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A Handbook of Gravity-Flow Water Systems for small communities

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surveyor. The surveyor makes a backsight reading on the last station; the vertical angle of the backsight should be equal (but of opposite sign) to first sighting. The surveyor then shoots the foresight to the assistant ahead, and then everyone advances one station. Each elevation can then be calculated using the average of two vertical angles.

When a survey is closed, the difference in elevation readings of the two surveys should agree to within 6% of the original surveyed elevation change.

Example: The elevation from source to reservoir site of a system was originally measured to be 55 meters. The closing survey from reservoir back to source measured an elevation of 53 meters. The difference between the two surveys is 2 meters, which is 3.6%, which is within the allowable limit of 6% (2/55 x 100% = 3.6%).

Accuracy: The accuracy of a calculated elevation is dependent upon the accuracy of the surveying equipment and techniques. A common practice, especially when using electronic calculators, is to calculate elevations to several decimal places (such as "4.679" or "6.341", etc.). Such "precision" is easy to compute with the calculator, yet is actually a false accuracy. Calculations to such a high degree of accuracy imply that the surveying equipment and techniques are equally accurate, which definately is not so.

Accepted engineering and scientific practices state that no instrument is any more accurate than one-half of the smallest scale division. Thus, an Abney scale calibrated in one-degree divisions may not be read more accurately than plus/minus 0.5-degree. A tape measure whose smallest division is in centimeters cannot measure any more accurately than plus/minus 0.5-centimeter. Although the human eye may be able to read the scale more accurately than this, the manufacturer did not design the instrument to be that accurate. Therefore, it is wrong to do so.

Another constraint on the accuracy of the survey are the conditions under which it is conducted: field measurements are inherently less accurate than laboratory measurements.

Under field survey condititons in Nepal, the following standards of accuracy should be adopted:

vertical angle: plus/minus 0.5-degree
ground distances: plus/minus 0.1 meters (10 cm)
calculated elevations: with extremely meticulous technique, an accuracy of plus/minus 30 cm can be obtained, but for general surveying, an accuracy of plus/minus 0.5-meter is correct.

4. DESIGN PERIOD, POPULATION, AND WATER DEMANDS

4.1 INTRODUCTION

This chapter will present the manner of calculating the daily water demand of a village. The population growth rate for that regional area of Nepal is used to project the village's current population to the future population after 15-25 years. The water demands of the village are then calculated, based upon the future population.

4.2 DESIGN PERIOD

Community water supply systems should be designed and constructed for a 15-25 year lifespan. The choice of either a 15, 20, or 25-year design period is made by the surveyor, based upon the amount of potential change that he 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 new highway or airfield is slated for construction, a shorter design period should be considered, because the long-range water demands cannot be accurately forecasted.

4.3 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 factor for the design period, given in Figure 4-1.

Example: A village in the Far Western hills of Nepal has a current population of 436 people. The design period was selected to be 20 years. What is the design population?

Future population = current population + 34%
= 436 x 1.34
= 584 people

Within the design report, the design period and population forecast should be carefully indicated, as should any special criteria for their selection.

GEOGRAPHIC AREA	1961-1971 average annual growth rate	PERCENTAGE INCREASE 10-yrs	15-yrs	20-yrs	25-yrs
FAR WESTERN DEVELOPMENT REGION					
Mountains	1.4	14	22	30	39
Hills	1.5	16	25	34	45
Indian border districts	2.3	25	40	56	75
Surkhet Valley	2.3	25	40	56	75
Plains	3.4	40	66	96	130
WESTERN DEVELOPMENT REGION					
Mountains	1.1	12	18	25	33
Hills (northern)	1.6	17	27	38	48
Hills (southern)	2.1	23	36	51	68
Plains	3.7	44	73	110	150
CENTRAL DEVELOPMENT REGION					
Mountains	1.0	11	17	23	30
Hills	1.6	18	27	38	48
Kathmandu Valley	1.3	13	20	28	36
Plains	3.8	45	74	110	150
EASTERN DEVELOPMENT REGION					
Mountains	1.1	12	18	25	33
Hills	1.5	16	25	34	45
Plains	4.1	50	84	120	170

Note: All figures derive from the 1952-54, 1961, and 1971 census data. The 10-25 year growth figures are based upon 1961-1971 average annual growth rates, computed by C. Johnson.

FIGURE 4-1
POPULATION FORECAST TABLE

4.4 WATER DEMANDS

The total water demands for the village at the end of the design period is the sum of the per capita demand plus special need demands.

Per capita demand is the water required per person of the project village population. A per capita demand of 45 litres per person per day is the present design standard. This figure derives from World Health Organization (WHO) studies, and includes allowances for personal washing, drinking, cooking, and a portion of domestic animal needs.

When a marginal water source is encountered, and the target figu[illegible] of 45 LPCPD (liters per capita per day) cannot be met, then one may go as low as 230 liters per household per day. This figure is based upon minimal needs, and assumes 8-10 persons per household.

Special need demands are those required by additional facilities in the village, such as schools, health posts, government offices, etc. The amount of water needed daily by these facilities is given below, based upon WHO ideal target usages:

Facility	Daily Demand (liters) Ideally	Minimally*
Schools -day students	10 liters/student	6.5
-boarding students	65 liters/boarder	42
Hospitals & Health Posts	500 liters/bed	325
Health clinics (no beds)	2500 liters/day	1625
Government Offices	500-1000 liters/day (depending upon size)	325-560

*Minimal figures are 65% of ideal

The village's total daily water requirements will be the sum of the per capita demand plus the special needs demand, as projected for the end of the design period.

5. TYPES OF SYSTEMS

5.1 INTRODUCTION

There are several types of gravity-flow water systems, each type being determined by certain design characteristics. These systems fall into two general catagories: open systems, and closed ones.

An open system derives from the concept that the taps can be left open and flowing continuously all day long, and still provide constant and steady flow. This means that the safe yield of the source(s) is sufficient enough to supply all tapstands directly, without requiring a reservoir tank.

A closed system is one where the safe yield of the source cannot provide continuous flow to all taps, or where the safe yield is such that a reservoir tank is necessary to store water for peak demand periods which the source alone could not meet. All tapstands on the system must have a faucet, either of the self-closing or manually-operated type.

Both catagories of systems may require break-pressure tanks, but an open system will never require a reservoir tank. At all tapstands, regardless of the type of system, a control valve must be installed to proportion and regulate the flow between taps.

From these two catagories, there are five different types of systems which can be built, as discussed below.

5.2 OPEN SYSTEMS WITHOUT FAUCETS

This type of system has continual, 24-hour flow from the taps, with no faucets to shut off the water. The primary advantage to this system is that there are no faucets that can be abused, worn out, broken, stolen, etc. The primary disadvantage arises out of the copious amounts of wastewater issuing forth all day and night. Strategic location of taps to make efficient use of wastewater (such as irrigation of nearby fields, etc) and construction of non-erodible drainage channels to carry these flows away will minimize the problems of large wastewater quantities.

5.3 OPEN SYSTEM WITH FAUCETS

The problems of copious wastewater flows from an open tapstand can be eliminated by installing faucets on some of the tapstands. Provisions must be made for handling overflow water from the lowest break-pressure point (i.e. reservoir tank, break-pressure tank, etc.), since excess water will overflow at that point.

This type of system is one of the more desirable types, since it requires no reservoir tank, provides more than sufficient water for the villagers, and has minimal wastewater problems.

5.4 CLOSED SYSTEM WITH RESERVOIR

A reservoir tank is required when the peak water demands of the village cannot be met by the source alone. The reservoir stores wate from low-demand periods(such as overnight) to supplement the source flow during peak demand periods (such as early morning). A reservoir system is able to provide water at any time demanded, but depends upon faucets and pipeline being well maintained (a broken faucet or a leaky pipeline will not allow the reservoir to fill).

A reservoir system may actually be less expensive to build than an open system, since usually a smaller pipe size can be used between the source and reservoir. The savings in pipe cost can offset the cost of the tank (refer to Section 5.7).

5.5 CLOSED SYSTEM WITH INTERMITTENT SERVICE

There are some topographic situations where the yield of the source and geography of the terrain act in such a way that the system must be designed with one (or more) break-pressure tanks located downstream from the reservoir tank. This arrangement has required an intermittent supply system: except for a few hours each day (ie- in the mornings and evenings), the water is shut off at the reservoir tank to allow it to refill. Without doing this, the tank would never refill, since it would be constantly draining out through the lower break-pressure tanks.

This intermittent system is the least-desirable type to build. Hydraulic problems, such as air entrapment, can complicate the draining and refilling of the pipeline each day; there will be increased wear on the control valves at the reservoir; support of the system caretaker requires considerable village organization; negative pressures in the pipeline during system shut-down can suck in polluted groundwater via small leaks; and since the entire water demand period is compressed into just a few hours (rather than spread out over the full day), the taps must be designed to deliver greater flows, which in turn requires larger pipe sizes and substantially increases the cost of the system.

Fortunately, it is possible to avoid intermittent systems by installing float-valves (also known in Nepal as "ball-cocks") in the downstream break-pressure tanks.

5.6 CLOSED SYSTEM WITH FLOAT-VALVES

As mentioned above, there are some situations where it is inescapably necessary to install break-pressure tanks downstream from the reservoir.

Float-valves are installed in these break-pressure tanks, and act on the same principle as those commonly used in household toilets. These valves automatically adjust the flow in the pipeline to exactly match the amount demanded by any open taps. When all taps are closed, the break-pressure tank fills with water, lifting the float and gradually closing the valve until the flow is cut off. This allows the upstream reservoir tank to refill.

Sturdy-quality float-valves are now becoming part of the standard supplies provided by UNICEF for water supply projects in Nepal. Locally available float-valves (usually manufactured in India), although not of high quality, can also be used and offer the advantage that, if broken, they can be easily and inexpensively replaced by the villagers themselves.

5.7 OPEN SYSTEM VS CLOSED SYSTEM

The decision to build a system as either open or closed is governed by several factors: pipeline profile, safe yield of the source, design population, and availability of construction materials. In some instances, the decision is an obvious one, and in other cases the designer must evaluate the economics of both types before making a decision.

As mentioned above in Section 5.4, a reservoir system may be a more economical system than an open system. An open system will usually require a large-size pipe between the source and the village, whereas if a reservoir tank was constructed, then a lot of that pipe could be replaced with a smaller-size. The designer should always investigate both of these alternatives if it is possible that a system may be built as an open one. However, sometimes the specific pipe sizes, or enough cement, may not be quickly available, in which case the alternate system may have to be built if delays in construction are to be avoided.

5.8 LIMITED EXPANSION

One aspect with which planning and designing a water project is concerned is the extendibility of the system. Although the population is projected through the end of the design period, the physical growth of the village may expand in such directions that the villagers may wish to add one or two more tapstands to the system at some future date. It is also possible that the village's population growth may in fact be much greater than initially assumed, resulting in the design water demands long before the end of the design period. This section discusses possible means of limited expansion of the system to resolve these problems, provided that preparations are made during the initial survey, design, and construction of the system. These expansion possibilities are only aimed at meeting these unexpected needs for the duration of the initial design period. It is presumed that by the end of the original design period,

the condition of the system and the new village water needs will require a major overhaul of the system, or even construction of an entire new one.

Additional taps: The need for additional taps can be minimized by trying to predict in which directions the village is likely to expand in the future, and locate tapstands accordingly. Although this anticipation of the future will rarely be easy to make, the geography of the land around the village will sometimes set limits on expansion (such as rivers, cliffs, direction of ridges and hills, etc.).

If additional tapstands are needed, no changes in the pipeline will be necessary if the villagers are willing to slightly reduce the flow from the other taps to make water for the new taps. Decreasing the flow of four tapstands by 20% will allow the addition of a fifth one to the line. The system designer should indicate in the design report just where additional tapstands may be added, and what flow re-adjustments would be necessary. This information should be in the project file at the LDD office, and also should be discussed with the village leaders.

Increased water demands: This problem can only be solved if there is another water source located above the intake or reservoir level, so that the flow from the new source can be added to the existing one. In the future, water purification schemes may become available to many projects, thus a near-by water source which is currently unacceptable may some day be able to be added to the system. Despite such increased flows, additional water storage capacity may be required. This can be accomplished by either of two methods: select the original reservoir site so that a second tank may be constructed next to it and cross-connected; or alternatively, design the first tank such that its walls can be raised enough to add another 30-50 centimeters of water depth to the tank.

Again, the design report should indicate just how future expansion of storage capacity has been planned for.

5.9 PHASED EXPANSION

Expansion of a system does not necessarily have to remain within the domain of the original system. There may be a small ward or village close at hand which, currently, has its own adequate water supply and does not have to be included in the system initially. After a number of years, however, that small population may have outgrown its water source, and then consideration should be given as to how the original system may be extended to incorporate it.

The best way to accomplish this is to plan for it when initially preparing the system design. Certain pipelines of the system would have to be of a larger size than otherwise necessary, and the reservoir should be designed so that it can be expanded as discussed above.

Branchpoint tees and/or control valves for the future extension may be installed at the time of initial construction. The design report should indicate after how many years it is intended to extend the system, and the matter discussed with the LDD regional engineer and village leaders.

6. HYDRAULIC THEORY

6.1 INTRODUCTION

In this chapter, the basic hydraulic principles that govern the behavior of gravity-flow water systems will be presented. It will not be possible to understand this material in a single, cursory reading, yet full understanding of these concepts is necessary befor any person can properly design such a system. The designer should read, study, and repeatedly refer back to this chapter until he is satisfied with his knowledge of these principles.

The next chapter shall discuss special strategies in designing a pipeline section where there are potential air-blocks.

6.2 ENERGY

To move water, whether moving it uphill, downhill, 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 water system is "powered" by gravitational energy. The amount of such energy in the 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 (ie- buried into the ground) and their relative elevations cannot 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 profile of the system, at some points there may be a minimal amount of energy (ie- low pressure), while at other points there may be an excessive amount of energy (ie- high pressure). A poorly designed or constructed system will not conserve energy properly enough to move the desired 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 location of control valves, break-pressure tanks, reservoirs, tapstands, etc.

6.3 HEAD: The Measure of Energy

On the Earth's surface, fresh water weighs 1 gram per cubic centimeter (1 g/cm^3). A column of water one centimeter square and

100 centimeters high (1 x 1 x 100 cm) would therefore weigh 100 grams. The same column 1000 cm high would weigh 1000 grams (1 kilogram). The area at the base of this column is one square centimeter (cm^2) and supports the entire weight of the column. Therefore, the pressure at the base of this column is 1 kg/cm^2. The same column 20 meters high (2000 cm) would weigh 2 kgs, and exert a pressure of 2 kg/cm^2; a column 30 m high exerts a pressure of 3 kg/cm^2; a column 43 meters high exerts a pressure of 4.3 kg/cm^2, and so on.

In hydraulic work, rather than repeatedly calculate water pressures, it is an easier practice to simply report the equivalent height of the water column. Technically, this is called the 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 1.4 kg/cm^2 is reported as 14 meters of head; a pressure of 4 kg/cm^2 is 40 meters of head; 5 kg/cm^2 is 50 meters of head, etc.*

6.4 FLUID STATICS: 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 descended but that swimming horizontally at a constant depth produced no change in pressure. This common experience serves to illustrate a major principle in hydraulics:

> 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 6-1. The water pressure at point A is determined by the depth of water at that point. The pressures at points 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	1 kg/cm^2	10 meters
B	.2 kg/cm^2	20 meters
C	3.5 kg/cm^2	35 meters

* the pressure exerted by other fluids, such as mercury, oil, etc, can also be reported as equivalent heads of that fluid. Barometric pressure, for example, is often measured as "millimeters of mercury".

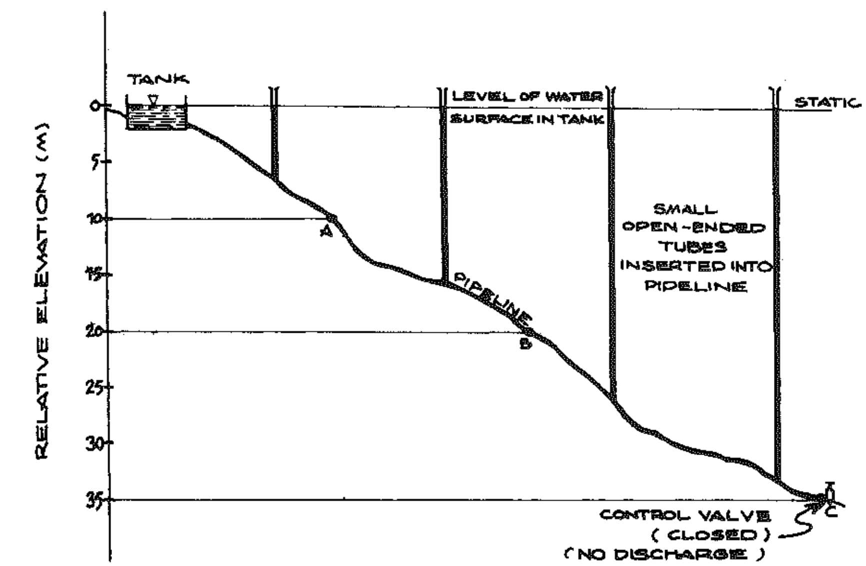


FIGURE 6-1 . STATIC EQUILIBRIUM

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.

If small tubes were inserted into the pipeline, as shown in Figure 6-1, the water level in each tube would rise exactly to the static water level. The height of water in each tube is the pressure head exerted on the pipeline at that point.

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

6.5 FLUID DYNAMICS: Water in Motion

Now suppose that the control valve at point C in Figure 6-1 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 glass 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 6-2.

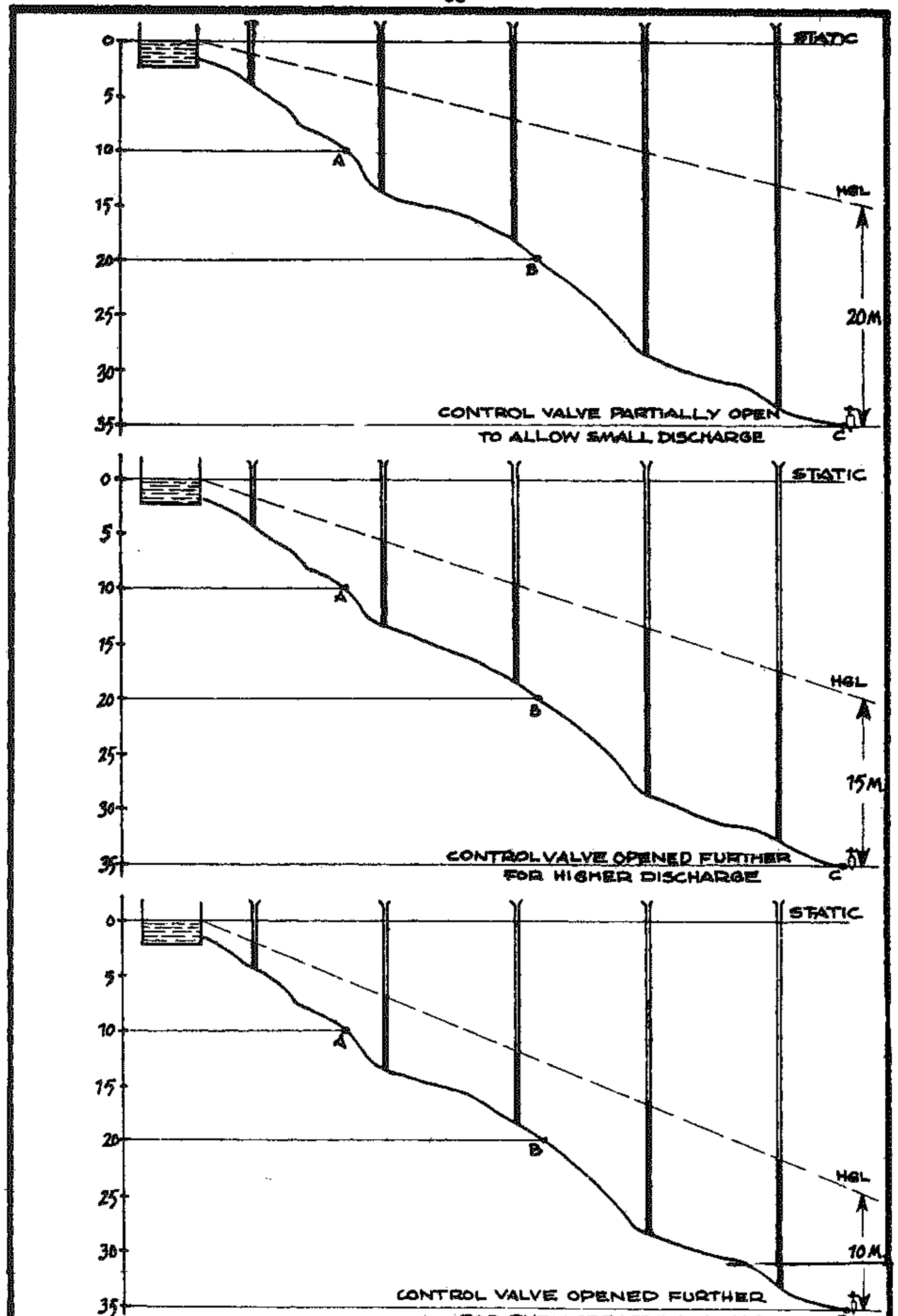


FIGURE 6-2 DYNAMIC EQUILIBRIUM

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 heights 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. A different flow establishes a different dynamic equilibrium, and a new HGL.

6.6 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 (ie- 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 interfaces (such as the surfaces in tanks or discharges at tapstands) is zero. Thus, the HGL must always come to zero wherever the water comes into contact with the atmosphere.

Since frictional losses are never recovered, the HGL always slopes down along the direction of flow. The steepness of the slope is determined by the rate at which energy is lost to friction. Only under static conditions is the HGL perfectly horizontal, although for practical purposes the HGL may be plotted as horizontal for extremely low flows in large pipes (where the headloss is less than 1/2-meter per 100 meters of pipelength). For practical purposes, the HGL will never slope upwards.

Appendix A gives a more mathematical discussion on the HGL, with relevant examples of how it applies to a gravity-flow water system.

6.7 FRICTION: Lost Energy

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.

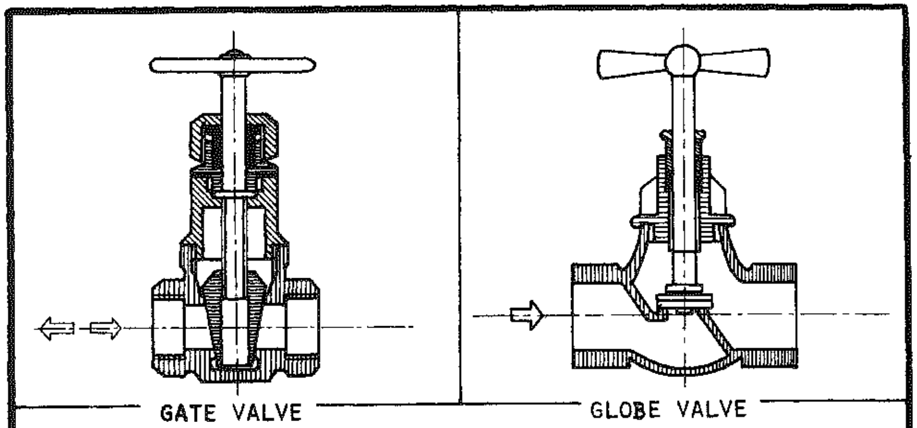
The magnitude of energy lost due to friction against some obstacle is determined by several factors. The major factors would be the roughness of the obstacle, and the velocity of the flow. Minor factors would include water temperature, suspended particles, dissolved gases etc.

The diameter of the 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 greater the frictional losses.

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

6.8 VALVES: Variable Friction Devices

An excessive amount of energy (ie- high pressure) can cause the pipe to burst. One method of controlling excessive amounts of energy is to install control valves at strategic points throughout the system. A valve is a device which can be adjusted to create greater frictional losses as the water flows through it. There are two types of control valve : gate valves, and globe valves. Both are shown below in Figure 6-3:



GATE VALVE GLOBE VALVE

FIGURE 6-3 CONTROL VALVES

Gate valves: Gate valves serve as on/off control valves, for the purpose of completely cutting off the flow. Generally, they are located at the outlets of intakes, reservoirs, strategic break-pressure tanks, and at major branchpoints. They are not recommended for use in regulating flow (ie- partially open or closed) since the water will erode the bottom edge of the gate and result in a leaky valve whenever it is meant to be closed. Direction of flow through the valve is unimportant.

*flow, velocity, and pipe size are all related by the Equation of Continuity presented in Technical Appendix A.

Globe valves: These valves are designed for regulating flow through the system. They are best located near discharge points, so that it is easier to measure the flow through the valve. They are generally located at discharge points in reservoirs, strategic break-pressure tanks, and at every tapstand*. Direction of flow through a globe valve is important: there is an arrow stamped on the valve that indicates the proper direction of flow, and care must be taken to see that the valve is installed correctly.

6.9 FRICTIONAL HEADLOSS FACTORS

It is obvious that to properly design a system, the designer must be able to determine how much energy will be lost to friction by the time the flow reaches various critical points in the system. Frictional headloss tables are used for this purpose. The common method is to report the amount of frictional headloss per unit length of pipe, for a specific flow. Typically this would be expressed as "meters of headloss per 100 meters of pipelength", or as "m/100m" or "%".

The frictional headloss tables for both HDP and GI pipe are given at the end of this book. These headloss factors are never perfectly accurate since frictional losses are affected by many different factors which may vary from system to system. For this reason, it is necessary to always include a margin of safety when plotting the HGL (see Section 6.13).

Example: What are the frictional headlosses in the pipeline section below?

a) 1350m of 32mm HDP @ 0.45 LPS:

Frict'l headloss factor = 2.56m/100m

1350 x 2.56/100 = 34.6 meters headloss

b) 730m of 2" GI pipe @ 1.30 LPS:

Frict'l headloss factor = 1.84%

730 x 1.84/100 = 13.4 meters headloss

c) 2075m of Class IV 50mm HDP @ 1.40 LPS:

Frict'l factor = 3.22%

2075 x 3.22/100 = 67 meters headloss

Frictional headlosses can be rounded off to the nearest 1/2-meter, or even to the nearest meter.

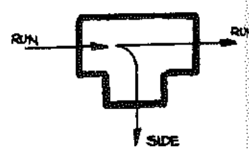
Frictional headlosses of flows through fittings such as elbows, reducers, tees, valves, etc, are given as equivalent pipelengths.

* the 1/2" globe valve used on tapstands is known in Nepal as a corporation cock.

6.10 EQUIVALENT PIPELENTHS OF FITTINGS

A pipeline fitting (such as an elbow, tee, valve, etc) acts as a concentrated point of frictional losses. The amount of headloss in the fitting depends upon the shape of the fitting, and the flow through it. The headlosses are computed by determing the equivalent length of pipe necessary to create the same amount of headloss. For fittings, this is commonly given as the L/D Ratio (length/diameter). The L/D ratios for various fittings are given below:

Fitting	L/D Ratio
Tee (run-side)	68
Tee (run-run)	27
Elbow (90°, short-radius)	33
Union	7
Gate valve (fully open)	7
Free entrance	29
Screened entrance	150



Example: What is the equivalent pipelength of a 1-1/2" GI elbow?

1-1/2" x 33 = 50" = 126 cm = 1.26 meters

Where fittings are located at isolated points along a long pipelength, the amount of headloss they generate is considered minor compared to the normal headloss through the pipe. Such headlosses do not have to be shown on the plotted HGL when the pipelength is more than 1000 diameters. For the common pipe sizes used in CWS projects, these losses can be ignored if the pipe section is longer than:

20mm HDP: 20 meters
32mm HDP: 32 meters
50mm HDP: 50 meters
63mm HDP: 63 meters
90mm HDP: 90 meters

When several fittings are located close together, however, the total headloss is actually greater than the sum of individual headlosses through each fitting. Thus, special concern must be given to selecting the proper pipe sizes for the GI plumbing of a tank outlet This is explained in Technical Appendix G.

Since a valve is adjustable, it can be set for any equivalent pipelength. This is discussed further in Section 6.13 and Figure 6-7.

6.11 PLOTTING THE HGL

To illustrate the basic principles of plotting the HGL, the simple system of Figure 6.4 will be used. In this example, the pipe sizes have already been selected. The desired flows out of each tap are 0.225 LPS, the safe yield of the source is 0.50 LPS. The elevations and pipelengths are given for the source, Tap # 1 and Tap # 2.

The HGL is plotted in sections (technically called reaches) between strategic points in the system.

First reach: In this example, the first reach is from the intake to the end of the 50mm HDP pipe section. The desired flow through this reach is 0.45 LPS (ie- the sum of the tap flows), and since the safe yield of the source is greater than 0.45 LPS, no reservoir tank is required (ie- the system can be built as an open system, with or without faucets).

340m of 50mm HDP @ 0.45 LPS
Frict'l factor = 0.30%
340 x 0.30/100 = 1 meter headloss

This is plotted on the profile.

The second reach ends at the first tap, 480 meters of 32mm HDP pipe. The desired flow is still 0.45 LPS.

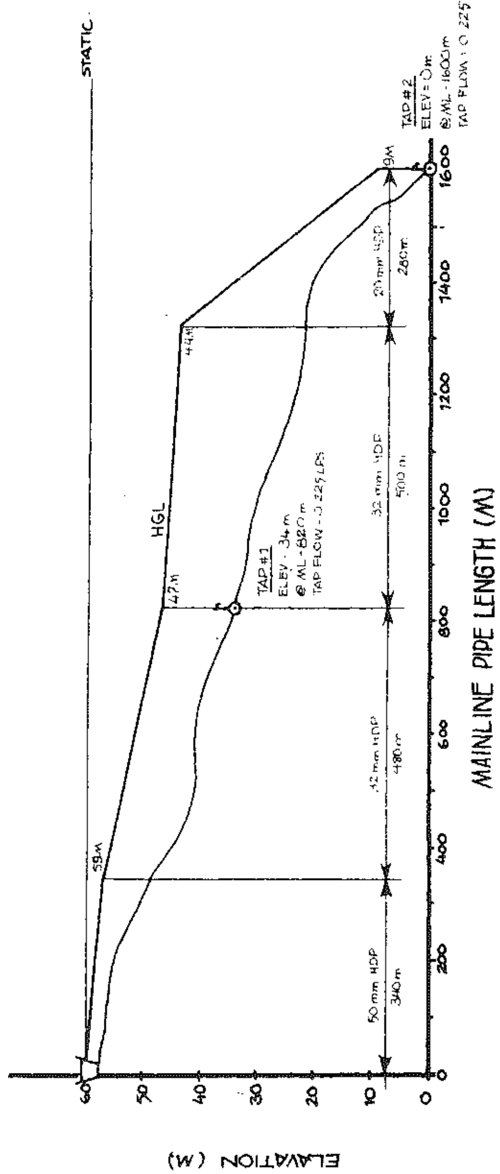
480m of 32mm HDP @ 0.45 LPS
Frict'l factor = 2.56%
480 x 2.56/100 = 12 meters headloss

The residual head at Tap # 1 is therefore 13 meters (refer to Chapter 15.4 for standards on acceptable residual heads for tapstands).

The third reach is from Tap # 1 to the end of the 32mm HDP pipe section: 500 meters of pipe. The desired flow in this section is now only 0.225 LPS (ie- flow for just the remaining single tap).

500m of 32mm HDP @ 0.225 LPS
frict'l factor = 0.78%
500 x 0.78/100 = 4 meters headloss

The HGL at this point is now 17 meters below the static level, meaning that a total of 17 meters of head has been lost to friction by the flow between the source and end of this reach.



FRICTIONAL HEADLOSSES(M/100m)

	Q=0.225 LPS	0.450 LPS
20 MM.	S = 12%	40.00%
32 MM.	0.78%	2.56%
50 MM.	—	0.30%

FIGURE 6-4 PLOTTING THE HGL (EXAMPLE SYSTEM)

The final reach is 280 meters of 20mm HDP pipe, carrying a flow of 0.225 LPS.

280m of 20mm HDP @ 0.225 LPS

frict'l factor = 12.0%

280 x 12/100 = 34 meters headloss

The residual head at Tap # 2 is 9 meters.

Observe that the HGL only changed slope at points of-new pipe sizes and/or new flows. To allow only the desired 0.225 LPS out of each tap, globe valves must be installed in the tap pipeline (tapline) and adjusted so that precisely the 0.225 LPS comes out of the faucets. When adjusted like that, the valve for Tap # 1 will be burning off 13 meters of head, and the valve for Tap # 2 will be burning off 9 meters of head. The effects of residual heads are discussed in Section 6.13.

6.12 REQUIRED HGL PROFILES

The plotted HGL of Figure 6-4 represents the hydraulic profile of the system specifically when both taps are open. Naturally, there will be a different profile if just Tap # 1 is open, or just Tap # 2 is open, or if both taps are closed (ie- the static profile). Normally, it is not necessary to calculate the HGL profiles for the various combinations of open/closed taps in a system. The HGL should only be plotted for the two extremes: all taps open, and all taps closed. As can be seen in Figure 6-4, both of these cases have been plotted on the single profile. This allows the designer to easily determine points of high and low pressure in the system, to ensure that they are within allowable limits (as will be discussed in Sections 6.14 - 6.16).

6.13 RESIDUAL HEAD: Excess Energy

The significance of residual heads at tapstands, 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 flow has reached the discharge point. It represents excess gravitational energy.

Installing a control valve at the discharge point will burn off residual head. (For this purpose a globe valve, not a gate valve, should be used).

Whilst this will reduce the quantity of flow, it may produce more desirable pressure characteristics within the pipeline.

A more specific discussion on design residual heads follows:

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 in Figure 6-5:

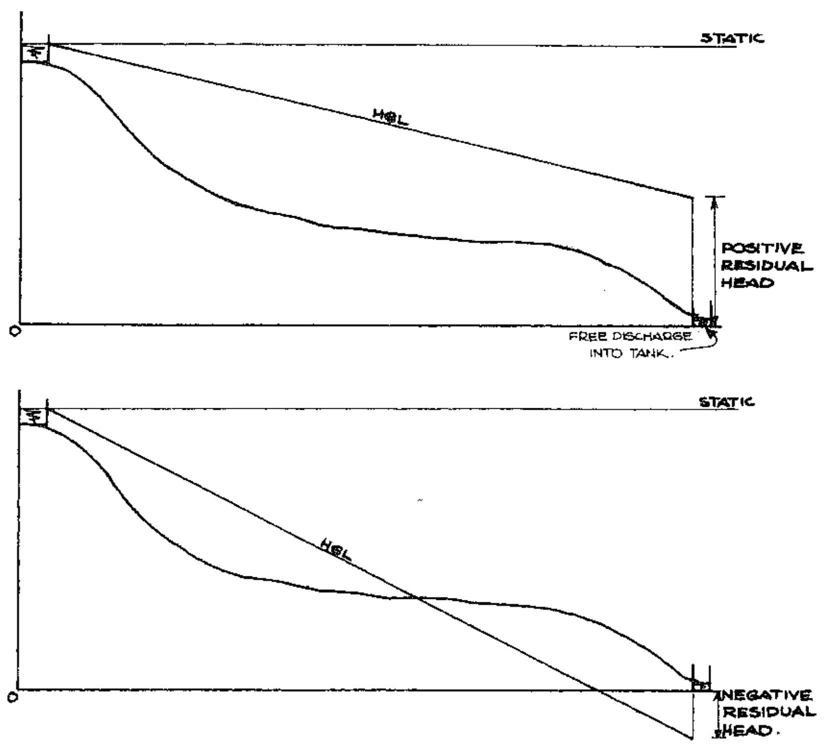


FIGURE 6-5 EXAMPLES OF POSITIVE & NEGATIVE RESIDUAL HEADS

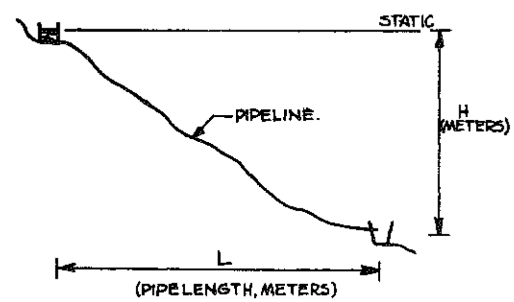
Negative residual head: This indicates that there is not enough gravitational energy to move the deisred quantity of water, hence this quantity of water will not flow. The HGL must be replotted using a smaller flow and/or larger pipe size.

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 through the pipe; as flow increases, the frictional headlosses will decrease the residual head. The flow will increase until the residual head is reduced to zero.

Natural flow: When the residual head of a pipeline discharging freely into the atmosphere is zero, then the maximum amount of flow is moving through the pipe. This is the natural flow of the pipe, and is the absolute maximum flow that can be moved by gravity. The natural flow of the pipe can be controlled by selective pipe sizing.

Figure 6-6 shows the calculation of the natural flow of an example pipeline.



NOTES:

1. H = VERTICAL ELEVATION BETWEEN UPPER AND LOWER BREAK-PRESSURE POINTS (I.E. MAX AVAILABLE HEADLOSS)

2. FREE DISCHARGE OF FLOW INTO LOWER TANK.

EXAMPLE:

WHAT IS THE NATURAL (MAXIMUM) FLOW THROUGH THE PIPELINE SECTION SHOWN, WHERE:

H = 50M.
L = 472 M.
PIPE = CLASS III 32MM. HDP.

SOL'N

THE NATURAL FRICT'L FACTOR.

$FN = \frac{50(100)}{472} = 10{\cdot}6\ m/100m$

FROM THE FRICT'L HEADLOSS TABLE, IT IS DETERMINED THATA HEADLOSS FACTOR OF 10.6m/100m FOR CLASS III 32mm. HDP PIPE OCCURS FOR A FLOW OF 1.00 LPS

FIGURE 6-6 CALCULATION OF THE NATURAL FLOW OF A PIPELINE

If the natural flow of a pipeline is greater than the safe yield, then the pipe will drain faster than it can be filled, and the result will be that the pipe will not flow full. In such a case, the HGL will lie on the surface of the water inside the pipe. A non-full flowing pipe is not under any pressure (except where the pipe flows full in U-profiles). If there are no tapstands located along a pipeline section that is not flowing full, then this is of no consequence.

However, if there is a tapstand, then it is very important that the pipeline be kept flowing full (ie- under pressure) to ensure the proper functioning of the tap.

Pipelines that otherwise will not flow full must have a control valve at the discharge point. This control valve will burn off the residual head, rather than allowing the flow to increase too much. The control valve is adjusted until the desired flow is discharged; at that setting, it is burning off exactly the correct amount of head.

In practice, control valves are adjusted under the hydraulic conditions where all taps are opened. As mentioned earlier, different HGL profiles will occur when different combinations of taps are opened and closed. For each possible combination, new residual heads will occur at the discharge points. Since it is not desirable to have the villagers constantly re-adjusting control valves every time a tap is opened or closed, the actual discharges will fluctuate. However, such fluctuations will be small and are negligible.

The amount of frictional headloss of flow through a valve is reported as the equivalent pipelength of the valve (see Section 6.10). Figure 6-7 given an example of calculating the equivalent pipelength of a valve, and calculates the fluctuating flows for the example system of Figure 6-4.

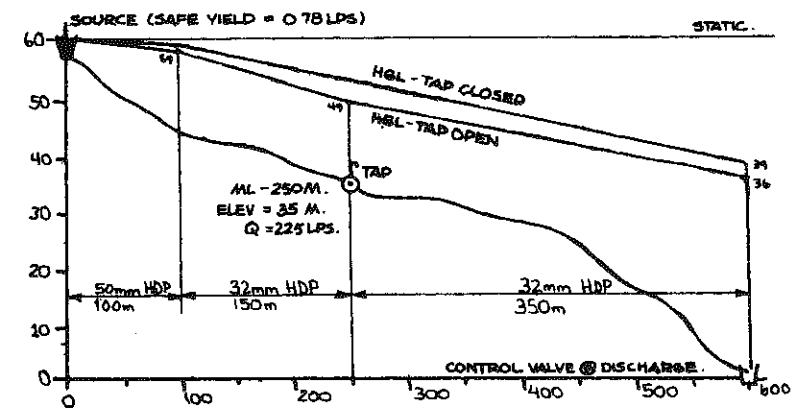
Since every tapstand requires some amount of residual head, then it is obvious that every tapstand requires a control valve. Control valves at discharges into reservoir or break-pressure tanks are only required where it is necessary to keep a specific flow in the pipeline, or to keep the upstream section of the pipeline flowing full (due to tapstands or branchpoints along that section). Without the control valves, the desired flow cannot happen in the pipeline, and the real hydraulic profile will not match the plotted HGL.

6.14 MAXIMUM PRESSURE LIMITS

As discussed already, it is seen that pipe sizes are selected because of frictional headloss considerations. However, there is yet another consideration which determines what type of pipe must be selected. This consideration is pressure, and will dictate whether Class III HDP pipe, Class IV HDP pipe, or galvanized iron (GI) pipe must 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 for each of these pipes is discussed below:

Class III HDP pipe: Maximum pressure rating = 6 kg/cm^2 (60 meters of head). This is the standard pipe used in Nepal where pressures do not exceed 60 meters of head. Since the other classes of pipe are much more expensive, the system should be designed to use as much Class III as possible.

Class IV HDP pipe: Maximum pressure rating = 10 kg/cm^2 (100 meters of head). This class is used where pressure exceed 60 meters of head but not 100 meters. Its wall thicknesses are greater, which allow it to



TAP OPEN

First Reach (Source--Tap)
Q= 0.78 LPS
L= 100m of 50mm HDP + 150m of 32mm HDP
Headlosses= 1m (50mm HDP section) + 10m (32mm HDP section)
Residual head @ Tap= 14m; Tap flow= 0.225 LPS

Second Reach (Tap--Tank)
Q= 0.55 LPS
L= 350m of 32mm HDP
Headloss= 13m
Residual head @ Tank discharge= 36m

This residual head at the tank discharge will be exactly burned off when the control valve at the discharge is adjusted to allow precisely 0.55 LPS into the tank. At this setting, the equivalent pipelength of the valve is 974m (ie- the length of 32mm HDP pipe required to burn off exactly 36m of head at 0.55 LPS flow).

TAP CLOSED

The equivalent pipeline of the system is:

100m of 50mm HDP
500m of 32mm HDP
974m of 32mm HDP (the equivalent pipelength of the valve)

To learn the new discharge flow into the tank, it is necessary to calculate the natural flow of the equivalent pipeline (ie- the flow at which 60m of head will be burned off by 100m of 50mm HDP + 1474m of 32mm HDP). By trial-and-error calculations and interpolations from the Frictional Headloss Table, the flow is found to be about 0.575 LPS. At this flow, the headlosses are:

100m of 50mm HDP @ 0.46 m/100m =	0.46m
500m of 32mm HDP @ 4.05 m/100m =	20.25m
974m of 32mm HDP equivalent pipelength @ 4.05 m/100m	=39.44m
TOTAL HEADLOSS	=60.15m

Thus, when the tap is open, the discharge into the tank will be 0.55 LPS, and when the tap is closed, the discharge will be slightly less than 0.575 LPS.

FIGURE 6-7 EQUIVALENT PIPELENGTH

withstand greater pressures, but it is much more expensive than Class III and therefore should not be used except where pressure requires it (it should not be used because of more suitable headloss factors).

GI pipe: Maximum pressure rating = 25 kg/cm^2 (250 meters of head). Galvanized Iron pipe used in CWS projects in Nepal is manufactured in India. GI pipe is used where pressures exceed 100 meters of head, or where proper burial of the pipeline is not possible. Current LDD policies set limits on the amount of GI pipe to be used in a project, therefore consultation with the regional engineer is necessary when a system appears to require a lot of GI pipe.

In all the above pressure ratings, for HDP pipe as well as GI pipe, there is a large safety factor. Thus, the above pressures can be safely exceeded by a few meters, but only when absolutely necessary. In the case of HDP pipe, the manufacturers state that the working lifetime of the pipe is 50 years when it is properly joined, buried, and pressures do not exceed the class rating. In the case of GI pipe, the safety factor is even larger, but it must be kept in mind that the pipe corrodes over the years, reducing wall thicknesses and therefore reducing its strength.

6.15 U-PROFILES & MULTIPLE PIPELINES

A special pressure problem to many systems in mountainous regions is the U-shaped profile, similar to the example below:

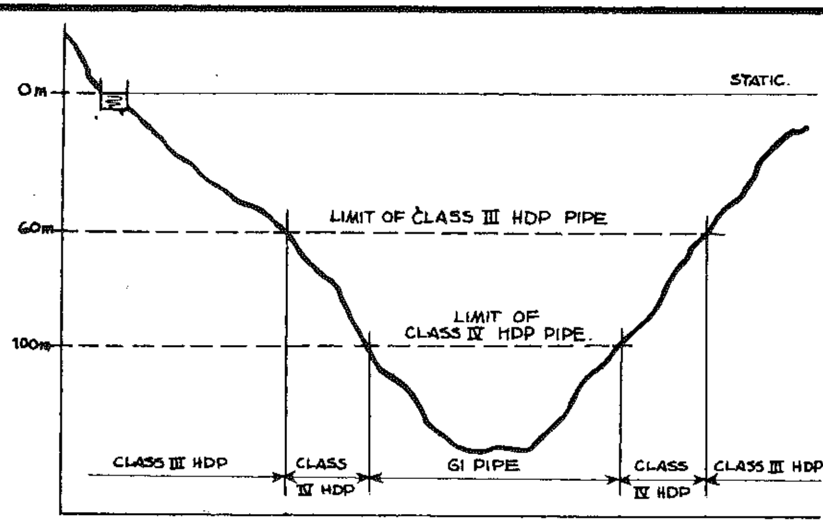
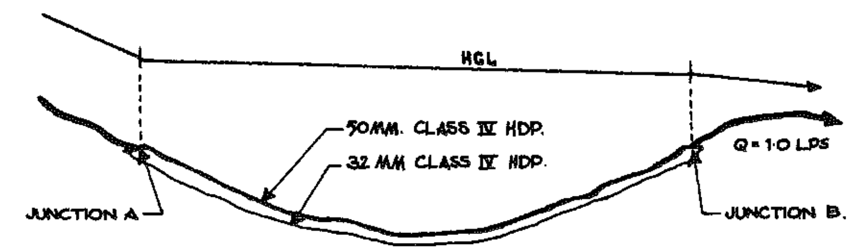


FIGURE 6-8 EXAMPLE OF U-PROFILE

It is apparent from the figure above that, under static conditions, the pressures in U-profiles can be quite high. Sections where the pressures exceed 60 meters of head will require Class IV HDP pipe, and where there is more than 100 meters of head GI pipe will be required.

Although Class IV is usually available in all sizes, there may be times when a particular Class IV size cannot be had. In such cases, it is possible to select a combination of smaller pipe sizes to be put down in parallel that will provide suitable headlosses (such a combination may be even less expensive than a single larger pipe size). Figure 6.9 gives the procedure for determining how the flow will divide itself between two pipes of unequal diameters.



The pressure at Junction A must be equal for all three pipes, since they are all joined at a common point, and likewise the pressure at Junction B must also be the same for all pipes. This implies that both pipes in the multi-pipe section must lose equal amounts of head. As these pipes are equally long, then there must be an equal rate of frictional headloss in both pipes. Thus, the flow will automatically divide itself between the two pipes such that each pipe has a frictional headloss factor equal to the other.

In the example drawn above, a flow of 1.0 LPS will flow through a multi-pipeline of 32mm and 50mm HDP (both of Class IV series). Consulting the frictional headloss table for these two pipe sizes, it can soon be determined that the only way that the flow can divide itself is approximately as follows:

Class IV 32mm HDP @ 0.225 LPS
" " 50mm HDP @ 0.75 LPS

It can be seen that for the above flows, the frictional headloss factor for both pipes would be approximately 1.1 m/100m.

This same principle applies also to multi-pipeline section of three or more pipe sizes.

FIGURE 6-9 DIVISION OF FLOW BETWEEN PIPES OF UNEQUAL SIZES

6.16 MINIMUM PRESSURE LIMITS

It is possible when plotting a HGL to discover that, due to the topographical profile of the pipeline, the HGL will actually "go underground;" that is, it will cross below the goundlevel profile and pass some distance underground before emerging again. An example is shown in Figure 6-10:

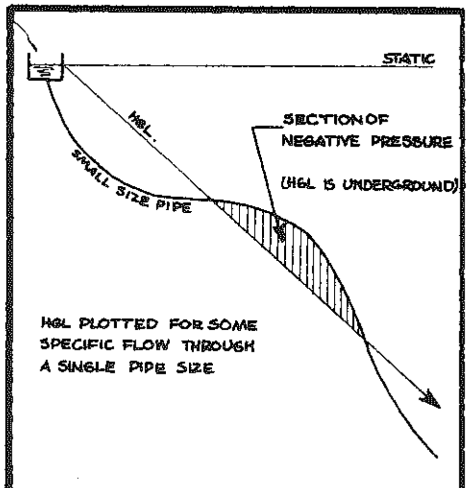


FIGURE 6-10
IMPROPERLY-DESIGNED HGL

The pressure in the pipe along the section where the HGL is underground is a negative pressure (measured as "negative head"). Negative pressure in the pipeline means that the water is being siphoned through (ie- sucked from below rather than pushed from above), a condition that is undesirable in CWS systems. Such negative pressures can suck in surrounding, polluted groundwater via leaky joints. Large negative pressures can also cause problems with dissolved air in the water (such air can come out of solution in the water and form trapped pockets of air at high points in the pipeline; more on air-blocks in the next chapter).

Therefore, as a general standard design, do not design any system where the HGL will fall less than 10 meters above the ground, except when unavoidable. Never allow the HGL to go underground at all.

Figure 6-11 shows the same profile, with pipe sizes selected to keep the HGL at least 10 meters above the ground.

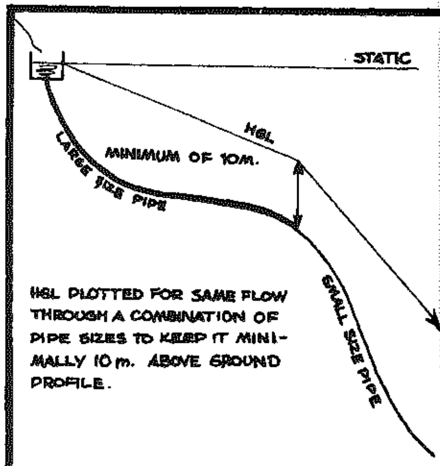


FIGURE 6-11
PROPERLY-DESIGNED HGL

6.17 VELOCITY LIMITS

The velocity of flow through the pipeline is also another matter of consideration. 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 are :

maximum: 3.0 meters/second
minimum: 0.7 meters/second

The corresponding flows for the various sizes and classes of HDP pipe are (in LPS):

	20mm	32mm (III)	32mm (IV)	50mm (III)	50mm (IV)	63mm (III)	63mm (IV)	90mm (III)	90mm (IV)
Maximum:	0.60	1.85	1.62	4.64	3.96	7.33	6.27	15.00	12.76
Minimum:	0.14	0.43	0.38	1.08	0.92	1.71	1.46	3.50	2.98

The frictional headloss tables indicate low flows with an asterisk (*), and do not give headloss factors for flows greater than the recomended ones.

When a pipeline carries a low flow, provisions must be made for sedimentation problems: a sedimentation tank should be built at the intake site, and washouts located at strategic low points to allow flushing out of settled matter. Refer to Chapter 12 for information on sedimentation tanks, and to Chapter 7.6 for information on locating washouts.

6.18 SUMMATION

This chapter has presented the design methods needed to select pipe sizes and classes, and how to arrange the pipe to keep the HGL within acceptable limits above the ground profile. One more final consideration, that of air-blocks, must be discussed. This will be done in Chapter 7; and then Chapter 8 will present the specific procedures for turning a topographic survey into a properly designed system.

7. AIR-BLOCKS & WASHOUTS

7.1 INTRODUCTION

This chapter considers the details of determining whether or not a pipeline is likely to be affected by trapped pockets of air which could interfere with the flow. If the designer determines that his system is a likely victim of air-blocks, he can then refer to Technical Appendix B for the analysis and procedures needed to deal with these air-blocks.

The chapter also discusses washouts, which allow settled sediments to be periodically flushed out of the pipeline.

7.2 AIR-BLOCKS, Introduction

An air-block is a bubble of air trapped in the pipeline, whose size is such that it interferes 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 within 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.

Generally, there will be no problems 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 7-1 below:

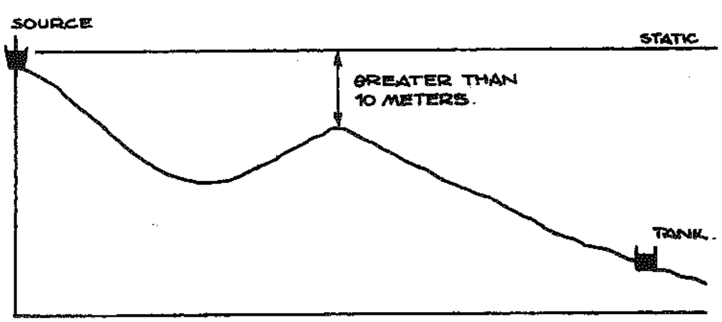


FIGURE 7-1 PROFILE WHERE AIR-BLOCKAGES WILL NOT INTERFERE WITH FLOW

Air-blocks analysis should be done in U-profile systems similar to that shown in Figure 7-2 below:

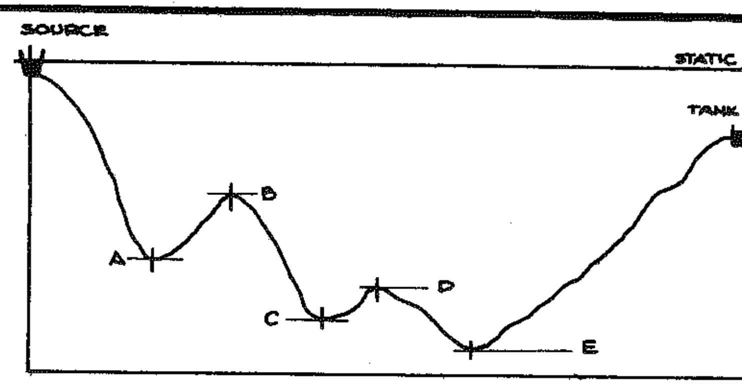


FIGURE 7-2 PROFILE WHERE AIR-BLOCKAGES MAY INTERFERE WITH FLOW

7.3 AIR-BLOCKS: Pipeline Design Practices

These are guidelines for arranging pipe sizes in such a way so as to minimize trapped air and potential air-blocks. Only after such an arrangement has been analysed and found inadequate should air-valves be installed.

1) Arrange the pipe sizes to minimize the frictional headloss between the source and first air-block.

2) Use larger-sized pipe at the top, and smaller-sized pipe at the bottom of the critical sections where air is going to be trapped (sections BC and DE in Figure 7-2). Pipe sizes elsewhere do not affect the air-blocks.

3) The "higher" air-blocks (ie- closer to the static level) are the more critical ones. Eliminate or minimize them first.

7.4 AIR-VALVES

Airvalves provided by UNICEF are sturdy devices, and operate automatically. Maximum pressure rating is 60 meters of head. Details of installation are shown in Figure 7-3.

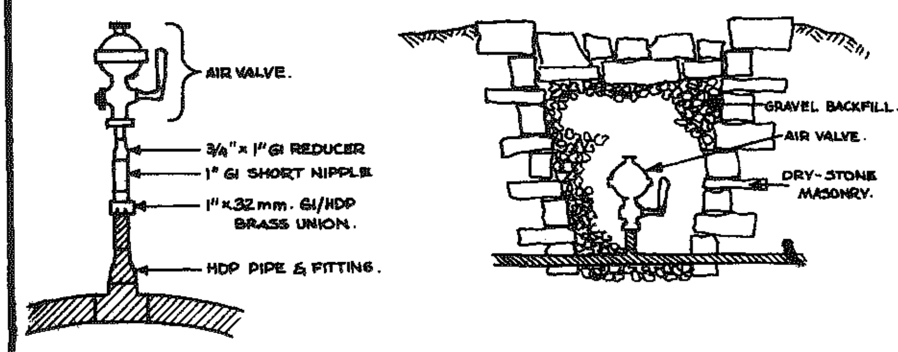


FIGURE 7-3
DETAILS OF AIR-VALVE & INSTALLATION

7.5 ALTERNATIVE AIR-RELEASES

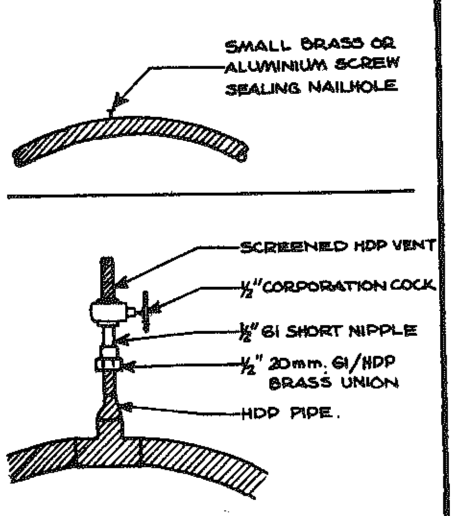


FIGURE 7-4
ALTERNATIVE AIR RELEASES

At times when the above air-valves are not available, there are two alternative methods for allowing trapped air to be released from the pipeline: install a normal control valve, or puncture the pipe with a nail and seal it off with a brass or aluminum screw. Although these alternative methods are not as expensive as an air-valve, they are not automatic, and require manual operation by the villagers. At times when the pipeline is being refilled with water, the valve is opened (or the screw is removed), allowing trapped air to escape. To discourage tempering with these air-release devices, they should be well buried (removing the handle from the valve will also keep unauthorized persons from opening it).

7.6 WASHOUTS

Over a period of time, suspended particles carried in the flow will tend to settle out, particularly at low points in the pipeline or where the flows are low enough so that the flow velocity drops below 0.7 meters/second. Reservoirs usually allow most of these particles to settle, but pipeline 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 should be located at the bottom points of major U-profiles, especially those upstream from the reservoir tank. 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.

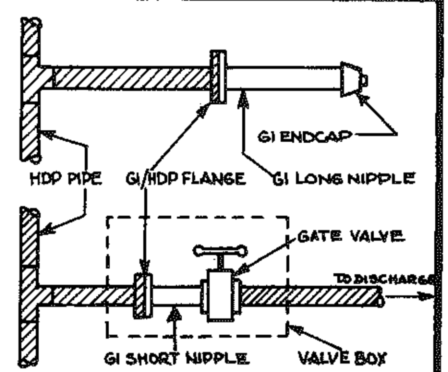


FIGURE 7-5
WASHOUT DESIGNS

The washout pipes should be of the same size as the pipeline at that point. Endcap-type washouts will require that the pipeline will be completely drained before the end-cap can be replaced (since it is impossible to put it back on while there is water gushing out of it), which is not so with a washout that has a gate valve (a globe valve is definately not suited for this type of work). Handles should be removed and valves well buried to discourage tampering. Endcaps should be torqued lightly with a wrench (so that they cannot be removed by hand) but not extremely tightly, since they will tend to rust onto the pipe and be very difficult to remove at a later time. Figure 7-5 shows some washout designs.

8. PIPELINE DESIGN

8.1 INTRODUCTION

The concepts of hydraulic theory, descriptions of various factors which influence flow, techniques for determining pressures and the HGL, have all been presented so far. In this chapter, all of it will be brought together to show how it is practically applied in the design of a real system.

The pipeline design phase begins with the graphic plotting of the topographic survey (from the initial survey of the system) and ends when all sections of the pipeline (ie- mainline, sourcelines, branch-lines, and taplines) have been designed in their final form. Blueprints are then made of the design.

This chapter will present standards and guidelines for preparing the pipeline drawings, example designs for mainlines, branchlines, source collection lines (ie- for systems with multiple sources), and a pipeline section of combination pipe sizes.

8.2 PIPELINE DRAWINGS

The purpose in plotting the profile is to create a visual, easy-to-understand picture of the topographic elevations along the pipeline. Because the profile contains so much information on it, it is necessary that it be carefully laid out so that it is not cluttered, sloppy, difficult to read, or incomplete.

Graph profile: The profile is initially plotted on graph paper of centimeter divisions. Vertical scale should be either 1 cm = 5 meters or 1 cm = 10 meters; horizontal scale 1 cm = 50 meters or 1 cm = 100 meters. Each sheet should contain a title block (as shown in Figure 8-1) and the axes laid out as shown in Figures 8.2 and 8.4. The profile, title block, axes, and tapstand sites are done in ink, but tank locations and HGLs are worked in pencil until properly designed, and then inked. The designs must be approved by the LDD regional engineer.

Tracing profile: When the pipeline design is finished and approved, it is traced onto tracing paper. Dark-color ball-point pens, soft-tip markers, or drafting pens should be used. Lettering must be neatly printed. All tanks, taps, washouts, air-valves, branch points, strategic points, etc, should be labeled with distance and elevation. Tapstand flows must be indicated if they are not the standard 0.225 LPS. Pipe sizes and lengths must be indicated.

Blueprinting: When the final tracing is completed, it is ready for blueprinting. A sheet of ammonia-sensitized paper, slightly larger than the tracing paper, is laid out on the tracing paper and then both are rolled through a fluorescent-tube light box. The ammonia paper is then slipped into another air-tight box, where it is exposed to ammonia

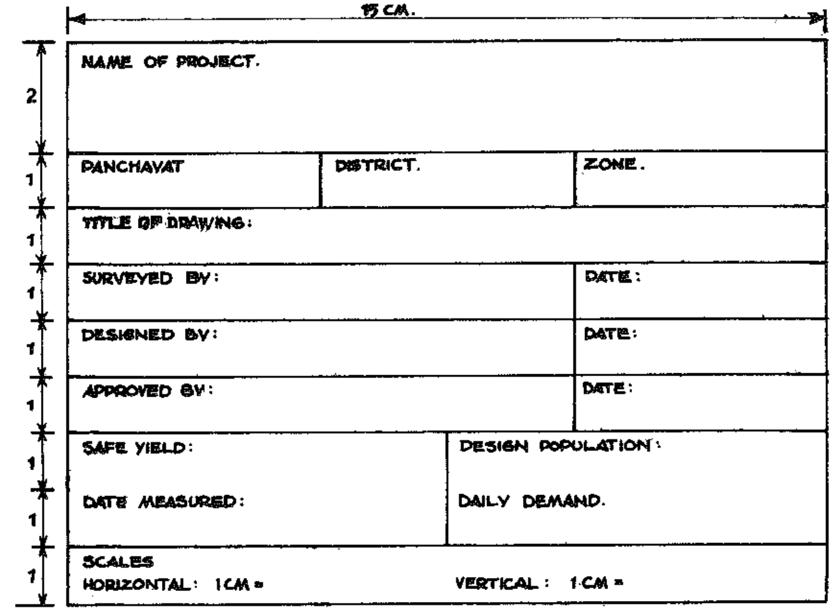


FIGURE 8-1 TITLE BLOCK

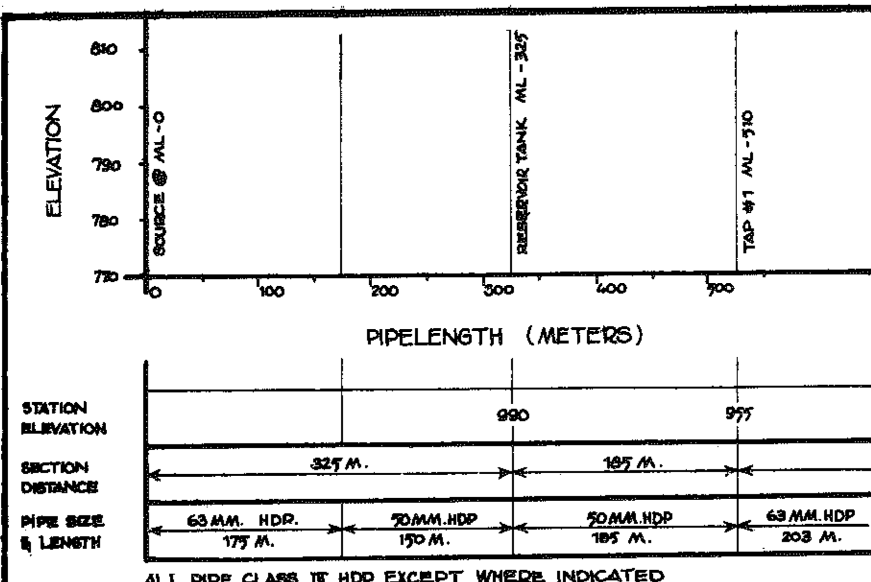


FIGURE 8.2
PORTION OF SAMPLE DESIGN SHOWING PROFILE AXES

vapors for a few minutes to "develop" the blueprint. Size of a blueprint page should be about the same as the graph paper size (larger-sized sheets are awkward to handle). The total number of copies of each blueprint is determined by LDD, so consult the regional engineer.

General plan view & key plan: In addition to the profile design, blueprints should be made of the general plan of the system, which shows the rough layout of the system, with village landmarks indicated. A key plan of the system is also made, showing the relative arrangements of tanks, control valves, branchlines, tapstands, etc. An example is shown in Figure 8-3.

NOTE: Since these design examples were worked out, new frictional headloss tables for HDP pipe were obtained. The new tables are now in the back of this book, and are not the ones referred to in the following examples.

8.3 DESIGN EXAMPLE: Mainline

Figure 8-4 will be used as the design example of a mainline. The basic procedure for designing a pipeline is to divide it at strategic points (usually tanks and tapstands). The pipeline section between each of these points is called a reach. For each reach, determine the desired amount of head to be burned off, and the length of pipeline, and with these determine the desired frictional headloss factor. From the Headloss Table, select the pipe size which is closest to that desired frictional factor. If no size is suitable, then using two different pipes in the reach can be done. The method for determining the length of these combination pipes is given in Section 8-6.

When designing the pipeline, the designer can begin at the source and plot his way downstream, or begin at the end and plot his way upstream, or begin at the ends and plot towards the middle, depending upon his intuitive feelings. With experience, he will develop more intuition at where to best begin. In this example, however, plotting will begin at the source and proceed downstream.

Reservoir calculations:

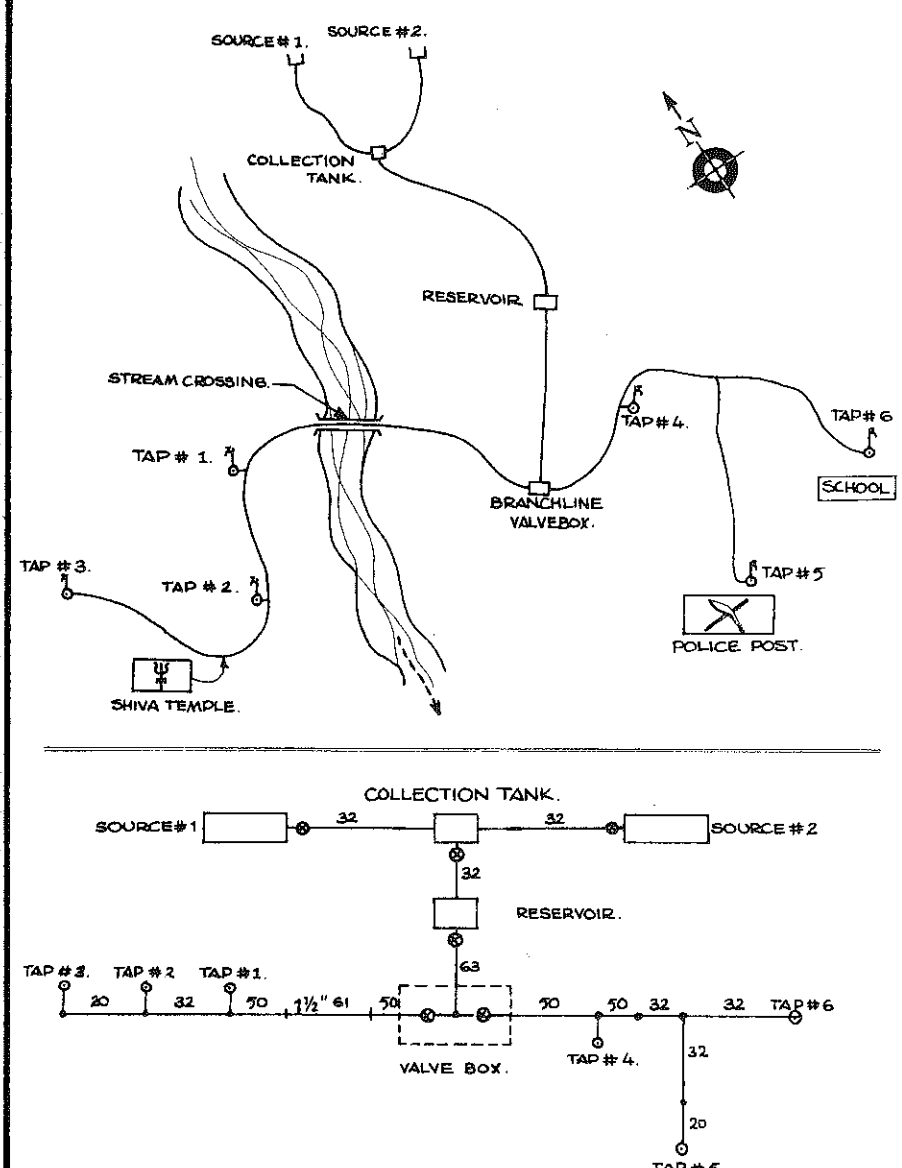
Safe yield of source = 1.40 LPS

Demand by 6 taps @ 0.225 LPS = 1.35 LPS

Therefore reservoir tank not required

Preliminary pressure analysis:

This profile contains a major U-profile, so it is best to begin by examining it there. If Class III pipe were used along the bottom of the U-profile, the pressure in the pipeline would exceed 60 meters of head before the flow could make it back out of the U-profile. Therefore, Class IV pipe must be used, with a break-pressure tank located 100 meters above the bottom of the U-profile. This tank would therefore be located at ML-600, elevation of 900 meters. The Class IV pipe would have to begin at ML-870 (which is at an elevation 60 meters below the break-pressure tank) and run until ML-1720.



NOTES:
1. ALL PIPE CLASS III HDP, SIZE IN MM.
2. ALL CONTROL VALVES SHOWN ARE GATE VALVES OF CORRESPONDING PIPE SIZES.
3. SEE DESIGNS FOR INTAKES, COLLECTION TANK, RESERVOIR, & VALVEBOX DETAILS.

FIGURE 8-3 PLAN VIEW & KEY PLAN OF EXAMPLE SYSTEM

From the break-pressure tank to the source is 100 meters of elevation. It would be possible to use Class IV in this section, but it actually is less expensive to install another break-pressure tank and use Class III pipe. Thus, another break-pressure tank will be required 60 meters lower than the source. This would put it just downstream from Tap #1 (for the sake of convenience, let's locate the break-pressure tank 5 meters lower than the tap, which would place it at ML-400). This will be break-pressure tank #1 (BT #1), and the next one downstream will be break-pressure tank #2 (BT #2).

A third break-pressure tank must be 60 meters lower than BT #2, which puts it at an elevation of 840m, between Taps #2 and #3.

The next break-pressure tank (BT #4) must be 60 meters lower than BT #3, which puts it at an elevation of 780m, between Taps #3 and #4.

From BT #4, there is 80 meters of vertical distance to the last tap, which would require either another break-pressure tank or Class IV pipe. In this case, it is rather likely that 20mm HDP pipe will be used to reach the last tap, and such pipe is provided only in Class IV. So for the time being, the design will proceed assuming that no break-pressure tank will be required (this will have to be specially checked once the total design is completed, however).

So the four break-pressure tanks have been tentatively located as follows:

BT #1: ML-400, elev 950m

BT #2: ML-600, elev 900m

BT #3: ML-2240, elev 840m

BT #4: ML-2540, elev 780m

and Class IV pipe: ML-870 (elev 840m) - ML-1720 (elev 840m)

Now, the pipeline will be designed, reach-by-reach, beginning at the source.

<u>First Reach (Source - Tap #1)</u>

Design flow = 1.35 LPS (flow for 6 taps)
Pipelength: 380 - 0 = <u>380 meters</u>

1000m (elev of HGL @ source)
-955m (elev of Tap #1)
<u>-15m</u> (desired residual head @ Tap #1)
30m DESIRED FRICTIONAL HEADLOSS

$$\frac{\text{desired headloss}}{\text{pipelength}} = 30/380 \times 100\% = 7.89\% \quad \text{(desired frictional headloss factor)}$$

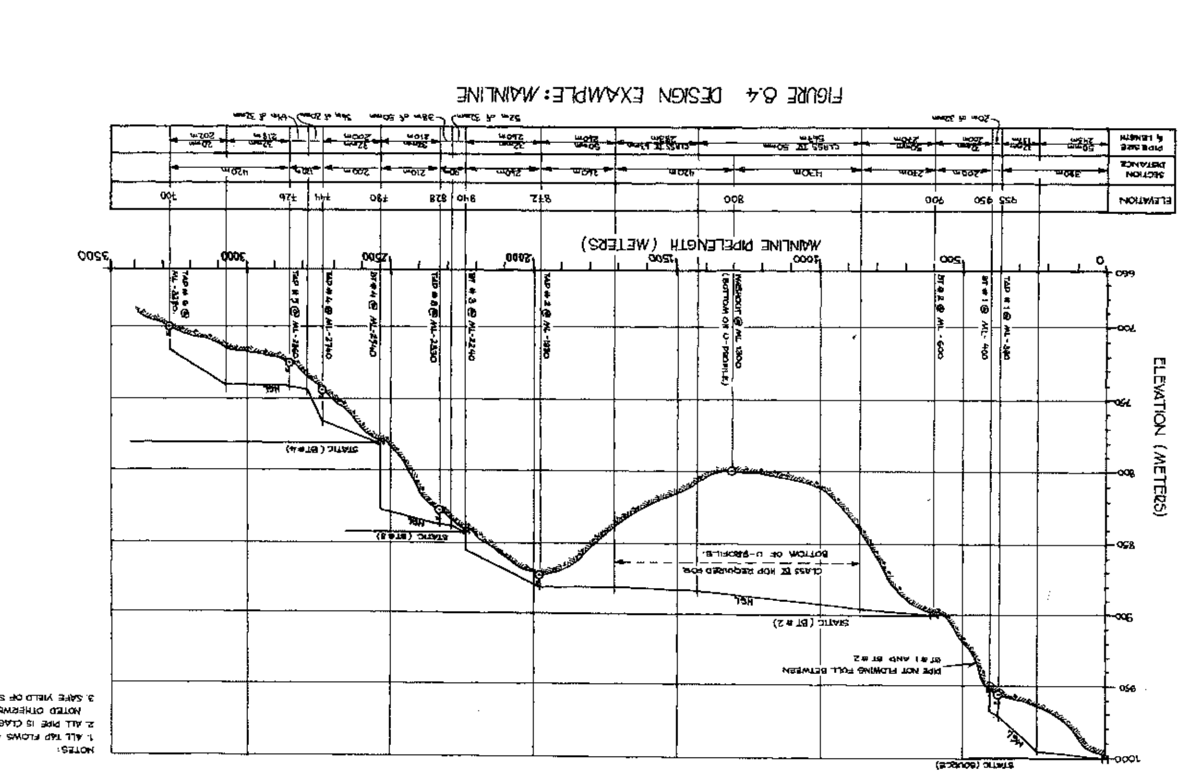


FIGURE 6.4 DESIGN EXAMPLE: MAINLINE

Consulting the HDP Frictional Headloss Table (at the back of this Handbook) for a flow of 1.35 LPS, it is seen that the headloss factor of 32mm HDP pipe is too high (18.15%) while that of 50mm HDP pipe is too low (2.08%). Thus, a combination of both of these pipe sizes is needed to produce exactly the desired headloss. Calculations* indicate the following lengths are needed:

243m of 50mm HDP @ 2.08% creates	5m	of headloss
137m of 32mm HDP @ 18.15% creates	25m	of headloss
	30m	TOTAL HEADLOSS

Elev of HGL @ Tap #1 = 970m

Second Reach (Tap #1 - BT #1)

Design flow = 1.125 LPS (flow for 5 taps)
Pipelength: 400 - 380 = 20m

970m	(elev of HGL @ Tap #1)
-950m	(elev of BT #1)
-10m	(desired residual head @ BT #1)
10m	DESIRED FRICTIONAL HEADLOSS

10/20 x 100% = 50% desired frict'l factor

Consulting the Headloss Table, it is seen that no factor is given for this flow for 20mm HDP pipe (because the flow velocity would be too high), therefore there is no choice but to use 32mm HDP pipe:

20m of 32mm HDP @ 12.6% creates 3m of headloss

actual residual head = 17m (acceptable)

BT #1 must be constructed with a globe valve at its discharge, adjusted to allow exactly 1.125 LPS flow. For convenience sake, a gate valve can be installed on the outlet of the tank, permitting the downstream pipeline to be shut down without cutting off the flow for Tap #1.

The HGL is now at the surface level of the water in the tank, at 950m elevation.

Third Reach (BT #1 - BT #2)

Design flow = 1.125 LPS
Pipelength: 600 - 400 = 200m

Since there are no tapstands along this reach, there is no reason why the pipe must flow full. Select the smallest size that will allow the design flow through:

* see Section 8-6 for combined pipe calculation example

950m	(elev of HGL @ BT #1)
-900m	(elev of BT #2)
0m	(desired RH @ BT #2)
50m	MAXIMUM ALLOWABLE HEADLOSS

50/200 x 100% = 25% maximum allowable frict'l factor

32mm HDP is the smallest pipe size that has a frictional factor less than 25%, so this is the pipe size to be used. Since there is no need to maintain pressure in the pipeline, the pipe is allowed to discharge freely into BT #2. Gravity will drain the line faster than it will fill, so it won't flow full.

Fourth Reach (BT #2 - Tap # 2)

Design flow = 1.125 LPS
Pipelength: 1980 - 600 = 1380m (including 850m of Class IV)

900m	(elev of HGL @ BT #2)
-872m	(elev of Tap #2)
-7m	(minimum desired RH @ Tap #2)
21m	MAXIMUM DESIRED HEADLOSS

21/1380 x 100% = 1.52% desired frict'l factor

This reach will require 530m of Class III pipe and 850m of Class IV pipe. Once again, combination pipe sizes are required. Class III 50mm HDP pipe will be used for the entire Class III length:

530m of 50mm HDP @ 1.40% creates 7m of headloss

(therefore only 14m of allowable headloss left)

The proper combination of Class IV pipe is:

567m of 50mm HDP @ 2.12% creates	12m	of headloss
283m of 63mm HDP @ 0.70% creates	2m	of headloss
	14m	of total headloss

Thus, the pipe arrangement for the entire reach is:

270m of Class III 50mm HDP pipe
567m of Class IV 50mm HDP pipe
283m of Class IV 63mm HDP pipe
260m of Class III 50mm HDP pipe

Elev of HGL @ Tap #2 = 879m

A washout is located at the bottom of the U-profile, since it is a major low point in the system, and there is no reservoir tank to allow sedimentation to occur. The presence of a washout does not affect the hydraulic profile of the system (except when the washout is opened.).

<u>Fifth Reach (Tap #2 - BT #3)</u>

Design flow = 0.90 LPS (flow for 4 taps)
Pipelength: 2240 - 1980 = <u>260m</u>

879m	(elev of HGL @ Tap #2)
-840m	(Elev of BT #3)
-10m	(desired RH at BT #3)
29m	DESIRED HEADLOSS

29/260 x 100% = 11.2% desired frict'l factor

The only pipe size which gives a close frictional factor is 32mm HDP:

260m of 32mm HDP @ 8.9% creates 23m of headloss

actual residual head = 16m (acceptable)

BT #3 must have a globe valve at its discharge so that the exact desired flow of 0.9 LPS comes through. A gate valve may be installed on the outlet to cut off downstream flow without effecting the upstream taps.

elev of HGL @ BT #3 = 840m

<u>Sixth Reach (Tap #3 - BT #3)</u>

Design flow: 0.90 LPS
Pipelength: 2330 - 2240 = <u>90m</u>

840m	(elev of HGL @ BT #3)
-828m	(elev of Tap #3)
-7m	(minimum allowable RH @ Tap #3)
5m	MAXIMUM ALLOWABLE HEADLOSS

5/90 x 100% = 5.56% Maximum allowable frict'l factor

For this flow, 50mm HDP pipe must be used:

90m of 50mm HDP @ 0.99% creates 1 meter of headloss

actual residual head = 11m (acceptable)

elev of HGL @ Tap #3 = 839m

<u>Seventh Reach (Tap #3 - BT #4)</u>

Design flow: 0.675 LPS (flow for 3 taps)
Pipelength: 2540 - 2330 = <u>210m</u>

839m	(elev of HGL @ Tap #3)
-780m	(elev of BT #4)
-10m	(desired RH @ BT #4)
49m	DESIRED HEADLOSS

49/210 x 100% = 23.33% desired frict'l factor

Again there is no choice but to use 32mm HDP:

210m of 32mm HDP @ 5.3% creates 11m of headloss

actual residual head = 48m (acceptable)

This residual head is getting close to the maximum allowable limit of 56 meters. High residual heads increase wear and tear on control valves, reducing their lifetimes and requiring more frequent replacement.

elev of HGL @ BT #4 = 780m

<u>Eighth Reach (BT #4 - Tap #4)</u>

Design flow: 0.675 LPS
Pipelength: 2740 - 2540 = <u>200m</u>

780m	(elev of HGL @ BT #4)
-744m	(elev of Tap #4)
-15m	(desired RH @ Tap #4)
21m	DESIRED HEADLOSS

21/200 x 100% = 10.5% desired frictional factor

Again, there is no choice but for 32mm HDP:

200m of 32mm HDP @ 5.3% creates 11m of headloss

actual residual head = 25m (acceptable)

elev of HGL @ Tap #4 = 769m

<u>Ninth Reach (Tap #4 - Tap #5)</u>

Desired flow: 0.45 LPS
Pipelength: 2860 - 2740 = <u>120m</u>

769m	(elev of HGL @ Tap #4)
-726m	(elev of Tap #5)
-15m	(desired RH @ Tap #5)
28m	DESIRED HEADLOSS

28/120 x 100% = 23.33% desired frict'l factor

A combination of 32mm HDP and 20mm HDP pipe sizes are used:

67m of 20mm HDP @ 40% creates 27m of headloss
53m of 32mm HDP @ 2.56% creates 1m of headloss

actual residual head - 15m (perfect!)

elev of HGL @ Tap #5 = 741m

Tenth Reach (Tap #5 - Tap #6)

Design flow: 0.225 LPS
Pipelength: 3280 - 2860 = 420m

741m	(elev of HGL @ Tap #5)
-700m	(elev of Tap #5)
-15m	(desired RH @ Tap #6)
26m	DESIRED HEADLOSS

26/420 x 100% = 6.2% desired frictional factor

A combination of 32mm HDP and 20mm HDP is needed:

218m of 32mm HDP @ 0.78% creates 2m of headloss
202m of 20mm HDP @ 12.0% creates 24m of headloss

actual residual head = 15m (perfect!)

In this last reach, it can be seen on the profile that the final 16m of 32mm HDP pipe will be exposed to a static pressure greater than 60m of head. However, the maximum static pressure on this pipe would only be about 66m of head, which is a tolerable amount. However, the designer may also use Class IV 32mm HDP for some or all of this reach, or install another break-pressure tank. If he is not sure, the designer should consult with the LDD regional engineer.

Final Check

Once the designer has tentatively completed selecting pipe sizes, he must go back over the design, checking that at no point the pressures under static conditions are excessive. When this is done, he inks in the final HGL and tank locations, then gets the design approved by the LDD regional engineer.

8.4 DESIGN EXAMPLE: Branchline

Figure 8-5 is an example profile of a branchline with two taps (not related to the design example of Figure 8-4). In this example, the mainline has already been designed, so that the residual head of the branchpoint is known, as is the static level.

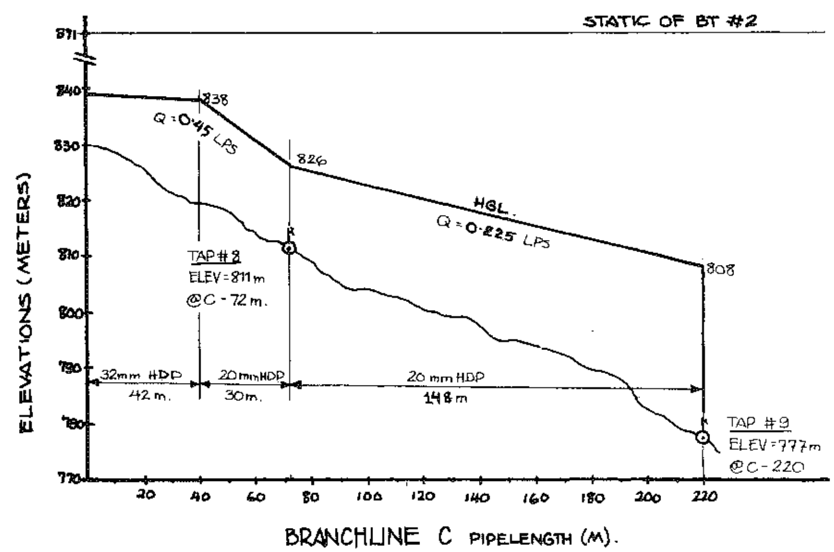


FIGURE 8-5
DESIGN EXAMPLE: BRANCHLINE

Branchline "C", First Reach (B'point - Tap #8)

Design flow: 0.45 LPS (flow for 2 taps)
Pipelength: 72 - 0 = 72 meters

839m	(elev of HGL @ b'point)
-811m	(elev of Tap #8)
-15m	(desired RH @ Tap #8)
13m	DESIRED HEADLOSS

13/72 x 100% = 18.1% desired frict'l factor

A combination of 32mm HDP and 20mm HDP pipes is needed:

42m of 32mm HDP @ 2.56% creates 1m of headloss
30m of 20mm HDP @ 40% creates 12m of headloss

actual residual head = 15m (perfect')

elev of HGL = 826m

Branchline "C", Second Reach (Tap #8 - Tap #9)

Design flow: 0.225 LPS
Pipelength: 220 - 72 = 148m

826m	(elev of HGL @ Tap #8)
-777m	(elev of Tap #9)
-15m	(desired Rh @ Tap #9)
34m	DESIRED HEADLOSS

34/148 x 100% = 23.0% desired frict'l factor

The only possible choice is 20mm HDP:

148m of 20mm HDP @ 12% creates 18m of headloss

actual residual head = 31m (acceptable)

The final pressure check for static conditions indicates that no pressures exceed the pressure ratings of the pipes. If there was such a point, then either break-pressure tank #2 (on the mainline) would have to be moved down (and thus require re-designing the mainline once more) or installing a break-pressure tank along the branchline itself

8.5 DESIGN EXAMPLE: Collection Lines

It is not uncommon to have a system which must combine several small sources to obtain a useful safe yield flow. In such cases, it is easiest to bring the individual sourcelines together at a common collection or sedimentation tank. This tank, of course, acts as a break-pressure point and the HGL would have to be plotted as such. If the sources were at different elevations, there would be no problem of hydraulic interference between the sources.

However, it is not always possible to install such a break-pressure point. In such cases, the sourcelines are joined together directly to the mainline, as shown in Figure 8-6 below:

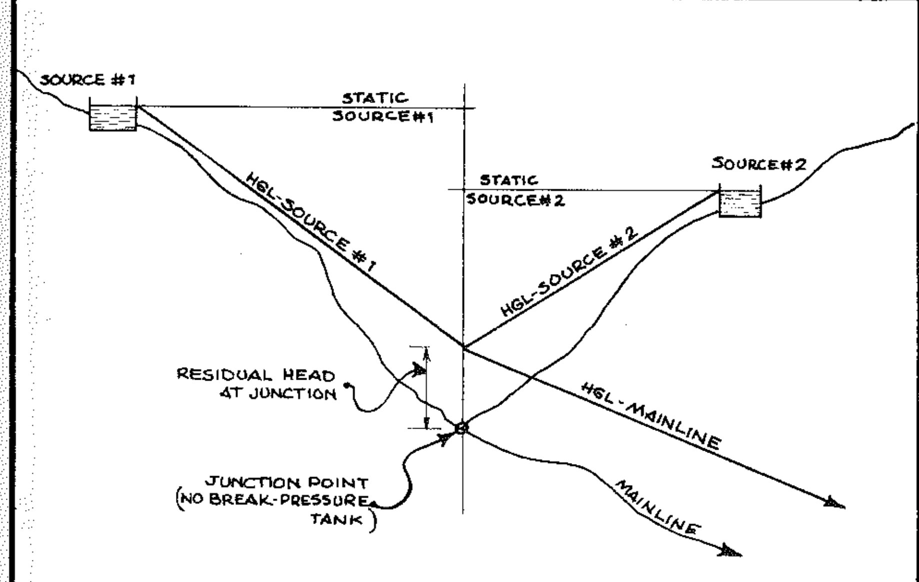


FIGURE 8-6 DESIGN EXAMPLE: COLLECTION LINES

With this type of junction, if the sources are at different elevations then it is possible that the pressure from one will interfere with the flow from the other.

The principle of properly joining the sources at a common point is to realize that the flow from each source will be such that there will be only one possible residual head at the junction. Thus it is necessary to design the sourcelines in such a way that they all meet at a common residual head at the junction.

Procedure: Plot the HGL from a single source to the junction. Then select the other pipe sizes of the other sources so that, for the desired flow out of each source, the HGLs all intercept the HGL of the first source; that is, they all have an equal residual head. From that point, continue plotting the HGL for the mainline using the total flows.

8.6 DESIGN EXAMPLE: Combination Pipe Sizes

When designing a pipeline section, there may be no single pipe size available that gives the desired frictional headloss factor. In that case, a combination of pipe sizes is used: one pipe which is "too small" and one which is "too large". The lengths of each pipe must be long enough so that the sum of the headloss of each is equal to the total desired headloss. Refer to Figure 8-7:

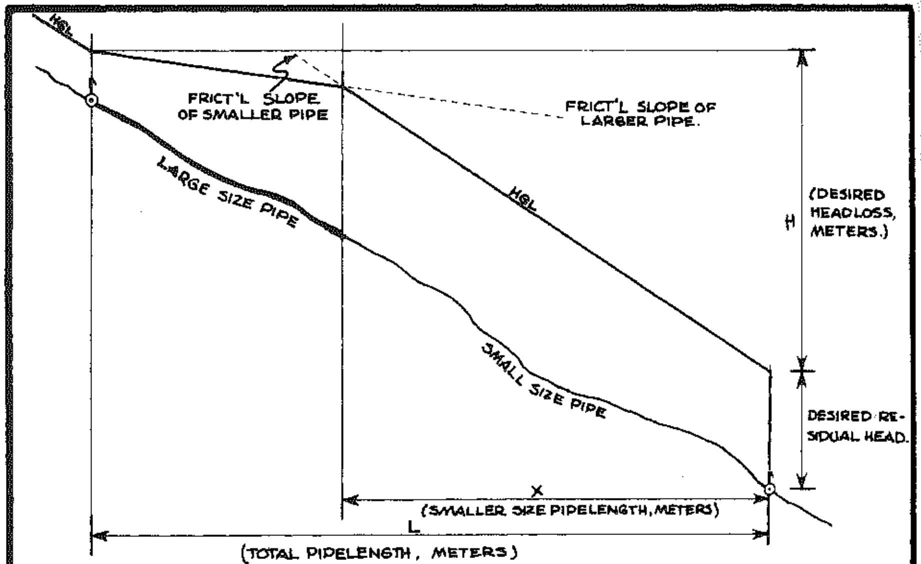


FIGURE 8-7 DESIGN EXAMPLE: COMBINATION PIPE SIZES

Since the total pipelength, design flow, and desired headloss are all known, the lengths of the two pipe sizes can be determined by the following equation:

$$X = \frac{100H - (Fl \times L)}{Fs - Fl}$$

Where:
H = desired headloss (m)
L = total pipelength (m)
X = small-size pipelength (m)
Fl = frict'l factor, large pipe (%)
Fs = frict'l factor, small pipe (%)

When the length of the smaller-sized pipe is calculated, it is then subtracted from the total pipelength to determine the length of the larger-sized pipe. See Technical Appendix C for the derivation of the above formula.

8.7 DESIGN EXAMPLE: Excessive Residual Head

There may be points in a system where the residual head at a discharge point is excessively high (ie- greater than 56 meters). This can particularly happen to tapstands located in positions such as shown in Figure 8-8:

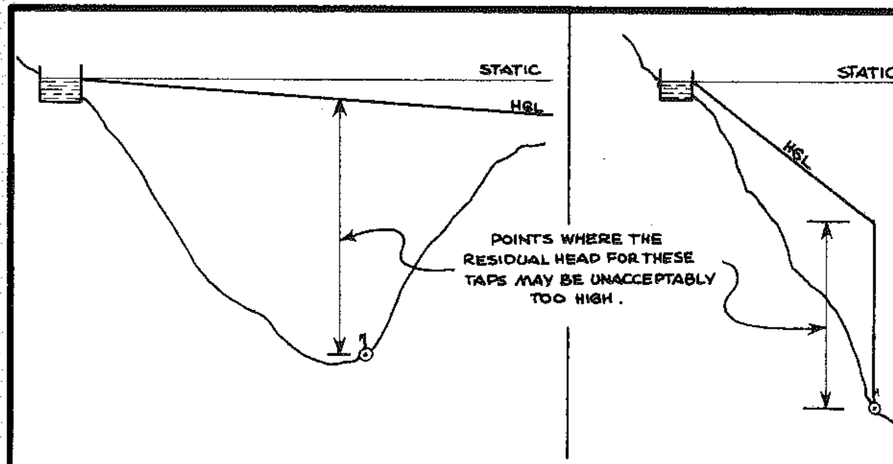
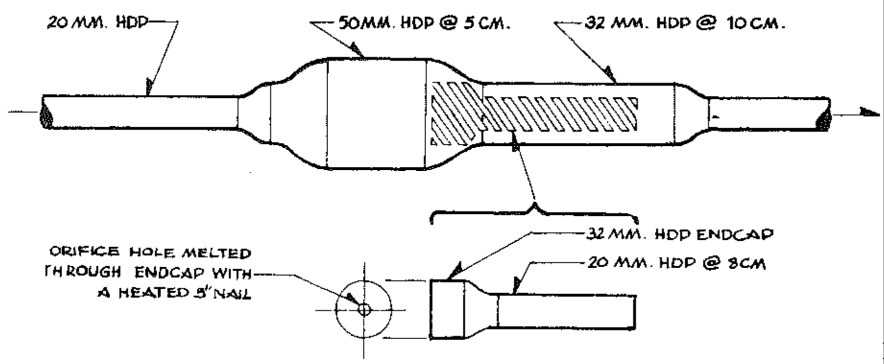


FIGURE 8-8 EXAMPLES OF HIGH RESIDUAL HEADS AT TAPS

For such cases, it is possible to install a device which creates high frictional losses in only a short length of pipeline. This sort of frictional diffuser can be easily manufactured in the field, using HDP pipe and fittings. A design for this is shown on the next page.



APPROXIMATE HEADLOSS:

$\boxed{H = 369\ Q^2}$ WHERE: H = HEADLOSS (M), Q = FLOW (LPS)

FIGURE 8-9
FRICTIONAL DIFFUSER

The orifice in the endcap is made using a hot 3" nail to melt a hole. For a flow of 0.225 LPS, this diffuser will create approximately 18 meters of headloss. Adding more holes decreases the headloss; if the headloss of a single diffuser is not enough, add a second one. The pipe section must be of Class IV HDP if the diffuser will be subjected to a static pressure greater than 60 meters of head.

The diffuser is installed just upstream of the tapstand, as shown in Figure 8-10:

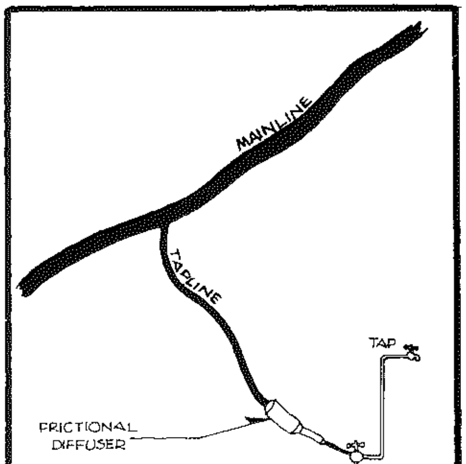


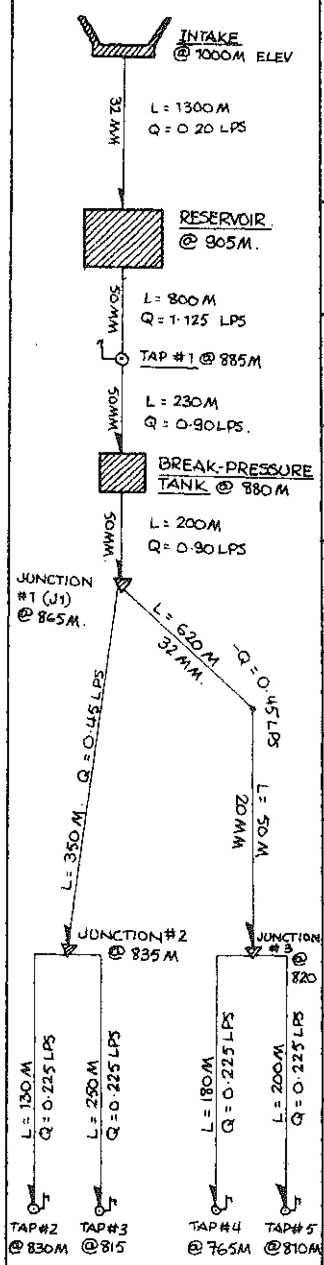
FIGURE 8-10
INSTALLATION OF DIFFUSER

Refer to Technical Appendix D for details on calculating the headlosses through an orifice, for designing similar frictional diffusers.

8.8 TABULATED PROCEDURE

Figure 8-11 shows a tabulated format procedure developed by LDD to organize the design of a pipeline. The format shown helps to keep the different numbers from becoming confused, and allows precise planning of the pipeline in each reach.

However, this procedure is not fool-proof. It is still necessary to plot the resulting HGLs on the graph profile, to determine that pressures and residual heads are within allowable limits at all points in the reach.



REACH STATION I	STATION II	① FLOW (LPS)	② PIPE LENGTH (M)	③ PIPE SIZE	④ FRICT'L FACTOR	⑤ HEAD LOSS	⑥ HGL @ STATION 1	⑦ ELEVATION OF STATION II	⑧ HGL @ STATION II ⑧=⑥-⑤	⑨ RESIDUAL HEAD OF STATION II ⑨=⑧-⑥	REMARKS
INTAKE	RESERVOIR	0.20	1300	32 MM.	–	–	–	–	905	–	FREE DISCHARGE (PIPE FLOW NOT FULL)
RES	TAP#1.	1.125	800	50 MM.	1.48%	12M	905	885	893	8	
TAP# 1.	BPT	0.90	230	50 MM.	0.99%	2M	893	880	891	11	HGL REDUCED TO 880 M
BPT	J1	0.90	200	50 MM	0.99%	2M	880	865	878	13	
J1	J2	0.45	350	32 MM	2.56%	9M	891	835	882	47	
J2	T2	0.225	130	20MM	12.0%	16M	882	830	866	36	O.K.
J2	T3	0.225	250	20 MM	12.0%	30M	882	815	852	37	O.K.
J1	*	0.45	620	32MM	2.56%	16M	878	*	*	*	COMBINATION OF 32MM & 20MM HDP
*	J3	0.45	50	20MM	12.0%	6M	*	820	856	36	
J3	T4	0.225	180	20MM	12.0%	22M	856	765	834	69	NEED FRICT'L DIFFUSER
J3	T5	0.225	200	20MM	12.0%	24M	856	810	832	22	O.K.
	TOTAL = 4310 M.										

FIGURE 8-11
TABULATED DESIGN EXAMPLE

9. SYSTEM DESIGN & ESTIMATES

9.1 INTRODUCTION

Once the profile has been plotted and the final pipeline sizing has been approved, the designer enters the next phase of planning the project: the extensive designing of the system components (such as intakes, reservoir tank, break-pressure tanks, tapstands, etc) and preparing the detailed estimates of material, labor, and money that will be required to implement the project.

In the past, most CWS projects in Nepal have been surveyed, designed, and constructed by the same person. This allowed for a fairly informal manner of designing a system, since the actual construction overseer was intimately familiar with the thoughts of the designer! Detailed plans were not so necessary, as long as he kept the rough design notes and calculations that he had made.

However, there is now an increasing trend in Nepal towards turning over the completed project design to a fresh person who will be the one who oversees its construction. In these cases, the overseer cannot get by with just an estimate sheet and a few cryptic notes in an unfamiliar format. Designers are now required to be more professional, and their designs more detailed, so that a person unfamiliar with the project can take over with minimal loss of information. Unless the designer specifically details just how he intended the system to be constructed, the overseer cannot be expected to build the system according to the designer's materials and cost estimate.

Because of the knowledge and experience that is building up, LDD will soon be able to create standardized designs for most components of a system. Such standardized designs will detail plan specifications, material requirements, labor estimates, etc, and will greatly reduce the task of the designer. However, certain components of the system, such as intakes, will always have to be "custom-designed" for each individual system. Therefore, the designer is still required to develop a clear, professional technique for passing his ideas along to the overseer.

9.2 DESIGN TECHNIQUE

It is the designer's responsibility to prepare a complete record of his designs for the system, which can be given to the overseer. Such a record must include drawings, calculations, and estimates for each individual component of the system. With such records, the overseer is able to compare actual construction requirements against the estimates, and modify accordingly. If some unexpected problems come up during construction, the overseer can judge exactly what new materials he will require to overcome them.

A small exercise book or notebook, of the type commonly used by students, is perhaps the best way of keeping all project notes and calculations contained in one place. The designer should divide the notebook into sections, each section devoted exclusively to a single component of the system. The final section is for totaling up all the materials, labor, and costs.

The contents of each notebook section should contain the information listed in the following discussions.

9.3 PIPELINE SECTION

Sub-divided into mainline, branchlines, taplines, etc. A record of all GI and HDP pipe sizes and lengths; all fittings (tees, elbows, reducers, unions, etc). Washouts and airvalves. Control valves and valveboxes. A rough key plan of the pipeline. Trenchline calculations (volume & labor of excavation). Required tools. See Figure 9-1 for a sample estimate.

9.4 INTAKE SECTION

Sketches of each source area, showing locations of structures. Rough design drawings of each structure (such as intake, collection tank, etc). Construction calculations (volumes of excavation, sand, cement, gravel, stone, brick, slate, etc). Labor (skilled/unskilled). Specific diagram of pipes and valves (with size and lengths). Roofing details. Required tools. Special instructions. If the intake works are particularly complex, a separate blueprint should be prepared. See Figure 9-2.

9.5 SEDIMENTATION TANK SECTION

Only required if there is a sedimentation tank. Design flow, detention time, capacity calculations. Rough design plan. Calculations (excavation, materials, labor). Specific details of pipes and fittings. Roofing details. Required tools. Special instructions. If very big or complex, prepare blueprints.

From	To	Pipe	Fittings
ML-0	ML-540	Class III 63mm HDP (540m)	63mm tee (Tap 1) 63mm tee (Tap 2)
ML-540	ML-800	Class III 50mm HDP (260m)	63 x 50mm reducer
ML-800	ML-808	1½" GI (8m)	1½" x 50mm GI/HDP flange set 1½" x 50mm GI/HDP flange set
ML-808	ML-1000	Class III 50mm HDP (192m)	50mm tee (b'point A) 50mm tee (airvalve #1)
ML-1000	ML-1670	Class III 32mm HDP (670m)	50 x 32mm reducer 32mm tee (Tap 3) 32mm tee (Tap 4)
A-0	A-200	Class III 32mm HDP (200m)	50 x 32mm reducer (b'point A)
A-200	A-430	Class IV 20mm HDP (230m)	32 x 20mm reducer 20mm tee (Tap 5)

FIGURE 9-1
EXAMPLE PIPELINE ESTIMATES

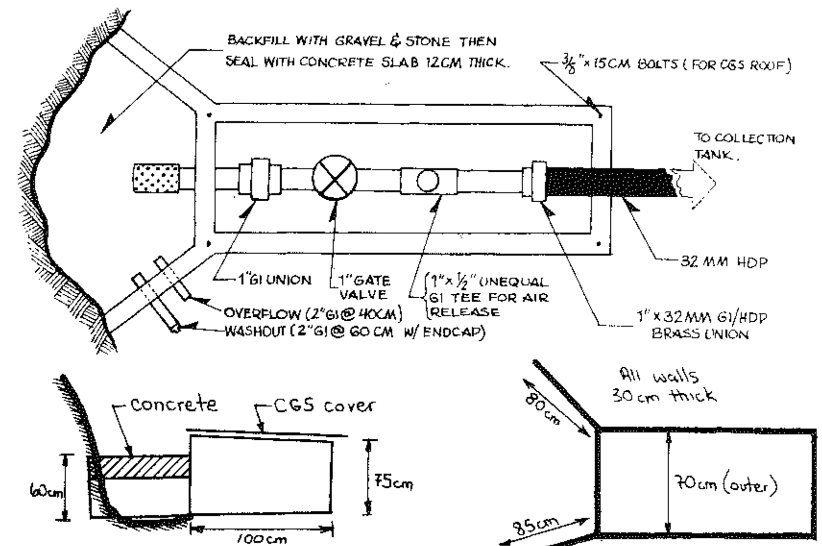


FIGURE 9-2
EXAMPLE ROUGH-DESIGN PLAN

9.6 BREAK-PRESSURE TANK SECTION

Sub-section for each design of break-pressure tank in the system (ie- masonry, HDP, float-valve, etc). Each sub-section should include estimates for the individual design, and for the total number of such designs. Basic drawing of each type. Construction calculations (excavations, materials, labor). Specific arrangements of pipes and fittings. Roofing details. Required tools. Special instructions. See Figure 9-3.

9.7 RESERVOIR TANK SECTION

This is the most important section of the design notebook, since no other single component will consume so much material and labor. Careful drawings of the design are required (wall dimensions, floor construction, pipe arrangements, roofing, etc). Construction calculations (excavation, materials, labor). Required tools. Special instructions. A separate blueprint should be prepared. For an example design and estimate, refer to Chapter 14.8

9.8 TAPSTAND SECTION

As with break-pressure tanks, a sub-section for each different tapstand design. Estimates for each individual design, and for the total number of such designs. Drawings of each type of tapstand. Volumes of cement, sand, gravel, brick, stone, slate. Labor (skilled and unskilled). Pipe sizes, lengths, and fittings. Required tools. Drainage details. See Figure 9-4.

9.9 SPECIAL COMPONENT SECTION

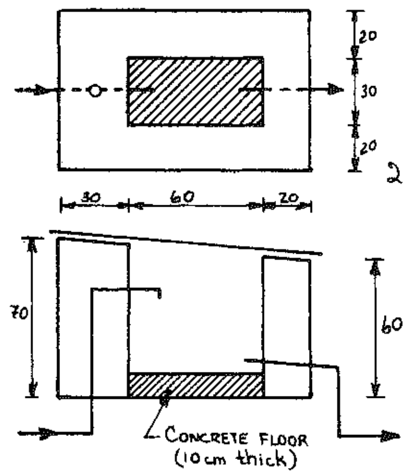
For special components of the system: suspended crossings, gully crossings, pipeline valveboxes, frictional diffusers, etc. For each of these, detailed drawings and estimates are required.

9.10 TOOL LIST SECTION

Every different type of tool that must be supplied (ie- brought out to the field site) for working on the system; quantity of each tool. Refer to the REFERENCE TABLE III at the end of this handbook for a recommended tool list.

9.11 TOTAL ESTIMATES

Total materials: Sub-divided into two lists: all materials locally provided (sand, stone, slate, etc) and all materials brought in (provided by government and UNICEF). Unit price, and total unit cost.



Stone masonry @ 1 4 mortar

ESTIMATES PER SINGLE TANK

VOLUME OF MASONRY

$0.3 \times 0.2 \times 0.6 = 0.036\ m^3$

$0.3 \times 0.3 \times 0.7 = 0.063\ m^3$

$2 \times (1.1 \times 0.2 \times 0.65) = \underline{0.286 m^3}$

$0.385 m^3$ TOTAL

35% = mortar = $0.135 m^3$

25% = Cement = $0.034 m^3$

100% = Sand = $0.135 m^3$

VOLUME OF CONCRETE @ 1:1½:3 mix

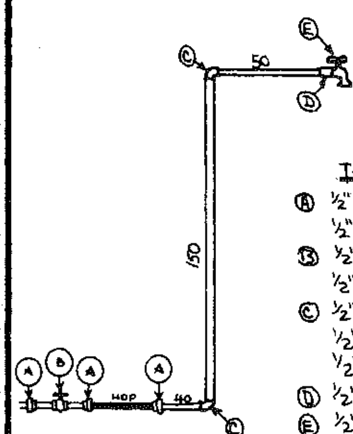
$0.3 \times 0.6 \times 0.1 = 0.018 m^3$

50% = mortar = $0.009 m^3$

etc.

BREAK-PRESSURE TANKS #1 & #2

FIGURE 9-3
EXAMPLE MASONRY CALCULATIONS



DETAILS OF PIPE & FITTINGS

	Item	TOTAL QNT'Y
Ⓐ	½" x 20mm GI/HDP Brass Union (3 per tapstand)	12
	½" GI NIPPLE (2 PER TAPSTAND)	8
Ⓑ	½" Globe valve	4
	½" GI Pipe @ 40 cm*	160 cm
Ⓒ	½" GI elbows (2 per tapstand)	8
	½" GI pipe @ 150 cm*	600 cm
	½" GI Pipe @ 50 cm*	200 cm
Ⓓ	½" GI SOCKET	4
Ⓔ	½" BRASS Faucet	4

(*All GI pipe threaded @ both ends)

TAPSTANDS #1-#4

FIGURE 9-4
EXAMPLE GI PIPE ARRANGEMENT & ESTIMATE

Portering: Once the amount of materials has been determined, calculate logistical details: number of portering trips, cost per trip, total portering cost. How much is paid by government, and how much portered voluntarily (this is determined by LDD policy, so discuss with the regional engineer).

Total labor: Total man-hours of skilled labor, wages, and total skilled labor costs; total man-hours of unskilled labor, equivalent wages (ie- one-half of skilled labor wage), and total equivalent cost.

Total project cost: Total costs of materials, portering, and both skilled & unskilled labor.

Contributions: Separate listing of all materials and costs that are contributed by the government, by UNICEF, and by the villagers.

9.12 SUMMARY

It is emphasized once again that the designer must very carefully prepare accurate and clear notes of his designs, so that the overseer may easily understand how the project is envisioned.

Each of the above sections will be individually discussed in some of the following chapters, so that better ideas of the materials required can be had. A table of estimates can be found in REFERENCE TABLE VIII at the end of this handbook.

10. PIPELINE CONSTRUCTION

10.1 INTRODUCTION

No other phase of a CWS project is likely to consume so much of the labor, or run into more difficulties, than the construction of the pipeline. Difficult terrain, all too common in the rugged countryside of Nepal, can prolong this phase far beyond what would resonably be expected which drains away village enthusiasm, which in turn prolongs the work even more. Motivating the villagers is a major aspect of the overseer's job.

It is important, therefore, that the pipeline work be done properly the first time. To have to locate some internal blockage that is the result of carelessness, or to have to rejoin pipe already buried, or to have to redig the trenchline because of erosion problems which could have been foreseen, are all discouraging tasks.

Although the above problems are not completely avoidable, they can be minimized. This chapter shall deal with the proper technical procedures for constructing the pipeline, and will also discuss some typical problems that experienced CWS overseers in Nepal have been confronted with, and how they overcame such difficulties.

10.2 PROJECT ORGANIZATION

In most projects, especially where the water system is keenly needed, the villagers are quick to organize themselves into a work force, and to divide up responsibility and work among themselves. It is not necessary for the overseer to involve himself with the bookkeeping/timekeeping records (except for skilled labor), for the villagers can do this better themselves. The division of responsibility and work likewise is best decided by them, for they will do it according to their own established social customs and procedures. The results may not necessarily appear to be equitable to the overseer (especially if he is a non-Nepali), but the important matter is that all the villagers are content and in agreement with the decisions.

The point where the overseer should exert his influence is in planning the overall construction schedule: which sections of the project will be started first, or saved until later.

Experienced overseers in Nepal have made certain observations and recommendations:

The role of the overseer should be that of a technical consultant, assisting the villagers in the system construction. During the first few days of work, one or more village individuals will emerge as natural foremen, quick to understand the needs of the overseer and able to influence and direct the village laborers as required Within a few days, these foremen will be fully capable of directing the routine trench digging, allowing the overseer to initiate construction of masonry works, etc.

There will be high initial enthusiasm of the workers for the first few days or weeks of digging. The work force during this time will never be bigger, nor will the villagers ever work so hard. It is a good time to tackle the most difficult portions of the trenchline. Try to avoid anything which will slow down the work rate (such as lack of materials, or scheduling work just before major holidays, etc).

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. Digging the trenchline one coil-length at a time is good, laying the pipe down and then immediately burying it. A steady rate of completing the pipeline keeps the enthusiasm high.

The division of work will usually mandate that each villager, or household, or ward, is responsible for digging a certain length of mainline, as well as all of their own particular branchline or taplines. If possible, get the entire village to dig the mainline first, before having the individual sections of pipeline dug.

From the very beginning, establish standards and procedures that must be followed. Getting the trenchline consistantly dug to the proper depth is easiest if the overseer insists on it being so before putting down any pipe in it. Once this routine is established, and firmly maintained, there will be less problems later on.

Communicating with the villagers is very important. The overseer must explain not only what needs to be done, but also why it must be so. Once the villagers see the reasons, they are far more motivated to do the job properly, since it is in their best interests to do so It is not enough that a few village leaders alone know, or even that the foremen know: the typical, average worker on the system must also know. For a non-Nepali overseer, there is a considerable language and cultural barrier which must be overcome, but the effort is worthwhile. As an aid to this, refer to Figure 10-1, presented in both English and Nepali.

10.3 TRENCH WORK

The proper depth of the trenchline should be one meter (100 cm) deep. There is no specific width of trench necessary (in practice, the width of the trench will be determined by the size of the digger, approximately 40 cm). The manufacturers of HDP pipe claim that the lifetime of the pipe, when properly joined, buried, and if not subjected to pressures greater than the pressure rating, is 50 years.

When the pipe is buried one meter deep, it is adequately protected against the weight (and sharp hooves) of heavy animals walking over it; it is well below the depth reached by Nepali farm plows (about 10 cm); it is insulated against freezing temperatures; and there is plenty of overburden (cover of dirt) to allow for erosion over the lifetime of the system. This is all discussed in Figure 10-1.

The reason that we must bury the pipe underground is to protect it against the heavy animals which may walk on it, and to prevent the pipe from being damaged or broken. If the pipe is buried properly 1 meter (3 feet) underground, it will stay protected for the next 25 years. If it is not properly buried, then it will soon be broken and no water will reach the tapstands. It is better to bury the pipe properly now, then to have to build a whole new system after just 5 or 10 years.

We will dig and bury the pipeline in lengths of 200 meters at a time. Each worker will be assigned to dig a length of 3 meters (10 feet), which he must dig to the proper depth. There must be no sharp rocks in the trench which can cut the plastic pipe.

When the full length of trench is ready, we will join the new pipe to the old, then put it in the trench and bury it. When burying the pipe, only dirt should be used to completely cover it. No bushes, leaves, or tree branches should be used. Large stones should be put on top of the trench.

The shovels, hammers, and rock-picks provided by the government are not the personal property of any man, but belong to the project. These tools must be brought to work every day. When digging the trenchline, if a large rock is encountered then these tools must be shared. The workers will take turns hammering at the rock until it is removed, and then the tools should be passed along to where another rock must be removed.

कुलो सनेर पाइप गाड्नु पर्ने मुख्य कारणाहरू यि हुन कि गाइ-वस्तुहरू हिंड्दा पाइप कुल्चिने, बिग्रिने वा माँचिने सम्भावना हुन्छन् । यदि १ मीटर (३ फीट) कुलो सनेर पाइप जमीन मुनि गाड्न सकियो भने त्यस पाइपलाई २५ बर्ष सम्म जोगाएर राख्न सकिन्छ । यदि पाइप राम्रोसँग गाडिएको छैन भने चाँडै नै बिग्रिन सक्छ, जस्ले गर्दा धारामा पानी आउँदैन । त्यसकारण ५ वा १० बर्ष पछि पुरै सानै पानी योजना फेरि बनाउनु भन्दा पाइपलाई पहिले नै उचित तरीकाले गाड्नु राम्रो हुन्छ ।

हामीहरू एक पटकमा २०० मीटर लामो पाइप गाड्छौं । प्रत्येक व्यक्तिले ३ मीटर (१० फीट) लामो कुलो सन्नु पर्छ, अनि कुलो चाहिनेजति गहिरो हुनै पर्छ । कुलो सन्दा चुच्चो परेको ढुङ्गा त्यसै छाड्नु हुँदैन किनभने पाइप (प्लास्टिकको) काटिने सम्भावना हुन्छ ।

जहिले कुलो पूरा सनेर तयार हुन्छ, अनि हामीहरू नयाँ पाइप पुरानो पाइपमा जोडेर कुलोमा गाडि दिन्छौं । पाइप पुर्दा पहिले माटोले मात्र पुर्नु पर्छ । झारपात वा रूखको हाँगाहरू हालेर पाइप कहिल्यै पनि पुर्नु हुँदैन । पाइप माटोले पुरिसकेपछि मात्र ठुला ठुला ढुङ्गाहरू माथिबाट राख्न सकिन्छ ।

सरकारले दिएको साबेल, घन र ढुङ्गा सन्ने पिक, योजनाका सामानहरू हुन्, कुनै पनि मानिसले आफ्नै सम्पति हो भनेर मन्ठान्नु हुँदैन । यि सन्ने सामानहरू दिन दिनै कामगर्न आउँदा लिएर आउनु पर्छ । कुलो सन्दा ठुलो ढुङ्गा भिक्नु पर्‍यो भने त्यसबेला सन्ने सामान जस्लेपनि माग्न सक्छ । ठुलो ढुङ्गा न भिकेसम्म सन्ने मानिसहरूले आलो पालो गरी ढुङ्गामा घनले हान्नु पर्छ, अनि त्यस पछि अरू ठुलो ढुङ्गाहरू भिक्नु पर्ने ठाउँमा यि औजारहरू पठाइदिनु पर्छ ।

FIGURE 10-1

The pipeline should ideally follow the same route that the original survey was conducted along. However, it is not unusual to have to make detours due to impassable rock areas, land erosions, or because of an original survey along an impractical route. When such re-routing is necessary, the overseer must re-survey the new section to determine how it will affect the HGL of the system, and to see whether additional pipe is necessary.

The pipeline should be kept as far away from erodible points as possible: landslide areas, gullies, streams or riverbanks, etc. When passing through a terrace, keep the trenchline "inside" (as close to the back of the terrace as possible), and when cutting down the faces of terraces, run the trenchline diagonally across the face (refer to Figure 10-2). Motor roads should be crossed perpendicularly and the trench dug as deep as possible up to 150 cm.

Due to hard, rocky ground along some sections, it will not be possible to always get the trench 100 cm deep. The overseer should try to learn what sort of traffic can be expected to walk over the pipeline (human, animal, farming, etc), how vulnerable the section will be to erosion, and from this information he should decide if the soil cover will be adequate. If not, it will be necessary to substitute GI pipe along that section. Ideally, the surveyor has already determined all the places where GI pipe will be needed, but practically speaking there will be sections, not visible from a surface walkover, where additional GI pipe will be needed.

When crossing landslides, gullies, and/or streams, a suspended pipeline may be necessary. Refer to Section 10.11 and Technical Appendix E for further discussion of these special problems.

Experience advises digging the trenchline in sections equal to the length of the coil of pipe to be buried in it. Each worker is assigned a three-meter length of the trench to dig to the proper depth (a villager can typically dig this length in soft, easy soil in one day, or two days if the ground is harder). Pick-axes and crowbars that are provided by the government are not individually owned by the villagers, but should be moved up and down along the trenchline to be used where needed.

The trenchline should be free of all sharp rocks which can cut into the HDP pipe (after the pipe is laid it tends to contract, which can force it to kink around sharp stones). When the entire section is dug, it should be inspected along its full length by the overseer before he allows the pipe to be uncoiled and laid.

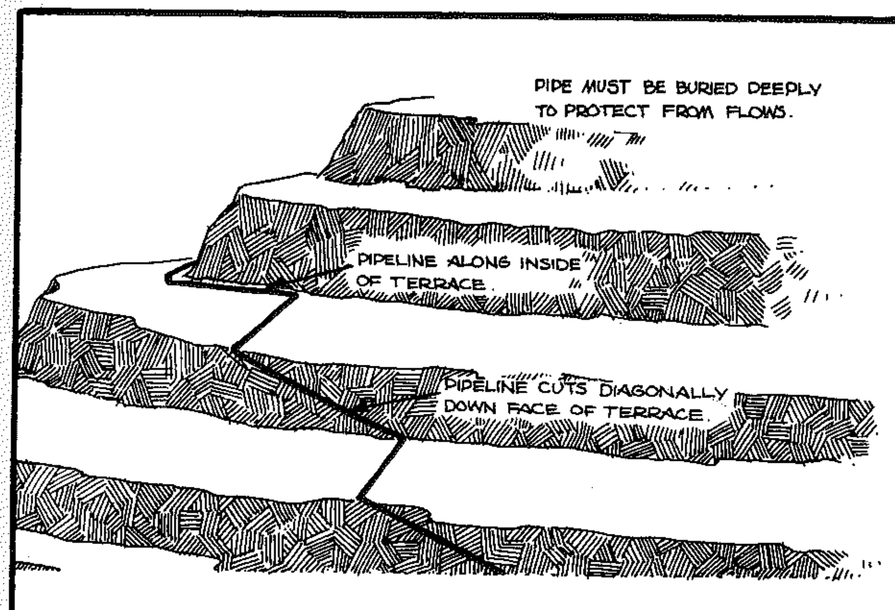


FIGURE 10-2 TRENCHING ACROSS TERRACED FIELDS

10.4 PIPE LAYING

The HDP pipe is supplied in coils of 25-300 meters in length, depending upon the size and class. The HDP pipe must be uncoiled carefully, otherwise kinks will form which crimp the pipe, as shown in Figure 10-3. Crimped pipe must not be used, since it has been considerably weakened at the crimped point. Such sections should be cut out about 10 cm on either side of the crimp, and the pipe ends rejoined.

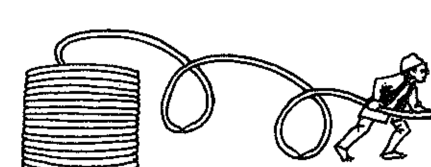


FIGURE 10-3

KINKING PIPE BY IMPROPER UNCOILING

A practical manner of uncoiling the pipe is to have it supported by a length of bamboo, and slowly unwound, as is shown in Figure 10-4. Larger coils can be unrolled along the side of the trench, then carefully put into position.

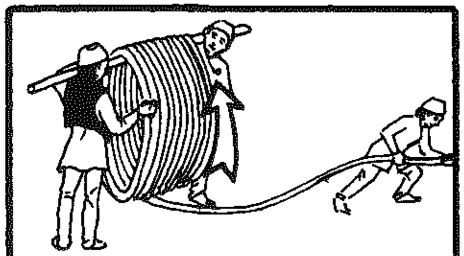


FIGURE 10-4 RECOMMENDED METHOD OF UNCOILING PIPE

10.5 HDP PIPE JOINING

HDP is joined by the technique of butt-welding: using a heated steel plate to melt the ends of the pipe, which are then pressed together and allowed to cool. This is discussed in detail below:

Once the pipe has been laid out (either in the trench or next to it), it must be checked for internal blockage This can be done by covering the pipe end by mouth and blowing forcefully into it. If the air flows freely, there is no total blockage. Partial blockages can occur with dirt, stones, or sticks that have ended up deep inside the pipe*. A small stone can be dropped into one end of the pipe and "walked" through the whole length. Its tumbling passage is easily heard,and therefore can be used to locate blockages. Refer to Section 10.12 for techniques to minimize such blockages.

When the pipe has been checked and cleared of blockages, it is prepared for joining as follows:

1) The pipe ends are cut perpendicular with a hacksaw and leveled off with a flat file. A pocketknife is then used to trim the rough plastic filings off. The prepared pipe ends should be clean and smooth, and when the two ends are mated together, there should be no gaps of more than one millimeter.

2) The pipe-joining crew should make a practice attempt before actually using the heating plate, so that they are familiar with the motions necessary to make that specific joint. When making the actual joint, one person must be positioned to report progress on the underside of the pipe as it is being melted. A pocket mirror can help too.

* the most common internal blockages are caused by the wooden plugs that some HDP manufacturers use to seal the ends of the pipes. These plugs sometimes get forced into the pipe.

3) The temperature of the heating plate is crucial, 220°C on both sides of the plate. To determine this, a white Thermo-Cnrom crayon is used to make a small mark on the hot plate. This mark should turn from white to brown in just two seconds. A hotter plate will turn the mark brown much faster, and a colder plate will take longer (or not change the color at all). A too-hot plate will only melt the plastic it touches, without heating deeply into the pipe which is necessary for the homogeneous, solid fusion of the two pipe ends. A plate that is too cool will also fail to melt enough plastic. Both types of joints will be imperfect and brittle, easily cracked or snapped apart.

4) When the heating plate is at the proper temperature, it is slipped into a Teflon envelope (which prevents the melted plastic from sticking to the heating plate). The plate is then held between the pipe ends, which are firmly pressed against it. When the pipe is properly heated, there will be a lip of melted plastic around the perimeter of the pipe ends. This lip should be equal and even all the way around.

5) The pipe ends are separated from the heating plate, which is removed out of the way, and then are <u>carefully</u> brought together. Contact must be even and balanced, and done exactly right the first time: once the melted ends touch, they cannot be taken apart and realigned. The pipe ends should be pressed together firmly but not excessively hard, until the joint has cooled down to where it can be touched by hand, without hurting. It should then be laid on the ground carefully and not moved or disturbed for several minutes more.

6) The joint is tested by flexing it vigorously and examining it visually. A proper joint is as strong as the rest of the pipe, and cannot be cracked or broken apart. A weak, meek flexing of the pipe serves no purpose. If a weak joint is passed over and buried, it can easily be cracked apart by the earth and water pressures acting on the pipeline once it is in service. It is much easier to re-join the pipe <u>before</u> it is buried, so test it strongly!

7) When the joint has been successfully tested, the pipe is fully laid in the trench and stretched out. One person can actually walk on top of the pipe, flattening in down into the trench and visually examining to see that no projecting stones will damage the pipe once it is buried.

10.6 BACKFILLING

Backfilling the trench should be done as soon as the pipe has been laid, to minimize exposure to sun and curious villagers, both of which are detrimental to the pipe. Ideally, the backfill should be screened and compacted in 10 cm layers, but practically speaking this is difficult to get the villagers to do. The chief concerns of backfilling should be to prevent any organic materials (such as leaves, sticks, bushes, etc) from being used, and prevent rocks and stones from being dumped directly on the pipeline (after the pipe has been covered with about 50 cm, it is allowable to use rocks in the backfill). Because the backfill will tend to settle, the dirt should be mounded up over the trenchline to compensate, as in Figure 10-5.

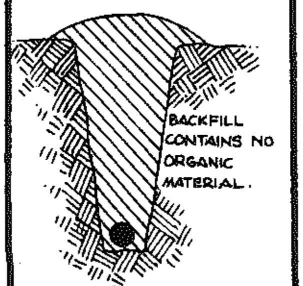


FIGURE 10-5
TRENCH DACKFILLING

The pipeline should be fully buried except for a three-meter section at each joint. This joint section should only be about half-buried until the pipeline has been filled with water and allowed to stand at full static pressure for 24 hours. This makes it very easy to locate leaking joints, since water will have seeped up to the surface within that time. Once all joints have been tested, they can be fully buried.

Road crossing. When the trench under a road is 120 cm deep or more, it can be backfilled normally except that the backfill should be compacted regularly as it is added, with all large rocks and stones removed.

When the trench is between 100-120 cm deep, the pipe should be laid on a bed of sand, covered with a further 30 cm of sand and then backfilled as above.

When the trench is less than 100 cm deep, the pipe should be bedded on sand, and further covered by 20 cm more of sand. On this, a reinforced-concrete (RCC) slab 10 cm thick should be poured. Once the slab has hardened, backfilling procedes as above. Refer to Chapter 19.12 for details of RCC slabs. Figure 10-6 illustrates backfilling across a road.

Shallow trench and embankments: Along some sections it will not be possible to get the pipeline buried deeply (if at all). If such sections are of HDP pipe, then special earthwork is required for adequate protection of the pipe. Refer to Figure 10-7

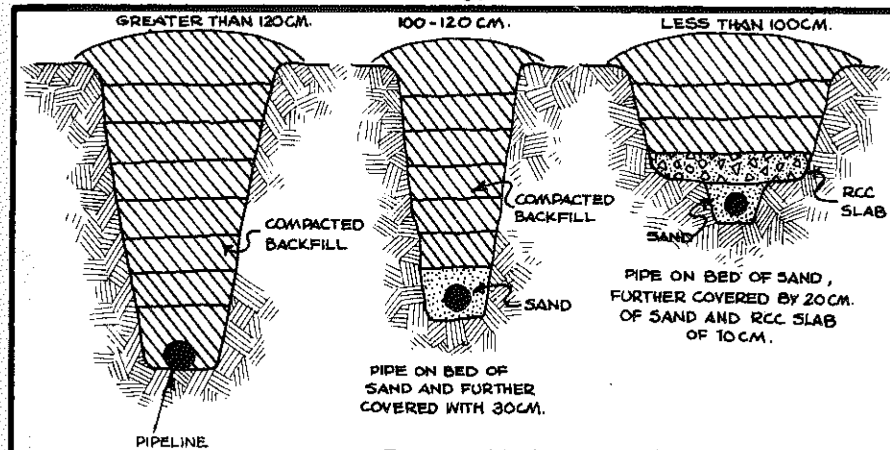


FIGURE 10-6
ROAD BACKFILLING

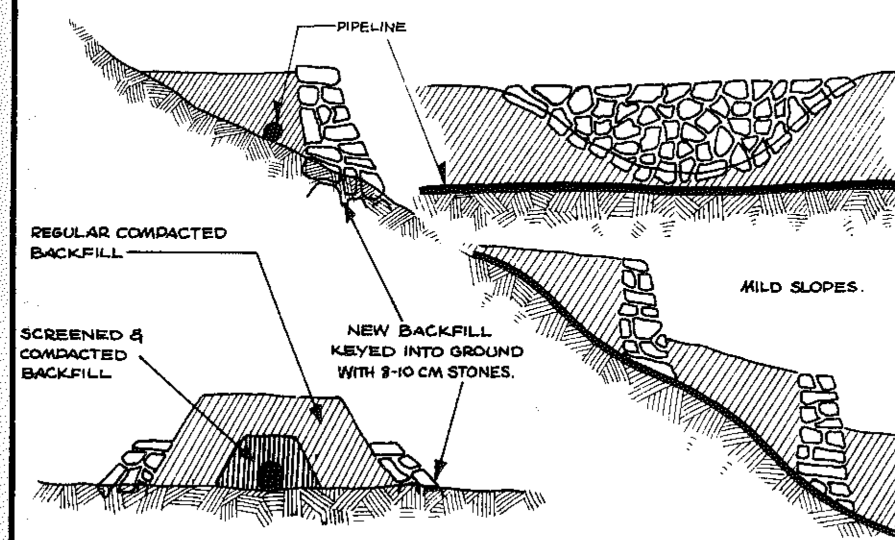


FIGURE 10-7
BACKFILL EMBANKMENTS

Steep slopes: Where the trench cuts down a steep incline, the backfill is vulnerable to easy erosion by rainfall, which will tend to wash all backfill to the bottom of the slope. Facing the trench with stone, as shown in Figure 10-8, will help to protect the backfill.

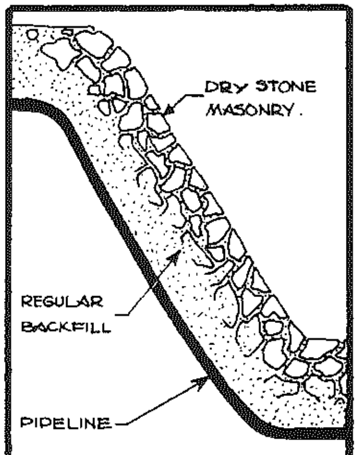


FIGURE 10-8
BACKFILLING A STEEP SLOPE

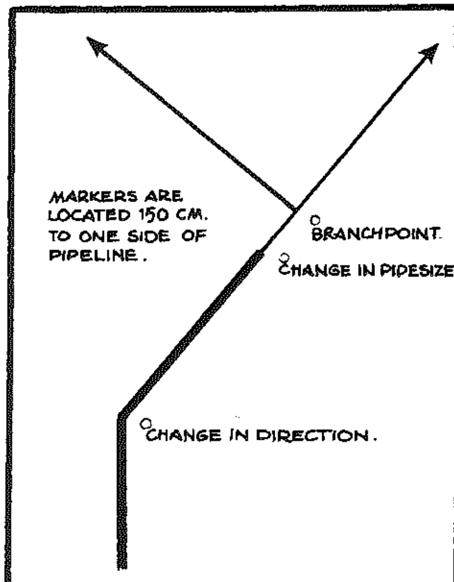


FIGURE 10-9
MARKING THE PIPELINE

10.7 MARKING THE PIPELINE

Within a surprisingly short time, all visible traces of the buried pipeline will cease to exist, especially in sections through jungle or cultivated fields. Should it become necessary to relocate some particular point in the pipeline, human memory, especially after several years, can no longer be relied upon.

To aid relocation efforts, the pipeline should be marked with permanent markers at strategic reference points, similar to that shown in Figure 10-9.

Markers should be located exactly 150 cm to one side of the pipeline, with a notch cut in it to indicate which side the pipeline is. This is because if they are ever dug up for maintenance work, it is not sure that they will be replaced in their exact position.

Markers should be located at the following strategic points:

- at all branchpoints;
- at all reducers (changes of pipe sizes);
- changes in pipeline direction;
- every 200 meters in open terrain, 50 meters in jungle.

A record of each marker should be kept, a copy with the villagers and a copy for the LDD project file.

10.8 REJOINING BURIED PIPE

In special circumstances, it will become necessary to dig up a section of the pipeline to either repair a bad joint or to locate a blockage. At such times, it becomes a difficult task to rejoin the pipe ends. The larger the pipe, the more difficult the job.

There is no easy way of doing this. It requires a lot of excavation to create enough room for the joining crew to work. A standard excavated area is shown in Figure 10-10. The exact dimensions of the area depend upon the size of the pipe being worked on.

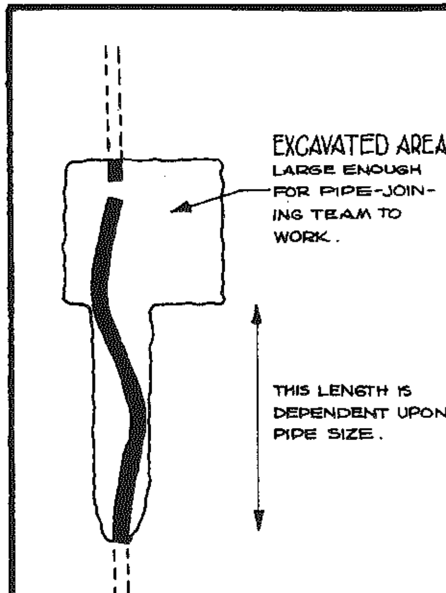


FIGURE 10-10
REJOINING BURIED PIPE

The fundamental procedure is to dig up several meters along one pipe end, so that there is a bit of slack. The two pipe ends are cut to a separation of about 30 cm. A new pipe section of about 40 cm is welded onto one pipe end. When it has cooled down and been properly tested, the pipe is flexed and joined to the other end. If this joint fails, there is still enough extra pipe so that by carefully cutting out the bad joint, another joint can be made.

Once the pipe is successfully rejoined, it is flattened down into the trench as deeply as possible and partially reburied until the joints have been tested under 24 hours of static pressure.

10.9 FIELD-CONSTRUCTED HDP FITTINGS

Although most HDP fittings are normally available, sometimes out at the project site the overseer may run out of one particular type of fitting. In such cases, it is possible to make the same fitting by using pieces of HDP pipe. Such locally produced fittings, if properly made, are as strong as the regular pipeline.

The "Technical Training Manual #3", published by LDD/UNICEF/SATA in Section 2.4.4 gives excellent diagrams on how to produce elbows, bends, tees, angled branchpoints, and reducers out of HDP pipe.

10.10 GALVANIZED IRON (GI) PIPE

Galvanized iron (GI) pipe is primarily used in the various tanks and tapstands of the system. However, it can be used in the pipeline along sections which will be subjected to excessively high pressures or where proper burial of an HDP pipeline would not be possible.

The GI pipe sizes used for CWS projects in Nepal are 1/2", 1", 1-1/2", 2", and 3" (size refers to inner diameter). The pipe is supplied in lengths up to 6 meters long.

Cutting: GI pipe is cut using a hacksaw, and the rough edges are trimmed with a flat file. Use of oil during the cutting will help prolong the life of the hacksaw blades. If machine oil is not available, then cooking oil is an acceptable alternative. Even water is better than nothing.

Threading: This is done using adjustable pipethreaders. Although it is sometimes possible to make the threads with a single cut, it is recommended that they be made by a series of shallow cuts, adjusting the die teeth to make a deeper cut each time. This technique will prolong the life of the die teeth. Lubricating oil is absolutely necessary and should be used extravagantly.

Even if the pipe has already been cut and threaded in a workshop, it is an advisable practice to bring pipethreaders, extra die teeth, and a pipe vise out to the project site, since inevitably there will be some threads which are damaged and must be cut anew.

When cutting threads, check for a proper fit using several different fittings. Despite the "standardization" of pipe sizes and threads, all fittings are not created equal, and if the entire lot of pipe is cut to fit a misfit fitting, then there will be much repetition of labor. Always test threads with at least three different fittings.

Transporting: To protect threads during transporting, coat them with oil or grease and then screw on a fitting. Exposed threads are sure to corrode and/or be damaged.

Caulking: When a fitting is screwed onto GI pipe, it is necessary to use some method of making the threaded connection watertight, especially if the joint will be under high pressure. Although caulking compounds and pipe dope are useful, it is just as effective to wrap the threads heavily with thin string (such as kite string or thread), and to screw on the fitting tightly.

Caution: When using a pipewrench to tighten fittings, care must be taken not to screw it on so tightly that the fitting is split open! It is not necessary to screw the fitting on as tightly as humanly possible. It is the caulking which makes the connection watertight, not brute strength.

10.11 SPECIAL PROBLEMS

Along certain sections of the pipeline, it is sometimes unavoidable to run through undesirable terrain such as across landslides, over gullies or streams, etc. Where such crossings are short (less than 6 meters), there is usually little technical difficulty. The use of GI pipe, suitably anchored, will usually suffice.

However, for longer spans, where alternate routing of the pipeline is not possible, it may be necessary to use a suspended crossing over the unstable area. Refer to Technical Appendix E for discussion of these crossings.

Landslides: Generally, there is no choice with a landslide area but to use a suspended crossing. The anchor points of the crossing must be on stable ground, and the suspended pipeline must be high enough to avoid being struck by sliding or falling debris.

Gully crossings: Gullies are eroded paths, usually sharp-banked, created (and enlarged) by run-off of rainwater. They are typically dry in clear weather, but may be semi-permanent streams during the monsoon season.

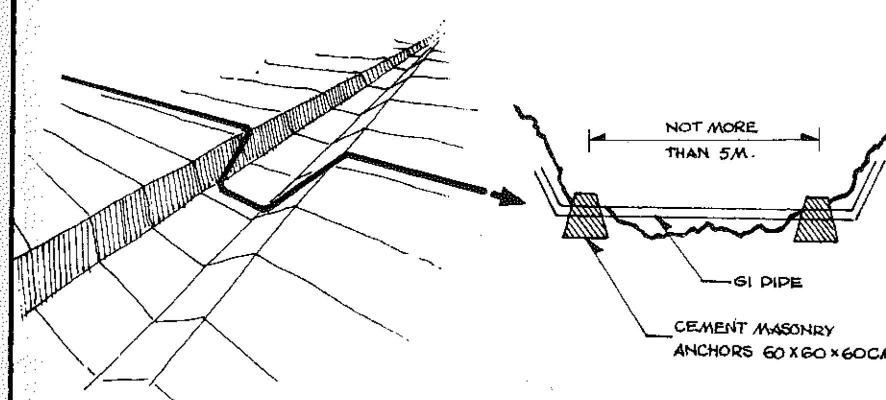


FIGURE 10-11
GULLY CROSSING

Discussion with the villagers will give an idea how extensive the run-off is. Narrow, deep gullies can be crossed by a span of GI pipe above the bottom of the gully, clear of the maximum flood level and anchored in the banks of the gully (similar to the crossing shown in Figure 10-12). Broader gullies should be crossed by GI pipe buried as best as possible, and anchored down using drystone masonry or gabions,as shown in Figure 10-11. Refer to Technical Appendix H for information on gabions.

Stream crossings: Narrow streams can be crossed similarly to narrow gullies, but additional attention must be paid to ensuring that the banks of the stream directly below the crossing point will remain stable. Building dry-stone masonry embankments or gabions (refer to Technical Appendix H) is recommended. Larger or wider streams will require a suspended crossing. In all cases, the height of the pipeline must be sufficient enough to prevent it from being struck by debris floating down the stream, especially at the maximum flood levels. Figure 11-12 illustrates:

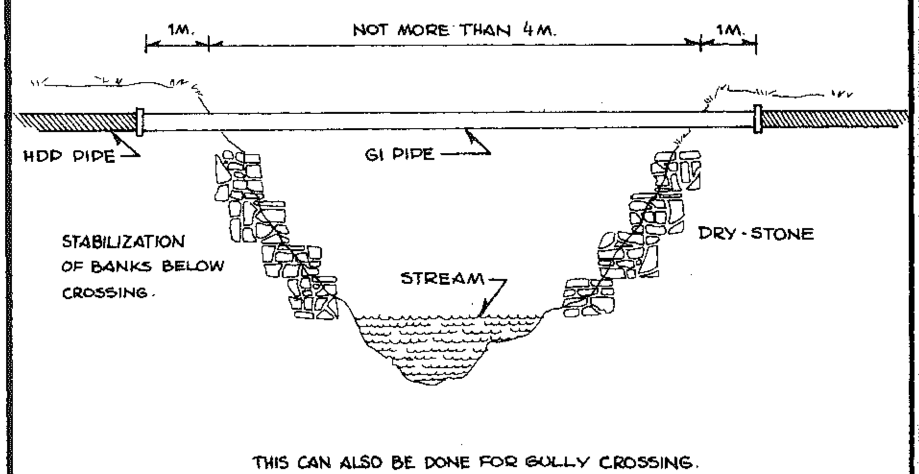


FIGURE 10-12
STREAM CROSSING

10.12 IMPORTANT CONSIDERATIONS

Construction of the pipeline requires more than just technical expertise. The overseer must be aware of human problems frequently encountered in the villages of Nepal.

Children should be considered to be compulsive saboteurs of the system. Although they do not do so deliberately, their curiosity leads to much damage and repetition of work. Open pipe ends, exposed pipeline, fresh masonry all will attract attention, with frustrating results.

Adult villagers, especially strangers passing through, will also be curious about the system and can cause some damage. Exposed fittings can get stolen, and sometimes sections of the pipeline itself are cut out, to be carried off elsewhere.

Heavy animals such as cows and water buffalos can crush or crimp the HDP pipe if they happen to step directly on it. Open trenchlines, expecially along a cowpath, pose a serious danger to these animals, since they can easily stumble into it and break a limb (some animals have even broken their necks and died, resulting in a serious loss for some family).

Some manufacturers of HDP use green (freshly-cut) wooden plugs to seal the pipe mouths of the coils. With the passage of time, these plugs will dry out and shrink and become loose. They can either fall out, or fall into the pipe. If the coil is not tested, these plugs will quickly swell up with water once the pipeline is in service, creating a very tight blockage, not suspected until the water is turned on. At such times, searching a pipeline that is several hundred meters long for a single wooden plug just 10 cm long is a very unhappy task!

The prevention of these problems is not absolute, but the overseer can take some practical steps to minimize their occurence. He should make the villagers understand the difficulties of repairing a damaged or plugged pipeline, and obtain their cooperation in protecting it. The following suggestions are based upon experience in Nepal:

1) NEVER leave a pipe end open and exposed, even for just one night. As soon as the pipe coils have been transported to the village the overseer must seal off every unplugged pipemouth. Simple plugs, cut from branches, can be used for this purpose. They are jammed into the pipemouth, then firmly nailed in place by a few ½" nails, driven directly through the HDP pipe into the plug. The plug is then cut off flush with the end of the pipe, leaving nothing for curious hands to grab and twist. It is especially important to do this for a pipe left overnight in the trenchline. Carrying a small matchbox full of these ½" nails is quite easy and worthwhile.

2) NEVER leave the pipeline exposed in the trench. As soon as it is laid, the pipe should be buried except for a 3-meter stretch at each joint, which should be partially buried until the joint has been tested under static pressure for 24 hours. At the end of the pipeline, where work has ended for the day, the pipe end should be plugged, big rocks should be carefully piled onto the pipe, and the trench filled with thorny bushes.

3) Thorny bushes can be piled around fresh masonry structures, and one or two villagers sleep next to it if necessary, until the cement has set and all pipes are firmly bonded in place.

4) Do not leave control valves exposed. Install them only after the valvebox has been completed and has a secure cover. Removing the valve handle will also discourage tampering.

5) When pipe coils are being transported from the roadhead to the village, and from the village to the worksite, make sure that the villagers do not attempt to re-coil the pipe to a more convenient shape or size.

Many times this has led to excessive amounts of crimping in the pipe, requiring a lot of lost pipe and much labor re-joining it.

11. INTAKE WORKS

11.1 INTRODUCTION

The first point of flow in a water system is at the source, where water is collected at an intake and funneled into the pipeline. This chapter will discuss various types of intake works, such as spring and stream intakes, dams, source protection, etc.

Due to the uniqueness of a source, there never will be a standard design that can be universally built for every system. However, the intake works should incorporate standard design features, which allow for adequate control of the water, opportunities for sedimentation, and prevention of further contamination. These design features will be the basic theme of this chapter. It is up to the designer to incorporate them into his plan for the intake works. The construction overseer must also be aware of these principles, so that he can make modifications in the event of unforeseen problems.

The fundamental purpose of the intake works is to collect water from one or several points and focus this flow at a single point: the entrance to the pipeline. If the water is dirty, it must be allowed to sit relatively undisturbed for a period of time. The water must be protected as much as possible against further contamination (from rain run-off, grazing animals, and curious villagers). And it must be built in such a way to last for the lifetime of the system.

The number of possible ways to design the intake for a source is infinite, influenced by factors such as available materials, source flow, flood levels, ground stability, topography of the area, etc. This chapter will present several different designs, all of which have been successfully used in the past, and from which the designer can modify and develop a suitable intake for his own system.

The next chapter will present the technical details of sedimentation tanks, which may be required by silty sources such as streams.

11.2 SITE LOCATIONS

Although it is obvious that intake works must be built at the source, there is still flexibility when it comes to locating the actual structures. Water catchments can be built as part of the total intake structure, or can be used as starting points where water is collected and piped down to a nearby site which is more suitable for building settling chambers, sedimentation tanks, or collection tanks.

The most important consideration must be the problems of the monsoon season flooding. Intake structures must be located at points where they will not be threatened directly by flood waters, or indirectly by land erosion over the years. Careful questioning of the villagers must be done to obtain as accurate an idea of monsoon flows as possible.

Intake works should not be built in or near gullies, or at points where unstable ground above them can carry them away in a landslide, or on top of swampy ground (soft dirt saturated by the underground water table).

11.3 EXCAVATION, FOUNDATION, & CONSTRUCTION

The tank site should be staked out with wooden pegs and string, and excavated to a depth of 30 cm (if solid rock is not encountered sooner), and the floor of the excavation leveled off. For a spring intake, the flow should be diverted away from the excavation to keep it as dry as possible. The excavation of catchment walls should be deep enough to cut off underground seepage from the spring (discussed in the next section). The floor of the excavation should be hard and firm. A layer of lean concrete 10 cm in depth is put down, and compacted to ensure proper settling. The cement mortar is put down directly on the concrete, and the masonry footing is laid directly on the mortar. The footing should be 10 cm wider than the wall on each side, and the wall should not be less than 30 cm wide. The height of the footing should be 10 cm. The cement mortar should be of 1:4 cement: sand ratio (refer to Chapter 19 for discussion on cement mortar and masonry). Figure 11-1 illustrates the cross-section of an intake wall.

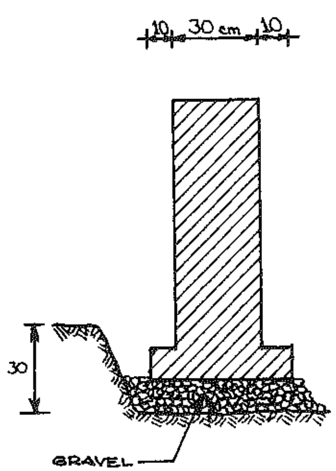


FIGURE 11-1
FOUNDATION OF INTAKE TANK

11.4 CATCHMENT OF FLOW

This is the component where the source flow is captured. In a spring intake, it is typically watertight walls surrounding the source point.

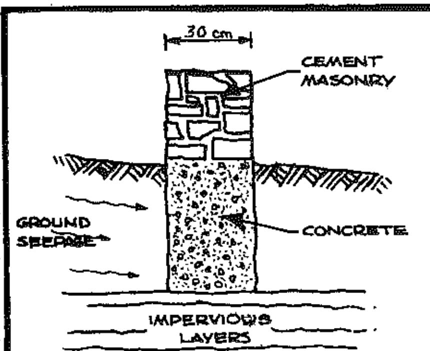


FIGURE 11-2
WATERTIGHT WALL FOR INTAKE TANK

For a stream intake, it is typically a small pool of water with an intake pipe at its bottom, or a surface channel leading the water to the sedimentation tank.

Where watertight walls must be built to contain the ground flow of a spring, they should penetrate as far into the ground as necessary to cut off seepage flow beneath the intake. Since such foundation trenches will be impossible to keep dry, it is possible to pour a fairly-dry concrete mix into the trenches. As long as the concrete is contained by dirt walls or wooden forms, it cannot be physically washed away and will be alright. Once the concrete has set and hardened slightly, the regular masonry wall can be built on it (refer to Figure 11-2).

For a stream intake, where the depth of the intake point is less than 40 cm, it is necessary to create a basin of water, which will be relatively quiet and therefore allow settlement of the heavier suspended particles (such as sand, leaves, etc). An intake pipe can be located in the bottom of the pool (protected as shown in Figure 11-3). Such an intake should have 40 cm of water depth over it to deter interference from humans and animals, and to protect it from floating debris. An alternative design for such a basin is to dig a channel from the basin to the settling tank. Such a surface channel, unless in extremely sandy or porous soil, does not have to be of masonry, so it is therefore less expensive to built.

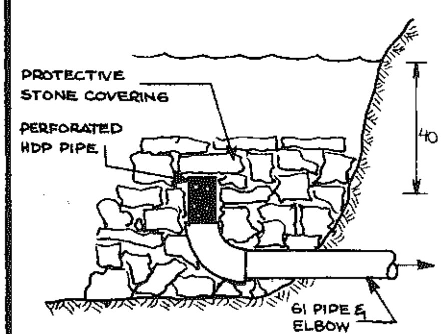


FIGURE 11-3 EXAMPLE OF STREAM INTAKE

Section 11.12 discusses stream catchments, dams, and basins.

11.5 SCREENING

Suspended particles in the flow can add to the wear and tear on the HDP pipe, so it is desirable to eliminate such particles as much as possible. Screening can remove a lot of these particles, and sedimentation removes much of the remainder.

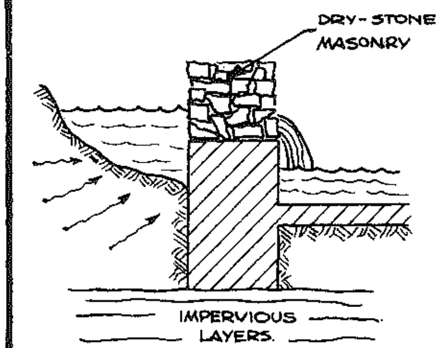


FIGURE 11-4
DRY-STONE MASONRY WATER SCREEN

There should be at least two screening points in the flow: a coarse filter to remove larger floating and suspended debris, and a fine-mesh screened intake over the pipe end.

The coarse screen can be done by a dry-stone masonry wall, as shown in Figure 11-4. This is just a section of masonry wall put together by rocks or bricks closely set together but without any cement mortar. This wall section is easily dismantled for cleaning and maintenance purpose, and then easily reconstructed.

The screened intake of the pipeline should be of a fairly small mesh (a good size usually available in the bazaars is 20 meshes per inch), made of brass screening if available. Chapter 20.2 discusses an easy way to make a screened intake using HDP pipe.

A medium-size screened intake can be made using HDP pipe that is perforated with a hot nail to make dozens of holes. This is then affixed to the GI pipe of the outlet.

11.6 SEDIMENTATION

Sedimentation is the process whereby the water is allowed to sit relatively undisturbed for several hours. In the resulting lack of turbulence, the finer suspended particles sink and settle out of the water. Since the chief source of turbulence in water is due to the velocity of the flow, then the slower the flow through the sedimentation chamber, the more effective the sedimentation process.

The size (capacity) of the chamber depends upon the type of source, the amount of flow, and whether or not there is a reservoir tank further downstream. Sedimentation requirements may be nothing more than a small chamber in the intake structure, or may be a large, separate tank (such tanks are discussed in the next chapter).

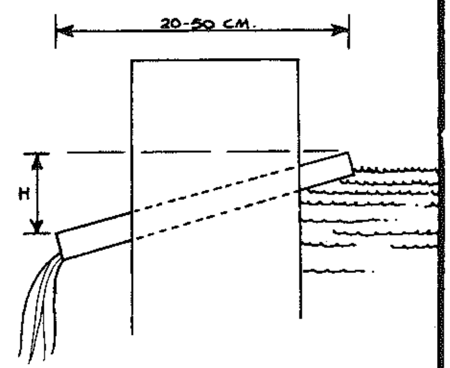
Spring sources: These are typically cleaner, and usually such a system will require a reservoir tank. So extensive sedimentation is not usually necessary (unless it is a rather dirty source, or there is no reservoir). A chamber with a dry-stone masonry filter and a screened intake (such as discussed in the proceeding section) is usually sufficient.

Stream sources with reservoirs: A separate sedimentation tank should be built, with a detention time of 15 minutes.

Stream sources without reservoirs: Also require a separate sedimentation tank. A detention time of not less than 60 minutes.

11.7 SERVICE PIPES

Because HDP pipe does not bond to cement mortar or concrete, all pipes set into masonry walls must be of galvanized iron (GI). There are three different service pipes in an intake structure: washouts, overflows, and outlets.



OVERFLOW CAPACITY (LPS).

GI PIPE SIZE	H=5CM.	H=10CM.
1"	0·62	0·85
1½"	1·4	1·9
2"	2·1	3·0
3"	5·2	7·3

FIGURE 11-5 OVERFLOW DESIGN

Washouts: allow the draining of the intake so that settled sediments can be washed out, and maintenance work performed. The GI pipe size of the washout should be either 2" or 3". The washouts should be set slightly into the bottom of the chamber, and can be closed with an endcap. Washouts should not be set into masonry walls less than 30 cm wide (to ensure enough bonded strength so that a pipewrench can be used for removing the endcap). Each separate chamber, and the catchment basin, should have its own washout. Water from the washouts must be carried away (by a surface drainage channel) in such a manner that does not cause erosion of the intake foundation.

Overflows: allow excess water to be safely diverted away from the tank without causing erosion. The size of the overflow must be selected so that it can pass the maximum flood flows during the monsoon season. The typical overflow for small tanks is a short length of GI pipe set into the wall of the tank. Figure 11-5 shows this arrangement, and gives some maximum flows that each size pipe can handle, pitched at 5 cm and 10 cm. Any number of pipes can be used if a single pipe is insufficient. The overflow water must be disposed of in the same manner as the washout flow.

Outlets: The outlet pipe is the starting point of the pipeline. The size of the outlet can be determined using the information given in Technical Appendix G, but should not be smaller than the HDP pipe size for the design at that point. The mouth of the pipe should have a screened intake. A gate valve is needed, with an air-vent located just downstream. Figure 11-6 shows the pipe arrangement of a typical tank outlet.

11.8 CONTROL VALVES & AIR-VENTS

Gate valves should be installed on each outlet pipe, so that the pipeline can be drained for maintenance purposes.

Globe valves are not necessary when the entire flow of the source is to be used. A globe valve is needed only when a portion of the source flow is to be used (as is usually the case with a stream intake). Such a valve allows only the design flow into the pipeline, and forces the excess water to overflow and be returned back to the source. This valve must be located at the discharge point of the first downstream tank, or at a point where, if it is accidentaly closed, it will not cause excessive pressures to burst the HDP pipe.

Air-vents are located just downstream from a gate valve. These serve to allow air to escape from the pipeline without bubbling out through the intake and interfering with the flow. They also allow air into the pipeline whenever the gate valve just upstream is closed so that the pipeline drains (when this happens, the draining water will set up a suction pressure in the pipeline, which can draw in polluted groundwater through leaks; an air-vent allows air to be drawn in instead). An air-vent can be either of ½" GI pipe or 20mm HDP pipe. The mouth of the air-vent must be higher than the overflow level of the tank. The end of the air-vent should be directed downwards (to prevent dirt and dust from settling into it) and should be screened (to prevent insects from crawling in).

11.9 ROOFING

The roofing on the intake structures must be secure enough to prevent curious people from interfering with them, and should seal the source off against any further contamination from surface run-off of rain, grazing animals, leaves, etc. Accessways are required so that the intake can be cleaned and repair work performed; an opening to allow a man in should be at least 60 cm square.

Common roofing schemes in Nepal are:

Slate roofing: constructed by the villagers, if slate is locally available. Requires a lot of wood for beams and rafters.

CGS roofing: corrugated galvanized steel roofing sheets, nominal size 3' x 10', effective size 70cm x 320cm. Refer to Technical Appendix F

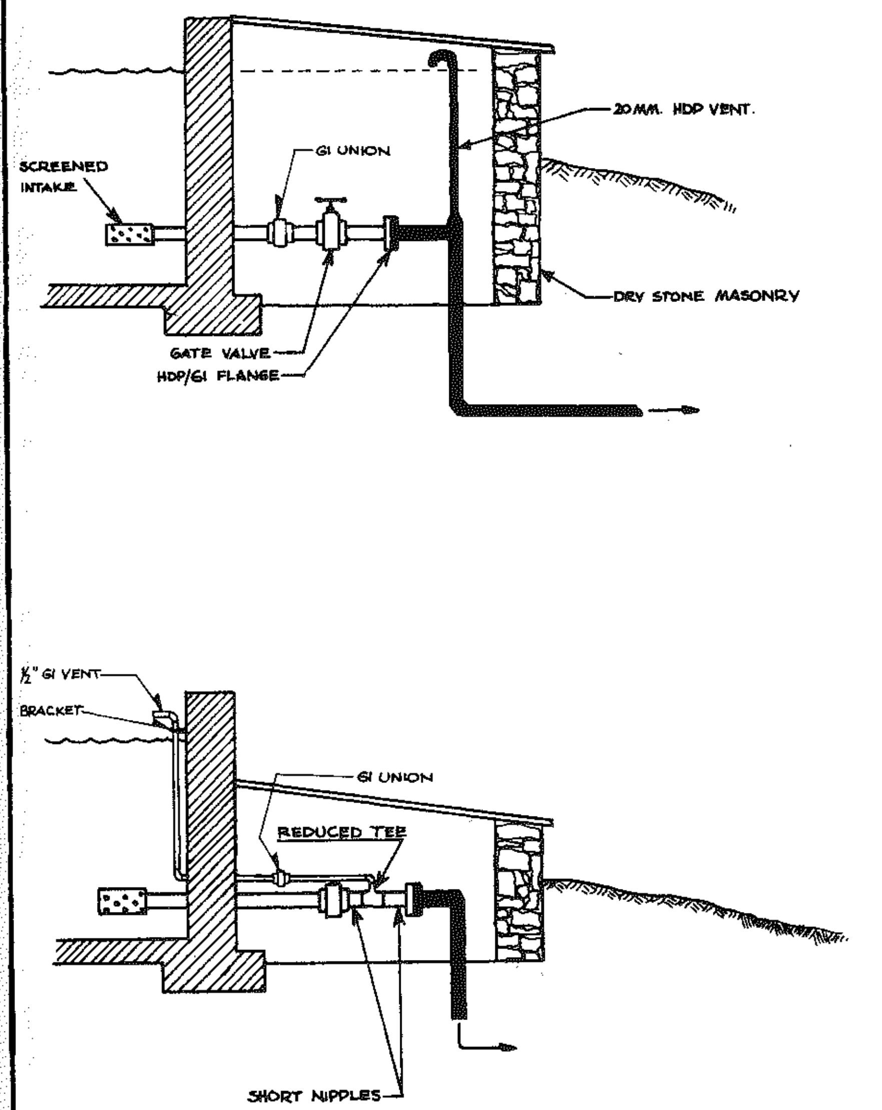


FIGURE 11-6 TYPICAL OUTLET PIPING

Concrete slab roofing: Either of reinforced concrete (RCC) or reinforced (RF) bricks. These are the ideal roofs, since they will totally seal off the intake, and last the lifetime of the system. However, such roofs do require additional materials and costs. Refer to Chapter 19.13 for technical details.

11.10 PROTECTIVE MEASURES

It is important that, once the source water has been collected, it must be protected from further contamination. Thus, measures must be taken to seal off the flow from as much of the external environment as possible.

Surface run-off of rain must not be allowed to flow into the catchment of springs, therefore the intake structures should be minimally 30 cm

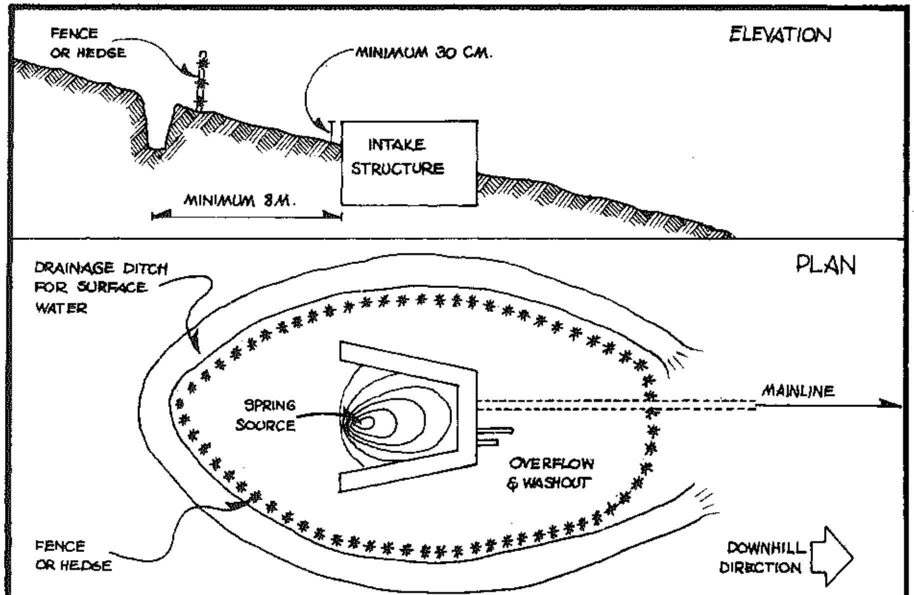


FIGURE 11-7
PROTECTION OF A SPRING CATCHMENT

above ground level. Dirt should be mounded against the tank walls so that water will be turned away, and the source should have a drainage ditch dug around its uphill side. This channel should be deep, and can be lined with dry-stone masonry. Each year, particularly just before the monsoon season, it should be cleared of accumulated debris.

The catchment of a spring source can be roofed over with a concrete slab, and buried for further protection.

If necessary, retaining walls of gabions or dry-stone masonry should be built to stabilize the land around the intake works, especially if erosion is foreseen to be a major problem over the lifetime of the system.

Re-forestation and planting of grass and bushes directly above spring sources greatly aids in maintaining the flow from the source (vegetation allows surface water to seep into the ground rather than disappear quickly as surface run-off. Such water can add to the yield of the source).

If necessary, fencing should be built around the structures to keep away grazing animals, children, etc. Discuss such measures with the LDD regional engineer.

Figure 11-7 Shows suitable protection for a spring catchment

11.11 MULTIPLE SOURCES

Some systems will actually be supplied by the combined flows from two or more sources (particulary if the sources are low-yield springs). Such multiple sources can be handled in any convenient manner, as determined

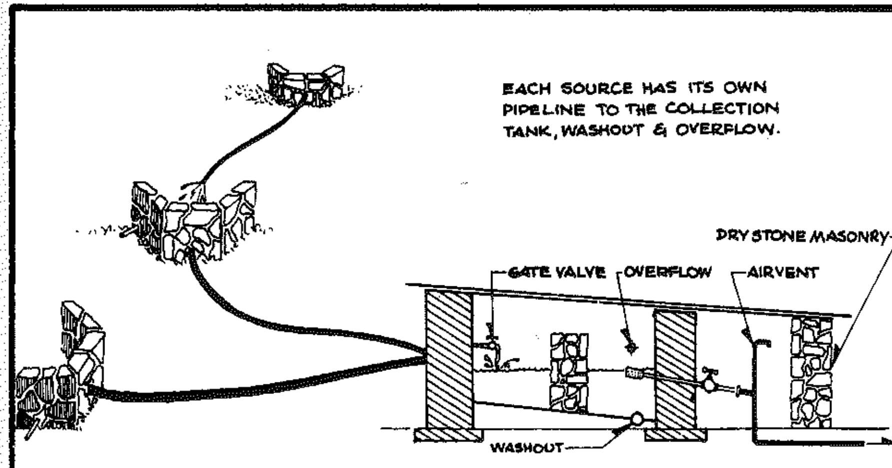


FIGURE 11-8
MULTIPLE SOURCES & COLLECTION TANK

by the distances and elevations between the sources. Flow from higher sources can be piped directly into lower sources, or each source may have its own individual pipeline to a single collection tank or sedimentation tank. Each catchment requires its own washout and overflow pipes, but a gate valve can be located at the discharge point into the collection tank. It is not necessary that each catchment have its own settling chamber, so long as the total flow has the opportunity to settle.

Figure 11-8 shows such a possible arrangement of catchments and collection tank.

11.12 STREAM CATCHMENTS: Dams and Basins

This section deals with construction of total or partial dams across streams, to form a sheltered basin of water for a stream intake. The purpose of the basin is to allow adequate water depth over the mouth of an intake pipe, and to allow the heavier sediments to settle out (since turbulent streams carry sand, and even small stones).

Figure 11-9 shows both a total dam, and a partial dam:

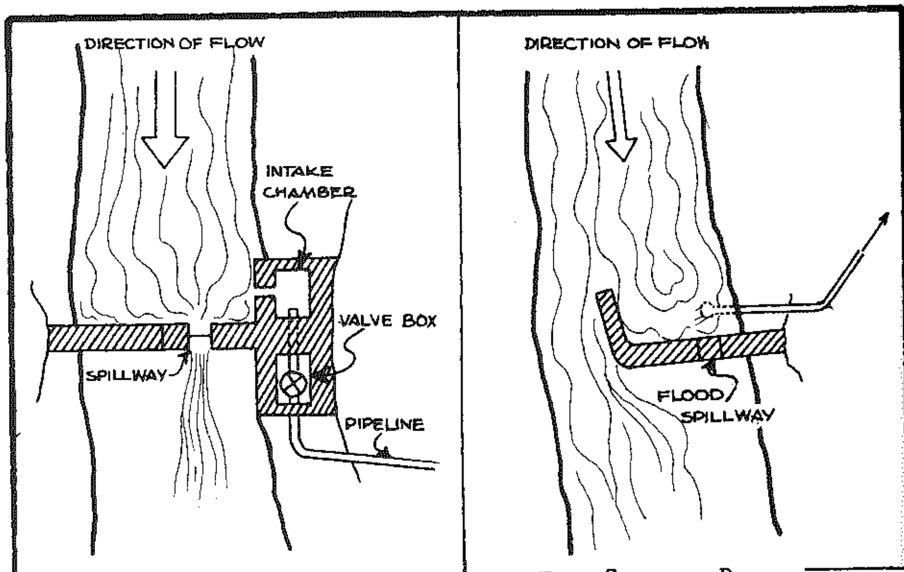


FIGURE 11-9
TOTAL & PARTIAL DAMS

The important concepts that must be kept in mind when designing dams are as follows:

- when water is backed up to the maximum flood level, it must not flood the surrounding land;

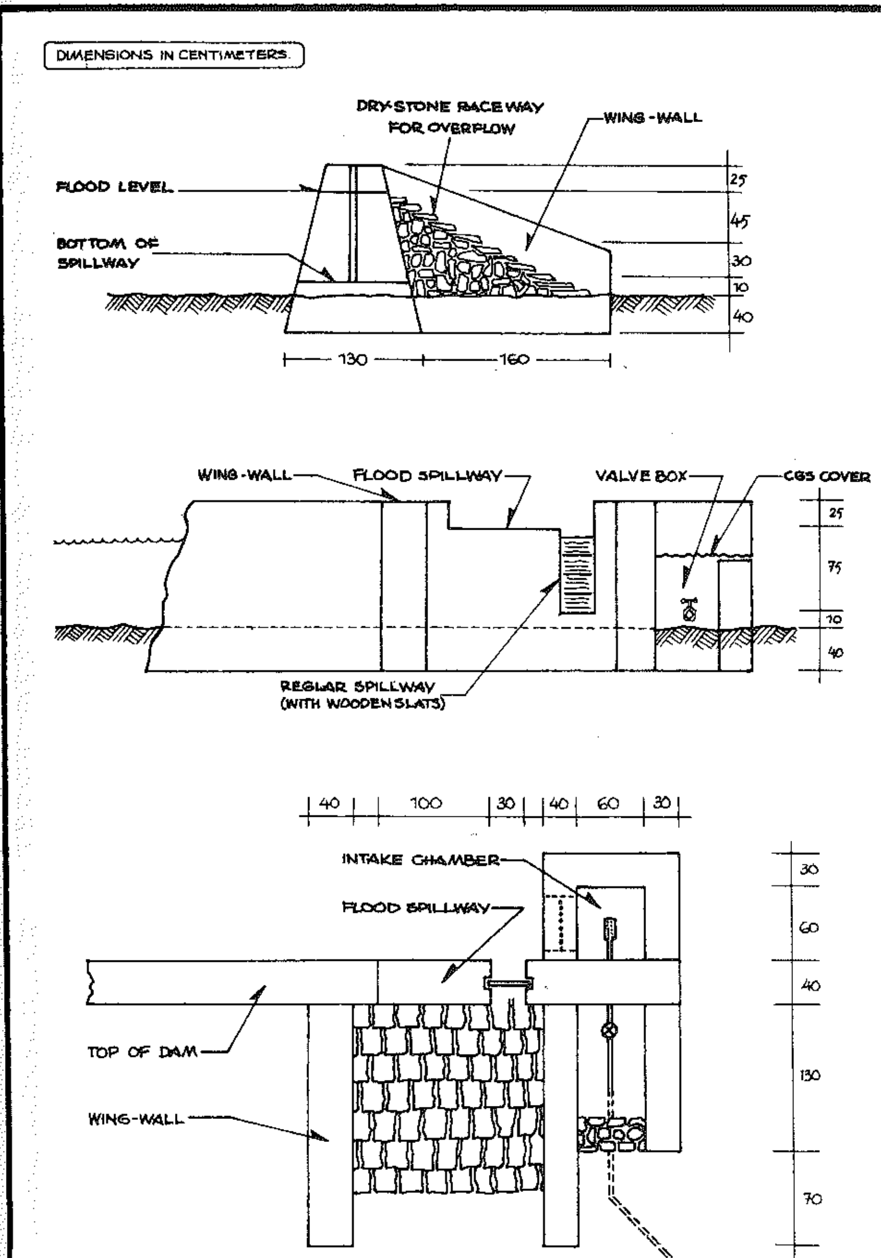
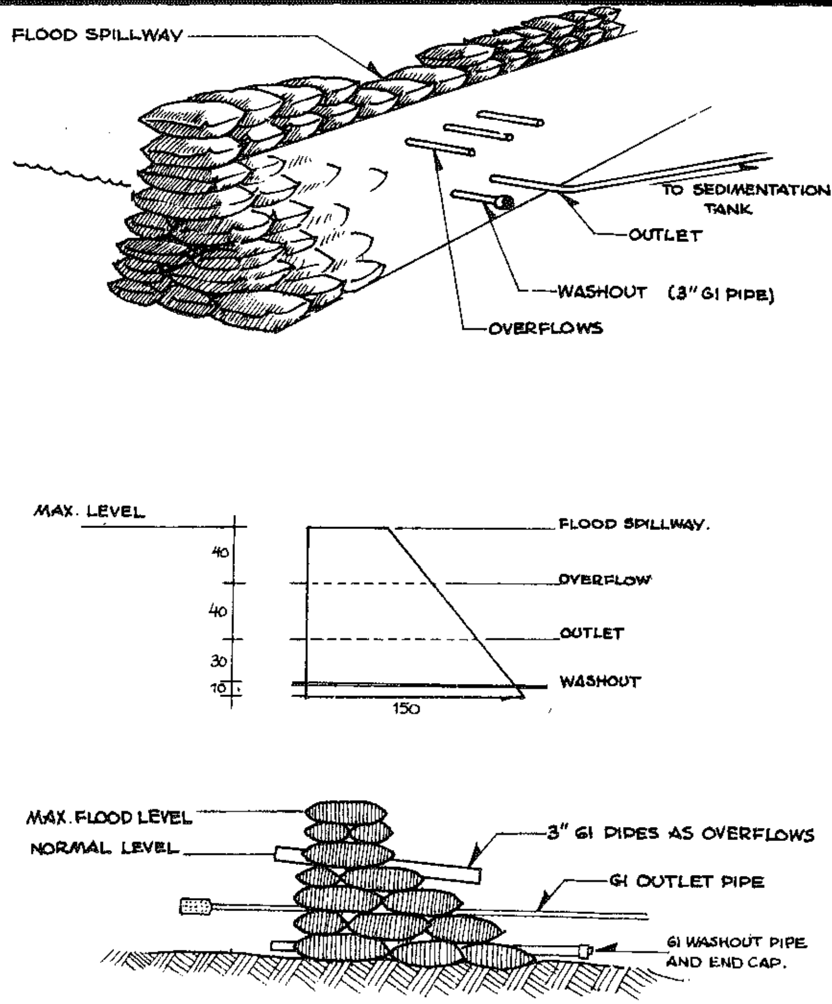


FIGURE 11-10
MASONRY DAM DETAILS



NOTES:

1. OVERFLOWS = 3" GI X 150 CM; WASHOUTS = 3" GI X 200 CM W/ ENDCAP } MAX FLOW = 15 LPS FOR EACH.
2. WASHOUTS MUST BE LOCATED DIRECTLY BELOW INTAKE.
3. VOLUME PER BURLAP (JUTE) BAG = 35 LITERS
   { EACH BAG @ 1:6:8 CEMENT : SAND : AGGREGATE.
   { 28 BAGS PER 1-M^3 OF DAM.

FIGURE 11-11
BURLAP (JUTE) BAG EMBANKMENT

- excess water (ie- overflow) can be safely handled without causing erosion and collapse of the dam or stream banks;

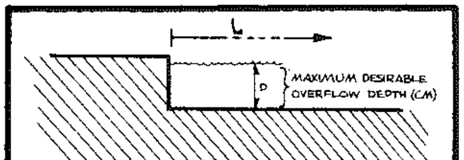
- a total dam will probably be used as a bridge for human, and possibly animal, traffic across the stream, especially if it is conveniently located.

A dam may be built of cement masonry, or by embankments of concrete-filled burlap (jute) sacks.

Cement masonry dams: A cement masonry dam can only be built when the stream flow is completely diverted away from the fresh masonry. Temporary, diverting dams can be made using sand-filled burlap sacks. Dimensions of a good masonry dam are shown in Figure 11-10. The wooden slates of the spillway are removable, which allows full draining of the basin (which, in turn will carry away much of the accumulated silt in the vicinity of the intake chamber).

Burlap (jute) embankments: An easier type of dam to construct is an embankment of concrete-filled burlap (jute) bags. The bags are filled with a fairly-dry concrete mix (1:6:8 cement:sand:gravel) and sewn shut. They can be placed directly in the water as long as there is no hard current flowing against them (fresh cement bags can have a protective facing of ordinary sand-filled bags in front of them, or a diverting dam can be used to absorb most of the hard currents). The burlap material holds the concrete into position until it has set; the bags will mold themselves tightly together under their own weight, so that they'll interlock solidly. Lengths of 10mm Ø rebar can be driven vertically through several layers of bags, "spiking" them together. GI pipe can be easily set in place as the bags are being layered. Several washout and overflow pipes (of 3" GI pipe) may be required, depending upon the maximum flood flows of the stream. Washouts should be placed in the vicinity of the intake pipe, so that silt can be washed away whenever the basin is drained. Figure 11-11 shows details of this type of dam.

Spillways: Both type of dams should have emergency spillways. These are low points along the top of the dams which will overflow first with high flooding flows. This overflow will be confined to a special channel, which will carry the excess flows away without causing erosion problems. The spillways should have masonry wing-walls, and a bed of dry-stone masonry to absorb the hard flow currents of the overflowing water. Figure 11-12 presents the overflow capacity of spillways of different depths. For example, a spillway 20 cm deep and 90 cm long can handle an overflow of more than 124 LPS.



OVERFLOW CAPACITY OF SPILLWAYS
(PER 30CM. OF L)

D	LPS
5	4.8
10	14.3
15	26.7

D	LPS
20	41.4
25	57.1
30	75.6

FIGURE 11-12
SPILLWAY DESIGN

11.13 EXAMPLE DESIGNS

Figure 11-13 shows different examples of designs and intake structures that have been successfully used for developing stream and spring sources in Nepal.

For futher designs, refer to the "Technical Training Manual No.5" published by LDD/UNICEF/SATA.

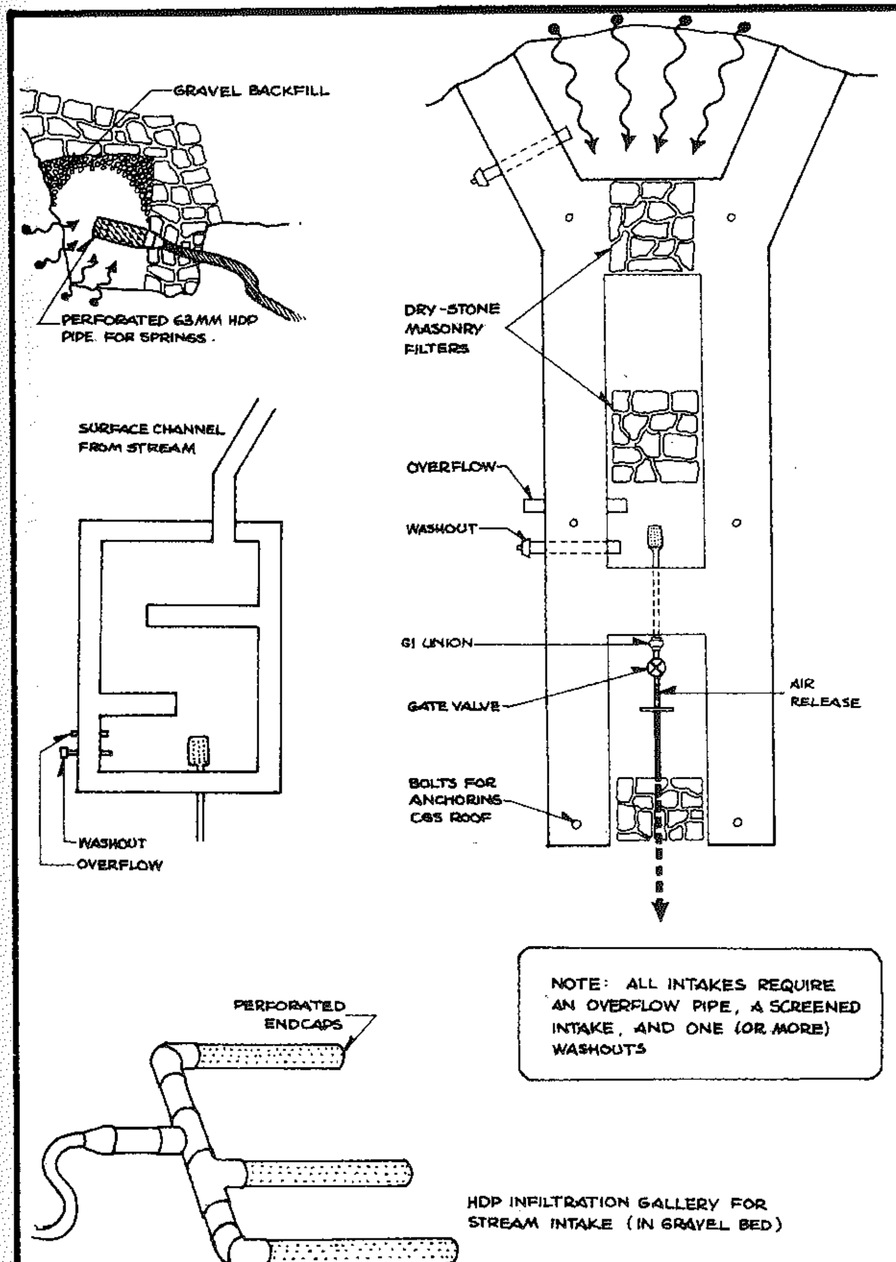


FIGURE 11-13
EXAMPLE INTAKE DESIGNS

12. SEDIMENTATION TANKS

12.1 INTRODUCTION

Water from stream sources and large springs typically contains suspended particles, since the turbulence of large flows can churn up clay, silt, sand, and even small pieces of gravel. Such particles carried in the flow can give the water a dirty, unappetizing appearance and taste, and also add substantially to the erosion of the HDP pipe. If, however, the water is allowed to sit relatively quietly in a tank for some period of time, much of these suspended particles may sink and settle out to the bottom of the tank. This process is called sedimentation, and is accomplished in sedimentation tanks, specially designed for this purpose.

Sedimentation tanks should be built for all systems using stream sources, and for those spring sources where the water is visibly dirty or cloudy. This chapter will present the technical procedures for designing adequate sedimentation facilities for a system.

12.2 SETTLING VELOCITIES

When sediment-laden water is allowed to sit quietly without any turbulence, the suspended particles will tend to sink downwards under the influence of gravity. Typical settling velocities for various particles are given below:

Type of Particle	Diameter (mm)	Settling Velocity (cm/Min)
Coarse sand	1.00	600
	0.50	318
Medium sand	0.50	318
	0.25	156
Fine sand	0.25	156
	0.10	48
Very fine sand	0.10	48
	0.05	15.6
Silt	0.05	15.6
	0.01	0.924
Fine silt	0.01	0.924
	0.005	0.0385
Clay	0.01	0.154
	0.001	0.00154

Smaller particles (e.g. fine clay or bacteria) either do not settle or have a negligible settling rate.

From the above information, it can be calculated that a settling period of about 20 minutes will allow some fine silts and larger particles to settle out of the upper layers of water in the sedimentation tank. This clean surface water is "skimmed off" and channeled into the pipeline.

12.3 DETENTION TIME

The period of time that the water spends in the sedimentation tank to allow settling is called the detention time. The amount of detention time required depends upon several factors: quantity of flow, amount of suspended particles and their size, surface area of water in the tank, presence of a reservoir tank further downstream (if the system requires one). A reservoir tank will allow about 10 hours of undisturbed settling time each night, so such a system does not require as large a sedimentation tank.

Recommended detention times:

Small, clean spring sources: no sedimentation tank*

Systems with reservoirs: 15-20 minutes

Systems without reservoirs: 60 minutes minimum

12.4 CAPACITY

When the detention time has been selected, the required capacity of the sedimentation tank can be calculated:

$$C = Q \times T$$

where: C = capacity (liters)
Q = flow (LPS)
T = detention time (seconds)

12.5 TANK SPECIFICATIONS

The dimensions of the sedimentation tank may be adjusted in such a way as to accomodate any particular site location, but certain design characteristics must be incorporated into the design:

L/W Ratio: The length/width of the water surface area should be at least 4. This allows the initial discharge turbulence to die away.

Water Depth: The depth of water is best between 75-100 cm.

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* however, the intake should have a settling chamber with the prescribed screening, as discussed in Chapter 11.5 & 11.6

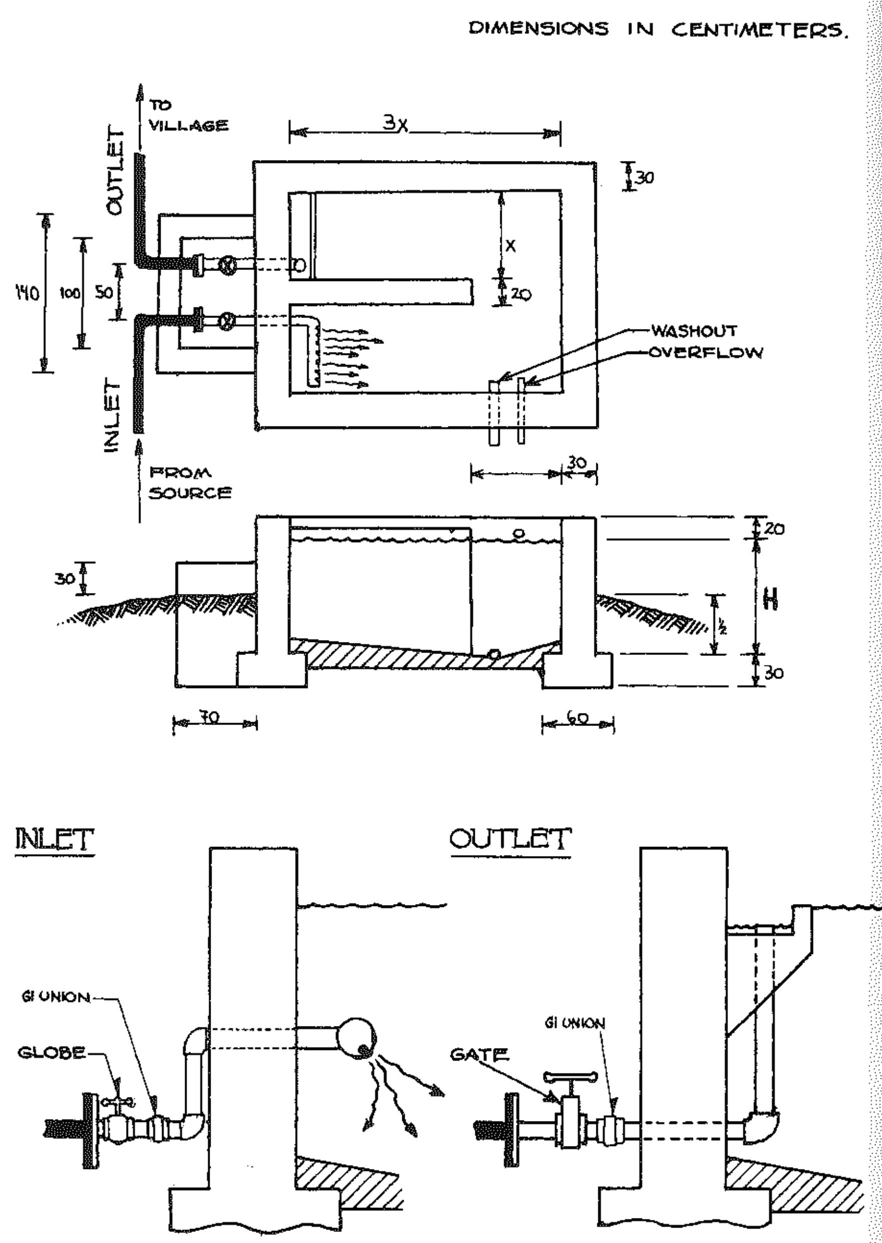


FIGURE 12-1
RECOMMENDED SEDIMENTATION TANK

Inlet: The discharge of the flow into the tank should be distributed as evenly as possible across the width of the water path. The depth of discharge should be about halfway between the surface and the floor of the tank, as shown in Figure 12-2. The pipe should be of 1" or 1½" GI, with a perforated length of larger HDP pipe. A globe valve is needed to regulate the flow.

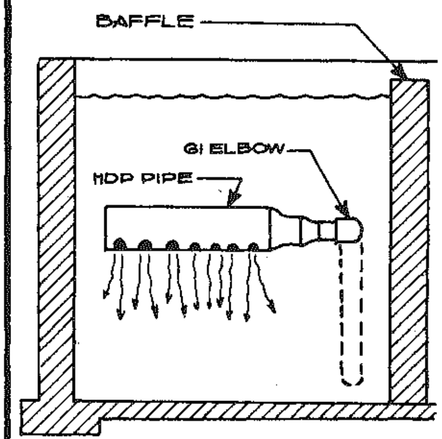


FIGURE 12-2
DETAILS OF INLET

Outlet: The outlet should be designed to collect just the very surface layer of water, from across the full width of the water path. The easiest way to accomplish this is with a collection gutter, as shown in Figure 12-3. The outlet piping should be of GI pipe, according to flow:

GI Size	Flow (LPS)
1"	up to 0.35
1½"	" " 0.85
2"	" " 1.40
3"	greater than 1.40

The outlet should have a gate valve with air-vent.

Washout: The washout should be at least 2" GI pipe with an endcap, set in the bottom of the tank, with suitable drainage for the washout flow.

Overflow: As presented in Figure 11-5.

Flow velocity: The water velocity flowing through the tank should not exceed 0.50 cm/sec. Greater velocity may create turbulent currents which hinder the sedimentation process. The velocity is calculated as follows:

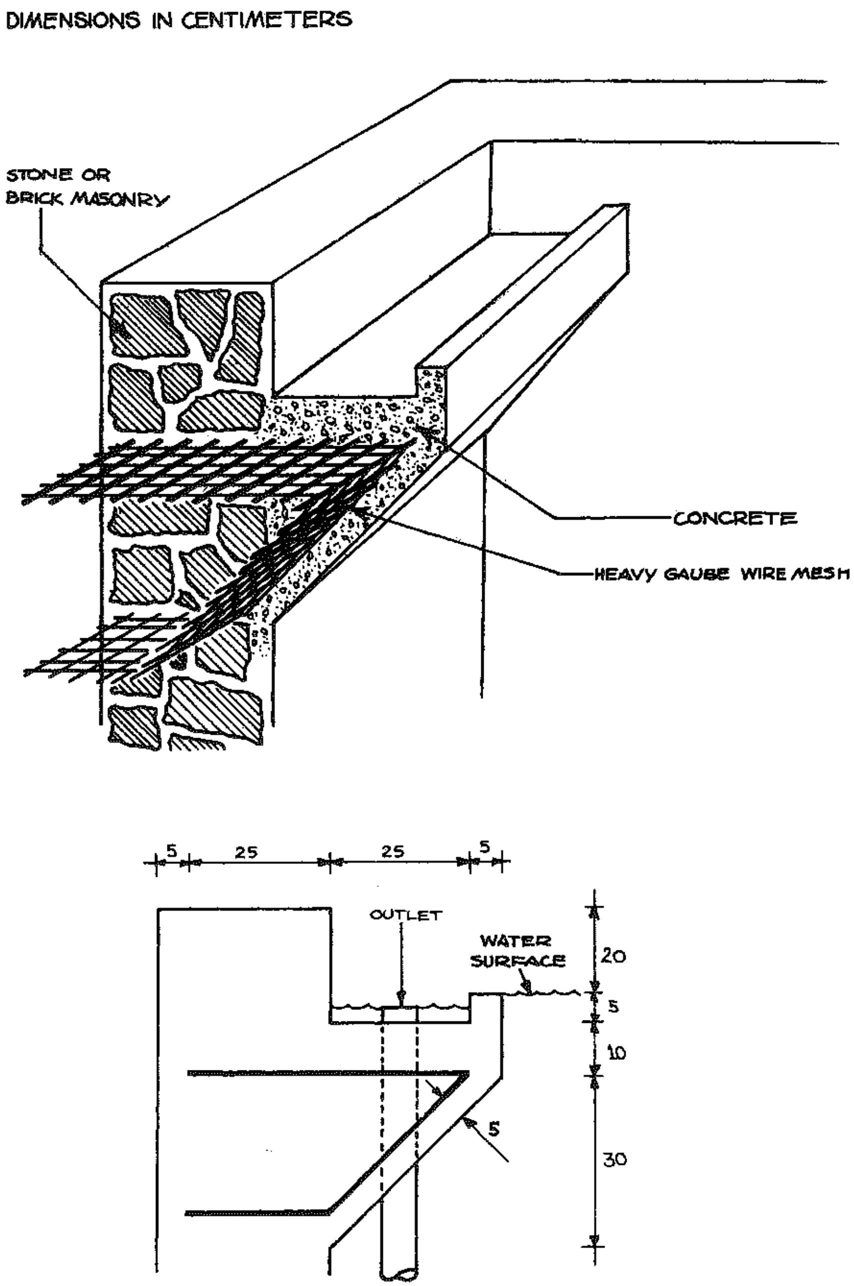


FIGURE 12-3 DETAILS OF COLLECTION GUTTER

$$V = \frac{1000Q}{WD}$$

where: V = velocity (cm/sec)
Q = flow (LPS)
W = width of water path (cm)
D = depth of water (cm)

Baffles: One or more partitioning walls (baffles) may be used to subdivide the surface of the water so the L/W ratio is improved without increasing the external size of the tank. Baffles should extend the full depth of water, and be of masonry construction.

Excavation, foundation, & walls: The depth of excavation should be enough to half-bury the walls. The floor of the excavation should be level and firm (compacted if necessary). The walls should be built on a masonry footing (as discussed in Chapter 11.3). The walls should be 30 cm thick, of 1:4 cement:sand masonry. The tank floor and plastering is done per specifications in Chapter 19.12 & 19.13.

A recommended design for a sedimentation tank which meets (or exceeds, the above specifications is shown in Figures 12-1, -2, & -3.

13. BREAK-PRESSURE TANKS

13.1 INTRODUCTION

The function of a break-pressure tank is to allow the flow to discharge into the atmosphere, thereby reducing its hydrostatic pressure to zero, and establishing a new static level. Strategic placing of break-pressure tanks can minimize the amount of Class IV and GI pipe which must be used in a system (except where there are U-profiles). In Chapter 8.3, the design example of the mainline included four break-pressure tanks, and discussion was presented about the various strategies to locate them.

It is anticipated that shortly the LDD office will have developed standardized designs for break-pressure tanks, complete with detailed estimate lists. Therefore, this chapter will present just the basic design principles and characteristics of such tanks which have been successfully constructed in Nepal.

13.2 TYPES OF TANKS

Currently, break-pressure tanks can be constructed of cement masonry (with/without float valves) or HDP pipe. Investigation is underway about developing pre-fabricated break-pressure tanks of HDP, GI sheet metal, and ferro-cement.

13.3 MASONRY TANKS

There is no minimum required capacity for a break-pressure tank, as long as water is able to drain from it as fast as it is discharged. The dimensions of the tank are more influenced by the size of the fittings (such as control valves, float valves, etc) which must fit inside of it (and size of the pipewrenches which must be able to swing around inside as well). The tanks can be designed so that they are easily covered by a half-sheet of CGS roofing, or by a small RCC slab, or by slate (if locally available).

Specifications for masonry break-pressure tanks are as follows:

Excavation, Foundation, & Walls: Excavation for a tank should be 30 cm into firm soil, and the floor of the excavation leveled and compacted. A layer of gravel and masonry footing should be built, as specified in Figure 11-1. Minimum height of the wall above ground should be 20 cm; the ground should be pitched away from the tank and have a drainage ditch to divert rain run-off. Drainage provisions must be made for the washout and overflow, and surrounding ground should be stabilized if necessary. Masonry walls should be minimally 20 cm thick (30 cm if there is a GI pipe inbedded in it) of 1:4 cement:sand mortar, plastered accoring to Chapter 19.12.

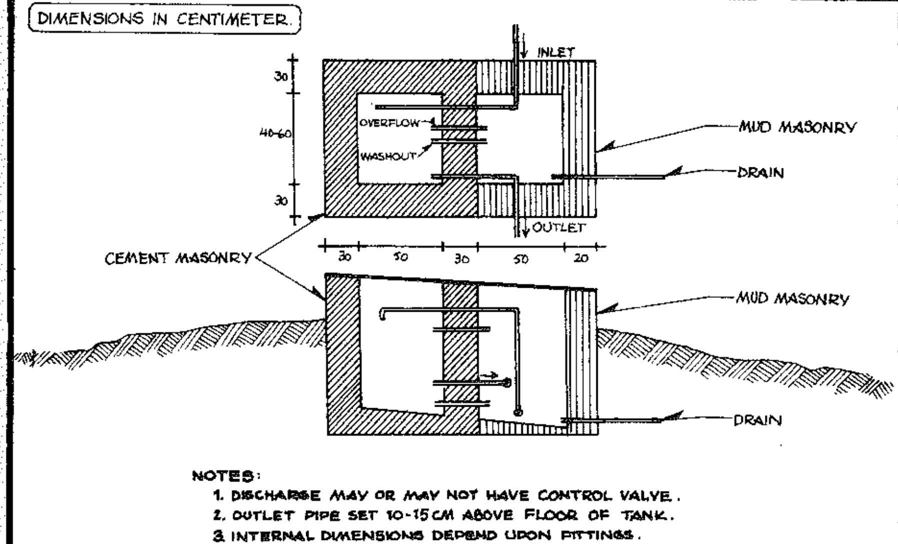


FIGURE 13-1
EXAMPLE DESIGNS OF MASONRY BPTs

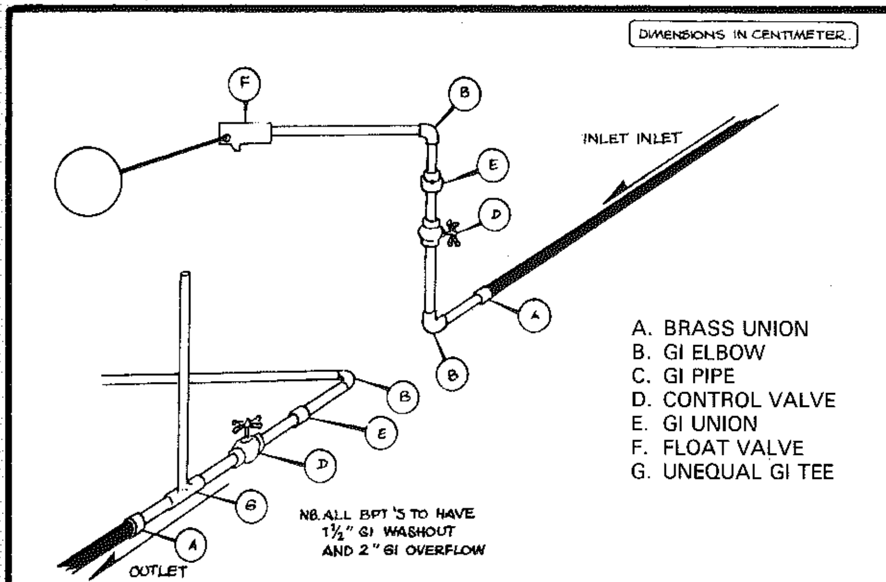


FIGURE 13-2
GI PIPE ARRANGEMENTS FOR BPTs

Inner dimensions: Must be large enough to accomodate the fittings which will be installed. Minimum width = 40cm; minimum length = 40 cm (80cm if a float valve is to be installed), and depth to be 50cm. Can be adjusted to accomodate dimensions of CGS roofing.

Control valves: If a globe valve is needed to regulate the discharge flow, it can be installed directly on the discharge. If a float-valve is to be installed, then a gate valve should be installed as well (as shown in Figure 13-2) so that the flow can be shut off when installing/removing the float-valve. NOTE: Make sure that the height of the roof will not interfere with the float-valve operation, and that the tank overflow is set higher than the float-valve.

If a gate valve is to be installed on the outlet pipe, then a valvebox can be built onto the tank, or the gate valve installed in an external valvebox of GI pipe (refer to Chapter 16.8).

Service pipes: The inlet pipe should be of GI pipe, with the discharge flow directed directly downwards towards the floor of the tank (if allowed to spray against the walls, the plaster will soon be eroded away). The outlet pipe should be of GI pipe one size larger than the pipeline it connects to, and should be located 10-15 cm above the floor of the tank (this will create a "cushion" of water in the bottom of the tank, which will absorb much of the energy of the discharge flow). All tanks should have an overflow (refer to Figure 11-5) and also a washout of 1½" GI pipe.

Figure 13-2 gives some specifications of the GI pipe for a masonry break-pressure tank.

Roofing: Break-pressure tanks can be covered with CGS sheeting, a reinforced concrete (RCC) slab, or slate. Either CGS or RCC roofing is recommended if the tank has any internal control valves, since these are the most secure covers. For a CGS cover refer to Chapter 20.4; for an RCC slab refer to Chapter 19.15; and for slate refer to the villagers.

Additional ideas: Placing a hard, flat rock directly below the discharge will provide even further protection to the floor of the tank. Outlets may be screened if desired.

13.4 HDP TANKS

HDP break-pressure tanks have several advantages and disadvantages, some of which are as follows:

Advantages: Lightweight; quickly and easily fabricated in a workshop; quick and easy to install; require small sites; provide good protection of the flow from contamination; made from materials which are usually readily available (excess HDP pipe and reducers).

Disadvantages: Not as sturdy as masonry tanks; more difficult to install control valves (require external valveboxes); require some protective dry-stone masonry.

HDP break-pressure tanks should be installed only where the flow has been well-screened so that sediments cannot accumulate and clog the tanks. The "snorkel" of 50mm HDP should have several screens, since the outermost ones are susceptable to being punctured by children (refer to chapter 20.2 for ideas on screening HDP pipe). A stone masonry valve box is needed to protect the snorkel and a drain pipe and ditch to carry away any overflow.

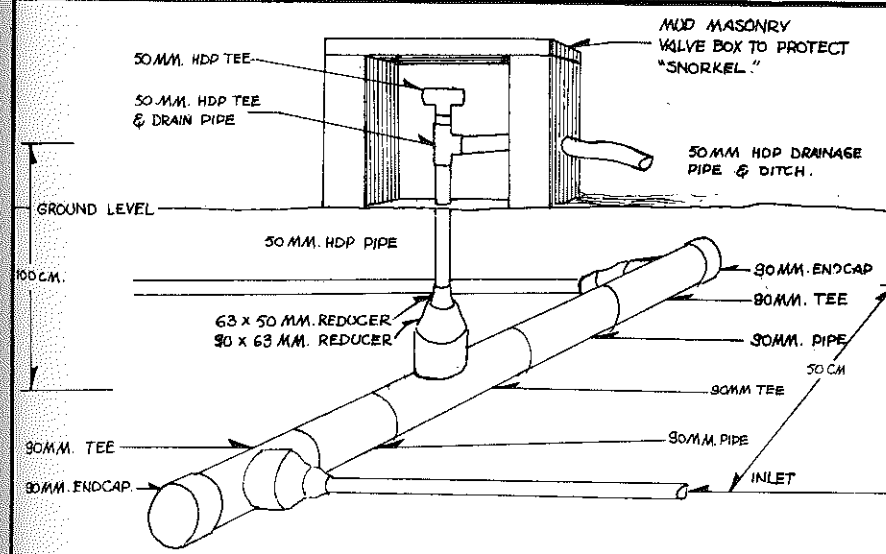


FIGURE 13-3
HDP BREAK-PRESSURE TANK

The rate of flow out of the HDP break-pressure tank will be dependant upon the head of water acting at the outlet pipe. This head will be limited by the height of the snorkel overflow tee above the outlet pipe. Consequently, the HDP break-pressure tank should be at least one meter below the overflow tee, i.e. about 90cm below ground level.

14. RESERVOIR TANKS

14.1 INTRODUCTION

The construction of the reservoir tank will be the most visible effort of the entire system. It will require the coordinated tasks of dozens of people, ranging from the masons who lay the stone to the assistants who mix cement, to the men, women (and sometimes even children) who collect stones from the fields or porter sand up from the rivers. When completed, the reservoir becomes a public monument of the village and a source of pride to the villagers, especially if the project is considered a successful one.

When it comes to designing the reservoir tank, the most common attitude of the villagers is "the bigger the better!". While this is an understandable idea, there 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 (as discussed in Chapter 4) and the safe yield of the source. The actual dimensions of the tank are determined by its capacity, the conditions at the site selected, and rules of economical design.

This chapter will present all the procedures and knowledge that is necessary to design and construct practical reservoir tanks. At the end of the chapter is an example design.

14.2 THE NECESSITY FOR A RESERVOIR

Although the village water needs are based upon a minimum requirement of 45 liters per person per 24-hour day, in actuality just about all of this water will be demanded during daylight, a period of 10-12 hours. The reservoir tank serves to store water that is provided by the source during low-demand periods (such as overnight) for use during high-demand periods (such as early morning).

A system will require a reservoir when:

- The safe yield of the source will not directly provide 0.225 LPS for each tap;
- The daily water demand is greater than the yield of the source during the daylight hours;
- The pipeline distance from source to village is so far that it is more economical to use a smaller pipe size and build a reservoir tank

14.3 CAPACITY

To determine how large a reservoir tank must be- it is necessary to calculate how much water is demanded at various times during the day, and compare this to how much water is yielded by the source for those same time periods. The difference will either mean that water will be drawn out of the reservoir or will flow into the reservoir.

The maximum size of a tank should not be greater than needed to store the water yielded by the source during the night. It is possible to sometimes design a tank to take advantage of the higher yields during the monsoon season if the dry-season safe yield is not enough.

The daily demand pattern of a typical village would be somewhat similar to either of the schedules below:

Schedule 1

6:00 AM - 8:00 AM ......30% of total daily water need
8:00 AM - 4:00 PM ......40% " " " " "
4:00 PM - 6:00 PM ......30% " " " " "
6:00 PM - 6:00 AM ......Negligible water demand

Schedule 2*

5:00 AM - 7:00 AM ......10% of total daily water need
7:00 AM -11:00 AM ......25% " " " " "
11:00 AM - 1:00 PM ......35% " " " " "
1:00 PM - 5:00 PM ......20% " " " " "
5:00 PM - 7:00 PM ......10% " " " " "
7:00 PM - 5:00 PM ......Negligible water demand

* This schedule observed by C. Johnson

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

The second schedule is based upon direct observation by Johnson of a 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.

In practical applications, use whichever schedule requires the smaller-sized tank, for the villagers will adjust their demand patterns to whatever schedule the tank can provide.

Capacity design example:

The projected population of a village is 400 persons, with no other special water needs. Safe yield of the source is 0.45 LPS, and five tapstands are to be built.

Since the source is not large enough to supply more than two of the tapstands by itself, a reservoir tank is required. Using the two demand schedules, the following water demands are calculated:

TIME PERIODS	SUPPLY	DEMAND	DIFFERENCE	
Schedule 1				
6 AM - 8 AM (2 hrs, 30%)	3240	5400	-2160	(water withdrawn)
8 AM - 4 PM (8 hrs, 40%)	12960	7200	+5760	(tank overflows)
4 PM - 6 PM (2 hrs, 30%)	3240	5400	-2160	(water withdrawn)
	Largest deficiency = 2160 liters			
Schedule 2				
5 AM - 7 AM (2 hrs, 10%)	3240	1800	+1440	(tank overflows)
7 AM - 11 AM (4 hrs, 25%)	6480	4500	+1980	(" ")
11 AM - 1 PM (2 hrs, 35%)	3240	6300	-3060	(water withdrawn)
1 PM - 5 PM (4 hrs, 20%)	6480	3600	+2880	(tank refilling)
5 PM - 7 PM (2 hrs, 10%)	3240	1800	+1440	(tank overflows)
	Largest deficiency = 3060 liters			

For this example, the required storage capacity is determined by Schedule 1, at 2160 liters. For practical design, consider this 2200 liters (2.2 cubic meters).

Capacity design example:

The projected population of a village is 780 persons, with no other special water needs. Safe yield of the source is 0.45 LPS, and five tapstands are to be built.

Again, a reservior tank is required.

TIME PERIODS	SUPPLY	DEMAND	DIFFERENCE	
Schedule 1				
6 AM - 8 AM (2 hrs, 30%)	3240	10530	-7290	(water withdrawn)
8 AM - 4 PM (8 hrs, 40%)	12960	14040	-1080	(" ")
4 PM - 6 PM (2 hrs, 30%)	3240	10530	-7290	(" ")
	Largest deficiency = 15660 liters			
Schedule 2				
5 AM - 7 AM (2 hrs, 10%)	3240	3510	- 270	(water withdrawn)
7 AM - 11 AM (4 hrs, 25%)	6480	8775	-2295	(" ")
11 AM - 1 PM (2 hrs, 35%)	3240	12285	-9045	(" ")
1 PM - 5 PM (4 hrs, 20%)	6480	7020	- 540	(" ")
5 PM - 7 PM (2 hrs, 10%)	3240	3510	- 270	(" ")
	Largest deficiency = 12420 liters			

In this example, the required capacity is determined by Schedule 2, at 12420 liters. For practical designing, consider this to be 12500 liters (12.5 cubic meters).

14.4 SHAPE

When the required capacity of the reservoir tank has been calculated, it is then time to begin determining the shape and dimensions of the tank. This is usually a compromise procedure that may have to be repeated two or three times before the optimum design is discovered.

All other factors being equal, the most economical tank shape is circular, then nearly-circular, then square, and then rectangular. For ease of construction, certain shapes are easier than others:

Circular tanks: The most economical shape to use, but not easy to construct, especially for small diameters.

Octagonal (8-sided) tanks: The best shape to use, but not easy to construct for diameters less than 2½ meters (or capacities smaller than 3200 liters).

Hexagonal (6-sided) tanks: Good for tanks between 1700-3200 liters (diameters not less than 2 meters).

Square tanks: This is the traditional shape, and easiest to construct for small capacities (such as mini-tanks, break-pressure tanks, etc).

Rectangular tanks: The least-economical shape, especially as one side becomes much longer than the other. However, due to physical constraints of the site, it may be necessary to use this shape. Keeping it as nearly square-shaped as possible will make a more economical design.

Special note for CGS-roofed tanks: When a square or rectangular tank is to be roofed with CGS, it is easier to slightly adjust the dimensions of the tank so that it is neatly covered by the sheets (such as "5 sheets wide by 1½ sheets long"). This helps to minimize the amount of CGS sheet cutting, which is a relatively difficult task. For the multi-sided tanks this is not so easy to do, but should still be kept in mind.

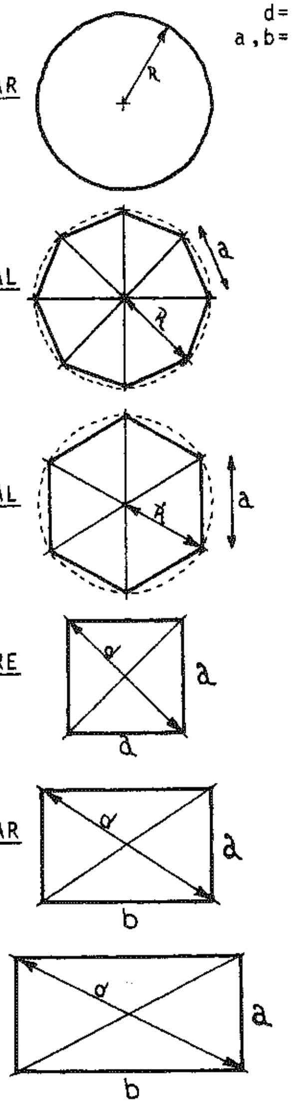
Figure 14-1 is a table of these various tank shapes, giving the simple mathmatical equations for determining their dimensions once capacity and water depth have been selected.

14.5 WALL DESIGN

The type of walls used in construction of these reservoir tanks are known as "gravity-walls": they resist being overturned (by the hydrostatic water pressure) by virtue of their weight alone. The design of the wall is determined by the material of construction (ie- brick or stone) and the selected water depth.

$A = \frac{C}{10H}$

A= water surface area (m^2)
C= capacity (liters)
H= water depth (cm)
L= length of perimeter (m)
d= diagonal (m)
a,b= sides (m)



CIRCULAR: $R = 0.564\sqrt{A}$, $L = 6.28R$

OCTAGONAL: $R = 0.598\sqrt{A}$, $a = 0.76R$, $L = 6.08R$

HEXAGONAL: $R = 0.623\sqrt{A}$, $a = R$, $L = 6R$

SQUARE: $a = \sqrt{A}$, $L = 4a$, $d = 1.414a$

RECTANGULAR (1:1½): $a = 0.816\sqrt{A}$, $b = 1\frac{1}{2}a$, $L = 5a$, $d = 1.803a$

(1:2): $a = 0.707\sqrt{A}$, $b = 2a$, $L = 6a$, $d = 2.236a$

FIGURE 14-1 SHAPES & DIMENSIONS OF RESERVOIRS

Water depth: Although it is possible to select any depth of water, for the designs presented in this book certain water depths are more economical than others:

brick masonry: 60, 90, or 105 centimeters depth

stone masonry: 65, 95, or 115 " "

These are the water depths that should be first selected and trial designed. Only if the resulting dimensions of the tank cannot be used should other water depths be considered.

Masonry: Stone masonry is generally heavier than brick masonry, and therefore does not require as large a volume to resist the hydrostatic pressures. For the design table of Figure 14-2, the following specific weights were used:

brick masonry: 2120 kg/m^3

stone masonry: 2450 kg/m^3

A safety factor against over-turning of 1.5 was used.

External walls: These are the outside walls of the tank. Hydrostatic pressure is exerted on only one side, and they are partially backfilled for additional support.

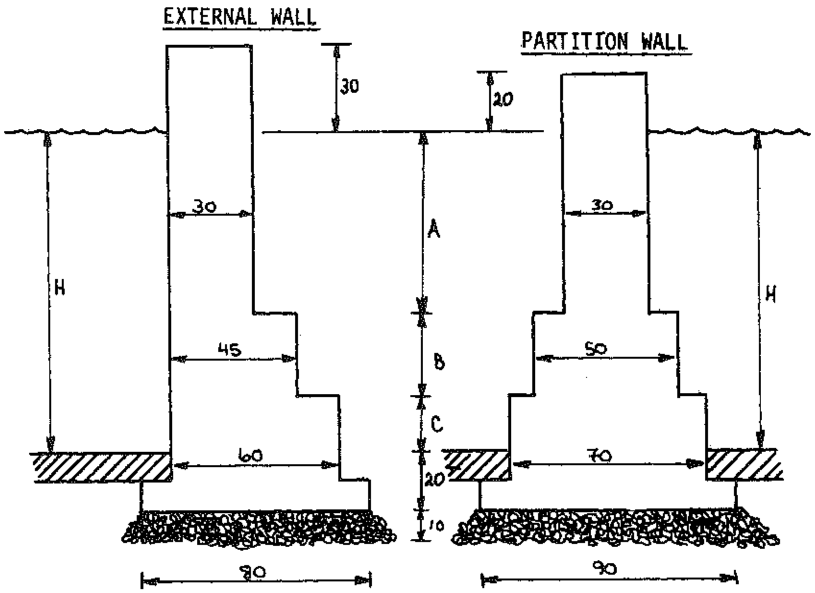
Partition walls: This is a wall which divides the inside of the tank exactly in half. This allows half of the tank to be drained for maintenance purposes while the other half is still providing some service. In practical use, however, such walls use a tremendous amount of extra material and labor, and their use has never been proven worthwhile in Nepal. Partitioned tanks also must be somewhat larger, to replace the storage capacity displaced by the partitioning wall.

Wall design table: Figure 14-2 allows the quick design of the external and partition (if desired) walls of a reservoir tank. constructed of either brick or stone masonry, for various water depths.

14.6 SERVICE PIPES

The pipe arrangement of the reservoir tank requires some particular attention, especially if the tank is partitioned. A reservoir typically requires an inlet (discharge), outlet, by-pass, overflow, and washout. Refer to Figure 14-3.

Inlet: The inlet can be of 1" GI pipe. Where a free discharge is planned, only a single gate valve is required, but if a controlled discharge is required then a globe valve is also necessary (the globe valve can be located inside the tank, and the villagers warned that it must not be adjusted; the gate valve can be inside the valve box. The actual point of discharge should be on the opposite side of the tank from the outlet, so that maximum opportunity for sedimentation is provided.



WATER DEPTH H	EXTERNAL WALL stone masonry A	B	C	brick masonry A	B	C	PARTITION WALL stone masonry A	B	C	brick masonry A	B	C
50	50	-	-	50	-	-	50	-	-	50	-	-
55	55	-	-	55	-	-	55	-	-	55	-	-
60	60	-	-	60	-	-	60	-	-	60	-	-
65	65	-	-	50	15	-	65	-	-	50	15	-
70	55	15	-	55	15	-	55	15	-	55	15	-
75	60	15	-	60	15	-	60	15	-	60	15	-
80	65	15	-	60	20	-	65	15	-	60	20	-
85	65	20	-	60	25	-	65	20	-	60	25	-
90	65	25	-	60	30	-	65	25	-	55	20	15
95	65	30	-	55	25	15	60	20	15	60	20	15
100	60	25	15	60	25	15	60	25	15	60	20	20
105	65	25	15	60	30	15	60	25	20	60	20	25
110	65	25	20	Not permissible			60	25	25	Not permissible		
115	65	30	20				65	25	25			

Notes: these are gravity-walls with a safety factor of 1.5 against overturning, based upon stone masonry @ 2450 kg/m^3 and brick masonry @ 2120 kg/m^3. All dimensions above in centimeters. Approximate depth of excavation: D= $\frac{1}{2}$H + 30

FIGURE 14-2
WALL DESIGN TABLE

Outlet: For a pipe arrangement similar to that shown in Figure 14-3, the following sizes of GI pipe should be used in the outlet:

GI pipe size	Outlet flow
1"	0.33 LPS
1½"	0.80 LPS
2"	1.30 LPS
3"	3.30 LPS

The outlet should be installed with a gate valve and air-vent (refer to Chapter 11.8).

By-pass: The by-pass line is a direct connection between the inlet and outlet lines, so that when the tank is shut down, at least some of the flow can be diverted into the mainline. A gate valve serves to shut off the by-pass when the tank is in use, and is only open when the flow into the reservoir is cut off for maintenance work. When a by-pass is used, it is important to consider static pressures, since the break-pressure effect of the reservoir has been eliminated.

Overflow: The overflow is sized according to Figure 11-5, but since the reservoir tank will be overflowing frequently, special care must be made to ensure drainage of the overflow water does not cause erosion problems.

Washout: The washout should be of 2" GI pipe, with a gate valve. The floor of the tank should be pitched down to the washout, and the washout pipe imbedded in the bottom of it (refer to Chapter 19.13).

Partitioned tanks: A partitioned tank will require just about complete duplication of control valves, since one sub-tank must be isolated from the system at a time. Figure 14-4 shows the general service pipe arrangements for the necessary cross-connections.

14.7 CONSTRUCTION

This section will present the general steps in construction of a reservoir, listing important considerations of each step.

Site selection: The site selected for the reservoir should be on stable ground which will not be threatened by landslides or erosion.

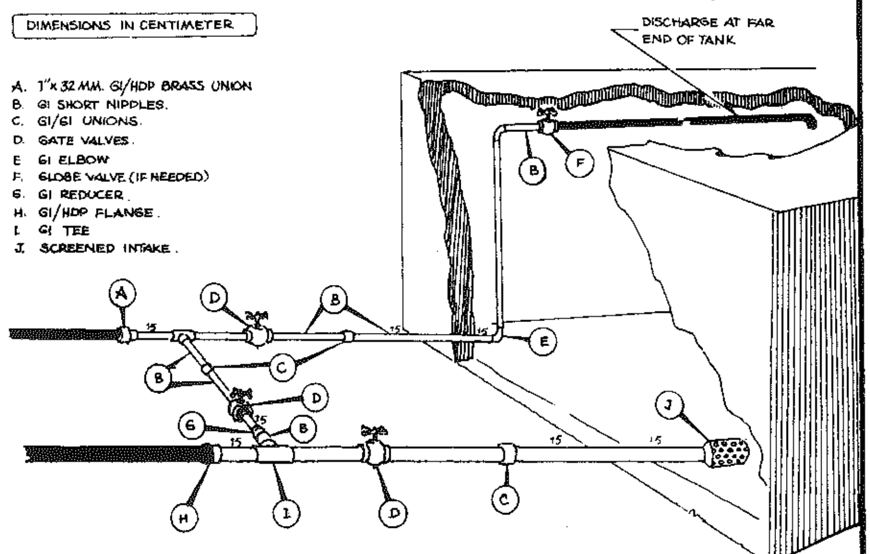


FIGURE 14-3
RESERVOIR GI PIPE ARRANGEMENT

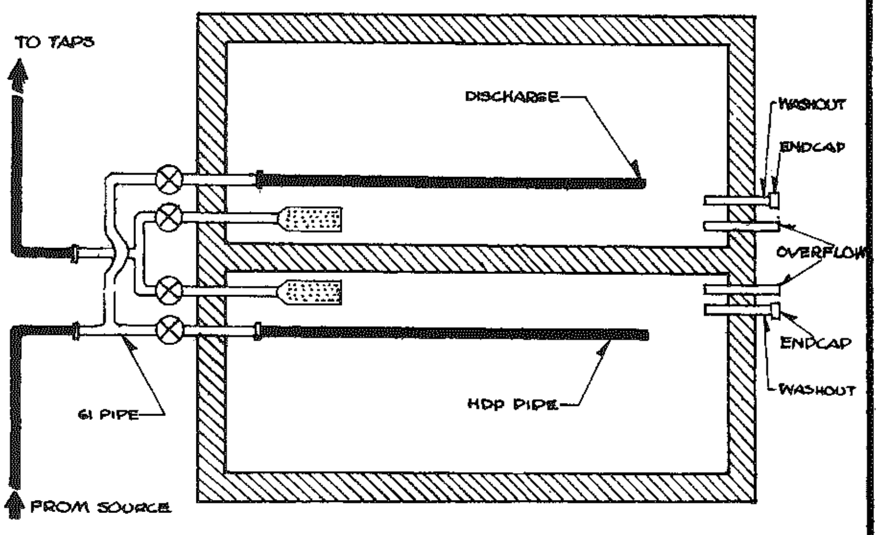


FIGURE 14-4
SERVICE PIPES FOR PARTITIONED RESERVOIR

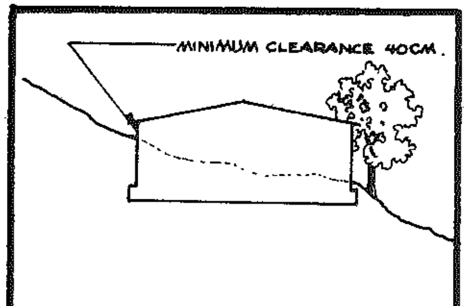


FIGURE 14-5
RESERVOIRS ON SLOPING GROUND

A level ground site is desirable since it will require less excavation, but no site should be used where any wall of the tank will be backfilled too deeply (a minimum of 40 cm of wall must rise above the ground) refer to Figure 14-5. There must be room for stockpiling the construction materials(stone, sand, gravel, etc) and adequate room for the cement-mixing crews to work (for large tanks, the cement-mixing pads can actually be located inside the tank). For a project where many masons will be working, it may be desirable to have two or more mixing pads.

Excavation: The depth of excavation for the tank will depend upon the nature of the soil in the site. Approximate depths of excavation are given in Figure 14-2. In sloping ground, the deepest-buried wall must still rise above the finished ground level by at least 40 cm. Minimum excavation must establish a perfectly level floor, with foundation trenches 30 cm deep for the wall footings. Although gravity-type walls do not require the support of backfill, some excavation is advisable to ensure that the tank is firmly imbedded in the ground, especially if it is on sloping ground. When the excavation is completed, the foundation trenches are staked out (using string and wooden pegs).

Foundations: The foundation trenches should be as wide as the wall footings and 30 cm deep. A bed of gravel or lean concrete 10 cm deep is put down and leveled, then a masonry (or concrete) footing 20 cm high. The regular masonry wall is built up upon this footing. Refer to Figure 14-6.

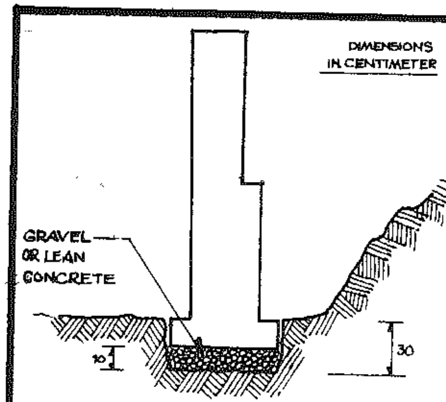


FIGURE 14-6
RESERVOIR WALL FOUNDATION

Wall construction: The masonry walls are of 1:4 cement:sand mortar. As they are built up, especially if the tank is a deep one, stepping-stones or foot rungs (made of 3/8" rebar) must be set into the walls directly below where the accessway is to be located (refer to Figure 14-7). The rungs can be easily fashioned by the village blacksmith, to the dimensions shown in the figure. Rungs or steppingstones should be spaced 30 cm apart vertically. Refer to Chapter 19 for details on cement, mortar, and masonry.

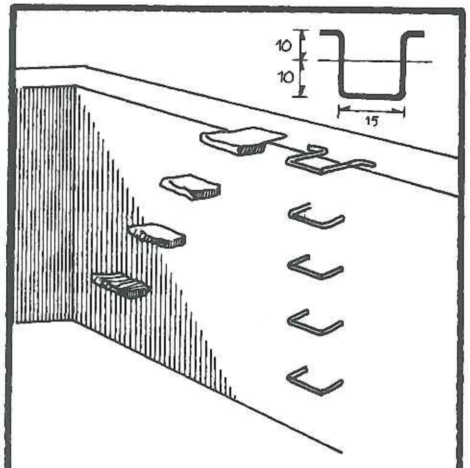


FIGURE 14-7
STEPPINGSTONES & RUNGS

Roofing: Since direct exposure to sunlight can cause dehydration of plaster and concrete, it is recommended that the roof of the tank be completed before the walls are plastered or the floor has been poured. Having the tank securely locked will also deter children from entering it when the plaster and concrete are still setting. Chapter 19.14 presents concrete roof slabs, and Technical Appendix F discusses other types of roofing.

Plastering: It is recommended that plastering be done before putting down the floor. Specifics of plastering are given in Chapter 19.12.

Floor: The floor of the tank may be either of masonry (ie- mortared brick or stone) or concrete (either reinforced or non-reinforced). A bed of gravel or crushed stone must be put down, roughly pitched so the floor will slope downwards to the washout. Technical details of creating a water-proof tank floor are presented in Chapter 19.13. As soon as the final concrete or plaster has set, the tank should be filled to a depth of about 30 cm to help the curing process (a deep depth of water would exert too much pressure on the floor which the cement would not be strong enough to support). After two weeks the tank can be filled completely and checked for any visible leakage.

Finished grading: The ground around the reservoir should be mounded so that rain run-off will not head towards the tank. The surrounding land should be stabilizied against erosion. If there is generally heavy rain run-off, then suitable drainage channels should be made. The drainage channel for the overflow should also be carefully constructed, and perferably should carry the water to where it can be utilized (such as for an animal water-hole, or for irrigation of a nearby garden).

Maintenance should include a yearly draining and cleaning of the tank. with plastering and other repair work as necessary.

14.8 DESIGN EXAMPLE

A reservoir tank of 16,000-liter capacity is to be constructed o rubble-stone masonry, with CGS roofing and a non-reinforced concrete This section will present the design calculations and estimates for materials and labor for roofing, masonry, excavation, and floor (exclu are GI service pipes). For specific labor and estimate analysis rates, see REFERENCE TABLE VII at the end of this handbook.

Preliminary calculations:

Water depth selected to be 65 cm (=0.65m)

Required water surface area = capacity/depth
$= 16.0m^3/0.65m$
$= 24.62m^2$

Internal dimensions for a square tank = $\sqrt{24.62}$ = 4.96m
= 5.0m x 5.0m

The area to be covered by the roof includes the water surface, the top of the tank walls (each 30 cm wide) and a 10cm overhang:

Roofing area = 5.0 + 0.3 + 0.3 + 0.1 + 0.1 = 5.8m x 5.8m

Adjusting these dimensions to accomodate the effective dimensions of a CGS sheet (3.0 x 0.7m):

8 sheets wide = 5.6m

2 sheets long = 6.0m

So the final internal dimensions of the tank are 4.8m x 5.2m (subtracting overhang and walls), which gives a final capacity of $16.22m^3$ (16,220 liters) which is acceptable.

Wall dimensions: Having selected the water depth, it is possible to select the dimensions of the masonry walls, using Figure 14-2. The resulting cross-section of wall and foundation trench are shown in Figure 14-8.

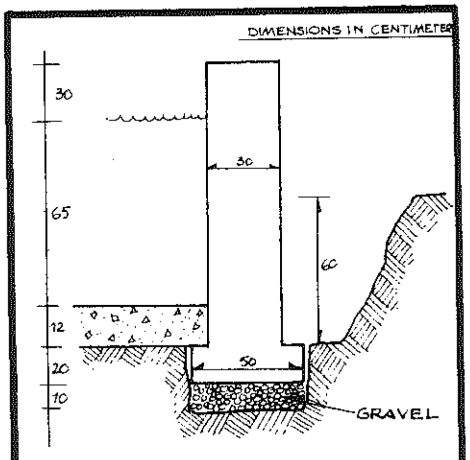


FIGURE 14-8
WALL OF DESIGN EXAMPLE

Cross-sect'l areas of:

masonry = $0.42m^2$

gravel = $0.05m^2$

The final external dimensions of the tank (inclusive of footings) is 5.6m x 6.0m, and excavation dimensions are therefore 7.6m x 8.0m (which allows an extra meter outside of the walls for the masons to work), by 60cm deep.

Excavations:

volume of main excavation = 7.6 x 8.0 x 0.6 = $36.48m^3$

volume of foundation trench excavations:

(5.6 + 5.6 + 6.0 + 6.0) x 0.3 x 0.5 = $3.48m^3$

TOTAL VOLUME OF EXCAVATION = $40.0m^3$

TOTAL LABOR OF EXCAVATION = 22.0 man-days (unskilled)

Masonry:

total length of masonry walls = 23.2m

cross-sect'l area of walls = $0.42m^2$

volume of masonry = 23.2 x 0.42 = $9.74m^3$

volume of crushed stone (foundation trenches) = 23.2 x 0.05

therefore TOTAL VOLUME OF CRUSHED STONE = $1.16m^3$

Rubble-stone masonry is 65% stone and 35% mortar, and mortar is 100% sand and 25% cement (for 1:4 cement:sand mortar):

Total volume of stone = 0.65 x 9.74 = $6.33m^3$

Total volume of mortar = 0.35 x 9.74 = $3.41m^3$

Total volume of sand = $3.41m^3$

TOTAL VOLUME OF CEMENT = 0.25 x 3.41 = $0.85m^3$

TOTAL MASON LABOR = 9.74 x 1.4 = 13.64 man-days (skilled)

TOTAL LABOR = 9.74 x 3.2 = 31.17 man-days (unskilled)

Floor slab:

(The floor slab is non-reinforced concrete, 12cm thick of 1:1½:3 cement:sand:gravel mix)

volume of crushed stone foundation = 4.8 x 5.3 x 0.1 = $3.0m^3$

volume of concrete = 4.8 x 5.2 x 0.12 = $3.00m^3$

volume of crushed stone in concrete = 1.0 x 3.0 = $3.0m^3$

TOTAL VOLUME OF CRUSHED STONE = 3.0 + 3.0 = $6.0m^3$

Total volume of sand = 0.5 x 3.0 = $1.5m^3$

TOTAL VOLUME OF CEMENT = 0.33 x 3.0 = $1.0m^3$

TOTAL MASON LABOR = 1.1 x 3.0 = 3.3 man-days (skilled)

TOTAL LABOR = 4.0 x 3.0 = 12.0 man-days (unskilled)

Roofing:

(The roof will require 16 CGS sheets, supported at mid-span by beams (therefore 3 beams required), each beam supported at mid-span by a 1" GI pipe column).

Interior span of tank = 4.8m

Span of each beam = 2.4m

Dimensions of each beam = 5 x 10 x 540 cm

TOTAL VOLUME OF WOOD = 3 x (0.1 x 0.05 x 5.4) = $0.081m^3$

TOTAL CARPENTER LABOR = 0.081 x 18 = 1.46 man-days (skilled)

TOTAL LABOR = 0.081 x 18 = 1.46 man-days (unskilled)

3 pieces 1" GI pipe @ 0.95m

6 pieces 1" GI threaded flanges

20 pieces 3/8" x 5" bolts w/washer & nut

½-kg 2" nails

6 pieces 3/8" rebar @ 0.60m (for anchoring beams to walls)

Plastering:

(Plastering according to specifications of Chapter 19.12, 3 coats @ 1 cm thick)

Plaster area = $(4.8 + 4.8 + 5.2 + 5.2) \times 0.7 = 14.0m^2$ per coat
Spatterdash (1:4 plaster): sand = $14.0 \times 0.1 = 0.14m^3$
cement = $14.0 \times 0.0025 = 0.035m^3$
Second coat (1:3 plaster): sand = $0.14m^3$
cement = $14.0 \times 0.003 = 0.042m^3$
Final coat (1:2 plaster): sand = $0.14m^3$
cement = $14.0 \times 0.005 = 0.07m^3$

Total volume of sand = $0.42m^3$
TOTAL VOLUME OF CEMENT = $0.15m^3$
Total plastered area (ie- 3 coats) = $3 \times 14.0 = 42.0m^2$
TOTAL MASON LABOR = $42.0 \times 0.14 = 5.9$ man-days (skilled)
TOTAL LABOR = $42.0 \times 0.22 = 9.2$ man-days (unskilled)

Total Materials & Labor:

Total volume of crushed stone = $7.16m^3$
Labor of crushing stone = $7.16 \times 1.4 = 10.0$ man-days (unskilled)
TOTAL VOLUME OF CEMENT = $2.00m^3$ = 2000 liters = 63 bags
TOTAL UNSKILLED LABOR = 86 man-days
TOTAL SKILLED LABOR = 24.3 man-days

(List of required fittings....)
(List of required tools....)

Figure 14-9 are drawings of the final tank design.

NOTES:
1. GI PIPE SIZES & ARRANGEMENT NOT DETAILED HERE.
2. REFER TO FIGURE 14-8 FOR DETAILS OF WALL DIMENSIONS.
3. EACH ROOF BEAM HAS A 1"GI PIPE COLUMN @ MID-SPAN.
4. CGS ANCHOR BOLTS SPACED AT 65 CM.

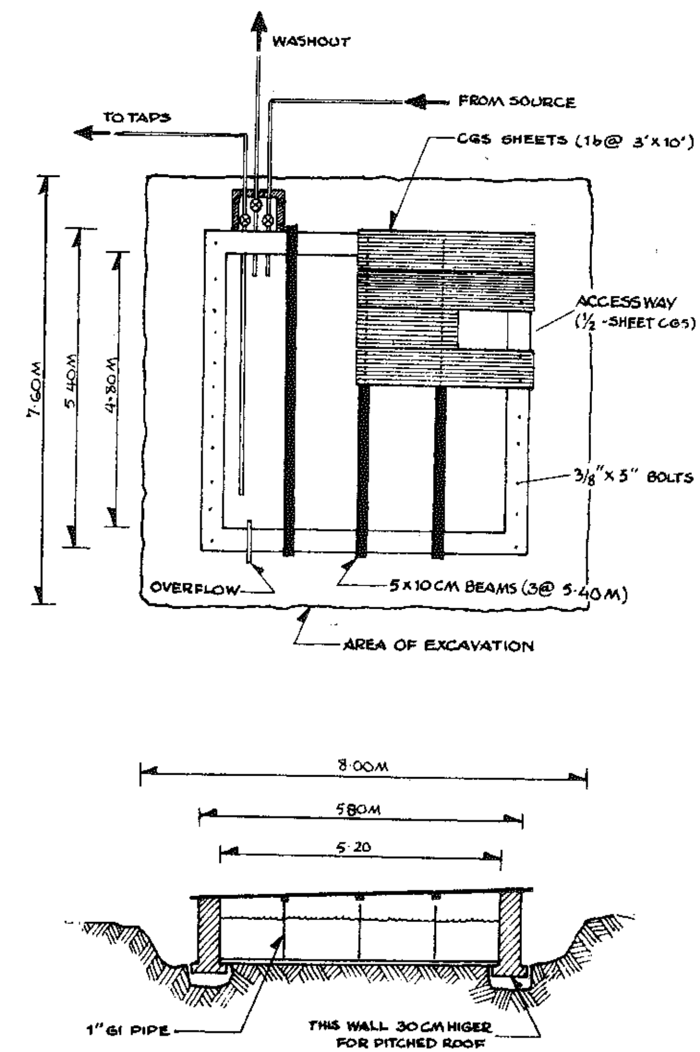


FIGURE 14-9
RESERVOIR FOR DESIGN EXAMPLE

15. PUBLIC TAPSTANDS

15.1 INTRODUCTION

The tapstands are the most frequently-used component of the entire system. No other structure will face more abuse than these, and no other structure will have to fit in so closely with local social and cultural needs.

A tapstand is more than just a physical structure. It will become a new and important gathering point of the village, where women will be washing clothes and men bathing themselves. Not only the tapstand itself, but the immediate surrounding area must also be carefully selected and planned. Properly designed and built, the tapstand will be a clean, attractive, and inviting place. Poorly completed, and it will be a dirty, muddy, unhygienic eyesore.

In addition to being the point to collect water, the tapstand area must allow room for clothes-washing as well as bathing.

Apart from the water-rights of the source, no other part of the system is apt to become so embroiled in politics, arguments, or disputes. The number of desired tapstands, and their location, will be a frequent source of heated debate.

These are all considerations which must be kept in mind and equitably resolved if the system is to be a successful one.

15.2 TAPSTAND LOCATIONS

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

The number of taps required in a system will be greatly influenced by the geographical lay-out of the village. Isolated wards, no matter how small, will require their own. The school (if any or several) and health post should also each have one. It would not be unusual for the leading political person of the village to desire his own tap (this can be used to advantage: such a person can be a useful ally in organizing and motivating the work force, especially if he stands to gain a tapstand). It has also sometimes happened in Nepal that certain caste groups will want their own tap, for reasons of religious purity.

These are not all unreasonable requests. While it is not desirable that the total number of taps becomes excessively large, it is generally acceptable to add another one or two taps if the project (as a whole) will gain from this. A small investment of materials may go a long way towards goodwill, motivation, and success.

The location of the tapstands should be based upon a number of considerations: is it well-located to serve those families that will depend upon it, is there an adequate drainage point for the waste-water; is the area large enough to allow for several users at once (washing, clothes, bathing, collecting water, etc).

A site near, but not directly on, a main trail is good. A sunny, sheltered site will encourage bathing (even in the cold season). A small water-hole for animals may be dug nearby to collect waste-water (and prevent the animals from coming directly to the tapstand to get their water). Overflow from the water-hole can be channeled to a nearby garden or field.

15.3 FLOW

The standard tapstand flow is 0.225 LPS (13.5 liters/minute). Such a tap will adequately serve a population of 200-230 persons. Where a tapstand will be serving only just a few households then the flow can be cut down a bit, and conversely the flow may be increased for a more densely-populated area (a double - or triple-faucet tapstand may also be built, refer to Section 15.5).

The design flow is achieved by installing a ½" globe valve at the base of the tapstand, and adjusting it until the desired flow is delivered. This valve is then securely locked up, to prevent furhter tampering. The faucet at the discharge serves only as an on/off control valve.

15.4 RESIDUAL HEAD

The residual head at the tapstand is important: if too high, it will cause accelerated erosion of the interior of the control valve; and if too low, will result in low flows.

The following residual heads are recommended:

Absolute minimum:	7 meters
Low end of desired range:	10 "
Most desirable:	15 "
High end of desired range:	30 "
Absolute maximum:	56 "

These standards are somewhat liberal; Wagner & Lanoix recommend a range of 10-50 meters.*

The static pressure when the tap is closed must not exceed the pressure rating of the tapstand pipe, and tapline.

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* "Water Supply for Rural Areas & Small Communities" (WHO, 1959)

15.5 STRUCTURAL CONSIDERATIONS

A tapstand may be constructed of brick, stone, or wood, using mortar or dry-stone masonry. Regardless of what it is constructed with, it must be designed and built to survive heavy use and abuse, especially if located in a schoolyard.

A masonry tapstand of cement mortar should have a supporting column 50cm x 50cm around the GI pipe, and should be on a footing imbedded 30cm below ground level. Mortar should be 1:4 and the exterior can be plastered if the villagers so desire. The faucet should protrude far enough so that the water vessels can be easily filled; it need not protrude, however, more than 30cm. Since the water vessels are typically carried by a headstrap, a low bench added to the tap (either of cement, mud mortar, or dry-stone masonry) will be helpful to facilitate lifting the vessel. A concrete or cement-mortared "apron" should provide enough room for several persons to work at once. A non-erodable drainage channel should carry the wastewater to a suitable drainage point.

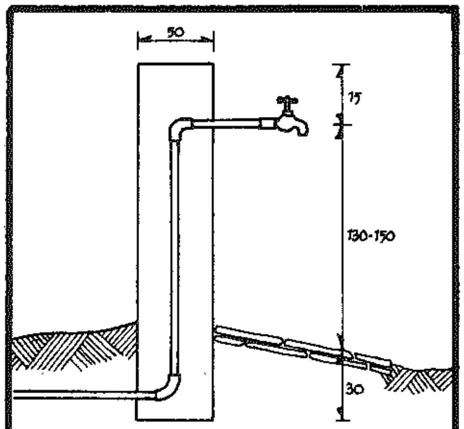


FIGURE 15-1
MASONRY TAPSTAND

Dry-stone tapstands can be used when the tapstand can be built into an embankment, or where there are skilled rock-cutting masons who can carefully fit together a solid tapstand structure; these types require minimal (if any) cement mortar. Both of these types are shown in Figure 15-2.

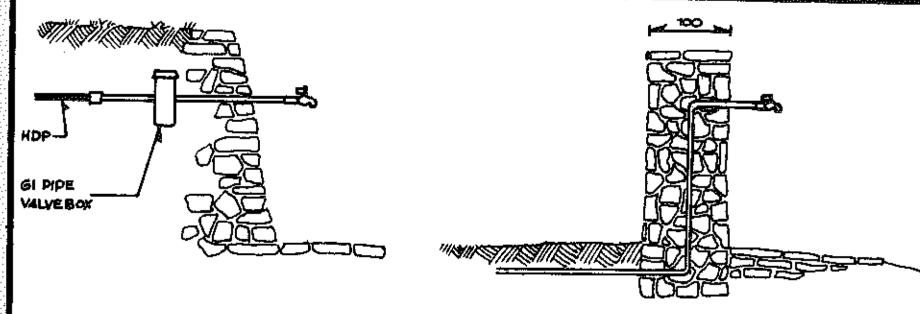


FIGURE 15-2
DRY-STONE TAPSTANDS

Wooden tapstands do not have such a long lifetime as masonry tapstands, due to the moist environment which promotes rotting. However, occasionally some projects will temporarily require such tapstands until cement can be obtained to build the proper one. In such cases, a wooden post, 15cm square minimally by 100cm longer than the height of the faucet, can be quickly installed. The village blacksmith is able to make a few iron staple-nails with which the GI pipe can be firmly nailed to the post. This type of tapstand is shown in Figure 15-3.

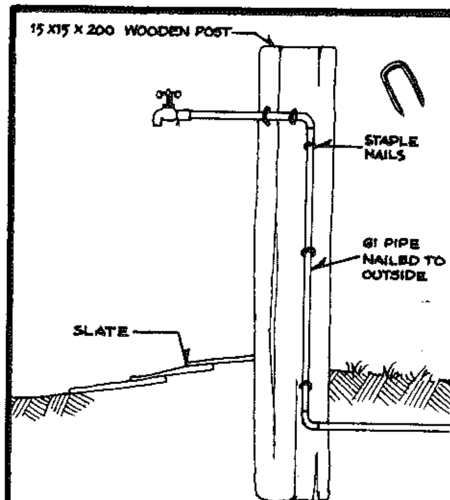


FIGURE 15-3
TEMPORARY WOODEN TAPSTAND

A more permanent wooden tapstand can be made with a post of the same size, but a channel cut in the back of it so that the GI pipe can be installed inside of it, as shown in Figure 15-4. A wooden cap over the top of the post will prevent rainwater from seeping into the wood, and the surrounding area around the post should be slated to minimize seepage into the ground. The post should be set into a bed of gravel, and backfilled with more gravel, so that water drains freely downwards and doesn't soak the post. The post itself should be thoroughly painted with a wood preservative or varnish, to inhibit rot.

GI pipe: Tapstands use ½" GI pipe, and require a faucet and a globe valve.* The height of the faucet should be 120-150cm above the apron (a schoolyard tapstand should have a faucet somewhat lower for smaller students). The control valve should be located in a securely-locking valvebox (see Chapter 16) that prevents tampering. Figure 15-5 shows typical GI pipe arrangements and dimensions for single - and multi-faucet taps:

*In Nepal, a ½" globe valve is known as a "corporation cock"

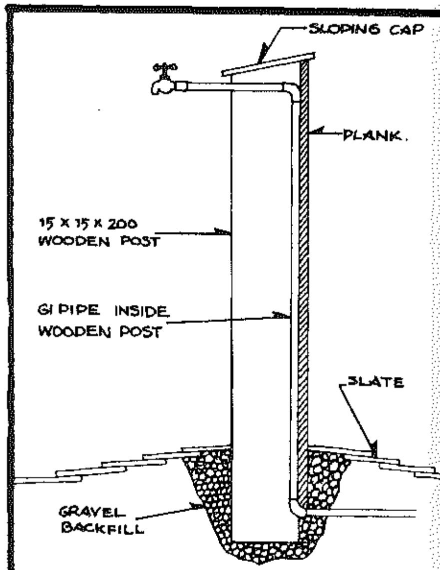
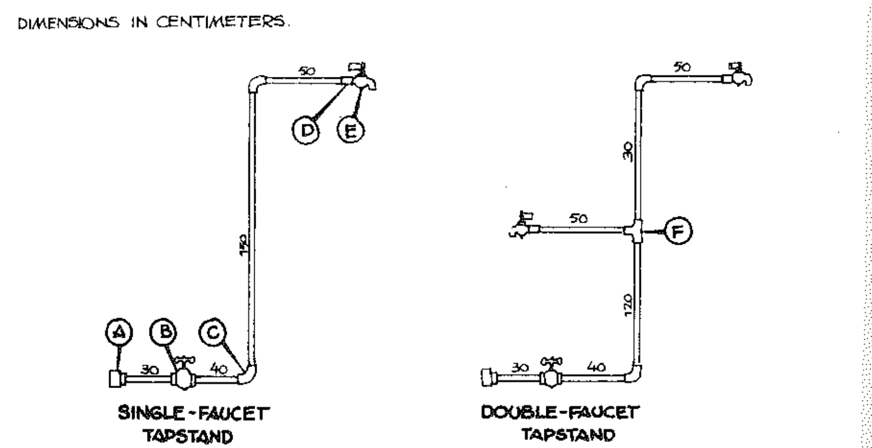


FIGURE 15-4
WOODEN TAPSTAND



(A)	½" x 20MM GI/HDP BRASS UNION	(D)	½" GI SOCKET
(B)	½" GLOBE VALVE	(E)	½" BRASS FAUCET
(C)	½" GI ELBOW	(F)	½" GI TEE

FIGURE 15-5
GI PIPE ARRANGEMENTS FOR TAPSTANDS

Multi-faucet tapstands: Where the population density of a village is quite high, it is possible to economize on the number of tapstands by constructing ones with two or three faucets. In such cases, one faucet should be set about 30cm lower, and the control valve adjusted so that not less than 0.20 LPS flows from each tap when all taps are open. Multi-faucet tapstands are not required anywhere except where it is expected that there will be more than 200 persons using the tap, or where an unusual water demand schedule will result in a great number of persons trying to use the tap at once (such as bathing at a bazaar tapstand).

Drainage: The waste-water from tapstands must not be allowed to collect in muddy puddles, where it can stagnate and become a breeding

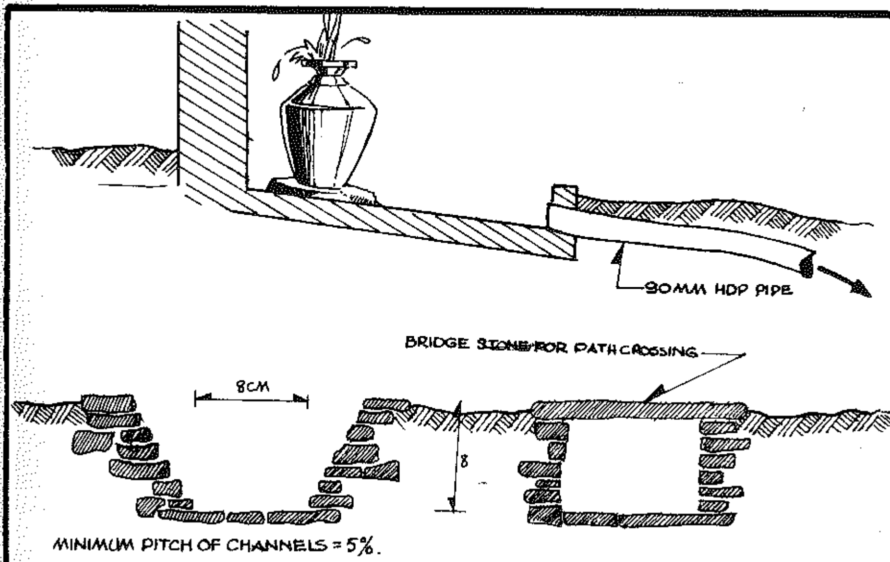


FIGURE 15-6
DRAINAGE CHANNELS

place for mosquitoes and disease. Waste-water should be carried away by a non-erodible channel (or 90mm HDP pipe) to a suitable drainage point (such as a water-hole for animals, or a nearby garden or field). Such channels do not have to be made of mortared masonry but should be made of brick or stone that is carefully fitted. Drain channels should be bridged at path crossings. The drainage point must definitely be at a lower elevation, and the minimum slope of the drain channel should be 5%. Refer to Figure 15-6 for drawings of some drainage channels.

Finishing: The ground around the tapstand should be finished in such a way that it is stable, quick-draining, and quick-drying. Animals should be precluded from walking over, or through, the tapstand area, and therefore some fencing may be needed.

The tapstand may be plastered or left natural, depending upon the quality of the masonry and village desires. In fact, much of the tapstand construction should be according to the wishes of the villagers, with the overseer providing guidence along those lines discussed in this chapter.

Some villagers may be content with just an open pipeline. Such arrangements invariably create problems that for outweigh the expense and labour of building a sound tapstand.

Various drawings of tapstands are shown in Figure 15-7.

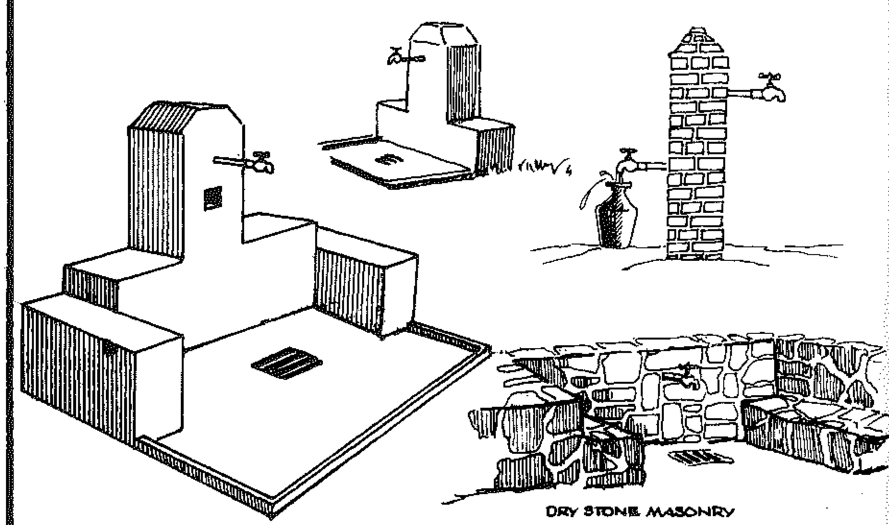


FIGURE 15-7
VARIOUS DESIGNS OF TAPSTANDS

16. VALVEBOXES

16.1 INTRODUCTION

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

Valveboxes can be attached to the structures (as is common with tanks) or located independently along the pipeline (such as at strategic branchpoints or near tapstands). They can be constructed of masonry, GI pipe, HDP pipe, or reinforced concrete (RCC), depending upon the materials available, size and number of valves, how often they will be operated, etc.

16.2 DESIGN CHARACTERISTICS

Regardless of what they are constructed with, all valveboxes must be built with the following characteristics:

Secure cover: The valve must be protected by a strong and secure cover which cannot be undone or opened by ordinary persons. Covers can be bolted down, nailed down, buried, or even welded down (as is the case with a HDP pipe valvebox). Valvebox covers can be RCC slabs (see Chapter 19.15), GCS sheeting (refer to Chapter 20.6) or wooden planks.

Free-draining: No valvebox should have a solid floor, so that any leakage or ground seepage can quickly drain away. A bed of gravel or crushed stone is recommended.

Adequately large enough to allow the valves to be removed easily and replaced, without having to tear down the valvebox. If constructed of masonry, it must be large enough so that wrenches and pliers can swing freely. If constructed of GI or HDP pipe, they must be easily removable.

16.3 MASONRY VALVEBOXES

Masonry valveboxes are of either stone or brick. The lower portion of the box may be of dry-stone masonry, but cement-mortar masonry should be used for the top 40cm. The box should protrude about 10cm above ground level. The interior dimensions of the box must allow the valves(s) to be unscrewed from the pipe; in a box with two or more valves, staggering them will leave some more room. The pipeline should not be

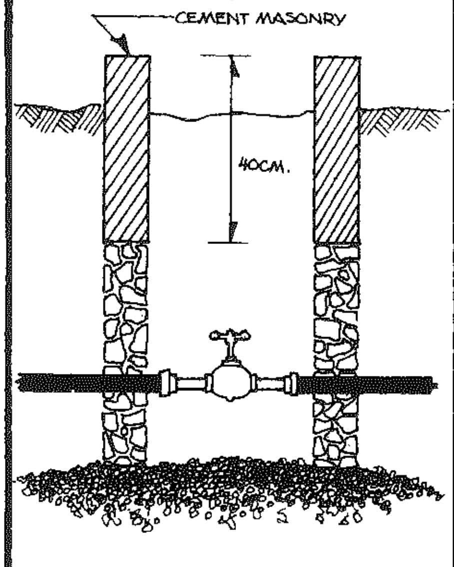


FIGURE 16-1
MASONRY VALVEBOX

cemented into the walls of the box. and the box should be built on a bed of gravel or crushed stone several centimeters deep to allow quick drainage of leakage or moisture. Refer to Figure 16-1.

16.4 RCC VALVEBOXES

Valveboxes made of reinforced concrete are not generally worth the effort for just a single box. However, when several boxes of the same dimensions are to be built, then a wooden form can be made and RCC valveboxes easily produced. The RCC valvebox can rest upon a lower portion of dry-stone masonry. The reinforcement should be of 3/8" rebar, the concrete should be 1:2:4 mix (with small-sized gravel), and the walls should be about 5cm thick. Bolts should be imbedded in the top for bolting down the cover (refer to Chapter 20.4). The dimensions of the interior are the same as for masonry valveboxes. Refer to Chapter 19 for details of RCC cementwork.

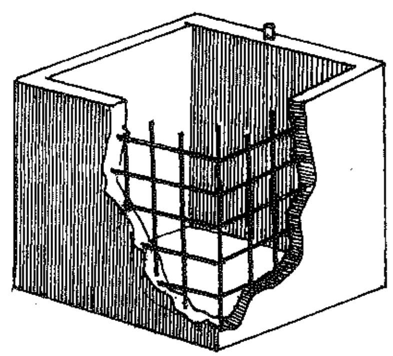


FIGURE 16-2
RCC VALVEBOX

16.5 GI PIPE VALVEBOXES

For a single valve, it is possible to use a length of GI pipe as the valvebox, as shown in Figure 16-3. The size of the GI pipe depends upon the size of the control valve; for valves used in Nepal, the following sizes can usually be used:

VALVE SIZE	GI PIPE SIZE
½"	2"
1"	3"
1½"	3"
2"	4"

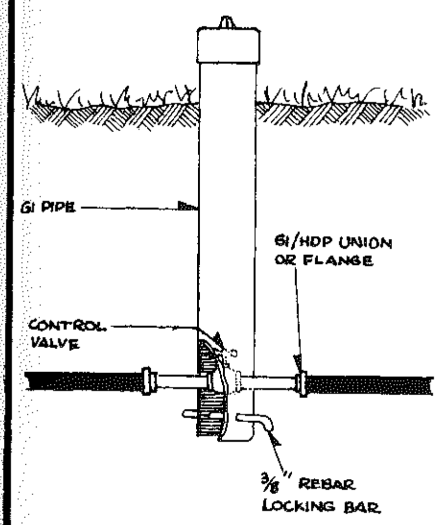


FIGURE 16-3
GI PIPE VALVEBOX

It is recommended, however, that the overseer personally check that the control valves will actually fit inside the GI pipe.

The base of the GI pipe is slotted to allow it to slip over the pipeline, and locked into place with a 3/8" bolt or hooked rebar (these are passed through two ½" holes drilled in the bottom of the pipe). An endcap screwed down with a pipewrench will be a secure cover. For operating the valve, a "key" of ½" or 1" GI pipe is used: the ends of this key are slotted so it can be slipped onto the valve handle and turned. Dry-stone masonry walls and gravel backfill are recommended, and painting the GI pipe will help to retard corrosion.

16.6 HDP PIPE VALVEBOXES

These are best suited for the ½" control valve of a tapstand, since these are valves that are not frequently adjusted. An HDP valvebox is of 90mm HDP pipe, secured to the pipeline in the same slotted manner as described above, but not extending to the ground surface. They are closed by welding on a 90mm HDP endcap, with enough clearance so that the cap may be cut off without damaging the valve inside. The same cap can be re-welded back on once the maintenance work is completed. Refer to Figure 16-4.

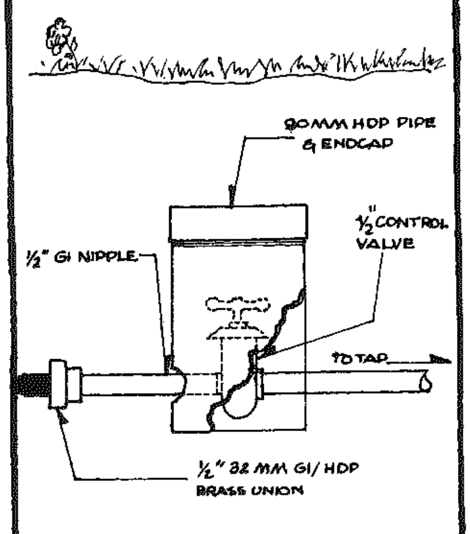


FIGURE 16-4
HDP VALVEBOX

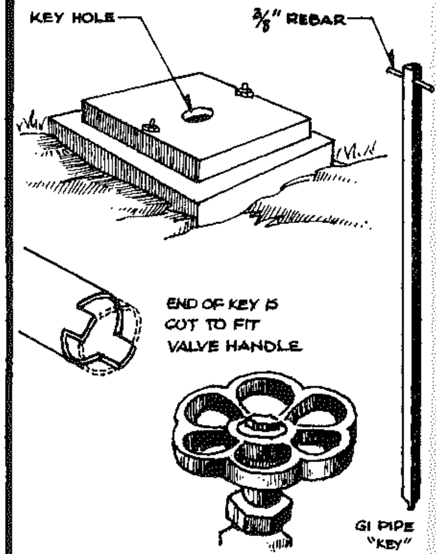


FIGURE 16-5
VALVEBOX WITH KEY

16.7 FREQUENTLY-ADJUSTED VALVES

For valves which must be operated fairly frequently, it may be better to leave a hole in the valvebox cover that is located directly over the valve handle. Then the valve can be operated using the "key" (described in Section 16.5) instead of repeatedly removing the cover. The hole in the cover should be just a few centimeters larger than the key pipe, and the cover should be high enough so that hands cannot reach down through the hole and operate the valve manually. Refer to Figure 16-5. A disadvantage of this type of valve box is that the hole is open, which leaves the valve liable to being tampered with by using a hand-made key.

16.8 ATTACHED VALVEBOXES

These are valveboxes attached to, or built into, some structure, such as intake tanks, break-pressure tanks, or reservoirs. Such boxes will usually consist of three masonry walls (of which the wall of the tank may be one) with a dry-stone masonry wall as the fourth. This dry-stone wall can be dismantled and the pipeline below it dug up without destroying any part of the valvebox.

Various drawing of some valveboxes are presented in Figure 16-6:

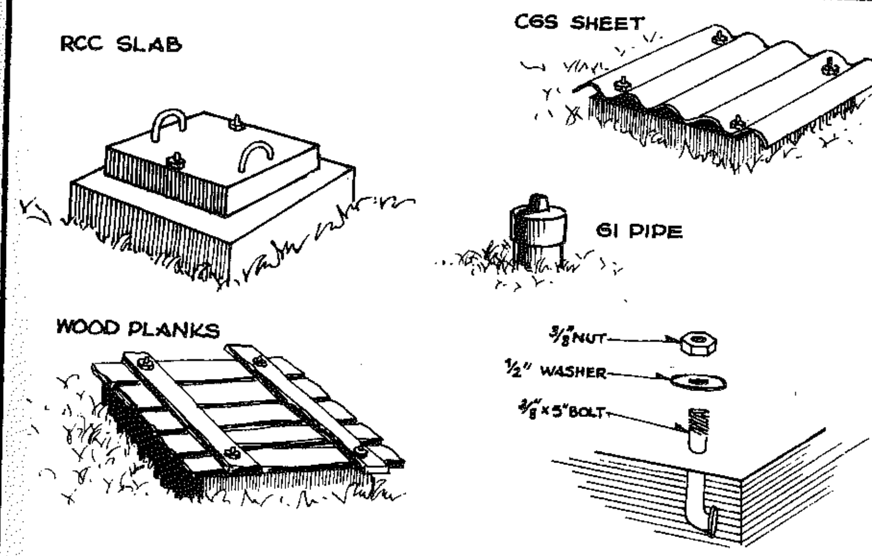


FIGURE 16-6
VARIOUS VALVEBOX COVERS

17. WATER QUALITY

17.1 INTRODUCTION

At the current time, there is no practical water treatment system which can be broadly used in Nepal. Thus, emphasis must lie in locating the cleanest possible source of drinking water, then properly securing it against further contamination.

Physical contaminants, such as suspended matter, can be removed or greatly reduced by allowing sedimentation to occur, as discussed in Chapter 12.

There are only two additional steps that can be practically employed towards improving water quality: slow-sand filtration, and aeration. This chapter will not attempt to present the technical details for these: the reader can find an abundance of such information among the sources listed in the Reference section of this handbook. Instead, general description and discussion will be presented so that the reader can gain a basic understanding of these procedures.

17.2 SLOW-SAND FILTRATION

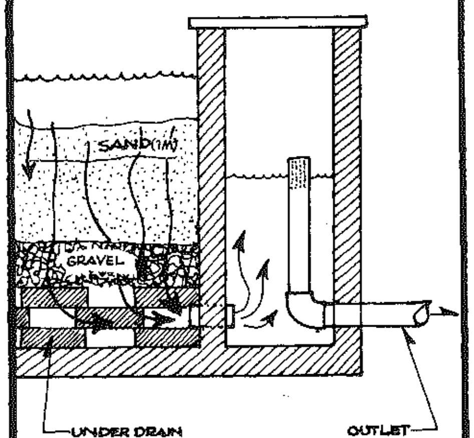


FIGURE 17-1
SLOW-SAND FILTER

A slow-sand filter is a large tank that has an under-drain system which is covered by a base layer of gravel and then a bed of filtering sand. The filter works by mechanically straining the water as it flows through the sand, and also by biologically attacking the organic impurities (the filter bed develops a "slime" of bacteria, which feed upon the organic impurities carried in the flow).

These filters are relatively simple to build and do not require highly-trained personnel for maintenance. However, a slow-sand filter has several serious drawbacks: at best efficiency, it can only filter about 0.002-0.003 LPS (7-11 liters/hour) per square meter of filter surface area. Thus, a large areas is required for providing even a minimal flow for the system. Additionally, although simple to maintain, they do require regular, reliable attention or else they can become sources of bacterial pollution rather than removers.

The decision to install such a filter involves much serious consideration and consultation with the villagers, the overseer, and the LDD engineers. Technical design of such a filter is best left to professional people.

17.3 AERATION

Aeration is the process of thoroughly mixing the water with air. Oxygen-enriched water loses its acidity (which is due to the presence of dissolved carbon dioxide) and reduces undesirable tastes and colors due to the presence of iron or other dissolved gases.

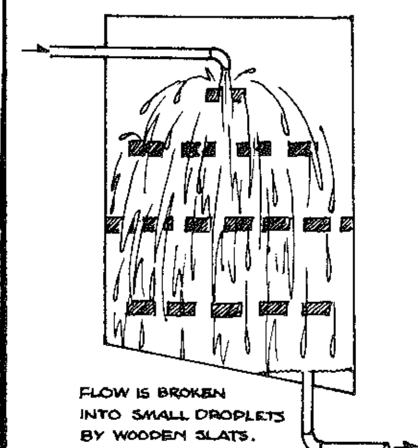


FIGURE 17-2
AERATION TOWER

The easiest method of aeration is to build a tower, as shown in Figure 17-2, which has several tiers of plastic screens or wooden slats. These mechanically break the water flow into small droplets, which, because of their increased surface area, can absorb oxygen quicker. Such a tower can be built as part of a sedimentation tank or collection tank, or even the reservoir.

17.4 FURTHER REFERENCE

For quick, basic information on slow-sand filters and aeration techniques, refer to "Water Supply for Rural Areas and Small Communities", Lanoix & Wagner (WHO, 1959), pages 175-180.

18. HYDRAULIC RAMS

18.1 INTRODUCTION

Hydraulic rams (hydrams) are coming into greater and greater use Nepal, allowing many villages that earlier could not use a gravity-flow water system (because the source was too low) to now have a drinking water system which is still economical to construct. Although the hydram is a pump, it requires no fuel or electricity. Instead, it operates by using the gravitational energy contained in a large amount of water falling a short distance to pump a small amount of water up a high distance.

This means that the hydram can be used to pump water from a low source up to a reservoir tank which is built higher than the village. From there, the water is distributed via a normal, gravity-flow pipeline.

This chapter will introduce the basic principles of a hydram, and present the technical knowledge necessary for a surveyor to conduct a field survey of a potential hydram project and determine if a hydram is feasible. Although the installation of a hydram requires special knowledge, there is no reason why a surveyor cannot properly identify a feasible hydram project, or a designer properly design such a system.

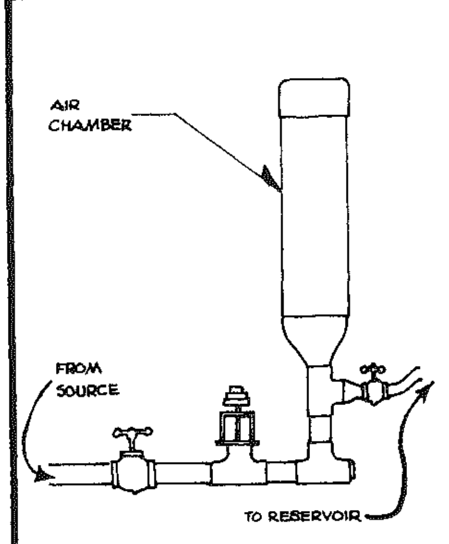


FIGURE 18-1
HYDRAULIC RAM

An excellent reference book is "Use of Hydraulic Rams in Nepal", by Mitchell Silver, printed by UNICEF/Nepal, 1977. Further information can be found in the sources listed in the Reference section of this handbook.

18.2 DESCRIPTION

Hydrams are available as commercially manufactured kits, or can be easily fabricated using GI pipe fittings and the services of a machine shop. Either type will be essentially similar to the one shown in Figure 18-1.

A large amount of water flowing from the source down the drivepipe compresses the air in the chamber, which then expands and drives a small amount of water up the delivery pipe. The quantity and height that the hydram can push water up to depends upon the quantity

and height of the water "falling" in the drivepipe.

A typical installation of a hydram system is shown in Figure 18-2:

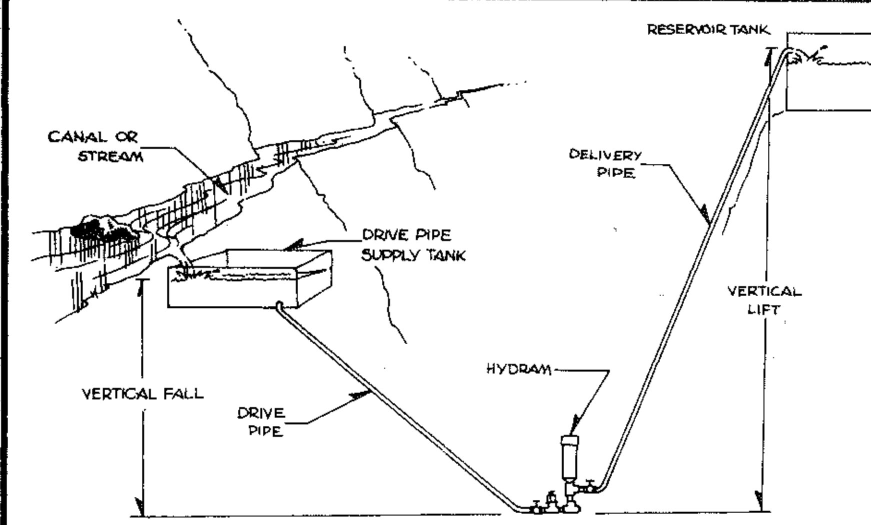


FIGURE 18-2
HYDRAM INSTALLATION

18.3 OUTPUT CALCULATION

A hydram, under optimum conditions, can rarely pump more than 25% of the source flow to a higher elevation. Thus, usually only very large springs, or streams, will be able to provide enough flow to serve a village. The higher the water must be pumped, the smaller the flow will be.

To calculate the approximate delivery flow of a hydram:

$$Qp = \frac{2 \times Hd \times Qd}{3 \times Hp}$$

Where:

Qp = delivery flow (LPS)
Hd = "falling" head (meter)
Hp = "lifting" head (not to exceed 100 meters)
Qd = "falling" flow (LPS)

These variables are indicated in Figure 18-2.

18.4 TECHNICAL CONSIDERATIONS

When studying a village for a potential hydram project, there are some technical details which must be kept in mind:

- If the source is a stream, it will be necessary to build a storage tank for the hydram (to ensure a regular, constant flow into the drive pipe)·

- Suspended particles and sediments will increase wear and tear on the pump, thus a sedimentation tank may have to be built;

- The drive pipe must be of GI pipe, and be as straight as possible, securely anchored or imbedded;

Thus, when studying a potential hydram project site, the surveyor must carefully select sites and terrain which will allow for the installation of tanks, drivepipe, etc, and must obtain accurate elevations and ground distances.

18.5 SPECIAL ARRANGEMENTS

It is possible to use several hydrams connected to a single delivery pipe; or to use the waste-water from an upper hydram to operate a lower hydram; or to incorporate a hydram into a break-pressure tank. These possibilities are illustrated in Figure 18-3.

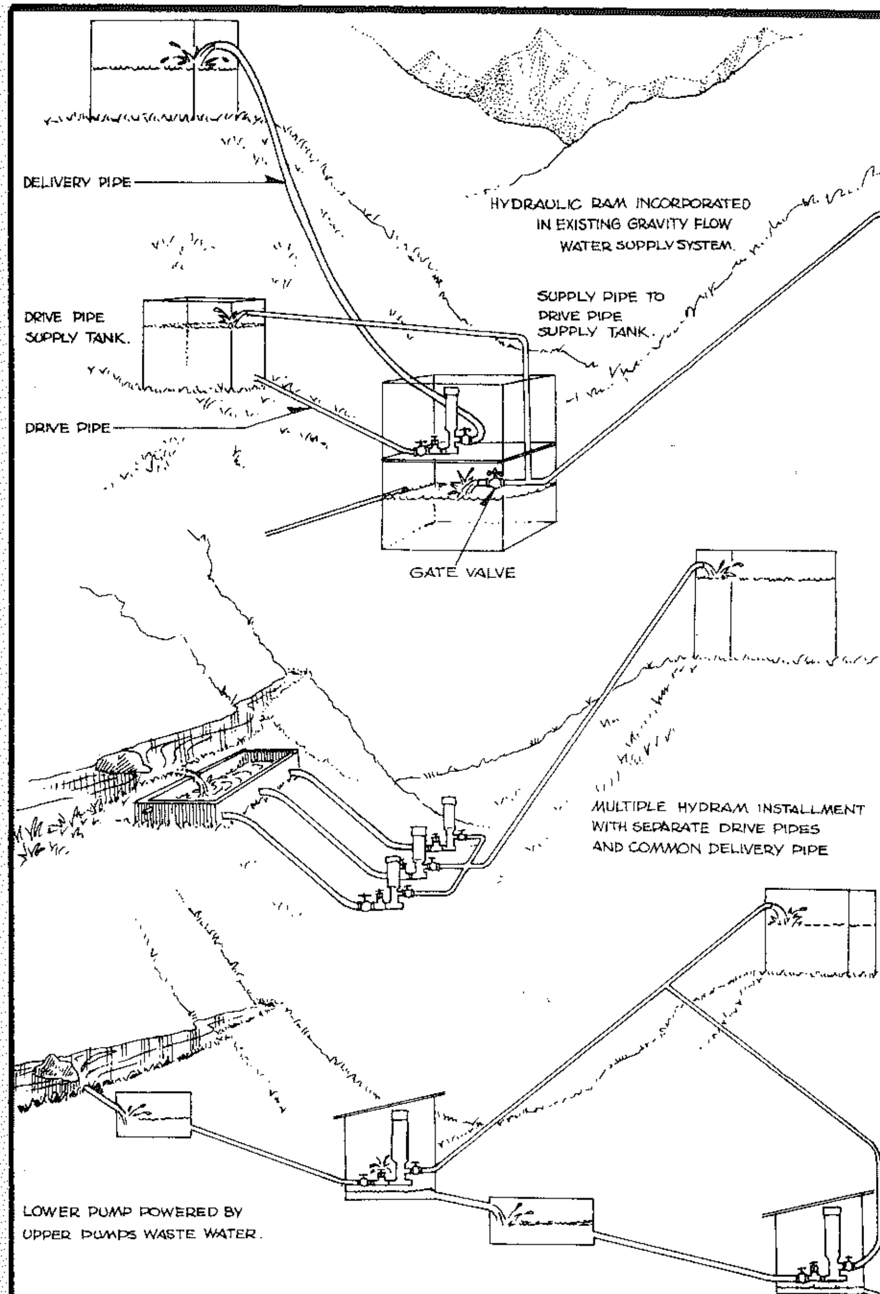


FIGURE 18-3
VARIOUS HYDRAM INSTALLATIONS

19. CEMENT, CONCRETE, & MASONRY

19.1 INTRODUCTION

Just about all structures constructed in water supply projects require the use of cement: mortar for masonry, plaster for waterproofness, and concrete for floor slabs. Proper knowledge of how to select the best materials, how to organize cement-mixing procedures, and how to make efficient and economical use of cement is all essential to the overseer.

This chapter is intended to be a fairly complete field reference. It will describe the various materials required for cementwork, their properties, and important considerations. It will discuss masonry of brick and stone, and concrete slabs for floors and roofs. It will present organizational procedures, helpful construction tips, and mention some common problems.

Some of the more important information is included in REFERENCE TABLE VIII at the end of this handbook.

19.2 DEFINITIONS & TERMS

The common cementwork vocabulary used in this chapter is listed here, with a brief explanation:

cement: serves as an adhesive, gluing together sand and stone. Typically, normal Portland cement is used: a gray powder, similar to flour.

mortar: a mixture of cement and sand in various proportions, depending upon desired strength. Used to cement together bricks or stones in masonry, and used to plaster walls for waterproofness.

concrete: a mixture of cement, sand, and aggregates (such as gravel or crushed stone) in various proportions. Can be poured to form slabs.

RCC: reinforced concrete. Concrete with reinforcing steel rods or bars imbedded in it for additional strength and support. Wire screening may also be used.

RF bricks: reinforced brick masonry, using reinforcement described above.

rebar: reinforcing steel bars or rods, used in RCC or RF brick.

aggregate: small pieces of stone mixed with cement and sand to form concrete. Coarse aggregates may be gravel, crushed stone, or crushed brick. Fine aggregate is sand.

gravel: usually found along rivers and streams: small pebbles and stones, worn fairly smooth and rounded by the action of water.

crushed stone: large pieces of rock or stone broken down to aggregate size, by manual labor using sledge hammers.

crushed brick: pieces of broken-up brick.

19.3 CEMENT

Cement is a mixture of chalk or limestone, and clay, which is fired and then ground into a fine powder. Additional materials may be added to impart certain properties to the cement (such as to make it quick-setting, low-heat, rapid-hardening, etc). Ordinary cement is a gray powder, commonly known as "Portland cement". This is the type commonly provided for water supply projects in Nepal.

Properties of cement: Portland cement is used for ordinary construction projects. Cement mortar or concrete has high compressive (crushing) strength, but relatively low tensile (stretching) strength. When water is added to a mortar or concrete mixture, it forms a fluid mass which is easily worked and placed into position. Within an hour (depending upon temperature and mix) the cement begins to set, losing its plasticity. Within 4 hours it has finished setting and can no longer be worked. From the time that setting begins, the cement is undergoing a chemical hardening process which will continue for at least a year, although it most-rapidly hardens during the first few days.

STRENGTH OF PORTLAND CEMENT CONCRETE
(Per cent of ultimate strength at various ages)

3 days	approx.	20%
7 days	"	45%
28 days	"	60%
3 months	"	85%
6 months	"	95%
1 year	"	100%

Hydration: When water is added to a dry cement mixture (for either mortar or concrete), it begins a chemical reaction with the cement known as "hydration". This reaction causes the cement to set and harden, giving off heat in the process. The rate of hydration is accelerated by heat and humidity, therefore cement will set and harden faster at warmer temperatures, and vice versa for colder temperatures (freezing of cement completely kills the hydration reaction, which will not continue even if the cement is thawed out. Refer to Section 19.18). The hydration reaction requires moisture, but the heat generated by hydration tends to cause evaporation of the moisture in the mix. Thus it is necessary to prevent the rapid drying-out of the cement, especially during the first few days. Once hydration ceases, the cement will gain no further strength.

Setting: When water is added to a cement mix, there is a period of about 30-60 minutes in which the mix is plastic and easily worked into position. However, after that period, the mix begins to set, becoming stiffer and stiffer. Within a few hours, the setting should be complete. Once setting has begun, the mix should not be disturbed, which would weaken the mix. Onset of setting can be determined by pressing the blunt end of a stick or pencil into the mix: resistance to penetration will suddenly increase when setting begins.

Hardening: This is the process whereby the cement mix gains strength. Hardening begins as soon as setting begins, but continues for at least a year.

Both setting and hardening are influenced by temperature: heat accelerates the rates of both.

Curing: Curing is the process of keeping the cement mix properly wetted, to ensure that there is enough moisture for the hydration reaction to continue. It is especially important during the first few days after pouring a concrete mix, when the cement most rapidly gains its strength.

Packaging of cement: One liter of Portland cement weighs approximately 1.44 kgs. Cement is typically factory-packed in bags of 50 kgs each, so therefore each bag should ideally contain nearly 35 liters of cement. However, some cement is lost during shipping and portering. For practical purposes, the amount of cement per bag should be considered as follows:

burlap (jute) bags : 32 liters

paper bags : 34 liters

Storage of cement: Cement easily absorbs moisture from the air, and as a result loses strength during long periods of storage. Typical losses are as follows:

Period of storage	Loss of strength
3 months	20%
6 "	30%
12 "	40%
24 "	50%

When storing cement at the project site, it should be stacked in a closely-packed pile, not more than 10 bags high (to keep the bottom bags from bursting). Close-packing also reduces air-circulation between the bags, which is good. The pile of cement should be raised on a platform above the floor. The room or storage shed should have as little air circulation as possible, and if a long storage period is anticipated, the pile should be further covered by plastic or canvas tarpaulins. Paper bags of cement will resist aging much better than burlap bags, thus paper bags should be on the outside of the pile, and the burlap bags should be the first used in construction.

Aged cement will form lumps. All lumps should be screened out of the cement, and no lumps should be used which cannot be easily crumbled

by the fingers. If old cement (ie- field stored for more than 6 months) must be used, increase the amount of cement in the mix by ½-1 parts (depending upon how lumpy it is).

19.4 WATER

Water in the cement mix serves two purposes: first, to take part in the hydration reaction of the cement; and secondly, to make the mix fluid and plastic enough so that it can be easily worked and placed.

Quality: Water that is fit for drinking is usually fit for mixing cement. Water unsuited for drinking may still be used, if tested as follows:

Using water of known suitability (ie- drinking water), make 3 cakes of cement paste, each approximately 1-2 cm thick by 6 cm in diameter. At the same time, make 3 identical cakes using the unknown water. Comparing the two types, observe the setting time, the "scratchability" (using a fingernail) and strength after a few hours, 24 hours, and 48 hours. Only if both types of cakes are equally strong should the unknown water be used.

Quantity: Water is necessary for the hydration of the cement, but too much water added during mixing results in a weaker strength. The quantity of water generally needed to make the mix easily workable is much more than is needed for the hydration reaction. Therefore, no more water should be added than necessary to make the mix easily workable. The ideal quantities of water depend upon the amount of cement in the mix, and approximate guidelines are given along with the mix proportions for concrete, in Section 19.11.

Once the cement has finished setting, further addition of water does not weaken it. In curing concrete, this is a necessary action to prevent the surface of the slab from drying out too quickly.

19.5 SAND

Sand is used in both mortar and concrete (in the latter, it is sometimes referred to as "fine aggregate"). Proper sand is well-graded (ie- containing grains of many sizes mixed together). Sand of a uniform size, such as beach sand or very fine sand, is not suitable (but can be mixed into coarser sands).

Sources of sand: Sand found in land deposits is known as "pit sand". Such grains are generally irregular, sharp and angular. Sand carried by water, such as found along banks of rivers or lakes, is known as "river sand". Such grains are generally rounded and smooth, due to the action of water.

Both types of sand are suitable for cementwork, so long as they are well-graded and clean.

Quality: Sand containing clay, silt, salt, mica, or organic material is not good, since such contaminants can weaken the strength of the cement if they are present in large quantities. There are easy field tests which can be conducted to determine the quality of a sand source:

a) A moist handful of the sample sand is rubbed between the palms of the hands. Suitable sand will leave the hands only slightly dirty.

b) Decantation test: a drinking glass (or other clear glass container) is half-filled with the sample sand, and then filled 3/4-full with water. The glass is then shaken vigorously, and allowed to sit undisturbed for an hour or so. The clean sand will settle immediately, and the clay and silt will settle as a dark layer on top of the sand. The thickness of the clay/silt layer should not be more than one-seventeenth (6%) of the thickness of the sand.

Dirty sand can be washed by rinsing repeatedly with water.

Bulking of sand: Damp sand that contains up to 5-6% water will swell up and occupy a greater volume than if it were perfectly dry. This is known as "bulking". A moisture content of 5-6% can increase the volume by over 30%. Additional water content reduces the bulking, until saturated completely (saturated sand occupies nearly the same volume as it does when dry). Thus when using slightly damp sand, it is necessary to use an extra amount of sand in the mix if it is to be proportioned by volume. Very damp sand (such as freshly washed) is measured as if it were dry. If the mix is proportioned by weight, the bulking is of no consequence.

19.6 AGGREGATES

Aggregates is the general term for the material mixed with cement and water to form concrete. Sand is a fine aggregate, and larger material is a coarse aggregate.

Coarse aggregates may be gravel (generally river-worn, rounded rocks) or crushed rock and brick.

Stones of granite, quartzite, basalt, or having rough non-glossy surfaces are best. Hard limestones are good, soft sandstones are not. Limestones and sandstones are porous and therefore not good for water tank floor slabs, but can be used for roof slabs (same applies for crushed brick).

Aggregates must be clean and well-graded. Smaller rounder aggregates (such as river gravel) are better for waterproof floor slabs.

Sizes of aggregates: Aggregates should be well-graded so that air voids between pieces are minimal. Largest sizes should be:

For roof slabs: 3/8" (10mm)

For unreinforced or lightly reinforced slabs: 3/4"-1" (20-25mm)

Crushed brick: Pieces of broken-up brick may be used as aggregate in concrete, but due to their porous nature should not be used for floor slabs of water tanks. When using crushed brick aggregate, pieces should be thoroughly soaked in water prior to mixing, to prevent absorption of moisture from the mix (which will interfere with the hydration reaction).

19.7 REBAR REINFORCEMENT

Reinforcement of concrete is only needed for slabs which are large in area or will be put under great hydrostatic pressure (ie- deep water depth). An RCC slab can be thinner than a non-reinforced slab. The presence of the reinforcement helps to distribute the stresses and forces uniformly over the entire mass of concrete.

Reinforcing bar (rebar): Is available in many sizes, but for typical water supply projects only the following diameters are needed: ¼", 5/16", or 3/8" (6mm, 8mm, or 10mm).

Wire-mesh screen: (also known as "wire-mesh fabric") can also be used as reinforcement in slabs. The size of aggregate in the concrete mix should be smaller than the size of the mesh (using a piece of the screen to sift the aggregate is the best way of ensuring this).

Spacing of rebar: The spacing of the rebar must distribute the cross-sectional area of steel uniformly across the cross-sectional area of the slab. For a floor slab, the area of rebar must not be less than 0.225% of the total cross-sectional area of the slab, and for a RCC roof slab is must not be less than 0.30%. The following table can be used:

Type of slab	Thickness (cm)	Spacing of Rebar (cm) 6mm	8mm	10mm
floor	8	15	30	40
roof	8-9	12	21	33
roof	9-11	10	17	27
roof	11-13	8	14	22
roof	13-15	7	12	19
roof	15-17	6	11	17

Placing of rebar: The reinforcement is made as a grid, with the size of the squares according to the table above. The rebar rods can be tied together with thin wire or string. The rebar must have a minimum of 3 cm of concrete covering. For a roof slab, the rebar is set 3 cm from the bottom of the slab, and for a floor slab the rebar is set 3 cm from the top of the slab (Refer to Figure 19-1).

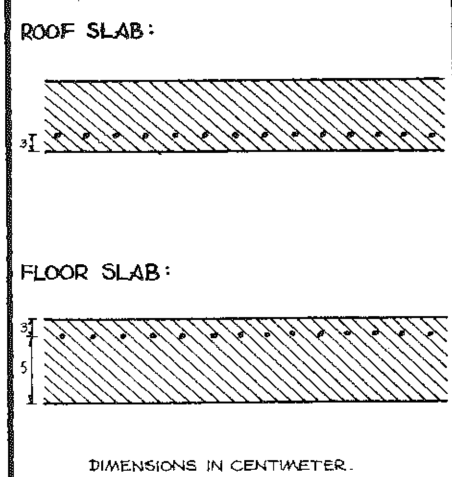


FIGURE 19-1
REBAR PLACEMENT IN SLABS

The rebar must be securely fastened so that it cannot be shifted around while the concrete is being placed (the rebar can be supported on pieces of non-porous rock, but NOT brick or wooden stakes).

Reinforced (RF) brick slabs: When a roof slab is to be of reinforced brick, different rebar spacing is required, depending upon the thickness of the slab, and the size of the bricks. Refer to Section 19.14 for technical details of this type of roof slab. A floor slab of brick requires no reinforcement (refer to Section 19.13).

19.8 CEMENT MIXING

For convenience, it is usually easiest to mix cement at the construction site, so it is necessary to ensure that there is an organized system for delivering cement, sand, aggregates, stone or brick, and water. It is particularly important when mixing and pouring concrete that it be done in a continuous operation, without long delays caused by lack of materials.

Mixing pad: Cement should never be mixed on the ground. A mixing pad of brick, slate, concrete, or even a CGS sheet should be made. It should be large enough to allow mixing of convenient-sized batches, without overflowing: 1.5 square meters is adequate. If possible, build a small lip around three sides of the pad so that materials may not get accidently washed off.

When a number of masons are working at once, or concrete is being mixed, it is usually better to have two or more pads.

Proportioning: Although the most accurate method of proportioning cement, sand, and aggregates is by weight, in a field site this is not so easy to arrange. The common method is to mix by volume, using a small bucket. Measuring by shovelfuls is not accurate. Mortar should be mixed in smaller batches than concrete, but no batch should be so large that it is not used in 30 minutes.

Dry-mixing: All ingredients are first thoroughly dry-mixed together, using shovels and trowels, until the mix is of a uniform color and consistancy.

Wet-mixing: Water is added slowly, a small quantity at a time. Each time water is added, the mix is thoroughly "turned over" a few times with shovels. Water is added until the mortar or concrete is at the desired consistancy. The wet-mix can be adjusted as follows:

Too wet: add sand (and aggregate)

Too dry: add water

Too stiff: add sand

Too sandy: add cement

Tools & Manpower: A cement-mixing team should minimally have three persons: two for mixing, one for adding water and ingredients. Each team should have two shovels and two trowels, a small bucket (for measuring proportions) and a large bucket (for transporting the mix to the masons).

19.9 MORTAR

Cement mortar is used for masonry construction of walls, and for plastering. Grout is used to cement rebar anchor rods into rocks and imbedding GI pipes into the masonry.

Typical mixes: Proportions of cement to sand, by weight or by volume:

Type of mortar	Cement:sand
Ordinary masonry	1:4
Reinforced brick roof slabs:	1:3
Spatterdash (1st coat plaster)	1:4
Rough plaster (2nd coat)	1:3
Final plaster (3rd coat)	1:2
Grout:	1:1 - 1:1½

Volumes of mortar: The total volume of mortar is equal to the total volume of sand in the mix. The cement mixes with water to form a paste which fills in the voids in the sand. Thus, a 1:4 mix requires 100% sand and 25% cement; a 1:3 mix requires 100% sand and 33% cement, etc.

Quantities required to make one cubic meter (1 m^3) of various mortar mixes:

Mortar mix	Sand (m^3)	Cement (m^3)
1:4	1.0	0.25
1:3	1.0	0.33
1:2	1.0	0.50
1:1½	1.0	0.67
1:1	1.0	1.00

19.10 MASONRY

Because the masonry walls of the tanks are required to be as waterproof as possible against the hydrostatic pressure of the water inside, particular attention must be paid to the workmanship of the masons. It must be made clear to them that a masonry wall built the same as walls for their houses is not adequate, and that the walls of the tanks must be carefully laid down according to directions.

Brick masonry: Bricks are usually locally manufactured in Nepal, and are of various shapes and quality. The exact dimensions of local bricks should be obtained for making the estimated requirements. The total volume of brick masonry is approximately 25% mortar and 75% brick. Bricks should be soaked in water for several minutes prior to being used (this prevents them from absorbing too much moisture from the mortar) but not soaked excessively.

Masons who are experienced at building houses with brick and mud mortar will be inclined to build tank walls in the same manner; laying down a bed of mortar, then placing the bricks tightly together on top of it, then laying down another mortar bed for the next course. The result is a network of unobstructed channels between the bricks where water will have easy leakage. Proper brick masonry for waterproof walls requires spacing the bricks one centimeter apart, and carefully filling in the joints with mortar. Bricks should be laid in patterns that do not result in a straightline joint from the inside to outside of the wall. Refer to Figure 19-2 for various points on brick masonry.

The top course of bricks should be completely clean and wetted before putting down the mortar bed for the next course. If the mortar on the top course has begun to set, the joints should be scraped down approximately one centimeter deep and refilled with fresh mortar. The walls should be built up evenly, so that the weight is distributed uniformly: no section of a wall should be more than 15 courses (approximately 1 meter) higher than the lowest section.

Once the mortar has set, the masonry should be wetted regularly (several times per day) for several days.

Dressed-stone masonry: Also known as "Ashlar masonry". In this type of masonry, the stones are carefully cut to rectangular dimensions, making "stone bricks". Such masonry requires skilled masons, and much

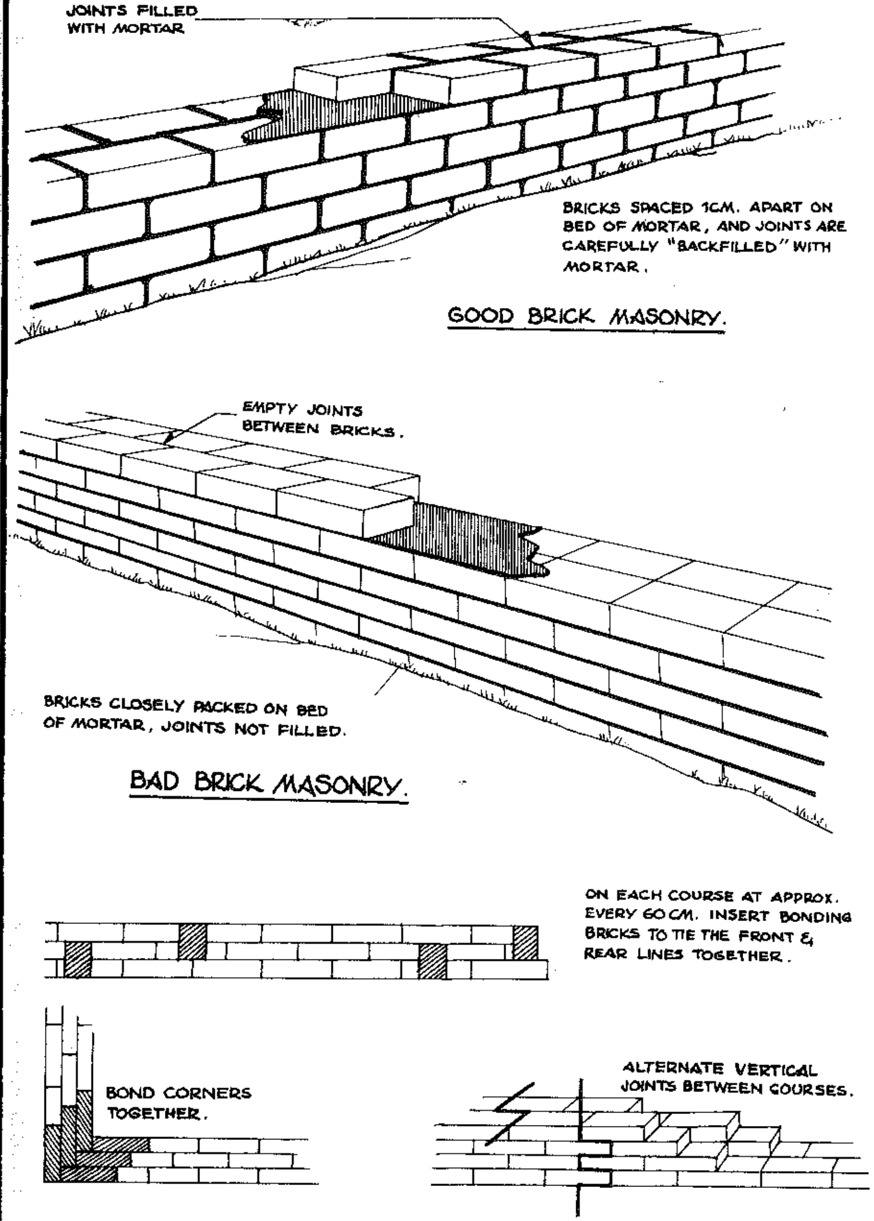


FIGURE 19-2
BRICK MASONRY TECHNIQUES

time and labor. Ashlar masonry is approximately 30% mortar and 70% stone. Refer to Figure 19-3:

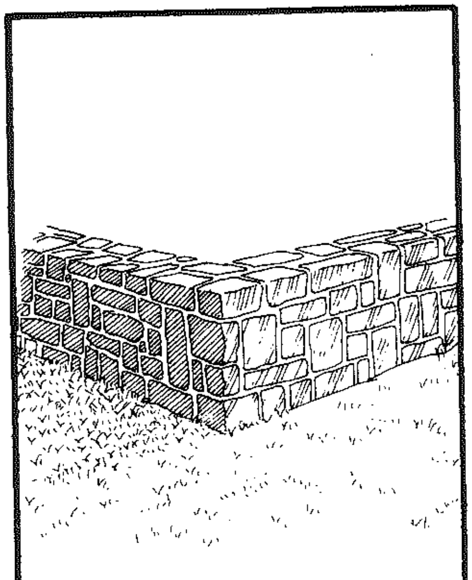


FIGURE 19-3
DRESSED-STONE MASONRY

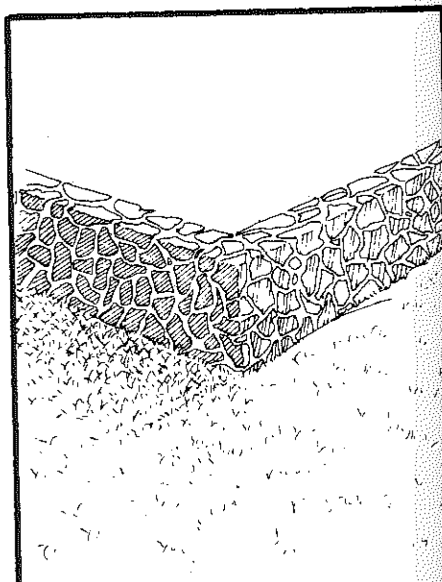


FIGURE 19-4
RUBBLE-STONE MASONRY

Rubble-stone masonry: This is the most common type of masonry used in Nepal. The stones are roughly shaped by the masons, and the resulting wall is similar to that shown in Figure 19-4. The stones should be lightly tapped down into the mortar, then securely fixed using mortar and pieces of crushed gravel. No stone should span completely from the inside to the outside of the wall. With this type of masonry, it is very easy to leave air voids between the stones, so care must be taken that this does not happen. For estimate purposes, this type of masonry is approximately 35% mortar and 65% stone.

Setting GI pipe: GI pipe is set into masonry walls on a bed of grout (1:1 or 1:1½ mortar). A minimum length of 30cm of pipe should be imbedded, and the more the better. Once the pipes have been placed, they must not be disturbed at all for several days. Building a protective dry-stone masonry wall will protect against accidental dislodgement (this can happen quite easily otherwise, for the worksite is the scene of much activity). The pipemouths should be plugged up to keep any mortar from accidently falling into them.

19.11 CONCRETE

Concrete is used for pouring floor and roof slabs of tanks. The size and type of aggregates depends upon the purpose of the slab, its reinforcement, and its thickness (all discussed in Section 19.6).

Typical mixes: The following proportions are recommended for concrete, proportions by either weight or volume:

Normal RCC work (roof slabs): 1:2:4 (cement, sand, aggregate)

Waterproof slabs (tank floors): 1:1½:3

The concrete is proportioned and mixed as already discussed in Section 19.8.

Water: For the above mixes, the approximate amount of water needed is 3/4 parts water per part of cement (1:3/4 cement:water) by volume.

Volumes of concrete: The total volume of the concrete mix is never less than the total volume of aggregates. Typically, air voids make up 50% of the aggregate volume, and these voids must first be filled by the mortar. Excess mortar then adds to the volume of the concrete.

For the above mixes, the following volumes of cement, sand, and aggregate are necessary to produce one cubic meter ($1\ m^3$) of concrete:

Concrete mix	cement (m^3)	sand (m^3)	aggregate (m^3)
1:2:4	0.25	0.5	1.0
1:1½:3	0.33	0.5	1.0

Segregation: This is the separation (due to gravity) of the aggregates in the concrete. The heavier aggregates will tend to sink to the bottom, and water will rise to the surface. The result is a poorly mixed concrete which will be weak. Segregation usually happens transporting the concrete from the mixing pad to the work site, therefore the mixing pad should be as close to the final pouring point as possible, and the concrete should be re-mixed with a trowel before pouring.

Placing the concrete: A bucket of concrete should never be dumped from any height since segregation of the aggregates will occur. Concrete should be placed in strips about 15-20 cm wide, never as piles (refer to Figure 19-5). If a fresh layer is to be put down on top of an earlier layer, then the second layer should be put down before the first has begun to set (within 30 minutes). Rough leveling of the concrete can be done, but extensive trowelling will cause the cement paste to rise to the surface of the slab.

Before it sets, the concrete must be thoroughly compacted.

Compacting: This is the process of settling the concrete so that it contains no air voids. This is accomplished by "rodding"the concrete: poking a length of rebar into the concrete and stirring it up and down.

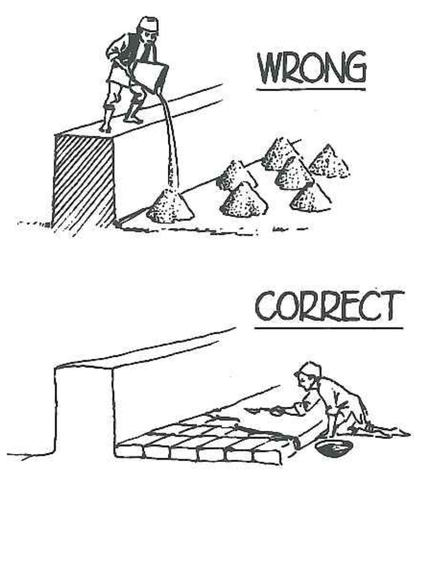


FIGURE 19-5
PLACING CONCRETE

The concrete should be carefully rodded in all corners and around reinforcement. Over-rodding, however, will cause segregation. After rodding, the concrete should be tamped level again, using a flat board of wood. Sprinkling loose cement on the surface of the slab (to absorb excess water) is not good: such a layer will easily crack, crumble, and powder.

Waterproofing floor slabs: A day after the concrete has been placed, a water proofing plaster may be put down. A grout mixture of 1:1 proportions should be worked into the surface of the slab with a wooden float. Only a thin layer of plaster is needed, just enough to seal the surface pores of the slab and smooth it over. Waterproofing compound can be added to the grout (refer to Section 19.17).

Curing: As soon as the concrete has set (within a few hours), the floor slab should be flooded with a few centimeters of water. More than this will put too much hydrostatic pressure on the concrete, which may not be strong enough to support it. After one day, the water may be drained off for waterproofing (as described above), but once the waterproofing plaster has set it should be re-flooded and kept that way for several days (after 3 days, if all else is finished, the tank may be filled fully and put into service).

When first flooding the slab, care must be taken that the discharge flow doesn't erode the fresh concrete.

If the slab is being poured over a period of several days, the surface of each section must be covered over with a tarpaulin and constantly wetted. This method must also be used for drying a roof slab.

Improper curing will allow the surface of the slab to dry out and shrink, while the interior mass remains unchanged. The resulting tensions will cause the surface to crack, reducing waterproofness. Too much loss of moisture will stop the hydration reaction, and no further strength will develop (even if the concrete is thoroughly flooded again).

19.12 PLASTERING

Plastering masonry walls adds to their waterproofness. Sever coats of increasing richness (ie- cement content) are better than or two thick coats.

All walls will receive three coats of plaster, each 1 cm thic should be plastered at least 5 cm above the overflow level.

Spatterdash: This first coat is a rough plaster of 1:4 mort is applied by spattering the plaster against the walls, using a t This coat is NOT troweled smooth, and the resulting surface is ex bumpy and irregular. This provides a good rough surface for the layer to adhere to.

Second coat: A mortar mix of 1:3, applied to the spatterdash coat. That coat is left with a rough surface.

Third coat: The final coat is a 1:2 mortar mix, which is finally troweled smooth and clean.

Only one coat of plaster per day should be applied.

Volumes of plaster: For a plaster coat 1 cm thick, the following quantities of cement and sand are needed for each square meter of plastered surface:

Plaster mix	Cement (m^3)	Sand (m^3)
Spatterdash (1:4)	0.0025	0.01
Second coat (1:3)	0.0030	0.01
Third coat	0.0050	0.01

19.13 FLOOR SLABS

Tank floors may be of mortared brick or stone, or concrete (either non-reinforced or reinforced). The floor slabs of tanks must be as waterproof as possible. Concrete slabs should use carefully-selected aggregate, and mortared slabs should be plastered.

Foundations: Tank floors are put down on a bed of gravel or crushed stone, averaging 10cm deep. This gravel bed should be pitched down towards the washout pipe. An easy way of accomplishing this is to radiate several strings from the mouth of the washout pipe to various points around the perimeter of the tank, as shown in Figure 19-6. Each string is at the desired slope, and the gravel bed is set down according to the nearest string. The gravel should be several centimeters below the washout pipe (otherwise it will not be possible to fit the floor slab underneath it !).

Mortared brick: A brick floor slab should consist of two layers of brick, laid flat on beds of mortar, carefully spaced with good mortared joints in between. The mortar should be 1:4 mix. The line of bricks should be different between the two layers. The floor is then plastered with two coats (the first at 1:3 mix, and the second at 1:2 mix,

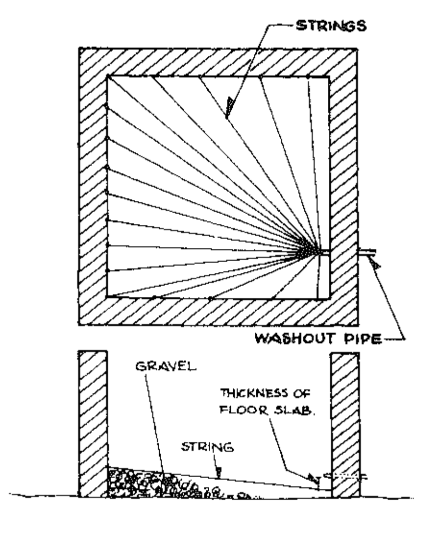


FIGURE 19-6
PITCHING FLOOR GRAVEL

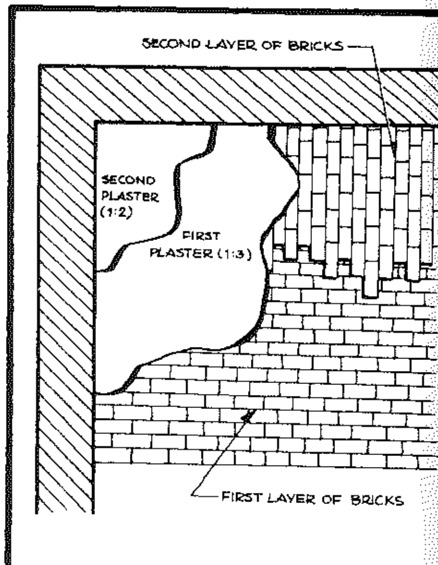


FIGURE 19-7
BRICK FLOOR

no spatterdash coat needed). As soon as the second plaster coat has set, the tank should be flooded a few centimeters deep (for curing the mortar) for several days. Refer to Figure 19-7.

Mortared stone: This type of flooring is more difficult to put down, especially with rubble-stone masonry. Care must be taken to prevent air voids in the masonry. This type of floor should be 15 cm thick, mortared and plastered as prescribed above.

Reinforced (RCC) concrete: The aggregate for concrete slabs should be small and well-graded, maximum size being 18mm. Rounded river gravel is excellent for this type of slab. A reinforced slab should be 8cm thick, with the reinforcement according to the specifications of Section 19.7, and the concrete work according to Section 19.11.

19.14 ROOF SLABS

Although roof slabs of concrete or reinforced brick require extra materials and labor, such roofs are structurally quite strong, and should never need replacing during the lifetime of the system. A slab roof also effectively seals off the tank from external contamination.

An accessway must be left in the roof, approximately 60cm x 60cm. Bolts should be imbedded for securing the accessway cover.

Structurally, the roof slab must be "tied" into the masonry walls of the tank. This is done by imbedding rebar rods in the walls, then bending them over and securing the roof reinforcement to them. This is shown in Figure 19-8:

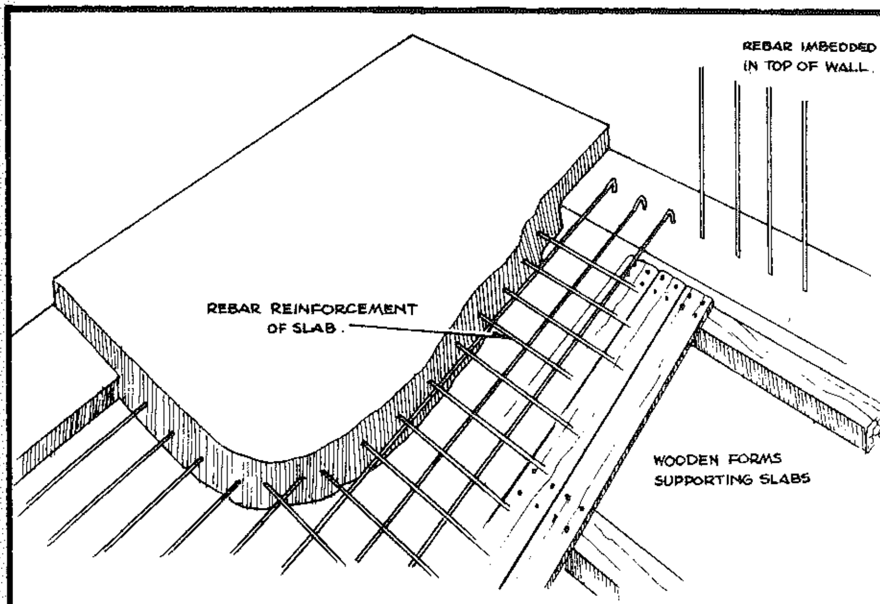


FIGURE 19-8
RCC ROOF SLAB

Columns: No unsupported span should be more than 4 meters. Support can be made using GI pipe as columns:

GI pipe size	Max height of column
1½	178 cm
2"	278 cm

The ends of the column must be threaded and have flanges (which act as bearing plates). The bottom flange of the column should rest on the floor slab, with additional concrete on it to hold it firmly in place (rebar studs can be left protruding from the floor slab to help anchor the concrete cover).

A "beam" of 1" GI pipe spans the interior, supported directly on the column. This GI pipe beam should be in the exact centre of the roof slab.

Figure 19-9 shows the arrangement of a GI pipe column and beam.

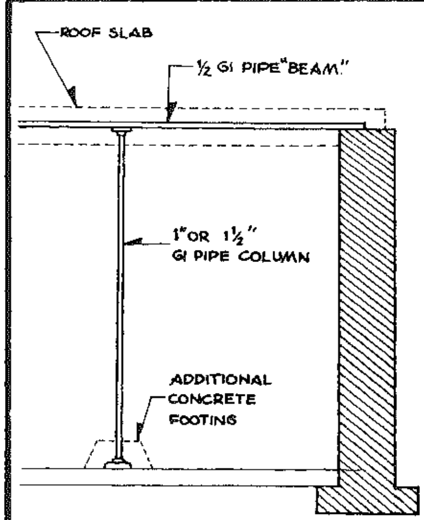


FIGURE 19-9
GI PIPE COLUMNS

RCC slabs: The amount of rebar in an RCC roof slab must not be less than 0.30% of the cross-sectional area of the slab. The thickness of the slab should not be less than 1/30 (3.33%) of the longest span of the tank, but the thickness should not be less than 8 cm. The concrete mix should be 1:2:4, aggregates should be well-graded of crushed stone or brick (if crushed brick is used, the roof should be plastered with a 1 cm coat of 1:3 mortar). Largest size of aggregates should be 10mm. Refer to Section 19.7 for details of rebar.

Reinforced (RF) brick slabs: Roof slabs of RF brickwork offer considerable savings in cement, compared to an RCC slab. The thickness of an RF brick slab should not be less than 1/30 (3.33%) of the longest span of the tank. The size of the rebar should be either 5/16" (8mm) or 3/8" (10mm), and is spaced between the bricks. Bricks are laid in one direction, spaced 4 cm apart. The rebar must not touch any brick (such contact would allow moisture from the brick to corrode the rebar). The cement mix is 1:3 mortar, with enough water to make it easily worked into position around the rebar and between the bricks. A second pouring of mortar may be required, to bring the slab to the proper thickness (due to settlement of the mortar). Figure 19-10 shows different brick arrangements for roof slabs of various thickness.

Curing: Curing the roof slab is very important, since otherwise it may not develop the full strength it needs to support its own weight (and the weight of those who will stand upon it). Unfortunately, curing a roof slab is not so easily accomplished as for a floor slab. Direct exposure to sunlight will greately hasten the drying-out of the slab if care is not constantly taken to prevent this.

The rim of the slab should have a low wall (of brick or dirt) around the edge of it, about 20 cm high. The slab itself should be covered with several centimeters of sand, which is then thoroughly wetted using several buckets of water. The sand is then covered using a plastic or canvas

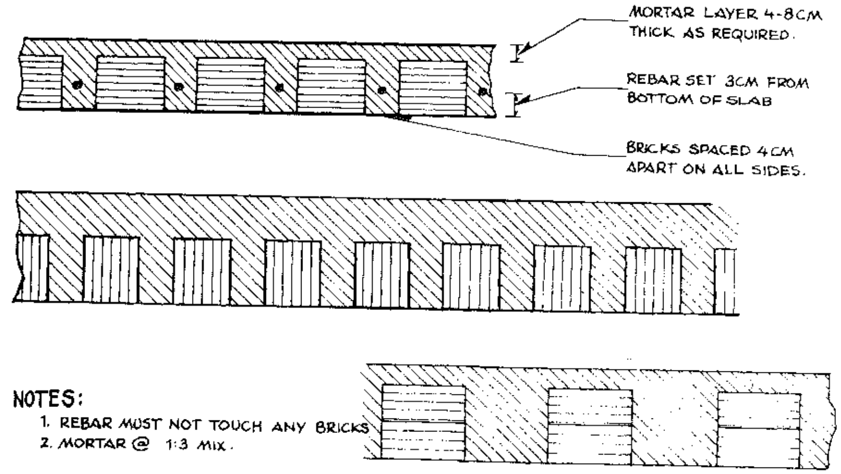


FIGURE 19-10
RF BRICK ROOF SLAB

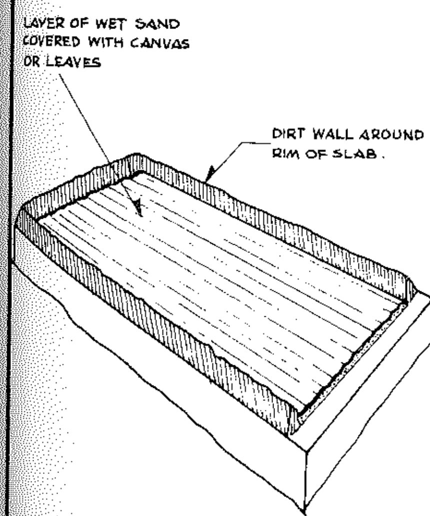


FIGURE 19-11
CURING ROOF SLABS

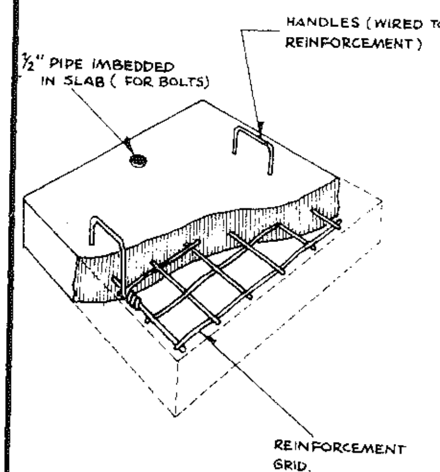


FIGURE 19-12
SMALL COVER SLABS

tarpaulin, straw mats, or several layers of banana tree leaves. The sand is re-wetted at least three times per day, for a week, after which the surface can be cleared off and forms stripped away.

19.15 SMALL SLAB COVERS

Small RCC slabs (less than 100cm square) can easily be made for covers of valveboxes, break-pressure tanks, or accessways in larger tanks.

For such slabs, the reinforcement is best done using large-mesh wire screen, but small size rebar can also be used.

A simple wooden form can be constructed. The rebar is firmly set in place, and short pieces of ½" GI or 20mm HDP are fixed into position where the 3/8" bolts will pass through. Handles of wire or rebar should be tied to the reinforcement, so that the slab can be lifted.

The thickness of the slab should not be less than 5 cm, and not more than necessary to cover the rebar with 2½ cm of concrete on both sides.

The concrete mix should be 1:2:4, with small-size aggregate small enough to fit through the mesh if wire screen is used.

After the concrete has been poured, the slab should be covered with sand and kept wetted for three days. After that time, if the form is needed to make more covers, the slab can be carefully removed from the form and kept in a shady place for several more days, being constantly wetted. Covering the slab with wet burlap (jute) sacking will help to keep it moist.

When the concrete has been cured for several days, the slab may be plastered with a 1:3 mortar, to give it a smooth, clean surface.

Figure 19-12 shows some details of the form and slab reinforcement.

19.16 FERROCEMENT TANKS

A new type of tank is currently being developed for use in Nepal, constructed of ferrocement. When practical construction methods have been finally worked out, such tanks will offer a considerable savings of cement and labor, compared to regular masonry tanks of equal capacity.

Essentially, ferrocement is made by wrapping light wire screening (such as chicken-wire) around the outside of a form, and then heavily plastering the screening with a 1:3 cement mortar. When the plaster has strengthened, the interior form is stripped away, and the inside of the screening is then equally plastered. The resulting wall is about 5 cm thick.

The same technique can be done for the roof of the tank.

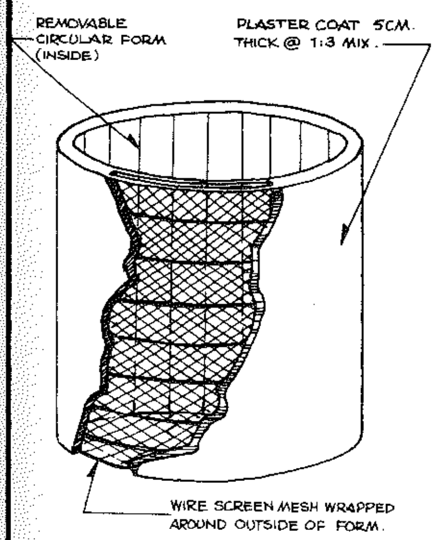


FIGURE 19-13
FERROCEMENT TANK

For quick technical reference, see "Ferrocement Water Tank" by Frans Dubbeldam, printed by the German Volunteer Service in Nepal. For general reference, refer to "Ferrocement Water Tanks and their construction", by S.B.Watt, printed by the Intermediate Technology Publications, London, UK.

19.17 WATER-PROOFING COMPOUNDS

Commercially-manufactured additives may be mixed into the mortar or concrete dry-mix, to improve the imperviousness of the resulting plaster or concrete. Such compounds are generally packaged in the quantity that should be mixed into a full 50-kg bag of cement. However, typical construction work of CWS projects rarely calls for mixing an entire bag of cement at once, which means that the compound must be sub-divided into smaller portions. This is most accurately done by weight, which is not easily done in the field.

No amount or type of additives will make a poorly-mixed or poorly cured plaster or slab waterproof. Therefore, the most improtant procedure is to see that the cementwork is properly done. Additional compounds are helpful, but not essential.

19.18 COLD WEATHER CONCRETING

When cementwork must be done where temperatures are expected to drop down to freezing levels, special precautions must be taken to protect the cement.

When concrete or mortar freezes, the hydration reaction is stopped permanently. Even when the cement is thawed and re-wetted, the chemical process does not resume, and the concrete develops no further strength than it had when it first froze.

Since the hydration reaction generates heat, the single best strategy is to insulate the cementwork to prevent the loss of this heat, especially during the first two days (when the rate of heat loss would be highest).

Padding the cementwork with straw and covering with mats or tarpaulins will be of special help. When re-wetting the cement (during curing), heated water should be used if possible. Protecting the cementwork against the wind is extremely important, and all protruding rebar should be wrapped with cloth (since steel is an excellent conductor of heat, these would be major points of heat loss).

The setting and hardening of cement is temperature-dependent, and will proceed more slowly at lower temperatures. Increasing the amount of cement in the mix by 20-25% will help generate more heat and earlier strength. Heating the aggregates and using hot water for mixing will improve the setting time (aggregate should not be heated hotter than can be touched by the hand, nor should the water be hotter than 140°F/60°C. Never heat the cement alone, or add hot water to cement alone).

The freezing point of the mix may be reduced by dissolving salt into the heated mixing water. Salt is added by weight, and should not exceed 5% of the weight of the cement. Each percentage of salt lowers the freezing point by about 1½°F (0.8°C), but salt cannot be used effectively for temperatures lower than 25°F (-4°C).

20. PRACTICAL TECHNOLOGY

"Necessity is the mother of Invention"

20.1 INTRODUCTION

Technical theory is useless without effective and practical methods of applying it. Years of experience in the field have yielded many practical construction techniques to supplement and implement the theory of CWS construction. This chapter will present some of these ideas.

20.2 SCREENED INTAKES

Screened intakes can be quickly made using light wire mesh or ordinary window screening, and HDP pipe.

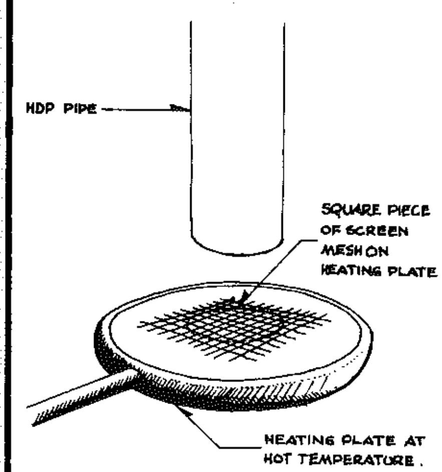


FIGURE 20-1
SCREENED INTAKE

The size of the HDP pipe should be one or two sizes larger than the outlet pipe. A square piece of screen, slightly larger than the mouth of the HDP pipe, is laid onto the hot heating plate. The HDP pipe is then pressed against the screen and heating plate until the melted lip is formed. The HDP is then given a slight twist, and re-moved from the plate. The screen will be perfectly welded across the end of the pipe. When it has cooled, the excess screening is trimmed away.

This screened intake takes a few seconds to make (indeed, it is possible to make several without reheating the plate), and the villagers can easily make new ones to replace those worn out.

20.3 JOINING HDP & GI PIPE

At points of low pressure, HDP and GI pipes can be joined without using flanges or brass unions.

Threading: Depending upon the size of the HDP pipe, it can sometimes be threaded (just like GI pipe) and screwed in GI fittings. This is particularly possible with Class IV pipe, which has thicker walls. A short "nipple" of Class IV HDP can be threaded into a GI fitting, and regular Class III HDP pipe welded on.

Expanding: The HDP pipe can be heated and softened over a fire, and then a threaded GI pipe or nipple can be jammed/screwed into it.

The HDP/GI joints should only be used at low-pressure points. They are useful for making a discharge pipeline of tank washouts and overflows, and for putting HDP screened intakes onto the outlet pipes, and for air-vents, etc.

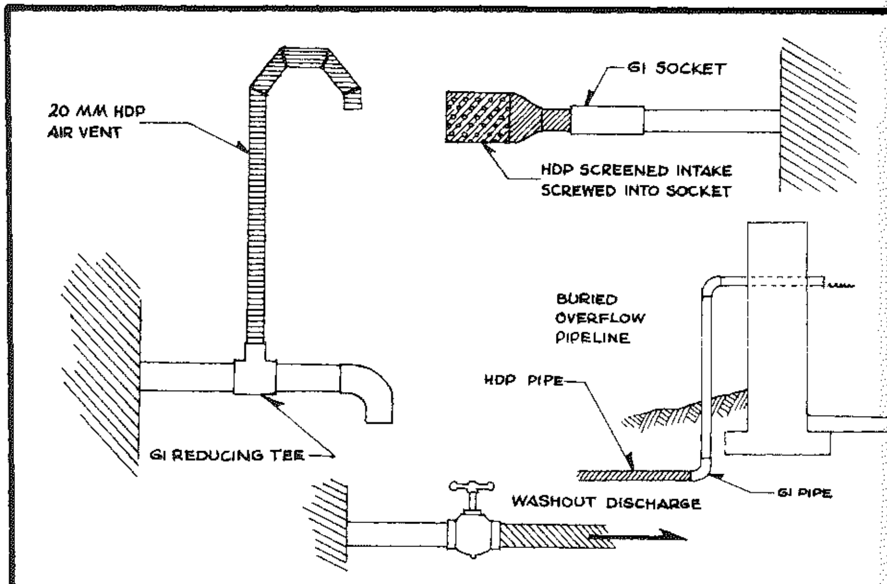


FIGURE 20-2
DIRECT HDP/GI JOINING

20.4 IMBEDDING BOLTS IN MASONRY

The best size of bolts to use is 3/8" x 5" (10mm x 15cm) with two washers and nut. Clamp the head of the bolt into a vise, and slip a length of ½" GI pipe (75-100cm long) over it. Using the pipe as a lever, bend the bolt over 90°. Set the bolt into the fresh mortar or concrete, leaving about 4 cm protruding (a longer bolt may have to be used if a thick RCC slab cover is to be bolted down).

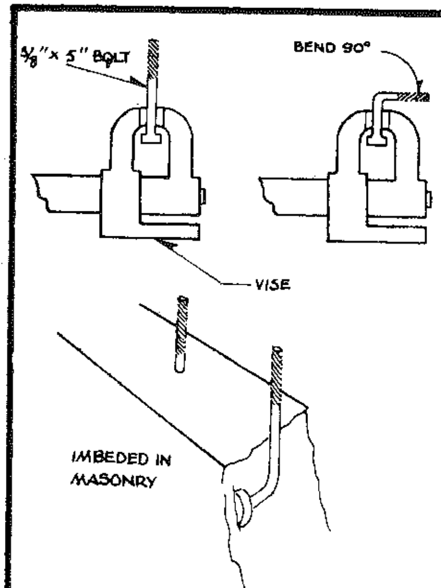


FIGURE 20-3
BOLTS IN MASONRY

This type of imbedded bolt will never "spin" when a wrench is used to remove the nut.

Refer to Figure 20-3.

20.5 ANCHORING BEAMS TO MASONRY

Using the same technique as described above, a 60cm length of 3/8" (10mm) rebar is bent 90° and imbedded in the top of the masonry walls. A ½" (12mm) hole is drilled in the wood beam, which is then slipped over the protruding rebar. The rebar is then hammered over, locking down the beam to the wall.

Beams anchored in this manner are easily removed when it comes time to replace them.

Refer to Figure 20-4.

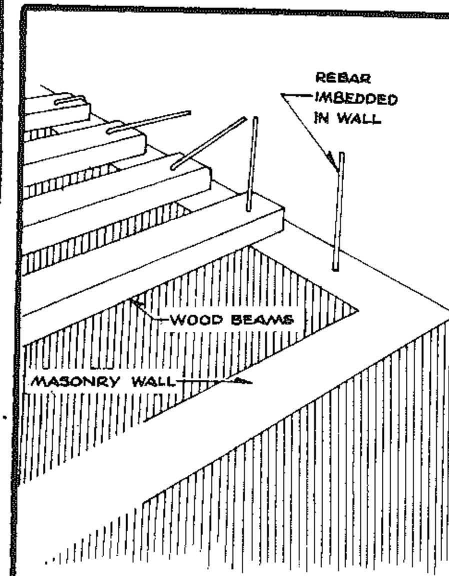


FIGURE 20-4
BEAMS IN MASONRY

20.6 LOCKING DOWN CGS COVERS

Valvebox or accessway covers of CGS roofing are quite quick and easy to construct. However, they suffer from one serious weakness: where holes have been cut in them (for bolts), it is very easy for someone to enlarge the hole and then slip it over the washer and nut that are supposed to be locking it down.

To prevent this, a special washer can be made by the village blacksmith. It is made from a piece of flat iron, and measures 5 cm across. This special washer is large enough to completely protect the hole in the CGS and prevent people from enlarging it. The nut should be tightened down so that there is no way of shifting around the CGS cover.

Refer to Figure 20-5.

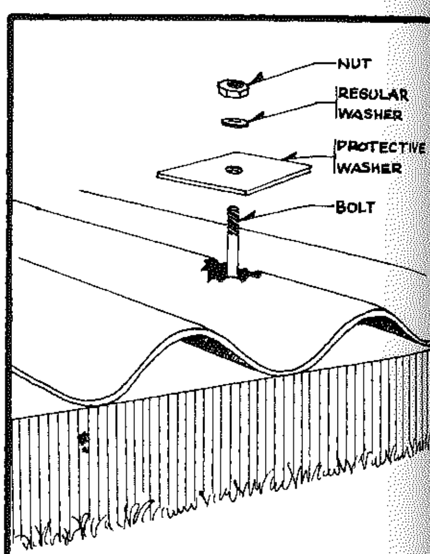


FIGURE 20-5
CGS COVERS

20.7 FASTENING BOLTS TO BEAMS

In the same fashion described already in Section 20.5, the bolts are bent 90°. They are then securely fastened to the side of the wooden beam, using 1½" or 2" nails, as shown in Figure 20-6.

20.8 BRACKETS FOR AIR-VENTS

An air-vent that must extend outside the protective confines of the valvebox should be made with ½" GI pipe, firmly mounted to the tank wall. Such a mounting bracket can be made using small-size rebar, fashioned into shape by the village blacksmith (refer to Figure 20-7).

These brackets should be mounted about one meter apart; the brackets are imbedded directly into the masonry at the time of construction.

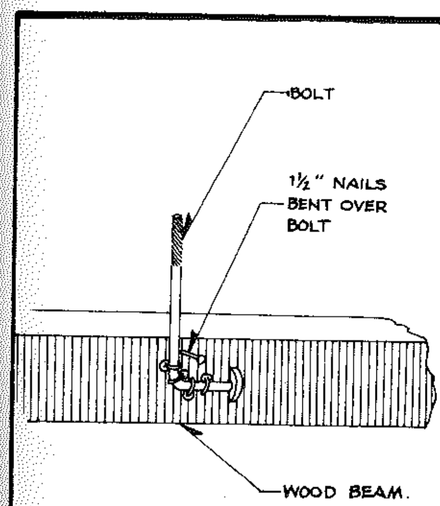


FIGURE 20-6
SECURING BOLTS TO BEAMS

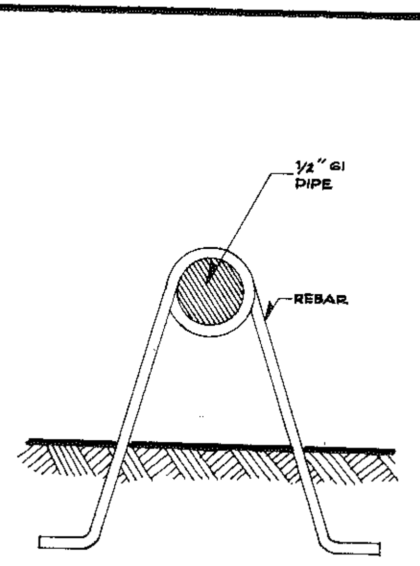


FIGURE 20-7
BRACKETS FOR AIR-VENTS

TECHNICAL REFERENCES

The following list of publications was used in compiling this handbook, or would be useful for further, in-depth study. In addition, the reader is encouraged to investigate the technical libraries of UNICEF, LDD, Peace Corps, German Volunteer Service, and other agencies in Nepal which are involved in similar projects, as well as the personal reference materials of the engineers of LDD.

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