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Gravity Flow Water Systems

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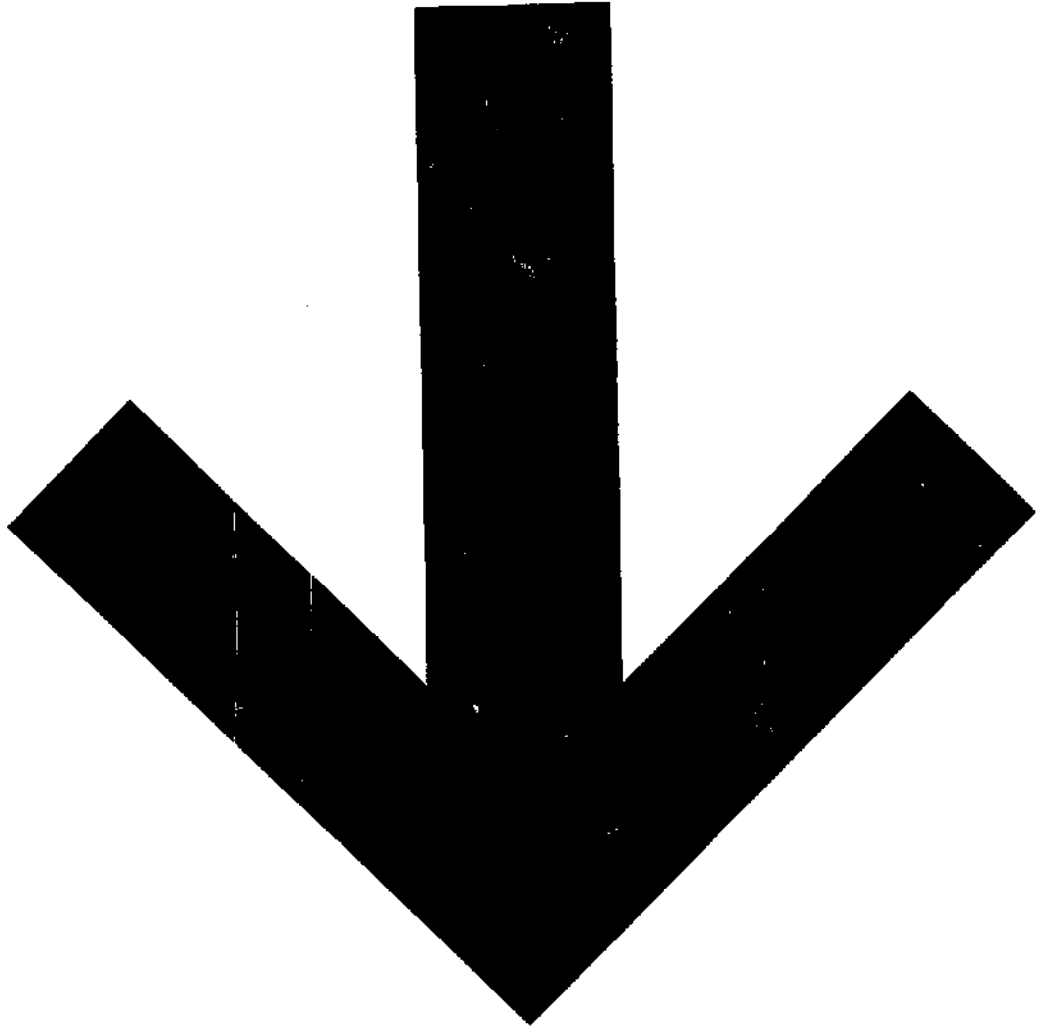
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# **GRAVITY FLOW Water Systems**

**PRACTICAL DESIGN NOTES  
FOR  
SIMPLE RURAL WATER SYSTEMS**

**A. SCOTT FAIIA  
Sanitary Engineer**

**1982**



## GRAVITY FLOW WATER SYSTEMS

### PRACTICAL DESIGN NOTES FOR SIMPLE RURAL WATER SYSTEMS

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## 1. INTRODUCTION

- 1.1. These design notes primarily cover simple gravity flow water systems. This is often the only feasible alternative for many rural areas at the present time and in many countries a major portion of the funds available for water systems are allocated to gravity flow systems. However, field inspections of completed projects and the literature indicate that there is a lack of capable design personnel and a substantial number of systems do not function properly due to poor design. Emphasis in the notes is placed on those aspects most neglected or misunderstood. Certain topics such as maintenance, community participation and health education are beyond the scope of this manual but must be given due consideration in constructing any type of water system.
  
- 1.2. These design notes present simple examples and explanations to illustrate some of the basic principles of gravity flow systems. Suggested guidelines \* for design parameters are also presented. The theory of water system design is extensively covered in other publications. The emphasis here is placed on practical methods that have been tested in the field and have given acceptable results. However, it must be emphasized that there are no set or standard solutions for the design of a water system. Attempts to implement water supply programs using a limited number of standard designs can be expected to produce poor results. Each community is unique and will require its own carefully prepared design by a person thoroughly familiar with local conditions.
  
- 1.3. The notes are based on several years experience in Indonesia and have been used for training of field staff responsible for site selection, design, and implementation. They are intended for use by persons responsible for planning, designing, and implementing, rural water systems. While a technical background or prior familiarity with water systems would obviously be useful it is intended that the material can be understood by persons without such a background. Since the guidelines are based on Indonesian conditions they should be applied with caution in other social, cultural and physical settings.

\* an asterisk denotes terms contained in the glossary in Appendix F. Such terms are so marked the first time they appear in the text.

1.4. The type of systems discussed are simple branched systems as in the example in Appendix E. Several dozen systems have been constructed using the design guidelines and they generally serve populations of 1,000 to 5,000 and are 2 to 8 kilometers in length. The largest system constructed using the suggested guidelines contained 59 distribution reservoirs of 6 m<sup>3</sup> capacity, seven break pressure tanks, and over 20 kilometers of distribution pipe. The flow in the system is approximately 15 l/s and this serves a 1982 population of approximately 12,000.

## 2. THE WATER SOURCE

### 2.1. General Considerations

2.1.1. The water source should be free of fecal contamination and must supply continuously a minimum amount of water to the system (purity and reliability). Both of these aspects are difficult to measure with accuracy. Quality and quantity will fluctuate around some mean value under natural conditions and and the variations from the mean tend to increase as the natural ecological balance is disturbed. A major change such as transformation from forest to agriculture, can be expected to significantly affect quantity and quality. Decreased yields and the total drying up of the springs are not uncommon.

2.1.2. The source also must be protectable and containable. In muddy areas and soft soil it is sometimes impossible to collect and protect the water. Areas of seepage rather than true springs are also difficult to deal with. Limestone areas must be thoroughly investigated because the spring could easily shift position altogether and the possibility of contamination is greater.

2.1.3. Another key consideration is the source's availability for use in the system. It is frequently not possible to use water previously used for irrigation, already allocated to other planned schemes or from sacred areas.

### 2.2. Estimation Of Quantity

2.2.1. Accurate measurement of quantity would require frequent monitoring over a period of several years and should preferably include a drier than average year. However, this information is generally not obtainable and an estimation must be made from point measurements\*. As many point measurements as possible should be taken and they should be during the driest part of the year. Experience and judgement are critical. Note should be taken of the condition of the catchment area, vegetation, land use etc. Comments of long-time local residents regarding reliability and change over time are also useful. These however, must be weighed carefully because people ~~always~~ tend to overestimate flows during dry periods and will sometimes falsely report that a source never dries up for fear of losing the project.

### 2.3. Estimation Of Quality

2.3.1. Quality generally fluctuates much less than quantity but it would also require monitoring over a long period to establish an accurate estimation. Point measurements should include periods of both high and low flows. In general, the greater the fluctuations the greater the amount of data that must be collected. The main considerations in assessing quality are bacterial quality, consumer acceptability and chemical quality.

2.3.2. Measurement of bacterial quality through testing for specific disease causing agents is not practicable. Instead an indicator organism is used to assess the likelihood that the water is contaminated by harmful pathogens. The most suitable indicator organism at present is fecal coliform\* as determined by the membrane filter technique\*. The majority of fecal coliform organisms are not harmful to man but their association with fecal matter indicates that organisms dangerous to health may be present. However, the link between presence of fecal coliform and contamination by fecal matter is not yet firmly established for tropical areas and further research is needed. The establishment of guidelines for bacterial quality is complicated by several other factors. Firstly, most sources under consideration are unprotected and the simple act of cleaning up the area and protecting the source may often result in dramatic improvements in quality. However, for political and social reasons, it is often impossible to carry out such improvements without making a commitment to complete the water system regardless of changes in quality. Secondly, a rigid standard\* may result in the rejection of a source which is superior in quality to existing sources. The replacement of a source containing several hundred thousand fecals per 100 ml by one with only 100 fecals per 100 ml can result in dramatic improvements in health. Thirdly, there is sufficient evidence to suggest that improving the quantity of water available without improving the quality will still result in significant health benefits.

2.3.4. In view of the above, only the most tentative bacterial quality guidelines can be proposed. Each situation must be judged on its <sup>own</sup> merits. The obvious goal is no fecal coliforms present in any sample taken from the proposed source. If some contamination exists then it is best if the average of all samples is less than 50 fecals/100 ml and if no single sample



exceeds 100 fecals/100 ml. For levels greater than these it is best to provide some form of treatment to improve quality if the source is to be used.

- 2.3.5. All water sources should also be monitored several times a year after source improvement and pipe installation. This is useful to document changes brought about by the improvements and to guard against contamination.
- 2.3.6. Consumer acceptability of the water is of prime importance. The best designed system in the world will be useless if the consumers do not accept and use the water. A simple visual inspection and sample testing of water along with a survey of the intended users will usually suffice to determine acceptability. It is very important to avoid excess iron in the water that will turn tea or rice black. If a source has not previously been used for drinking then the water should be boiled and tea made to check for any undesirable changes. Previously unused sources are sometimes also objectionable for cultural reasons.
- 2.3.7. Extensive chemical testing is both costly and time consuming and is usually omitted if the water is acceptable to the consumer. However, any unusual circumstances should be noted and guarded against. For example, the presence of excess carbon dioxide may not be noticed by consumers but it would rapidly corrode steel piping.

### 3. GENERAL SYSTEM DESIGN AND DESIGN PARAMETERS

#### 3.1. Type Of Service

3.1.1. The level of service provided in rural Indonesia is generally distribution at public taps for all types of domestic water use i.e. drinking, bathing, laundry, and toilet. Properly designed and maintained toilet facilities are not a health hazard. However, if there are serious doubts that the people will not use such facilities properly or are totally unfamiliar with them then it is better not to include them in the design. In such cases a simple facility strategically located would be in order.

3.1.2. The distribution points and facilities should be freely accessible to all intended users, and should be located according to population density. Their design and placement should facilitate and encourage water use thus maximizing project benefits. As general guidelines, no more than 10 to 20% of intended users should have to walk more than 100 meters to obtain water and the number of users per water faucet should be between 30 to 100. For example, a small system serving 1,000 people could have 6 distribution points (either standposts, reservoirs or public baths) each with four faucets or approximately 41 persons per faucet. The lower range in the design figure should be used for multiple purpose systems while the higher figures are acceptable for systems with restricted use, e.g. drinking water only.

#### 3.2. Water Usage - Design Figures

3.2.1. Multiple use systems should generally be designed for a per capita use of 60 liters/day. If sufficient water is available the figure could be 80 or 100 liters per day. If water is extremely scarce and was to be supplied for drinking purposes only the minimum acceptable figure would be 20 liters per person per day. These figures include an allowance for wastage.

#### 3.3. Storage Capacity

3.3.1. Storage of water is often necessary and will be influenced by the nature of the water source and the design of the system. For example if the flow of the water source is just 1 l/s (86 m<sup>3</sup>/day) and the average daily usage\* is also 86 m<sup>3</sup>/day then a certain amount of storage must be provided.

This is because the flow of water is constant throughout the 24 hour period while the usage is not. Therefore, during periods of low usage, such as during the night, part of the 86 m<sup>3</sup> will flow from the source and not be available for use unless it is stored. If the flow is high enough then the storage is not necessary provided that the pipe is large enough to provide sufficient water at the time of peak usage<sup>\*</sup>. However, even with a sufficiently large source it is preferable to provide some storage at the point of use. This aspect is discussed in more detail in paragraph 3.3.2. Figures 1A to 1D depict several possibilities for storage placement.

- 3.3.2. Fig 1A depicts the general schematic for placing storage at the point of use. In most cases this is the preferred type of design for the following reasons :
- a. The inflow to each reservoir can be regulated so that each area receives a set allotment of water. If the people at that reservoir tend to waste water then they can only waste their allotment but not that of others. In a stand-pipe system with storage at the source, wastage would be much greater if taps were inadvertently left open.
  - b. The small reservoir acts to break pressure in the system. This means that faucets at the point of use have only the head of the reservoir itself on them and will last for a longer, ~~period of time~~. Reduced head at the point of use also reduces wastage.
  - c. Storage at the point of use means that the main distribution pipe is in use at all times. It can therefore be of a smaller diameter and thus reduce costs. (Influence on total cost will depend on the flow of the source compared to average daily usage as this will influence storage costs).
  - d. It is generally easier to obtain community support and cultivate feelings of ownership and consequently improve maintenance through the construction of small scattered reservoirs as compared to one large distant reservoir and standpipes. Additionally, the construction of small reservoirs allows each segment of the community to work at its own pace during construction and a lack of community organization will be less likely to impede the project.

Figures 1A - 1D

STORAGE PLACEMENT

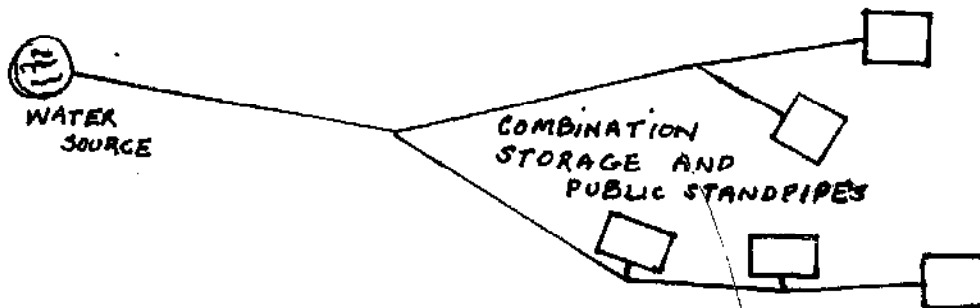


Fig. 1A - Storage at the point of water usage. This is the preferred schematic for placement of storage and requires the smallest pipe diameter. Its advantages are outlined in paragraph 3.3.2.

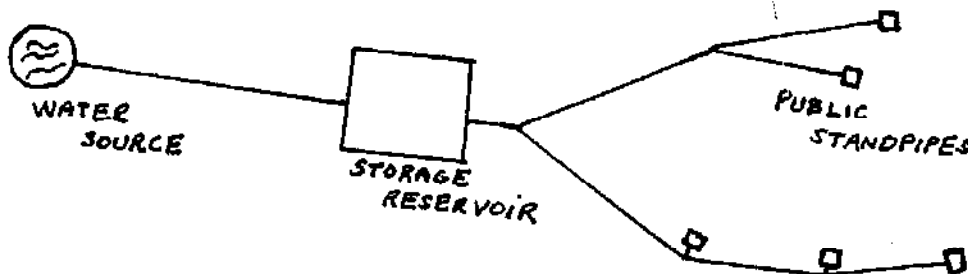


Fig. 1B - Storage far from source but still above distribution network, distribution from storage to public standpipes. The pipe line to the point of storage is the same as in Fig. 1A but after the point of storage is the same as Fig. 1C and 1D. This option should only be used when storage at the point of use is not possible.

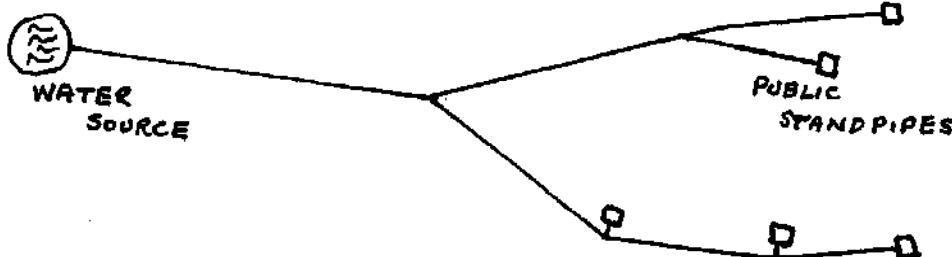


Fig. 1C - No storage provided, distribution direct to public standpipes. This type of system requires a larger diameter pipeline. The cost is sometimes less than that of the system in Fig. 1A but use of storage as in Fig. 1A is preferable for reasons outlined in paragraph 3.3.2.

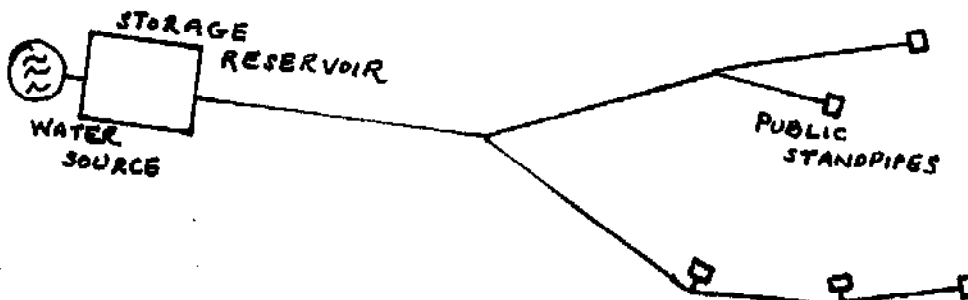


Fig. 1D - Storage provided at or near the water source then distribution direct to public standpipes. The pipeline diameter is the same as in Fig. 1C and this is the most expensive option.

e. In Indonesia the use of reservoirs can encourage water use and increase health benefits from the system. With a few additional walls the area can provide privacy for bathing and washing that is often explicitly requested by the community. The walls of the reservoir can also be used for displaying health education messages.

3.3.3. The following example of storage calculation is applicable when storage is to be provided at the point of use. (see Fig. 1A). Under normal circumstances the necessary storage is determined by comparing the supply curve with the consumption curve for the village. However, information on the consumption curve for Indonesian villages is not available and some other method is necessary. Based on recent experience in Indonesia it is recommended that the storage capacity be fixed at one half the average daily usage as determined by the population to be served and per capita consumption. For example, a system supplying 1,000 people with 60 liters/day would have an average daily usage of 60 m<sup>3</sup> and would thus require 30 m<sup>3</sup> storage capacity. If storage were provided at the point of use and there were 6 distribution points, as in the example in paragraph 3.1.2., it could be accomplished with 6 distribution reservoirs each 5 m<sup>3</sup> with four taps. This solution would be suitable if the population distribution were uniform. If it were not uniform then the reservoirs would be of different size and could have different numbers of faucets. For example, the main heavily populated area could be served by a 10 m<sup>3</sup> reservoir with 8 faucets. Three areas of medium density with reservoirs of 5 m<sup>3</sup> and 4 faucets and three areas of low density with reservoirs of 2,5 m<sup>3</sup> and two faucets. The important point is that the total storage capacity and number of faucets remains the same (or approximately the same) but their placement is according to field conditions. In practice the numbers do not divide so neatly and some adjustments are necessary. For example, if the design population were 961 the needed storage capacity would be 28,8 m<sup>3</sup>. This may be rounded up to 30 m<sup>3</sup> so that a standard design for tanks of 5 m<sup>3</sup> or such could be used. The use of standard designs for certain system components can be very convenient and economical. The design figures are rough approximations only, and can be modified if necessary

3.3.4. There are situations where the distribution system is fed from a central storage reservoir above the village. (See Figs. 1B and 1D)

This is recommended primarily when the population density is too great to allow for placement of the storage reservoirs within the village. Because of their smaller size, standpipes can be more easily placed at strategic locations. The storage capacity necessary for such a system will be determined by the ratio of the estimated minimum flow\* of the source to the average daily flow\* as determined by the design population and per capita usage. If the ratio is between one and two then storage should be equal to one half of the average daily usage. If the ratio is between two and three then the storage should be one quarter of average daily usage.. If the ratio is between three and four then the storage should be one eighth of average daily usage. If the ratio is greater than four then no storage is necessary (See Fig. 1C). If the ratio is less than one then the source doesn't have enough water to supply the design population with the projected per capita use. The above figures are rough approximations because detailed information on the demand curve for water is not generally available for rural areas in Indonesia.

3.3.5. For the example discussed in paragraph 3.3.3. (storage at the point of use) the average daily usage was 60m<sup>3</sup>/day or  $\sphericalangle$  0,7 l/s and the required storage 30 m<sup>3</sup>. If the storage were to be provided above the village and the estimated minimum flow of the source were 1,0 l/s then the required storage would be equal to half the average daily usage or 30 m<sup>3</sup> (ratio of source yield to average daily flow between one and two). If the estimated minimum flow of the source were greater than 2,8 l/s (ratio greater than four) then no storage would be necessary. Design of the distribution pipe for these cases is discussed in section 4.3.

3.3.6. The placement of storage facilities and distribution points requires detailed field surveys and extensive contact with the villagers. It can not be done from behind the drafting table in the office.

$\sphericalangle$  an average daily flow of

#### 4. PIPELINE DESIGN

##### 4.1. General

4.1.1. Sizing of pipe depends primarily on three main factors : design flows\*, length of pipe, and available head\*. The first is controlled by the designer while the second two are fixed for a given village and source. (The maximum design flow is usually limited to the estimated minimum flow of the source). Calculation of the head loss\* for the given flow and distance will indicate which size pipe is necessary to ensure that the desired flow will be obtained.

##### 4.2. Design Flows For Storage In Reservoirs At The Point Of Use :

4.2.1. The design flow for systems with storage at the point of use is the same as the average daily flow. In cases where there is sufficient water available this flow is determined according to the population served (plus expected increases for 10-15 years) and the per capita use. For example, a village has 1,000 inhabitants and village statistics indicate that population growth is 2% per year. The expected population in 15 years would then be 1,293 persons. If the percapita use is estimated at 80 liters per day then the average daily flow is 103.4 m<sup>3</sup> or 1.2 l/s (to convert m<sup>3</sup>/day to l/s divide by 86.4). This is the design flow that will be used to calculate head losses from the source to the first reservoir (first point of use). Flows used in calculating losses for subsequent sections of the pipeline will be reduced by the amount used at each distribution point. (See examples of head loss calculations in paragraph 4.4.5 and sample design in Appendix E.)

4.2.2. If no figure for population growth is available then 2% may be assumed. However, some villages in Indonesia have nearly reached the saturation point and population growth is very low due to migration out of the village. If this is the case then a figure of 1% or even zero should be used.

##### 4.3. Design Flows For Public Standpost System

4.3.1. When no storage is provided in the village then there is no water flowing in the pipe if the faucets are closed and the system not in use. In effect, the main distribution line is underutilized and it is best to avoid this situation. The design flows must be altered to account for this. If the taps are fully open 25% of the time (six hours per day) then the design flow would be 4 times the average daily flow and this is the recommended design figure.

For the example in paragraph 4.2.1., if there were no storage in the village, then the design flow would be  $4 \times 1.2$  l/s or 4.8 l/s. This obviously requires a larger diameter pipe and will increase the cost of the distribution lines. If the hypothetical case occurred where water use were very concentrated in time (say all use between 6 am to 9 am) then a design of this type would result in an actual per capita consumption lower than the design figure or in a number of users lower than the design figure. Again the recommended design figure is based on recent experience in Indonesia because adequate data on peak usage is not generally available for the area. The storage required in this situation will depend upon the flow of the source as discussed in paragraph 3.3.4.

#### 4.4. Calculation Of Head Losses

##### 4.4.1. General

Calculation of head losses is based on empirical formulas derived from experimental data. To avoid the necessity of repeated calculations values are taken directly from charts, tables or slide calculators. The circular waterflow calculator developed by M.H. Mear & Co in England is recommended because it is sufficiently accurate and convenient to use, particularly for quick estimations in the field. Appendices A and B will assist in understanding head loss calculations and should be reviewed by those readers without adequate background in fluid mechanics.

##### 4.4.2. General Method Of Calculation

Head losses are generally calculated for several pipe diameters using the design flow and measured length for that section of pipe. If there are any fittings or bends then 5% should be added to the calculated losses to account for this. A further 5% should be added in case there are any errors in measuring available head or distance. The calculated values for head losses are then compared to the available head on the measured profile. The smallest diameter pipe with a calculated head loss less than the measured head available is chosen for that section of pipe. Although this general principle is fairly simple the actual application of it is complicated by the withdrawal of water from the pipeline, the fixed diameters of available pipe, the possible occurrence of negative pressures, and the need to reduce pressure in the pipeline when there is excessive head. It is here that the experience, judgement, and ingenuity of the designer come into play in matching field conditions to available resources to produce the desired result in the most economical fashion. These aspects are further dis-



cussed below.

#### 4.4.3. Excessive Head

In designing the pipeline it is necessary to ensure that the pressure or head exerted on the pipeline and fittings does not exceed acceptable limits. This is accomplished by breaking the continuity in the pipeline at appropriate points. A 1 m<sup>3</sup> concrete break pressure tank divided into two compartments is normally used but in some cases a distribution reservoir can also serve this function. The maximum allowable head will depend on the type of the pipe used and the fittings involved. The following are recommended guidelines :

- a. For steel pipe with no valves in the lower end a maximum head of 200 meters is allowable although 100 meters is more reasonable. A cutoff valve or means of diverting the water is necessary at the top end to prevent flow in the pipe in case of repairs. Because there are no means to stop the flow at the lower end the full static head of 200 meters will never be attained.
- b. For steel pipe with valves and other fittings it is best to limit the maximum static head to 50 meters. That is if all valves are closed and water were stationary in the distribution pipe then the lowest point in each section should have no more than 50 meters head.
- c. For PVC the maximum allowable head is the manufacturers standard for which the pipe is guaranteed. If this exceeds 50 meters than care must be taken to avoid excessive pressure on any fittings in the line.

The above recommendations should be viewed as approximate and can be adjusted to conform with field conditions. For example, if the total head were 65 meters it would not be absolutely necessary to install a pressure release tank. Similarly the placement of tanks on a long line would not be at exactly 50 meters intervals of head but would depend upon the topography and the land available for the tanks.

There are also instances where conformance to the recommended guidelines is not possible or desirable. For example, if the pipeline descended steeply from the source 150 meters down into a valley and then rose 100 meters on the other side to reach the village the washout valve at the low point crossing the valley

would be under a head of 150 meters under static conditions. However, the use of pressure release tanks would probably mean that the water would not have enough head to reach the village. In such cases special effort should be made to use the best quality fittings available and to minimize the chances of attaining the full static head.

#### 4.4.4. The Hydraulic Grade Line (HGL)

The Hydraulic Grade Line (HGL) is defined by the difference between the head loss in the pipeline and the static head. The difference between the ground profile and the HGL is the pressure in the pipeline while the water is flowing. If an opening were made in the pipeline and a tube connected to it then the water would rise to the level of the HGL. The HGL should always lie above the profile. If it does not, then the water may still flow but at sections where the profile lies above the gradient there is a negative pressure which can cause air or pollution to enter the pipeline. Those sections of the pipeline where negative pressures occur should be redesigned to eliminate them. Figures 2A and 2B illustrate this.

#### 4.4.5. Pipeline Design Examples

Once the approximate location of the break pressure tanks has been decided upon then calculations to determine pipe size can be carried out for each continuous section of the pipeline (where no breaks in pressure occur). It is important to look at the total section involved and not just the sections between withdrawals where changes in flow occur.

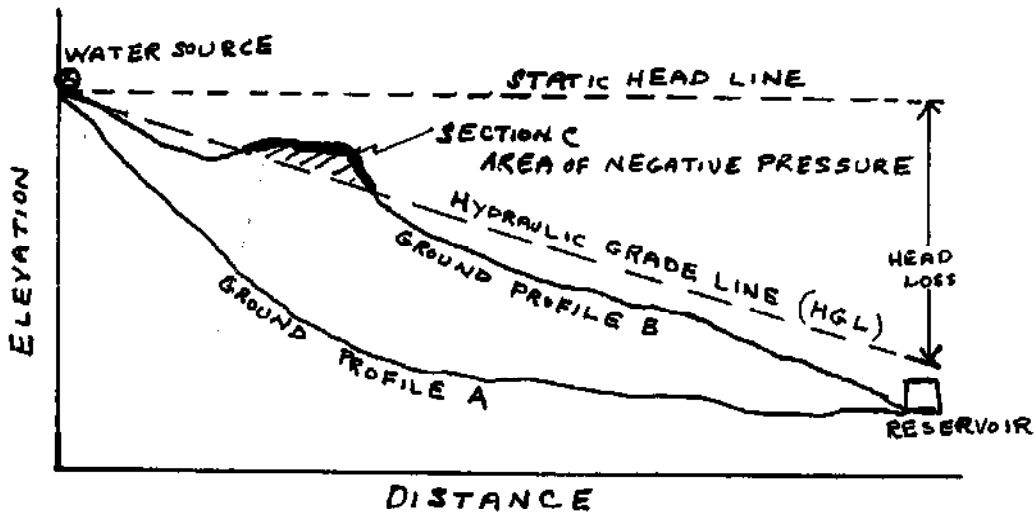
##### Example 1

A spring with a flow of 0.5 l/s is 1,000 meters from the village and the difference in elevation (available head) is 20 meters. It is planned to convey the entire flow to a small reservoir in the village. What size pipe is recommended? On the water flow calculator (or similar charts) a flow of 0.5 l/s and a length 1,000 meters using 1.5 inch pipe indicates a head loss of 11 meters. For 1.25 inch pipe the head loss is 26 meters. Thus the required flow will not be obtained with a 1.25 inch pipe and a 1.5 inch pipe is too large. The most economical solution is a combination of two pipe sizes. By trial and error it is found that 500 meters of 1.5 inch GI pipe with a flow of 0.5 l/s has a head loss of 6 meters and 500 meters of 1.25 inch GI pipe has a head loss of 13 meters.

Figure 2

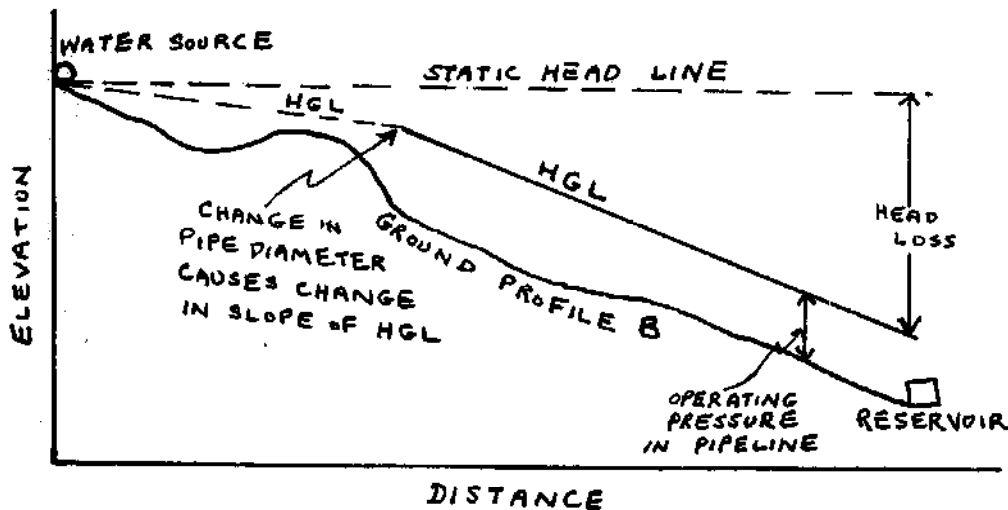
CHECKING OF THE HYDRAULIC GRADE LINE

Figure 2A



If the pipeline followed ground profile A then the choice of pipe with the given HGL is acceptable. If the pipeline followed ground profile B then negative pressure would exist in section C so the pipeline should be re-designed. See Fig. 2B

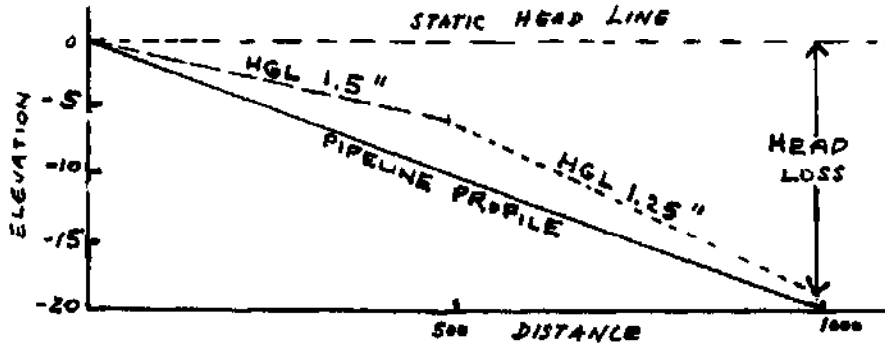
Figure 2B



The pipe diameters have been changed thus changing the HGL. Use of a larger diameter pipe near the source ensures that the HGL lies entirely above the ground profile and is acceptable. Note that two pipe diameters are now used between the source and reservoir. For each diameter the HGL has a different slope. The slope is directly dependent on the head loss so a smaller pipe has a steeper slope.

Thus the total head loss for the 1,000 meter pipeline is 19 meters which closely matches the available head. The pipeline profile and HGL are plotted in Fig. 3.

Figure 3  
(Example 1)



Example 2

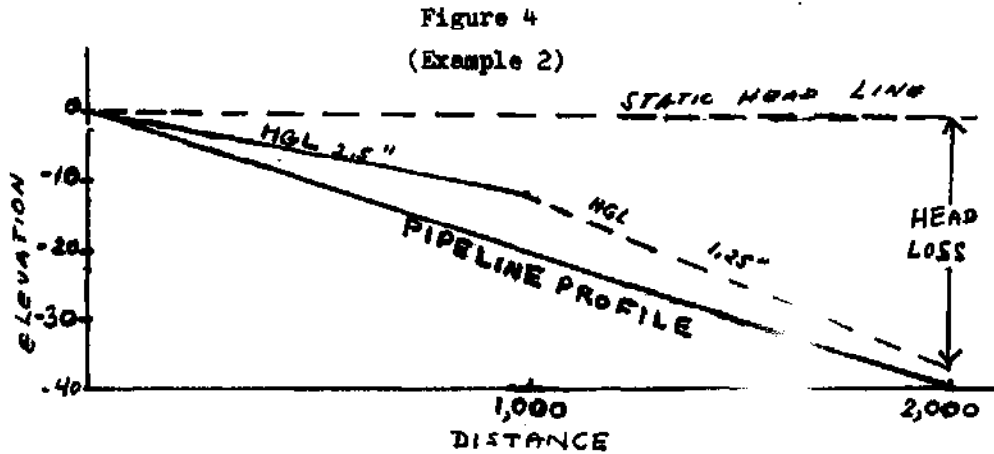
A water source is 1,000 meters from Kampong Kering and it is 1,000 meters further to Kampong Kering Sekali. The difference in elevation between the source and Kering is 20 meters and between Kering and Kering Sekali it is also 20 meters. The design flows are 2.0 l/s from the source to Kering and 0.5 l/s from Kering to Kering Sekali. What are suitable pipe diameters ?

Calculated Head Losses for various pipes are as follows :

length (meters)	flow (l/s)	pipe diameter (inches)	head loss (meters)	Available head (meters)
1,000	2	3	5	20
1,000	2	2.5	11	20
1,000	2	2	33	20
1,000	0.5	2	3	20
1,000	0.5	1.5	11	20
1,000	0.5	1.25	26	20

A suitable selection of pipe would be 2.5 inch pipe for the first 1,000 meters and 1.25 inch pipe for the second 1,000 meters. The total head loss is then 37 meters which closely matches the total available head of 40 meters. Note that the second 1,000 meters has a head loss of 26 meters and an available head of only 20 meters.

This is allowable because there is excess head available from the first 1,000 meters of the pipeline and the HGL is always above the pipeline profile. See Fig. 4.



Example 3

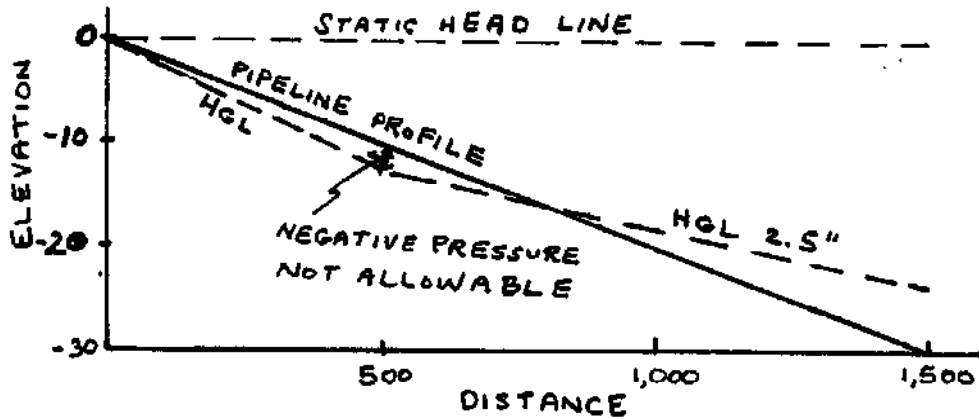
A spring with a flow of 3 l/s is 500 meters from the bathing area and the mosque is 1,000 meters further. A flow of 1 l/s will be used to serve the bathing area and 2.0 l/s for the mosque. The difference in elevation is 10 meters between the spring and bathing area and 20 meters between the bathing area and the mosque. What pipe sizes are recommended ?

Some calculated head losses are as follows :

length (meters)	flow (l/s)	pipe diameter (inches)	head loss (meters)	Available head (meters)
500	3	3	5	10
500	3	2.5	12	10
500	3	2	36	10
1,000	2	3	5	20
1,000	2	2.5	11	20
1,000	2	2	33	20
500	2	2.5	6	-
500	2	2	16	-

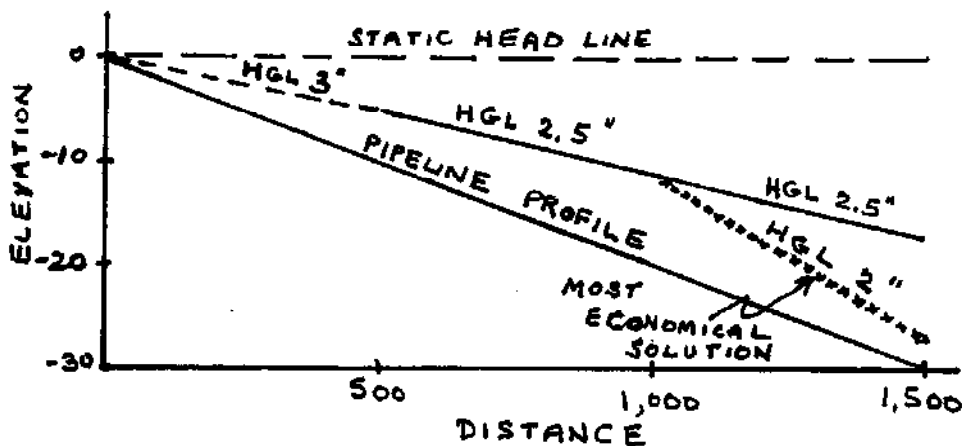
If 2.5 inches pipe is used for the entire 1,500 meters the total head loss is 23 meters which is less than the total available head of 30 meters. Thus the desired amount of water of may flow. However, the HGL as plotted in Fig. 5 falls below the pipeline profile and this is not allowable.

Figure 5  
(Example 3)



An acceptable alternative solution would be to use 500 meters of 3 inch pipe followed by 1,000 meters of 2.5 inch pipe. The total head loss would then be 16 meters and excess available head would be 14 meters. A more economical solution would be 500 meters each of 3, 2.5, and 2 inch pipe which would convey the full design flows. The HGL for these two solutions are plotted in Fig. 6

Figure 6  
(Example 3)



## 5. SUMMARY OF SUGGESTED DESIGN GUIDELINES

- 5.1. Bacterial Quality : No fecal coliforms in any sample from the proposed source. If this is not possible, then an average of less than 50 fecals/100 ml for all samples with no single sample exceeding 100 fecals/100 ml. For levels above this, simple treatment is advisable such as slow sand filtration.
  
- 5.2. Distance to Distribution Point : No more than 20% of intended users to walk more than 100 meters to obtain water.
  
- 5.3. Number of Users per Water Faucet : Between 30 and 100, lower range for multiple use and upper range for drinking water use only.
  
- 5.4. Per Capita Usage : Multiple use systems at a minimum of 60 l/day with up to 100 l/day preferred if sufficient water is available. Minimum of 20 l/day for supply of drinking water only.
  
- 5.5. Storage Capacity : For storage at the point of use one half of the average daily usage (ADU). For storage above the point of use (standpipes only) dependent upon the ratio of source yield to average daily flow. For a ratio between one and two then storage one half of ADU; for a ratio between 2 and 3 then storage one eighth of ADU; for a ratio greater than four then no storage required.
  
- 5.6. Design Flows : If storage is at the point of use; use the average daily flow. If storage is above the point of use (standpipes only) then use four times the average daily flow.

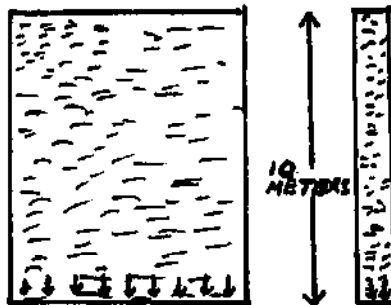
A P P E N D I X A

GENERAL EXPLANATION FOR FLOW AND HEAD LOSSES IN CLOSED PIPES

1. Pressure Exerted by a Column of Water

A column of water exerts a force due to the weight of the water. The pressure, or force per unit area, is dependent on the height of the column of water. Therefore, head or water pressure is usually expressed in terms of the equivalent height of water needed to exert that pressure. The pressure under static conditions is not dependent on pipe size. See Fig. A 1.

Figure A 1

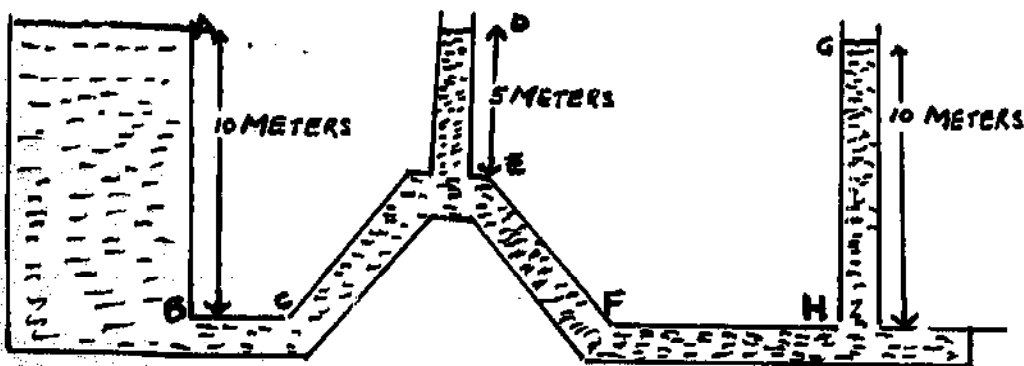


The pressure at the bottom of each column of water is the same. It is 10 meters of head or 10 kg/cm<sup>2</sup>. The pressure midway in each column would be 5 meters of head or 5 kg/cm<sup>2</sup>.

2. Pressure in a Static System

In a system under static conditions the pressure at any point is dependent on the difference in height between the point in question and the highest point in the system. If an opening is made in the pipe in any part of the system and a tube connected to it then the water level will rise until it is the same as the highest point. See Fig. A 2.

Figure A 2





The system is static and no flow occurs.

The pressure or head at points B,C,F, and H is the same, i.e. 10 meters. The pressure or head at point E is 5 meters of the difference in height between points A and E. If the pipeline were opened and a tube connected to it at point C or F then the water would rise 10 meters and be at the same level as points A,D, and G.

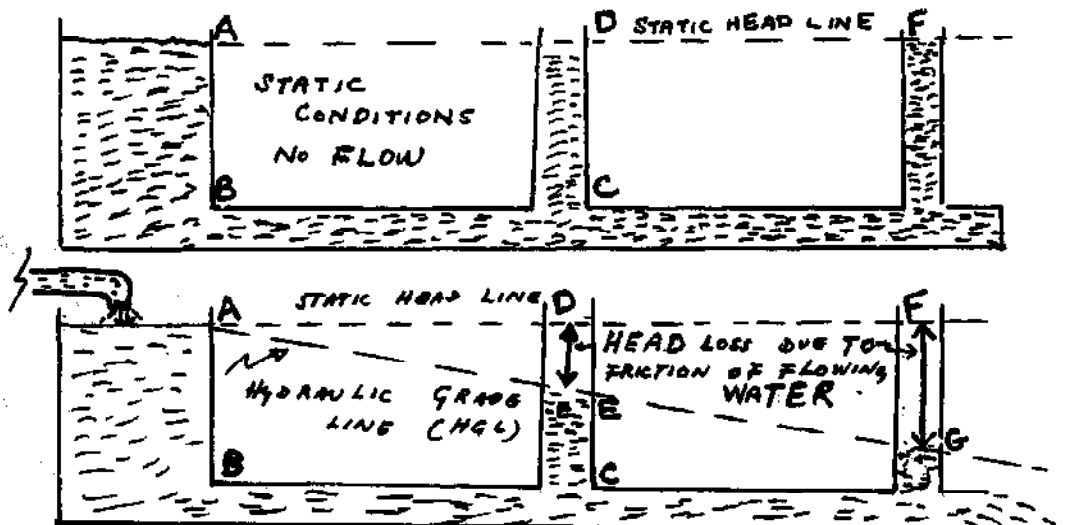
Fig. A2

### 3. Pressure in a Flowing System

When water in the pipeline is flowing, then the pressure is no longer dependent solely on the height difference with respect to the highest point. There is a loss of pressure or head due to friction between the water and the pipe. The pressure or head at any point is equal to the static head\* (relative height difference) minus the head loss\* due to friction. Because of the head loss the water will not rise to the same level as the highest point but only as high as the pressure or head at that point. Head loss occurs only when water is flowing.

See Fig. A 3

Figure A 3



Under flowing conditions the pressure is no longer the same and the pressure at point C or G is not sufficient to raise the water level to points D or F. The height difference between points D and E or points F and G is the head loss due to friction in the pipeline. If the flow were stopped the water level would return to points D and F.

#### 4. Factors Influencing Head Losses

The amount of head loss is influenced by the following factors :

a. The length of pipe.

The longer the pipeline the greater the head loss. This loss is directly proportional to the length i.e. the head loss for 200 meters of pipe would be twice that for 100 meters under the same conditions.

b. The diameter of the pipe.

The smaller the diameter of the pipeline then the greater the friction will be for the same flow of water. The differences are not proportional.

c. The flow of water in the pipe.

The higher the flow of water in a given pipe the greater the head loss due to friction. Friction increases as the square of the velocity.

d. The pipe material.

The smoother the inner surface of the pipe the lower the head loss. Thus, since PVC pipe is smoother than steel or cast iron it has a lower head loss for identical conditions.

e. The number of fittings or bends in the pipeline.

A straight pipeline would have a lower head loss than one of the same length with fittings or bends.

Some of the practical consequences of the above in designing a gravity flow system are :

a. Increasing pipe diameter between any two points will produce a larger flow of water between those two points.

b. Adding many bends and fittings reduces the flow.

#### 5. Pipe Design

In designing a gravity flow pipeline the available head  $\angle$  obtained from field measurements. From the number of people to be served and the projected per capita consumption the desired flow of water is calculated.

A pipe size is then chosen with a head loss less than the available head at the desired flow of water and pipe length. The head loss is taken from charts and tables. (see Section 4.4. and Appendix B)

$\angle$  and distances:

6. Factors Affecting the Hydraulic Grade Line (HGL)

In Figure A 3 the level of the water surface defines the hydraulic grade line (HGL). The slope of this line is constant for a given pipe diameter and given flow and is directly related to the head loss. Changing the pipe diameter of the flow will change the slope of the hydraulic grade line as in Fig. A 4. Changing the amount of water flowing in the system also affects the HGL as in Fig. A 5. The HGL is compared with the ground profile to determine if the pressure in a pipeline is adequate. (See paragraph 4.4.4.)

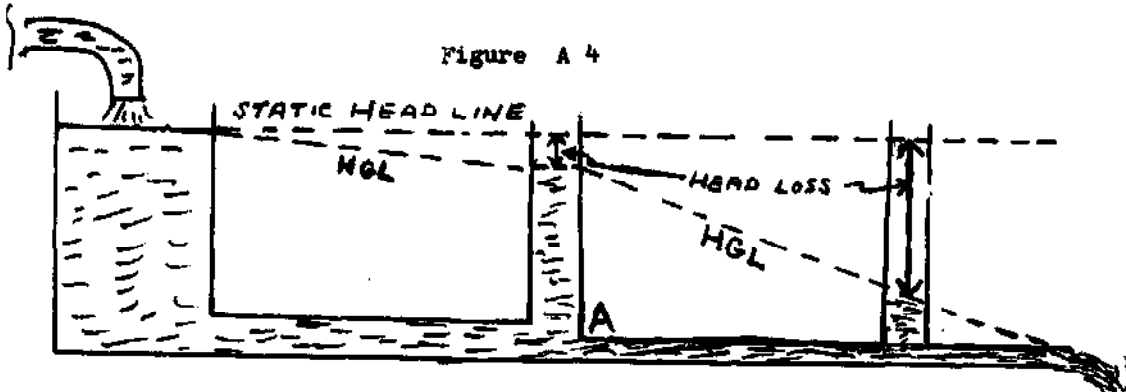


Figure A 4

After point A the pipe diameter is reduced and the slope of the HGL increases due to an increased head loss for the smaller diameter pipe. The flow of water is the same in both pipe diameters.

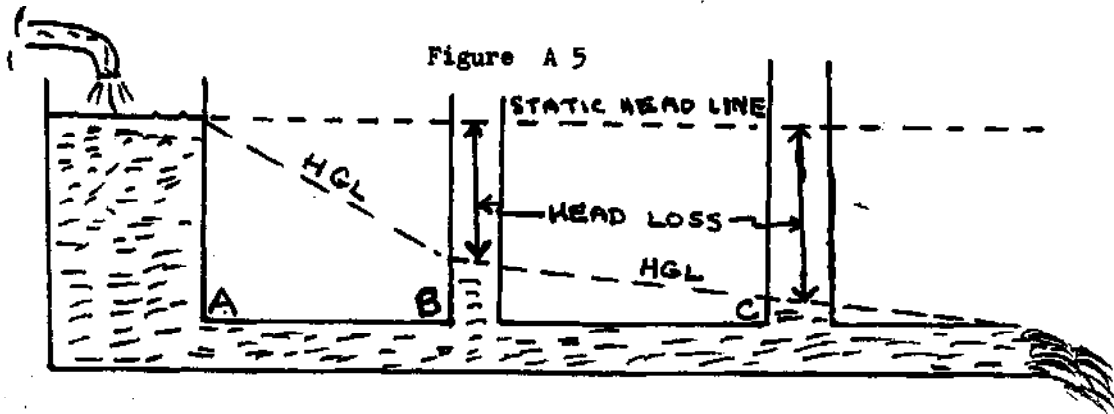


Figure A 5

At point B a portion of the flow is withdrawn from the system. The flow between points B and C is less than the flow between points A and B. The pipe diameter remains the same so the slope of the HGL is reduced because of the reduced head loss,

A P P E N D I X B  
THE MEAR WATERFLOW CALCULATOR

The Mear waterflow calculator is based on the Colebrook-White formula for flow in pipes. It provides solutions to the equation for a wide range of conditions without the necessity for cumbersome calculations.

In using the calculators there are 5 variables, they are :

- a. H = the head loss in millibars; 98 millibars is equivalent to 1 meter of head. In practice when reading the calculator 100 millibars can be considered as equivalent to 1 meter head.
- b. F = Coefficient of friction. This is related to the pipe material and on the calculator the most common types of pipe are listed.
- c. L = The length of pipe which on the calculator ranges from 1,5 to 150,000 meters.
- d. D = the bore of pipe (pipe diameter) which on the calculator ranges from 16 to 6,000 mm
- e.  $Q^*$  = the flow which on the calculator ranges from 1 l/s to 1000 m<sup>3</sup>/s

If any four of the above variable are known or fixed then the value of the fifth variable can be found on the calculator.

Example 1

A source is 2 kilometers from the village and the difference in elevation is 50 meters. What will the flow of water be if a 2" PVC pipe is installed? Compare this flow with the flow in a 2" GI pipe.

- a. Set the bore of pipe 51 mm opposite 2,000 meters.
- b. Set the plastic pipe arrow opposite the pressure loss of 5 bars (50 meters)
- c. Read the flow for PVC pipe opposite the flow arrow : 2.3 l/s
- d. Keep the bore of pipe 51 mm opposite 2,000 meters.
- e. Set the GI pipe arrow opposite the pressure loss of 5 bars
- f. Read the flow for GI pipe opposite the flow arrow : 1.8 l/s

Example 2

A water source is 1,500 meters from the main reservoir in a village and it is necessary to convey a flow of 1 l/s to it. Galvanized iron pipe must be used. The difference in elevation is 35 meters.

What size pipe is necessary ?

- a. Set the flow arrow at 1 l/s
- b. Set the arrow for galvanized iron pipe at 3.5 bars (35 meters)
- c. Read the bore of pipe opposite 1,500 meters. It is 41.5 mm or 1.63 inches.

There is no pipe available with a diameter of 1.63 inches. If we use 1.5 inch pipe the flow will be less than 1 l/s if we use 2" inch pipe the flow will be more. Check as follows :

- a. Set the bore of pipe 51 mm (2 inches) opposite 1,500 meters.
- b. Set the arrow for galvanized iron pipe at 3.5 bars
- c. Read opposite the flow arrow 1.75 l/s
- d. Set the bore of pipe 38 mm (1.5 inches) opposite 1,500 meters
- e. Set the arrow for galvanized iron pipe at 3.5 bars
- f. Read opposite the flow arrow 0.8 l/s

Since 1.5 inches pipe is too small and 2" pipe is too large a combination of the two can be used. Try 750 meters 2" pipe and 750 meters 1.5" pipe :

- a. Set the bore of pipe of pipe 51 mm 2" opposite 750 meters
- b. Set the flow arrow opposite 1.0 l/s
- c. Read the pressure loss opposite the galvanized iron arrow 650 millibars or 6.5 meters.
- d. Set the bore of pipe 38 mm 1.5" opposite 750 meters
- e. Set the flow arrow opposite 1.0 l/s
- f. Read the pressure loss opposite the galvanized iron arrow at 27 bars or 27 meters.

The total head loss for the 1,500 meters then equals 27 + 6.5 or 33.5 meters. This more closely matches the available head of 35 meters.

The Mear Water Flow Calculator is available from :

M.H. Mear and Co. Ltd.  
56 Nettleton Road,  
Dalton, Huddersfield  
England.

A P P E D I X C

CONSTRUCTION NOTES

1. CAPTERING (SPRING PROTECTION)NOTES

- 1.1. The construction of a captering will depend on the physical characteristics of the spring and surrounding area. Several possible situations are depicted in figures C1, C2, and C3. In addition the considerations in paragraph 1.2 - 1.16 below must be thoroughly understood and applied.
- 1.2. A captering has two primary functions : it collects and conveys the water for distribution and it protects the water from contamination. It normally does not provide storage although this is sometimes possible. In many instances it is not feasible to excavate near the source to provide storage capacity. A common error that occurs when a combination reservoir/captering is constructed is that most of the storage is above the natural water level of the source. This can lead to problems. Therefore, it is better to convey the water from the source (protected area) to a convenient location close by where any necessary storage can be provided and the water supplied to the distribution system.
- 1.3. It is best if the captering and source can be completely covered with a backfill or more than 1 meter of clay or other impermeable material. A manhole may be constructed to allow access to the source but it is not required. Several variations are shown in figures C1, C2, and C3.
- 1.4. In installing the foundation it is important to dig down to, but not through, the impermeable base layer. The water flow should be horizontal rather than upward through a permeable layer. The water level should never be allowed to rise above this level.
- 1.5. The water level should not be allowed to rise above the original natural outflow of the spring. This level should be carefully recorded with some respect to some permanent nearby object. Any overflow or withdrawal must be below this level. Only upon the recommendation of an experienced hydrogeologist should a higher withdrawal or overflow be installed. The danger is that allowing the water level to rise will create back pressure that may result in a reduction of yield or loss of the spring. Dozens of water systems in Indonesia do not function properly due to problems created by back pressure.

FIGURE C 1  
COLLECTING CHAMBER  
BELOW SOURCE

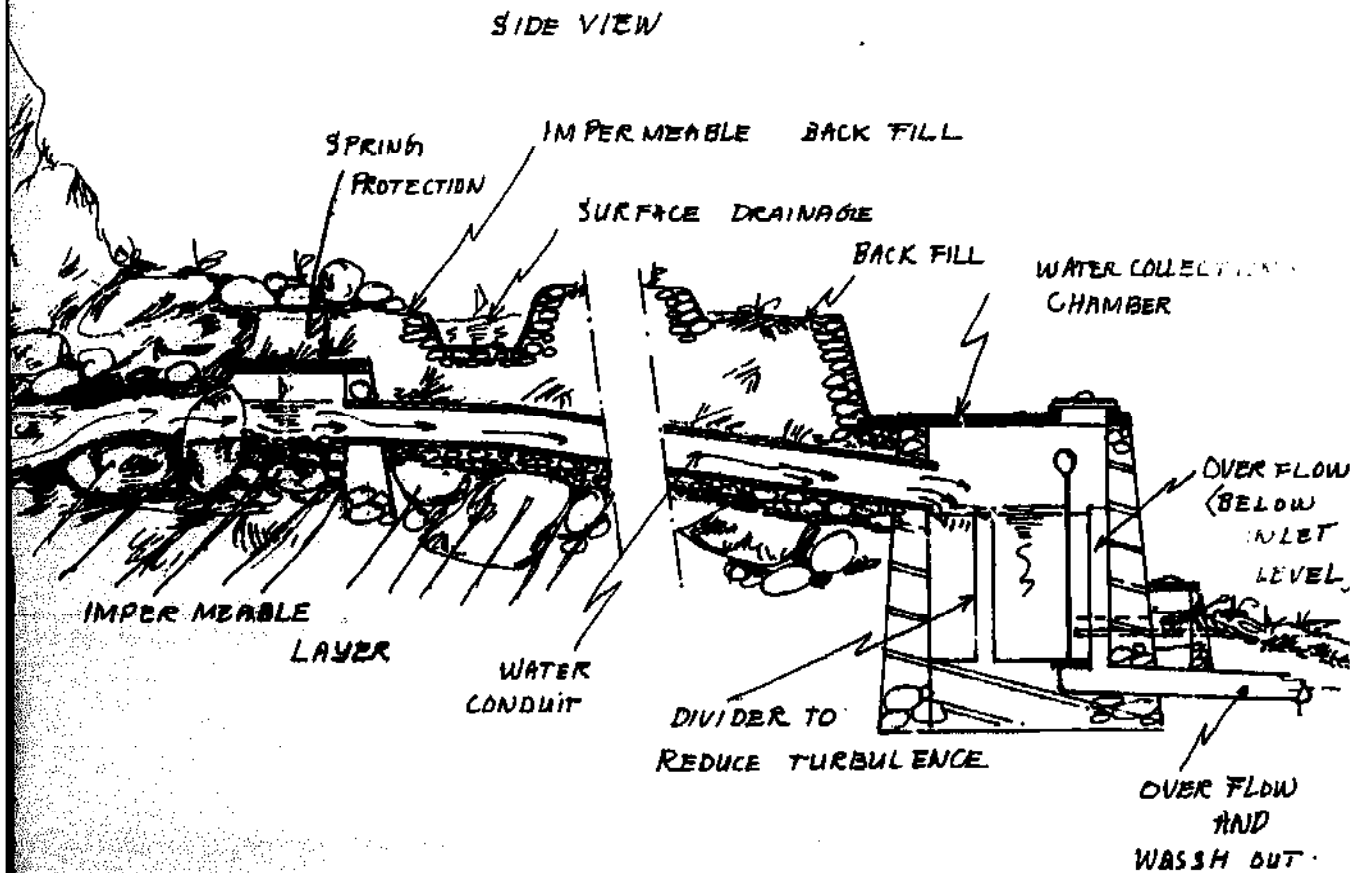
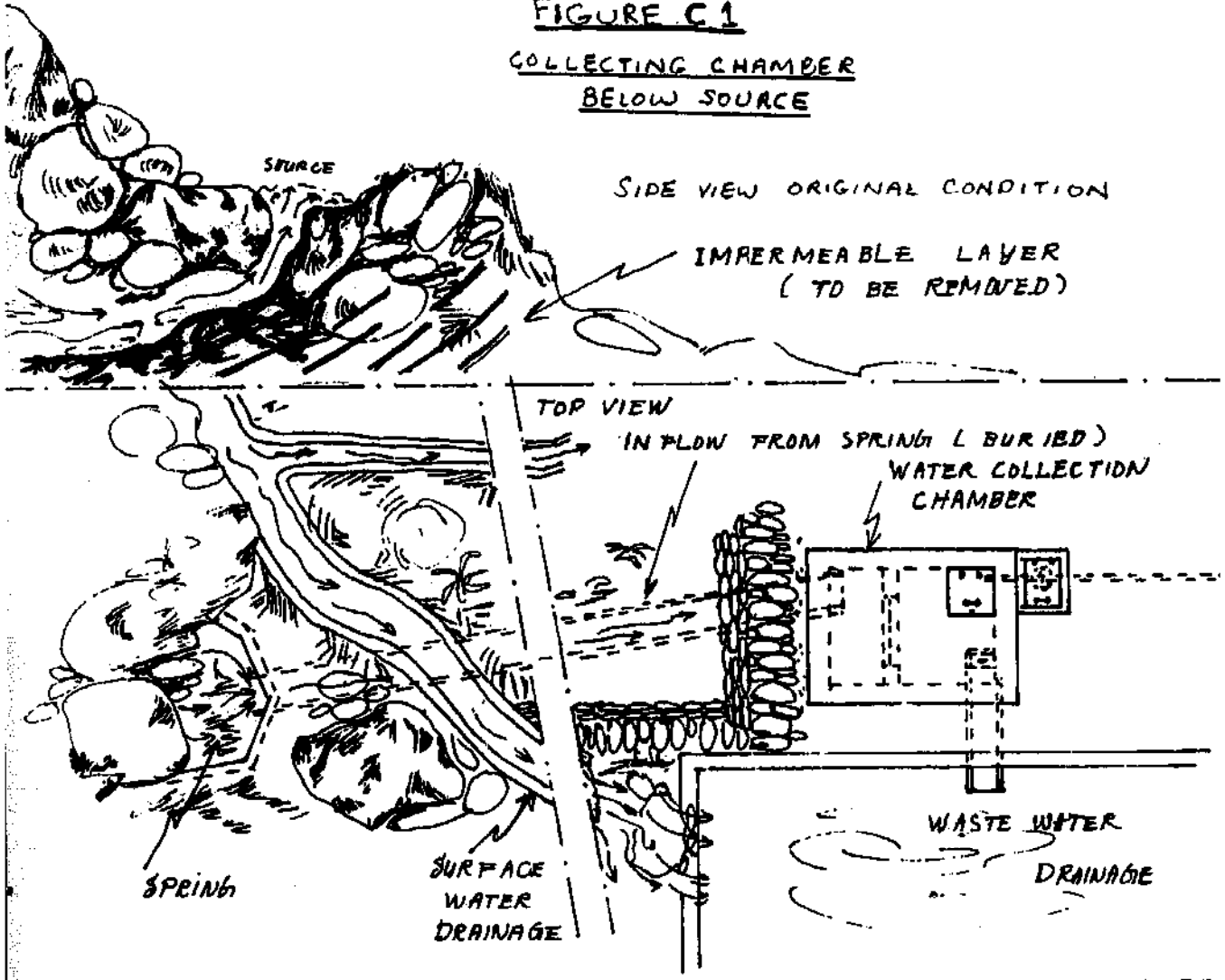
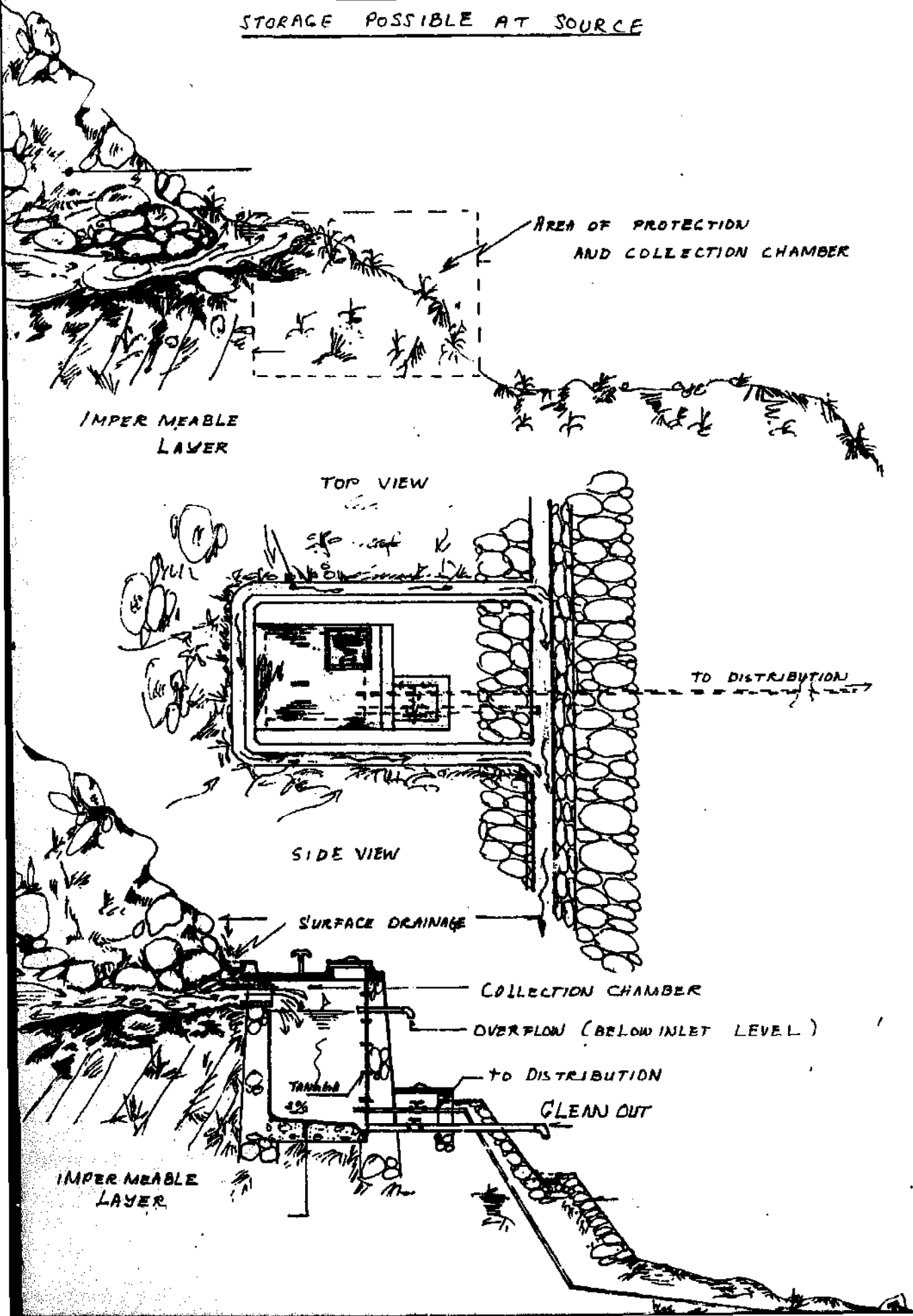


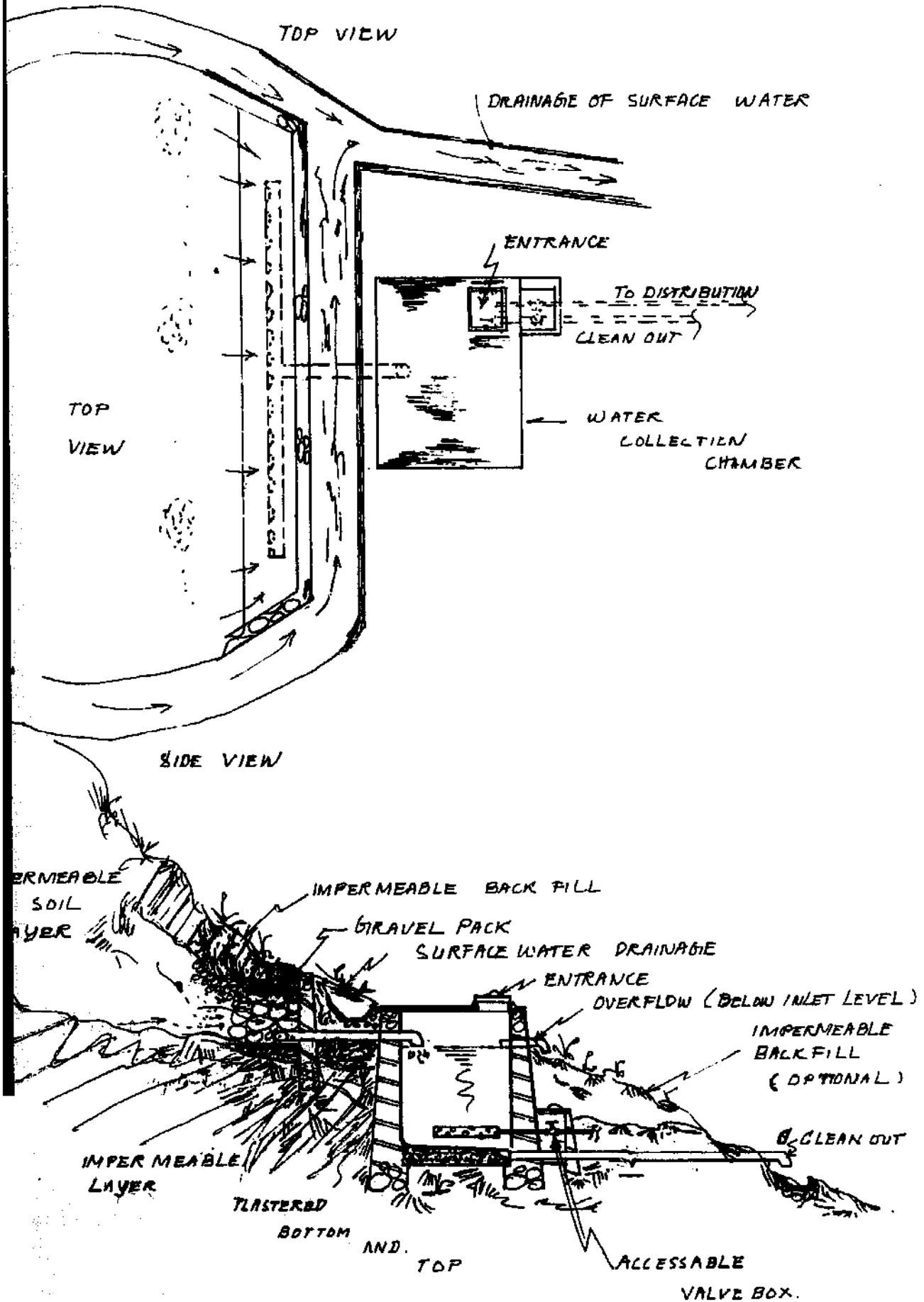
FIGURE C.2  
STORAGE POSSIBLE AT SOURCE





# FIGURE C 3

## SEEPAGE OVER A WIDE AREA



- 1.6. The foundation, walls, covering, and all joints should be watertight.
- 1.7. There should be no trees or vegetation whose roots may eventually enter and disturb the source protection. However, any existing large trees should be left undisturbed so as not to disturb the source.
- 1.8. The outflow conduit from the capturing should be large enough and of sufficient slope to accommodate the maximum anticipated flow.
- 1.9. Arrangement should be made for diversion and drainage of any surface water.
- 1.10. If it is possible to stop the flow of water in the collecting pipe then there must be provision for an overflow of sufficient size.
- 1.11. If there is an overflow pipe then it must be of sufficient diameter and it must have a strainer. The open area of the strainer should be 2-3 times the cross sectional area of the overflow pipe. Openings in the strainer should be small enough to prevent entry of small animals. The offtake in a water chamber or reservoir should also have a strainer.
- 1.12. If there is an overflow then there must be suitable arrangements for drainage of any excess water.
- 1.13. If water from the capturing is channelled to a water chamber or storage reservoir then the overflow of the chamber/reservoir must be at a lower level than the inflow from the capturing.
- 1.14. The date of construction should be engraved somewhere on the structure and any buried structures should be identified by a small concrete marker.
- 1.15. In so far as possible the source should be protected from any disturbance. Fencing is recommended for a distance of 10 meters in the front of, 10 meters on either side, and 50 meters to the rear of the source. Protection and/or reforestation of the catchment area should be strongly encouraged.
- 1.16. If possible, arrangements should be made so that the yield of the source can be easily measured.

## 2. RESERVOIR CONSTRUCTION

- 2.1. Reservoirs provide storage to offset fluctuating demand and low source flows. They may improve water quality through sedimentation if they allow for some retention time. They can also serve to break pressure in the system. Important considerations in their construction are listed below.
- 2.2. There should be a clean out pipe in each compartment of the reservoir. The cleanout should be placed at the lowest point, and the floor of the reservoir should slope towards it. It is not necessary to have an expensive valve to control this. A simple plug is sufficient.
- 2.3. Joinings of the wall and floor should be slightly rounded to facilitate cleaning.
- 2.4. Any offtakes should be 5 to 20 centimeters above the reservoir floor. The larger the reservoir, the higher the level. The level of the offtakes with respect to the ground will depend on the usage of the reservoir.
- 2.5. The level of the inlet pipe should be approximately 10 to 20 cm below the cover of the reservoir. In smaller reservoirs (generally those with an inflow pipe of 1" diameter or less) the inlet should be fitted with a float valve so that the flow is stopped when the reservoir is full. The float should be placed so that the flow is stopped before the water level reaches the level of the inlet pipe. If the float valve is not installed then an elbow should be used to direct the flow downwards. In larger reservoirs an elbow or baffle are required to reduce turbulence.
- 2.6. The inlet pipe should be fitted with a globe valve to control the inflow. The valve should be protected from possible tampering.
- 2.7. The overflow should be placed at a slightly higher level than the inflow so that it will come into play only if the float valve malfunctions. The overflow also serves as an air ventilation pipe.
- 2.8. The overflow pipe diameter should be large enough to accommodate the maximum expected flow. It is normally larger than the inflow pipe because the inflow is under pressure. For example, an inflow pipe of ¾" diameter might require an overflow of 1" diameter.

- 2.9. The overflow and offtakes to serve the distribution pipe should be screened. The open area of the screen should be 2 to 3 times the cross sectional area of the pipe to which it is attached. For example, a 1½" pipe has a cross sectional area of 1.23 inches<sup>2</sup>. If the screen is made by drilling holes in PVC with a ¼" bit than each hole is approximately 0.5 inches<sup>2</sup> and at least 50 to 75 holes should be drilled.
- 2.10. There should be a manhole on the reservoir which is just large enough for a man to enter. The design of the cover should be such that rain-water cannot enter the reservoir. A concrete manhole cover with a removable iron handle is recommended.
- 2.11. The float valve and overflow pipe should be close enough to the manhole so that they may be inspected and serviced without having to enter the reservoir. They should not interfere with entering the reservoir.
- 2.12. The top of the reservoir should be sloped so that rainwater does not collect on it.
- 2.13. There should be provision for proper drainage of any wastewater or overflow. The slope should be at least 2% and it should be directed to a soakage pit, pond or flowing ditch. An open, concrete lined, drain 10 x 10 cm is preferable. Proper drainage of wastewater is unfortunately totally neglected in practice even though it is critical to achieving desired health benefits. In fact, poor drainage may create more health problems than the provision of clean water alleviates.
- 2.14. Reservoirs larger than 80 m<sup>3</sup> should be divided into two compartments and provision made so that the inflow can be directed to either compartment. There should be a separate offtake for each compartment. Thus each compartment can be separately cleaned without interrupting the flow in the system.
- 2.15. Reservoirs greater than about 40 m<sup>3</sup> should have built in iron rung steps on the outside to facilitate access to the top.
- 2.16. Any exposed iron or pipes should be painted.
- 2.17. The date of construction should be engraved somewhere on the concrete for future reference.
- 2.18. In calculating reservoir volume the effective depth is from the offtake to the highest level the water can reach.

### 3. PIPELINE

- 3.1. The choice of pipe material depends on physical, technical, and cultural factors. Although it is presently more expensive galvanized iron is preferred for use in Indonesia. This is primarily to prevent unauthorized tapping into the system. It is also frequently not possible to bury the pipe and iron pipe must be used in these areas.
- 3.2. The pipeline must have air release valves at all high points. Any air present in the water will tend to collect at high points and block or reduce the flow of water.
- 3.3. The pipeline must have washout valves at all low points.
- 3.4. Union joints in galvanized iron pipe should be installed at minimum every 120 meters and near every fitting. In rough terrain they should be installed at least every 50 meters.
- 3.5. PVC pipe must be buried at least 50 cm below the natural ground level.
- 3.6. All GI pipe installed above ground should be secured and anchored with concrete pillars at suitable intervals. The installation of pipe above ground has several advantages : it is not possible to "lose" the pipeline several years after installation : it may last longer due to reduced corrosion; and trenching is not necessary. The disadvantages are : it is more subject to tampering and certain kinds of stress ; and expensive concrete pillars are required for support.

A P P E N D I X D  
STEPS IN SURVEY AND DESIGN

I. The Field Survey

1. Survey the source and the village; initiate contact with the community.
2. From survey results decide on the feasibility of constructing the water system.
3. For systems considered feasible a detailed survey of the source, distance, and ground levels is made. Some one from the village should assist in this. At this time tentative locations for distribution points are decided upon in consultation with the villagers, taking into account their own wishes, population distribution etc.

II. The Design Process

The survey data is used to make preliminary design roughly as follows :

1. Decide on general system design, type of distribution facilities, etc.
2. Decide of how much of the population can be served by the system. Decide on per capita usage and calculate the average daily usage taking into account population growth.
3. From the type of system and average daily usage determine the desired storage and its placement. The placement of the distribution points must be fixed at this time.
4. From the placement of the distribution points make a general sketch of the system including all relevant distances and elevations. Based on the population distribution the average daily flows required at each point can be calculated and noted on the sketch.
5. From the average daily flows and the type of system the design flows for the pipe are calculated.
6. From the elevation profile and general scheme of the system the need and placement of break pressure tanks is determined and recorded on the general sketch.
7. From the design flows and elevations the head losses for various diameter pipes are calculated for each unbroken section of the pipeline.
8. The most appropriate pipe is chosen for all sections of the pipeline based on the calculated head losses. The hydraulic gradients are then plotted on the profile. Pipe diameters should be changed where necessary in order to eliminate any negative pressure.
9. Detailed drawings, specifications, materials lists and budgets can now be prepared.

A P P E N D I X E

SAMPLE DESIGN FOR DESA GEMBIRA

1. General

1.1. The village of Desa Gembira has a population of 850 divided into two kampongs, Gembira I with 605 persons and Gembira II with 245 persons. A water source with an estimated minimum flow of 1 l/s is located approximately 2,300 meters from the village. A sketch map of the village and a ground profile are presented in Figures E1 to E4.

2. Design Parameters

2.1. Population

The design population will be the expected population in 10 years at a 2% growth rate or  $850 \times 1.22$  equal to 1,037 persons. For design purposes this may be rounded up to 1,050 persons.

2.2. Water Usage

It is preferable to supply the maximum amount of water possible but a per capita supply of 100 liters per day would require 105,000 liters/day or a flow of 1.2 l/s. Since the estimated minimum flow of the source is only 1 l/s this is not possible. A per capita use of 80 liters/day would require an average daily flow of 0.97 l/s and this is possible.

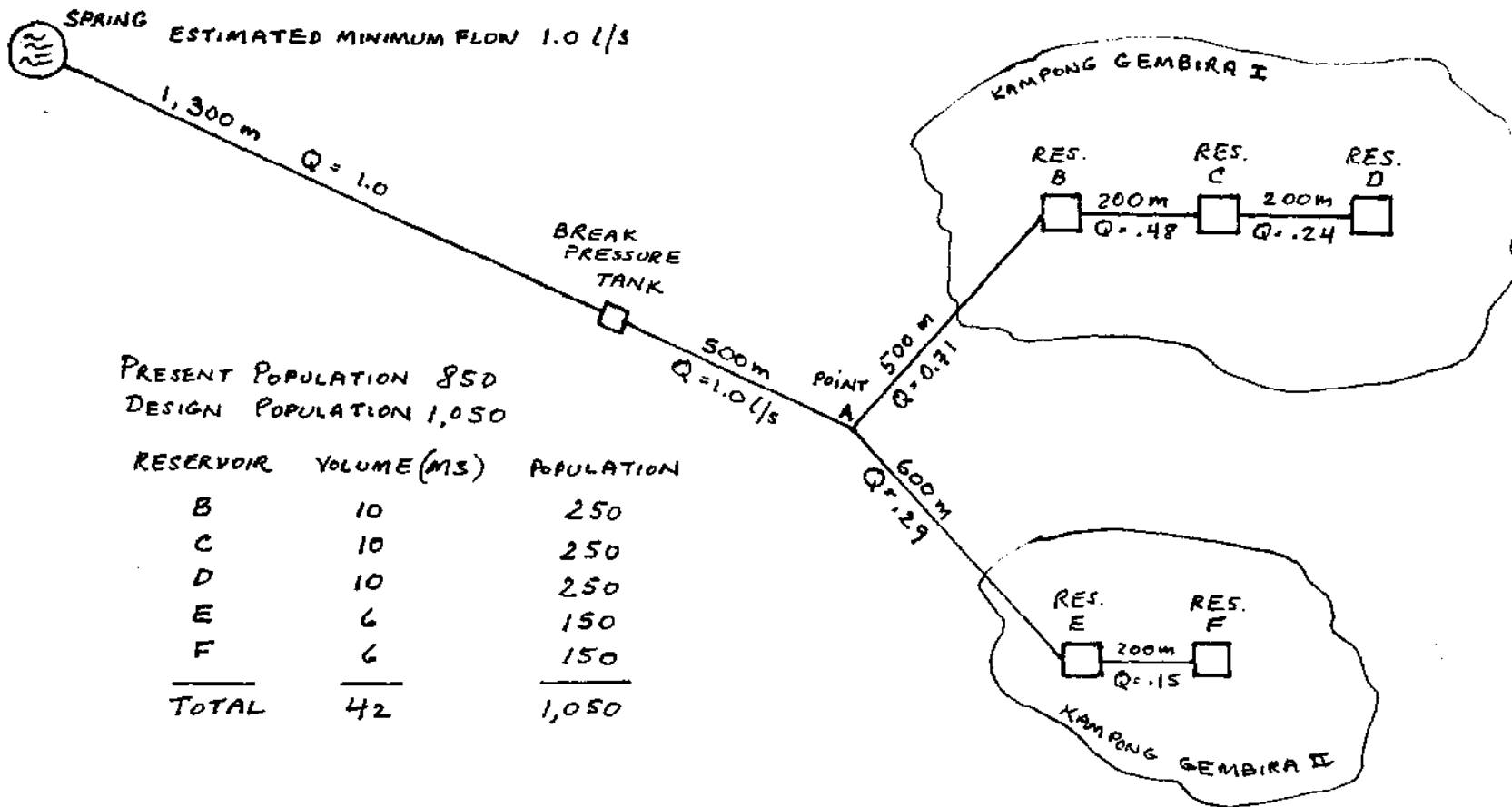
2.3. Storage Requirements

A per capita use of 80 liters/day means that the average daily usage is  $80 \times 1,050$  or 84,000 liters. In order to reduce the size of the main pipe storage will be located in the village. The recommended storage is then one half of 84,000 liters or 42 m<sup>3</sup>. Based on the present population distribution  $245 \div 850 \times 42$  m<sup>3</sup> or 12.1 m<sup>3</sup> should be in Gembira II and  $605 \div 850 \times 42$  m<sup>3</sup> or 29.9 m<sup>3</sup> should be in Gembira I. Based on the village sketch it has been decided that the water will be distributed to 5 public reservoirs, three in Gembira I and two in Gembira II. The three reservoirs in Gembira I will be 10 m<sup>3</sup> each for a total of 30 m<sup>3</sup> (rounded up from 29.9 m<sup>3</sup>) The two reservoirs in Gembira II will be 6 m<sup>3</sup> each for a total of 12 m<sup>3</sup> (rounded down from 12.1 m<sup>3</sup>). With this distribution of reservoirs no one has to walk more than 100 meters to obtain water.

2.4. Number of Faucets

The number of persons per faucet should be between 30 and 100 so the number of faucets for Gembira I should be between 6 and 20

**FIGURE E1**  
**MAP OF DESA GEMBIRA**



PRESENT POPULATION 850  
DESIGN POPULATION 1,050

RESERVOIR	VOLUME (M <sup>3</sup> )	POPULATION
B	10	250
C	10	250
D	10	250
E	6	150
F	6	150
<b>TOTAL</b>	<b>42</b>	<b>1,050</b>



and for Gembira II between 2 and 8. In order to facilitate usage a higher number is preferable. Thus 6 faucets at each of the 3 reservoirs in Gembira I and 4 faucets at each of the reservoirs in Gembira II give a total of 26. The average number of persons per faucet is 42 for Gembira I and 37 for Gembira II. Half of the faucets can be placed on one side of the reservoir and half on the opposite side. Thus, one area can be used by females and the other by males.

### 2.5. Design Flows

The path of the pipeline is sketched in Figure E1. The water will flow continuously into the reservoirs so the design flows will be the same as the average daily flows. At the projected per capita use of 80 l/day the average daily flow is 0.97 l/s but the spring has an estimated minimum flow of 1.0 l/s. Therefore, 1.0 l/s can also be used in designing the pipeline.

From the source to the junction at point A the design flow used is 1.0 l/s. At point A this flow is divided with 0.71 l/s flowing to Gembira I and reservoir B. The remainder of 0.29 l/s will flow to Gembira II and reservoir E. At reservoir B 0.23 l/s is taken and the remainder of 0.48 l/s flows to reservoir C. At C 0.24 l/s is taken and the remainder of 0.24 l/s flows to reservoir D. At reservoir E 0.14 l/s is taken and the remainder of 0.15 l/s flows to reservoir F. The design flows are noted on the pipeline route and profile.

## 3. Pipeline Design

### 3.1. Pressure Release Tanks

Inspection of the profiles in Fig. E2 to E4 indicated that a pressure release tank is necessary 1,300 meters from the source. The static head at this point is 55 meters. The point of greatest pressure is 1,100 meters from the source but it is not possible to place the tank at this point.

### 3.2. Pipe Material

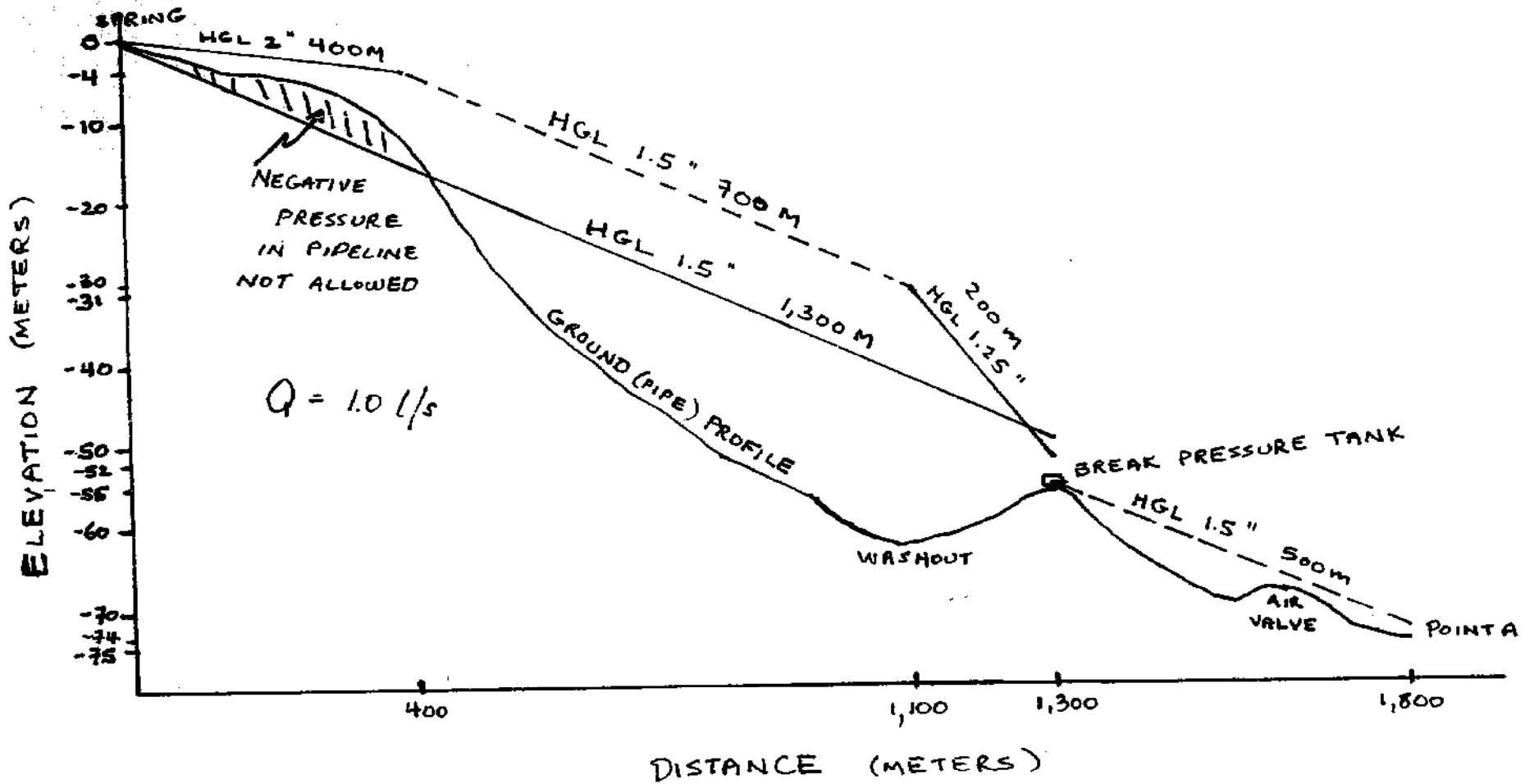
Galvanised iron pipe will be used due to difficulties in burying many sections.

### 3.3. Pipe Size from Source to Pressure Release Tank

The distance to the tank is 1,300 m and the available head 55 m with a design flow of 1.0 l/s. Calculated head losses are as follows :

# FIGURE E2

GROUND PROFILE FROM SOURCE TO POINT A



# FIGURE E3

GROUND PROFILE AND HGL FROM POINT A TO RESERVOIRS B, C, AND D

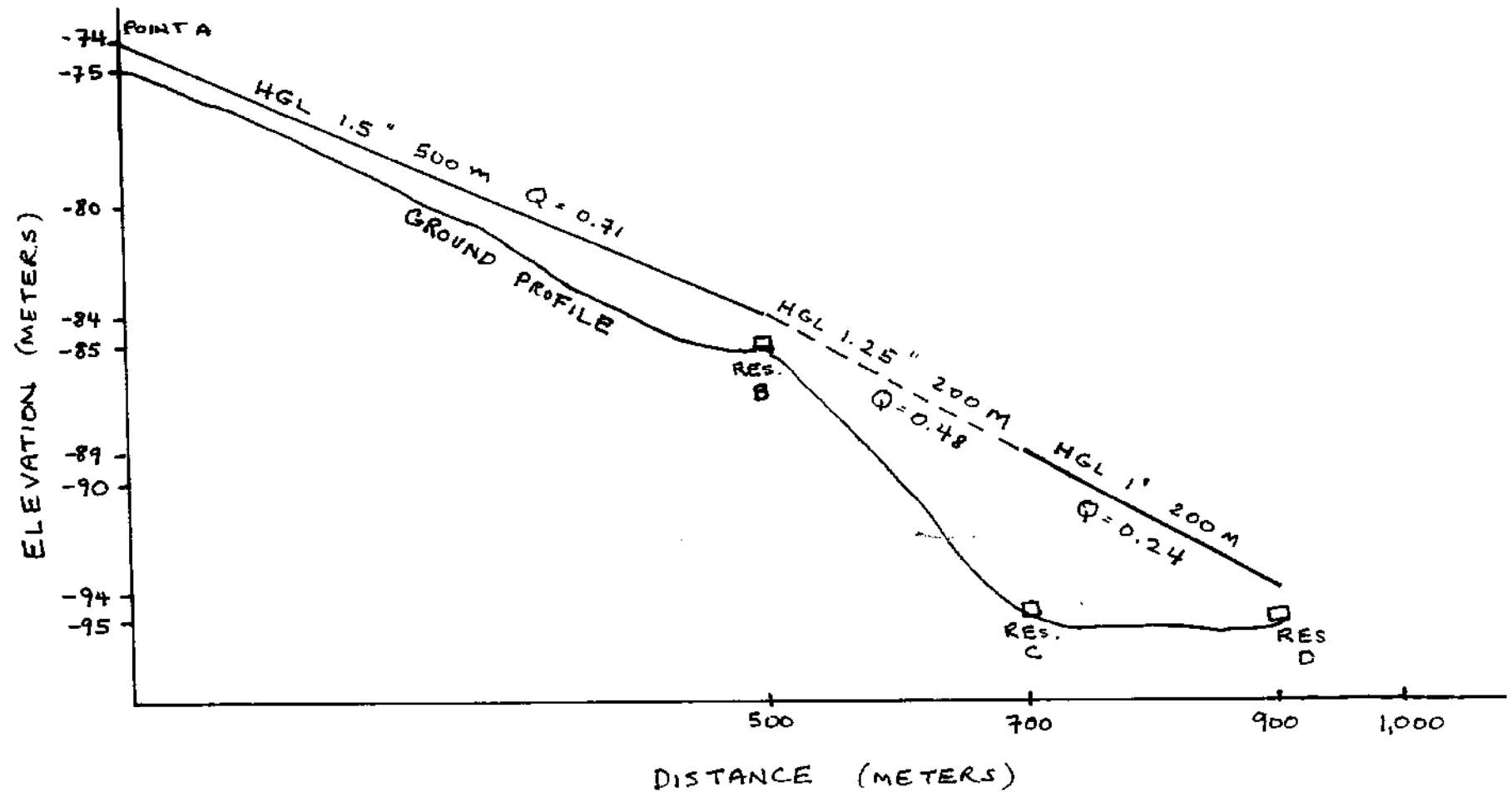
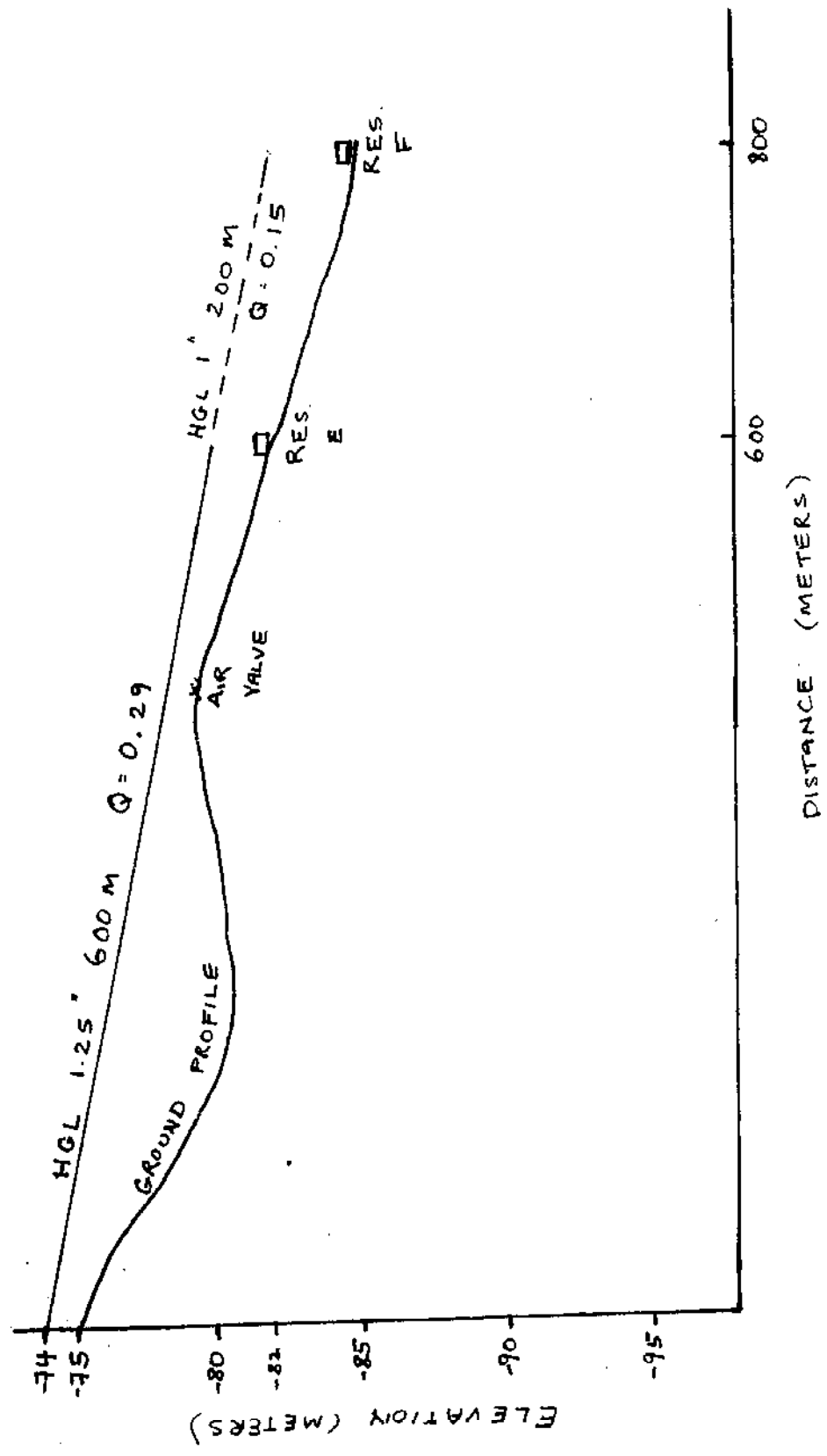


FIGURE E4  
GROUND PROFILE AND HGL FROM POINT A TO RESERVOIRS E AND F



length (meters)	flow (l/s)	pipe diameter (inches)	head loss (incl. 10% extra) (meters)
1,300	1.0	2	12
1,300	1.0	1.5	50
1,300	1.0	1.25	115
400	1.0	2	4
900	1.0	1.5	34
700	1.0	1.5	27
200	1.0	1.25	21

From the calculated head losses it appears that 1.5 inch is sufficient but a check of the HGL on the profile indicates a negative pressure in the first 400 meters. Thus a 2 inch pipe must be used to avoid the negative pressure and the remaining pipe can be a combination of 1.5 inch and 1.25 inch which is found by trial and error. With 400 meters of 2 inch, 700 meters of 1.5 inch and 200 meters of 1.25 inch the total calculated head loss is 51 meters.

#### 3.4. Pressure Release Tank to Junction at Point A

Calculated head losses are as follows :

length (meters)	flow (l/s)	pipe diameter (inches)	head loss (incl. 10% extra) (meters)
500	1.0	2	5
500	1.0	1.5	19
500	1.0	1.25	51

With the available head of 20 meters a 1.5 inch pipe is appropriate.

#### 3.5. Point A to Reservoirs B,C,D.

Calculated head losses are as follows :

length (meters)	flow (l/s)	pipe diameter (inches)	head loss (incl. 10% extra) (meters)	Available head (meters)
500	0.71	1.5	10	10
500	0.71	1.25	23	10
500	0.71	2.0	3	10
200	0.48	1.5	2	10
200	0.48	1.25	5	10
200	0.48	1.0	16	10
200	0.24	1.25	1	0
200	0.24	1.0	5	0
200	0.24	0.75	18	0

From Point A to Reservoir B a 1.5 inch pipe is appropriate. From reservoir B to C a 1.25 inch pipe is adequate and there is extra head available of  $10 - 5 = 5$  meters. Since reservoirs C and D are at the same elevation this head will allow the water to flow to reservoir D. Thus between reservoirs B and D the total head loss is 10 meters and the total available head is also 10 meters.

3.6. Point A to Reservoir E and F

Calculated head losses are as follows :

length (meters)	Flow (l/s)	Pipe diameter (inches)	head loss (incl. 10% extra) (meters)	Available head (meters)
600	0.29	1.5	3	7
600	0.29	1.25	6	7
600	0.29	1.0	19	7
200	0.15	1.0	2	3
200	0.15	0.75	6	3

If 1.25 inch pipe is used from point A to reservoir E and 1.0 inch pipe from reservoir E to F then the total head loss is  $6 + 2$  or 8 meters which closely matches the available head of 10 meters. The pipe sizes, HGL, and design flows are all noted on the profile.

A P P E N D I X F

G L O S S A R Y

- Available Head** : The actual difference in elevation between the two points in question. For example, the difference in elevation between the outflow pipe of the capturing and the overflow pipe of the first reservoir would be the available head used in selecting an appropriate pipe.
- Average Daily Flow** : The flow of water necessary to supply the Average Daily Usage if the water were flowing continuously. It is used as the basis for selecting design flows for pipes. The Average Daily Usage in m<sup>3</sup> divided by 86.4 gives the Average Daily Flow in l/s that is necessary to supply that amount.
- Average Daily Usage** : The average volume of water which flows through the water system during a 24 hour period. It is based on the total population served and the projected per capita usage and is usually expressed in cubic meters (m<sup>3</sup>). The Average Daily Usage is a hypothetical quantity. In reality, on some days water usage is greater and on some days it is less, it is only rarely the same. However, this figure is the basis used for determining other design parameters such as storage volume and design flows. The Average Daily Usage for a system supplying 1,000 persons with 100 liters per day is; 1,000 x 100 liters or 100 m<sup>3</sup>.
- Coliform Bacteria** : Defined as rod shaped gram negative bacteria that ferment lactose at 35° C. They are widespread in the environment and not necessarily harmful to man. They were formerly widely used as an indicator organism for water quality but are being replaced by fecal coliforms. See also Fecal Coliforms.
- Design Flows** : The flow used in the calculation of head losses to determine the pipe diameter. It is chosen by the designer based on the number of users, level of service, and type of storage to be provided and is thus related to the average daily flow.

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**Estimated Minimum Flow :** The best estimate of the ten year low flow from the water source that can be made with the available data. Often the lowest point measurement taken reduced by an appropriate factor taking into account all relevant factors. It is best to be conservative in estimating the minimum flow to be used in designing the system.

**E. Coli :** See Fecal Coliforms

**Fecal Coliforms :** Also referred to as Escherichia Coli or E. Coli. These are part of the coliform group but the fermentation test is conducted at 44° C. At this temperature the non-fecal organism die from heat shock. It is believed that all fecal coliforms originate in the intestines of man and higher mammals and that they will not reproduce outside the host. There is some evidence that this does not hold true for tropical areas but until further research is completed the presence of fecal coliforms must be regarded as evidence of fecal contamination. Fecal coliforms themselves are usually not harmful to man but their presence indicates that the water may contain disease causing organisms which may cause illness in those who consume it.

**Guideline :** A value or rule derived from known facts or figures to be used in decision making or designing. The decision maker may choose to deviate from the guideline. This is opposed to a standard or criteria which is generally a rule established by authority that cannot be deviated from.

**Head :** The pressure or force per unit area that is available or must be overcome in order to transport water. Head may be supplied by gravity or by mechanical means such as a pump. Although it is a pressure it is generally referred to in meters or feet which is the equivalent pressure that would be exerted by a standing column of water of that height.



**Head loss** : A loss of pressure (or head) in a closed pipeline due to friction between the pipe and the flowing water. The head loss increases if the volume of water increases, if the distance it is carried increases, if the diameter of the pipe is made smaller, or if the inside surface of the pipe is rougher. Calculated head losses are compared with the available head to determine if the desired flow of water will be obtained under conditions given.

**Membrane Filter Technique** : A method for determining concentrations of coliforms and/or fecal coliform bacteria in water sample. The water sample is filtered through a small membrane that retains the bacteria. The bacteria on the membrane are then cultured (allowed to grow) for 24 hours at a set temperature in an incubator. The number of bacteria colonies growing are then counted and related to the volume of water filtered to estimate the concentration of bacteria in the original water sample.

**Peak Usage** : The highest flow of water expected to occur on any given day. This flow usually lasts for only a very short time period and does not necessarily occur every day. For designing standpipe systems it is taken as four times the average daily flow.

**Point Measurements:** Measurements of flow, quality etc. taken at one instant in time. These measurements represent the actual condition only at the exact time of taking the measurement. Where continuous monitoring is not possible then a number of point measurements are used to estimate other quantities of interest such as the average flow, highest yearly flow, ten year low flow etc.

**Q** : A general symbol used by engineers to represent the quantity of water flowing. It could represent any flow such as that of a spring, river, or pipeline etc. It is normally expressed in liters per second or cubic meters per day.

**Standards** : See guidelines

- Static Head** : The various pressures that would be obtained in the water system if it were full of water and the water is not flowing. It is different for each point in the system and depends on the elevation relative to the highest point in the system. It is the same as the available head.
- Ten Year Low Flow** : The lowest flow from the water source that is expected to occur, on the average, once every ten years.