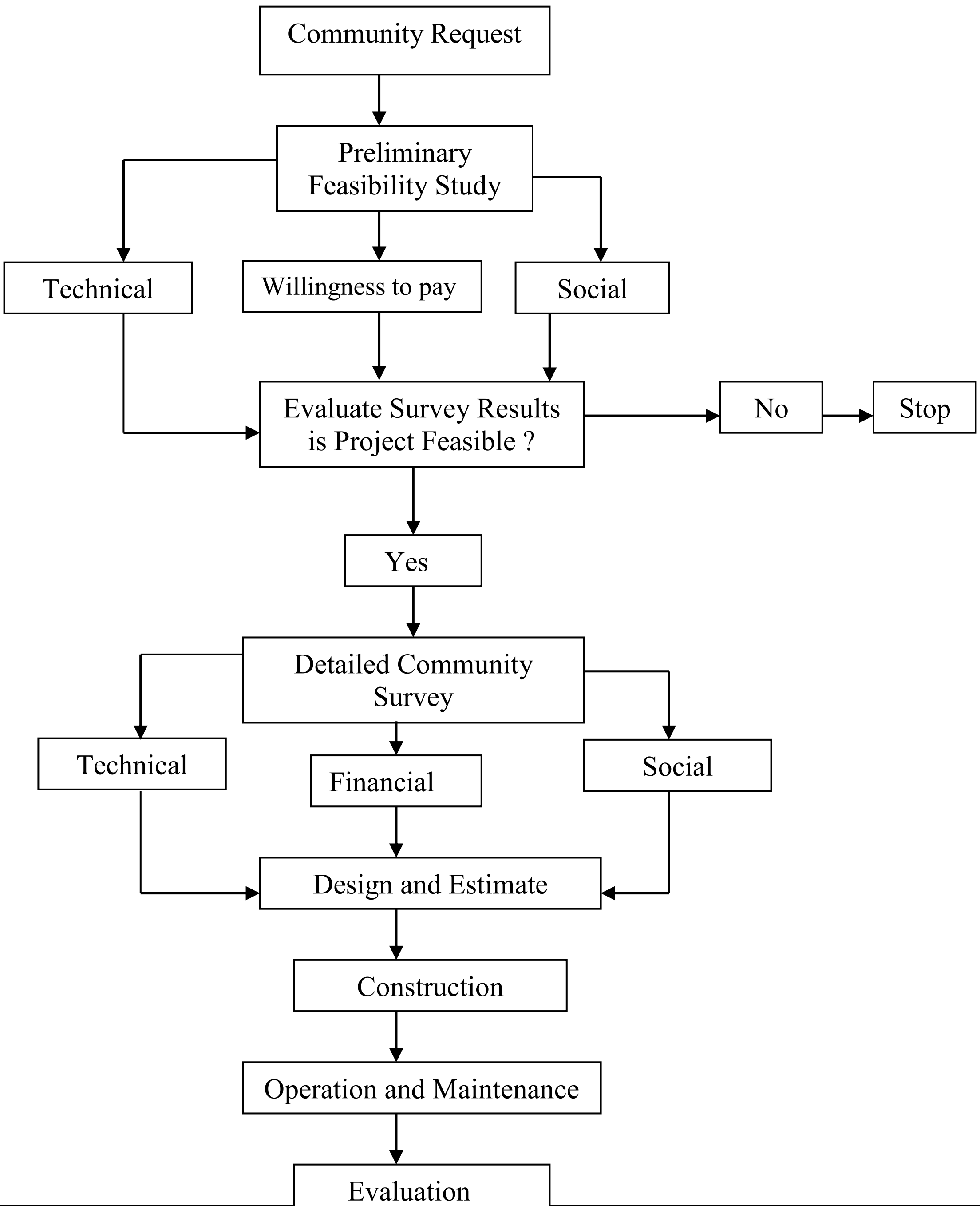
- Gravity-flow systems do regular basic maintenance, especially the care of the taps/faucets. If arrangements are not made to carry out the maintenance, the system will eventually fail.

2. ORGANIZATION OF A GRAVITY - FLOW WATER SYSTEMS

This organizational structure refers to projects in many developing countries. The approach needed in a particular situation may differ.

Figure 2.1 shows the stage of implementation of a gravity - flow water system. The tasks required to complete each stage are discussed in detail in the text which follows.

Figure 2.1 Stages in Project Implementation



2.1. Community request

If gravity flow water system have been built nearby, communities should be well aware of the technology and its benefits through promotional work undertaken by the government or NGOs. Thus it is expected that the community will make a request for a gravity-flow water system.

The request should be more than just a demand for the technology. It can be in the form of a survey of the community showing the needs of the people and the input that they are willing to donate to the project.

The community request should give an idea of whether:
- the community needs the technology,
- the technology is appropriate to the abilities and the social and cultural patterns of the community,
- there is sufficient demand for the technology.

2.2. Preliminary Feasibility Survey (Pre Feasibility Study)

A preliminary survey is made by a team of experts. It should have two objectives:
- to determine whether a gravity flow water supply system is feasible,
- to get a better understanding of the community structure and to evaluate whether the community request is a true indicator of community interest in the project.

Preliminary Survey Tasks

Technical
- Visit all potential water sources.
- Assess the quality and flow of water.
- Attempt to estimate lowest dry season flow.
- Examine potential pipeline routes.

Social
- Discuss project with community leaders.
- Estimate population to be served.
- Assess potential of community participation.
- If possible, examine condition of past community participation projects.

Willingness to pay
- One the most important issues in designing and implementing a water supply system is how to ensure the financial sustainability of the project. This can involve predicting what users will be willing to pay for water in the future. In order to know this:
  - Conduct the willingness to pay survey of the community for sustaining the system.

" Willingness to pay (WTP) is that the maximum amount that an individual states he/she is willing to pay for a good or service" - in this case improved water supply offered"

The survey is to be made by preparing a questioner and to be filled and analyzed by the Woreda/district experts located close to the community. For RWSEP case the questioner is prepared and will be available to Woreda Water Resources Development Teams.

Once the preliminary survey is complete, the project team will have an idea of whether the project is feasible or not. Despite whether there is a general felling for or against a project amongst the study team, the data which has been collected in the survey needs to be analyzed by the programme and its staff. There are three basic criteria, which can determine project feasibility:

- Is the project technically possible?
- Is the community ready and willing to participate to the fullest extent possible and will this participation be sufficient to complete the project successfully and provide the planned benefits?
- Is the community willing to pay of any amount for sustaining the scheme?

Only if these three criteria are met can the project be considered feasible and the detailed survey be undertaken.

2.3. Detailed Community Survey (Detailed Design)

The detailed survey has very specific objectives. Once this survey is complete, the project team should have a clear idea of how the project will implementation including a realistic projection of the amount of community participation that the project will rely on. The whole project team (engineer, social workers, surveyors and others) should be involved in the survey so that they can familiarize themselves with the community and the community with them.

A meeting should be held with the community leaders who have been identified in the preliminary survey before the start of the community survey to discuss the survey and to introduce the project team. The community leaders should be continuously consulted, involved and informed of decisions and actions taken by the project team.

Once the feasibility survey is completed, the preliminary location of standpipes and the pipeline route are presented to the community leaders for their comments, contributions and approval.

There are three different types of information required, technical, social and financial. The information to be gathered will include:

a) Technical

a.1) Water Sources Survey

- Select water sources with the best potential using previously gathered data.
- Measure minimum rate of flow of the source.
- Measure quality of the water from the source by conducting physical and chemical analysis. The results of the analysis must comply with the WHO water quality guideline. The guideline is shown in Annex 1 of the manual.
- Evaluate desirability of the water source, including:
  - potential for contamination
  - potential for future expansion of gravity - flow system
  - ease of construction of the intake
  - how difficult it will be to protect the intake works against erosion, floods and contamination.
- Ensure that the community has water rights to the source.

Water rights

The final aspect of source investigation must include resolving the water rights of those people currently depending upon that source for their water. Although it is not the designer's responsibility to become involved in setting this question, it is his/her responsibly to make sure all disputes are resolved satisfactorily. If such problems cannot be solved, he/she should consider alternative sources. In the past, some projects have been deliberately sabotaged by villagers who felt they were not being considered fairly. At such times, there have been unhappy consequences, and much wasted time, labor and materials.

a.2) Topographical and Soil Surveys

- Evaluate potential pipeline routes and complete a topographical survey.
- Survey subsoil depth along the route to evaluate whether sufficient depth of soil exists to protect the pipeline (usually the trench for pipe laying be 0.6m wide and 0.8m deep).
- Locate potential sites for sedimentation tanks and reservoir.
- Locate potential sites for public fountains.
- Choose the pipeline alignment, which is not necessarily, the shortest, but one, which will minimize complicated construction procedures and overall cost.

Topographic Survey

There should be a topographic survey to be made along the proposed pipeline route. Such a surrey can be done using a theodolite.

The theodolite is a high precision instrument, and requires special training in its use. A two - men team is required, one for sighting through the instrument and an assistant, who is holding a vertical scale (rod) several meters tall. Although surveying with theodloite will yield measurements accurate to within a few centimeters, it is a relatively slow method. The accuracy of this instrument is not usually needed for the entire length of a pipeline survey, though it is sometimes useful to use to measure the depth of a U - profiles, or for accurate positioning of break pressure tanks.

a.3) Local Building Techniques and Supplies

- Assess the skills of local artisans, for example, are they skilled in the construction of reservoirs.
- Locate sources of building materials, such as sand and stone.
- Using the information gathered from the community survey, sketch a system layout including mainline and branch locations as well as potential public fountain sites.

b) Social

b.1) Population

An accurate population survey is necessary. Census data may be used if it is recent and known to be reliable but it is better to survey the population directly.

A population survey should identify and enumerate all people who will depend on the water source for their needs. This should include patients in health posts, students in school, workers in government offices, etc.

If the water is also required for animals, data on the number of animals should be collected during the survey.

b.2) Water Use Practice

- Determine present water sources.
- Examine the uses that are made of water.
- Estimate the water use requirements of the average person.
- Include allowance for future needs in the water estimate including increased demands as a result of newly introduced sanitation patterns.

b.3) Community Participation

- What will be the contribution of the community during the construction of the system? Based on the response of the community and observation made by the study team:
  - Identify, estimate and value the material contribution of the community.
  - Estimate and value the labor the community can contribute.
  - Estimate and value the cash contribution of the community if willing.
- Find out which people in the community will work. Will it just be men or will women and children also help?
- What is the best time of the year construction? For example, are these religious festivals or harvest times when workers could be occupies elsewhere?

c) Financial

One the most important issues in designing and implementing a water supply system is how to ensure the financial sustainability of the project. This can involve predicting what users will be able and willing to pay for water in the future.

- Analyze the willingness to pay survey result carried out during preliminary feasibility survey.
- Estimate the water tariff required to sustain the scheme irrespective of the ability and willingness to pay of the community (here full or operation and maintenance cost recovery should be decided).

The water tariff/users fee finally to be set will be the compromise of the above two important approaches in setting users fee.

2.4. Based on the results of the detailed community survey, the design and cost estimate of the project can be carried out

2.5. Construction

The construction of a gravity - flow water system is the most exciting part of a project. However, to make it successful it must be well organized. The plan listed in Table 2.1 is an example of a typical construction plan used in a gravity water scheme.

The time of construction should be arranged so that the maximum community contribution can be achieved.

Construction proceeds by sections to reduce organizational problems. Whoever is constructing the scheme i.e contractor or artisans, materials are purchased and transported to the area and are stored in an appropriate manner. For example if PVC pipes present must be stored in shade, cement has to be stored in dry place.

2.6. Operation and Maintenance

The operation and maintenance of a system are an important part of the implementation process. With care, a gravity - flow water system will operate continuously for many years without problem. Therefore, for this purpose the appropriate responsible body namely WATSANCOs, scheme operators and maintenance personnel must be organized and trained.

2.7. Evaluation

In this phase the project results are monitored at regular intervals and from the monitoring results we can get feed back for future projects implementation.

Note

1. For low cost and relatively smaller sized projects the pre- feasibility and the detailed community survey can be made together although not much preferable.

2. The report contents of the pre- feasibility survey and the detailed community surrey are shown in Annex 2 of this manual.

Table 2.1 Example of gravity system water supply construction schedule

Activity	Weeks											
	1	2	3	4	5	6	7	8	9	10	11	12
Source												
• Locate source	x											
• Buy and transport materials		x	x									
• Construct the source		x	x	x	x							
Reservoir												
• Determine location	x											
• Clearing and digging			x									
• Buy and transport materials			x	x	x			x	x			
• Floor slab construction				x	x							
• Wall construction						x	x	x				
• Roof construction								x	x	x		
• Fittings installation					x		x			x		
• Finishing										x	x	
• Fill water and test												x
Pipeline												
• Route identified	x											
• Buy and transport materials			x	x	x	x						
• Dig trench					x	x	x	x	x	x	x	
• Lay pipe					x	x	x	x	x	x	x	
• Test pipes for leakage						x	x	x	x	x	x	x
Public fountains												
• Locate fountains	x											
• Buy and transport materials							x	x				
• Clear the area								x				
• Construct the fountains									x	x		
• Install pipes and fittings									x	x		
• Test the pipes and fitting for leakage											x	x
Valve boxes												
• Locate boxes												
• Buy and transport materials								x				
• Construct the boxes										x	x	

3. DESIGN PRINCIPLES AND PARAMETERS OF THE DIFFERENT COMPONENTS OF A GRAVITY FLOW WATER SYSTEM

3.1 Locating an adequate and clean water source

3.1.1 Source Investigation

In investigating a source for a water supply system should not be confined to only the most convenient source. At this time, water treatment techniques are not generally practical and recommended for rural areas, thus it is necessary to locate the cleanest source possible, even if it is not the closest one to the village. Rather than relaying upon village description alone, the surveyor should personally visit all possible sources. Quantity and quality of flows must be determined, means to develop the intake works must be studied, and water rights must be investigated.

From the information collected on population survey, the surveyor can calculate the daily water requirements of the village. No source is feasible if, in 24 hours, it can not provide the daily amount of water required.

3.1.2 Flow - measurement techniques

In most investigations, accurate flow of measurement of a source will require some earthwork usually drainage channel. After the channels have been constructed, wait preferably for an hour for the water to achieve steady, constant flow, before attempting any measurements.

Discussed below are two simple methods for measuring the flows of springs and streams. Always measure the flow for several times, and deviant should be repeated. Question the villagers closely about seasonal variations in the flow.

3.1.2.1 Bucket and stopwatch methods

Springs flows are most conveniently measured by using a wide mouthed container (of known capacity) and bucket of capacity 10 - 20 liters is usually available in the village. For the most accurate results, the capacity of the container should be such that it requires at least 15 seconds to fill and smaller containers should only be used if nothing larger is available. An ordinary watch (that has a second counter) can be used for timings, but it is best in this case if two persons work together, one concentrating on the stopwatch, the other filling the container. The flow is calculated as:

$$\mathbf{Q_{safe} = V/t} \qquad \text{(Formula 3.1)}$$

Where, Q_{safe} = Safe (minimum) flow (liters/second)
V = Capacity of container in liters
t = Time to fill container (seconds)

Materials required for flow measurement:

- Pickaxe and shovel
- Piece of pipe
- A wide mouthed container of known capacity (volume)
- Stop watch (having a second counter)

Procedure for measuring flow of a source:

- The minimum(safe) flow has to be measured in the driest months of the year (April or May).
- Lead the water from the spring in one direction using a piece of pipe or a trench dug for this purpose.
- Once the spring starts to flow though the pipe or trench initially the flow is higher. The reason is that there is a stored water in the source. After some time the natural flow of the spring will start. Therefore it is preferable to measure the spring flow after one hour so that we can get the natural flow of the source.
- Prepare the container so that it can be filled with water from the source within some seconds or minutes depending on the flow of the spring and the capacity of the container. And, record the time required to fill the container.
- Measure the spring yield using the same procedure as above at least three times.
- Calculate the flow of the source in each trial using formula 3.1.
- The safe yield of the spring is calculated by averaging the three different trials flow results.
- We can calculate the 24 hours yield of the spring by multiplying the safe yield by 86400seconds (24 hours).
- We can compare the 24 hours yield of the spring with the daily water requirement of the village and we can decide whether the source can be built or not.

3.1.2.2 Velocity - area method

This method requires more work and is not as accurate as the other method, yet particularly for wide streams it can be easier to use.

Measure the surface water velocity of the streams by timing how long it takes a drifting surface float (such as a block of wood) to move down a measured length of the stream (this measured section must be fairly straight and free of obstacles, for the 6 - 10 times the average water depth).

Measure the cross - sectional area of the stream. The measurements should be repeated several times, averaging the results together. The average stream velocity is 85% of the surface velocity, and the flow is calculated as :

$$\mathbf{Q_{safe} = 850 \times V \times A} \qquad \text{(Formula 3.2)}$$

Where, Q_{safe} = Safe (minimum) flow (liters/second)
V = Surface velocity (meters/second), V = distance traveled/time taken
A = Cross sectional area of channel (m^2), A = Depth x Width

This method of flow measurement is applicable to streams of water depth at least 30 centimeters. Figure 3.1 illustrates the velocity - area method of measurement.

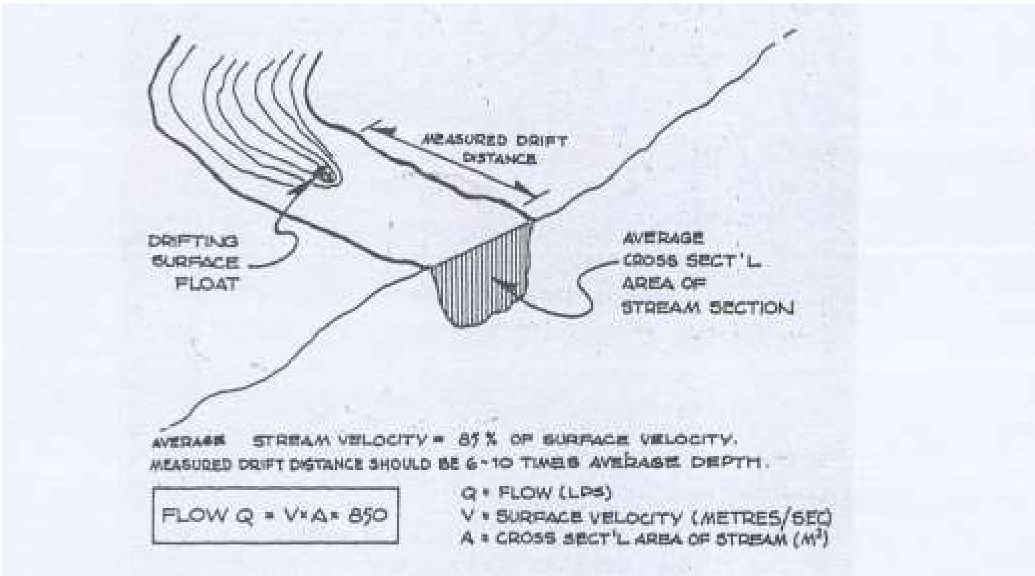


Figure 3.1 Velocity - area method of measurement

3.1.3 Safe (minimum) yield

The safe yield of the source is typically the minimum flow of the source during the dry season as determined in section 3.1.2. The safe yield is the flow of water that the source can be counted upon to deliver all year round, and it is this flow that is used in designing water systems. Unless the source is measured in April or May (the driest season) the villagers must be consulted to determine as accurately as possible what the safe yield of the source is. Should the water flow be critical, measurements should be repeated during the dry season, or stand-by sources also selected.

The maximum flows should also be determined by questioning the villagers. As the safe yield is important for pipeline and reservoir design, the maximum flow is also necessarily for estimating structural protection of the intake and overflow requirements.

For a water source to be considered as adequate, the 24 hours minimum yield (safe yield) of the source should cover the maximum daily demand of the people.

The 24 hour yield of the source (V_d) is determined by multiplying the safe yield of the source by the time in seconds within 24 hours.

$$\mathbf{V_d = Q_{safe} \times t} \quad \text{(Formula 3.3)}$$

Where, V_d = The 24 hour yield of the source in litters.
Q_{safe} = the safe yield of the source in liters
t = time in seconds in 24 hours = 86400seconds

3.1.4 Water quality

Drinking water quality as described by World Health Organization (WHO) guideline is such that the water is suitable for human consumption and for all usual domestic purposes including personal hygiene. Therefore, ideally drinking water should:

- be free from micro-organisms known to be disease causing. It should also be free from bacteria indicative of faecal contamination.
- be free from turbidity, undesirable colours, odours and taste.
- not contain excessive concentration of chemicals which may be harmful to health or economically undesirable.
- not contain excessive concentration of undesirable dissolved gasses.

In order to check the water quality is acceptable for health or not physical and chemical analysis should be made during water source identification and bacteriological analysis be made before the water supply system is in use.

The recommended WHO guideline for water quality is shown in Annex 2 of this manual.

3.2 Design period, population growth and water demand

3.2.1 Design period

Design period is the number of years from the date of design to the estimated date when the maximum conditions of the design will be reached.

Community water supply systems with distribution networks should be designed and constructed for 15 - 25 years life span. The choice of either 15, 20 or 25 years design period is made by the design team, based upon the amounts of potential change that he/she can foresee for the village. A remote area, far from future development efforts, might well be designed with a 25 year water demand projection. However, in an area where a near future development is assumed to be made, a shorter design period should be considered, because the long range water demands can not be accurately forecasted. For most of the rural water supply schemes the design period is proposed to be 20 years.

3.2.2 Population forecast

Selection of the design period leads directly to an estimate of the village population for the last year of that period. This design population is calculated using the current village population and the population growth rate for the design period. Now a days in Ethiopia the rural area growth rate of population is 2.83%.

There are different methods of forecasting population. The most widely used is the geometric rate method presented as below.

$$\mathbf{P_d = P_o\,(1 + i/100)^{n}} \qquad \text{(Formula 3.4)}$$

Where, P_o = the present population of the village
i = the population growth rate in %
n = the design period in years
P_d = the population of the village at the end of the design year (period).

3.2.3 Water Demand

The total water demands for a village (rural area) at the end of the design period is the sum of the per capita demand (domestic demand), commercial demand, public demand and loss and waste. If found to be necessary animal demand can also be included. The demand components are discussed in detail in the following sections.

a) Per capita demand/domestic water demand

This is the water required for persons of the projected village population. A per capita water demand of at least 15 - 20 liters per capita per day (l/c/d) is the present design standard in Ethiopia for rural areas.

The demand of 15l/c/d recommenced is based on the following uses:

- For drinking = 4 litters/capita/day
- For cooking = 6 litters/capita/day
- For body washing = 2 litters/capita/day
- For washing utensils = 1 litters/capita/day
- For cloth washing = 2 litters/capita/day

$$\mathbf{D_d = P_d \times I_d} \quad \text{(Formula 3.5)}$$

Where, D_d = Design domestic demand in liters
P_d = the population of the village at the end of the design year (period).
I_d = Per capita daily demand in l/c/d

b) Public demand

This is the water required for schools, hospitals, hotels, offices, military camps, cottage industries etc. Based on some ideal usage recommended in books, the amount of water needed daily by those facilities is given below.

Table 3.1 : Recommended public water demand for different institutions

It. No.	Facility	Demand in liters
1	Day school	2 - 3 liters/student/day
2	health posts and clinics (no bed)	20 liters/staff/day and 2 - 4 liters/patient/day
3	Government offices	2 - 3 liters/worker/day

In the absence of data the public demands can be estimated as 10 - 20% of the per capita water demand. For our design purpose we can use 15%.

c) Commercial demand

This is the amount of water required for commercial centers such as hotels, tea rooms, restaurants, local drink houses (tella bet), bus station, laundry etc. If there are number of such commercial centers in the village and even if there is a plan to construct hotels in the future, assessment on the water consumption from the existing commercial centers can be made and also projection in demand for the future planned commercial centers be made and the demand is included in the study. For bars and tea rooms 200l/bar can be taken.

d) Animal demand

As animals are the basis for the life of the rural community, it is preferable to include animals demand also in the design if the source has adequate yield. Although the water sector policy of our country recommends cattle trough not to be considered if a flowing stream is available at a distance of 3Kms from the village, visual observation and discussion with the beneficiaries are required to be made whether to include in the design or not.

Table 3.2 : Water demand for different types of animals as recommended by Agricultural Offices

It. No.	Type of animal	Average demand in liters	Minimum demand in liters
1	Cattle (cow and ox)	15 liters/cattle/day	10 liters/cattle/day
2	Goats	3 liters/goat/day	2 liters/goat/day
3	Sheep	5 liters/sheep/day	3.5 liters/sheep/day
4	Pack animals(donkeys, mules and horses)	6 liters/animal/day	4 liters/animal/day

To simplify the process of the design, irrespective of the size of the animals usually an average animal demand of 5 liters/animal/day is recommended to be used in the design of rural water supply schemes.

e) Loss and waste

Water losses include water used at the treatment plant for back washing, leakage in the pipes and unmetered usage, such as overflow from service reservoirs.

Usually loss and waste is estimated to be 10 - 20% of the sum of domestic, commercial, public and animal demand depending on various factors. In our case 10% can be used for loss and waste in the design. the loss and waste will be more if the system components mainly pipes are constructed poorly.

f) Average daily demand (A_d)

The average daily demand is the total sum of the domestic demand, commercial demand, public demand, animal demand, and loss and waste.

g) Maximum daily demand (M_d)

The water consumption in a year varies from day to day. The maximum day demand is the highest demand in a day of specific year. The ratio of the maximum day consumption to the average daily consumption is the maximum day factor (M_f). The maximum day factor is taken as 1.1 - 1.3, usually 1.2. This maximum daily demand is usually happened to be in dry seasons. When the water problem of the community is very sever and the source has a low yielding capacity, this factor can even be taken as 1.

Maximum daily demand, M_d = M_f x Average daily demand (Formula 3.6)

This is the water consumption which is used to design the source capacity, capacity of storage reservoirs, capacity of treatment plants and design of gravity main pipe (pipeline from source to reservoir).

h) Maximum daily flow (Q_{max})

This is flow in litters per second required to deliver the maximum daily demand of the people at most within 24 hours.

$$Q_{max} = \frac{\text{Maximum daily demand } (M_d)}{\text{time}} \qquad \text{(Formula 3.7)}$$

Where , Q max = Maximum daily flow in liters/second
M_d = maximum daily demand in liters
t = time in seconds in 24 hours = 86400seconds

i) Peak hour demand (P_d)

The demand of water in any system is not a constant, it fluctuates up and down throughout a 24 hours period. The peak hour demand is the highest demand of water expected to occur over the 24 hours period. This is the time when the most water is used and often times, all taps are fully open. In general, peak demand accounts for 20 - 25% of the operating time (4-6 hours per day) usually in the day time hours around meal times.

The peak hour demand is the peak hourly demand of the maximum day. This demand is greatly influenced by the size of the town and social activity pattern. In practice the peak hour factor (P_f) at the maximum day varies from 1.1 to 4 depending on local conditions such as size of the town and social activity pattern, and is determined by the designer.

For our purpose in the rural areas, a peak hour factor of 2 is recommended to be used.

$$\text{Peak hour demand, } P_d = P_f \times \frac{\text{Maximum daily demand}}{\text{24 hours}} \qquad \text{(Formula 3.8)}$$

P_d = Peak hour demand in liters/hour and the maximum daily demand is in liters/day

The distribution pipes (pipes after the reservoir to the public fountain or to any point) should be designed for the peak hourly demand.

j) Peak hourly flow (Q_{peak})

This is flow in litters per second required to deliver the peak hourly demand of the people at most within 24 hours. It is calculated using

$$Q_{peak} = \frac{\text{Peak hour demand } (P_d)}{\text{time}} \qquad \text{(Formula 3.9)}$$

Where , Q_{peak} = Peak hour flow in liters/second
P_d = Peak hour demand in liters/hour
t = time in seconds in 1 hour = 3600seconds

3.3 Reservoir and sedimentation tank

3.3.1. Reservoir (Storage tank)

3.3.1.1 The necessity for a reservoir

Although the village water needs are based upon a minimum requirements of 15 - 20 liters per person per 24 hours day, in actuality just about all of this water will be determined during daylight, a period of 8 - 12 hours. The reservoir tank serves to store water that is provided by the source during low demands periods (such as overnight) for use during high demand periods (such as early morning).

District reservoirs are located high enough to give adequate pressure in the pipeline and are situated to divide the network into manageable areas. The decision to build reservoirs depends either on the flow rate of water from the source or its location.

In general a reservoir is needed if the source cannot supply all of the water needed in a working day (within 8 - 12 hours), but can supply it in 24 hours.

A system will require a reservoir when:

- The daily water demand is greater than the yield of the source during the daylight hours (8 - 12) hours. In our case the people are expected to collect their 24 hours demand within 8 hours.
- A reservoir may also be installed to save money (the pipeline distance from source to village is too far that it is more economical to use a smaller pipe size and build a reservoir tank). A study should be undertaken to determine weather it is cheaper to:
  - put in a large pipe from the source that would supply enough water in a working day, even during peak hours; or
  - put in a reservoir to cope with peak demands and a smaller, cheaper pipe from the source to the reservoir.
- Storage reservoir is needed to distribute or equalize pressures and to furnish water for emergencies and breakdowns.

Small reservoirs may also be located at each public fountain/standpipe. This arrangement may be necessary if some groups of users are drawing much more water than other groups.

3.3.1.2 Capacity of reservoir

When we come to the designing of the reservoir tank, the most common attitude of the villagers is " the bigger the better". While this is an understandable ideas, here is no point in building any tank so large that the source will never be able to fill it up during the overnight re-filling period. The storage capacity of the reservoir is actually determined by the projected village water needs and the safe yield of the source. Just for the sake of observation in supply and demand, the daily demand pattern of a certain village rural village is shown in Annex 3 of this manual

The maximum size of a storage tank should not be greater than needed to store than water yielded by the source during the night.

The capacity or size of the storage tank is usually determined by comparing the supply curve with the demand curve. This is done by looking at the ration of the safe (minimum) source flow, Q_{safe} over the maximum daily flow, Q_{max}.

$$\textbf{Ratio of storage requirement, Sr} = \frac{\textbf{Safe yield of source } (Q_{safe})}{\textbf{Maximum daily flow } (Q_{max})} \quad \text{(Formula 3.10)}$$

After calculating the ratio use the following table to estimate the storage capacity (requirements) of the reservoir.

Table 3.3 : Table showing storage requirements of reservoir

It. No.	Ratio (S_r) = $\frac{Q_{safe}}{Q_{max}}$	Recommended storage volume (m^3)	Remark
1	< 1	The source is not adequate and not recommended for construction	
2	1 - 1.5	1/2 x Vmax (half of the maximum daily demand)	Usually for actual design 1/3 x Vmax is recommended because the 1/2 x Vmax demand is needed only in the final years of the design period
3	1.5 – 2	1/3 x Vmax (one third of the maximum daily demand)	
4	2 – 3	1/4 x Vmax (one fourth of the maximum daily demand)	
5	3 – 4	1/8 x Vmax (one eighth of the maximum daily demand)	
5	> 4	No storage is required as the source is too sufficient to supply the 24 hours demand within maximum of 6 hours.	To have at least some water incase of source maintenance 1/8 x Vmax capacity tank can be constructed.

In many practical cases the ratio of the storage requirement falls between 1 and 2. Therefore as a rule of thumb, storage (reservoir) capacity can often be taken as one third of the maximum daily demand of as shown in formula (3.10) i.e. 33% of the maximum daily demand, which is equivalent to 8 hours flow. However, all the above figures shown in Table 3.3 are approximate based on field experiences and may be adjusted to fit specific circumstances.

Reservoir capacity, $R_c = \frac{\text{Maximum daily demand}}{3}$ (Formula 3.11)

In any case the maximum capacity of reservoir should not be more than 12 hours flow of the source or 50% of the maximum daily demand.

Example,

Given, $V_{max} = 105m^3$, $Q_{safe} = 2$ lit/sec and $Q_{max} = 1.2$ lit/sec

Therefore, $S_r = \frac{2 \text{ lit/sec}}{1.2 \text{ lit/sec}} = 1.6$ which is between 1 and 2.

Hence, $R_c = 1/3 \times V_{max} = 1/3 \times 105m^3 = \mathbf{35m^3}$

3.3.1.3 Reservoir site selection & materials of construction

The site selection for the reservoir should be on stable ground, which will not be threatened by landslide or erosion.

The reservoir can be made from masonry, concrete, sheet metal etc. And the shape can be circular or rectangular.

3.3.2 The sedimentation tank

This is a rectangular tank that stills the water to allow solids to settle out. It has a minimum volume of water equivalent to the amount of water flowing through it in two hours. It has a depth of 1meters or more and a length to width ratio 4:1 or more. Water enters one end at half depth and is removed at the other end at the top. The tank can be built according to the most economical design available.

Sedimentation tanks are never considered to be storage for design purposes, and so they are always kept full.

For our purpose as we are thinking to have a spring having clear water, the construction and design of sedimentation tank is not dealt in depth.

3.4 Design of pipelines

3.4.1 Hydraulic Theory

3.4.1.1 Introduction

In this chapter the basic hydraulic principles that govern the behavior of gravity flow water system will be presented. In order to properly design a water supply system, the designer should read, study, and repeatedly refer back to this chapter until he/she is satisfied with his/her knowledge of these principles.

3.4.1.2 Energy

To move water, whether moving it uphill, down hill or horizontally, requires energy. As its name implies, in a gravity - flow water system the source of energy is the action of gravity upon water.

A gravity flow system is " powered" by gravitational energy. The amount of such energy in a system is determined by the relative elevations of all points in the system. Once it has been constructed, all points in the system are **immovably fixed** (i.e buried into the ground) and their relative elevations can not change. Thus, for any system, there is a fixed, specific quantity of gravitational energy available to move water.

As water flows through pipes, fittings, tanks etc. some energy is lost forever, dissipated by friction. Due to the changing topographic profiles of the system, at some points there may be a minimal amount of energy (i.e low pressure) , while at other points there may be an excessive amount of energy (i.e high pressure). A poorly designed or constructed system will not conserve energy properly enough to move the designed quantities of water through the pipeline.

The purpose of pipeline design , therefore, is to properly manipulate frictional energy losses so as to move the desired flows through the system, by conserving energy at some points and burning it off (by friction) at other points. This is accomplished by careful selection of pipe sizes and strategic locations of control valves, break pressure tanks, reservoirs, tap stands etc.

3.4.1.3 Head (the measure of energy)

In hydraulic works, rather than repeatedly calculate water pressure, it is easier practice to simply report the equivalent height of the water column. Technically, this is called **head** and represents the amount of gravitational energy contained in the water. In the metric system of units, head is always measured in meters.

By this practice, a water pressure of 140KN/m^2 is reported as 14 meters of head, a pressure of 400KN/m^2 is 40 meters of head , 500KN/m^2 is 50 meters of head etc.

3.4.1.4 Fluid static (water at rest)

Any person who has ever dived to the bottom of a lake or swimming pool quickly learned that the water pressure increased as he/she descends down but that swimming horizontally at a constant depth produced no change in pressure. This common experience serves to illustrate a major principle in hydraulics as:

" Water pressure at some depth is directly related to the vertical distance from that depth to the level of the surface, and is not affected by any horizontal distances."

Consider the system shown in Figure 3.2. The water pressure at point A is determined by the depth of water at the point. The pressures at point B and C are likewise determined by the height of the vertical distance from those points to the level of the water surface.

Point	Water pressure	Head
A	100KN/m^2	10 meters
B	200KN/m^2	20 meters
C	350KN/m^2	35 meters

In a pipeline where no water is flowing, the system is termed being in static equilibrium. In such systems, the level of the water surface is called the static level, and the pressures are reported as static heads.

In Figure 3.2, the control valve is closed and no water is flowing in the pipe. If small open ended tubes were inserted into the pipeline, the water level in each tube would rise exactly to the static water level. The height of water in such tube is the pressure head exerted on the pipeline at that point.

Since no water is flowing, there is no energy lost due to friction and the static level is perfectly horizontal.

3.4.1.5 Fluid dynamics (water in motion)

Now suppose that the control valve at point C in Figure 3.3 is partially opened, allowing a small flow of water through the pipeline (and also assume that the tank refills as fast as it drains, so that the surface level remains constant). The water levels in each vertical tube decrease a bit. As the valve is opened further and further to allow greater flows through the pipeline, the water levels in the tubes drop even lower, as shown in Figure 3.3.

It can be seen that the water heights in these tubes form a new line for each new flow through the system. For a constant flow, the line formed by the water height will remain steady. The system is now said to be in dynamic equilibrium. The line formed by the water levels in the tubes is called the **hydraulic grade line**, commonly abbreviated as HGL. The line represents the energy level at each point along the pipeline. Different flows establish a different dynamic equilibrium, and a new HGL.

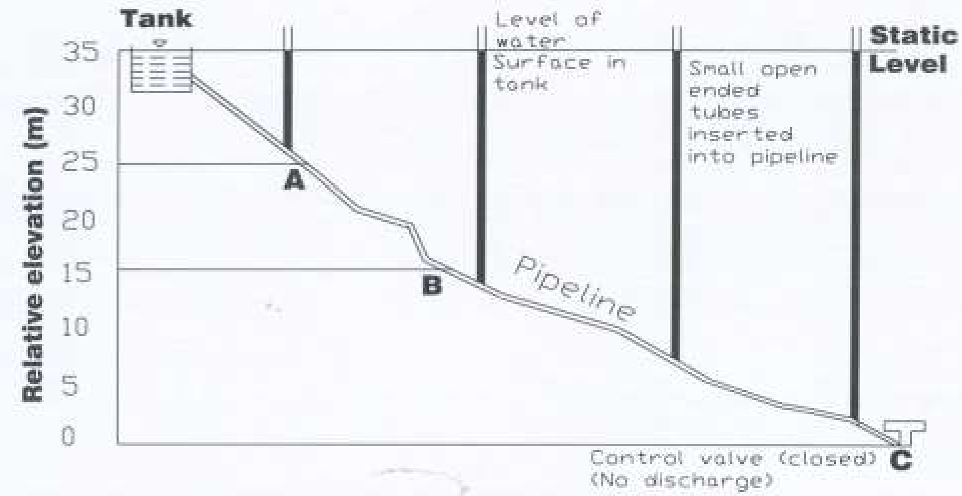


Fig 3.2 Static Equilibrium

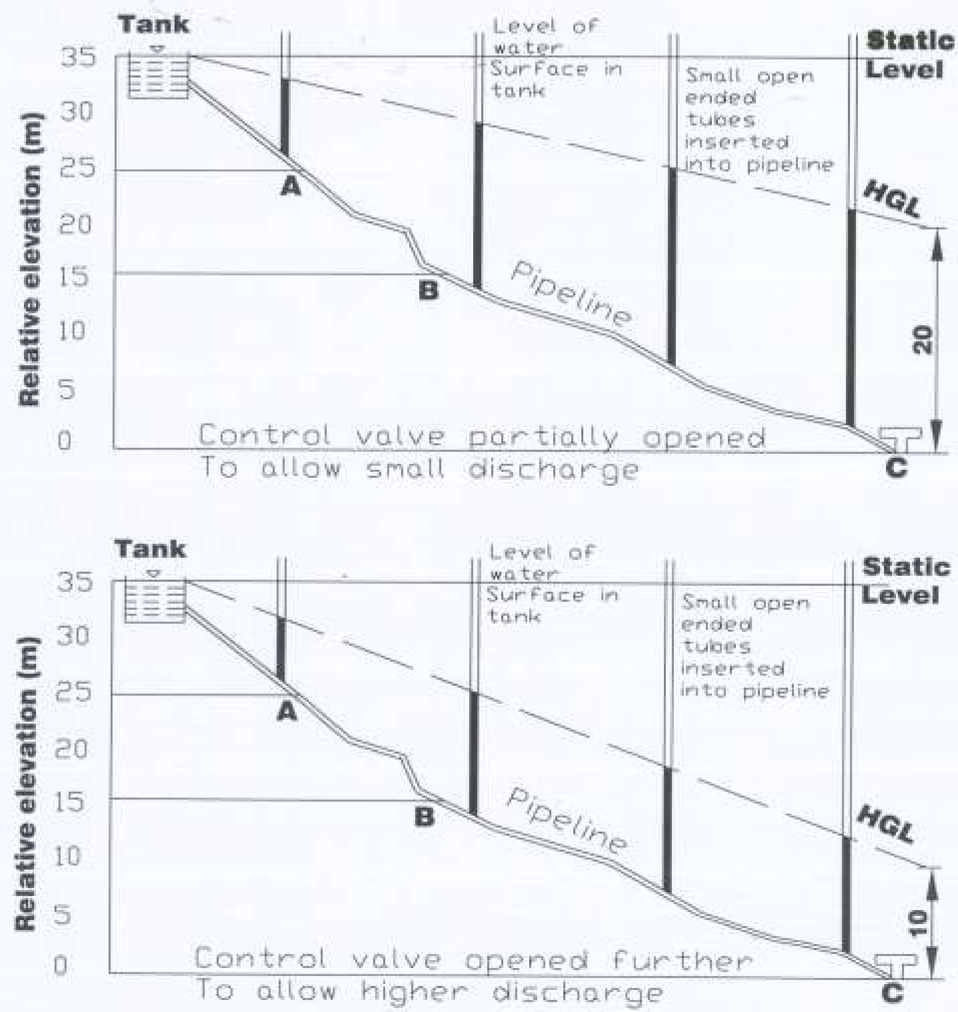


Fig 3.3 Dynamic Equilibrium

3.4.1.6 Hydraulic Grade Line (HGL)

The HGL represents the new energy levels at each point along the pipeline. For any constant flow through the pipe there is a specific, constant HGL. The vertical distance from the pipeline to the HGL is the measure of pressure head (i.e. - energy), and the difference between the HGL and the static level is the amount of **head lost by the friction of the flow.**

The water pressure at air/water interface (such as the surfaces in tanks or discharges at tap stands) is zero. Thus, the HGL must always come to zero whenever the water comes into contact with the atmosphere.

Since frictional losses are never recovered, the HGL always slopes dawn along the direction of the flow. The steepness of the slope is determined by the rate at which energy is lost to friction. It should always be above the pipeline, but if the pipe line rises, it is possible for the HGL to fall below the pipeline. This indicate the presence of negative residual head, this means there is no enough gravitational energy to move the desired quantity of water. This situation should be avoided and should be checked for in the design stage.

Only under static conditions as the HGL perfectly horizontal, although for practical purposes the HGL may be plotted as horizontal for extremely low flows in large pipes (where the head loss is less than 1/2meter per 100 meters of pipeline). For practical purposes, the HGL will never slope upwards.

Steps in plotting the hydraulic grade line,

1. Determine the elevations of the hydraulic grade line at the desired points (using Table 3.6).
2. Starting from the start or end of the pipeline, indicate points of desired head. At least at each distribution point or junction points and if possible even at intermediate points.
3. Plot the HGL according to the exact head losses at each point. Remember, when determining the available/free head at each new point along the pipeline, be sure and subtract the head losses from preceding point.

3.4.1.7 Friction (lost energy or head Loss due to friction)

As mentioned at the beginning of this chapter, a system has a specific amount of gravitational energy, determined by the relative elevations of points in the system. As water flows through the pipeline, energy is lost by the friction of the flow against pipe walls, or through fittings (such as reducers, elbows, control valves, etc), or as it enters/discharges from pipes and tanks. Any obstruction to the flow, partial or otherwise, causes frictional losses of energy.

The head loss is the reduction of pressure in a pipe which may be due to friction in the pipe and pipe fittings, or valves and can be expressed as a change in head.

The magnitude of energy lost due to friction against, some obstacle is determined by several factors, The major factors would be the roughness of the obstacle, and the velocity of the flow. Minor factors would include water temperature, suspended particles etc.

The diameter of pipe, and the amount of flow through it, determine the velocity of the flow. The greater the flow, the faster the velocity, and the greater the frictional losses. Likewise, the rougher the surface of the obstacle, the grater the frictional losses.

Frictional loses are not linear: doubling the flow does not necessarily double the losses: usually, losses are tippled, quadrupled, or even greater.

The Hazen - William's formula is widely used in the design of water distribution systems.

The head loss due to friction in the pipe is calculated using the Hazen - Willam's Formula as follows:

$$\mathbf{H_f = \frac{12.25 \times 10^9 \times L \times Q^{1.85}}{C^{1.85} \times d^{4.87}}} \qquad \text{(Formula 3.12)}$$

Where, H_f = Head loss due to friction in meters
L = Length of pipe in meters
d = Internal diameter of pipe in millimeter
Q = Discharge in liters/sec
C = Friction coefficient, C = 120 for new GI pipe,
C = 110 (recommended for GI pipes in design)
C = 100 for old GI pipe and
C = 150 for PVC pipes

The pipe diameters mainly in use for gravity water systems are shown in Table 3.4 below.

Table 3.4: The millimeter and the inch equivalent of the internal diameter (d) of different pipes

Smaller diameter pipes			**Larger diameter pipes**		
"d" in millimeter (mm)	**"d" in meter (m)**	**" d " in inch (in)**	**"d" in millimeter (mm)**	**"d" in meter (m)**	**" d " in inch (in)**
12.5	0.0125	1/2	50	0.05	2
19	0.019	3/4	63	0.063	2 1/2
25	0.025	1	75	0.075	3
32	0.032	1 1/4	90	0.09	3 1/2
40	0.04	1 1/2	100	0.1	4

3.4.1.7.1 Losses in fittings and valves (Minor losses)

A few fittings and valves used to control public fountains and reservoirs do not significantly increase head losses. However, if a large number are used throughout the system, to regulate flow or there are a significant amount of bends and joints in the pipeline, allowance would be made for the increase of head losses. In general 5% (preferably) to 10% of the head loss in the pipes can be added to account for the minor losses.

3.4.1.8 Frictional Head Loss Factors

The common method in head loss calculation is to calculate the head loss using the frictional head loss factor (f). The frictional head loss factor is expressed as " meters of head loss per one meters length", or as " m/1m ".

For the steel pipes of C = 110, the Hazzen William's formula can be simplified as follows,

$$H_f = \frac{2.05 \times 10^6 \times Q^{1.85}}{d^{4.87}} \times L = f \times L \qquad \text{(Formula 3.13)}$$

Where , f = Frictional head loss factor
L = Length of pipe in meters (m)

Nomograms or tables are often used in solving pipe flow problems involving the Hazzen - Williams's formula which shows the frictional head loss factor (f) for different flows and pipe diameters. The Table prepared for GI pipe with C = 110 is shown in Table 3.5 of this design manual.

3.4.1.9 Residual head (excess energy or free head of available head) , (F_h)

The significance of residual heads at tap stands, reservoirs, and break pressure tanks must be understood by the designer before a proper system can be planned.

Residual head is the amount of energy remaining in the system by the time that the desired flows has reached the discharge point. It represents excess gravitational energy.

When plotting the HGL for a flow which discharges freely into the atmosphere (such as into a tank or out of a tap), the residual head at the discharge point may turn out to be either positive or negative, as shown below.

Positive residual head. This indicates that there is an excess of gravitational energy, that is, there is enough energy to move an even greater flow through the pipeline, if allowed to discharge freely. A positive residual head means that gravity will try to increase the flow though the pipe As flow increase, the frictional head loss will decreases the residual head. The flow will increase until the residual head is reached zero. See Figure 3.4.a below.

Negative residual head. This indicates that there is not enough gravitational energy to move the desired quantity of water, hence this quantity of water will not flow. The Hydraulic Grade Line (HGL) must be replaced using a smaller flow and/or larger pipe size so that the head loss due to friction can be reduced. See Figure 3.4.b below.

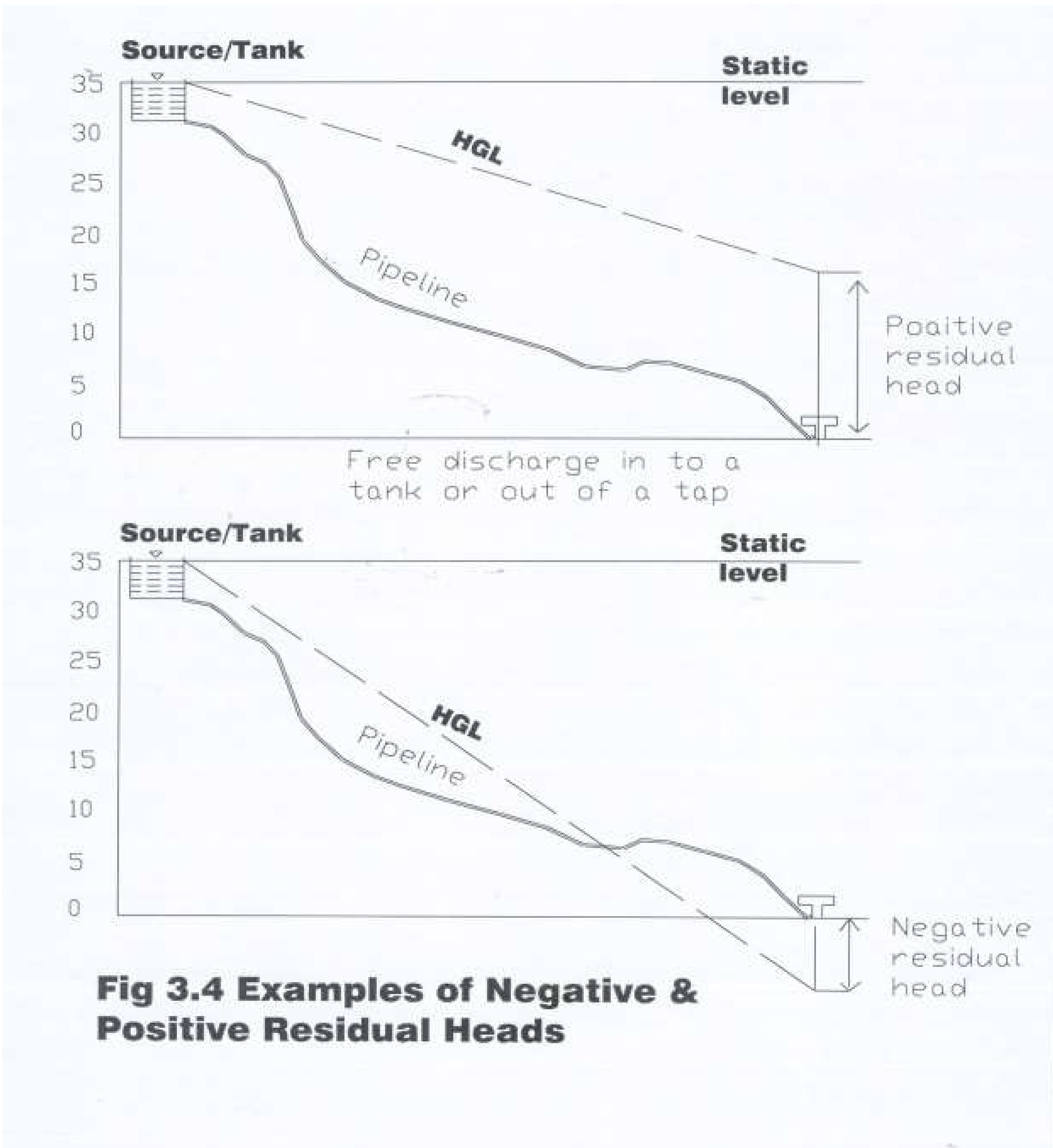


Fig 3.4 Examples of Negative & Positive Residual Heads

In public fountains or at any point in a pipe line the excess/free/residual head is the difference in meters of water between the reservoir or break pressure tank or source elevation, and the sum of the elevation of the point and head loss in the pipe from the reservoir/break pressure tank/source to the point.

Free head at any point , Fh = Elevation of reservoir or source - Elevation of the point - Sum of head losses in the pipe from the reservoir or source or BPT to the point) (Formula 3.14)

Example, Given elevation of reservoir = 2050m,
Elevation of the public fountain = 2000m,
Head loss from reservoir to fountain = 23 m

Therefore, Free head (F_h) at public fountain = 2050 m - (2000m + 23m) = 27m

3.4.1.10 Maximum and Minimum Pressure Limits in Pipes

As indicated already it is seen that pipe sizes are selected because of frictional head loss considerations. However, there is yet another consideration which determines what type and diameter of pipe must be selected. This consideration is pressure , and will dictate whether Polyvinyl Chloride (PVC) or Galvanized Iron (GI) pipes to be used. The choice is determined by the maximum pressure that the pipe will be subjected to (these maximum pressures are always the result of static pressure levels). The maximum pressure limits recommended for each of these pipes is written below.

PVC pipes - maximum pressure rating=10 bars = 100 meters of water column.
GI pipes - maximum pressure rating = 25 bars = 250 meters of water column.

3.4.1.11 Velocity limits

The velocity of flow through the pipeline is also another matter of consideration in pipe line design. If the velocity is too great, suspended particles in the flow can cause excessive erosion of the pipe; and if the velocity is too low, then these same suspended particles may settle out of the flow and collect at low points in the pipeline, eventually clogging it if left unattended.
The recommended velocity limits in the design of pipelines are:

- Maximum = 2 meters/second - 2.5 meters/second
- Minimum = 0.5 meters/second - 0.7 meters/second
- The most preferable velocities are within the range of 0.7 m/s to 2 m/s

The velocity of flow in the pipe using the selected standard pipe is calculated as follows:

$$\mathbf{V = 4Q / 3.14 \times d^2} \qquad \text{(Formula 3.15)}$$

Where , V = velocity of flow in m/s
d = internal diameter of the pipe in meters (m)
Q = discharge through the pipe in m^3 /s

3.4.1.12 Pipeline Design

After knowing all the above theories and design guidelines, the main purpose of the pipeline design is to determine and most economical and technically acceptable pipe type (PVC or GI) and the pipe diameter to be laid with the prevailing geological condition and to convey the required amount of water.

3.4.1.12.1 Layout of Water Distribution System

The commonly used layouts of water distribution networks are :
a) Tree or dead end systems
b) Gridiron system

The tree system comprises of a trunk line to which are joined mains that are connected to sub-mains. The sub - mains further branch out to become laterals which are further reduced in size to branches. The advantages of this system are ease of calculating pipe systems, use of comparatively small sized pipes and use of fewer fittings. The disadvantage is disruption of water supply beyond a repair point. For rural water supply systems the tree or dead end system is widely used.

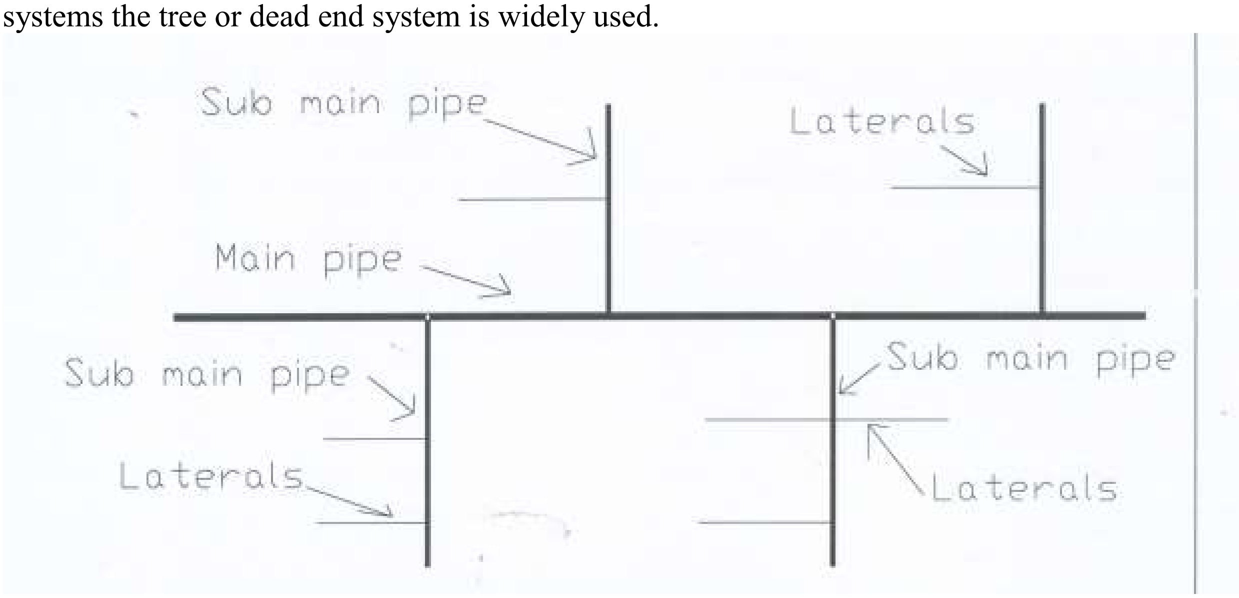


Fig. 3.5 Layout of tree/dead end system

3.4.1.12.2 Possible layout of tree /dead end gravity water supply system

The following are four common layouts for a simple rural water supply gravity system.
a) Placing storage tank at the point of use (Figure 3.6.a)

General comments on the layout are:
- The inflow to each reservoir can be regulated so that each area receives a set of allotment of water. If the people at each reservoir tend to waste water then they can

only waste their allotment and not that of others. In a public fountain system with storage at the source, wastage would be much greater if faucets left open.
- The small reservoir acts as break pressure tanks in the system. This means that faucets at the point of use have only the head of the reservoir itself, and will last for a longer period of time because of the reduced pressure. Reduced head at the point of use also reduces wastage.
- Storage at the point of use means that the main distribution pipe is in use at all time. It can therefore be smaller diameter and thus reduces costs. Influence on total cost will depend on the flow of the source compared to average daily usage as this will influence storage costs.
- It is sometimes easier to obtain community report and cultivate felling of ownership and consequently, improve maintenance through the construction of small scattered reservoirs as compared to one large distant reservoir and fountains. Additionally, the construction of small reservoirs allow each segment of the community to work at its won pace during construction and lack of community participation will be less likely to impede the project.

b) Placing storage tank between source and distribution points (Figure 3.6.b). This system has many beneficial features.

General features on the layout are:
- The pipeline to the point of storage is small.
- The tank serves as a break pressure point in the system and can be placed to regulate pressure at the distribution points.
- Only one storage tank (larger size) need to be constructed and maintained.
- Water may be treated to improve quality at one center point.
- Public fountains can be easily placed at the desired distribution point.

c) No storage, and distribution is direct to public fountains (Figure 3.6.c)

General features on the layout are:
- This type of system requires a larger diameter pipeline, which in many cases, increase the cost of the overall system substantially.
- This system also requires a source flow to provide peak demand without storage. Means the safe yield of the spring should be high so that it can yield the maximum daily requirement of the people within 8 -12 hours.

d) Storage tank is near or at the water source, then distribution is direct to public fountains (Figure 3.6.d)

General features on the layout are:
- This type of system requires a larger diameter pipeline, which in many cases, the most expensive option.
- In certain cases, however, storage may be included with construction of the intake and prove cost-effective for small systems.

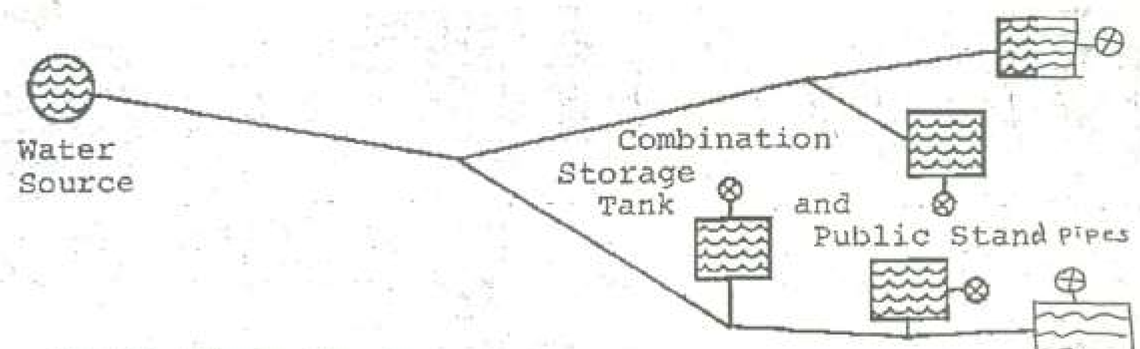


Fig. 3.6.a Placing the storage tank at the point of use

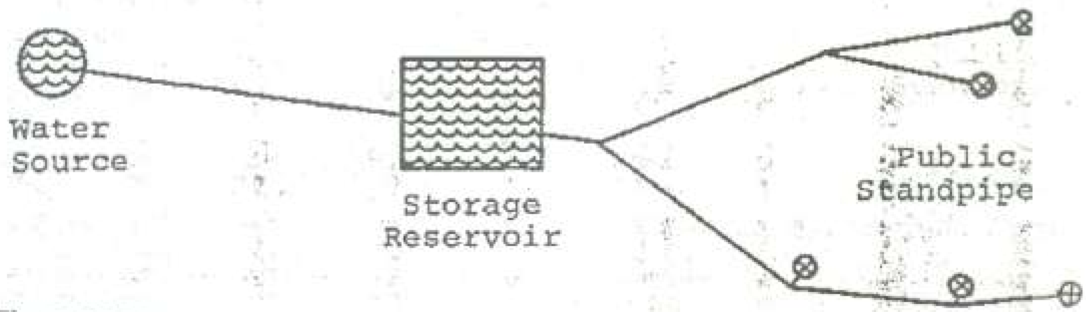


Fig. 3.6.b Placing the storage tank between source and distribution

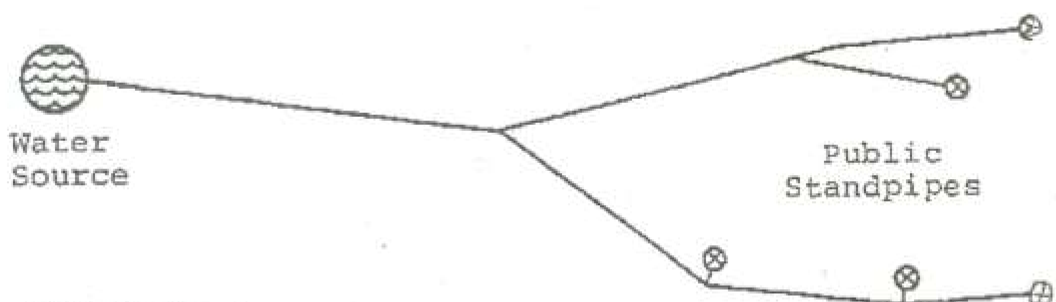


Fig. 3.6.c No storage tank & distribtion is direct to fountains

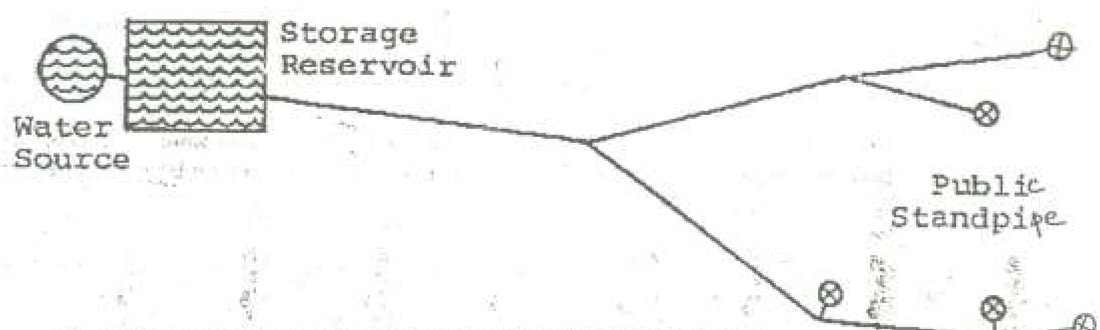


Fig. 3.6.d Storage tank is near the water source

3.4.1.12.3 Pipes design criteria

a) Conveyance system: - the rising and gravity mains (pipes from source to reservoir) shall be designed for a maximum daily demand.
b) Distribution networks:- the distribution network (pipes from reservoirs to public fountains or other points) shall be designed for the peak hour demand.

The pipe diameters mainly in use for gravity water systems are shown in Table 3.4

In deciding the pipe diameters, to start with the economical size of service pipe can be calculated using the empirical formula by Garg as:

$$\mathbf{d = 1.22 \times \sqrt{Q}} \qquad \text{(Formula 3.15)}$$

Where, d = the internal diameter of the pipe in inch
Q = discharge through the pipe in liters per second (l/s)

For more details in the design procedure see chapter 4 of this manual.

3.5 Break Pressure Tanks (BPT)

These are small tanks whose function is to allow the flow to discharge into the atmosphere, there by reducing its hydrostatic pressure to zero, and establishing a new static level.

These tanks are required when the pipe elevation is sufficiently below the source to exceed the pressure capacity of the pipe (i.e. more than 100 meters for PVC and 250 meters for GI pipes) or the residual head at public fountain/faucets is more than 60 meters. They are tanks that water flows into and out of them, controlled by a float valve.

Strategic placing of the break pressure tanks can minimize the amount of PVC and GI pipe which must be used in a system.

The break pressure tanks can be constructed from masonry or concrete or sheet metal like any tank/reservoir. And the shape can be circular or rectangular.

3.6 Public tap stands (public fountains)

The public fountains are the most frequently used components of the entire system.
The following are the considerations, which must be taken when locating a public fountain.

3.6.1 Public fountain location

Selecting the sites for the pubic fountains will be a process of compromises, since no single point is opt to meet all the ideal requirements.

The number of public fountains required in a system will be greatly influenced by the geographical layout of the village and the number of people living in the village. The school if any and the health institutions should also each have one public fountain.

The probable number of taps/faucets (N_p) for community use can be determined by dividing the design population by the average people serving capacity of one faucet as:

$$N_p = \frac{\textbf{Design population } (P_d)}{\textbf{No. of people to be served by one faucet}} \qquad \text{(Formula 3.16)}$$

The number of public fountains with average of four faucets is determined by dividing the number of faucets determined in the above formula by 4.

Concerning distance a public fountain should not be located in a maximum distance of 500 meters from the furthest beneficiary.

Of course the final number of taps will depend on the settlement pattern of the people in different gotts of the village.

3.6.2 Flow & number of people served per tap/faucet

A public fountain can have more than one faucet usually four. The number of faucets in a public fountain depends on the number of people living in that area and the standard flow of one faucet.

The most widely used faucet is 3/4" diameter. Different countries have different flow standards in faucets. For example in Malawi, the public fountains operate 16 hours/day at 0.075l/s to supply 160 people with 27 l/c/d in one faucet. In Nepal the design values are 12 hours per day at 0.225 l/s (maximum flow in 3/4" faucet) to supply 215 people with 45 l/c/d in one faucet.

In Ethiopia it is recommended that the public fountains operate 8 hours/day at 0.15l/s to supply 150 to 200 people with 25 - 20 l/c/d respectively in one faucet.

In practical designs it is recommended that one faucet to serve 150 people or 30 households/day including the queuing time and time required for filling of the pots. This implies if there are 600 people using one public fountain, it should have 4 faucets.

With the above principles, the number of public fountains and their location should be decided. Once their number and location decided, should be located on the pipeline lay out for further calculation of free heads and pressure limits.

3.6.3 Residual head (free head)

The residual head at the tap stand/public fountain is important. If too much, it will cause accelerated erosion of the interior of the faucet, and if too low, will result in low flows.

Therefore, when designing a public fountain the residual head in the public fountain should not be less than 5 meters and the maximum should not be more than 60 meters. Preferably the residual head is required to be within the range of 10 - 30 meters.

The static pressure when the faucet/tap is closed also must not exceed the pressure rating capacity of the pipe in the public fountain, and the tap line as presented in section 3.4.1.10 in this manual.

3.6.4 Structural considerations

A public fountain may be constructed of bricks, stone, concrete or HCB.

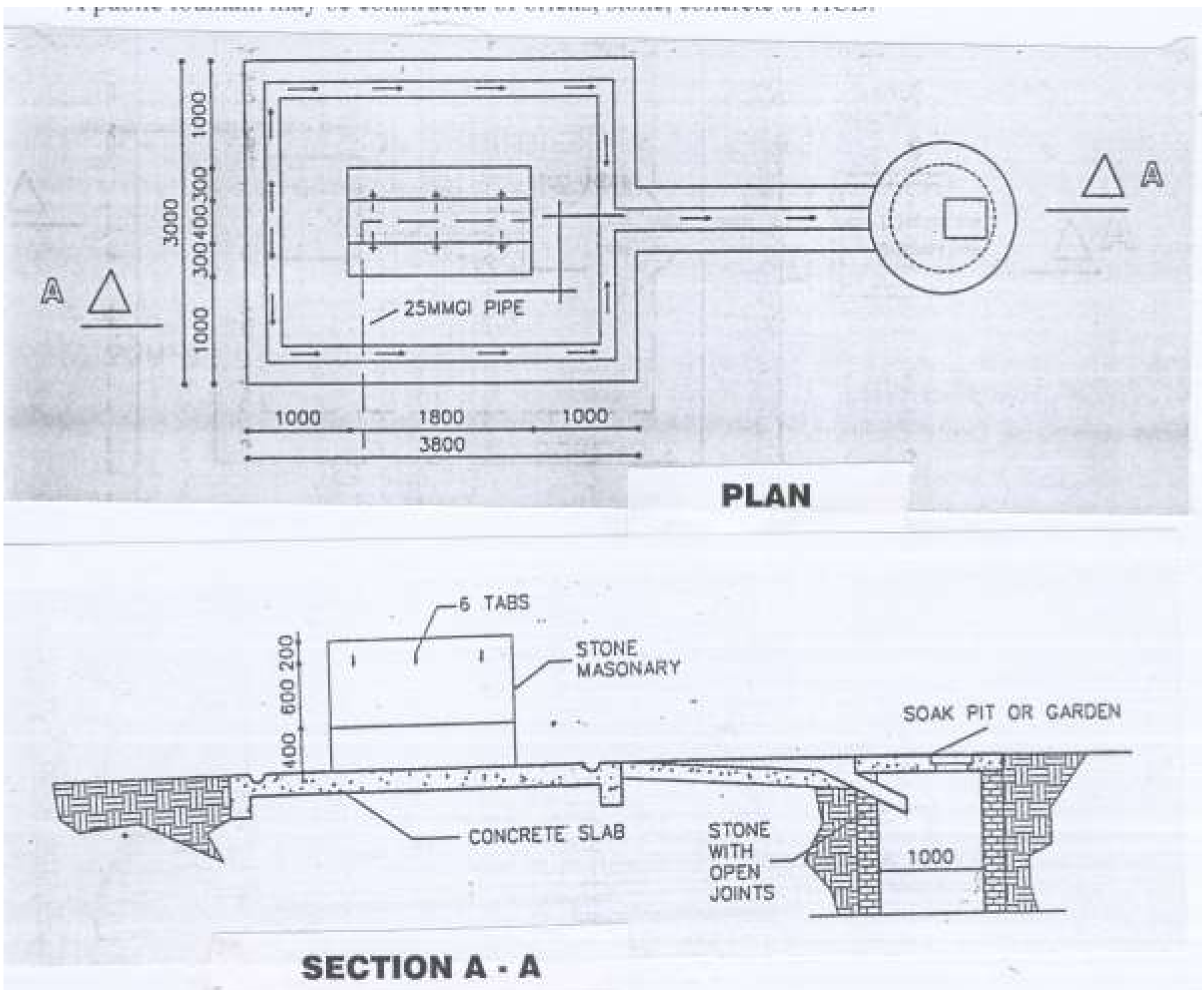


Fig. 3.7 One of the Public Fountain Designs with Six Faucets

3.7 Air Release Valves and Washouts

3.7.1 Air blocks

An air block is a bubble of air trapped in the pipeline, whose size is such that it interface with the flow of water through the section.

When the pipeline is first constructed, or subsequently drained for maintenance purposes, it is "dry", that is, all points are filled with air at atmospheric pressure. When water is allowed to refill the pipeline, air cannot escape from certain sections and is trapped. As pressure builds up, these air pockets are compressed to smaller volumes. In the process, some of the hydrostatic pressure of the system is absorbed by compressing these air pockets, reducing the amount of energy available to move water. If too much energy is absorbed by compressing air, then no flow will reach the desired discharge point until something is done about the air-blocks. For profiles/points which may require air release valves in a system see Figure 3.8.a.

Generally, there will no problem of air-blocks in a system where a tank is located at an elevation lower than the air-blocks, as long as the air-blocks are at least 10 meters below the static level. This is shown in Figure 3.8.b.

3.7.1.1 Air Release Valves

Air release valves should be located at every peak/high points on the pipeline. They release large bubbles of air that collects in the pipe which brings about resistance to flow condition in the pipes. They can also admit air to protect the pipeline if a break occurs. Their locations can be determined from the elevations of the pipeline. The best types are automatic ones as these require the least maintenance, but simple manual valves can also be used.

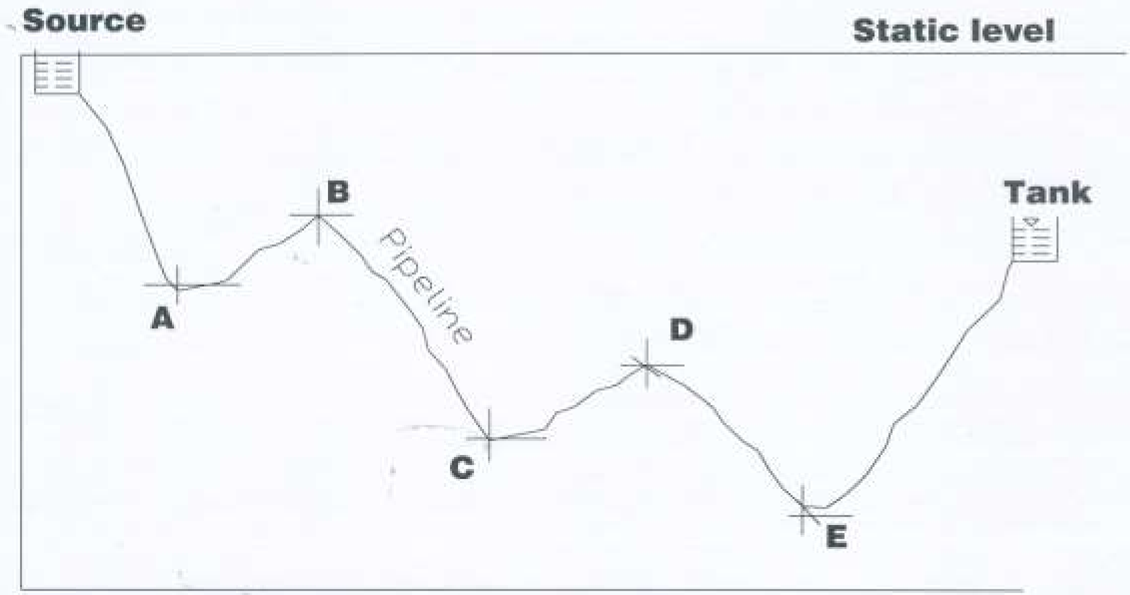
3.7.1.2 Types of Air Release Valves

There are air release valves which operate automatically.

At times when the above automatic air- valves are not available, there is alternative method for allowing trapped air to be released from the pipeline. Puncture the pipe with nail and seal it off with a brass or aluminum screw.

Although this method is not as expensive as an air valve, it is not automatic, and require manual operation by the villagers. At times when the pipeline is refilled with water, the valve is opened or the nail is removed, allowing trapped air to escape.

Details of automatic air release valves and manual/alternative air release valves are shown in Figures 3.9.a and 3.9.b.



Points A,C & E require washouts.
Points B & D require air-release valves.

Fig 3.8.a Profiles where Air-release & Washout valves may require

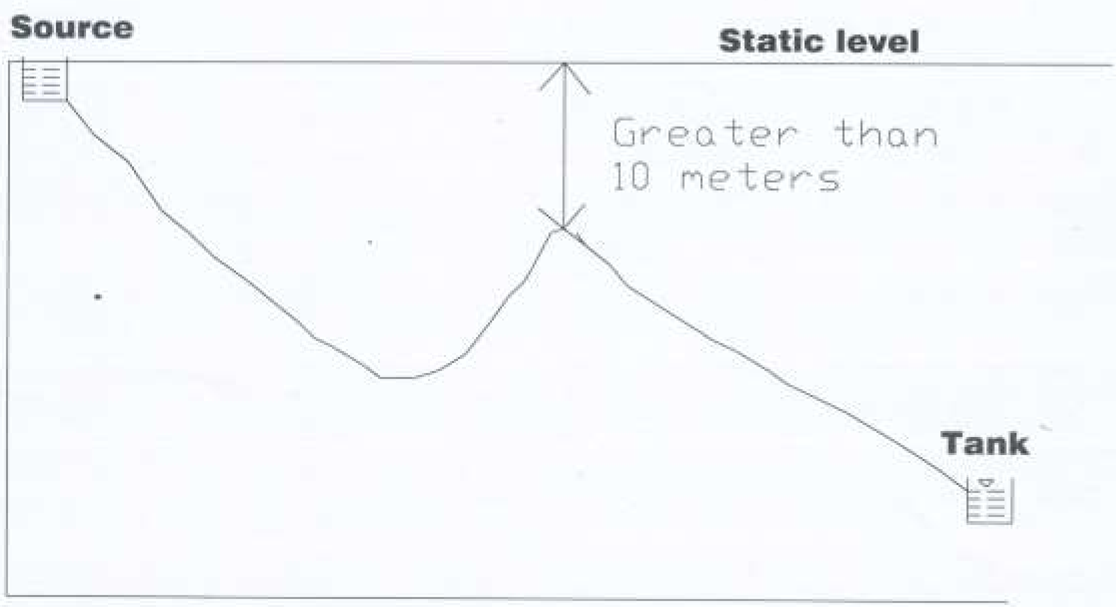


Fig 3.8.b Profiles where Air-blockage will not affect flow

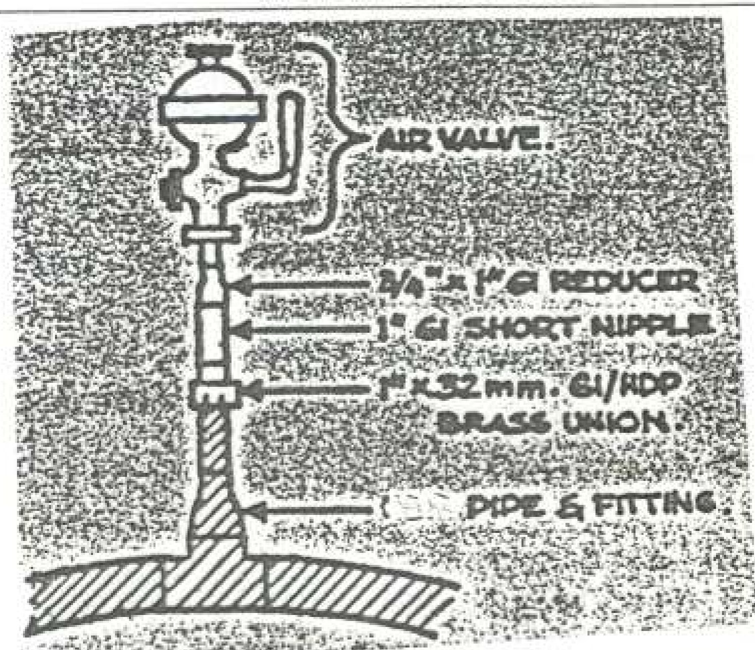


Fig. 3.9.a Automatic air-release valave

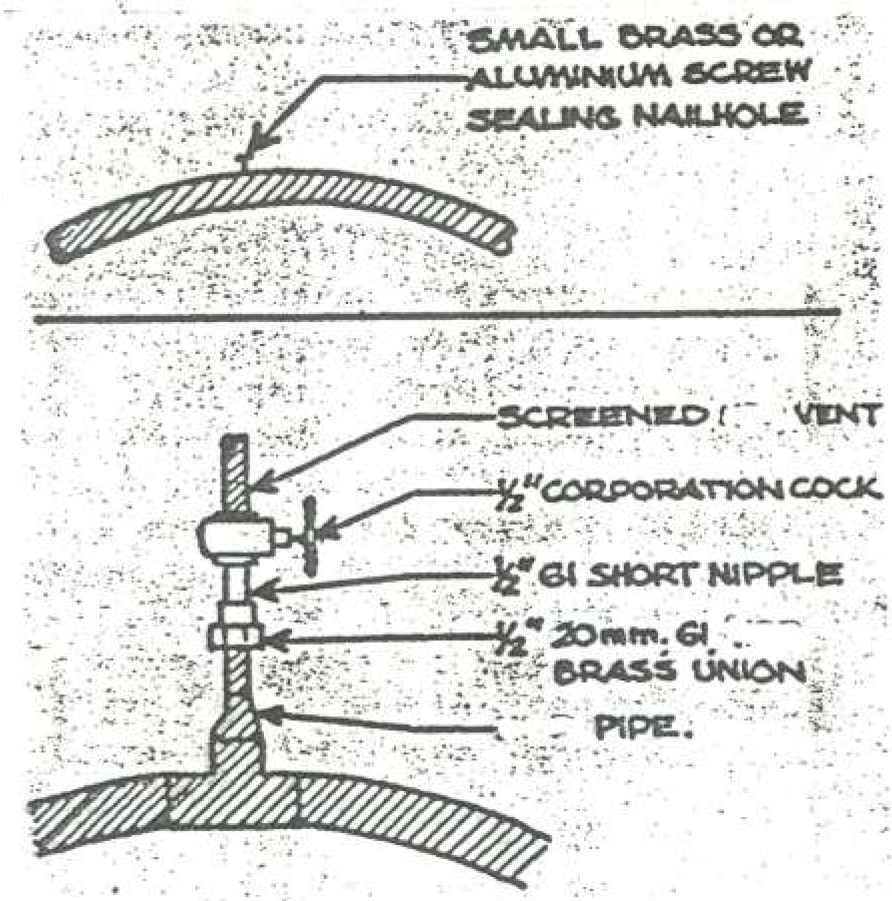


Fig. 3.9.b Manual air-release valave

3.7.2 Washouts / Flushing Devices

Over a period of time, suspended particles carried in the flow will tend to settle out, practically at low points in the pipeline or where the flows are low enough so that the flow velocity drops below 0.5 meters/second. Reservoirs usually allow most of the particles to settle but pipelines sections upstream from the reservoir do not benefit from this. Break - pressure tanks do not allow any sedimentation to occur, since flows through these are extremely turbulent.

Washouts are located at low points (at the bottom points of major U-profiles) in the pipeline especially those upstream from the reservoir tank and at the ends of the pipe sections with low flows. These consist of a tee joint that has a cap/plug or valve that can be opened to flush settled solids out of the pipe.

The number of washouts in a system depends upon the type of source (a stream yields more suspended materials than a spring), whether or not there is a sedimentation tank and/or reservoir, and the velocity of flow through the pipeline. For profiles/points which may require air release valves in a system see Figure 3.9.

The washout pipes should be of the same size as the pipeline at that point.

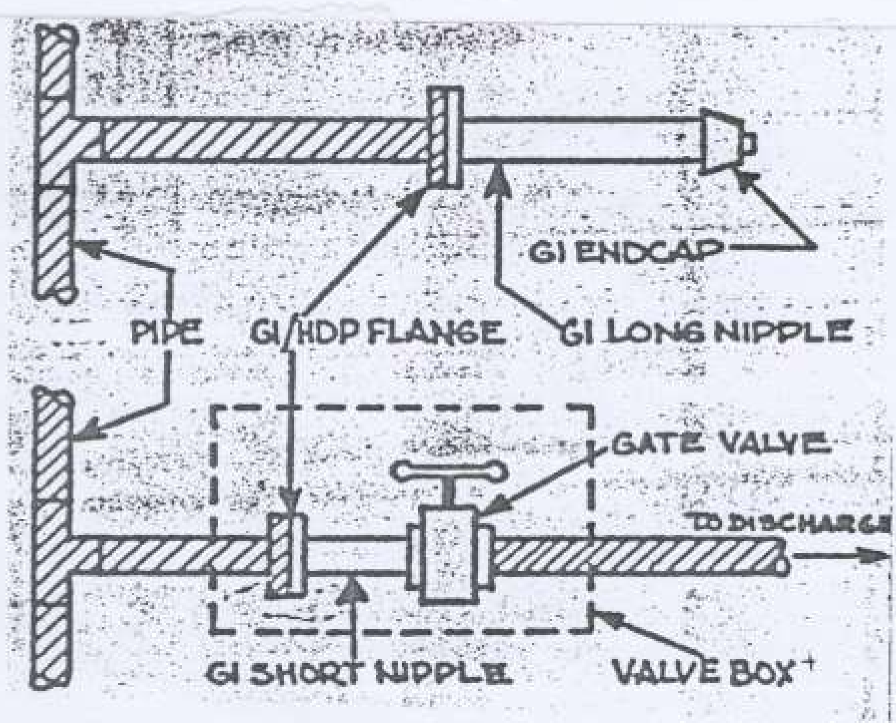


Figure 3.10 Some washout designs.

3.8 Valve Boxes

The purpose of the valve box is to protect a control valve from undesirable tempering which can upset the hydraulic balance of the system and disrupt flows.

Valve boxes can be attached to the structure (as is common in tanks) or located independently along the pipeline (such as at strategic branch points or near public fountains).

Valve boxes can be constructed of masonry, brick, HCB or concrete depending on the material available, size and number of valve, how often they will be operated etc. The boxes must have strong cover which can not be opened simply.

A valve box also must be adequately large enough to allow the valves to be removed easily and replaced, without having to demolish the valve box.

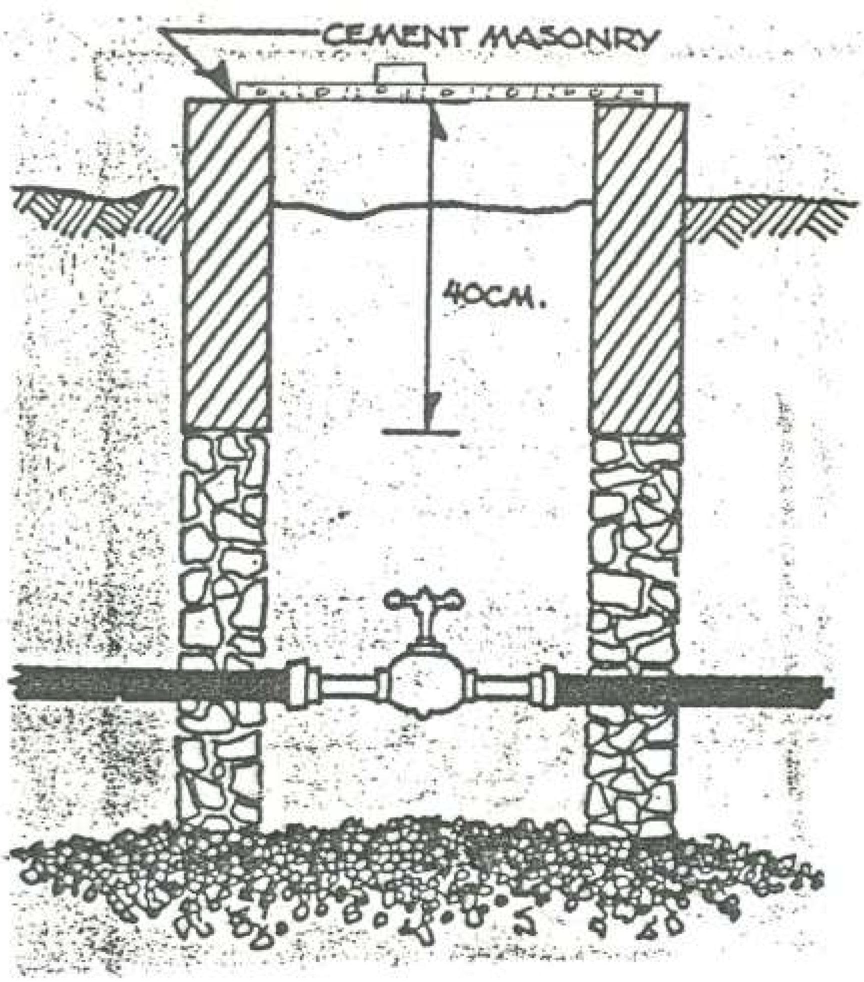


Fig. 3.11 Masonry made valve box design

3.9 Flow measuring device

In order to ensure effective water supply management in a system; flow measurement device (water meter) must be provided at certain points such as in pumping stations if any, source (intake), reservoirs outlets, house connections, public fountains etc.

3.10 Cattle trough

Cattle trough will be located within 50 meters far from the public (water) fountains when needed. The trough can be made from wood, stone or concrete.

3.11 Special component sections

For special components of the system such as suspended crossings and gully crossings should be identified where they are needed and the design drawings and cost estimates are needed to be prepared.

4. STEPS IN SURVEY AND DESIGN OF A GRAVITY- FLOW WATER SUPPLY SYSTEM

4.1 The Field Survey

1. Become familiar with the water source and the village; consult with the community with regard to the acceptability of the water source and purposed system and their commitment for participation.

2. From community survey result, decide on the feasibility of constructing the water system.

3. For systems considered feasible, a detailed survey is made including: source flow measurement, pipeline distances, and ground level profile.

Some people from the village should assist in this survey. At this time, tentative locations for distribution points are decided upon in consultation with the villagers, taking in to account their own wishes, population distribution, location of institutions etc.

4.2 The Design Process

The survey data is used to design the system generally as follows:

1. Decide on general system design layout such as type of distribution facilities, location of public fountains, reservoirs, cattle troughs if any, decide on the type of pipe to be used (either PVC or GI) at various sections etc.

2. Decide the important design criteria such as project design period, population growth rate, per capita water demand, maximum daily factor, peak hour factor etc.

3. Calculate the following values for the design period using the data collected and the design criteria decided in step 2 above:

- Total population to be served (Formula 3.4).
- Total domestic water demand.(Formula 3.5)
- Total public demand.(Section 3.2.3.b)
- Commercial demand .(Section 3.2.3.c)
- Total animal demand (Section 3.2.3.d) if any.
- Total loss and waste (Section 3.2.3.e)
- Average daily demand (Section 3.2.3.f)
- Maximum daily demand (Formula 3.6)
- Maximum daily flow in liters/second (Formula 3.7)
- The peak hour demand (Formula 3.8)
- Peak hour flow in liters/second (Formula 3.9)

4. Compare the maximum daily flow calculated in step 3 above with the minimum source flow determined from Section 3.1.2. And decide whether the source is a adequate for the village or not.

NB. **For a water source to be considered as adequate, the 24 hours minimum (safe) yield of the source should cover the maximum daily demand of the people.**

If it is decided for the source to be constructed, proceed to the following steps of design..

5. If you have well collected data on the supply and demand for a village of related nature, calculate the required capacity of the reservoir. Usually the capacity of reservoir is calculated using Table 3.3 or formula 3.11.

6. Design of the gravity main pipeline (pipe from source to reservoir).

- Calculate the ecumenical pipe diameter using formula 3.16.
  - Here the flow in the pipe to be used is the maximum daily flow.
  - In some cases if the minimum yield of the spring is greater than the maximum daily flow, if required the minimum (safe) yield of the source can be used in the design.
- Select the nearest standard economical pipe diameter from the list available in Table 3.4.
- Check the velocity of flow in the selected standard pipe using formula 3.15 and check this with the minimum and maximum allowable velocity limits shown in section 3.4.1.11. If the velocity is within the limits:
  - Calculate head loss in the pipe using formula 3.12.
  - Calculate the free head at the reservoir inlet using formula 3.14.
  - Finally decide on the diameter of the gravity main pipe.

7. Design of the distribution pipes (pipes from the reservoir to distribution points).

- Decide the flow to be distributed to each distribution point such as public fountains, institutional fountains and cattle troughs if needed. Here the flow to be convoyed by the pipes from the reservoir to any distribution point is the peak hourly demand (of course it is distributed to the distribution points based on their need for consumption).
- Propose the pipe diameters of the different pipeline routes based on the flow (here formula 3.16 can be used) or pipe diameters can be proposed from experience.
- If formula 3.16 is used, select the nearest standard economical pipe diameter from the list available in Table 3.4.
- Check the velocity of flow in each pipeline using formula 3.15 and check this with the allowable velocity limits shown in section 3.4.1.11.
- Calculate head loss in each pipeline using formula 3.12.
- Calculate the free heads at each distribution point (fountains, cattle troughs etc.) using formula 3.14 or Table 3.6.

- If the above calculated free heads satisfy the minimum and maximum recommended free heads shown in section 3.6.3 and the velocity in each pipe satisfy the allowable velocity limits put in section 3.4.1.11. Then the pipe diameters are accepted.
- But if the free heads at each distribution point and velocities in each pipe calculated in Table 3.6 do not satisfy requirements put in the respective sections, the following are proposed.
  - If the free head at any distribution point or faucet is less than 5 meters, then the following can be done:
    - the diameter of some pipes can be increased so that the head loss can be reduced keeping the velocity of flow within the range of 0.5 - 2.5m/s or,
    - the location of the public fountains can be changed to a lower elevation location or,
    - the reservoir can be at more higher elevation.
    - the head loss and free heads re-calculated.

  - If the free head at any distribution point or faucet is more than 60 meters, then the following can be done:
    - the diameters of some pipes can be reduced so that the head loss can be increased keeping the velocity of flow within the range of 0.5 - 2.5m/s or,
    - the location of the public fountain can be changed to a higher elevation location or,
    - if the free head is very excess and can not be minimized with diameter reduction, a break pressure tank can be proposed at some point
    - the head loss and free heads re- calculated.

N.B. The format to be used for pipeline design is shown in Table 3.6.

8. The most appropriate pipe is chosen for all sections of the pipeline based on the calculation shown in steps 6 and 7 above.

9. If required the true hydraulic gradients are then plotted on the profile at least by using (pointing) points of desired heads and known elevations.

10. From the elevations profile and general scheme of the system, the need and placement of break pressure tanks, air release valves, washouts, etc. are determined and recorded on the general sketch.

11. Detailed drawings, specifications, materials list and budget can now be prepared.

Table 3.5 : Table showing frictional head loss factors (f) for steel pipes, C = 110

For the steel pipes of C = 110, the Hazzen William's formula can be simplified as:

$$H_f = \frac{2.05 \times 10^6 \times Q^{1.85}}{d^{4.87}} \times L = f \times L$$

Where, f = Frictional head loss factor,
L = Length of pipe in meters (m)

For C = 110, and for different discharges and pipe diameters the following table is made so that it can be used when necessary.

Flow (lit/sec)	Pipe diameters in millimeters (mm)								
	12.5	19	25	32	40	50	63	75	90
0.1	0.13	0.017	0.0045						
0.15	0.28	0.036	0.012	0.003					
0.25		0.093	0.025	0.0074	0.0025				
0.3		0.131	0.034	0.01	0.0035				
0.35		0.174	0.046	0.014	0.0046				
0.4		0.223	0.059	0.018	0.006				
0.45		0.277	0.073	0.022	0.0074	0.0025			
0.5			0.088	0.027	0.009	0.003			
0.55			0.105	0.032	0.011	0.0036			
0.6			0.124	0.037	0.013	0.0042			
0.65			0.144	0.043	0.0146	0.005	0.0016		
0.7			0.165	0.05	0.017	0.006	0.002		
0.75			0.187	0.056	0.019	0.0064	0.0021	0.001	
0.8			0.211	0.063	0.021	0.0072	0.0023	0.001	
0.85			0.236	0.071	0.024	0.0081	0.0026	0.0011	
0.9			0.263	0.079	0.027	0.009	0.003	0.0012	
0.95			0.29	0.087	0.029	0.01	0.0032	0.0014	0.0006
1				0.096	0.032	0.011	0.0035	0.0015	0.0006
1.2					0.045	0.015	0.005	0.002	0.0009
1.4					0.06	0.02	0.007	0.003	0.0012
1.6					0.077	0.026	0.0084	0.004	0.0015
1.8					0.096	0.032	0.010	0.005	0.0018
2						0.039	0.013	0.0054	0.0022
2.2						0.047	0.015	0.0065	0.0027
2.5							0.019	0.0082	0.0034
3							0.027	0.011	0.0048
3.5							0.036	0.0154	0.0063
4								0.02	0.0081
4.5								0.0244	0.01

Example on the usage of the table,

- For a pipe of 40mm diameter, discharge of 0.6 lit/sec and 250 meters long pipe, the head loss can be calculated as, $\mathbf{H_f = f \times L = 0.013 \times 250m = 3.25m}$

Table : 3.6 Table for calculating frictional head loss in pipes

Reach		1	2	3	4	5	6	7	8	9	10	11
Station I	Station II	Discharge in (lit/sec)	Pipe diameter (mm)	Velocity of flow (m/s)	Pipe length between successive stations (m)	Head loss factor (f) in m/length of pipe	Head loss in the pipe including 5% minor losses (m)	HGL at station I (m)	Ground Elevation at station II (m)	HGL at station II, 9 = 7 - 6	Free head at station II, 10 = 9 - 8	Remark

NB. The elevations of the HGL at intakes (sources), reservoirs outlets and Break Pressure Tank (BPT) outlets is the same as the ground elevation of these points because these are the points where atmospheric pressure is Zero.

5. WORKED EXAMPLES ON DESIGN OF GRAVITY - FLOW WATER SUPPLY SYSTEM

Example 1

Design data

The village has a total population of 1600 at present divided into two main gots, got 1 and got 2 with 825 and 775 persons respectively. The people at each got also settle into two smaller menders having the population shown in the sketch map of the village. There is also a school having 800 students located at 2000meters distance from the source. A water source with an estimated minimum flow of 0.75 liter/second is located on the upstream of the village. The public fountains are to be used 8 hours per day.

Assume that GI pipes must be used for all pipes because of the rocky terrain. A sketch map of the village and the ground elevations of each important point is presented in Figure 3.12.

As there are number of flowing streams located at close distance from the gots, the source is not recommended for animals use.

Design the water supply system required for the village for the coming 15 years and plot the hydraulic grade line for each pipe length.

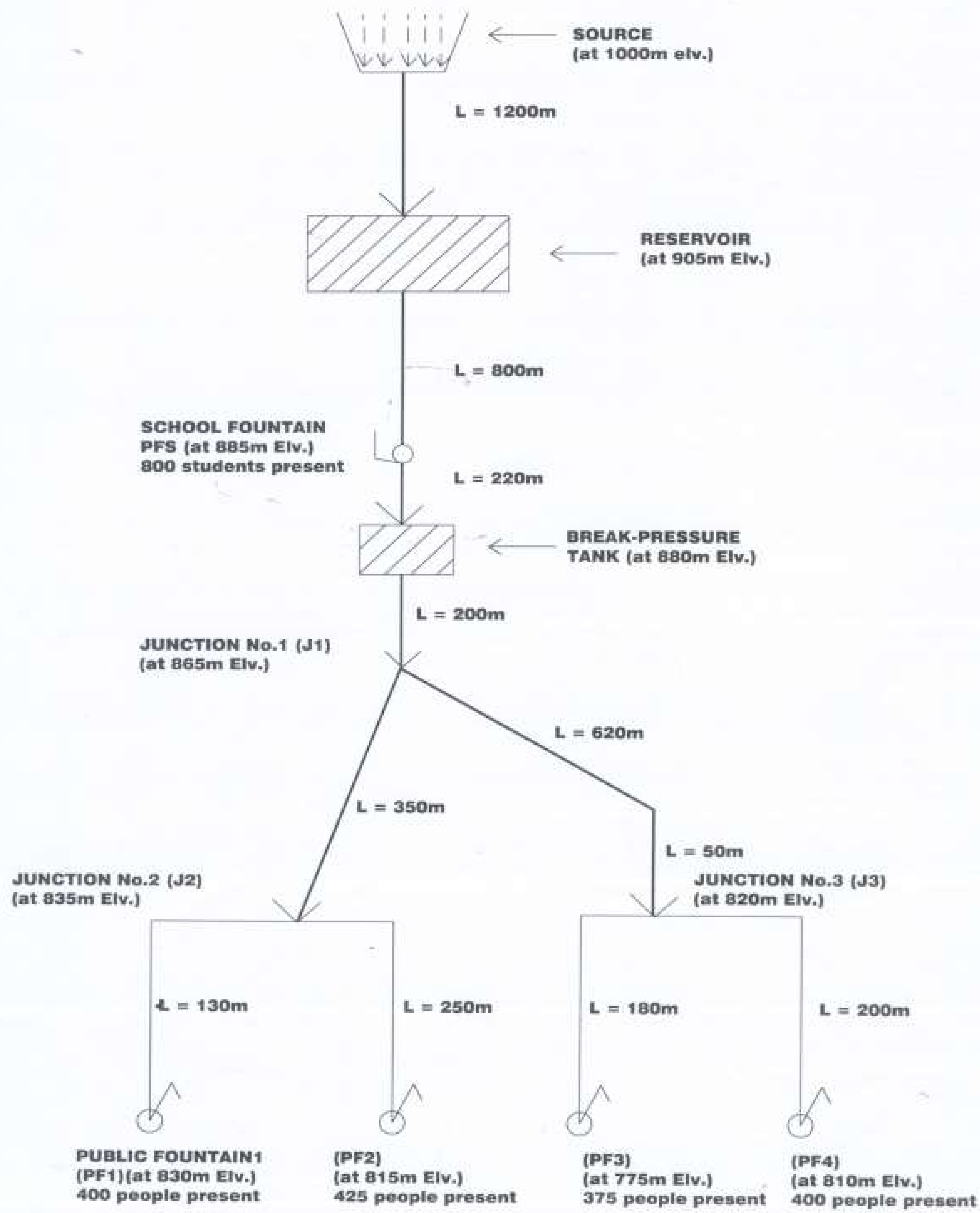


Figure 3.12 Map of village & elevations of important points for example 1

Design process of the example 1

The design process is shown in step by step as follows.

Step 1
The pipes are given to be GI, locations of scheme components made in Figure 3.12.

Step 2
The following design criteria and standards selected.
n = 15 years, i = 2.83%, M_f = 1.2, P_f = 2, Domestic demand = 15l/c/d

Step 3
a) $P_d = P_o\,(i + i/100)^n = 1600\,(1 + 2.83/100)^{15}$ = 2432 people
b) Domestic demand = 15 l/c/d x 2432 people = 36480 l/d
c) Public demand = 15/100 (36480 l/d) = 5472 l/d
d) Loss and waste = 10/100 (36480 + 5472) = 4195.2 l/d
e) Average daily demand = 36480 + 5472 + 4195.2 = 46147.2 l/d
f) Maximum daily demand (M_d) = M_f x Q_{ave} = 1.2 x 46147.2 l/d = 55377 l/d
g) Maximum daily flow (Q_{max}) = M_d/t = $\dfrac{55377 \text{ l/d}}{86400 \text{ s}}$ = 0.65 l/s
h) Peak hour demand (Q_{peak}) = P_f x M_d / 24 = 2 x 55377/24 = 4614.75 l/hour
i) Peak hour flow = 4615.74 l/3600sec = 1.3 l/s

Step 4
The safe (minimum) yield of the spring, Q_{safe} = 0.75 l/s
The maximum required daily flow , Q_{max} = 0. 65 l/s

As the safe yield of the spring is more than the maximum required daily flow, the source is adequate to the village and needs to be constructed.

As the minimum yield of the source is very close to the maximum daily flow required, in order to provide more water to the community we can use the minimum (safe) yield of the spring i.e. 0.75 l/s in the design of the whole system. But in order to show how the detailed estimate of demand is to be used in the design, the maximum required daily flow i.e. 0.65 l/s is used for the system components design.

Step 5
Capacity of reservoir = Maximum daily demand/3
Or S_r = 0.75 l/s / 0.65 l/s = 1.15 from Table 3.3 implies, recommended reservoir capacity is one third of maximum daily demand.
Reservoir capacity = $\dfrac{55377 \text{ l}}{3}$ = 18459 lit = **20 mcu**

Step 6
Design of gravity main pipeline (Pipe from source to reservoir)

Economical diameter, d = $1.22 \times \sqrt{Q} = 1.22 \times \sqrt{0.65}$ = 0.98 inch = 1 inch = 25mm

Check , Velocity, $V = 4 \times Q/3.14 \times d^2 = \dfrac{4 \times 0.00065\ m^3/s}{3.14 \times (0.025m)^2} = 1.3\ m/s$

As the flow velocity is within the permissible limit put in sections, the 1 inch or 25mm pipe diameter is enough.

Head loss in the gravity main, $H_f = f \times L$
For , C = 110, GI pipe, from table 3.5, for Q = 0.65 l/s and d = 1inch gives us f = 0.144
$H_f = f \times L = 0.144 \times 1200m = 172.8m$, including 5% minor losses , $H_f = 180.4m$

Free head at reservoir inlet = Elevation of source - Elevation of reservoir - Head loss from source to reservoir

Free head at reservoir inlet =1000m - 905m - 180.4 m = -65.2m (the water can not flow by gravity to the reservoir as the head loss is too much). Hence reduce the head loss.

To reduce the head loss we can increase the pipe diameter, try the next higher diameter i.e 1.25" = 32mm

$V = 4 \times 0.00065/3.14 \times (0.032m)^2 = 0.81$ m/s (OK)

Head loss due to friction, $H_f = f \times L = 0.043 \times 1200m = 51.6m$
Total head loss including 5% for minor losses = 1.05 x 51.6m = 54.2m
Free head at reservoir inlet = 1000m - 905m - 54.2m = 40.8m (OK)

Therefore , d = 1.25" or 32mm can be used for the gravity main pipe.

Step 7
Design of distribution pipes (pipes from the reservoir to distribution point).
These pipes are to be designed for the peak hour flow i.e 1.3l/s.

As put in the sketch of the town there are 5 important distribution points which need fountains namely the school, Mender 1, Mender 2, Mender 3 and Mender 4.

The peak hour demand is calculated taking into consideration of these 5 distribution points. Hence the peak hour flow should also be distributed to each distribution point so that the pipelines to each point be designed to convoy the required peak flow to each point.

- Peak flow for the school = 11/100 (1.3 l/s) = 0.143 l/s = 0.14 l/s
- Peak flow for Mender 1 (Q_{peak1}) $= \dfrac{(1.3 - 0.14) \times \text{Design population of mender 1}}{\text{total design population}}$
- Q_{peak1} = 1.16 l/s x 608/2432 = 0.29 l/s
- Q_{peak2} = 1.16 l/s x 646/2432 = 0.31 l/s
- Q_{peak3} = 1.16 l/s x 570/2432 = 0.27 l/s
- Q_{peak4} = 1.16 - 0.29 - 0.31 - 0.27 = 0.29 l/s
- Check, Q_{peak} = 0.14 + 0.29 + 0.31 + 0.27 + 0.29 = 1.3 l/s

For this demonstration purpose as we want to be more exact we use the actually calculated peak values for each distribution point design. But in some cases for simplicity of calculation we can take the peak flows to the nearest end 0 or end 5 numbers so that we can use Table 3.5 of this manual for calculating head losses for GI pipes of C =110.

The calculation for designing distribution pipes is shown in Table 3.7. As can be seen from the table the pipe diameters selected form each pipeline routs are found to be appropriate and economical as all satisfy the permissible velocity limits and the free heads at the distribution points are within the maximum and minimum range of allowable free heads.

To check the correctness of the calculation shown in the table, the following can be seen :

- Free head at J2 = Elevation of BPT - Elevation of J2 - H_f(BPT to J1) - H_f(J1 to J2) - = 880m - 835m - 2.9m -13.6m = 28.5m which is the same as calculated in the table.
- Free head at PF_3 = Elevation of BPT - Elevation of PF3 - H_f(BPT to J1) - H_f(J1 to J3) - H_f(J3 to PF_3) = 880m - 775m - 2.9m -(21.5+5.7)m - 20.4m = 54.5m which is the same as calculated in the table.

In the pipeline route from J1 to J3 combination of two pipe diameters are selected this is done just to show that it is also possible to use such a combination of pipes to be used to increase or decrease head losses when required.

In order to show that the presence of the Break Pressure Tank (PBT) in the example is appropriate see the following:

- If the BPT had not been present, water to the public fountains would have been distributed directly from the reservoir which is at an elevation of 905m. The elevation of the BPT is 880m which implies the reservoir elevation is 25 meters higher than the BPT. So if the BPT had not been present, the free head in all public fountains could have increased by 25m. And if we take the PF_3 for example, the free head would have been = 54.5m + 25m = 79.5 m which is by far more than the maximum allowable free head of 60m. So we can say that the placement of the BPT was very appropriate.

Step 8
The pipes chosen for all sections are shown in Figure 3.13.

Step 9
Plotting of the HGL

The HGL is plotted in Figures 3.14.a and 3.14.b.

Step 10
Placement of BPTs, air release valves and washouts.

The BPT is already located in the initial layout of the system. From the profiles shown in Figures 3.14.a and 3.14.b it is not as such needed to place air release valves or washouts.

Table : 3.7 Format for calculating frictional head loss in pipes for Example 1

Reach		1	2	3	4	5	6	7	8	9	10	11
Station I	**Station II**	**Discharge in (lit/sec)**	**Pipe diameter (mm)**	**Velocity of flow (m/s)**	**Pipe length between successive stations (m)**	**Head loss factor (f) in m/length of pipe**	**Head loss in the pipe with5% minor losses (m)**	**HGL at station I (m)**	**Ground Elevation at station II (m)**	**HGL at station II, 9 = 7 - 6**	**Free head at station II, 10 = 9 - 8**	**Remark**
Source	Reservoir	0.65	32	0.81	1200	0.043	54.2	1000	905	945.8	40.8	Free discharge (Pipe flow not full)
Reservoir	PF_S	1.3	50	0.66	800	0.018	15.1	905	885	889.9	4.9	OK
PF_s	BPT	1.16	50	0.6	220	0.014	3.2	889.9	880	886.7	6.7	HGL reduced to 880m
BPT	J1	1.16	50	0.6	200	0.014	2.9	880	865	877.1	12.1	
J1	J2	0.6	32	0.75	350	0.037	13.6	877.1	835	863.5	28.5	
J2	PF_1	0.29	19	1.02	130	0.123	16.8	863.5	830	846.7	16.7	OK
J2	PF_2	0.31	19	1.1	250	0.14	36.8	863.5	815	826.7	11.7	OK
J1	*	0.56	32	0.7	620	0.033	21.5	877.1	*	*	*	
*	J3	0.56	25	1.14	.50	0.109	5.7	*	820	849.9	29.9	Combination of 32mm & 25mm pipes
J3	PF_3	0.27	19	0.95	180	0.108	20.4	849.9	775	829.5	54.5	OK
J3	PF_4	0.29	19	1.02	200	0.123	25.8	849.9	810	824.1	14.1	OK

NB. The elevations of the HGL at intakes (sources), reservoirs outlets and Break Pressure Tank (BPT) outlets is the same as the ground elevation of these points because these are the points where atmospheric pressure is Zero.

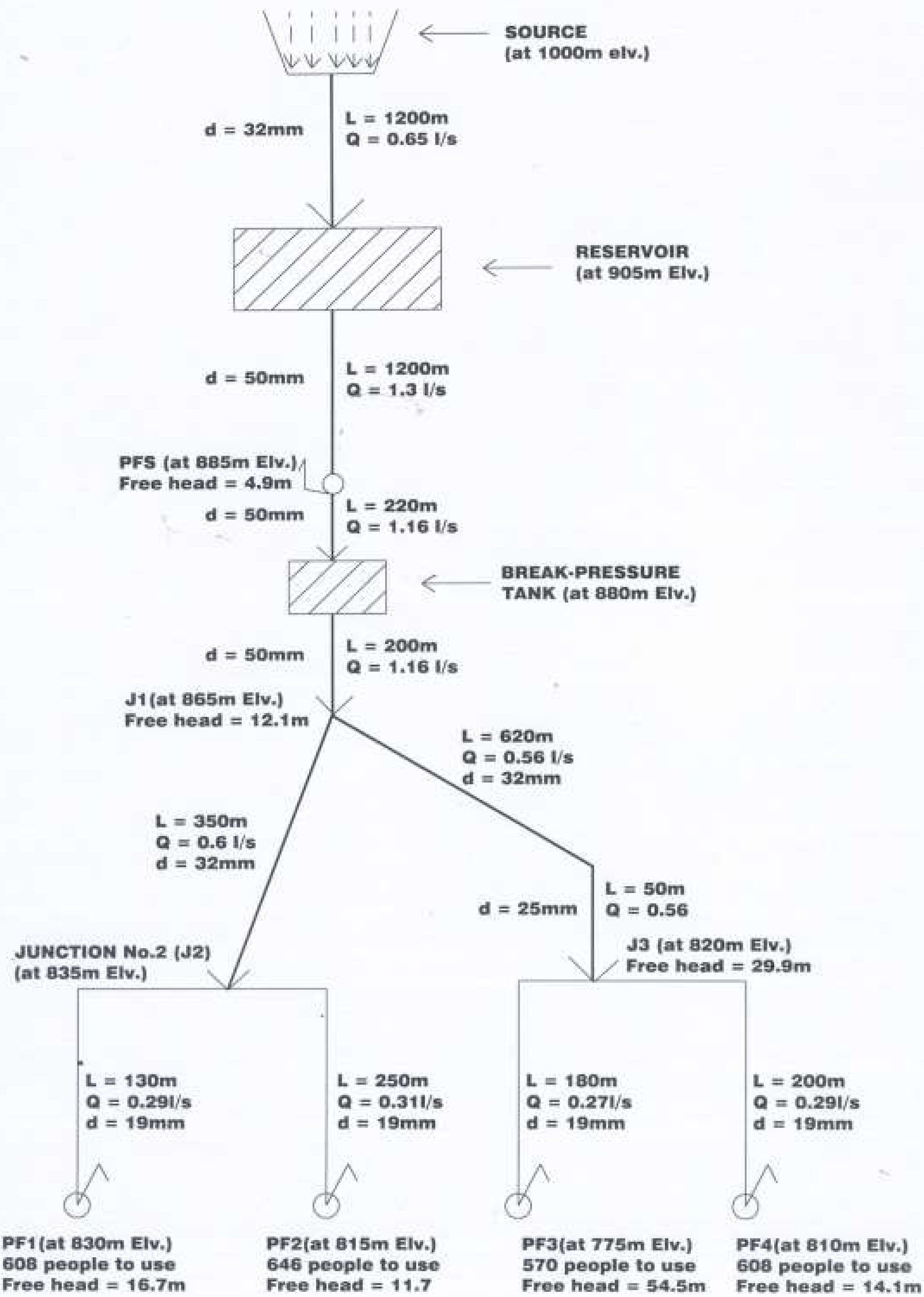


Figure 3.13 Designed pipes diameter, and free heads at fountains for example 1

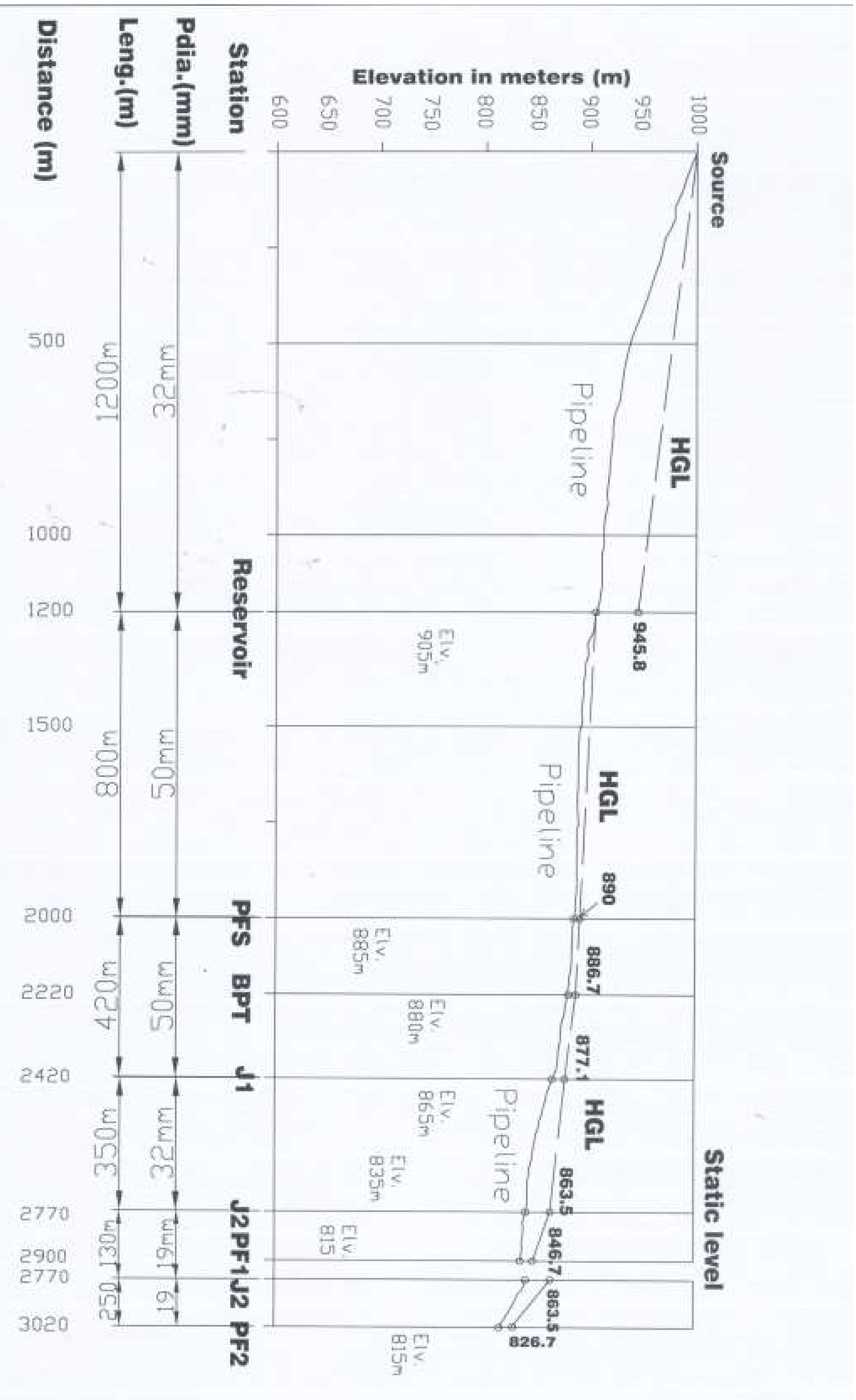


Fig. 3.14.a Hydraulic Grade Line of Example 1 from Source to PF1 & to PF2

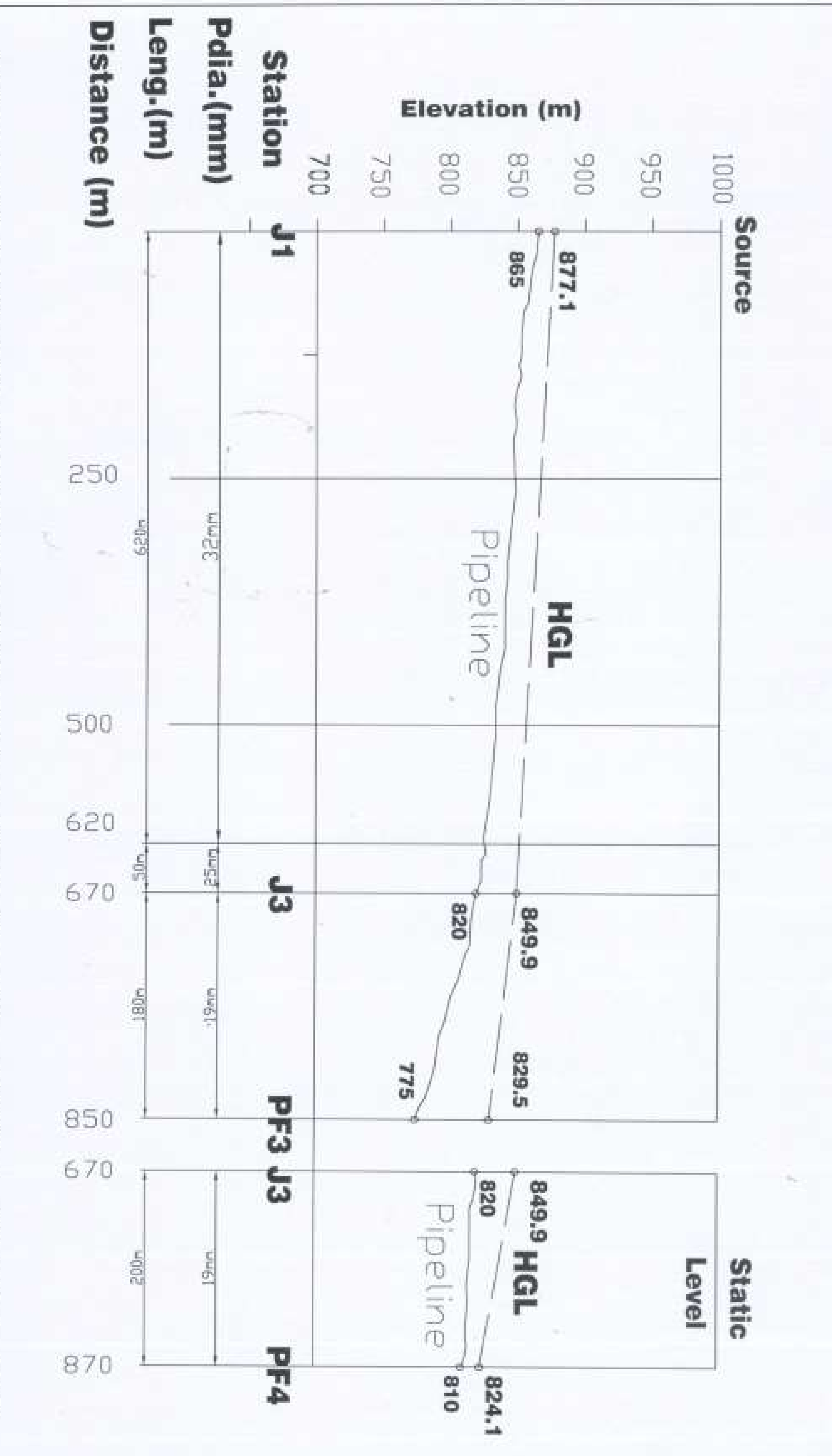


Fig. 3.14.b Hydraulic Grade Line of Example 1 from Junction 1 (J1) to PF3 & to PF4

Design Example 2

Design data

A village named Y has four smaller gots named A,B,C and D. Two of the gots named A and B, and gots C and D are located on different sides of the village. Each village serves certain number of people. The number of people located in villages A and B accounts 55% of the inhabitants and the remaining 45% of the inhabitants are located in villages C and D. A spring with an estimated minimum flow of 1 l/s which is just equal to the maximum daily flow required to the people is located on the upstream side of the village. In order to reduce the size of the gravity main pipe, its is decided that storage tanks of enough capacity to store 8 hours demand of each got population to be located in each got.

It is required that GI pipes to be used for all pipes due to nature of the terrain and in order to reduce the size of the main, storage tanks of enough capacity are located in each gott

(A sketch map of the village and the portion of the maximum daily flow to be distributed to each got is shown in Figure 3.15.

For plotting ground profile the following points with their elevation are provided

Point	Distance from close reference point (m)	Ground elevation (m)
Source (Spring)		1575
P1	350 m from spring	1560
P2	750m from spring or 400m from P1	1506
P3	1200m from spring or 450m from P2	1520
J1	1700m from spring or 500m from P3	1500
J2 (Res. A)	500m from J1	1490
Res. B	300m from J2	1480
J3 (Res. C)	600m from J1	1494
Res. D	300m from J3	1490

Design the pipeline (size of pipes) required to serve, recommend points where the air release valves and washouts are required if any and plot the hydraulic grade line for each pipe length.

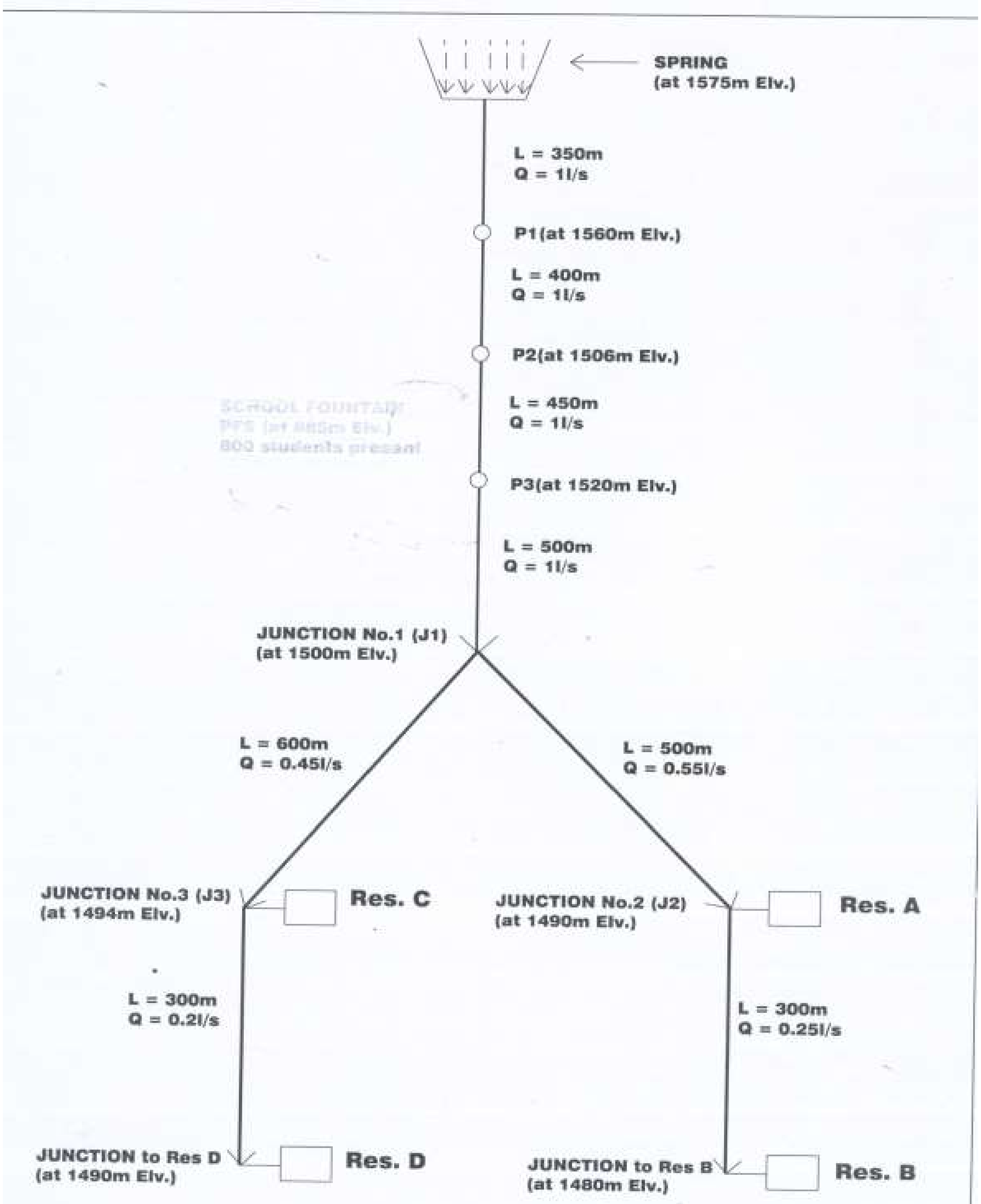


Figure 3.15 Map of village & elevations of important points for example 2

Design process of example 2

As it is clearly given in the question about the demand of each got, there is no need to calculate the demands hence directly go to the pipeline design.

The pipes are given to be GI, locations of scheme components is made in Figure 3.15.

1. Pipeline design

Based on the village sketch it has been decided that the water will be distributed to four public reservoirs. With this layout the water will be under each gott control. Costs will be reduced because the main pipe does not have to convoy peak hour flows of water (will convoy only maximum daily flows). The public reservoirs are to be situated on high points to obtain any acceptable pressure head at the tank outlet.

Design flows

The path of the pipeline is sketched in Figure 3.15. The water will flow continuously into the reservoirs so the design flows will be the same as the maximum daily flows. The peaking factor is therefore 1 and the flow of 1 l/ s will be used in designing the pipeline.

The design of the pipelines (determination of pipe diameters) for each pipeline routes is shown in Table 3.8 (trial1) and Table 3.9 (trial 2). So the selected pipe diameter for each route are shown in Table 3.9.

2. Plotting of Hydraulic Grade Line (HGL)

The ground elevations and the hydraulic grade line of the system are shown in Figures 3.17.a and 3.17.b.

3. Determination of location of air release valves, washout valves and break pressure tank (BPT)

- For the Figures 3.17.a and 3.17.b it can be seen that point designated by **X** is observed to be point of peak elevations hence in order to avoid air blockage air release valve is recommended in these points.
- Point designated by **Y** is observed to be point of low elevations hence washout valve is recommended.
- As can be seen from the Table 3.9, the maximum free head available is at each junction and distribution point is below 60 meters hence there is no a need for placing BPT is the system.

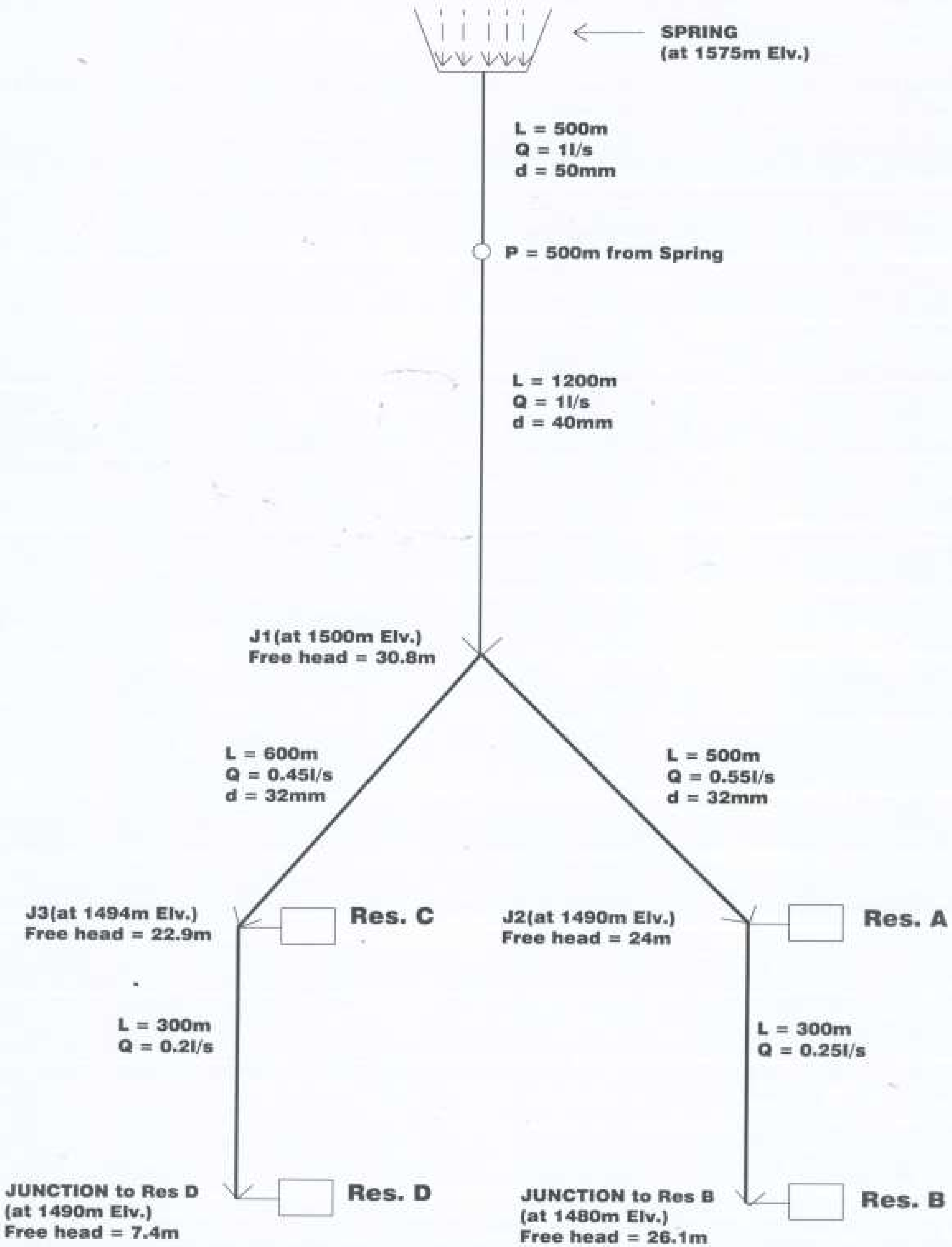


Figure 3.16 Designed pipes diameter, and free heads at fountains for example 2

Table : 3.8 Frictional head loss calculation in pipes for Example 2 (Trial 1)

Reach		1	2	3	4	5	6	7	8	9	10	11
Station I	**Station II**	**Discharge in (lit/sec)**	**Pipe diameter (mm)**	**Velocity of flow (m/s)**	**Pipe length between successive stations (m)**	**Head loss factor (f) in m/length of pipe**	**Head loss in the pipe with5% minor losses (m)**	**HGL at station I (m)**	**Ground Elevation at station II (m)**	**HGL at station II, 9 = 7 - 6**	**Free head at station II, 10 = 9 - 8**	**Remark**
Spring	J1	1	40	0.8	1700	0.032	57	1575	1500	1518	18	OK
J1	J2 (Res A)	0.55	32	0.68	500	0.032	16.8	1518	1490	1501.2	11.2	OK
J2 (Res A)	Res. B	0.25	25	0.51	300	0.025	7.9	1501.2	1480	1493.3	13.3	OK
J1	J3 (Res C)	0.45	32	0.56	600	0.022	13.9	1518	1494	1504.1	10.1	OK
J3 (Res C)	Res D	0.2	19mm	0.71	300	0.062	19.5	1504.1	1490	1484.6	-5.4	Not OK

NB. The elevations of the HGL at intakes (sources), reservoirs outlets and Break Pressure Tank (BPT) outlets is the same as the ground elevation of these points because these are the points where atmospheric pressure is Zero.

From the above table it can be seen that the free head at reservoir D (Res. D) is negative which is not allowed in the principles of gravity flow. Hence it needs adjustment to be made at least to make the free head at reservoir D inlet or just at junction to reservoir D, 5 meters, which is the minim free head allowed.

For this different approaches can be used and one solution is to reduce the head loss by increasing the diameter of any pipe in the system which contributes to head loss to Res. D. From the velocity data shown in Column 3 of the above table, we can say that there is a possibility only in pipe from Source to J1 that we can increase the diameter to reduce the head loss keeping the velocity at least 0.5 m/s. To be ecumenical we do not need to increase the diameter of the whole 1700 meters pipe and only increasing some portion of it may be enough so try to change the diameter of the first 500m pipes to50mm. The results of the second trial is shown in Table 3.10.

Table : 3.10 Frictional head loss calculation in pipes for Example 2 (Trial 2 and Final)

Reach		1	2	3	4	5	6	7	8	9	10	11
Station I	**Station II**	**Discharge in (lit/sec)**	**Pipe diameter (mm)**	**Velocity of flow (m/s)**	**Pipe length between successive stations (m)**	**Head loss factor (f) in m/length of pipe**	**Head loss in the pipe with5% minor losses (m)**	**HGL at station I (m)**	**Ground Elevation at station II (m)**	**HGL at station II, 9 = 7 - 6**	**Free head at station II, 10 = 9 - 8**	**Remark**
Spring	*	1	50	0.51	500	0.011	5.8	1575	*	*	*	
*	J1	1	40	0.8	1200	0.032	38.4	*	1500	1530.8	30.8	OK
J1	J2 (Res A)	0.55	32	0.68	500	0.032	16.8	1530.8	1490	1514	24	OK
J2 (Res A)	Res. B	0.25	25	0.51	300	0.025	7.9	1514	1480	1506.1	26.1	OK
J1	J3 (Res C)	0.45	32	0.56	600	0.022	13.9	1530.8	1494	1516.9	22.9	OK
J3 (Res C)	Res D	0.2	19	0.71	300	0.062	19.5	1516.9	1490	1497.4	7.4	OK

NB. The elevations of the HGL at intakes (sources), reservoirs outlets and Break Pressure Tank (BPT) outlets is the same as the ground elevation of these points because these are the points where atmospheric pressure is Zero.

Conclusion: From the above table we can see that the velocities in the different pipes and the free heads at different junctions are found to be within the allowable limits. Hence the diameters shown in the above table for the different lengths of pipes is recommended for the system.

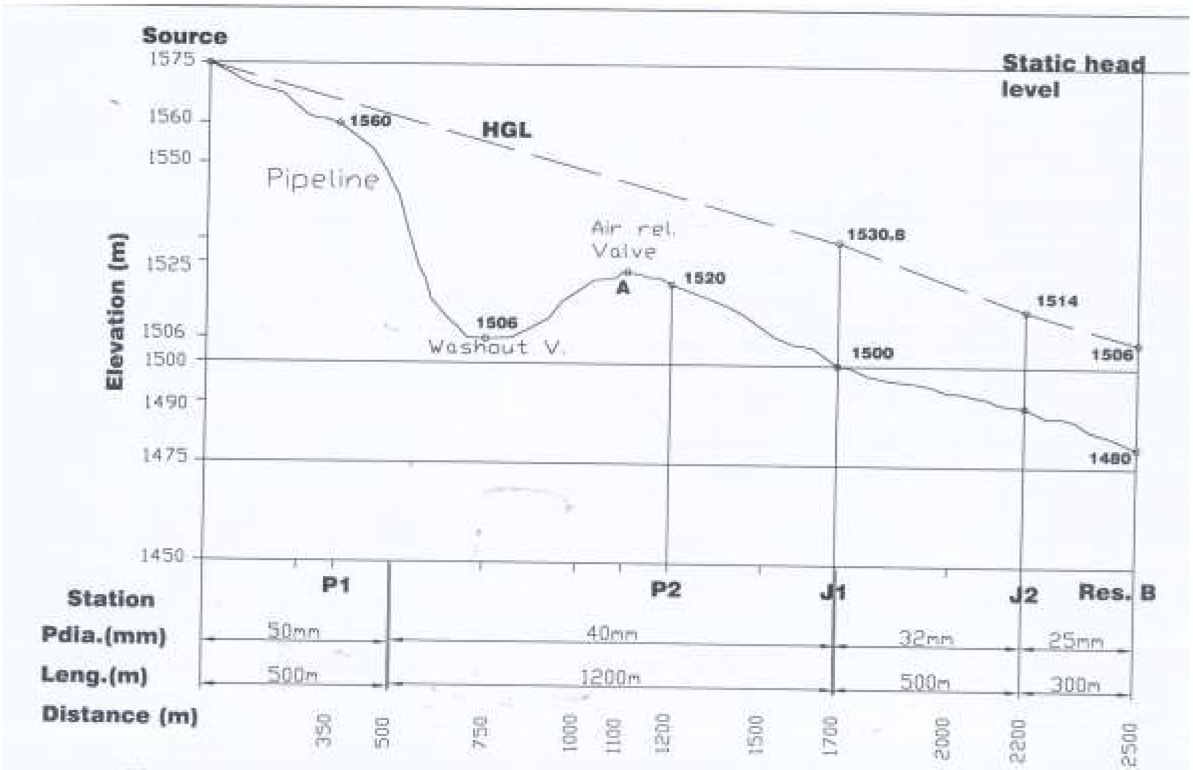


Fig. 3.17.a Hydraulic Grade Line of Example 2 from Spring to Res. A and Res. B

NOTE 1. As can be sen from the profile Point A (at about 750m from source) is found to be low point hence needs washout valve to be installed.
2. As can be seen from the profile Point B (at about 1100m from source) is found to be point of peak elavation hence air release valve is to be installed.

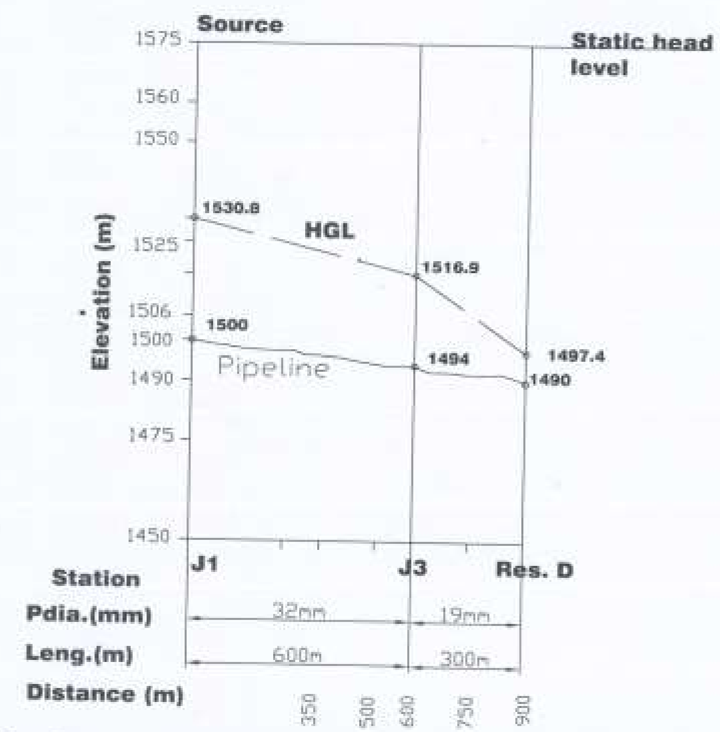


Fig. 3.17.b Hydraulic Grade Line of Example 2 from J1 to Res. C and Res. D

6. GUIDELINES FOR PREPARATION OF COST ESTIMATES and WATER TARIFF CALCULATION

There are essentially three types of costs experienced over the life cycle of the water supply investment projects. These are:

Capital costs - these are the costs initially required to construct the water supply system and should either be financed through the use of loans and grant assistance, and possibly also through up-front contributions from communities.

Recurrent costs - often termed operation and maintenance (O&M) costs and

Replacement costs - includes the replacement cost of any component of the water supply system when required.

The cost of the water supply includes all the above cost components which will occur at different times throughout the project life.

6.1 Capital costs

Capital costs should be divided into their main components, which will vary for each water supply technology type and could include:

6.1.1 Physical Investment Cost

The term "physical investment cost" as used here relates to the total cost of constructing physical components of the water supply system. The term includes the cost of materials, equipment and labor, but does not include the management cost of the client, cost of engineering design & supervision services and physical contingencies.

The cost is usually expressed in unit cost method. The unit cost of construction shall be determined for such items as site clearing, excavation, pipeline, concrete works, form works, reinforcement etc. These can be expressed as unit costs (per number, cubic meter, square meter, linear meter as appropriate) and shall include all the works involved in carrying out and completing the relevant items for each components of the system such as intakes, reservoirs, pipeline, public fountains etc. These costs shall be derived from recently quoted unit prices by contractors for similar works. For this the unit price of the Amhara Water Works Construction Enterprise which is in use now can be one basis for the estimate.

The physical investment cost for each component of the water supply system is calculated using the following table.

Table 6.1 Table in use for calculating physical investment cost

It. No.	Description of activity	Unit	Quantity	Unit price	Total price

6.1.2 Clients Management Cost

This cost shall be calculated as percentage of the physical investment cost to cover management cost of the client during the construction of the projects. Such costs include transportation & perdiem to be paid when travelling , stationers, telephone etc.

From past experience on works of similar water supply projects nature and scope, a two percent allowance shall be made to cover such costs.

6.1.3 Engineering Design and Supervision Services Costs

The cost of the engineering design fee is to be fixed as a lump sum fee as shall be stipulated, in the consultancy agreement that shall be signed between the client and various consulting engineers. It ranges from 2 - 6% of the physical investment cost from past records. Of course this cost is needed during the study and design of the project.

The cost of supervision services shall be determined as a percentage of the physical investment cost to cover the cost of the on site supervision, preparation of construction drawings and the consultant projects management. From past experience on such works and scope 6 - 8% allowance shall be assumed.

6.1.4 Project Base Cost

The term " project base cost" shall express the physical investment cost plus management cost, engineering design and supervision services fees.

6.1.5 Physical Contingencies

The physical contingencies represent the allowance required to cover the range of an unticipated discrepancies between the estimated cost and actual expenditures. These are expressed as a percentage of the project base cost. From past experience on works of water supply projects and scope, a ten percent allowance shall cover such contingencies.

6.1.6 Total Project Cost

The term " total project cost" as used here relates to the sum of the project base cost and the physical contingencies. In programmes such as RWSEP, the capital cost can be covered from different sources such as donors, government and community. In this case the summary of the total projects cost should be divided for each source of finance.

After calculating all the costs, the summary of the total project cost can be made using the following table for gravity water supply systems.

Table 6.2 Total Project Cost Summary for Gravity Water Supply System

It. No.	Cost Item	Amount in Birr	Remark
1	**Total physical investment cost**		
1.1	• Intake/source		
1.2	• Pipeline		
1.3	• Reservoir		
1.4	• Break pressure tank		
1.5	• Valve boxes		
1.6	• Public fountains		
1.7	• Others if any		
2	**Management Cost (2% of physical investment cost)**		
3	**Engineering design and supervision services cost (6% of the physical investment cost)**		
4	**Total project base cost (1+2+3)**		
5	**Physical contingency (10% of total project base cost)**		
6	**Total project cost (4+5)**		

6.2 Operation and Maintenance Costs

The operation and maintenance cost is the recurrent cost daily required for proper functioning of the waters supply facilities. The components of the operation and maintenance costs varies with the type of the water supply technology. Anyhow in motorized water supply schemes the costs includes the following. Of course with the deletion of some of the cost components the list below can also be used for gravity water supply systems.

- Power (Energy) Cost
- Chemicals cost used for water treatment if any
- Personnel cost for operating and managing the scheme
- Maintenance costs required to make the system work continuously
- Other costs

6.2.1 Power (Energy) Cost for Pumped Systems

If the water source is to reach to distribution points by pumping, a power cost is required to make the water available to communities. For this propose a generator or an EEPCO power if available in the area can be used.

If EEPCO power is used for running the system, the electric power required has to be calculated in Kilowatt hour and multiplied by the electric charge for one Kilowatts hour (Kwh).

If there is no EEPCO power in the area, generators are to be used as power source for pumping water. Hence fuel cost is calculated by multiplying fuel requirement in liter by fuel cost per liter. In addition, to estimate the cost of lubricants such as grease 10% be added on top of the fuel cost. For gravity schemes the power cost is not applicable.

6.2.2 Chemical Cost (applicable mainly for surface water sources)

There are some chemicals required for purifying water especially from surface water sources such as rivers. Therefore in such cases usually hydrated lime and calcium sulpahate used. The chemical cost is calculated by estimating the amount of chemicals required in kilogram for purifying a certain volume of water by the price of the chemical in kilogram. As springs does not need high treatment techniques, this component of the cost is not as such necessary to spring sources.

6.2.3 Personnel Cost

The monthly salaries of personnel required for operation and maintenance of the water supply system needs to be calculated. Of course this depends on the number and type of personnel to be employed and the amount to be paid for each personnel. Anyhow based on experience, the following personnel list and estimated salaries can be used as a basis for estimate. The actual cost calculation can be made with the nature of specific scheme and also by collecting data from related nature functioning schemes.

Table 6.3 Estimated number of personnel and salaries for operation & maintenance of a water supply system

It. No.	Type of personnel	Number required	Monthly salary	Remark
1	Manager	1	280	To be economical one person can be manager & accountant with some salary increment.
2	Accountant	1	240	
3	Cashier	1	220	
4	Plumber	1	200	Also can be employed on daily basis when required.

Table 6.3 Estimated number of personnel and salaries for operation & maintenance of a water supply system (Continued)

It. No.	Type of personnel	Number required	Monthly salary	Remark
5	Meter reader and bill collector	1	150	One of the tap attendants can be also bill collector with some salary increment.
5	Tap attendant	1	100	Depending on the location of taps one woman can be tap attendant for two taps.
6	Generator operator	1	220	Required only when there is generator
7	Guard	1	150	Will be employed if there is generator.

6.2.4 Maintenance Cost

Maintenance cost for all components of the proposed water supply system takes into account the materials and spare parts necessary for routine and non - routine maintenance.

The yearly maintenance costs to be determined using percentages of physical investment costs of each component. Of course in this case if it is thought to be a burden to the users, in calculating this cost the physical investment cost to be covered by the labor and materials contributions of the community such as trench digging and backfilling cost can be deducted.

The following table shall be used in the estimation of the maintenance cost for different types of works.

Table 6.4 Estimating the maintenance cost of water supply system components

It. No.	Water supply scheme component	Percent of yearly maintenance cost out of the physical investment cost of the component
1	Civil works (buildings, generator house, valve boxes etc.)	0.3 - 0.5
2	Pipelines	0.6 - 0.8
3	Reservoirs, intakes, treatment plants	0.3 - 0.7
4	Public fountains and water meters	1 - 5
5	Electrical and mechanical equipment	3 - 5

6.2.5 Other Costs

These costs include office supplies, uniform, perdiem & travel, postage & telephone, etc. It is assumed that these costs will be 3 - 4% of the above 4 costs combined.

6.3 Replacement Costs/Future Investment Costs

This cost component includes the replacement of any hardware component of the water supply system when the need arises.

The economic life of the different components of the water supply system should be estimated, including factor for depreciation. Depreciation can be estimated in a number of ways. In financial accounting, straight line method is often used. Which means if the expected life of an electric pump is 10 years, the cost of physical investment can be divided by 10 and this value would be recognized as an expense of cost each year.

The cost allocated for a replacement part must be high enough to ensure that when the major parts needed to be replaced, the managing authority has enough funds to immediately provide a replacement.

In this case in order to really calculate what portion of the investment cost and which components of the system and when to be replaced should be clearly known. In the water sector policy of our country it is clearly stated that urban water supply schemes the full cost of the component to be recovered by the users at the end of design period.

Anyhow when thinking of the cost recovery for the electromechanical parts such as pumps and generators should be replaced within 10 - 15 years and other components such as reservoirs, pipelines, sources/intakes, boreholes, public fountains etc. be replaced at the end of the design period of the scheme (may be from 15 - 25 years to which the scheme is initially designed).

In rural water supply schemes what portion of the physical investment cost to be recovered be made with detailed observation of the willingness to pay survey result made in the community, the possibility of getting credit facility in the future if matching fund is needed and the possibility of donors to be involved in the area in the future. Anyhow as much as possible it is proposed to cover some portion of the physical investment cost.

With detailed studies to be made in the area and continuous discussions with the people, the covering of the physical investment cost for gravity systems will start by assuming some percentages and revising it in the future. For example, to start with the following can be some basis for including cost recovery in the water tariff.

For pumped schemes:

- The electromechanical parts i.e. pumps and generator to be replaced within 15 years.
- Five percent of the cost of other components to be recovered within the first 5 years (from 1-5 years of the scheme service) period.
- Ten percent of the cost of other components to be recovered within the second 5 years (from 6-10years of the scheme service) period.
- Fifteen percent of the cost of other components to be recovered within the third 5 years (from 11-15years of the scheme service) period.

- Twenty percent of the cost of other components to be recovered within the fourth 5 years (from 16-20years of the scheme service) period if it is designed for more than 15 years.

If the above basis are followed, we can see that at the end of the 20 years design period, about 60% (assuming pumps and generators account 10% of the total cost) of the construction cost of the water supply system will be recovered.

For gravity systems with spring as source:

- Five to ten percent of the capital cost of all the components to be recovered within the first 5 years (from 1-5 years of the scheme service) period.
- Ten to fifteen percent of the capital cost of all the components to be recovered within the second 5 years (from 6-10years of the scheme service) period.
- Fifteen to twenty percent of the capital cost of all the components to be recovered within the third 5 years (from 11-15years of the scheme service) period.
- Twenty to twenty five percent of the capital cost of all the components to be recovered within the fourth 5 years (from 16-20years of the scheme service) period if it is designed for more 20 years.

If the above basis are followed, we can see that at the end of the 20 years design period, about 50% to 70% of the total construction cost of the water supply system will be recovered.

6.4. Water Tariff Calculation

6.4.1 Water tariff

One of the most important issues in designing and implementing a water supply system is how to ensure the financial sustainability of the project.

A common occurrence on village/small town water supply projects is that once the scheme is constructed, then those responsible for operation and maintenance do not invest sufficient money in this important task, so water services eventually deteriorate. To increase the prospects of sustainable schemes, reasonable and accurate operation & maintenance and depreciation cost estimates based on good operation and maintenance practice should inform future water tariff increase. It is also necessary to explain to key stakeholders why the proposed operation and maintenance expenditures are necessary. Time and effort is therefore well spent on predicting reasonably accurate cost estimates so that people have some confidence in the validity of the estimates for future financial planning

Tariff is normally a set of procedural rules used to determine the conditions of services and the types and levels of charges for water users in various categories.

Water departments and town council should consider the proper design of the tariff before the new water supply scheme is commissioned. The tariff structure will need to be agreed by key stakeholders including local authorities, the user population, the relevant government departments and possibly lending institutions.

6.4.2 What are the main purposes of a tariff ?

The main purposes of setting tariff for a water supply system are the following:

- **Adequate cost recovery** is often the primary purpose of a tariff. A decision needs to be made on the level of cost recovery. Should the tariff covers capital costs, or just operation and maintenance costs only?
- **Equity** usually means that users pay amounts which are proportionate to the benefits they get from the utility. This implies equals are treated equally.
- **Cross subsidization** is a term related to fairness but often explicitly incorporated into tariff design. It is assumed that tariff in developing countries should be used to redistribute income from richer households to poorer household. The idea behind progressive tariff is that the first block of consumption is priced with the operation and maintenance cost coverage and the following blocks are priced slightly above operation and maintenance cost. This should mean that large volume users subsidize small volume users.

6.4.3 Key issues for tariff setting

The following are the key issues in setting water tariff.

Table 6.5 Key issues in setting water tariff

It. No.	Issue	Potential impact on tariff policy
1	National policy priorities	National or state policy might impact on tariff setting. For example if government policy is to move towards full cost recovery, including capital costs, this should impact on tariff increase.
2	Cross subsidization of poorer communities	If one aim is to improve equity, tariff can be set at different levels for different user groups. This could be done by volume of use, or by type of user (commercial, domestic, etc.)
3	Consideration of the cost of water supply	A key decision is to determine whether tariff should aim to cover only the operational costs of a water supply system, or to include capital costs and provision for future expansion.
4	Willingness to pay of communities	This is an important factor and is becoming increasingly accepted as a key element of tariff setting. Tariff can be raised for those individuals/communities who are willing to pay more for the water supply.
5	Willingness to charge	Policymakers/politicians may be unwilling to increase water charges because they perceive that tariff increases are likely to be unpopular with the public. Orientation of policymakers is often required to demonstrate the benefits to all stakeholders of generating funds through increased tariff level.

6.4.4 Tariff and CAFES principles

A simple set of principles have been proposed to improve tariff-setting methodologies. These principles, often called CAFES, are described in Table 6.6 below, and it is suggested that policymakers and tariff setters need to consider all these factors in agreeing future \tariff structures.

Table 6.6 Tariff and CAFES principles

It. No.	Principle	Description of the principle
1	Conserving	The structure of tariff should influence consumption to the extent that consumers will purchase enough of the service without being wasteful.
2	Adequate	The funds generated must be sufficient to enable financial commitment to be met and some contribution to be made to future investment.

Table 6.6 Tariff and CAFES principles (Continued)

It. No.	Principle	Description of the principle
3	Fair	The tariff charges must be allocated between consumer groups in a fair and equitable manner paying particular attention to the needs of the poorer members of the community.
4	Enforceable	It must be possible to charge and collect the tariff, i.e. consumers must know that water will be disconnected if tariffs are not paid keeping other conditions satisfied by the water management body.
5	Simple	The tariff should be simple to administer an easy for consumers to understand.

6.4.5 Types of tariff

There are a number of tariffs used in the water sector, the most common ones are:

a) Flat tariff: tariff does not change with consumption.

This tariff is divided into two as follows:

Unmetered flat rate tariff: in this system each user household pays a fixed amount of money regardless of the volume of water used. And the payment amount is usually set by dividing the total operation & maintenance and depreciation cost required by the number of household using the system. This type of tariff is commonly used in hand pumps and on spot springs.

Metered flat rate tariff: in this tariff system each household pays fixed amount of unit rate of tariff for unit volume of water used such as in liters or meter cube. Hence the payment for water use will vary only with amount of water used. For example, if the price of 1meter cube of water is 2.5Birr and if a household uses 2 meter cube of water, the payment will be 5Birr. This type of tariff is common in public fountain users of piped water systems.

b) Progressive tariff: tariff increases as consumption increases.

In this system a different water prices would be applied to each cubic meters of water or certain blocks of cubic meters of water consumed.

For example, 2.5Birr/meter cube for 0 - 10meter cube water users and 3Birr/meter cube for 10 - 20 meter cube of water users. In this example if a household uses 15 cubic meters of water in a month, the total payment will be 2.5Birr/meter cube x 10meter cube + 3Birr/meter cube x 5 meter cube = 40Birr

This tariff is usually a structure of choice in developing countries because tariff in developing countries should be used to redistribute incomes from richer household to poorer households (cross subsidization). The idea behind this tariff structure is that the first block of consumption is priced to cover at least the operation and maintenance cost and subsequent blocks are priced slightly above the operation and maintenance costs. This should mean that large volume water users subsidize small users.

This tariff system is applicable mainly for yard and house connection users of a piped water supply system.

c) Lifeline tariff/Social tariff: it is a part of the progressive tariff, but the first X liters of consumption are either free or very cheap (only to cover the operation and maintenance cost). This is also preferred tariff type for developing countries in such a way to address the poorer family members.

In small towns/village water supply systems, a mixture of the metered flat rate system for public fountain users and the progressive tariff system for yard and house connection users are used.

6.4.6 Tariffs Guidelines in the Ethiopian Water Resources Management Policy

In the Federal Water Resources Management Policy of Ethiopia issued in June 1999, concerning water finance and tariff the following important points are put.

Finance

- Recognize water as a vulnerable and scarce resource and ensure as well as promote that all pricing systems and regulations should aim at conservation, protection and efficient use of water and equity of access to all.
- Ensure that pricing of water should neither too expensive as to discourage use and nor too cheap as to encourage misuse and waste.
- Provide subsidies on capital cost only to communities which can not afford to pay for their basic services based on established criteria and eventually phase out subsidy.
- Ensure that water supply undertakings shall adequately cover operation and maintenance costs and be based on the principles of " cots recovery".
- Ensure transparency and fairness in the management of water supply to foster willingness to pay and participation in the financial management of systems by users and communities.

Tariff

- Ensure that tariff structure are site specific, depending on the particularities of the project location, cost, users and others related traits of each project.
- Encourage rural tariff settings are based on the consumers willingness and ability to pay and urban tariff structures on the basis of ability to pay by convincing and explaining clearly its ultimate objective and advantages.
- Establish **"Social Tariff"** that enables low-income groups to cover operation and maintenance expenses.

- Establish progressive tariff structure that matches with consumption rates.
- Develop flat rate tariff for communal services like hand pumps and public fountains.

There is one major mis-undestanding by many people concerning rural water supplies tariff as the tariff is to cover only the operation and maintenance cost which is not the real content of the policy and are not willing to establish a tariff to cover at least some portion of the capital cost even in well to do communities. Therefore, it should be clearly understood that even in rural communities the social tariff is to be used only in those communities or households having very low income.

6.4.7 Tariff calculation from the point of technical requirements of the system

In the above sections the basic principles and the most likely cost components to be included in the water tariff setting were presented. Using these points as basis the calculation of the water tariff from the technical requirements of the system, i.e. irrespective of the willingness to pay survey result and the affordability/ability to pay of the users is presented below.

In general in piped water supply systems water tariffs are usually set in cost per unit volume of water used and put mathematically as follows:

$$\textbf{Water tariff} = \frac{\textbf{Functioning costs for a daily production of water in Birr}}{\textbf{Volume of water produced from the source daily in cubic meter}}$$

As discussed in detail above, one problem in the villages water supply schemes tariff setting is to decide which components of the costs are to be included in the tariff. Hence it is proposed to calculate the tariff in three different types assuming different cost coverage components. And the final decision can be made in discussion with the people and based on the willingness to pay survey result which is conducted during the pre feasibility study and also the affordability/ability to pay based on some studies conducted by the World Bank which is to be presented in the next section of this manual.

Hence the three alternatives tariffs from the technical requirements of the system are:

$$\underset{\textbf{(Social tariff)}}{\textbf{O \& M (minimum) cost tariff}} = \frac{\textbf{O \& M cost spend in a day in Birr}}{\textbf{Volume of water produced from the source daily in m}^3}$$

$$\underset{\textbf{(Real cost tariff)}}{\textbf{Replacement cost tariff}} = \frac{\textbf{(O \& M cost + replacement cost) in a day in Birr}}{\textbf{Volume of water produced from the source daily in m}^3}$$

$$\textbf{Total (full) cost tariff} = \frac{\textbf{(O \& M cost + recovery of capital cost) in a day in Birr}}{\textbf{Volume of water produced from the source daily in m}^3}$$

6.4.8 Treatment of inflation on tariff

It is recommended that estimates of future cost streams - whether capital cost, replacement costs or operation & maintenance costs used to make estimates of the likely cost of providing each option - are always in constant prices, which means that inflation is not included in the calculations for costing options..

Inflation does exist within the 'real' economy, therefore, those stakeholders who are responsible for increasing tariffs should take inflation into account as well as tariff increases that are needed to meet operation & maintenance and depreciation cots.

Financial planning must allow for expected inflation. If the average price of goods and services is increasing by 5 percent each year, the tariff that is eventually charged for the water supply options must also increase by a similar amount. Wages of staff, and the cost of components and parts, etc. are likely to be increased at 5 percent annually, and without a matching increase the tariff will not be sustainable and recover O & M costs.

Therefore, in order to have a sustainable tariff system on the tariffs shown to be calculated in section 6.4.7 above, additional 5 - 10% be added to account for inflation which is likely to happen.

6.4.9 Importance of willingness to pay survey and affordability in tariff setting

One the most important issues in designing and implementing a water supply system is how to ensure the financial sustainability of the project. This can involve predicting what users will be able and willing to pay for water in the future.

The water tariff calculation shown in section 6.4.7 above is only taking into consideration the technical requirement of the water supply system i.e. irrespective of willingness to pay survey result and the affordability/ability to pay of the users.

But in order to have a well accepted and sustainable tariff structure, the tariff calculated from the technical requirements of the scheme would be compromised with the willingness to pay survey result and the affordability of the users for the tariff.

6.4.9.1 Willingness to pay

" Willingness to pay (WTP) is that the maximum amount that an individual states he/she is willing to pay for a good or service" - in this case improved water supply offered"

Willingness to pay, is an expression of community's demand, is a strong pre-requisite for the financial sustainability of a water supply system. Willingness to pay, which is a useful yardstick for assessing the project feasibility, depends on the number of factors as:

- Demand for water and participation of communities
- Level of income

- Socio-cultural factors
- Perception of ownership and responsibility
- Service level and standard
- Price
- Relative costs to other services such as schooling, health etc.
- Reputation of service agency
- Transparency of financial management
- Perceived benefits
- Characteristics of existing sources
- Policy environment

The willingness to pay of the households can only be determined by conducting willingness to pay survey within the community. The survey is to be made by preparing a questioner and be filled and analyzed by the Woreda/district experts located close to the community. For RWSEP case the questioner is prepared and will be available to Woreda Water Resources Development Teams. The survey should be made by the latest during the pre-feasibility study of the project.

After the survey is conducted at field level, analysis is to be made and if the result of the analysis shows that the majority of the communities are not willing to pay, the detailed study or design of the project can not be made. If the result of the analysis shows that most of the households are willing to pay for the system, the result of the analysis is well kept to be used in tariff setting after compared and compromised with the tariff required form the technical point of view requirement of the system which is to be determined during the detailed design stage.

If the average willingness to pay survey result is found to be less than the operation and maintenance requirement tariff calculated in section 6.4.7 during the detailed design of the scheme, the result to be explained to the users if they are willing to revise their willingness to pay at least for covering the O & M tariff, and if they are willing to increase the project can be implemented and if they are not willing to increase the project need not to be implemented.

6.4.9.2 Affordability/ability to pay of users

Based on different studies conducted such as by World Bank, it is generally admitted that people should not have to pay more than 2 - 4 % of their income for water services (affordability criteria). A higher percentage of income expended on water will mean other important needs may not be fully met. Great care is therefore required when setting users' tariff.

The affordability data of the specific water supply system can be determined from willingness to pay survey. And this is determined by multiplying the average annual or monthly income of the household determined during the willingness to pay survey by 2 - 4%.

Of course compared to the willingness to pay survey, this is not as such a perfect measure of what people will pay for water. Anyhow, the purpose of knowing this criteria is that sometimes two extreme figures may come during the willingness to pay survey result as follows:

- The first category of data is that in order the water supply system to be constructed people may express higher amount of willingness to pay irrespective of their monthly or annual income.
- The other is that some people may express lessor amount of willingness to pay for water as compared to their income due to various reasons one may be assuming that the government to cover the operation and maintenance cost if they express lessor amount of willingness to pay.

Hence in such cases the affordability criteria may also be considered to set well accepted tariff.

6.4.10 Conclusion on tariff setting

As discussed in the above sections well accepted and proper tariff setting involves number of important elements. So a well prepared, well accepted and sustainable tariff is set by compromising the technical requirement of the system, the willingness to pay and the affordability to pay of the community.
Hence as much as possible the following are required to be met in setting tariff in their order of importance:

- Preferably & if possible the tariff to be set is to satisfy the real cost tariff, the average willingness to pay of the people and accounting maximum of 4% of the average income of the users or.
- In any case the average willingness to pay analysis result should not be less than the minimum or operation & maintenance cost tariff required from the technical requirements point of view.

In addition, the tariff is to be prepared for some period of years at a time and revision is required to be made on the tariff within certain intervals of time.

6.5. Worked Example in Tariff Calculation

6.5.1 Tariff setting example for gravity systems

Design data

A gravity water supply system of a rural village X has the following design data.

- Design period = 20 years
- In the first 5 years the average total daily water requirement for the village is 40 cubic meter.
- The system is constructed with the following initial investment cost shown in Table 6.7 (excluding the in kind and materials contribution made by the community)

Table 6.7. Physical investment cost for tariff calculation example of gravity system

It. No.	System component	Physical investment cost in Birr
1	Source development	25,000
2	Reservoir	45,000
3	Pipeline	395,000
4	Public fountains	30,000
5	Valve boxes	10,000
6	Cattle trough	10,000
	Total investment cost	**515,000**

- Based on the discussion made with the people, the following personnel list are required to be employed for the proper management of the system at least for early years of the system service.

Table 6.8. Personnel list and salary for tariff calculation example of gravity system

It. No.	Personnel title	Number required	Salary per month in Birr
1	Manager and accountant	1	350
2	Cashier	1	220
3	Plumber	1	200
4	Tap attendants	3	150

- It is also assumed that 10 percent of the capital cost has to be recovered within the first 5 years of the scheme use.
- Inflation is assumed to be taken as 10%.

Using the above design data and relevant section of this design manual, calculate the minimum tariff, real cost tariff and full cost tariff for the system per cubic meters of water and also per 20 liters of water.

Solution for tariff calculation example of gravity system

The detail calculation for the solution is shown below.

a) Operation and Maintenance Cost

a.1) Personnel cost

Table 6.9. Personnel cost calculation for tariff setting example of gravity system

It. No.	Personnel title	Number	Salary/month in Birr	Total salary in Birr
1	Manager and accountant	1	350	350
2	Cashier	1	220	220
3	Plumber	1	200	200
4	Tap attendants	3	150	450
	Total personnel cost /month			**1220**
	Total personnel cost /day = 1220Birr / 30days			**40.70**

a.2) Maintenance cost

Table 6.10. Maintenance cost calculation for tariff setting example of gravity system

It. No.	Component	Investment cost in Birr	% of yearly maintenance cost out of investment cost	Total maintenance cost in Birr
1	Source development	25000	0.4	100
2	Reservoir	45000	0.5	225
3	Pipeline	395000	0.7	2765
4	Public fountains	30000	4	900
5	Valve boxes	10000	0.4	40
6	Cattle trough	10000	0.4	40
	Total maintenance cost /year			**4070**
	Total personnel cost /day = 4070Birr / 365days			**11.15**

a.3) Other costs

Other costs account 4% of the sum of personnel and maintenance costs = 0.04 (40.7 + 11.15) **= 2.07Birr/day**

a.4) Total operation and maintenance cost = 40.7 + 11.15 + 2.07 = <u>52.57Birr/day</u>

b) Replacement Costs

As given in the design data 10% of the total investment cost to be replaced within the first 5 years

- Total cost to be recovered within five years = 0.1 x 515000Birr = 51500Birr
- 10 percent of the total cost to be recovered yearly = 51500Birr / 5 years = 10300Birr/year
- 10% of the total cost to be recovered daily = **10300Birr/ 365 days = <u>28.20Birr/day</u>**

c) Capital Cost Recovery

It is given that the total capital cost to be recovered within the coming 20 years.

- Hence capital cost to recovered yearly = 515000Birr / 20 years = 25750Birr/year
- The portion of the capital cost to be recovered daily = **25750Birr / 365days = 70.55Birr/day**

d) The three different Tariffs Calculation

The three different types of tariffs are calculated and shown in Table 6.11 below.

Table 6.11. Summary of calculation for tariff setting example of gravity system

It. No.	Cost component	Amount in Birr/day of each cost component for each tariff type		
		O & M cost tariff	**Real cost tariff**	**Total cots tariff**
1	Personnel cost	40.7	40.7	40.7
2	Maintenance cost	11.15	11.15	11.15
3	Others cost	2.07	2.07	2.07
4	Replacement cost	-	28.2	-
5	Total cost to be recovered	-	-	70.55
6	Sub total costs	53.92	82.12	124.47
7	Inflation (10% of sub total cost)	5.39	8.21	12.45
8	Total cost/day =(6+7)	59.31	90.33	136.92
9	Amount of water required/day in m^3	40	40	40
10	**Tariff / m^3 (8/9)**	**1.48Birr**	**2.26Birr**	**3.42Birr**
11	**Tariff / 20liters**	**0.03Birr**	**0.05Birr**	**0.07Birr**
12	**Tariff for 20 liters of water to the nearest practical payment (assuming one pot has 20 liters volume capacity)**	**10 cents/ 60 liters or 10cents/ 3 pots**	**5 cents/ 20 liters or 5cents/ 1 pot**	**15 cents/ 40 liters or 15 cents/ 2 pots**

6.5.2 Tariff setting example for pumped/motorized systems

Design data

The water supply system of a rural village has a borehole source. The system has the following design data.

- Design period = 15 years
- In the first 5 years the average total daily water requirement for the village is 50 m^3.
- The system is constructed with the following initial investment cost shown in Table 6.12 (excluding the in kind and materials contribution made by the community).

Table 6.12 Physical investment cost for tariff calculation example of pumped system

It. No.	System component	Physical investment cost in Birr
1	Borehole drilling & construction	120,000
2	Pump and generator	130,000
3	Generator house	25,000
4	Reservoir	45,000
5	Pipeline	395,000
6	Public fountains	30,000
7	Valve boxes	10,000
8	Cattle trough	10,000
	Total investment cost	**765,000**

- The following personnel list are required to be employed for the proper management of the system at least for early years of the system service.

Table 6.13 Personnel list and salary for tariff calculation example of pumped system

It. No.	Personnel title	Number required	Salary per month in Birr
1	Manager and accountant	1	350
2	Cashier	1	220
3	Plumber	1	200
4	Tap attendant and bill collector	1	180
5	Tap attendants	2	120
6	Generator operator	1	220
7	Guard	1	150

- The generator works 6 hours in a day with fuel consumption of 4 liters/hour.
- The cost of one liter of fuel is 4.5Birr
- It is also assumed that 5 percent of the capital cost other than pump and generator has to be recovered within the first 5 years of the scheme use.
- Inflation is also assumed to be taken as 5%.

Using the above design data and relevant section of this design manual, calculate the minimum tariff, real cost tariff and full cost tariff for the system per cubic meters of water and also per 20 liters of water.

Solution for tariff calculation example of pumped system

The detail calculation for the solution is shown below.

a) Operation and Maintenance Cost

a.1) Power Cost

A generator is used as a source of power. Hence fuel cost has to be considered for running it.

- Fuel cost/day = 6hours/day x 4lit/hour x 4.5Birr/lit. = 108 Birr/day
- Lubricant cost = 0.1 x 108 Birr/day = 10.8 Birr
- Total fuel and lubricant cost = **108 + 10.8 Birr = 118.8 Birr/day**

a.2) Personnel cost

Table 6.14 Personnel cost calculation for tariff setting example of pumped system

It. No.	Personnel title	Number	Salary/month in Birr	Total salary in Birr
1	Manager and accountant	1	350	350
2	Cashier	1	220	220
3	Plumber	1	200	200
4	Tap attendant and bill collector	1	180	180
5	Tap attendants	2	120	240
6	Generator	1	220	220
7	Guard	1	150	150
	Total personnel cost /month			**1560**
	Total personnel cost /day = 1560Birr / 30days			**52.00**

a.3) Maintenance cost

Table 6.15 Maintenance cost calculation for tariff setting example of pumped system

It. No	Component	Investment cost in Birr	% of yearly maintenance cost out of investment cost	Total maintenance cost in Birr
1	Borehole	120,000	0.4	480
2	Pump and generator	130,000	4	5200
3	Generator house	25,000	0.4	100
4	Reservoir	45000	0.5	225
5	Pipeline	395000	0.7	2765
6	Public fountains	30000	4	900
7	Valve boxes	10000	0.4	40
8	Cattle trough	10000	0.4	40
	Total maintenance cost /year			**9750**
	Total personnel cost /day = 9750Birr / 365days			**26.71**

a.4) Other costs

Other costs account 4% of the sum of power, personnel and maintenance costs = 0.04 (118.8 + 52 + 26.71) **= 7.9 Birr/day**

a.4) Total operation and maintenance cost = 118.8+52 + 26.71 + 19.75 = <u>205.41 Birr/day</u>

b) Replacement Costs

The electromechanical parts, i.e. pumps and generators to be fully replaced after 15 years and 5% of capital cost of other components to be replaced in the coming 5 years.

Replacement cost of pump and generator:

- Pump and generator cost to be recovered/year= 130,000Birr/15 years = 8667Birr/year
- Pump and generator cost to be recovered daily = 8667Birr / 365days **= 23.75Birr/day**

Replacement cost of other components:

As given in the design data 5% of the total investment cost to be replaced within the first 5 years

- Total cost to be recovered within five years = 0.05 x 515000Birr = 25750Birr
- 5 percent of the total cost to be recovered yearly = 25750Birr / 5 years = 5150Birr/year
- 5% of the total cost to be recovered daily = **5150Birr/ 365 days = <u>14.10Birr/day</u>**

Total replacement cost = 23.75 + 14.10 = <u>37.85 Birr/day</u>

c) Capital Cost Recovery

It is given that the total capital cost to be recovered within the coming 15 years.

- Hence capital cost to recovered yearly = 765000Birr / 15 years = 51000Birr/year
- The portion of the capital cost to be recovered daily = **51000Birr / 365days = <u>139.73Birr/day</u>**

d) The three different Tariffs Calculation

The three different types of tariffs are calculated and shown in Table 6.16 below.

Table 6.16 Summary of calculation for tariff setting example of pumped system

It. No.	Cost component	Amount in Birr/day of each cost component for each tariff type		
		O & M cost tariff	Real cost tariff	Total cots tariff
1	Power cost	118.80	118.80	118.80
2	Personnel cost	52.00	52.00	52.00
3	Maintenance cost	26.71	26.71	26.71
4	Others cost	7.9	7.9	7.9
5	Replacement cost	-	37.85	-
6	Total cost to be recovered	-	-	139.73
7	Sub total costs	205.41	243.26	345.14
8	Inflation (5% of sub total cost)	10.27	12.16	17.26
9	Total cost/day= (7+8)	215.68	255.42	362.40
10	Amount of water required/day in m^3	50	50	50
11	**Tariff / m^3** (9/10)	**4.31Birr**	**5.11Birr**	**7.25Birr**
12	**Tariff / 20liters**	**0.09Birr**	**0.10Birr**	**0.15Birr**
13	**Tariff for 20 liters of water to the nearest practical payment (assuming one pot has 20 liters volume capacity)**	**35 cents / 80 liters or 35 cents/ 4 pots**	**10 cents/ 20 liters or 10 cents / pot**	**15 cents/ 20 liters or 15 cents/ pot**

7. PIPES AND PIPEWORKING

7.1 Types of Pipe

The proper selection and use of pipe is a vital component of all gravity and pumped water systems. Therefore, it is important for all technicians and engineers to be familiar with the characteristics of various types of pipe and to learn the correct methods of working with pipes in the field. This chapter will deal with three types of pipe which are widely used around the world and commonly used for piped water system. These are:

a) Galvanized iron pipe (GI) is regular iron pipe that is coated with a thin layer of zinc. The zinc greatly increases the life of the pipe by protecting it from rust and corrosion. It usually comes in 6 meters lengths and joined together by threaded connections.

b) Plastic polyethylene pipe (PE) is black, light weight, flexible pipe that comes in large coils, 30 meters or more length. The pipe varies in density, and is joined by inserted fittings with clumps or heat fusion using a steel plate.

c) Polyvinyl chloride pipe (PVC) is rigid pipe, usually white or gray in color. It comes in three or six meters lengths, and is joined primarily by solvent cement, but can also be threaded. The pipe varies in density, and when buried, is extremely resistant to corrosion.

The following table lists some of the characteristics of the three types of pipe.

Table 7.1 Some of the characteristics of the three types of pipe.

It. No.	Comparison parameter	GI pipe	PVC pipe	PE pipe
1	**Life expectancy Resistance to corrosion underground**	Very long life expectancy of 30 years or more. However, joints are subject to rust and may break if not properly supported.	Long life expectancy if properly laid and backfilled.	Generally good life expectancy. However, has low stress resistances and poor rigidity.
2	**Resistance to corrosion or chemical inside pipe**	Will corrode in acid, alkaline, and hard water.	Very resistant. However, very soft or very hard water can corrode.	Very resistant. However, very soft or very hard water can corrode.
3	**Safe working pressures (meters of water column)**	Adequate for all pressures found in small scale water systems (i.e. up to 250 meter head of water, or 2500KN/m^2).	Rating from 60 - 100 meters head of water, i.e. 600KN/m^2 - 1000 KN/m^2.	60 meters head of water, i.e. 600KN/m^2 .

Table 7.1 Some of the characteristics of the three types of pipe (Continued)

It. No.	Comparison parameter	GI pipe	PVC pipe	PE pipe
4	**Resistance to puncturing and rodents**	Very high resistances	Good resistance	Low resistance
5	**Effect of sun and weather**	No, effect; however., threaded ends may rust.	Weakens with exposure.	Weakens with exposure.
6	**Ease of joining, laying, bending.**	Difficult to join, lay and bend. Very heavy.	Easy to join and lay. Rigid, but will bend on long radius. Can be bent by heating.	Easy to join and lay because of few joints and light weight. bends readily, but will collapse on short bends
7	**Cost**	Very high cost, especially in larger diameters.	Moderate cost	Low cost

7.2 Working with pipes

7.2.1 Galvanized iron pipes

Before the advent of plastic pipe, GI was the primary type of pipe used in water systems. Much of it is still in use today. GI has several advantages in a water system; it is very durable in the field, able to withstand high pressure heads, and minimally affected by water hammer. Leakage is also rare because the pipe is very hard to puncture and the threaded joints tend to seal themselves over time. GI pipe may be laid above ground, under roads, or across streams, performing well under all these conditions. The threaded joints, however, can be broken much more easily than the solid pipe, and therefore, must always be well supported.

GI has also a number of disadvantage its weight makes it difficult to transport, threaded joints are difficult and time consuming to make, certain kinds of water can corrode and rust the pipe, and it is difficult to repair in the field or tap-in new branch lines. The tools necessary for working with GI pipe are expensive. However, if such tools are properly maintained they can last a lifetime.

A variety of fittings are used to connect the pipe. Pipe threads are called "male" for outside threads, and "female" for inside threads. For more on the types of fittings and their use see annex 4. The pipes themselves usually comes in 6 meters length, and are factory threaded at both ends, usually one coupling is also provided. A variety of diameter sized are available, from small (3/8" 1/2" , 3/4" and 1") to large (4" and 6"), these sizes always referring to the inside diameter of the pipe. The outside diameter would measure at least 1/4" larger because of the wall thickness. Depending on their wall thickness and weight which determines their pressure resistance, GI pipes are divided in the three as light, medium and heavy class GI pipes.

Light class GI pipes are recommended up to maximum working pressure of 120 meters. According to ISO standard these pipes are yellow color marked on them by the standard organization of the importing country. For more on wall thickness and related measurements or such pipes see annex 5 of this manual. The cost is cheaper than the medium and heavy class GI pipes.

Medium class GI pipes are recommended up to maximum working pressure of 250 meters. According to ISO standard these pipes are blue color marked on them by the standard organization of the importing country. For more on wall thickness and related measurements or such pipes see annex 5 of this manual. The cost is higher than the light class but cheaper than the heavy class GI pipe. This class of pipe is the most recommended class of pipes for most rural water supply schemes due to its pressure resistance capacity.

Heavy class GI pipes are recommended for pressures more than 250 meters. According to ISO standard these pipes are red color marked on them by the standard organization of the importing country. For more on wall thickness and related measurements or such pipes see annex 5 of this manual. This is the most expensive and not widely used pipe.

7.2.2 Polyvinyl chloride (PVC) pipe

PVC is preferred pipe in small rural water systems. It has several advantages when used in a water system. It is light weight and easily transported, simple to join, cut and lay, low cost relative to GI pipe, it is very resistant to corrosion and when properly laid in a trench, will last long. However, it also has some disadvantages; it is easily punctured, will withstand only moderate pressure heads, weakens when exposed to weather, and must be laid underground in a particular manner /with high care/ in order to perform satisfactorily.

The tools necessary for working with PVC are handsaw or hacksaw, file, clean dry cloth, PVC solvent cement and applicator. Fittings consist of couplings, reducers, elbows, adapters, tees and plugs. They are joined together with the use of solvent cement. The pipe comes in three or six meters lengths and is usually gray or white. It also comes in various densities which correspond to the amount of pressure head that will withstand. Cutting, joining and laying PVC pipes is a simple process.

7.3 Trenching

The plastic pipes (PVC and PE) must be buried underground to provide satisfactory service . Therefore, it is necessary to dig a trench for the entire length of the pipeline.

Trenching is not easy job, even under the best of conditions and consequently, digging the trench is usually the most time consuming and labor intensive task in a water project.

If trenching and pipe laying are done properly, however, the life of the system will be greatly increased, and maintenance problems greatly reduced.

There is no specific width of trench necessary (in practice, the width of the trench will be determined by the size of the digger, usually trenches are dug with 60cm width). The depth of the trench is recommended to be 1 meters for plastic pipes and at least 70cm for GI pipes.

When the pipe is buried one meter deep, it is adequately protected against the weight (and sharp hoofs) of heavy animals walking over it, it is well below the depth reached by farm plows, there is plenty of overburden (cover of soil) to allow for erosion over the lifetime of the system.

Trench itself should be of uniform depth and gradation. The trench should have no sharp corners nor run a zigzagging manner. The bottom should be relatively smooth and free of rocks or sharp objects which could damage the pipe.

The trenching and the pipeline should ideally follow the same route as that of the original survey was conducted. However, sometimes it is necessary to have some detours/change in alignment to avoid such things as heavy erosion areas, extremely rocky places, or step gullies. When such re-routing is necessary, the designer or supervisor must re-survey the new section to determine how it will affect the hydraulic gradient of the system, and to see whether additional pipe is necessary.
.
The pipeline should be kept as far away from errodable points as possible such as landslide areas, gullies, streams or river banks, etc. Motor roads should be crossed perpendicularly and the trench dug as deep as possible up to 150cm.

When crossing landslides, gullies and /or streams, a suspended pipeline may be necessary. At times, GI pipe may be needed to cross streams, roads, or others areas where trenching is impossible. These areas should be marked out when the original survey is done. In determining the rest of the route, the surveyor should select the easiest course for trenching.

7.4 Pipe laying

Laying the pipe as continuously as possible is best. With time and rainfall, open trenches will fill in, requiring them to be cleaned out again before laying the pipe. In additional pipes and fittings should be joined properly to avoid leakage to occur in the pipe.

In order to ease maintenance of pipes **unions** (one of the pipe fittings) must be used in locations of gate valves, water meters, and at every 60 - 90 meters distance, i.e. 10 -15 pipes interval.

For sample pipes and fittings connected see figure 7.1 of this manual

7.5 Backfilling

Once the pipe has been laid in the trench, it should be backfilled as soon as possible. Therefore, one should not lay more pipes in one day than can be backifilled in the same day.

The pipe should be completely backfilled except for the two to three meters at each joint. The joints should be only partially covered until the line has been tested for 24 hours with working pressure.

When backfilling, the pipe should first be completely covered with dirt alone (no rocks or sticks) up to one third of the trench depth. This earth should be compacted to protect the pipe from surface pressures. The trench is then completely backifilled with the remaining soil. Rocks may be placed towards the top of the trench. Remember to compact the soil while backfilling, this will help stabilize the trench. Also, the top of the trench, when complete, should have a light crown to allow water to run off the trench rather than down it.

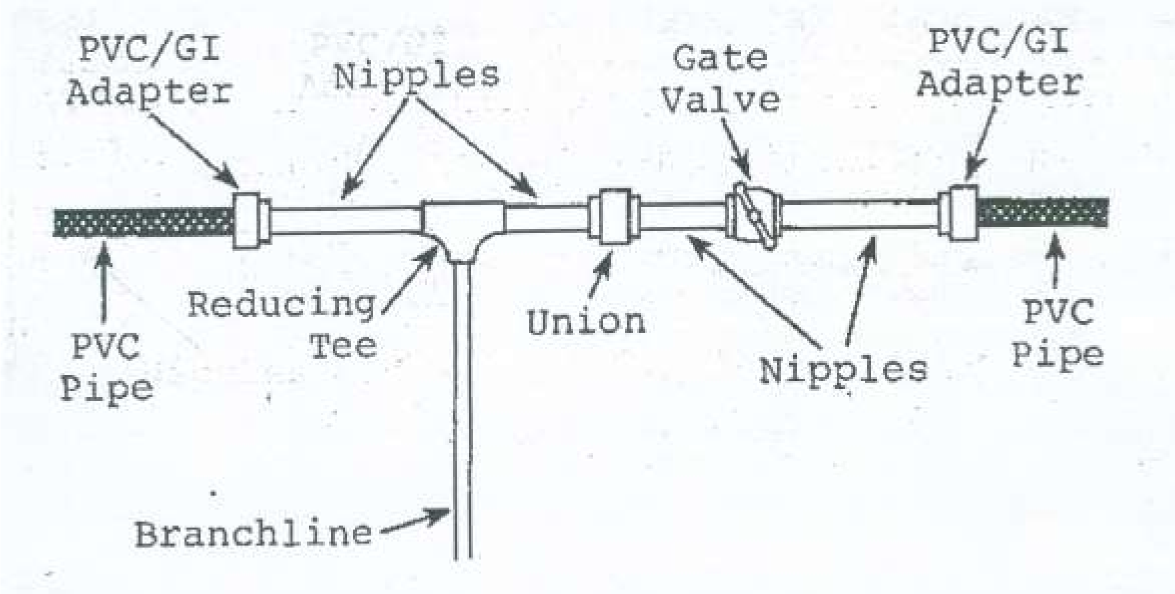


Figure 7.1 Sample pipes and fittings connection

REFERENCES

1. A Handbook of Gravity - Flow Water Systems for small communities
   by Thomas D. Jordan Jr.

2. Humanity Development Library for Sustainable Development and Human
   Basic Needs, Humanity Libraries Project - HDL 2.0 edition (CD)

3. Information and Training for Low - Cost Water Supply and Sanitation.
   Participants Notes.
   Author: D. Trattlles
   Editor : B. Latham

4. Study and Design of Seddie Rural Water Supply Distribution Network by
   Yohannes M., Andargie N., Mulatu A. and Wubegzier A.
   August 1999, Debre Markos

Annex 1 : WHO Water Quality Guideline

It. No.	Physico-Chemical Parameters	Unit	WHO Standard Recommended Max. or Min. - Max. Concentrations
1	Temperature	^{o}C	12 - 25
2	Turbidity	F.T.U	5 - 10
3	Colour	Pi-Co, Units	5 - 10
4	Odour		-
5	Taste		-
6	PH	No.	6.5 - 8.5
7	Total Dissolved Solids	Mg/l	1000
8	Conductivity	MS/CM	400 - 1200
9	Total Hardness as $CaCo_3$	Mg/l	100 - 500
10	Total Alkalinity as $CaCo_3$	Mg/l	-
11	Calcium Hardness as $CaCo_3$	Mg/l	10 - 100
12	Magnesium Hardness as $CaCo_2$	Mg/l	5 - 50
13	Hydrogen Supfide	Mg/l	0.5 (Fellable)
14	Carbon Dioxide as Co_2	Mg/l	-
15	Total Iron as Fe^{+++}	Mg/l	0.1 - 1.0
16	Manganese as Mn	Mg/l	0.05
17	Chloride as Cl	Mg/l	5 - 250
18	Fluoride	Mg/l	0.7 - 1.5
19	Nitrate as No_3	Mg/l	45
20	Nitrite as No_2	Mg/l	0.5
21	Ammonium as NH_4	Mg/l	0.05 - 0.5
22	Sulfate as So_4	Mg/l	200 - 250
23	Phosphorous as Po_4	Mg/l	2
24	Chromium Hexavalent as Cr	Mg/l	0.05
25	Silica as Sio_2	Mg/l	-

In addition to physico-chemical guidelines shown above, the bacteriological analysis needs to be made and <u>there should be no E.Coli per 100ml of sampled water.</u>

Annex 2. Guideline for Pre-Feasibility and Feasibility Report of a Water Supply Project

4. EXISING WATER SUPPLY AND THE NEED FOR THE PROJECT (*)

4.1 General **(*)**
4.2 Existing Water Sources, Coverage and Quality **(*)**
4.3 Water Tariff **(*)**
4.3.1 Cost of house, yard and public taps connection and operation **(*)**
4.3.2 Neighborhood vending and rate **(*)**
4.4 Water Consumption by Mode of Services **(*)**
4.5 Existing Water Systems Facility and Capacity **(*)**
4.6 Problems in the existing system **(*)**
4.7 Need for Rehabilitation **(*)**
4.8 Need for a Project **(*)**

5. POPULATION AND FUTURE WATER REQUIREMENTS (*)

5.1 Design Period **(*)**
5.2 Population Projection **(*)**
5.3 Targets for Level of Service
5.4 Projection of the Per Capita Water Demand **(*)**
5.4.1 Average Day Water demand **(*)**
5.4.2 Maximum Day Water Demand **(*)**
5.4.3 Peak Hour Demand **(*)**

6. ENGINEERING AND TECHNICAL ASECTS (*)

6.1 Design of Proposed Water System **(*)**
6.1.1 Selection of Potential Source **(*)**
6.1.2 Determination of Pump Capacity if any **(*)**
6.1.3 Design of Power Sources if any **(*)**
6.1.4 Design of Storage Reservoir **(*)**
6.1.5 Design of Distribution Networks **(*)**
6.1.6 Design of Service and Operation Buildings **(*)**
6.1.7 Design of other Structures if any **(*)**

6.2 Project Cost Estimates for the Designs **(*)**
6.3 Comparison of Alternative Schemes if any

6.4 Financial Aspects of the Recommended Source **(*)**
6.4.1 Construction Cost **(*)**
6.4.2 Construction Cost by Source of Finance and Financing Plan **(*)**
6.4.3 Operation and maintenance costs required **(*)**
6.4.4 Future tariff proposal **(*)**

7. CONCLUSION AND RECOMMENDATION (*)

7.1 Conclusion **(*)**
7.2 Recommended Actions **(*)**

<u>Note:</u>

In Rural Water Supply Study, it is enough to focus on the contents marked by (*) from the point of view of their importance

Annex 3. The Daily Water Demand Pattern of a Typical Village

In some villages where study is conducted in developing countries the daily water demand of a village can have close to the following schedules. The peak hour factor is determined usually from such schedules.

Schedule 1

Time	Percent of the daily water demand to be used by beneficiaries
6:00 AM - 8:00 AM	30% of total daily water demand
8:00 AM - 4:00 PM	40% of total daily water demand
4:00 PM - 6:00 PM	30% of total daily water demand
6:00 PM - 6:00 AM	Negligible water demand

Schedule 2

Time	Percent of the daily water demand to be used by beneficiaries
5:00 AM - 7:00 AM	10% of total daily water demand
7:00 AM - 11:00 AM	25% of total daily water demand
11:00 AM - 1:00 PM	35% of total daily water demand
1:00 PM - 5:00 PM	20% of total daily water demand
5:00 PM - 7:00 PM	10% of total daily water demand
7:00 PM - 5:00 AM	Negligible water demand

The first schedule is in general, theoretical pattern that is based upon the traditional Nepali custom of two major meals per day, including pre -meal hand washing, cooking and dish washing.

The second schedule ids based upon direct observation by Johnson of typical village in Western Nepal, after a water system had been completed for that village. Johnson feels that the other villages he observed generally conformed to that schedule.

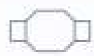
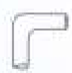
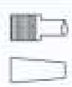
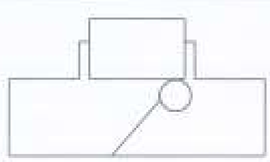
Example.

The projected population of a village is 720 persons, with no other special water needs. Safe yield of the source is 0.45l/s, What is the required capacity of the reservoir using schedule 1 and taking the maximum daily demand of 25 l/c/d.

Time	Supply in liters	Demand in liters	Difference of supply and demand
6:00 AM - 8:00 AM (2 hours, 30%)	0.45l/s x 3600s x 2hrs = 3240	720 p x 25l/c/d x 0.3 = 5400	-2160 (water withdrawn)
8:00 AM - 4:00 PM (8 hours, 40%)	0.45l/s x 3600s x 8hrs = 12960	720 p x 25l/c/d x 0.4 = 7200	+5760 (tank overflows)
4:00 PM - 6:00 PM (2 hours, 30%)	0.45l/s x 3600s x 2hrs = 3240	720 p x 25l/c/d x 0.3 = 5400	-2160 (water withdrawn)

Therefore, the capacity of the reservoir needed = **2160 liters (largest deficiency)**

Annex 4: Types of GI pipe fittings & their use

Fitting Name	Shape	Thread Type	Use of the Fitting
Pipe		External	To convoy water
Coupling or socket		Internal	To fit pipes together
Union		Internal	To join pipes together or with fitting for ease of maintenance
Plug		External	To close pipe ends
Tee		Internal	To branch the pipe in to three different directions
Cross Tee		Internal	To branch the pipe in to four different directions
Elbow		Internal	To bend the pipe 90 degree
Reducer		External for the larger dimater & Internal for the smaller diameter	To join two pipes having different diameters
Nipples		External	To join two fittings having internal thread
Gate Valve		Internal	To control (on or off) flows passing in the pipe in the valve installed direction
Faucet		External	To open or close a fountain
Water Meter		External	To measure the amount of water dischraged from source or reservoir
Check Valve		Internal	To permit water to flow in one direction only and is installed usually in borehole outlet pipes.

Annex 5 : Technical data of galvanized steel pipes confirming to BS : 1387 of 1985

Identific. Color by S.O.	Class	Internal pipe diameter		Outside pipe diameter				Wall thickness		Socketed pipe weight
				Maximum		Minimum				
		Inch	mm	Inch	mm	Inch	mm	Inch	Mm	Kg/meter
YELLOW	LIGHT (A)	1/2	15	0.843	21.4	0.827	21.0	0.079	2.00	1.00
		3/4	19 - 20	1.059	26.9	1.039	26.4	0.091	2.30	1.40
		1	25	1.331	33.8	1.307	33.2	0.102	2.60	2.02
		1 1/4	32	1.673	42.5	1.650	41.9	0.102	2.60	2.57
		1 1/2	40	1.906	48.4	1.882	47.8	0.114	2.90	3.28
		2	50	2.370	60.2	2.346	59.6	0.114	2.90	4.20
		2 1/2	63 - 65	2.991	76.0	2.961	75.2	0.124	3.20	5.87
		3	75 - 80	3.492	88.7	3.461	87.9	0.124	3.20	6.90
		4	100	4.484	113.9	4.449	113.0	0.142	3.60	10.03
BLUE	MEDIUM (B)	1/2	15	0.854	21.7	0.831	21.1	0.102	2.60	1.23
		3/4	19 - 20	1.071	27.2	1.047	26.6	0.102	2.60	1.59
		1	25	1.346	34.2	1.315	33.4	0.124	3.20	2.46
		1 1/4	32	1.689	42.9	1.657	42.1	0.124	3.20	3.15
		1 1/2	40	1.921	48.8	1.890	48.0	0.124	3.20	3.65
		2	50	2.394	60.8	2.354	59.8	0.142	3.60	5.17
		2 1/2	63 - 65	3.016	76.6	2.969	75.4	0.142	3.60	6.63
		3	75 - 80	3.524	89.5	3.469	88.1	0.157	4.00	8.64
		4	100	4.524	114.9	4.461	113.3	0.177	4.50	12.40
		5	125	5.535	140.6	5.541	138.7	0.197	5.00	16.90
		6	150	6.539	166.1	6.461	164.1	0.197	5.00	20.00
RED	HEAVY (C)	1/2	15	0.854	21.7	0.831	2.1	0.124	3.20	1.49
		3/4	19 - 20	1.071	27.2	1.047	26.6	0.124	3.20	1.90
		1	25	1.346	34.2	1.315	33.4	0.157	4.00	2.59
		1 1/4	32	1.689	42.9	1.657	42.1	0.157	4.00	3.85
		1 1/2	40	1.921	48.8	1.890	48.0	0.157	4.00	4.47
		2	50	2.394	60.8	2.354	59.8	0.177	4.50	6.30
		2 1/2	63 - 65	3.016	76.6	2.969	75.4	0.177	4.50	8.14
		3	75 - 80	3.524	89.5	3.469	88.1	0.197	5.00	10.60
		4	100	4.524	114.9	4.461	113.3	0.213	5.40	14.70
		5	125	5.535	140.6	5.461	136.7	0.213	5.40	18.20
		6	150	6.639	166.1	6.461	164.1	0.213	5.40	27.50

NB. In order to check that the pipe brought by the supplier is as ordered, measure the dimensions in caliper and the weight in a balance.

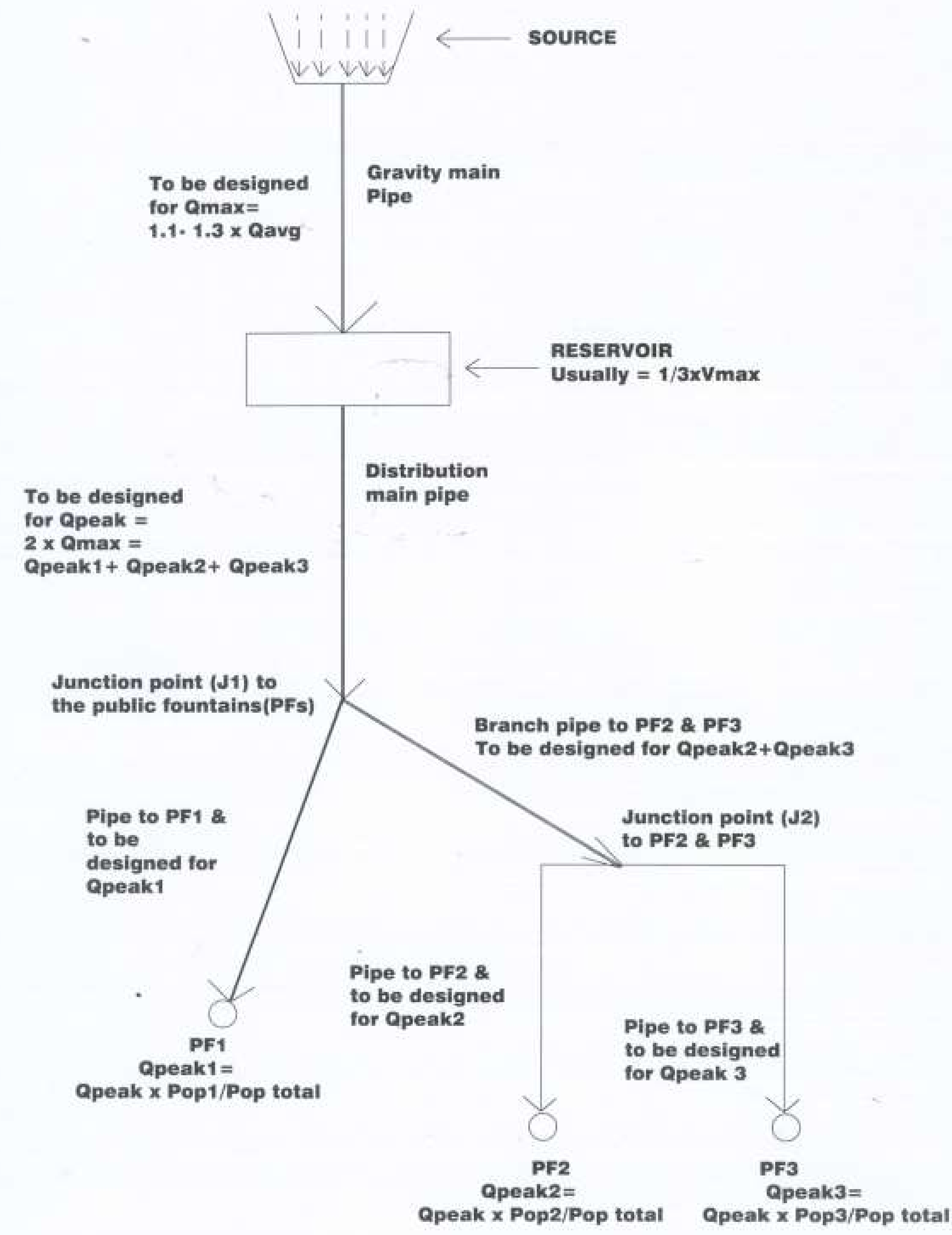


Figure 3.18 Sample flow distribution for design of different pipe componenets of gravity WS

Training Schedule for Design of Gravity Water Supply Schemes

Training Date: September 28 - October 1/2005

Days	Time in Ethiopian	Programme	Responsibility
28/9/05	**Day 1**		
	2:30 – 2:50	Registration	RWSEP
	2:50 – 3:00	Opening Speech	Mr. Arto
	3:00 – 3:10	Introduction of the programme and training objectives	Yohannes Melaku
	3:10 – 4:30	**INTRODUCTION TO GRAVITY FLOW WATER SYSTEMS**	Yohannes Melaku
	4:30 – 4:45	Tea break	RWSEP
	4:45 – 6:30	**ORGANIZATION OF A GRAVITY - FLOW WATER SYSTEMS** • Community request • Preliminary feasibility survey (pre feasibility study)	Tesfaye Yisagat
	6:30 – 8:00	Lunch break	
	8:00 – 9:30	**ORGANIZATION OF A GRAVITY - FLOW WATER SYSTEMS (Continued)** • Detailed community survey (detailed design) • Construction • Operation and maintenance • Evaluation	Tesfaye Yisagat
	9:30 – 9:45	Tea break	RWSEP
	9:45 – 11:30	**DESIGN PRINCIPLES AND PARAMETERS OF THE DIFFERENT COMPONENTS OF A GRAVITY FLOW WATER SYSTEM** • Locating an adequate and clean water source	Yohannes Melaku

Days	Time in Ethiopian	Programme	Responsibility
29/9/05	**Day 2**		
	2:45 – 4:30	• Design period, population growth and water demand • Reservoir and sedimentation tank	Yohannes Melaku
	4:30 – 4:45	Tea break	RWSEP
	4:45 – 6:30	• Design of pipelines	Tesfaye Yisagat
	6:30 – 8:00	Lunch Break	RWSEP
	8:00 – 9:30	• Design of pipelines (continued)	Tesfaye Yisagat
	9:30 –9 :45	Tea Break	RWSEP
	9:45 – 11:30	• Design of pipelines (Continued) • Break pressure tanks (BPT)	Tesfaye Yisagat
30/9/05	***Day 3***		
	2:45 – 4:30	• Public fountains, Air release valves & washouts, Valve boxes, Flow measuring device, Cattle trough and special component sections	Tesfaye Yisagat
	4:30 – 5:00	Tea Break	RWSEP
	5:00 – 6:30	**STEPS IN SURVEY AND DESIGN OF A GRAVITY- FLOW WATER SUPPLY SYSTEM** • The field survey • The design process	Yohannes Melaku
	6:30 – 8:00	Lunch Break	
	8:00 – 9:30	**WORKED EXAMPLES ON DESIGN OF GRAVITY - FLOW WATER SUPPLY SYSTEM**	Yohannes Melaku
	9:30 – 9:45	Tea Break	RWSEP
	9:00 – 11:30	**WORKED EXAMPLES ON DESIGN OF GRAVITY - FLOW WATER SUPPLY SYSTEM (Continued)**	Yohannes Melaku

Days	Time in Ethiopian	Programme	Responsibility
1/10/05	**Day 4**		
	2:45 – 4:30	**GUIDELINES FOR PREPARATION OF COST ESTIMATES CALCULATION** • Capital costs • Operation and maintenance costs	Yohannes Melaku
	4:30 – 4:45	Tea Break	
	4:45 – 6:30	**GUIDELINES FOR PREPARATION OF COST ESTIMATES CALCULATION (Continued)**	Yohannes Melaku
	6:30 – 8:00	Lunch Break	
	8:00 – 9:30	**PIPES AND PIPEWORKING**	Yohannes Melaku or Tesfaye Yisagat
	9:30 – 10:00	Tea break & closing of the workshop	RWSEP