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Village Water Systems

by: Carl R. Johnson

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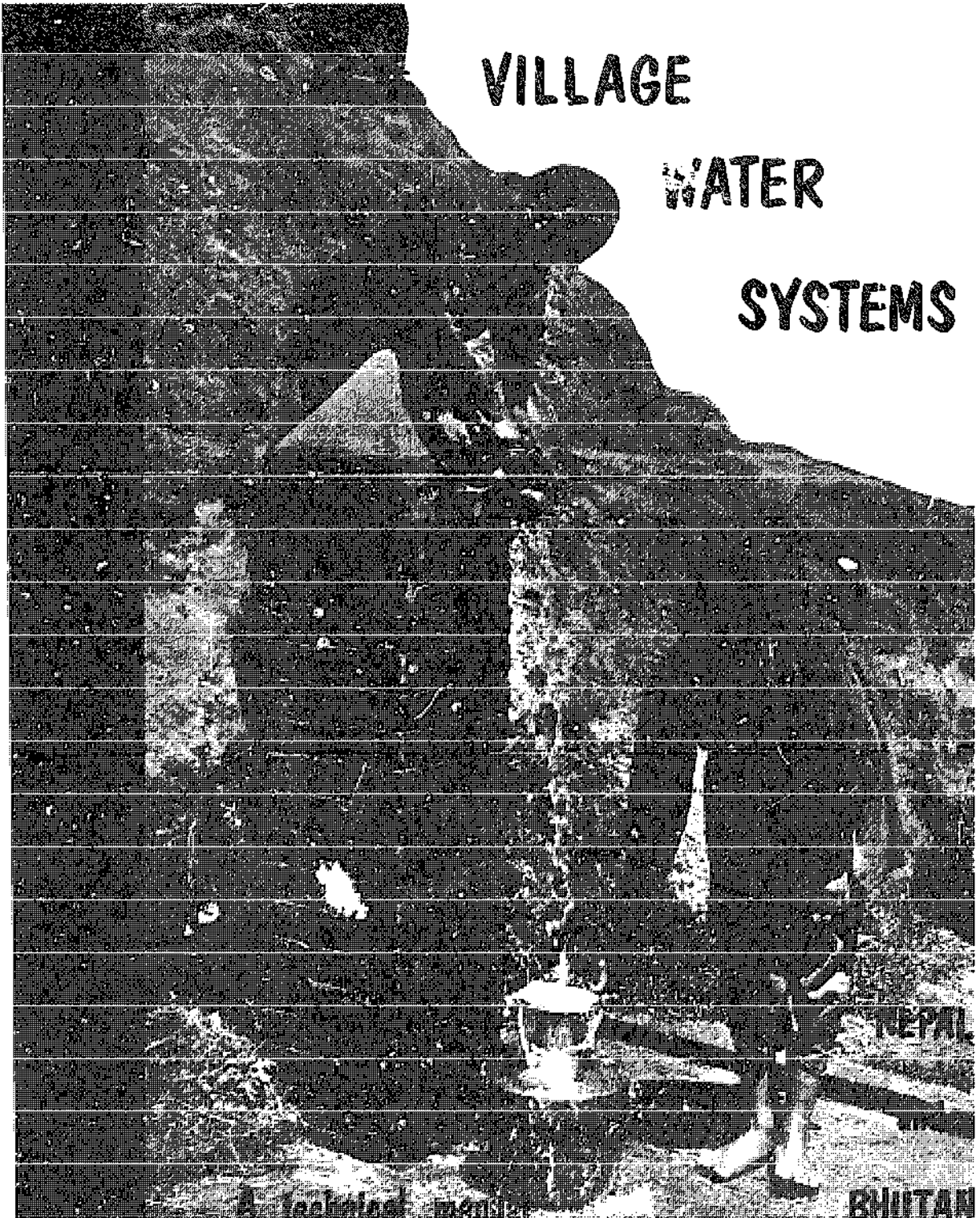
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VILLAGE

WATER

SYSTEMS



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Technical manual

BHUTAN

STANDARDS AND PROCEDURES
FOR THE DESIGN OF WATER SUPPLY SYSTEMS
IN RURAL AREAS OF NEPAL AND BHUTAN

(With special reference to the projects being implemented by the
Remote Area and Local Development Department (RA & LDD), HMG, Nepal.)

Compiled By

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ABBREVIATIONS AND SYMBOLS

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UNITS

cft - -	cubic feet	LPCPD - -	liters per capita per day
cm - -	centimeters	LPS - -	liters per second
cm ² - -	square centimeters	m - - -	meters
kg - -	kilograms	m ² - - -	square meters
kg/cm ² -	kilograms per square centimeter	m ³ - - -	cubic meters
km/hr -	kilometers per hour	mm - - -	millimeters
l - - -	liters	MPS - - -	meters per second

TERMS

GI - -	galvanized iron	no(s) - -	number(s)
HDP - -	high density polyethylene	oc - - -	on center
HGL - -	hydraulic grade line	OD - - -	outside diameter
ID - -	inside diameter	PVC - - -	polyvinyl chloride

SYMBOLS

H - -	head	Ø - - -	diameter
H _L - -	headloss	 - -	water surface exposed to atmospheric pressure
Q - -	flow	" - - -	inch
⊕ - -	centerline	√ - -	squareroot operative
@ - -	at		

1. INTRODUCTION

=====

This manual is especially written as a reference for persons engaged in rural water supply design and construction for the Remote Area and Local Development Department of His Majesty's Government of Nepal and its assisting agencies: UNICEF, WHO, American Peace Corps, and German Volunteer Service. The procedural and construction methods detailed in the text derive from current policy and practice of the Remote Area and Local Development Department (RA & LDD). The design information presented within reflects particular design problems of small water system construction in remote hill and mountain regions of Nepal. Therefore, while some procedural and construction aspects are relevant only to RA & LDD projects, the design methods may benefit a wider readership, including personnel working for other agencies in Nepal and Bhutan in the field of rural water supply.

The coverage of specific topics ranges from criteria and considerations for the design of certain water system features to fully developed standard designs, estimates, and specifications for other features. Design criteria are presented where standard designs can not be practically used, while the standard designs that are included are for guide purposes only and should always be carefully scrutinized in light of the specific conditions at the construction site.

The text of this manual has been written with measurements expressed in the International System of Units (SI). The one exception is the chapter on surveying which uses English units. The SI unit system has been adopted in keeping with the spirit of the universal trend toward metric based measurements. English units are used in the surveying chapter because most of the present surveying equipment available to overseers and engineers working for RA & LDD is calibrated in feet and inches.

Certain material presented in the text is based on field tests, empirical formulae and methods from other publications and is so referenced by notation or by reference number as listed at the back of the text.

2. VILLAGE SURVEY AND FEASIBILITY STUDY

=====

INTRODUCTION

The survey of the project site should fulfill four major requirements. These requirements are:

1. SOURCE EVALUATION
2. LENGTH OF PIPELINE SYSTEM AND RELATIVE ELEVATIONS
3. DETERMINATION OF MATERIAL, TRANSPORTATION, AND LABOR AVAILABILITY AND RATES
4. EVALUATION OF VILLAGE MOTIVATION AND COOPERATION

Each of these four requirements is discussed in detail in the following paragraphs.

SOURCE EVALUATION

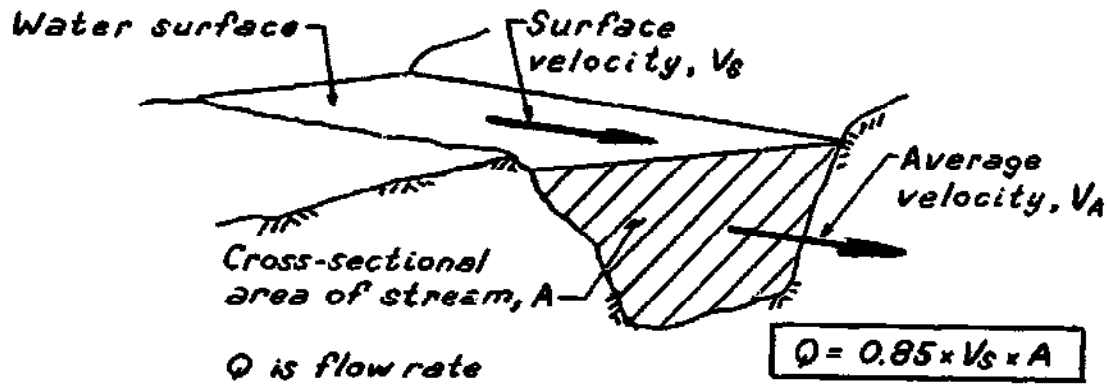
Evaluating the source(s) for the water system should be the first step in the survey process. The quantity and quality of the water must be determined, along with source development potential and investigating water rights disputes and conflicts.

Springs and streams are the two most common types of sources. The quantity of spring flow is most conveniently measured by calibrating a wide-mouthed collection vessel and timing its filling. Stream flows are usually too large to be measured by the previous method, so a velocity-area flow measurement should be made. The flow in cubic meters per second is equal to the water velocity in meters per second multiplied by the cross-sectional area of the stream perpendicular to the velocity direction measured in square meter units. This relationship is pictured in Figure 1.

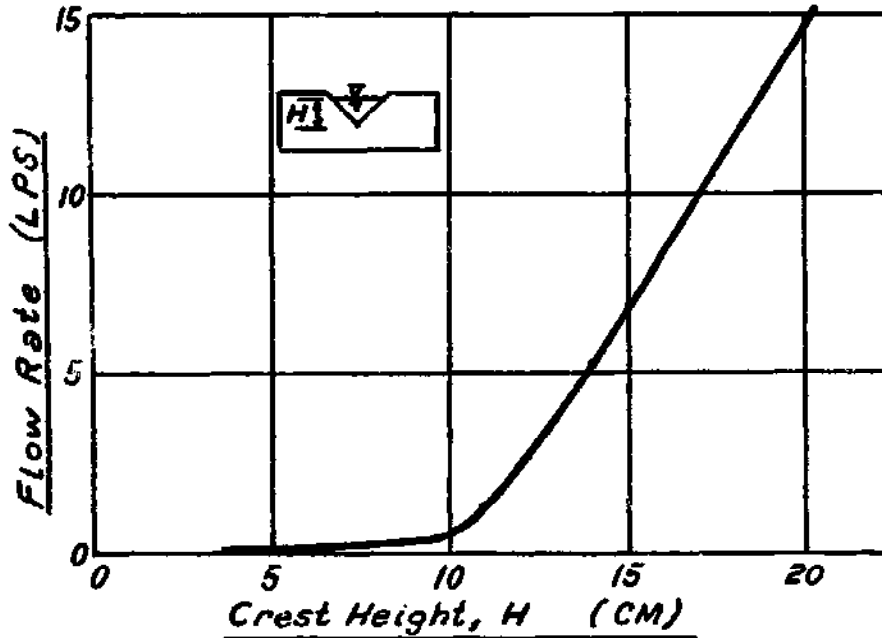
Measure a stream flow in the following way. Find a straight portion of the stream and measure the cross-sectional area of flowing water. A "straight" portion is one that is free of bends and obstructions over a distance of at least 6 to 10 times the average water depth. The average velocity is usually at mid-stream at a depth of 0.6 times the full depth at the section. However, an accurate enough measurement can be made by timing the movement of a surface float along a predetermined distance, and assuming the average velocity to be smaller by a factor of 0.85 to 0.90. Two or three measurements should be made and the average of these taken and reported. This method is applicable for water depths of at least 30 centimeters. (References # 1,6,7.)

Alternatively, a weir can be used to measure a small stream. A V-notch, 60° weir, as pictured in Figure 13, is recommended for the normal range of small flows encountered in village stream sources. The weir should be placed perpendicular to the stream (both horizontally and vertically) and a straight, unobstructed flow distance of at least ten times the weir crest width should precede the weir. The graph in Figure 1 shows flow rate as a function of water height passing over the notch ("H" in Figure 13) for a thin, 60° V-notch weir. Refer to the section in Chapter 5 on flow measuring devices and References # 6,13,17, and 21 for further information and data.

FIGURE 1. RELATIONSHIPS FOR STREAM FLOW RATE MEASUREMENT



A. VELOCITY-AREA METHOD RELATIONSHIPS



B. 60° V-NOTCH WEIR RELATIONSHIPS

The "SAFE YIELD" of the source is typically the minimum flow which occurs in the dry season. The "SAFE YIELD" is that amount of water which can always be withdrawn without fear of over-drawing from the source. Unless the survey is done in April or May (driest season), then the minimum flow will have to be determined by asking as many villagers as possible about minimum flow conditions. Maximum flows may also be determined by questioning, or more preferably, measuring monsoon flows directly. As the minimum flow, safe yield, is important for pipeline and reservoir design, and the maximum flow important for estimating structural protection and overflow requirements, accurate figures for flow extremes should be carefully established.

The quality of the source water must also be evaluated. Due to transportation problems and high costs, physical and chemical water testing is not presently feasible. However, biological contaminants can be tested for with a portable, battery-powered field testing unit. (See the water quality section of this manual for water quality standards.) Most of the time biological testing will not be done at the time of the initial survey. Therefore, the water quality assessment is based largely on the villagers' advice. They know through experience if the water in the source they have chosen is drinkable and of acceptable quality.

If a water source with turbidity (cloudy water), taste, and/or odor problems is encountered, then these problems might be readily amenable to a simple treatment scheme. (Refer to water quality section). In any case, investigate all potential water sources, choose the best, and make note of the water quality. As water quality can vary with season, it is wise to ask the residents about such variations in quality.

The third phase of source evaluation is making note of how the source can best be developed. Questions which ultimately must be answered are: How can the tank be protected from monsoon floods and contamination? How can water be most efficiently collected? And how stable and constant is the water source? Field notes concerning the topography, area description, geology, and answers to above questions will be useful in the subsequent design of the intake works.

The final aspect of source evaluation is investigation of any water rights disputes or potential conflicts which may arise after developing the source. While it is not the surveyor's duty to settle such difficulties nor to embroil one's self in these conflicts, one should make note of names and details and discuss such problems with local government officials.

PIPELINE SYSTEM TOPOGRAPHY SURVEY

The determination of pipeline lengths and relative elevations is the most time consuming part of the survey and the most important. Many surveys are short enough so that an ABNEY hand level and tape survey is of sufficient accuracy. Long surveys and surveys over rough terrain are most accurately done with a transit-stadia-and/or tape method. The care and use of the ABNEY hand level is described in detail at the end of this chapter.

The first step of the survey is to walk the proposed route and familiarize yourself with the terrain, tap and branch locations, and place names. While choosing best pipe routes remember that the side of a path or trail is best, as within the path rapid erosion can occur if the backfill is not properly

stabilized. Pipes laid through fields must be deep enough to preclude being plowed up.

While surveying, in addition to the basic numerical information entered in the field book, the following notes should be recorded along the route:

- Soil conditions - type, stability, erosion conditions
- Vegetation - (are fields burned over each year?)
- Special pipe protection requirements
- Special structure location feasibility (i.e. is this a good place for a tank, air release or washout valve placement?)
- Extra pipelength requirements
- Potential construction problems in the area
- Other information helpful in the design

Elevations should be established on permanent structures along the survey route. Such permanent elevation marks are called "benchmarks". A corner of a step, a prominent point of an immovable stone, and a well marked high point of a tree root are good benchmarks. Benchmarks are elevation reference points that are very conveniently used when resurveying in the future, surveying during construction, and for "closing" the present survey.

It is very important to "close" the survey. Closing is required on any branch or route of the survey and is simply resurveying from the last point back to the beginning. The closing survey need not be along the original route; however, it is easier to check for errors in benchmark elevations if the same route is surveyed for closing.

The following is an example to illustrate the use of closing a survey with an Abney hand level and steel taps. Say the survey starts at the source (at an arbitrarily set elevation of 5000 feet) and proceeds to a branch point whose elevation is found to be 300 feet lower than the source or 4,700 feet. Surveying back from the branch point elevation of 4,700 feet to the source, the elevation of the source is now found to be 5,025 feet. Assume the slope distance from source to branch point is 4,000 feet.

The question is, is this survey accurate enough with a potential error of 25-foot in elevation difference over 4,000-feet of distance? There is an 8.3% elevation error ($25/300$) over a distance of 4,000 feet. Given the accuracy limitations of the ABNEY and the problem of establishing accurate figures for water flow, two acceptable targets of error might be 20 feet $\times \sqrt{\text{SLOPE DISTANCE IN MILES}}$ and that the percentage error in elevation difference not exceed 6%. The survey in the example fails on both criteria, so another survey is required to find the error. The first criteria only permits an error of $20 \times \sqrt{(4000/5280)} = 17.3$ feet, while the percent error criteria is exceeded by 2.3%. Note that the target of accuracy for a transit survey should be about 0.5 feet $\times \sqrt{\text{DISTANCE IN MILES}}$ (Reference # 7.) Considering the time, effort, and expense involved in the construction of a village water system, it is well worth spending extra time on the survey to insure that the basic information is correct.

The route survey information should be summarized on a plan view map and a longitudinal profile. The former identifies locations of buildings, taps, branches, and distinctive topographical features; while the latter emphasises the distances and elevations of the route and pipeline system. If compass bearings are recorded, then the horizontal plan view can be drawn to scale, otherwise represent angular deviations as closely as possible.

The longitudinal profile when carefully plotted to appropriate horizontal and vertical scales, is an invaluable piece of information for the hydraulic design. Having designed the water system, the hydraulic grade line is drawn to scale on this longitudinal profile. Refer to the section on hydraulic design for details of HGL construction.

DETERMINATION OF MATERIAL, LABOR AND TRANSPORT AVAILABILITY AND RATES

The various questionnaire forms supplied by RA & LDD are self-explanatory as to their information requirements. Securing accurate data as to the availability and rates for material, labor and transportation is important for the viability of the project cost estimate.

VILLAGE COOPERATION AND MOTIVATION EVALUATION

The ability of the villagers to work together and carry out a drinking water construction project is something that is difficult to judge in advance, and better estimates of this "motivation factor" will come with experience. Nevertheless, it is necessary to get some idea of the motivation, cooperative ability, and potential social and political conflicts while doing the survey. Talking to people; examining past construction efforts; inspecting the conditions of roads, buildings, and public areas; and consulting with the local government officials will provide helpful clues to the possible success or failure of the construction effort. If it appears that a voluntary free labor construction effort would not be workable, discuss other options (such as use of the village land tax for paying laborers) with these officials.

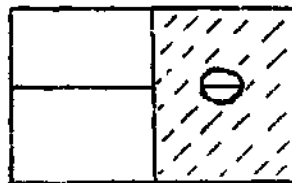
CARE AND USE OF THE ABNEY HAND LEVEL*

INTRODUCTION

The Abney hand level is a versatile reconnaissance instrument which may be used in a variety of applications for projects in less developed countries. The instrument consists of a square sighting tube equipped with a draw eye-piece on the observer's end and a horizontal metal cross hair on the objective end. Near the center of the sighting tube a 45° mirror is inserted covering half the field of view and reflecting half the sight line 90° up and through an aperture in the tube. Above the tube is a level bubble with an etched index mark at its center. The bubble is, in turn, affixed perpendicular to a movable index arm which coincides with the graduations on a nickel silver arc. Abney arcs may be graduated in per cent of grade, degrees or other special graduations. For the purposes of the present method of surveying, the per cent arc is used in all applications.

To use in producing a line of sight, the instrument is held to the eye and tilted up or down until the reflected image of the level bubble is centered on the index mark (Fig. 2). If the arc index is set on zero, the line of sight past the cross hair is level. If the arc index is set on any other value, the line of sight is at whatever percent elevation or depression the arc index has been set. For measuring an inclined sight line, the observer sets the cross hair on the sight line and then adjusts the arc index until the bubble comes into level. The angular value is then read from the arc.

FIGURE 2. SIGHT PICTURE WHEN ABNEY IS LEVEL, CROSS HAIR ON LEFT, MIRROR IMAGE OF BUBBLE AND BUBBLE INDEX MARK ON RIGHT.



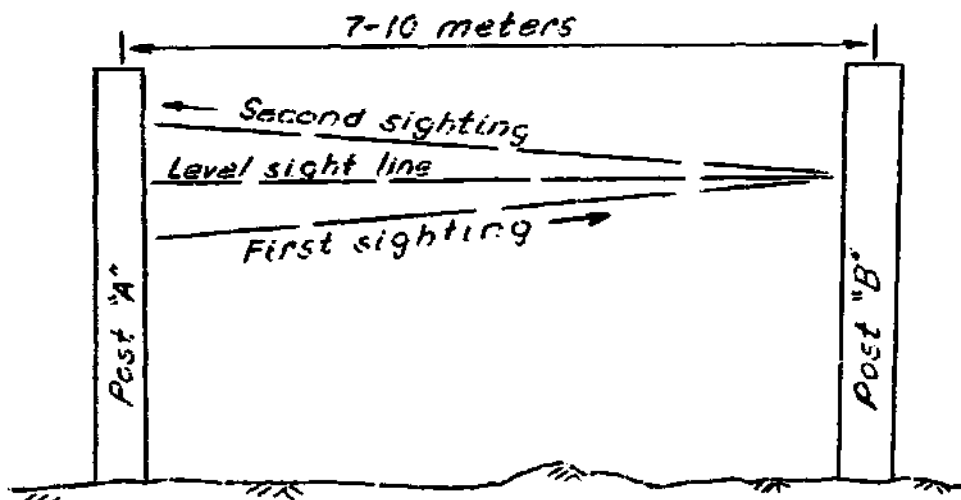
ADJUSTMENT OF THE ABNEY

Due to the peculiarities of its construction and use, the Abney is easily liable to creep out of adjustment. When used daily in the field, it should be checked frequently. No surveyor should ever begin a survey with an

* Note: This section is adapted from Peace Corps/Nepal training program information and is reprinted here in the original format. The "Note" at the end, however, was added by this writer.

unfamiliar level or one that has not been used for some time without first checking the level and adjusting if necessary. Also, if the instrument is ever dropped, it should be checked for level immediately. Adjustment of the Abney is a relatively simple task (see Fig. 3). Two posts or building corners are selected at a distance of 20 or 30 feet from each other. At post A the surveyor makes a mark at about eye level and then holds the level (the index set on zero) even with the mark. His assistant stands at post B and moves a pencil or target up and down the post until it lies in the line of sight. The surveyor then moves to post B, sets the level on the new mark and sights back to post A. The assistant sets a new mark to intersect this sight line. If the two marks on post A coincide, the sight is truly level and the instrument is in adjustment. If they do not, the assistant sets a third mark halfway between the first two. A line between this mark and the mark on post B is now truly level, and the surveyor adjusts the level bubble by turning the capstan mounting screws with an adjusting pin until the bubble remains centered when the level is held on the established level sight line.

FIGURE 3. ADJUSTING ABNEY LEVEL BY THE TWO POST METHOD



Many surveyors set up a permanent level sight line in their offices or on convenient outbuildings so that it is only the work of a moment to check the level and adjust if necessary.

USES OF THE ABNEY

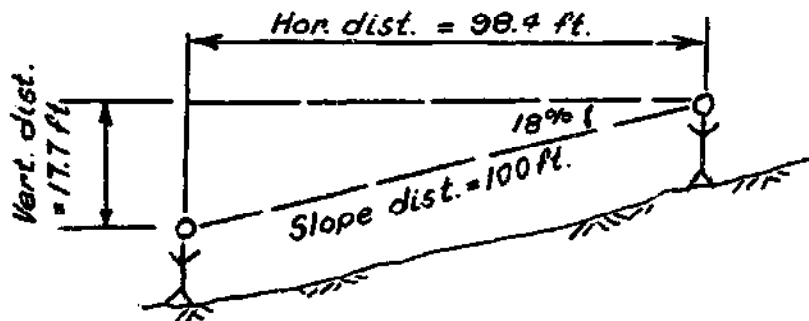
The Abney is so constructed that the plane of the bottom of the instrument is parallel with the sighting plane. With the index on zero, the level may simply be placed on any smooth surface and the bubble observed from above. While not as accurate as a 24 inch carpenter's level, the Abney is adequate for many rough leveling jobs and is particularly handy for setting short batter boards.

Many reconnaissance surveys require the rapid determination of rough differences in elevation without taking the time or trouble to achieve the degree of accuracy available through techniques of differential leveling. An example of such a survey might be a village gravity water supply system where it is necessary to determine roughly the difference in elevation between the

water source and distribution point to calculate head, and to measure the slope distance to estimate pipe requirements. By using the Abney, a steel tape and Calder's Tables, both figures may be derived simultaneously by trigonometric leveling.

Figure 4 shows the basic calculation in trigonometric leveling with the Abney. On sloping ground the surveyor and helper have measured off a slope distance of 100 feet. The surveyor then sights back through the Abney at the helper who is standing at the point just measured from. (Most surveyors simply shoot from eye to eye unless there is great disparity in their heights; the accuracy inherent in this method does not justify the greater precision implied in the use of a level rod.) The surveyor obtains a reading of 18%.

FIGURE 4. TRIGONOMETRIC LEVELING WITH ABNEY LEVEL AND TAPE.



By consulting Calder's Tables the surveyor determines that for a slope distance of 100 feet at 18%, the vertical distance is 17.7 feet and the horizontal distance is 98.4.

FIELD METHODS

Trigonometric leveling may be carried out alone or in conjunction with a compass traverse. If compass bearings are not taken, the surveyor and helper simply chain waist to waist, marking ground points by eye. The surveyor takes the Abney readings and enters the calculations in the notebook. In timber country many surveyors have the helper carry a flashlight to sight on, which reduces the need for brush cutting. On steep angles (greater than 10%) the arc is read to the nearest per cent. On flatter slopes the reading is taken to the nearest half per cent and vertical values interpolated from the tables.

When trigonometric elevations are carried in conjunction with a compass traverse, the surveyor generally takes the compass readings and makes the notebook entries while the helper takes the Abney readings and calls them back to the surveyor. Figure 5 shows a typical page of field notes for a compass traverse carrying trigonometric elevations.

**FIGURE 5. NOTES FOR A PORTION OF A COMPASS TRAVERSE
CARRYING ELEVATIONS TRIGONOMETRICALLY**

STA	BRG	SD	%	HD	VD	ELEV	RE	MAR	KS
A						1625			
	N3°W	90	-05	89.9	-4.5				
B						1620.5	PIPE	BRANCH	HERE
	S15°E	100	+21	97.9	20.6				
C						1641.1			

Notice in Figure 5 the importance of entering the sign of the Abney reading; without it one doesn't know whether to add or subtract the VD to obtain the correct elevation. Odd distances are easily computed from Calder's Tables by taking the data from the proper tens entry and obtaining and adding units and decimals (if any) by shifting decimal points.

NOTE:

When Calder's Tables are not available compute elevation changes as follows:

1. For vertical angle in degrees -
Change in elevation = slope distance X sine of vertical angle
Horizontal distance = slope distance X cosine of vertical angle
2. For vertical angle in per cent -
The angle measure in per cent is the tangent of the vertical angle multiplied by 100. Therefore, find the tangent by dividing "per cent" by 100. Look through the trigonometric function tables under the "tangent" column and find the angle in degrees whose tangent has just been computed. Now, knowing the angle in degrees, proceed as described in Step 1.

3. DESIGN PERIOD, POPULATION, AND WATER DEMANDS

=====

DESIGN PERIOD

Community water supply systems should be designed and constructed for a 15 to 25 year useful life span. The choice of either a 15, 20, or 25 year design period should be based on the amount of potential change that the designer foresees for the area. 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 is slated for construction in 12 years, a shorter period of 15 years should be adopted because longer period water demands can not be accurately forecasted. Careful consideration should be given to the choice of design period, and advice from RA & LDD engineers of the regional office sought.

POPULATION FORECAST

Selection of a design period leads directly to an estimate of the design population to be served. The chart on the following page (Figure 6) will serve as a helpful guide in predicting population trends for the different geographical regions of Nepal.

Within the design report, the design period and population forecast should be explicitly noted, as should be the criteria for their selection.

WATER DEMANDS

The total water demand for the village at the end of the design period is made up of the per capita demand and special demands. A per capita water usage of 45 liters per day, per person, is the present design standard. This figure derives from WHO studies. It includes allowances for personal washing, drinking, cooking, and livestock and domesticated animals' water needs.

When a marginal water supply source is encountered, and the target figure of 45-LPCPD can not be supplied, one may go as low as 230 liters per household per day. This figure is based on minimal needs and assumes 8-10 persons per household.

The per capita water demand, therefore, will be found by using the projected population for the design period and either the 45-LPCPD standard, if possible, or as low as the 230 liters per household per day.

The special water demands include projected requirements for schools, health centers, public institutions, government facilities, etc. Determination of the projected water needs of these special users can best be made by extrapolating past trends for each individual special use. Small villages will usually not have these special water demands. However, in the case where there does exist such a special demand, it may constitute a significant portion of the total water requirement. Consequently, an investigation of future expansion plans or other changes in the special use of water is

important for the integrity of the design.

The village's total daily water requirement will be the sum of the per capita demand, multiplied by the design year population, and the special water demands anticipated for the design year. Reservoir design, hydraulic design, and public tapstand placement will be based on this value for total daily water use and on water uses in different sections of the village.

FIGURE 6. RURAL WATER SUPPLY DESIGN
POPULATION GUIDE FOR NEPAL

GEOGRAPHIC AREA	1961-1971 Average Annual Growth Rate	PERCENTAGE INCREASE			
		10-Yr.	15-Yr.	20-Yr.	25-Yr.
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 1952-4, 1961, and 1971 Census data. 10-25 year growth figures based on 1961-1971 average annual growth rates computed by the author.

Note: The WHO consultant for RA & LDD suggests that a value of 1.75% average annual growth be used for all hill village water system designs.

4. OPEN SYSTEMS VS. CLOSED SYSTEMS OF SUPPLY

=====

An open system is defined here as a water supply system in which there are manually operated faucets on the taps and water flows continuously. (This definition does not exclude regulating valves used to proportion flows between taps). A closed system has self-closing or manually operated faucets on taps. Closed systems usually include a reservoir, and if topography or elevation changes warrant, one or more break pressure tanks. An open system may include break pressure tanks, but never a reservoir. A water supply system can be completely open, completely closed, or part open and part closed. Advantages and disadvantages exist for each type of system.

The open system has an immediate economic advantage over a closed system in that reservoir tank construction is avoided and open taps are less expensive than taps with shut-off valves. The water in an open system is always moving, hence such a system should not freeze, and the short detention time of the water lessens the chance for the growth and accumulation of contaminating microorganisms.

In order to design an open system there must be a sufficient flow of source water. If the minimum flow from the source supplies the peak period water demand flow rate then an open system can be designed. An open system can also be designed if the source flow is large enough to supply each tap with a flow of 13.5 liters per minute. Either criteria may be used, but the latter is best suited for situations involving 200 or fewer residents per tap.

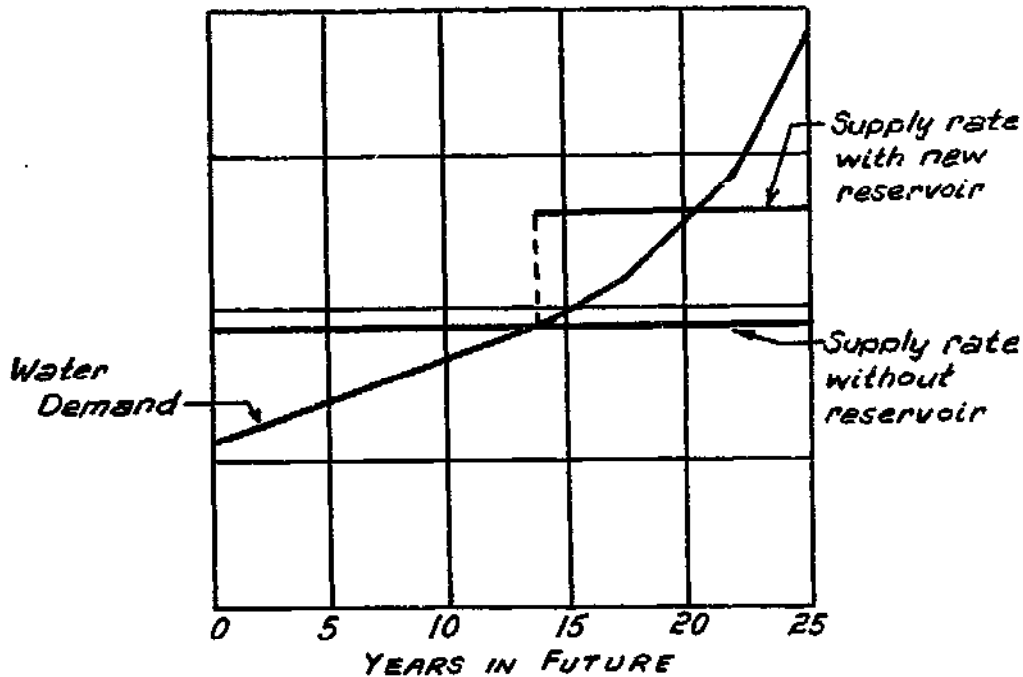
The largest single disadvantage of an open system is the problem of dealing with the copious amounts of wastewater issuing continually from the taps. Strategic location of taps to make efficient use of waste flows, and construction of non-erosible drainage channels to carry these flows will minimize the problems of the large wastewater quantities.

A closed system with a reservoir tank has some inherent advantages. First, the stored supply of water is useful in the event of a fire in the village. Second, the reservoir can be used for water treatment processes like sedimentation and disinfection. (Disinfection of rural water supplies in Nepal may become feasible in the future). A final consideration is that a reservoir tank is considered a status symbol in many Nepali villages.

One possible design approach is to construct an open system for a 15-year design period in anticipation of closing the system and constructing a reservoir tank at the end of 15 years to meet increased water demands. If the source provides enough water for a 15-year design population peak demand,

then such an approach might prove to be economical. The following example discusses some aspects of future, phased construction.

FIGURE 7. AN EXAMPLE OF PHASED CONSTRUCTION ECONOMICS



The graph shows the growth in water demand reflecting the village's population growth. Compare the construction cost of a complete system with reservoir now to the cost of a system without a reservoir at present, but adding the reservoir in 15 years. In remote areas of Nepal transportation costs are usually relatively excessive. The economic comparison should account for any future transportation cost reductions resulting from new road construction. Another factor which will tend to make future construction more attractive is the increasing availability of new fabrication materials. PVC brick and sheeting and fiberglass reinforced plastic (FRP) are likely to be more economical materials for remote hill areas in coming years.

5. INTAKE WORKS

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INTRODUCTION

It is not possible to present a standard design for an intake system because of the variety of sources and source geologies encountered in the rural water supply program. Consequently, this section will describe standard features and design criteria for intake works which will serve as a guide for the design of the structures.

STANDARD FEATURES

Most intake tanks will include the following features:

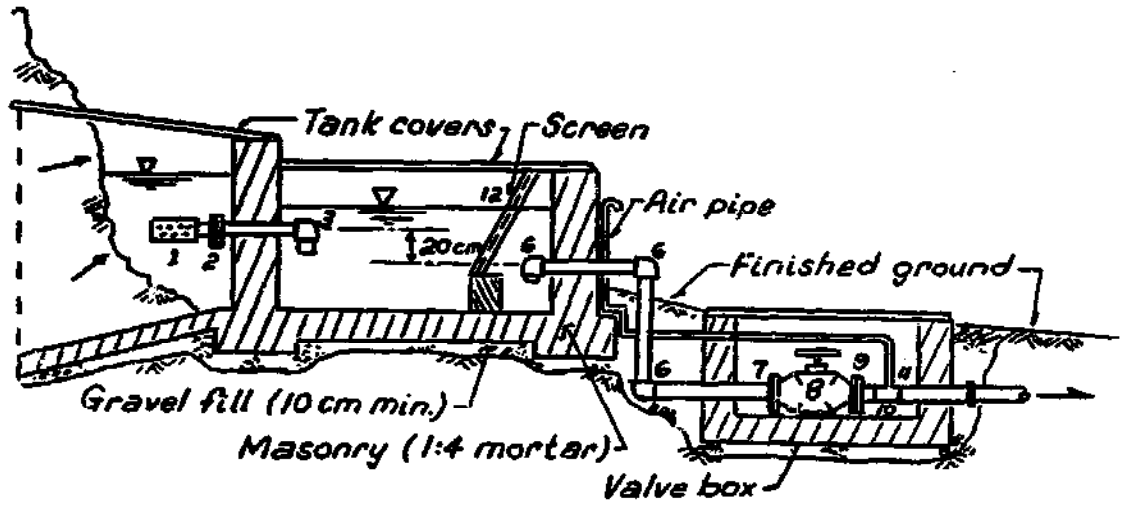
- Screened inflow piping
- Service pipe(s) (i.e., outflow pipe(s))
- Overflow piping
- Ventilation of tank
- Screens to protect service pipe(s)
- Shut-off regulating valve(s) on service pipe(s)
- Sedimentation compartment
- Locking accessway; locking valve box
- Fencing around source

In addition, a flow measuring device may be incorporated with the intake system.

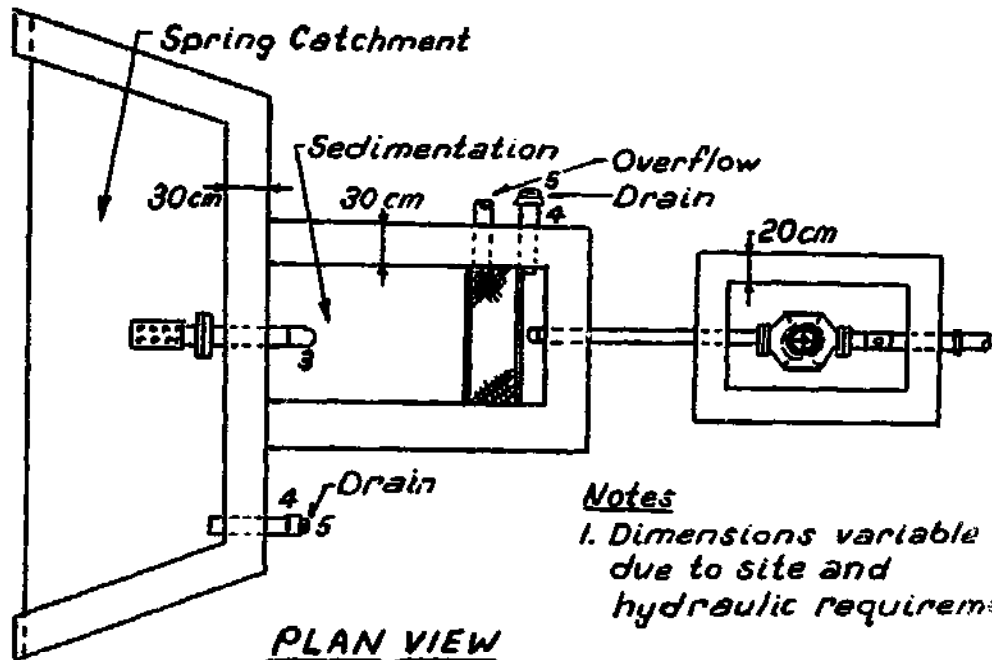
A design and estimate for one type of intake tank (designed for a spring source) is presented on the next two pages, Figures 8 and 9. It has most of the above mentioned features. This design and estimate was made by engineers at RA & LDD for their water supply program.

A simpler, alternative design is presented in Supplement A, page 96. This single compartment tank was proposed by the World Health Organization advisor for the RA & LDD as a more economic alternative. It features less mortared masonry, natural impervious formation floor, and simpler piping.

FIGURE 8. SPRING INTAKE SYSTEM



ELEVATION VIEW



PLAN VIEW

Notes

1. Dimensions variable due to site and hydraulic requirements.
2. Numbered fittings are listed on the following page.

FIGURE 9. ESTIMATE FOR SPRING INTAKE SYSTEM

List of Materials and Labor

1. excavation	appr. 20 m ³ =	700 cft	coolies	16 nos
2. stone-and gravel soling	2.20 m ³ =	77 cft		
	gravel	45 cft	mason	0.8 nos
	stone	45 cft	cooly	3.8 nos
3. cement concrete 1:2:4	2.78 m ³ =	100 cft	mason	3 nos
	sand	44 cft	cooly	12 nos
	gravel	88 cft	cement	22 cft
4. stone masonry 1:4	3.88 m ³ =	138 cft	mason	5.5 nos
	sand	50 cft	cooly	12.4 nos
	stone	166 cft	cement	12.4 cft
5. cement plaster ½" 1:3 (inside only)	12.5 m ² =	134 cft	mason	1.3 nos
	cement	2.7 cft	cooly	2.7 nos
	sand	8 cft		
6. roofing	slates	7 m ²	mason	2 nos
	wood	3.5 cft	carpenter	3 nos
	nails		cooly	8 nos
7. installing of fittings			mistry	1 no
			cooly	1 no

TOTALS:	cement	37.1 cft	mason	12.6 nos
	sand	102.- cft	carpenter	3 nos
	gravel	133.- cft	mistry	1 no
	stone	211.- cft	cooly	55.9 nos
	slates	7 m ²		
	wood	3.5 cft		

List of Fittings

The appropriate diameter depends on the hydraulical calculation. The perforated intake should be ½" larger than the continuing mainline of Ø x"

1. perf. int. flanged	x+½"	1 no	2. GI flange & acces.	x+½"	1 no
3. GI elbow	x+½"	1 no	4. GI socket	x+½"	2 nos
5. GI plug	x+½"	2 nos	6. GI elbow	x"	3 nos
7. GI flange	x"	1 no	8. Gate valve	x"	1 no
9. HDP flange & acc.	x"	1 no	10. HDP Tee equal	x"	1 no
11. HDP reducer	x"/x-1"	1 no	12. screen in wooden frame		2 m ²

FUNCTION OF INTAKE STRUCTURE COMPONENTS

The purpose of the sedimentation section in the above mentioned tank and in any intake tank is to allow the water to stand relatively still for a while so that heavy particles such as silt, sand, and stones will drop to the tank bottom. When taking water from a stream source these compartments will tend to fill with sediment quickly -- especially during the rainy season when the water will be carrying a heavier sediment load. Therefore, the sediment compartment should always be easily accessible to facilitate cleaning.

A short wall should separate the sediment compartment from the tank section containing the screens and outflow pipe. The outflow pipe should be placed 15 cm above the floor as some sediment will be carried into this section. Sediment should be cleaned from this section whenever it approaches within 5 cm of the outlet pipe.

Every intake tank has to be drained for cleaning, so a washout pipe, placed at floor level, should be installed. Pitch the floor at a minimum slope of 1:40 toward the pipe to facilitate drainage. Both the overflow pipe and washout pipe should be extended to a suitable location (for instance back into the stream) so that the overflows and drainage do not erode the area near the tank. A well-lubricated cap or plug on the end of the washout pipe will be sufficient if a gate valve is not available. Washout pipes should be 90 mm in diameter because of the sediment and debris which must typically pass through them.

Because the intake tank receives the initial flow of source water, two screens (coarse and fine) need to be installed to protect the rest of the downstream system. The coarse screen catches pebbles, sticks, etc. carried in heavy rains and protects the fine screen which stops the passage of smaller materials that would tend to settle in pipes and clog valves. The selection of screening material will, most likely, be severely limited. Brass, or other non-corroding screen should be used for the fine screen. A good substitute for a coarse screen is a rack built from wood. Openings should be 1 to 2 cm wide in such a rack, and the wooden bars 2 cm in diameter or 2 cm square.

An accessway in the intake tank roof should be installed so that depth of sediment and the conditions of the screens can be inspected, and so that sediment and screenings can be removed.

Intake tanks should be as small and shallow as possible. (deep tanks do not enhance sedimentation, as settling is proportional to water surface area.) If, due to site conditions or other design considerations, the tank must be deep or large, construct stone and mortar walls (1:4 cement mortar) to the dimensions described below:

- a. First 30 cm below water surface: walls 30 cm thick
- b. Next 100 cm: walls 45 cm thick
- c. Below 130 cm deep: walls 60 cm thick

Waterproofing for larger intake tanks should consist of two 0.5 cm layers of cement plaster, or two coats of sealant.

A schematic representation of an intake tank showing essential features and

their relative locations is presented in Figure 10. This drawing summarizes much of the previous discussion.

THE INTAKE TANK SITE

It is very important to locate the intake tank at a site that is geologically stable and that is protected from the ravages of monsoon flooding. A stream source is likely to be severely flooded in rainy periods, so the highest limit of flooding must be established (by observation and/or consultation with villagers on record high flood stages). Do not build the intake tank in this "flood plain"; instead seek a sheltered location away from the area, and divert water to the tank in an open channel. If the stream has a protected, year-round pool the simple arrangement pictured on the subsequent page (Figure 11) will make an intake tank unnecessary. The minimum depth of water in such a sheltered pool should be sufficient to provide protection and cover for this submerged intake structure. At least 40 cm of water depth should always cover the structure in order to deter vandals and to protect the structure from floating debris. A stream can be dammed to make this type of pool or to increase the depth in an existing pool.

It is always best to avoid streams as water sources because of the potential dangers of flooding and water contamination. Springs are the best sources, and when available they should be tapped in lieu of a stream source. However, springs are not completely free of problems. They, too, have increased monsoon flows which must be accommodated; they can move over a period of time; and they can become blocked by sedimentation.

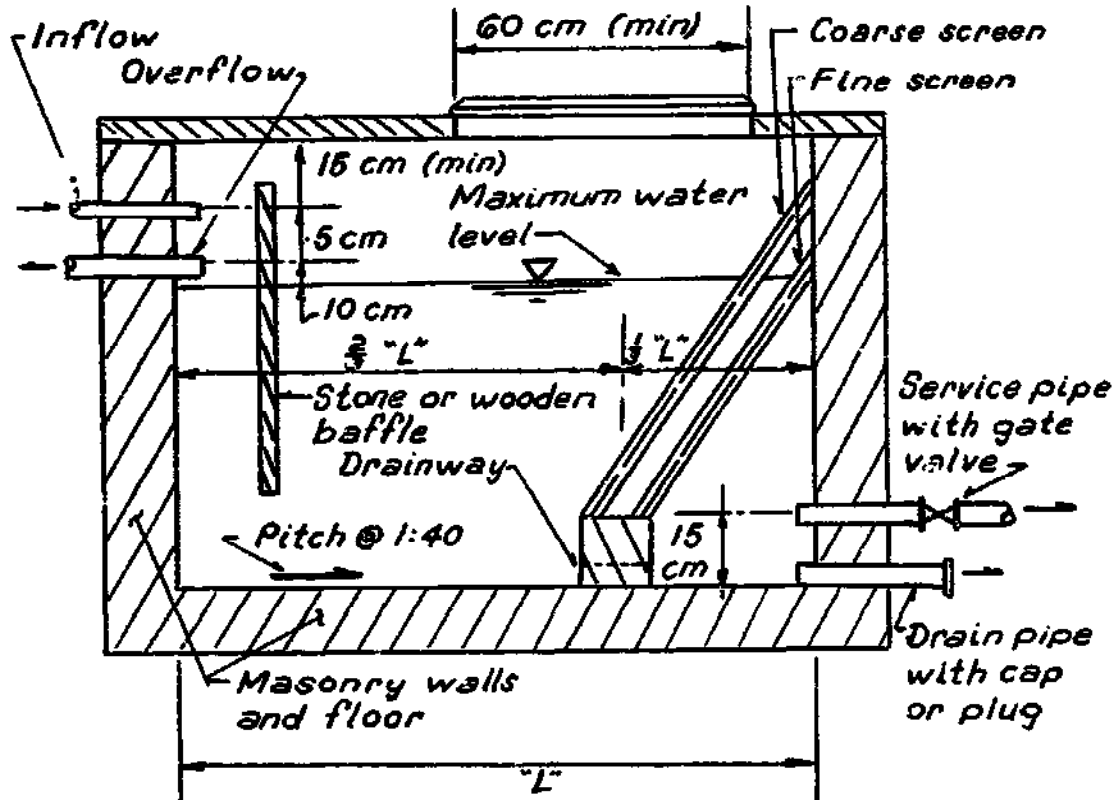
The intake structure for a spring source should overflow all flows in excess of the safe yield, i.e. minimum flow expected in an average year. This will insure two things: first, monsoon flows will pass around the structure without harming it; and second, the pipeline will always carry the design flow.

The tank overflow system must be of sufficient hydraulic and structural capacity to safely pass large flows. Thus a good estimate of peak source flow should be obtained.

A SPECIAL PROBLEM OF SHORT TERM MINIMUM FLOW

A situation may occur where the safe yield of the source is a small dry season flow which falls below the desired per capita supply. If this is so, and the designer wants to supply more water than the safe yield for most of the year (when there is more water available), then remedial steps can be taken to prevent low flow problems. The danger in sizing a gravity pipeline for a flow larger than the safe yield is that it will not flow full in a low flow period. Since the pipe should be designed for full flow, a partially full pipe may lead to such problems as clogging by sedimentation in the pipe, creating negative pressure reaches which may inhibit flow, or excessive air entrainment. These problems can be mitigated by installing a regulating valve downstream from the intake tank. The regulating valve should be placed far enough down the pipeline such that there are no high points in the pipeline below it, yet it must be upstream of a tank.

FIGURE 10. FEATURES OF A TYPICAL INTAKE TANK



Note: Provide ventilation pipe or vent holes in masonry at top of wall.

Baffle and screen location at $\frac{2}{3}L$ will prevent heavy debris from gathering on the screen.

Pipes are shown for relative elevation, only; place in other walls as site conditions may dictate.

FIGURE 11. DEEP STREAM INTAKE WORKS

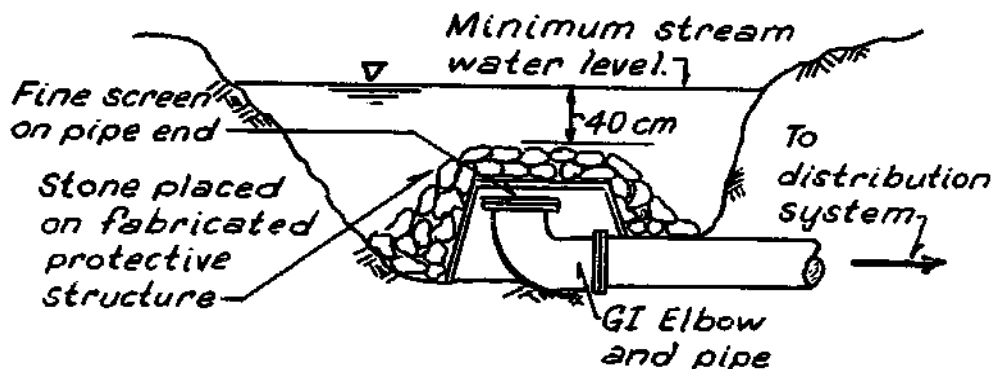
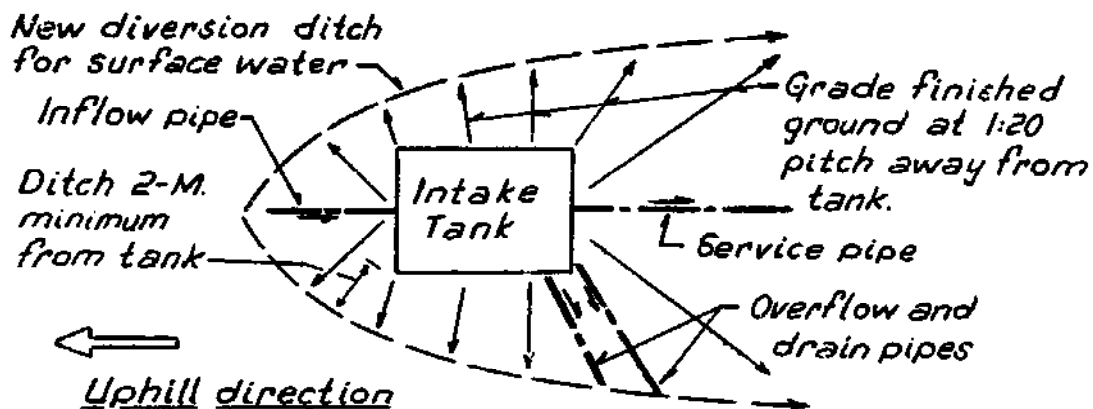


FIGURE 12. FINISHED GRADING FOR A HILLSIDE INTAKE TANK



By closing the valve slightly the capacity of the pipe upstream of it will be reduced and a lower source flow can still keep the pipe flowing full. Thus, in the wet season and in the months preceding the dry season, the valve will be opened fully to permit the flow of more water, while during the low flow period it is closed partially. The villagers will receive less water at this time, but the hydraulic integrity of the system will be preserved.

There are disadvantages inherent in this scheme. Not only are there limiting hydraulic constraints, but the village must have a responsible person who will understand the principles behind adjusting the valve seasonally. Other alternatives to strict safe yield design procedure are discussed in Appendix C.

SANITARY PRECAUTIONS FOR INTAKE WORKS DESIGN

The last set of criteria for designing an intake system involve sanitation. Given that a clean, potable supply of water has been found, the objective is to maintain it in this condition. This will involve protection from human and animal contamination and from polluting rainwater surface runoff.

A spring should be completely enclosed in a masonry structure and locked. Rainwater surface runoff should be diverted away from it with a ditch and grading as shown in Figure 12, the preceding illustration. This method of finished grading will keep the pollutants carried by the surface water from ponding near the tank and subsequently seeping into the system. Stabilize the finished ground and ditch with grass cover, stone paving, or other means to prevent erosion.

The intake tank for a stream source should be protected just like that for a spring source, except there is additional fencing required. The intake from the stream should be from an area which people and domesticated animals and livestock do not frequent, and there should be no houses or grazing lands upstream. Fencing off the immediate area and blocking access to the reach of stream above the intake should be done as best as possible. (It may be possible to work with the Forestry Department in reforesting and fencing this area). The closer the intake can be placed to the stream source the better. This location will not only minimize potential contamination, but will also receive smaller monsoon flood damage.

INTAKE TANK MAINTENANCE

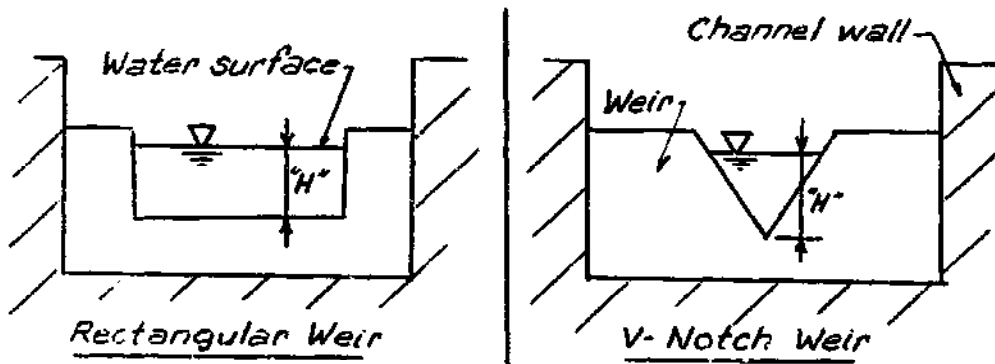
The intake tank will require routine maintenance: screen cleaning, sediment and debris removal, yearly pipe painting, and three times yearly valve lubrication. Screens should be checked weekly during the rainy season as most debris will enter at this time. Failure to clean the screens will result in clogging and mitigated water flow at the least, and screen destruction and pipe blockage with consequent total flow stoppage at the worst. Careful design of the intake tank will minimize maintenance work and the need for extensive repairs. An intake structure that is designed for easy cleaning will be kept cleaner by the village maintenance worker, and will consequently give better service than a structure that involves much effort for cleaning.

FLOW MEASURING DEVICES

Flow measuring devices were mentioned in the initial paragraph on intake systems as an optional feature. Though a village water system would only rarely require a device which measures the source flow, there are instances when metering may be desirable.

The easiest and simplest devices to design and construct for flow measuring are weirs and flumes. A weir, for instance, is simply a board with a rectangular or notch opening (as shown), while a flume is a contracting box-like structure.

FIGURE 13. WEIRS

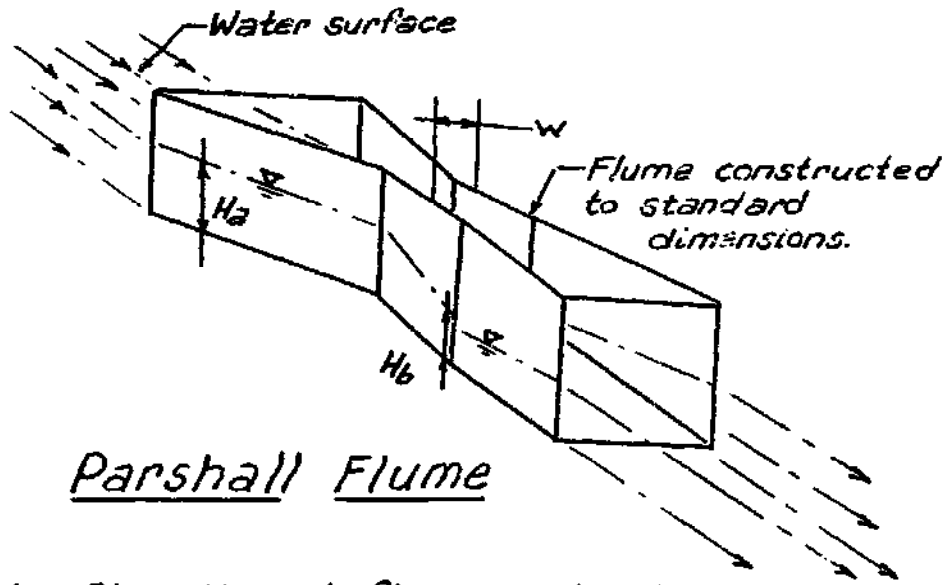


These devices are placed in a channel of designed dimension, and knowing the dimensions of the weir (length or angle), the flow of water can quickly be found by measuring "H", the depth of flow over the weir.

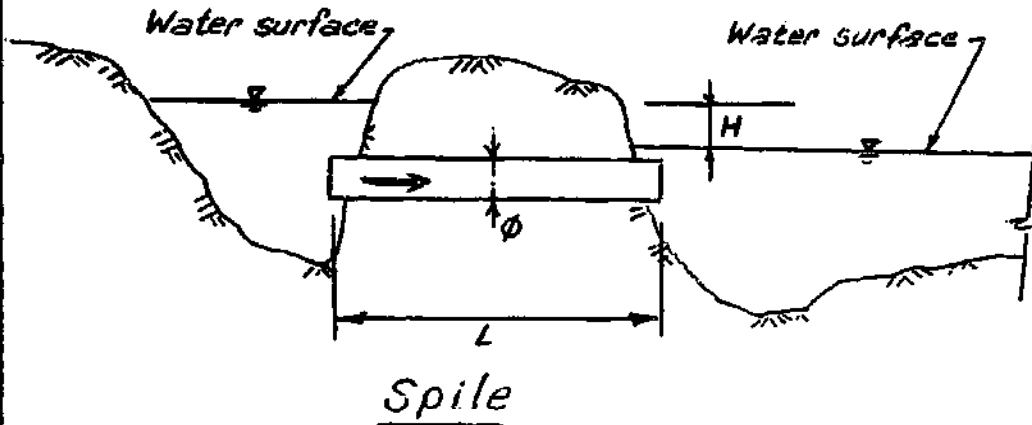
Such devices are of negligible cost and should be considered for water systems in more developed regions where information on flows can be collected and used in planning for future development.

Design information on flow measuring devices can be found in books on irrigation, hydraulics, and in some of the references cited at the end of this manual (See, for example, References # 3,6,12,13, and 17). This topic has been included here to acquaint the reader with the simplicity and usefulness of flow meters.

FIGURE 14. OTHER SIMPLE FLOW METERS



Note: Flow through flumes and spiles is a function of the parameters shown in the sketches.



Note that the drawings are not to scale.

6. PIPELINE DESIGN

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INTRODUCTION

This section includes four major topics: a) Procedure for Constructing a Hydraulic Gradient, b) Pipe System Design, c) Design for Branching Pipes, and d) Standards for Construction of Pipelines and Installation of Valves.

Most of the information presented in this section is based on the assumption that HDP (high density polyethylene) or PVC (polyvinylchloride) pipe of small diameter will be used in long reaches and GI (galvanized iron) pipe used only near structures. The design procedure in the second part is specifically written with respect to gravity systems and is therefore limited in its scope of applications.

PROCEDURE FOR CONSTRUCTING THE HYDRAULIC GRADIENT

The hydraulic grade line (HGL) is a representation of the height (above a given datum) to which water flowing in a conduit would rise were it not constricted by a closed conduit. If small open-ended tubes are inserted in a pipeline, the height of the water's rise in these tubes defines the HGL as shown in Figure 15. Notice that the height to which water will rise in these small tubes defines the headloss due to friction of the flowing water through pipes and valves. Water which is subjected only to atmospheric pressure will not rise in an open tube; therefore, the HGL in this situation is defined by the water surface. In tanks, open channels, and pipes not flowing full, the HGL is the surface of the flowing water.

Figure 16 on the bottom of the illustration and the following rules for HGL determination are offered as guides to help in constructing an HGL for any particular pipeline system.

1. In a gravity flow water system the HGL always falls in the direction of the flowing water.*
2. The HGL is coincident with the water surface in an open tank, a pipe not flowing full, and an open channel. It is at the centerline of a jet of water discharging freely into air.
3. In a pipe of constant size and friction factor, flowing full with a constant rate of flow, the HGL falls at a slope equal to the headloss per unit length for the flow condition.

* Neglecting minor change in sudden expansion which is very small compared to overall change in a long pipeline.

FIGURE 15. PHYSICAL INTERPRETATION OF THE HYDRAULIC GRADE LINE

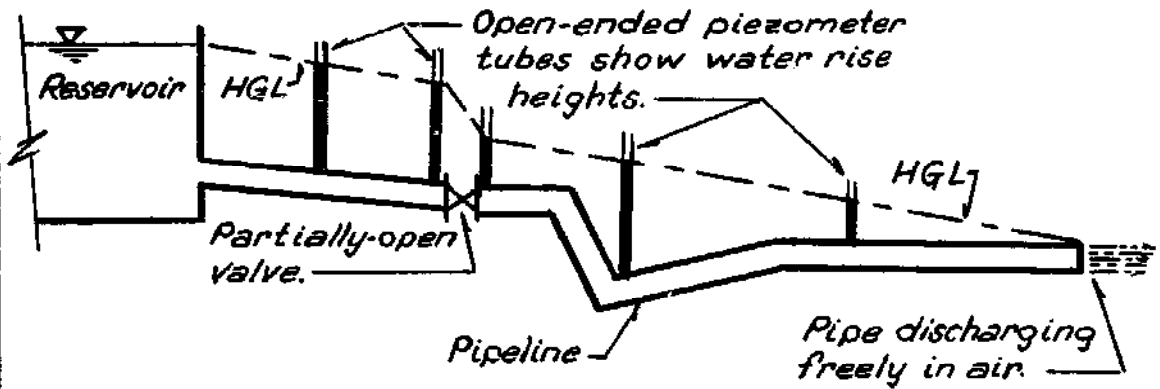
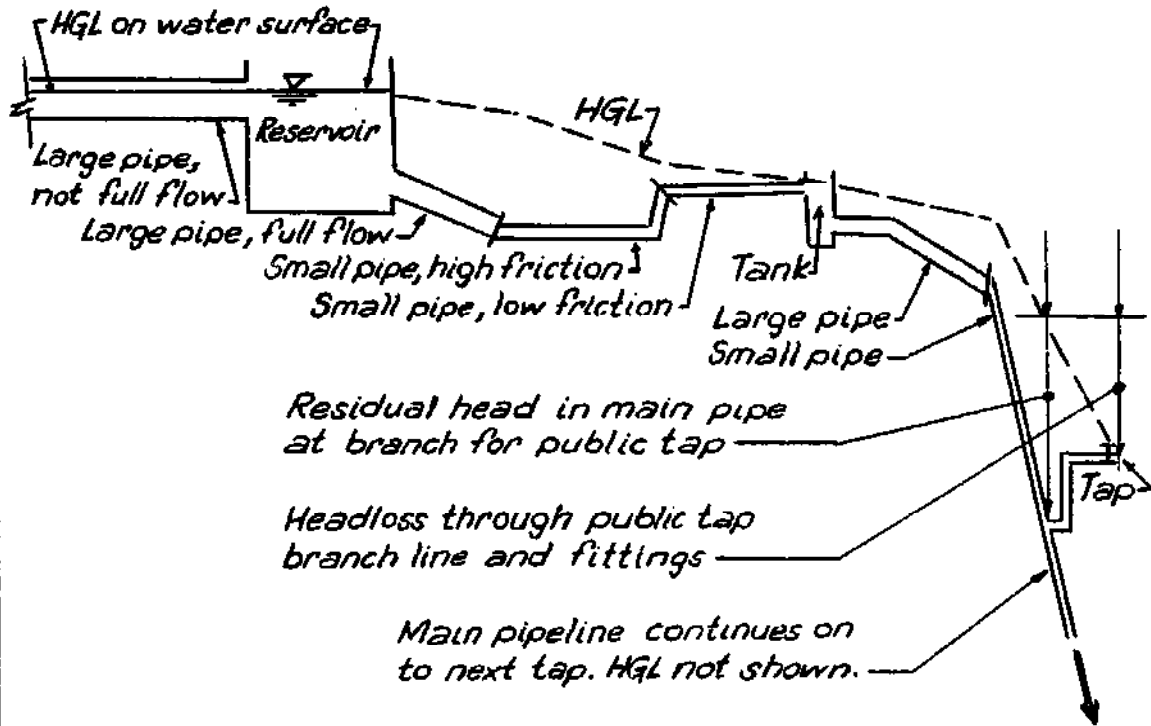


FIGURE 16. HYDRAULIC GRADE LINE IN A COMPLEX PIPELINE SYSTEM



4. The HGL will change slope under the following conditions:
 - a. Change in flow
 - b. Change in pipe size
 - c. Change in friction factor

5. When the pipeline length is greater than 1000 pipe size diameters, then minor headlosses in fittings, valves, and at entrances and exits become negligible in comparison to friction loss through the main length of pipe. Consequently, these minor losses would not have to be shown on the HGL profile. For the common sizes of pipe encountered in rural water supply work, the following limits for neglecting minor losses are presented:

15 mm pipe: greater than 13 meters, neglect minor losses
32 mm pipe: greater than 30 meters, neglect minor losses
90 mm pipe: greater than 80 meters, neglect minor losses

(Note: Values for minor losses are presented in Figure 17 in this section on hydraulic design).

6. In a pipeline in which water is not flowing (static condition) the HGL is a horizontal line from the nearest upstream free water surface.

In designing a pipeline it is very important that the HGL never falls below the grade of the pipe. When such a condition does occur it means that the water in the pipe must rise higher than it would if subjected to atmospheric pressure. This creates a negative pressure in the portion of pipe above the HGL. Needless to say, the occurrence of a negative pressure will impede, or completely interrupt the flow of water.

Refer to the appended TECHNICAL MEMORANDUM (Appendix C) for further information on the hydraulic gradient.

PIPE SYSTEM DESIGN

The procedure for designing a pipe system can be very mechanical, because in simple, gravity systems design simply involves matching required residual heads and friction losses to existing topographic conditions. It is assumed that the reader has had exposure to pipeline design, so this section will merely review and discuss the use of the design aids which are presented.

Pipe lines are designed from reach to reach. A reach extends between two tanks, a tank and a branch, or a branch and a tap, etc. The energy to move the water derives from gravity and is represented by the static head, h_s , the elevation change between the start and end of the reach.

As water flows through a pipe energy is lost in frictional resistance. Energy is also lost as water passes through valves, fittings, and tanks; these energy losses are termed "minor losses". When a reach of pipeline is shorter than 1000 pipe diameters, minor losses must be included in the headloss computation.

The following example illustrates the calculation of minor losses in a pipeline. Refer to Figure 17 listing "equivalent length" headlosses for various types of fittings. Consider a 50 mm diameter pipe which is 30 meters long. Note that 30 meters is less than 1000 pipe diameters, which would be 50 meters. Say that within the reach there are two right angle bends and one gate valve. The total "equivalent length", which accounts for energy loss in the pipe and the fittings and valve, is:

30.0 meters	Pipe Length	-	50 mm diameter
1.8 meters	90° Bend	-	50 mm diameter
1.8 meters	90° Bend	-	50 mm diameter
0.3 meters	50 mm Gate Valve		
<u>33.9 meters</u>	Total		

The total frictional headloss through the reach of pipe is found by using this length of 33.9 meters, the pipe diameter of 50 mm, and the flow, Q, carried in the reach. The pipe diameter and Q determine a unique value of headloss which is found in a table, in a graph, or in nomograph. Graphs and nomographs are rapidly used and of sufficient accuracy for rural water system design. Examples of a graph and a nomograph for determining friction headloss are presented on the subsequent pages. Note carefully the units of frictional resistance for each design aid.

The working head, h_w , is the sum of the static head, h_s , and the pressure head at the top of the reach. Subtracting the desired residual head at the downstream end of the reach under design yields the amount of head available for friction loss. Using standard sizes of pipe and strategically locating break pressure and reservoir tanks each reach is designed in turn. Where trial and error designs are necessary the nomograph saves time and calculations. (Refer to appended design examples for a step-by-step design procedure.)

VELOCITY AND PRESSURE CONSIDERATIONS

There are velocity and pressure constraints on pipeline design which are not explicitly accounted for in the above mentioned procedure. The velocity constraint can add complications in design because headloss and pressure may be suitable when water velocity is too high or too low. Ideally the velocity of the water in the pipe should be within the range of 0.7 to 3.0 meters per second. The minimum value precludes sedimentation of material in the pipe. The maximum velocity limit reduces abrasion and wear on the pipe wall due to high friction. If the water velocity is considerably in violation of these limits, then a new pipe size should be selected.

Every type of pipe has a maximum working pressure which must not be exceeded in design. Class III HDP is rated for 6 Kg/cm² while Class IV HDP has a working pressure of 10 Kg/cm². PVC and GI pipe have higher working pressures, and the designer must consult the manufacturer's product literature to ascertain the pressure rating for the particular pipe.

A special pressure problem common to many village systems in Nepal is that resulting from a "U" shaped pipeline. Such a line is illustrated on the

FIGURE 17. RESISTANCE OF COMMON VALVES AND FITTINGS

Pipe diameter (mm)	90° Bend	45° Bend	Tee	Gate valve (open)	Corporation cock (open)
15	0.5	0.3	1.1	0.1	2.6
25	0.9	0.4	1.8	0.2	4.7
40	1.4	0.6	2.7	0.3	6.8
50	1.8	0.8	3.4	0.4	8.7
65	2.1	1.0	4.4	0.4	10.3
75	2.6	1.2	5.3	0.5	13.1

Notes

1. Values tabulated for valves and fittings are in meters of the respective pipe size.
2. Numerical values in this table are interpolated for metric dimension pipe from test data presented in Crane Company Technical Paper # 409 (pp. 20, 21) as reprinted in Reference # 23 (p.81).

FIGURE 18. FRICTIONAL HEADLOSS GRAPH
FOR POLYETHYLENE PIPE

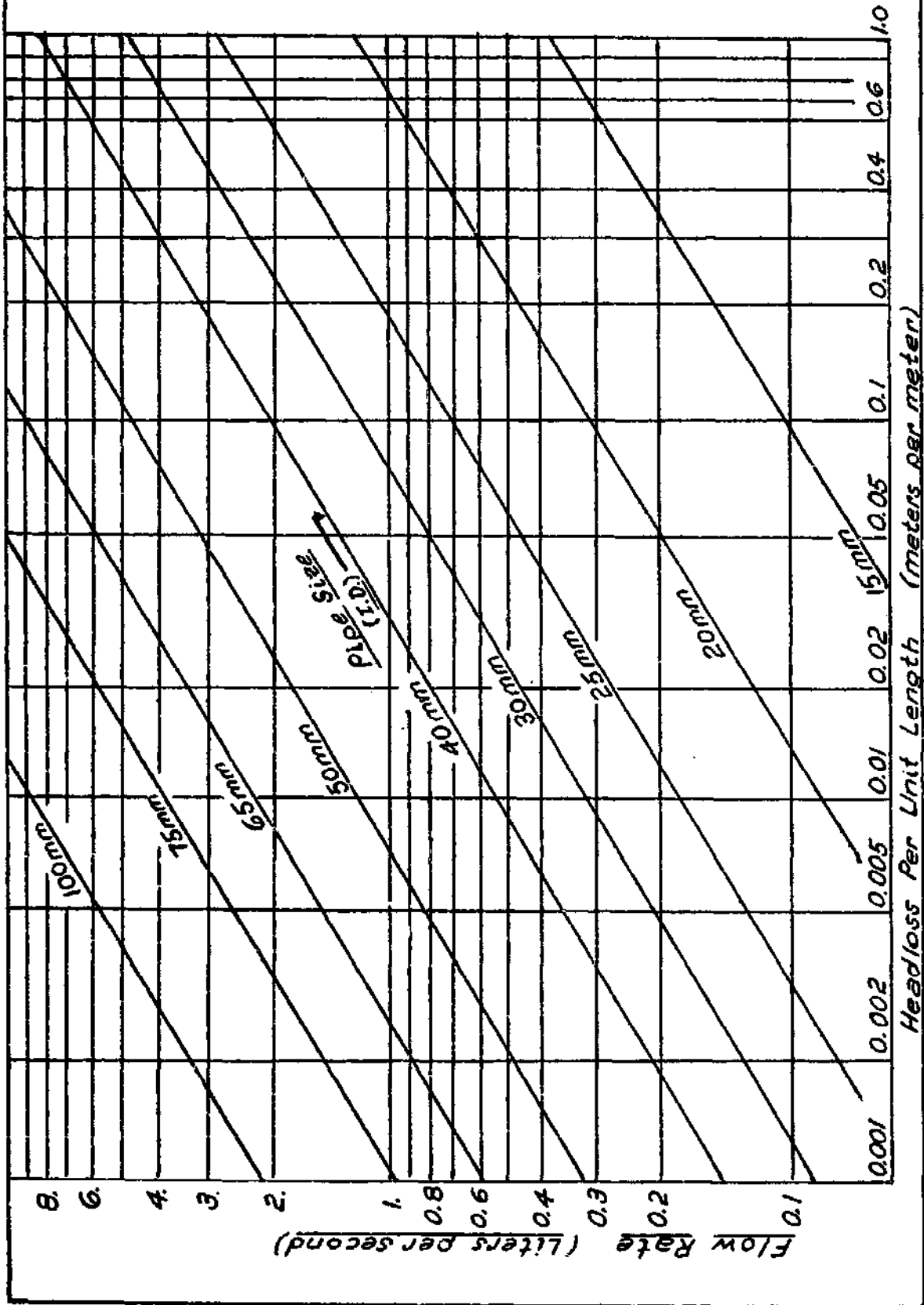


FIGURE 19. NOMOGRAPH FOR PIPE DESIGN

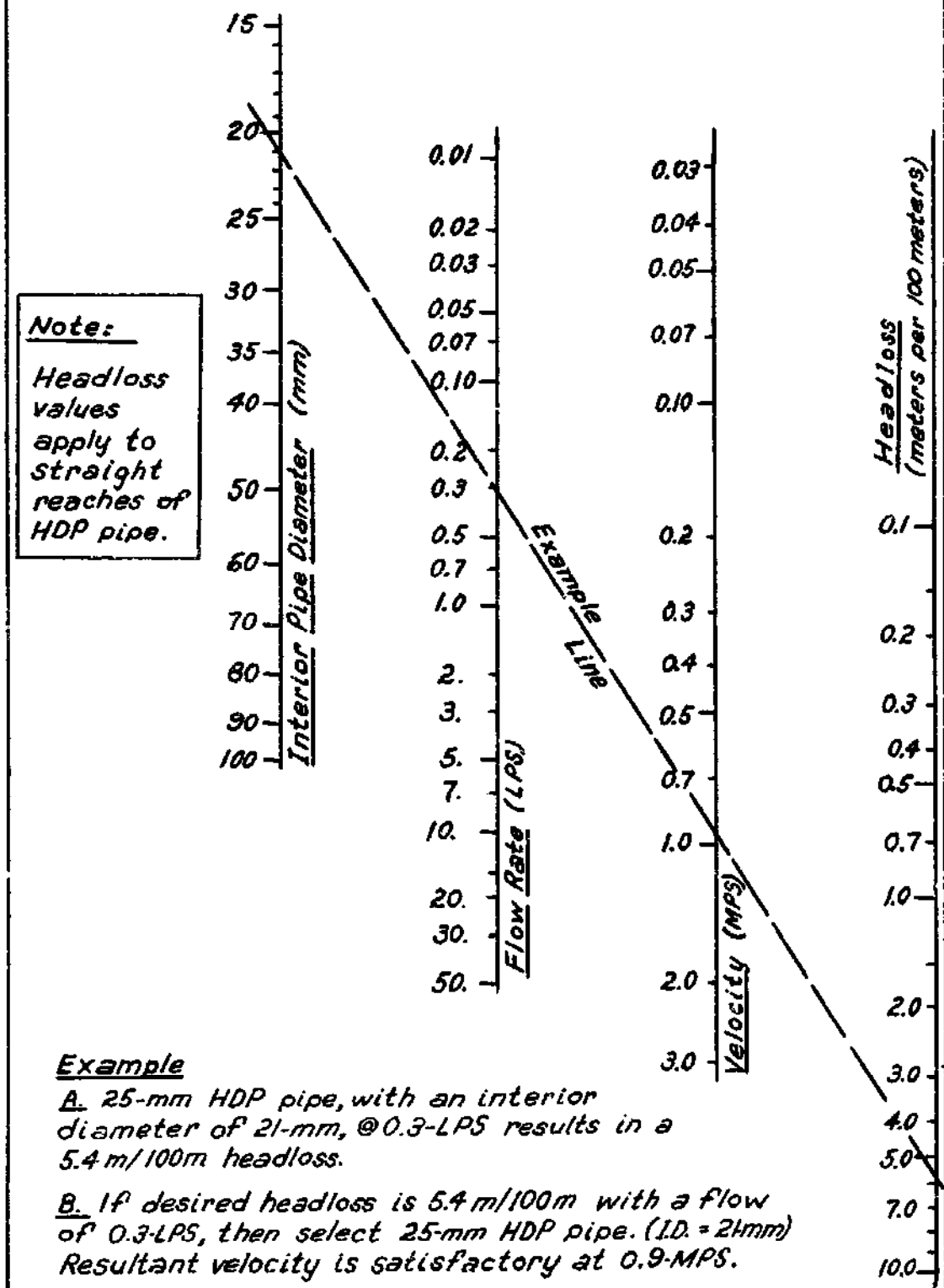


FIGURE 20. FRICTIONAL HEADLOSS TABLE FOR HDP PIPE

<u>FLOW</u>					<u>FLOW</u>				
<u>(LPS)</u>	<u>PIPE SIZES</u>				<u>(LPS)</u>	<u>PIPE SIZES</u>			
	<u>20x2</u>	<u>25x2</u>	<u>32x2</u>	<u>50x2</u>		<u>63x3.6</u>	<u>75x4.3</u>	<u>90x8.2</u>	
0.03	0.30				0.50	0.125	0.053		
0.04	0.52				0.55	0.147	0.060		
0.05	0.80	0.21			0.60	0.171	0.073		
0.06	1.09	0.30			0.65	0.195	0.088		
0.07	1.40	0.39			0.70	0.220	0.095	0.040	
0.08	1.81	0.50			0.80	0.282	0.118	0.051	
0.09	2.29	0.62			0.90	0.348	0.154	0.062	
0.10	2.80	0.76	0.19		1.00	0.410	0.180	0.075	
0.12		1.06	0.27		1.30	0.678	0.268	0.122	
0.14		1.40	0.35		1.50	0.855	0.374	0.156	
0.16		1.78	0.44		1.70	1.055	0.501	0.192	
0.18		2.18	0.54		2.00	1.40	0.610	0.255	
0.20		2.60	0.65	0.080	2.30	1.81	0.788	0.329	
0.22		3.10	0.78	0.094	2.50	2.10	0.910	0.380	
0.24		3.62	0.91	0.107	2.70	2.40	1.035	0.432	
0.26		4.20	1.05	0.122	3.00	2.90	1.25	0.520	
0.28		4.78	1.19	0.136	3.50	3.84	1.67	0.691	
0.30		5.40	1.35	0.150	4.00	4.88	2.14	0.878	
0.35		7.12	1.77	0.198	4.50	6.00	2.64	1.08	
0.40		8.96	2.22	0.252	5.00	7.20	3.20	1.30	
0.45		10.9	2.70	0.310	5.50	8.60	3.78	1.55	
0.50		13.0	3.20	0.370	6.00	10.1	4.38	1.82	
0.55		15.4	3.80	0.435	6.50	11.6	5.03	2.10	
0.60		18.0	4.43	0.504	7.00	13.3	5.75	2.40	
0.65		20.7	5.08	0.575	8.0	16.9	7.34	3.02	
0.70		23.5	5.80	0.650	9.0	20.4	9.10	3.69	
0.80			7.39	0.824	10.0	25.0	11.0	4.40	
0.90			9.22	1.01	12.0			6.20	
1.00			11.00	1.21	14.0			8.23	
1.30				1.98	16.0			10.5	
1.50				2.54	18.0			12.9	
1.70				3.15	20.0			15.5	
2.00				4.20					
2.30				5.45					
2.50				6.32					
2.70				7.21					
3.00				8.70					
3.30				10.4					
3.50				11.7					
3.70				13.0					
4.00				15.0					
4.30				17.2					
4.50				18.7					
4.70				20.2					
5.00				22.6					

Notes

1. Headloss values expressed as meters per 100 meters of straight pipe.
2. Pipe sizes listed as outside diameter in mm with respective wall thickness in mm.
3. Flow is expressed as liters per second.
4. Linear interpolation between values is possible.

following page (Figure 21). Whether water flows continually in an open system or if there are shut-off taps in a closed system, the maximum pressure criteria may be violated at the lower portion of the "U" if elevation changes are of large magnitude. This situation differs from an open system pipe of constant down-sloping grade because the weight of the water in the rising pipe section creates pressure at the bottom of the "U". High pressures in such a pipe system can be relieved by break tanks, by placing the pipe along a higher route, or by substituting pipe with a higher working pressure. A possible solution when only HDP is available is to run parallel lines of Class IV 20 mm pipe in sufficient number to replace a single line of larger diameter Class III HDP pipe. For example, if Class III 32 mm HDP can not be used because the maximum pressure is 7 Kg/cm^2 , then two lines of Class IV 20 mm HDP in parallel may be used if flow rate, friction loss, and velocity are acceptable. Note that as pipe flow relationships are not mathematically linear, substitution of two half size pipes for one full size will not, necessarily, result in the same flow situation.

DESIGN FOR BRANCHING PIPES

A not too uncommon situation in rural water supply systems is the case of two sources feeding into one main supply line. When the sources are at significantly different elevations care must be exercised in the hydraulic design to avoid the undesirable flow of water from the higher source back up the pipe and out of the lower source as shown in Figure 22 at the top of the illustration. An easy way to preclude this problem is to place a break pressure tank at the pipe junction which will accept inflows from both sources. However, it is possible that there might not be sufficient working head for the rest of the system if this is done. Alternatively, a break tank can be placed at the same elevation as the lower tank in the pipeline from the higher tank. This scheme may not be feasible if a suitable break tank site is not available. When break tanks can not be effectively employed the following design procedure may be considered.

Figure 22 (bottom illustration) shows a typical representation of a branching pipe and reservoir system. At the pipe junction, point "C", a unique, as yet unknown dynamic head exists. In order that there be no backflow from tank "A" to tank "B", the difference in the source elevations must equal the difference in headlosses from "A" to "C" and from "B" to "C". The subsequent design example will clarify the design process.

FIGURE 21. THE COMMON U-SHAPED PIPE PROFILE

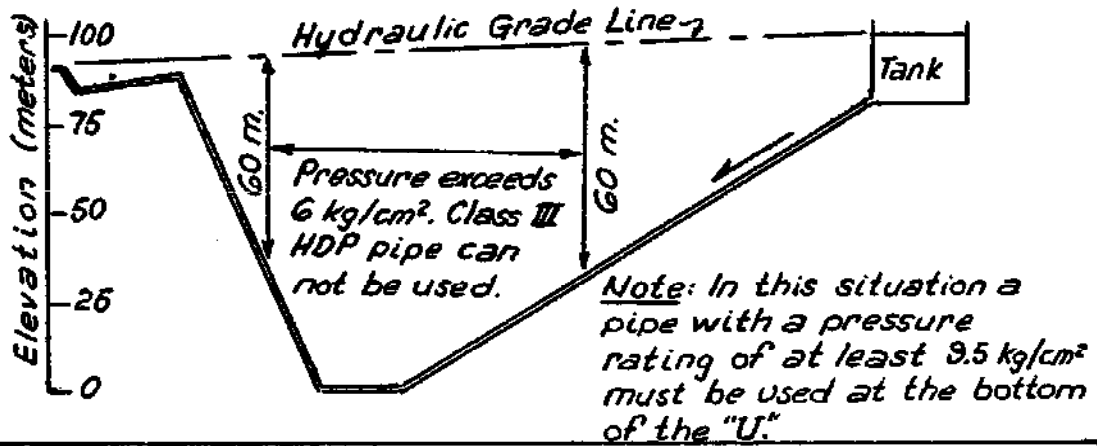
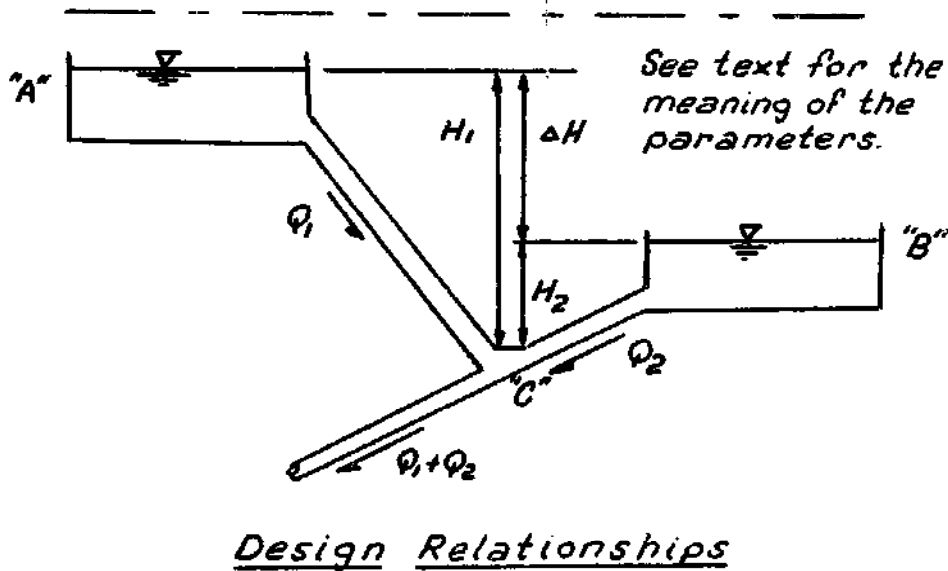
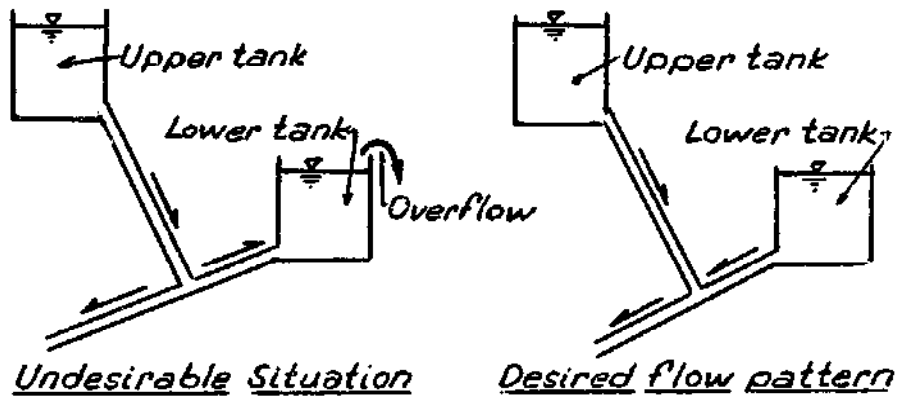


FIGURE 22. DESIGN FOR BRANCHING PIPES



DESIGN EXAMPLE FOR BRANCHING PIPES
=====

The following quantities are known:

$$\begin{array}{lll}
 Q_1 = 0.5 \text{ - LPS} & L_{AC} = 1000\text{-M} & H_1 = 150\text{-M} \\
 Q_2 = 1.0 \text{ - LPS} & L_{BC} = 500\text{-M} & H_2 = 100\text{-M}
 \end{array}$$

Let F_{AC} and F_{BC} be the respective friction headloss factors (Meters headloss per 100-Meters pipelength) for the two pipes:

In order for a unique value of dynamic head to exist at point "C": (HL_{AC} means headloss from "A" to "B")

$$H_1 - HL_{AC} = H_2 - HL_{BC} \text{ or, } \Delta H = \underline{\underline{HL_{AC} - HL_{BC}}}$$

where:

$$\begin{array}{l}
 \Delta H = 150 - 100 \text{ Meters} = 50\text{-Meters} \\
 HL_{AC} = F_{AC} \times \frac{1000\text{-Meters}}{100} \quad HL_{BC} = F_{BC} \times \frac{500\text{-Meters}}{100}
 \end{array}$$

Substituting the values and simplifying the numbers,

$$50 = F_{AC} \times \frac{1000}{100} - F_{BC} \times \frac{500}{100}$$

$$10 = 2 \times F_{AC} - F_{BC}$$

This relationship between F_{AC} and F_{BC} must be satisfied. By inspection of a friction loss table, as given below, the proper pipe sizes for the given flow conditions can be found.

FLOW	PIPE SIZE				
	12 mm	25 mm	40 mm	50 mm	65 mm
0.5 LPS	60	5.5	0.63	0.21	0.06
1.0 LPS		18.0	2.10	0.70	0.22

FRICTION FACTORS (METERS PER 100 METERS)
=====

By a trial selection from the table, a 25mm pipe for AC and a 50mm pipe for BC fit the headloss criteria closest.

$$\begin{array}{ll}
 10 = 2 \times F_{AC} - F_{BC} & F_{AC} = 5.5 \text{ for 25mm pipe} \\
 & F_{BC} = 0.7 \text{ for 50mm pipe} \\
 10 = 2(5.5) - 0.7 & \underline{\underline{\text{THEREFORE:}}} \text{ Use 25mm A to C} \\
 10 \approx 10.3 & \text{50mm B to C}
 \end{array}$$

STANDARDS FOR CONSTRUCTION OF PIPELINES AND INSTALLATION OF VALVES

The useful life of HDP pipe (or any pipe) will be greatly extended by careful and proper placement within the ground. An extra week or two of digging trenches deeper and properly backfilling will add years of usefulness to the pipeline.

The designs for placing pipe which are shown in Figure 24 will prevent erosion over the trench, preclude structural damage to the pipe by the weight of animal and human traffic, protect the pipe from chemical weathering by sun and rain, and will serve to deter vandalism. Any installation of pipe for rural water supplies should meet these four criteria. The author has found these pipeline placement standards meet the four criteria through his rural village construction experience.

The embankment and trench details are for small pipes (90 mm or less). Large pipes require higher standards for adequate protection. The area of screened, inorganic soil packed immediately around the pipe is a bedding which provides drainage and keeps large, sharp stone from cutting the exterior of the pipe wall. The backfill of excavated material free of large stone should be well compacted in 8 cm layers to avoid excessive settling of the trench. Since some settling will definitely occur, the trench should be mounded slightly so that the final, settled grade is smooth. Trenches which settle serve as ideal conduits for rainwater surface runoff, so mounding and soil stabilization is necessary to prevent erosion.

In areas where the soil is unexcavable, or only excavable to a small depth, an embankment of earth should be placed to protect the pipe. Again, compaction of the embankment and stabilization of the finished surface is required. In addition, the toe of the embankment should be "keyed" into the original ground with stone as shown in the sketch.

"Keying" is simply burying stones at 1/2 to 3/4 depth along the edge for 3 or 4 rows wide. These stones will lock the new edge in place and inhibit soil erosion. Slopes on embankment sides should not be steeper than those noted on the drawings. Shallower slopes are more stable.

The ground surface over a new trench should be restored to its original condition or improved. Replant grass or other ground cover and establish new plant cover where none existed before to minimize erosion of the fresh soil. Through forest areas a layer of duff (leaves, organic material, pine needles, etc.) about 5 cm deep will be sufficient on gentle slopes. Steep slopes in any area will present problems with erosion. Paving the disturbed area with stone or placing small diameter logs across the slope should help to slow the rush of surface water. These small logs should be staked into the ground and spaced at 30 cm intervals perpendicular to the route of the trench as shown below.

FIGURE 23. FINISHED TRENCH SURFACE STABILIZATION SCHEME

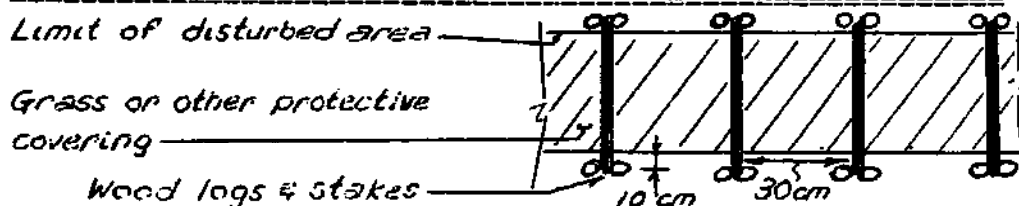
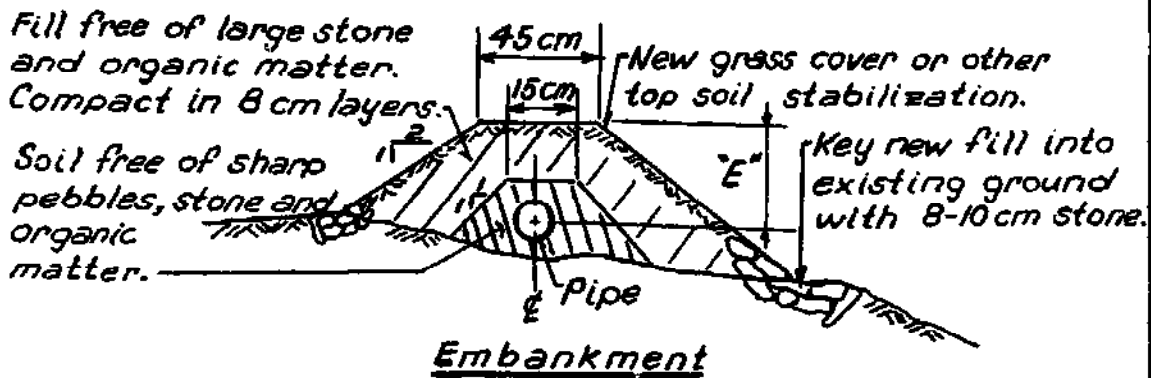
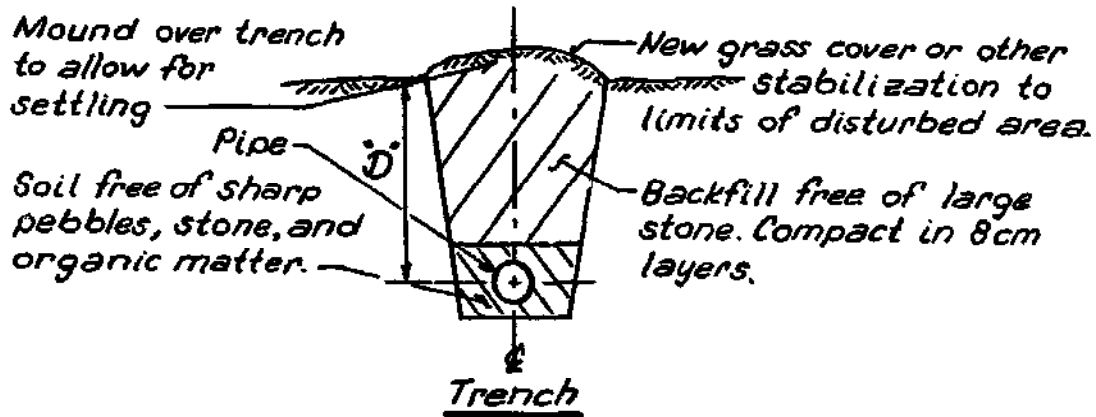
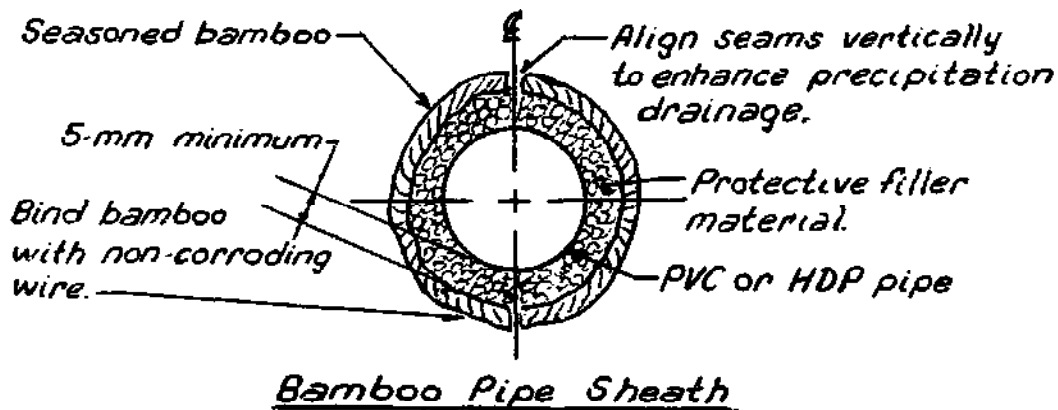


FIGURE 24. STANDARDS FOR PIPELINES



<u>Pipe</u>	<u>Max. diameter</u>	<u>"D"</u>	<u>"E"</u>
GI	90 mm	30-100 cm	30-80 cm
PVC or HDP	90 mm	80-100 cm	60-100 cm

Dimensions for Trench and Embankment Details



A successfully stabilized trench will, in time, blend in with the surrounding area and the exact location of the pipe consequently lost. Therefore, the pipe route should be marked in the following places:

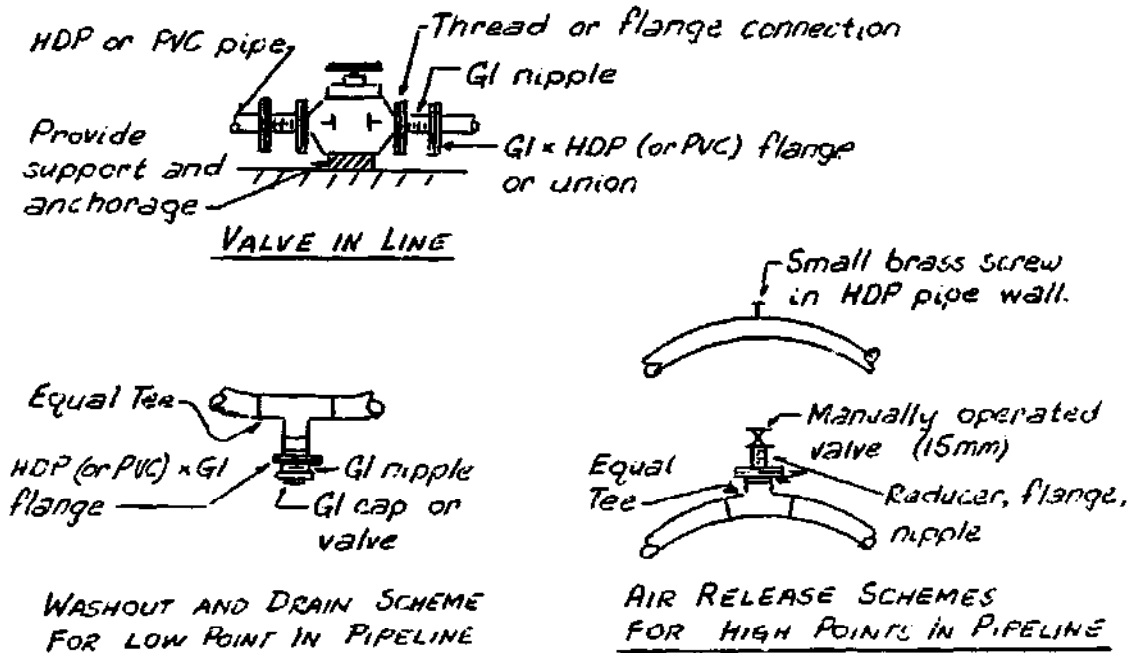
- At branches
- At changes in direction
- At changes in size
- At submerged valve boxes
- At every 25-40 meters on straight runs

A stone monument rising at least 15 cm above the ground and securely anchored in place will serve as a good marker. Alternatively a piece of upright pipe will also be sufficient. Pipe size, branch line size, direction change, and valve box location should be marked on the monuments. This information could be very valuable when trying to pinpoint trouble in the system.

Valves

There are four situations in which valves are needed in rural water system pipelines. First, gate valves or corporation cocks are installed downstream of tanks in order to close-off the line or the next tank downstream for cleaning, repair, and maintenance. Second, a valve is placed at a branch or

FIGURE 25. STANDARDS FOR VALVE PLACEMENT



Note: House all valves in protective, locking valve boxes of suitable dimension for maintenance and removal.

in a line to regulate (proportion) flow by creating "artificial" headloss. Third, an air release valve is placed at high points to allow entrained air to escape. These air release valves, if not available through the project funding agency, may be purchased at great expense in the market or ingeniously fabricated at the site. Fourth, a washout valve is placed at low points in the line. The washout valve is for draining the line and for removing sediment which might collect at the low point. Refer to the preceding illustration for designs of simple valve installations.

Globe valves and corporation cocks have high headlosses. These types of valves are best used for flow regulation. Gate valves, when completely opened, have a relatively low headloss. Use gate valves for situations where the valve is either open full or shut, but not partially open. For instance, a gate valve should be placed on the outflow line from a tank because such a valve is usually open full and shut only for system repair. Globe valves or corporation cocks should be located at branch points in the pipeline to divide the flow as required. Always use a gate valve or a plain cap or plug at a washout location. A globe valve should not be installed here because it will clog easily by debris emerging from the washout.

7. BREAK PRESSURE TANKS

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This section describes a standard design for a stone masonry break pressure tank, specifications for its construction, and an estimate of labor and materials required to build such a tank. A detailed standard design is possible because break pressure tanks are common features of rural water systems in Nepal. The design has been kept relatively simple to allow for the variety of conditions and construction material availability found in village construction projects.

The stone masonry tank is emphasized here because it is presently the most frequently used type of construction. The reader is encouraged, however, to look into alternate innovations such as plastic and sheet steel tanks which may be feasible and economical for a particular project.

CONSTRUCTION SPECIFICATIONS FOR BREAK TANK

Refer to the drawing of the standard design on the following page while reading the subsequent specifications for construction.

1. Excavation

Clear away organic soil and material, tree roots, and soft stone from the site. Excavate down to a gravelly soil or rock base. Place at least 8 cm of gravel fill under the tank and valve box to provide for drainage and to create a clean surface for the masonry superstructure. Thoroughly compact the gravel fill. When more than 8 cm is laid, compact in 8 cm layers.

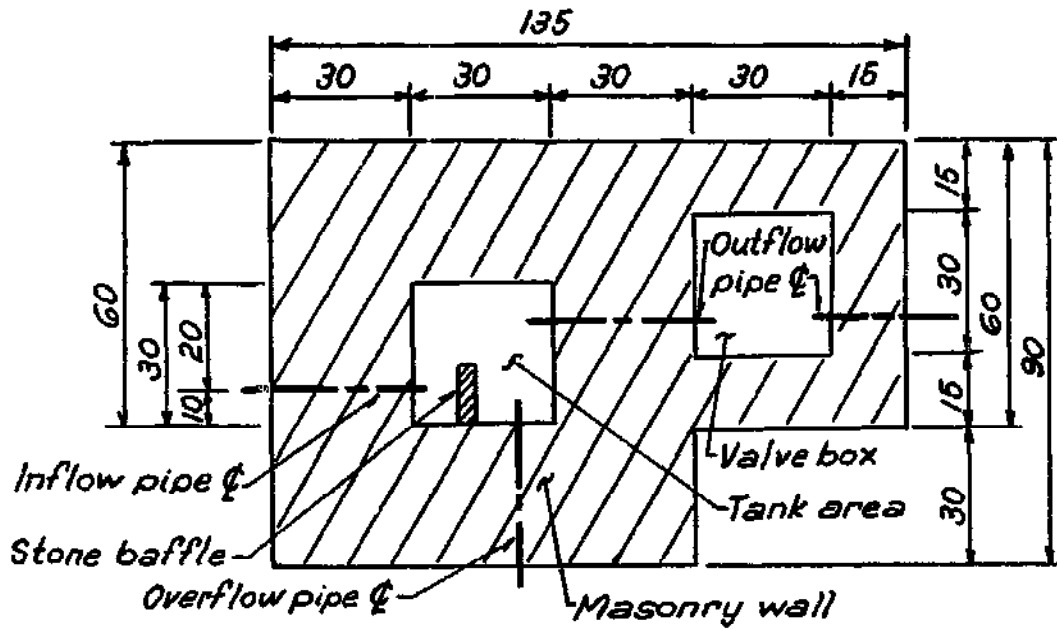
2. Masonry

Use a 1:4 cement to sand ratio for mortar and construct stone or brick masonry walls and floor to the approximate dimensions shown in the drawing. Coat the interior of the break tank with 12 mm of cement plaster made from a 1:3 cement to sand ratio. If other water proofing material is more readily available use it instead of cement plaster. It is not necessary to place a floor in the valve box area, nor is it necessary to line the valve box interior with plaster. Stone slab covers (or other roofing as is available) should fit tightly to preclude the entrance of debris and consequent contamination. Covers should preferably be bolted and locked. Include 2 or more 2 cm diameter air holes near the top of the tank in the masonry wall. These insure that atmospheric pressure exists in the tank, and they also provide ventilation.

3. Interior baffle and screen

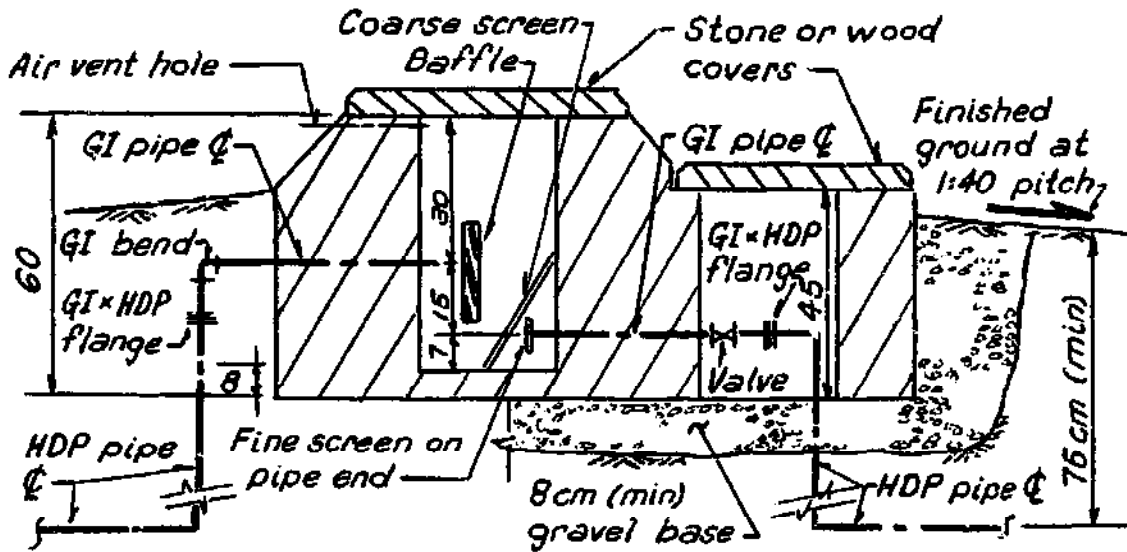
A stone baffle should be located opposite the inflow pipe as shown in the drawing. The baffle serves two purposes. First, it dissipates the energy in the emerging jet of water. Second, it protects the masonry walls from erosion by the jet of water. The baffle should be constructed so as to be easily removable and replaceable.

FIGURE 26. BREAK PRESSURE TANK DESIGN STANDARD



PLAN VIEW AT ELEV. 45 cm

- Notes: 1. Refer to Break Pressure Tank specifications for details on piping and construction.
2. All dimensions in centimeters.



ELEVATION VIEW

A coarse screen is required to protect the outlet pipe. It serves to stop debris and save the finer screen from damage. The coarse screen should be fabricated from non-corroding material and mounted on a wood frame. It should be durable enough to withstand the accumulation and impact of debris which may enter the tank. Secure the screen in place, but make it removable for cleaning and replacement.

4. Piping and fittings

Pipe centerlines should be located as shown on the drawing. Actual pipe sizes are determined by the required hydraulic capacity of the system. This design, however, assumes that 90 mm (O.D.) diameter pipe would be the largest size required. The outlet pipe should be as large or larger than the inlet pipe. In the situation where the hydraulic design calls for a smaller diameter pipe at the outlet, place a transition section of inlet pipe diameter reducing to outlet pipe diameter at the outlet in order to mitigate the headloss. The outlet pipe is placed 8 cm above the floor to avoid carrying sediment into the downstream pipeline. The 15 cm allowance between the inlet and outlet pipes compensates for headlosses within the structure.

GI pipe should be used in the locations noted on the drawing. PVC pipe is an acceptable secondary alternative. When GI pipe is to pass through a masonry wall, it should be roughened on its exterior and set into the fresh mortar in order to establish a strong bond. PVC pipe should also be roughened slightly, or spread some glue and sand on the PVC pipe exterior to give the mortar a surface to bind to.

Fine mesh screen flange units are available for current projects. They will keep fine debris from clogging valves. The valve shown on the drawing should preferably be a gate valve. A corporation cock valve may be used alternatively on pipes smaller than 32 mm. Connections between GI and HDP pipe can be made with unions or flanges.

HDP (or PVC) pipe and fittings should be used in the locations shown on the drawing. The vertical runs of HDP pipe are required to recover adequate trench depth as close to the tank as possible. Try to approach and leave the tank with the HDP pipe at a depth of 60 to 90 cm below the finished ground surface. At least 20 cm of soil cover should be allowed to safeguard GI pipe.

All GI pipe, fittings, and valves should be painted to retard corrosion. Remove rust and grease before painting.

5. Overflow pipe and overflow erosion control

The break tank will overflow when the system downstream of it is shut off or not in use. Therefore, a break tank could be overflowing all night long and during some time of the day. A well constructed overflow system is required to carry off this flow and to prevent consequent soil erosion.

The overflow pipe should be placed above the inlet pipe and should be of the same or larger diameter. GI or PVC pipe should be used for passing through the masonry wall. If only a short distances must be traveled to get the overflow suitably away from the tank, then continue the GI or PVC to this point. (Otherwise, for a long distance, switch to HDP.) Protect the overflow pipe with 15 cm of ground cover or, when near ground surface, with a bedding

and cover of stone. Exposed pipe above the ground should be GI to prevent breakage.

At the end of the overflow pipe precautions must be taken to prevent soil erosion by the issuing stream of water. Hopefully, the overflow can be put to efficient use such as irrigation, pokhari augmentation, or redirection into a stream. Whatever the final destination is, stabilize the water's path with a stone lined or grass covered channel of sufficient capacity to conduct the peak overflows. Care should be taken to avoid making low points where drainage water may collect in the channel, as these breed insects and waterborne disease organisms.

6. Grading and finishing the area around the tank.

Grade the finished ground surface away from the break tank and valve box at a slope of no less than 4 cm per meter as shown in the drawing. If the tank is set on a slope, construct a peripheral drainage ditch from about two meters up hill of the tank to lead rainwater surface runoff away from the tank. Surface runoff carries numerous pollutants, which if allowed to seep in around the tank will contaminate the water supply.

Stabilize the newly finished ground surface and drainage diversion ditches with local grasses (seed or sod), stone paving, or other plantings.

7. Cleaning and maintenance

The villagers left in charge of maintaining the water supply system should be instructed in the following points. The tank must be checked periodically, especially after heavy rains, and debris and sediment removed from it. The tank can be emptied of water by shutting off the valve on the structure (intake tank or other break tank) upstream of it. Painting of GI fittings, valves, and pipe can be done once a year or as required. Valve lubrication, with a non-toxic lubricant, should be applied three times yearly.

LABOR AND MATERIAL ESTIMATE

The break pressure tank illustrated on the previous page will require 0.56 m³ of stone masonry work and 0.84 m² cement mortar plastering.

- a. 0.25 m³ dry cement mortar (1:3) requiring:
 1. 100 Kg cement (2 bags)
 2. 0.2 m³ clean sand
- b. 0.8 m³ rough stone (to be cut and shaped)
- c. 0.5 m² flat stone slab cover
- d. Each 8 cm layer of gravel fill requires 0.1 m³ gravel.

The fittings and appurtenances for the break pressure tank include the below listed items:

- a. Inlet piping (of design diameter)
 - 60 cm GI inlet pipe
 - 1 - 90° GI elbow
 - 1 - GI x HDP flange or union connection

- b. Outlet piping (of design diameter)
 - 75 cm GI outlet pipe
 - 1 - valve (gate valve or corporation cock)
 - 1 - GI x HDP flange or union connection
- c. Overflow piping (of design diameter)
 - GI or PVC pipe of required length for the particular situation
- d. Appurtenances
 - Coarse screen - 0.2 m²
 - Screen frame made from wood
 - Fine mesh screen flange unit
(UNICEF # SLN 281 - 37,38 or equal unit)
 - Bolts and locks for stone covers as required
 - Paint - 1/2 liter
 - Lubricating oil (years's supply)

HDP bends can be field fabricated or standard, ready made bends can be employed.

The laborers needed for break pressure tank construction include a mason, a mistri, and unskilled helpers. The estimates for man-days of labor presented below will be useful in estimating skilled labor pay requirements; however, this author has found that unskilled villagers can build break tanks with the overseer's guidance, thereby eliminating paid skilled workers.

Mason	:	masonry	0.8
		plastering	0.1
		stone cutting and shaping	1.1
		total	<u>2.0</u>
Mistri	:	pipe fitting and screens	1.0
		painting	0.5
		total	<u>1.5</u>
Unskilled Labor:		excavation	0.8
		masonry	1.8
		plastering	0.2
		stonework	2.2
		gravel fill	1.2
		total	<u>6.2</u>

8. RESERVOIR TANKS

=====

This section on standards for reservoir tanks includes the following topics:

- Tank Site Considerations
- Sizing the Tank
- Standard Designs and Estimates
- Specifications for Reservoir Tank Construction

TANK SITE CONSIDERATIONS

A preliminary step of the design is selecting a tank site. Choose the site judiciously to possibly obviate an extra break pressure tank, to be easily accessible, to take advantage of the best soil and most stable terrain, and to economize on pipe system construction.

Note that a reservoir tank will drain out if a break pressure tank is located down stream of it. This problem can be avoided by installing a self-closing ball float valve on the break tank or building smaller tanks on the branches below all break tanks. Float valves are expensive, not readily available, and difficult to maintain. Two small reservoir tanks are significantly more expensive than a single larger tank. Therefore, reservoir siting will often involve economic as well as hydraulic considerations.

SIZING THE RESERVOIR TANK

A reservoir tank serves to store water during minimal water use time periods so that a greater supply is available for maximal water use periods. In remote areas of Nepal reservoir tanks, whether masonry or plastic, are extremely expensive features of a water system. Close attention to the proper design procedure will ensure a minimum sized, economic reserve water supply for the village.

A reservoir tank will not be needed in the following situation. Given a small village with a design year population of 250 persons (or less) per public tap, then each tap should be designed for a flow rate of 0.225 LPS. The maximum water use rate is 0.225 LPS multiplied by the number of public taps. Note that peak water demand analysis does not apply to public tap flows. Now, if the source's safe yield flow exceeds this maximum water use rate, then a reservoir is not needed and pipes are sized based on the number of taps each pipe reach will serve. (Refer to the section on Public Tapstands for further information on flows in pipelines which serve taps.)

When a source will not be sufficient in fulfilling peak period water demands, then a reservoir is designed on the basis of a daily water use schedule. The following pattern of water demand is suggested for use for rural water supply systems in Nepal:

* See Appendix E, page 97, for additional information.

6 AM - 8 AM	30% of daily demand
8 AM - 4 PM	40% of daily demand
4 PM - 6 PM	30% of daily demand
6 PM - 6 AM	negligible water demand

The suggested water demand scheme is not based on quantitative field studies, but is derived from qualitative observations. The two 30 per cent demand periods correspond to the water needs for the two daily meals; including uses for pre-meal ritual bathing, cooking, and dish washing. The mid-day 40 per cent demand primarily reflects the clothes laundering and livestock watering needs of the village. The designer should investigate the particular consumption habits at the project site and account for any anomalies in a revised water use pattern. It should be noted, however, that the presence of a new, piped water system, featuring conveniently located public tapstands, will probably change the water demand pattern of the pre-construction setting. That is, villagers will no longer be required to haul water over long distances from far away sources, and convenient public tapstand supply generally results in smaller peak demand amplitudes of longer duration compared to pre-construction peak demand periods. Therefore, a quantitative field study of pre-construction demand habits may be only of limited use for reservoir design.

This use pattern results in two daily peak demand rates - one in the morning; one in the evening. The reservoir stores water during the slack demand periods to augment supply for the morning and evening peaks. The peak demand flow rate for reservoir design is computed by multiplying the population for the design year by 0.00188 (assuming 45 LPCPD) to yield peak flow in liters per second.

There are a number of ways to transform the water use pattern and source safe yield into a size for the reservoir tank. The graphical method is often used, however, it is easy to make errors with this approach. The following tabular numerical method is suggested for use. The example illustrated in the table is for a population of 1000, a daily demand of 45,000 liters, and a source flow rate of 1.0 liters per second. Note that the total daily deficit between demand and supply is the required reservoir capacity.

Example of Reservoir Sizing

Time Period	Demand		Supply		Deficit ² (Liters)
	Rate (LPS)	Quantity ¹ (Liters)	Rate (LPS)	Quantity ¹ (Liters)	
6 AM - 8 AM	1.88	13,500	1.0	7,200	6,300
8 AM - 4 PM	0.625	18,000	1.0	28,800	-
4 PM - 6 PM	1.88	13,500	1.0	7,200	6,300
6 PM - 6 AM	0.00	-	1.0	43,200	-
Totals		45,000		86,400	12,600

Reservoir capacity = 12,600 Liters (12.6m³)

Notes:

1. This column of figures is obtained by multiplying the rate in "LPS" by the time interval in "seconds" to yield "liters".

e.g.: $1.88 \text{ LPS} \times 2 \text{ hours} \times 3600 \text{ seconds/hour} = 13,500 \text{ liters.}$

2. This column derives from subtracting the supply in "liters" from the demand in "liters".

e.g.: $13,500 - 7,200 = 6,300 \text{ liters.}$

Note that when supply exceeds demand in a particular time interval no entry is made in this column.

For the situation in which 45 liters per person per day is supplied, the graph of Figure 27 shows required tank sizes given the population and the source safe yield flow. Analysis of this graph shows that required capacity is related to source flow in the following manner:

<u>Hours of source safe yield flow to supply daily demand</u>	<u>Capacity of tank</u>
9 hours	0.10 x daily demand
12 hours	0.15 x daily demand
18 hours	0.33 x daily demand
24 hours	0.50 x daily demand

If the source can safely yield enough water to satisfy the daily demand in 9 hours, then, given the high cost of cement in Nepal, it is probably most economical to build the smallest tank (0.1 x daily demand) and to size the supply main from the source to the tank large enough to carry this flow rate. However, in a special situation with a tank not requiring much cement, say brick-powder and lime mortar ("surki") is used instead, and a very long reach of pipe from source to tank; then a smaller supply main diameter (which might only conduct enough flow to meet the demand in 12 hours) and a larger tank (perhaps 0.33 x daily demand) may be more economical. The economics of pipeline size construction versus reservoir size should be investigated in this situation. Compare the cost of a larger diameter pipe to a larger size tank in such an economic analysis.

STANDARD DESIGNS AND ESTIMATES

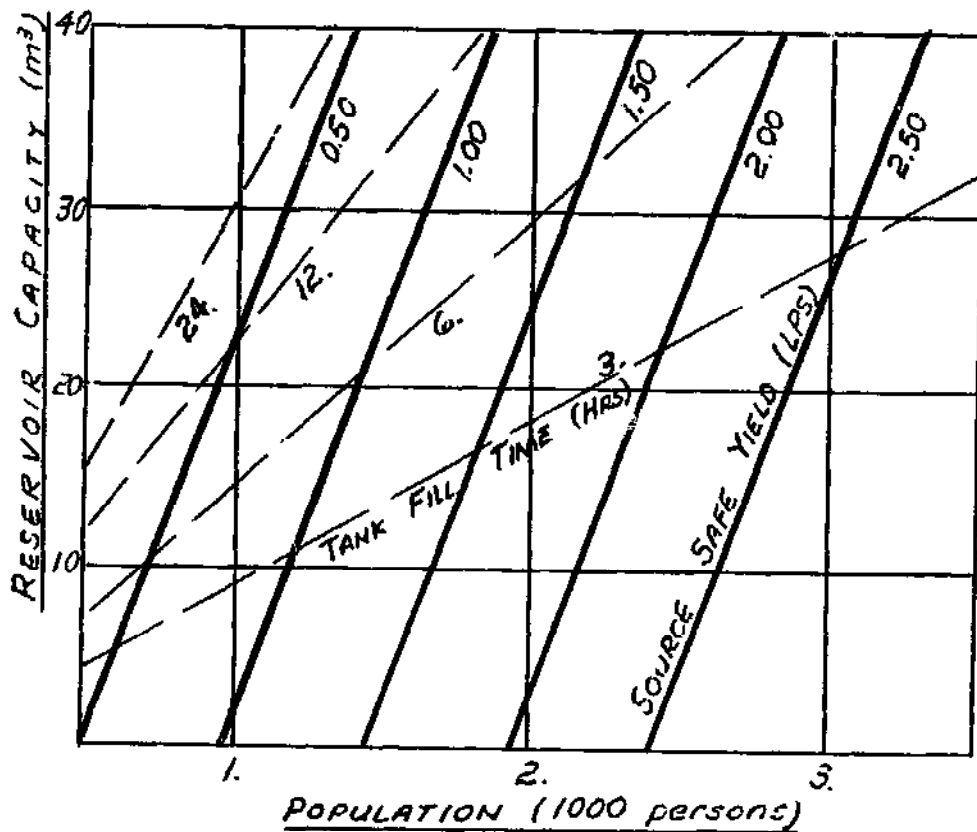
Once the capacity of the tank has been determined it must be dimensioned and designed. Figure 28 shows the relationship for economically optimal tank dimensions and tank capacity through 40 cubic meters. These dimensions for length, width, and water depth have been chosen to minimize roof area, floor area, and masonry wall volume given the required tank capacity. Thus, by choosing these dimensions, the design minimizes the materials (cement, stone, roofing, etc.) required for construction. This optimization scheme roughly approximates the minimum cost tank of desired design capacity.

Standard designs and estimates for tanks from 4 to 40 cubic meter nominal capacity are presented in the following pages. The tanks are all stone and mortar masonry, slate roofed, and are sized according to the optimal dimension scheme mentioned in the preceding paragraph. Each was designed to minimize cement usage and to maximize on the use of locally available materials. They all share some common features noted below:

- Tanks are square (length equals width)
- Gabled roofs of slate and timber construction
- Stone masonry floor and walls (cement mortar)
- Sand, gravel, and stone foundation
- GI piping in tank and at tank vicinity
- Exterior wall stepping and mid-wall buttressing

The drawings and details and estimates which follow are based on cement mortar construction. Designers wishing to further mitigate cement

FIGURE 27. RESERVOIR SIZING (45 LPCPD)



NOTE: The graph is based on the water demand pattern described on preceding pages, and is presented to display relationships rather than as an aid to obviate computation.

requirements by using locally available mortar or surki* should first investigate the structural strength and durability of this alternate mortar and then increase wall and buttress dimensions as required.

When using the standard design values and estimate figures interpolation for intermediate size tanks is possible. Tanks larger than 40 cubic meters are quite expensive and difficult to construct in remote areas and should be avoided unless absolutely necessary.

* Surki is crushed brick powder mixed with lime. Surki mortar masonry has allowable compressive and tensile stresses roughly fifty per cent those of portland cement masonry. It is, however, the best alternative mortar to portland cement available in the remote areas of Nepal.

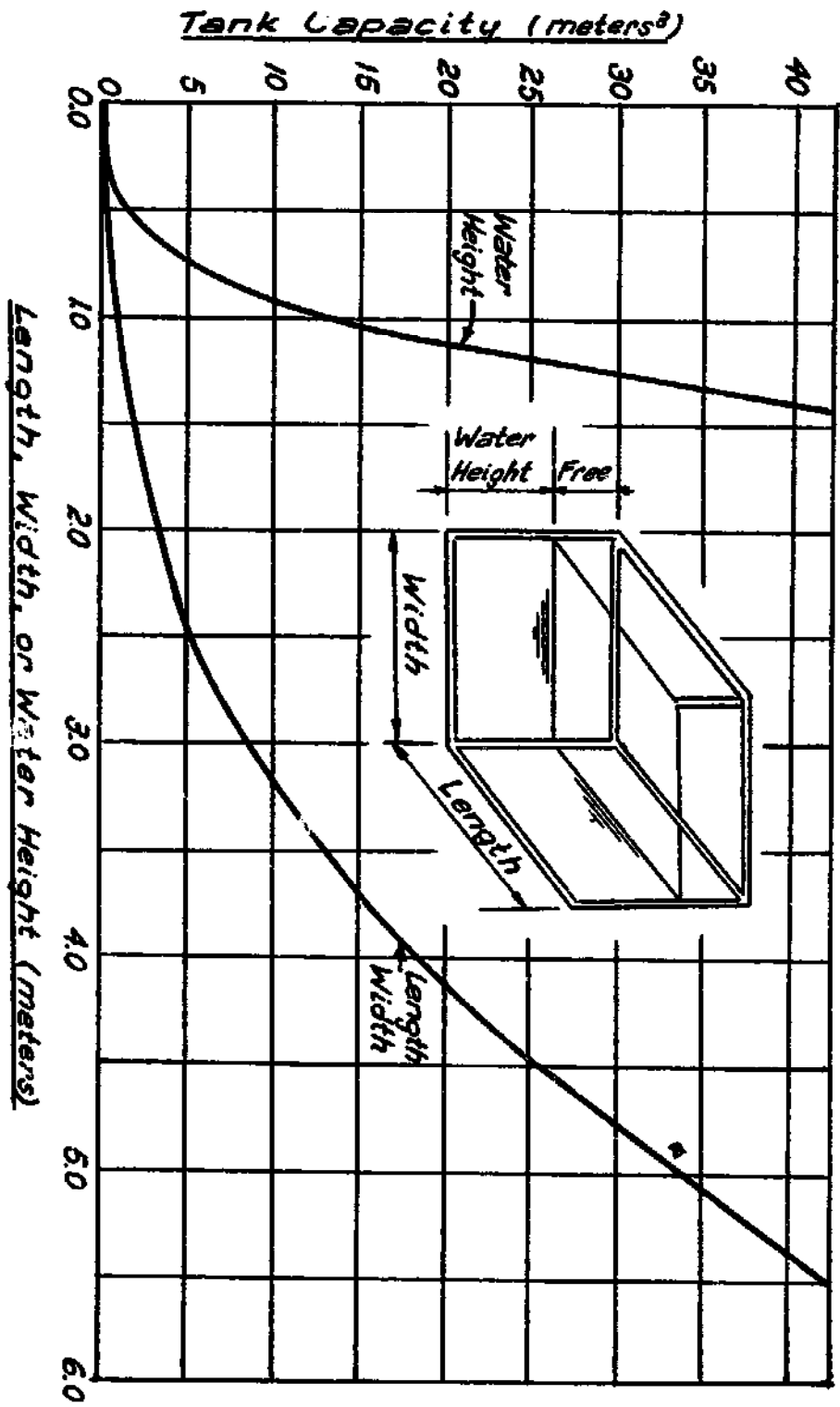
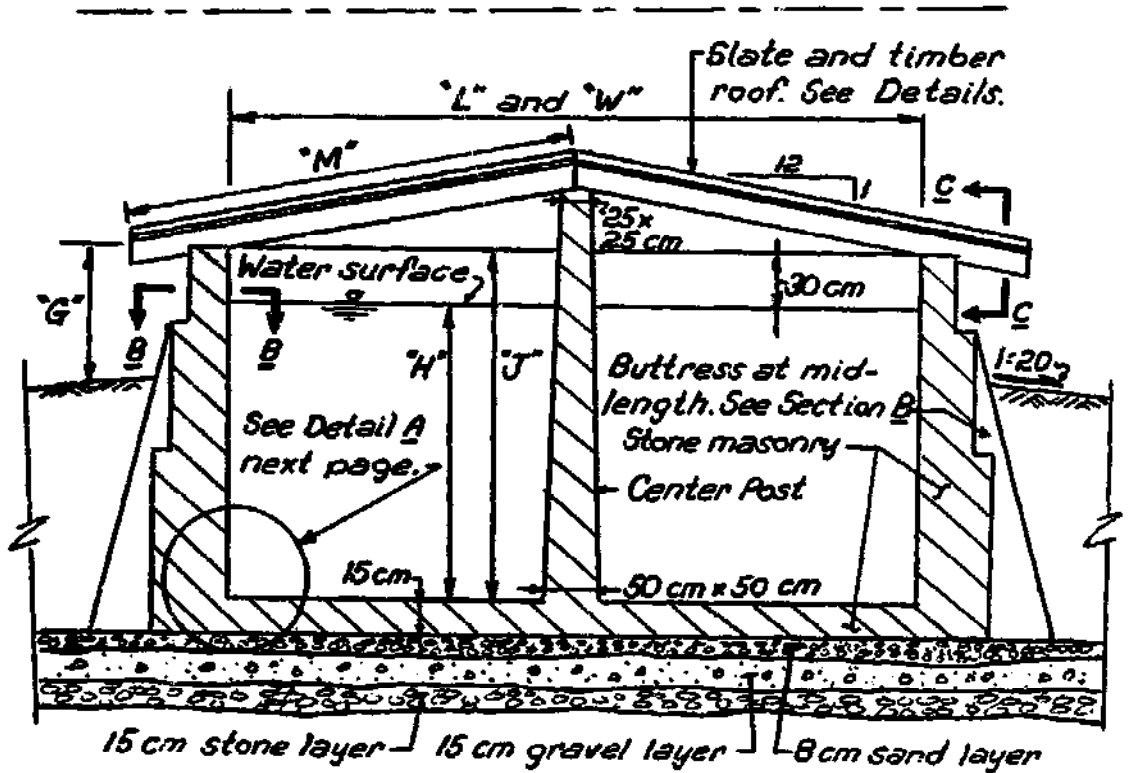


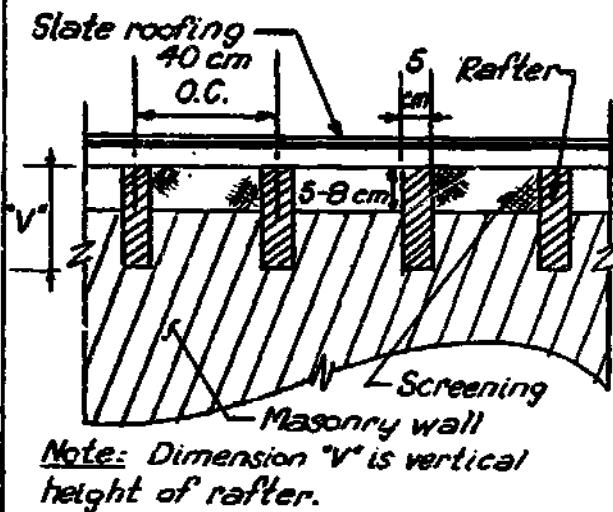
Figure 28. OPTIMAL DIMENSIONS VS. TANK CAPACITY

FIGURE 29. RESERVOIR TANK DESIGN STANDARD

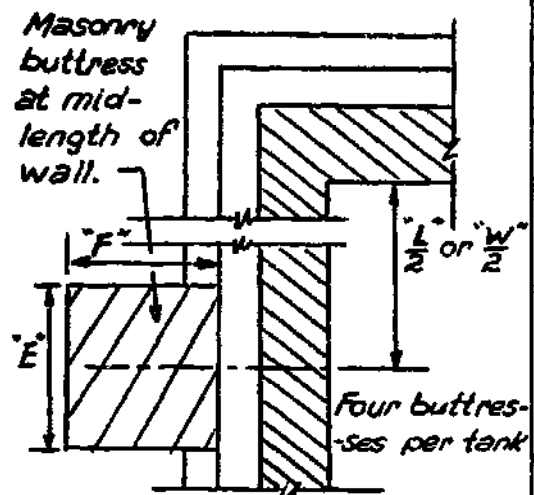
Refer to SCHEDULE OF DIMENSIONS (Figure 30) for dimensions denoted by capital letters in this figure. See also section of text on reservoir tank specifications.



Elevation View

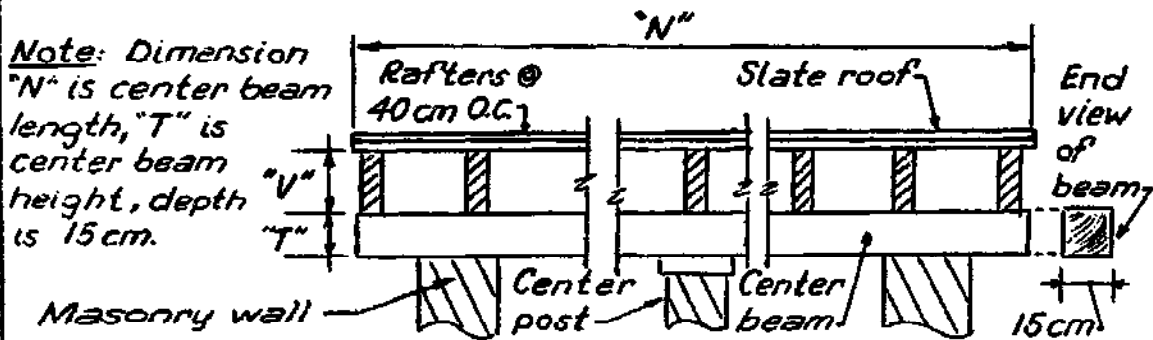


Section C-C



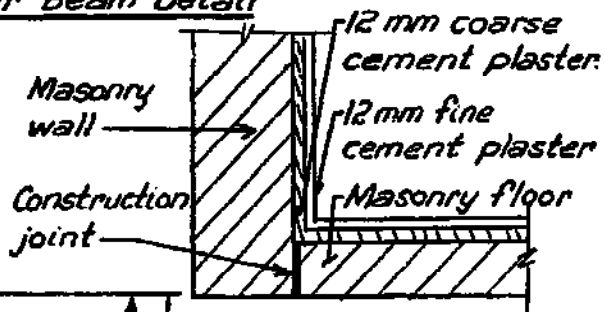
Section B-B

Figure 29. RESERVOIR TANK DESIGN STANDARD (CONTINUED)

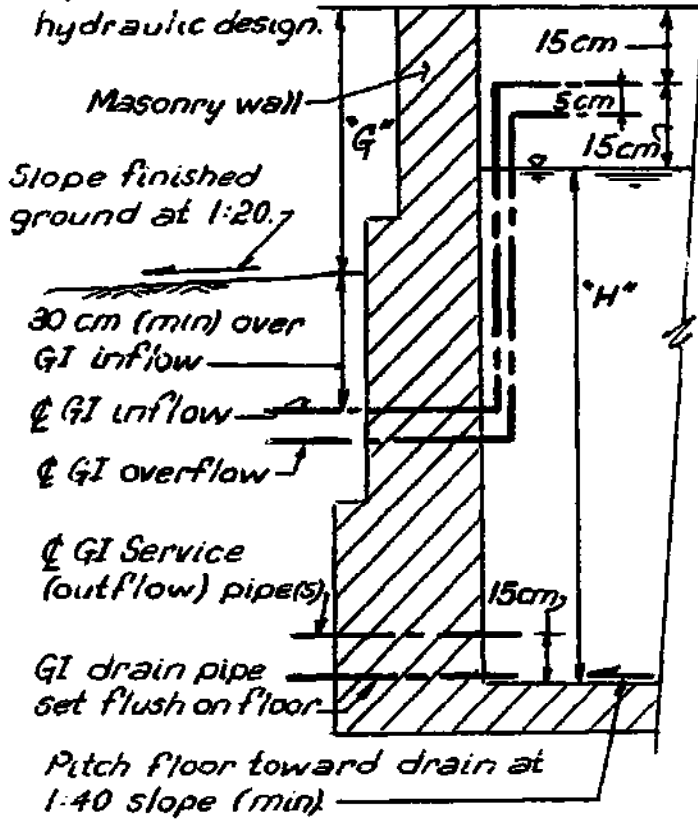


Center Beam Detail

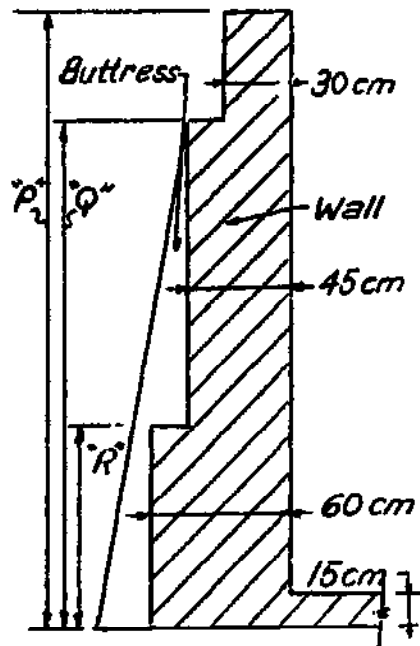
- Note regarding pipes:*
1. Detail shows relative pipe elevations, pipes need not be all in one wall.
 2. Pipe sizes as per hydraulic design.



Detail A



Pipe Placement Detail



Wall Dimension Detail

FIGURE 30. SCHEDULE OF TANK DIMENSIONS

Refer to the drawings for dimension location

	Actual Size (m ³)	Nominal Size (m ³)
"E"	0.30	0.30
"F"	0.50	0.50
"G"	0.60	0.60
"H"	0.75	0.95
"J"	1.05	1.25
"L"	2.25	2.90
"M"	1.35	1.70
"N"	3.25	3.90
"P"	1.20	1.40
"Q"	0.90	1.10
"R"	0.00	0.20
"T"	0.25	0.25
"V"	0.15	0.15
"W"	0.25	2.90
"E"	0.40	0.40
"F"	0.50	0.50
"G"	0.60	0.60
"H"	1.00	1.05
"J"	1.30	1.35
"L"	3.40	3.80
"M"	1.95	2.15
"N"	4.40	4.80
"P"	1.45	1.50
"Q"	1.15	1.20
"R"	0.25	0.30
"T"	0.20	0.20
"V"	0.15	0.15
"W"	3.40	3.80
"E"	0.60	0.60
"F"	0.60	0.60
"G"	0.70	0.70
"H"	1.10	1.15
"J"	1.40	1.45
"L"	4.10	4.45
"M"	2.35	2.50
"N"	5.10	5.45
"P"	1.55	1.60
"Q"	1.25	1.30
"R"	0.35	0.40
"T"	0.20	0.20
"V"	0.20	0.20
"W"	4.10	4.45
"E"	0.80	0.80
"F"	0.70	0.70
"G"	0.80	0.80
"H"	1.20	1.20
"J"	1.50	1.50
"L"	4.70	4.70
"M"	2.65	2.75
"N"	5.70	6.00
"P"	1.65	1.65
"Q"	1.35	1.35
"R"	1.35	1.35
"T"	0.45	0.45
"V"	0.25	0.25
"W"	5.00	5.00
"E"	0.90	0.90
"F"	0.75	0.75
"G"	0.90	0.90
"H"	1.30	1.30
"J"	1.60	1.60
"L"	5.15	5.30
"M"	2.85	3.00
"N"	6.15	6.30
"P"	1.75	1.75
"Q"	1.45	1.45
"R"	0.55	0.55
"T"	0.25	0.25
"V"	0.25	0.25
"W"	5.15	5.30

Notes:

1. All dimensions are meters.
2. Masonry center post not required in 4 and 8 m³ tanks.

Figure 31. STANDARD RESERVOIR TANK ESTIMATE SUMMARY

	Nominal Size (m ³)	4	8	12	16	20	24	28	32	36	40
Actual Size (m ³)		3.8	7.6	11.4	15.2	18.9	22.8	26.6	30.4	34.2	37.8
Excavation											
Volume (m ³)		18	28	37	44	48	56	62	68	71	74
Unskilled Labor		13	20	27	32	34	40	44	49	51	53
Foundation											
Sand (m ³)		2.5	3.4	4.0	4.7	5.3	5.8	6.4	7.0	7.4	7.6
Gravel (m ³)		2.5	3.4	4.0	4.7	5.3	5.8	6.4	7.0	7.4	7.6
Stones (m ³)		1.2	1.7	2.0	2.4	2.6	2.9	3.2	3.5	3.7	3.8
Unskilled Labor		3	4	5	6	7	8	9	10	10	10
Back Filling											
Unskilled Labor		2	2	2	2	2	3	3	4	4	4
Stone Masonry											
Volume (m ³)		6.3	9.8	12	14	15	18	21	21	24	25
Cement (1:4) (m ³)		0.53	0.82	1.0	1.2	1.3	1.5	1.7	1.8	2.0	2.1
Sand (1:4) (m ³)		2.1	3.3	4.0	4.8	5.0	6.1	6.9	7.2	8.2	8.4
Uncut Stone (m ³)		8.4	13	16	19	20	24	27	28	32	33
Mason		20	31	38	45	47	57	65	67	76	79
Unskilled Labor		42	65	80	94	100	121	137	143	160	168
Cement Plaster											
Area (m ²)		29	46	58	70	80	92	100	110	120	130
Cement (m ³)		0.12	0.19	0.24	0.28	0.32	0.37	0.38	0.39	0.39	0.39
Sand (m ³)		0.41	0.64	0.82	0.98	1.1	1.3	1.3	1.3	1.3	1.4
Mason		4	6	8	9	11	12	14	15	16	17
Unskilled Labor		9	15	19	23	26	30	32	36	38	40
State Roofing											
States (m ²)		7.7	12.4	16.6	20.2	24.2	27.9	31.1	34.4	36.6	39.8
5 cm. Natts (Kgs.)		1.2	2.0	2.6	3.2	3.8	4.4	4.9	5.4	5.8	6.2
Mason		3	4	6	7	8	10	11	12	13	14
Carpenter		3	4	5	6	7	8	8	9	10	12
Unskilled Labor		3	4	6	7	8	10	11	12	13	14

Note: one layer coarse (1:4) plaster 12 mm one layer fine (1:3) plaster 12 mm.

FIGURE 31. STANDARD RESERVOIR TANK ESTIMATE SUMMARY (CONTINUED)

Nominal Size (m ³)	4	8	12	16	20	24	28	32	36	40
Woodworking										
Total Volume (m ³)	0.42	0.59	0.72	0.82	1.1	1.3	1.4	1.8	1.8	2.0
Central Beam Length(m)	3.2	3.9	4.4	4.8	5.1	5.4	5.7	6.0	6.2	6.3
Dimensions (cm x cm)	15x25	15x25	15x20	15x20	15x20	15x20	15x20	15x25	15x25	15x25
Rafters:										
Number Required	18	20	22	22	26	28	28	30	30	32
Length (Each)	1.3	1.7	2.0	2.2	2.4	2.5	2.6	2.8	2.8	3.0
Dimensions (cm x cm)	5x15	5x15	5x15	5x15	5x20	5x20	5x20	5x25	5x25	5x25
Nails for slates (m ³)	0.12	0.20	0.27	0.32	0.38	0.44	0.49	0.55	0.58	0.63
Nails (10-12 cm) (Kgs.)	2.7	2.7	2.7	3.2	3.6	3.6	4.1	4.6	4.6	4.6
Roof Anchoring Bolts										
(15 cm.) (Nos.)	15	18	21	24	26	27	28	30	31	32
Carpenter										
Unskilled Labor	8	11	13	15	20	23	25	32	33	36
Installation of Fittings										
Mistri	2	2	2	2	2	2	2	2	2	2
Unskilled Labor	2	2	2	2	2	2	2	2	2	2
Screening (m ²)	0.46	0.56	0.65	0.74	0.84	0.93	1.0	1.1	1.2	1.2

FIGURE 31. STANDARD RESERVOIR TANK ESTIMATE SUMMARY (CONTINUED)

4	8	12	16	20	24	28	32	36	40
27	41	52	61	66	79	90	94	106	110
11	15	18	21	27	31	33	41	43	48
2	2	2	2	2	2	2	2	2	2
82	123	154	181	199	237	263	288	311	327
0.64	1.0	1.3	1.5	1.6	1.9	2.1	2.2	2.4	2.5
20	30	38	44	48	57	63	66	72	74
5.0	7.2	8.8	10.4	11.3	13.2	14.4	16.0	17.0	17.2
2.5	3.3	4.0	4.7	5.3	5.8	6.4	7.1	7.4	7.6
1.2	1.7	2.0	2.4	2.6	2.9	3.2	3.5	3.7	3.8
8.4	13	16	19	20	24	27	28	32	33
7.7	12	17	20	24	28	31	34	37	40
0.42	0.59	0.73	0.81	1.2	1.3	1.4	1.7	1.8	2.0
1.4	2.3	2.7	3.6	4.1	4.6	5.0	5.4	5.9	6.4
2.7	2.7	2.7	3.2	3.6	3.6	4.1	4.6	4.6	4.6
15	18	21	24	26	27	28	30	31	32
0.46	0.56	0.65	0.74	0.84	0.93	1.0	1.1	1.2	1.2

Schedule of Fittings (Each Size Tank)
(diameters as per hydraulic design)

4 - m GI Pipe	5 - m GI Pipe	5 - m GI Pipe	6 - m GI Pipe
2 - 90° Bends	2 - 90° Bends	1 - GI Union 2 - GI Nipples	1 - Gate Valve
1 - GI x HDP	1 - GI x HDP Union or Flange	1 - Screen - Flange Unit	1 - Cap or Plug (90 mm)
Union or Flange	Washout	for Mouth of Pipe	
		(UNICEF #SLN 281-37,38)	

SPECIFICATIONS FOR RESERVOIR TANK CONSTRUCTION

These specifications are meant to supplement the standard design drawings and to provide a basis for design for tanks which do not conform to the standards presented.

1. Excavation and foundation

Excavate down to a firm gravelly soil base or to rock and fill with sand, gravel and stone (12 mm - 25 mm diameter) at least to the minimal dimensions shown in the drawings. Compact sand and gravel in 8 cm layers. This foundation serves as a structural support and as an area for draining ground water. Sand, gravel and stone should extend about 60 cm beyond the tank wells.

2. Masonry

A 1:4 cement sand ratio was used for the standard designs and estimates in the wall and floor masonry. The 15 cm masonry floor should be built of two or three layers of mortared stone with 33% to 50% overlapping. Two coats of cement plaster are required: a coarse coat of 1:4 cement sand ratio and a fine coat of 1:3. Each coat should be 12 mm thick for adequate water-proofing.

Walls are stepped to increasing thickness to compensate for hydrostatic pressure. Walls can be tapered as an alternative to stepping. Interior or exterior stepping is suitable. Exterior stepping makes for easier interior plastering but more water can infiltrate from outside the tank. Interior stepping presents a cleaner exterior appearance for an exposed tank wall. The buttresses are purely structural features and care should be taken to insure their structural integrity. The masonry center post in tanks larger than 8 cubic meters may alternatively be made from timber which is treated with a wood preservative.

3. Roofing

Slate and timber roofs have been designed for the tanks as these materials are usually locally available. A zinc-coated steel sheeting roof or a temporary thatched roof are good alternatives. Provide a locking accessway in the roof. It should be at least 60 cm x 60 cm so that cleaning and maintenance work can be easily carried out. Compartmentalized tanks should have manholes for each compartment.

Timber not exposed to direct water contact should be painted, as painting will inhibit decay. The screening at the two end walls provides ventilation to keep the woodwork dry and to prevent the accumulation of gases in the tank.

4. Piping and fittings

Locate centerlines of pipes at the relative elevations shown in the drawings. Pipe sizes are determined by hydraulic design, but the overflow should be at least as large as the inflow, and the washout should be 90 mm in diameter. Pitch the floor to the washout pipe at a slope of 1:40.

(Refer to break pressure tank specification for the way to anchor GI pipes in masonry walls.) A by-pass pipe should be placed around the tank so that the system will function when the tank is being cleaned or repaired. As the reservoir is a major structure, maintenance and repair will involve considerable time. The by-pass line is important for insuring the flow of some water to the village when the reservoir is temporarily out of service. The by-pass line may require a break pressure structure to preclude excessive pressures at downstream

points while the reservoir is out of service. A screening device should be placed over the outlet (service) pipe(s).

Construct a lockable valve box in a location free from flooding by tank overflows. A gate valve should be placed on each service pipe. The gate valve has two purposes: a) to regulate the height of water in the tank, and b) to shut off the system downstream of the reservoir. The washout pipe does not require a valve since it is used infrequently. A well-lubricated cap or plug fitting can be used here, and removed when it is necessary to drain the tank. Paint all GI valves and fittings and pipe.

5. Overflow

The reservoir tank may overflow often so proper precautions should be taken to avoid soil erosion at the overflow point. Lead the overflow pipe away to a suitable location where the wasted water can be used efficiently. Any open channels conducting the overflows should be stone lined and of constant slope to avoid the formation of stagnant pools of water. At least 15 cm of ground cover should be placed over GI pipes and 45 cm minimum over PVC pipes which are used as overflows. The washout pipeline should be constructed in a similar manner to the overflow, because water rushing from the draining tank will erode unprotected soil. It is wise to conduct flows from both the overflow pipe and the washout pipe into a single wastewater channel which leads to a garden, field, pokhari, or stream.

6. Grading and finishing the area around the tank

The greatest potential source of natural contamination to the water supply is rain water runoff seeping into the tank. This surface runoff can be polluted with human and animal wastes, decaying organic matter, and harmful chemicals substances. To avoid surface runoff pollution of the reservoir, grade the finished ground away from the tank at about a 1:20 slope. Construct a diversion ditch to lead away surface runoff from the uphill side of the tank. This ditch should be similar to the diversion ditch depicted for the intake tank in an earlier section of this manual. Stabilize the newly finished ground (and ditch) surface with a new grass cover (seed or sod), stone paving or other locally available flora.

A fence might be helpful to keep people and animals away from the tank. A barbed wire fence will look less austere if integrated with shrubs and bushes. Locally constructed wooden fences can be quickly, inexpensively erected to keep animals away from the tank. Timber posts placed in the earth should be buried in gravel, stone or sand to inhibit rot from moist organic soil.

7. Cleaning and maintenance

The reservoir tank will tend to collect sediment at a faster rate than the smaller tanks in the system because water is detained in it for relatively longer periods. For this reason, the depth of sediment should be periodically checked, especially during the rainy season. When the sediment layer approaches 10 cm of the outlet pipe, which is 15 cm above the floor, the tank should be cleaned out. (Note that this sediment can best be used in a garden or agricultural field to augment and condition the soil cover as organic sediments have good nutritive and mineral content.) If sediment removal is anticipated to be a frequent task, the by-pass line constructed around the tank is especially important so that limited system operation can proceed during cleaning periods. Tank cleaning is also required when debris or

contaminating substances pollute the tank. Scouring the interior with lye*, and rinsing and drying thoroughly will serve to disinfect the tank after such a pollution occurrence. A thorough rinsing is required to remove all vestiges of lye from the tank. Two or three days of drying will aid in killing off any lingering pathogenic, anaerobic microorganisms.

Paint exposed GI pipe and valves yearly. Lubricate valves three times per year with a non-toxic lubricant.

* Make lye (pot ash) locally by draining water through a bed of wood ashes. This caustic liquid will be a fair disinfecting agent when used in copious amounts. Liquid chlorine or ammonia, if available, are better disinfectants.

9. PUBLIC TAPS

=====

DESIGN CONSIDERATIONS

The public taps are the most frequently used and most visible features of the rural water system. Depending on their design and construction they can be focal points of community pride or messy, muddy, poorly functioning units. The success or failure of the completed public tap will depend upon the amount of consideration given to the cultural, aesthetic and performance criteria of design.

A well designed tap will account for local water usage customs and traditions. This is the cultural factor. In Nepali villages religious bathing and washing rituals, caste and tribal relations regarding water use, and village politics are all aspects of the cultural design factor.

A well designed tap can also be a source of community pride and should be an inviting area at which people gather. This is the aesthetic factor. While the rest of this section deals with tap performance and utility, it is not meant to slight the importance of the cultural and aesthetic factors inherent in public tap design. Careful thought should be given to both of these; the designer should be familiar with local water use customs, and should seek input from the villagers as to their needs and ideas. Then, after doing this, the design can be completed with respect to the utility and performance criteria presented in the following paragraphs.

UTILITY AND PERFORMANCE STANDARDS

The tap must meet the following criteria:

- a. It must deliver water at the design rate.
- b. It must be built to accommodate the various uses of vessel filling, bathing, and laundering.
- c. It must be structurally sound, durable and long lasting.
- d. It must be a clean, sanitary unit with efficient drainage and efficient use of wastewater.

Each of these criteria is discussed in detail in the following paragraphs.

Design flow requirements

Public taps serving 250 or fewer people should have a design flow of 0.225 LPS. At this flow rate a typical village "gagaro" will fill in a minute. For taps serving more than 250 people increase the flow proportionately or use a maximum of 0.0015 LPS per tap user, depending on how much water is available. Do not design a tap for a flow less than 0.225 LPS.

A pipeline serving only one tap should be sized to carry the flow for one tap. A pipeline which leads to nine taps, however, does not have to be designed to carry the full flow for nine public taps. When designing pipes which will

serve six or more taps it can be assumed that not all the taps will be open simultaneously and a design based on probability theory can be made. Therefore, in a "closed" system with shut-off faucets size pipelines serving taps according to the following scheme. (The numbers are based on a 0.225 LPS design flow for each individual tap.)

- 1 - 6 Taps : assume all in use simultaneously - design pipe for 0.225 LPS per tap.
- 7 - 9 Taps : assume 85% in use simultaneously - design pipe for 0.19 LPS per tap.
- 10 - 12 Taps : assume 75% in use simultaneously - design pipe for 0.17 LPS per tap
- 12 - or more Taps : assume 66% in use simultaneously - design pipe for 0.15 LPS per tap.

The piping from the distribution line to an individual tap should be 20 mm HDP and half inch GI. In order to achieve the desired flow at the tap it is necessary to place a regulating valve in this line. Set the valve such that the maximum flow at the tap is equal to the desired design flow. This valve also will fulfill the dual purpose of a shut-off valve when the tap needs maintenance or repair. The regulating valve should be a gate valve (corporation cock if gate valve not available), and should be secured in a strong, locking valve box. The valve box may be located within the tap stand or underground. In either case, the valve should be tamper-proof because opening or closing this valve will perturbate the designed flow pattern for the water system.

Residual pressure criteria

Residual pressure at the tap is an important consideration. Too much pressure results in leakage and waste of water. Too little pressure results in small flows. The following table summarizes the range of residual pressures which should be used in rural water supply systems.

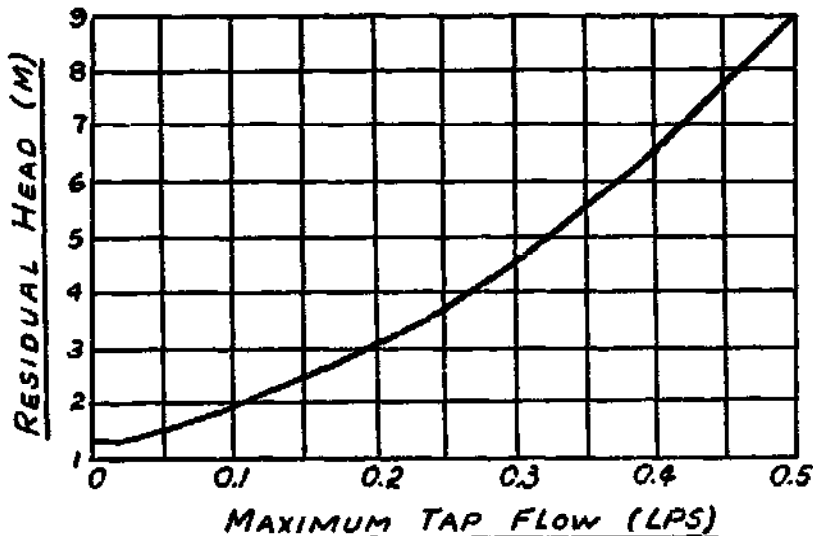
Standards for Residual Heads at Public Tap Stands

	<u>Kg/cm²</u>	<u>Meters of water</u>
Absolute minimum	0.7	7
Low end of desired range	1.0	10
Most desirable	1.5	15
High end of desired range	3.0	30
Absolute maximum	5.6	56

These limits have been adopted in light of the many constraints imposed by gravity system design and are rather liberal. Wagner and Lanoix (Reference # 26, p. 209) suggest a more conservative range from 10-50 meters.

During the design check for minimum pressure conditions at maximum flow (i.e. maximum headloss), and check maximum pressure criteria for the static fluid condition (i.e. no headloss). Note the graph of tap flow versus residual head presented below.

FIGURE 32. RELATIONSHIP OF TAP FLOW AND RESIDUAL HEAD



Note: *The relationship assumes a tap with half inch GI piping and 3 to 4 meters of 20mm HDP piping from main distribution pipe.*

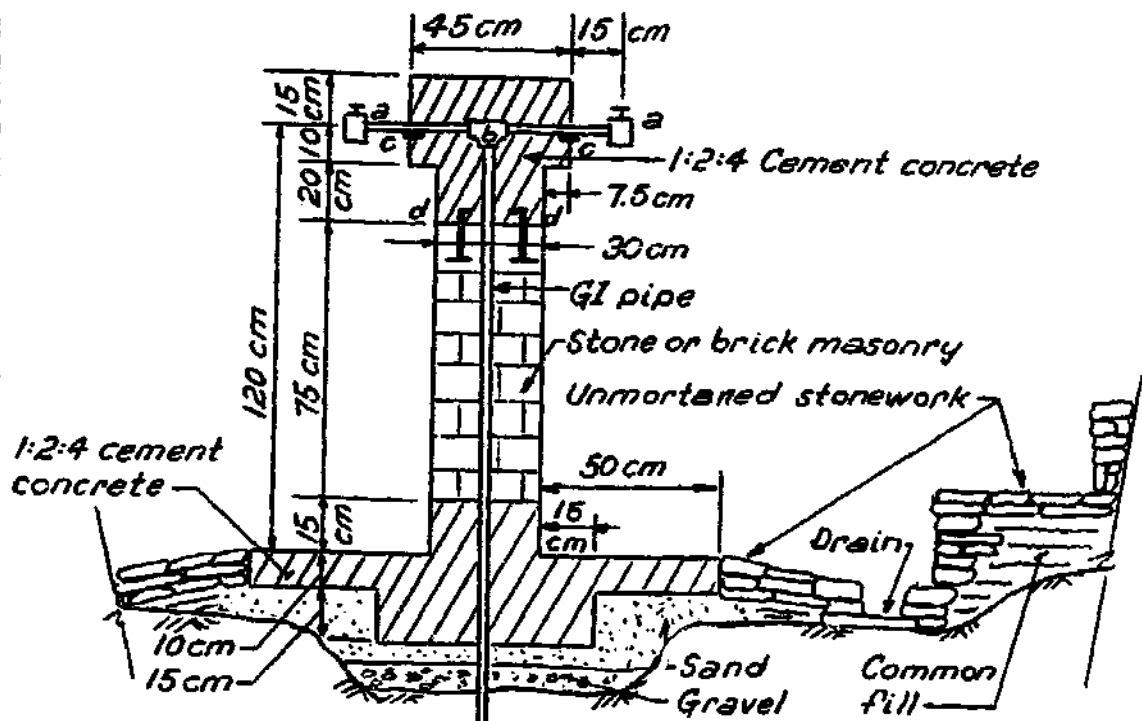
A standard public tap design

A standard design and estimate of materials and labor is presented on the next page (Figure 33.). This design was done by the engineers at RA & LDD for the rural water supply program. It is very likely that all the materials and fittings required for this design will not be available. Therefore, the information in the following sections is provided as a guide for standard construction procedures when building taps of designs not conforming to the illustrated standard design.

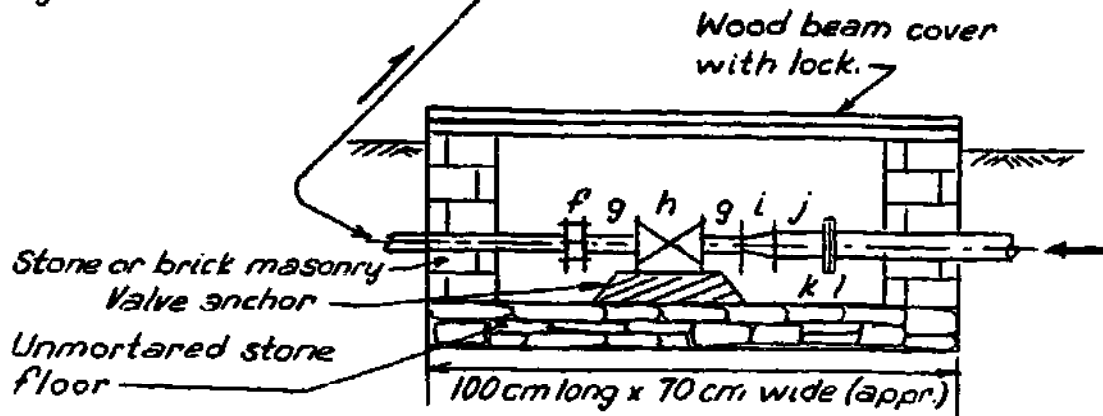
Structural criteria

The physical layout of the tap area should provide for convenient washing, laundering and vessel filling. Ample space should be afforded to each use. A 1.2 meter tap height is typically sufficient for these uses. However, at a tap where two or more faucets are provided, it is wise to set the faucets at different heights to accommodate both small and tall users. Since water vessels are usually strapped to the head for carrying, a bench of sufficient height can be advantageously provided under one of these faucets to facilitate lifting of the full vessel.

FIGURE 33. DESIGN FOR A PUBLIC TAP



Note: Fittings denoted by lower case letters are listed on the next page.



Valve Box Detail
(Scale enlarged)

FIGURE 34. PUBLIC TAP ESTIMATE

List of Materials and Labor

1. excavation		coolies	2 nos
2. stone soling and mud-stone work	140 cft	mason	1.4 nos
		cooly	7.0 nos
3. cement concrete 1:2:4	10 cft	mason	0.3 nos
		cooly	1.2 nos
		cement	2.2 cft
		sand	4.4 cft
		gravel	8.8 cft
4. stone masonry	16 cft	mason	0.6 nos
		cooly	1.4 nos
		cement	1.2 cft
		sand	4.8 cft
		stone	18.0 cft
5. installing of fittings		<u>mistry</u>	<u>1.0 no</u>
TOTALS:		mason	2.3 nos
		mistry	1.0 no
		cooly	11.6 nos
		cement	3.4 cft*
		sand	9.2 cft
		stone	18.0 cft
		gravel	8.8 cft

* 3.4 cft cement is approximately three 50-Kg bags

List of Fittings

a. jayson type tap	$\frac{1}{2}$ "	2 nos
b. GI tee equal	$\frac{1}{2}$ "	1 no
c. reinforcement iron plate		2 nos
d. J-hooks or ribar		2 nos
e. GI elbow	$\frac{1}{2}$ "	1 no
f. GI union	$\frac{1}{2}$ "	1 no
g. GI nipple	$\frac{1}{2}$ "	2 nos
h. GI gate valve	$\frac{1}{2}$ "	1 no
i. GI reducing socket	$\frac{1}{2}$ " / 3/4"	1 no
j. GI nipple	3/4"	1 no
k. GI flange	3/4"	1 no
l. HDP flange and accessories	20 mm	5 meters
GI pipe	$\frac{1}{2}$ "	

When stand posts can not be constructed to the standard design, be sure that the alternative design is rigidly secured in the ground. Stone (one upright piece) or wood posts should have one half of the total length below ground. Thoroughly compact gravel backfill around these posts to insure rigidity. A wood post will require special precautions to insure long life and resistance to decay. A moist environment catalyzes decay, so adequate drainage around the submerged wood post should be provided. A sketch and estimate for a wood post tap is presented in Figure 35.

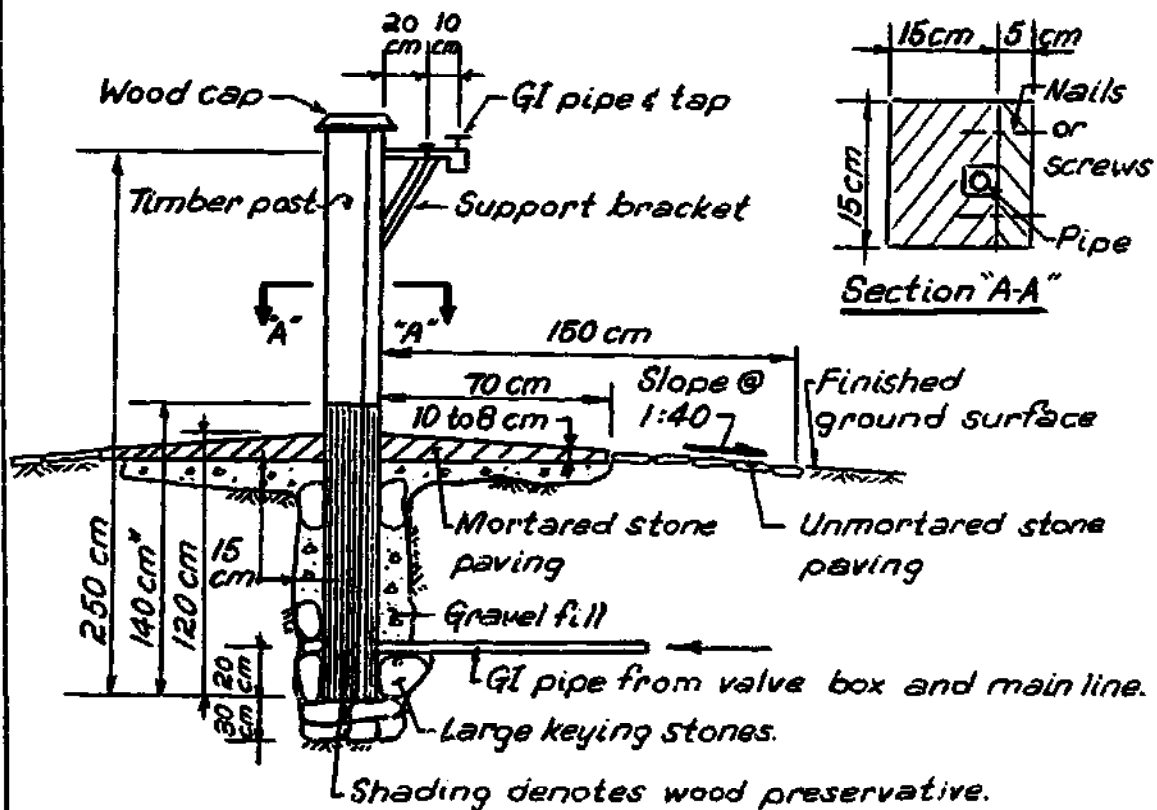
A GI pipe tap should not be cantilevered without support for more than 15 cm. Mud mortar tap stands should be plastered with cement plaster or with brick powder and lime plaster in order to protect the mud mortar from rain and splash erosion.

Sanitation and drainage criteria

There are a few basic sanitary precautions which will help the tap area stay clean and inviting. One might first start with a fence and gate to keep animals away from the area. Another precaution is to insure proper drainage of the wastewater. Try to find an efficient use for the wastewater such as irrigation or pokhari filling. Conduct the wastewater to its destination in a nonerodible channel. Wastewater should not be allowed to stagnate near the tap nor in the channel as such sluggish pools breed insects and waterborne microorganisms which are not conducive for maintaining potability of the water supply. It is always best to drain wastewater as far as possible from the tap stand site. Some possible schemes for draining waste flows from taps are shown in Figure 36.

The finished ground area around the tap should be paved and stabilized to preclude soil erosion, prevent muddy conditions, and to permit rapid drying of moisture. Concrete or mortared stone paving is the best ground cover for at least a 60 cm radius around the tap. Beyond this area, pave with unmortared stone to a radius of 2 meters. A good cover of grass or other erosion control vegetation should be established from the outer perimeter of the paved area. All of the finished ground surface around the tap should slope away at 5 cm per meter to insure that rain water does not collect at the site. Construct a rainwater diversion ditch on the uphill side of a tap site if required. Line the ditch with planted grass or stabilize with stone paving.

FIGURE 35. TIMBER TAPSTAND DESIGN



Notes: "Minimum height for preservative.
 1. Drainage channels not shown. See Drainage Standards.

Fittings

- 1 -- 1/2" tap
- 2 -- 1/2" GI 90° bend
- 1 -- 1/2" GI nipple
- 1 -- 1/2" gate valve
- 1 -- GI x HOP flange
- 5 meters 1/2" GI pipe
- 28 -- 7 cm screws or nails
- 4 -- 3 cm screws or nails

Labor

- unskilled -- 3.5 days
- mason -- 1.0 day

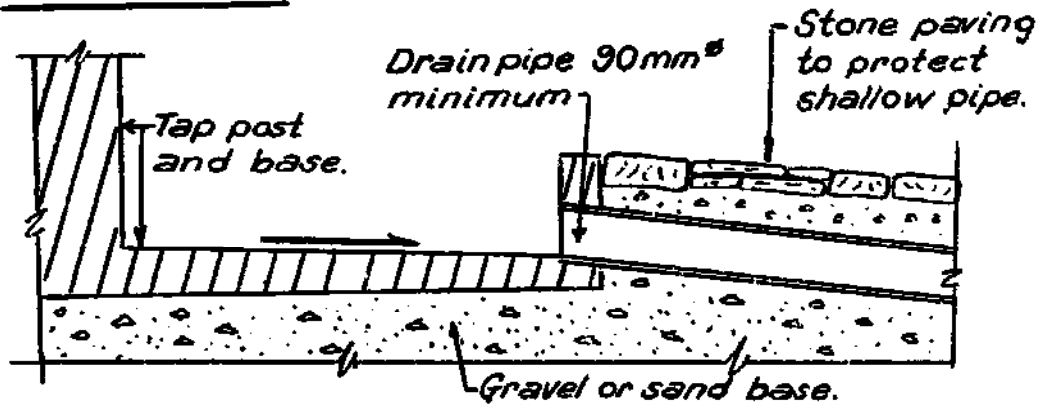
Materials

- 1 -- 15 x 15 x 250 cm timber
- 1 -- 5 x 15 x 250 cm timber
- wood cap and bracket
- 0.39 m³ gravel fill
- 0.42 m³ stone for paving
- 0.025 m³ sand for mortar
- 0.16 bags of cement for 1:4 mortar

- carpenter -- 1.5 days
- mistri -- 1.0 day

FIGURE 36. PUBLIC TAP DRAINAGE STANDARDS

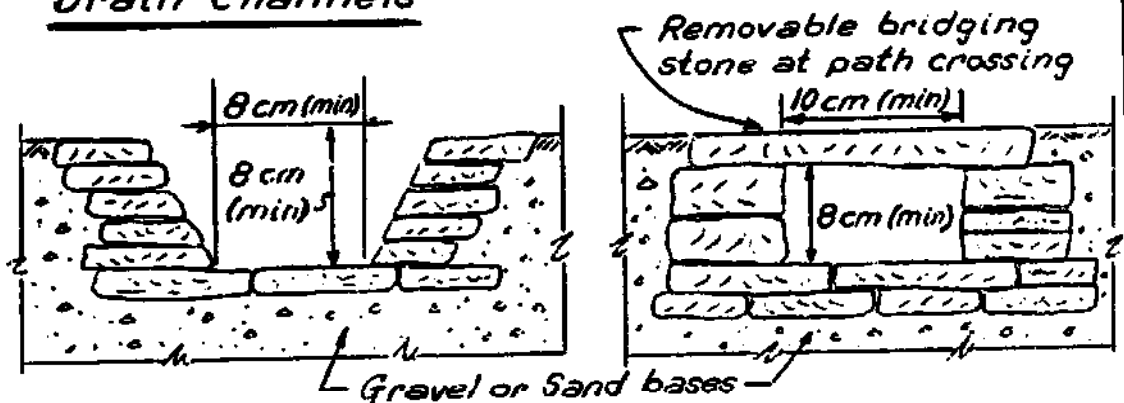
Drain Pipe



Notes:

1. Minimum drain pipe slope is 5%.
2. Drain pipes should be free of bends and ample space left at each pipe end for cleaning.
3. Recover standard ground cover over pipe in shortest possible length.

Drain Channels



Notes:

1. Minimum drain channel slope is 5%.
2. Channels lined with cut, shaped, tight-fitting stone or mortared stone.
3. Joints in between bridging stones should be tight to minimize debris accumulation in channel.
4. Channels must be kept clean for efficient drainage.

10. WATER QUALITY

=====

The material presented in the preceding sections has been written with the assumption that a suitable source water quality existed. The designs for each feature of the water system incorporate sanitary precautions to keep the water potable. A water system constructed according to the design criteria in this manual will be an important step towards insuring public health in the village.

This section is devoted to the topic of what can be done if the water source is only of marginal quality for human consumption. No effort has been made to describe detailed designs nor design criteria. Rather, this section will merely acquaint the reader with certain feasible testing and treatment schemes which may be used in village water systems.

As mentioned in the discussion of source evaluation, only biological water testing can be done in the field at the present time. The portable testing kits will indicate whether or not the source is contaminated by human and/or animal waste. A source found to be polluted by such wastes can be improved in two ways - finding the source of contamination and preventing it, or using a treatment scheme. If the village is in a relatively developed area, then disinfection or sand filtration might be possible. Each is expensive, each requires responsible management and maintenance. However, if the water need is great enough and if conditions look promising for building and maintaining either of these treatment systems, then an RA & LDD engineer and WHO advisor should be consulted for further investigation of treatment feasibility.

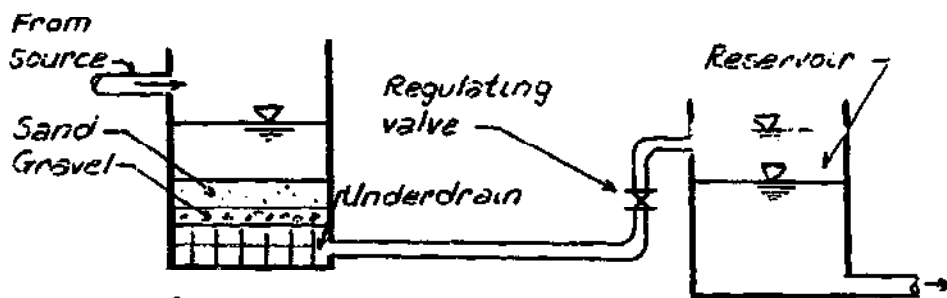
Aside from biological pollution, water may be physically or chemically polluted. Some sources of water may be markedly cloudy or turbid. Oftentimes the polluting particles will settle out when the water is given a chance to remain still over a period of time. Construction of a sedimentation tank or an oversized reservoir tank could provide the required treatment. Chemicals can be added to enhance settling, and lime is a commonly used coagulant which is available in many localities. Therefore, a cloudy water problem may be amenable for treatment even in a remote region.

A source may be polluted with iron which gives the water a red color and an offensive taste. This problem, and the problem of acidic water (carbon dioxide), may be suitably resolved by aeration and sedimentation, and aeration alone, respectively. An aeration unit can be simply made with a tower of wooden slats through which water droplets break, fall, and become exposed to oxygen in the air.

Aeration towers and sedimentation tanks are simple treatment units which could mean the difference between no water and good quality water for a village. As in the case of treatment for the removal of biological contaminants, there will be locations where aeration or sedimentation may be practical and places where their installation would prove unsuccessful. However, these water quality problems and many others that are corrected by basic treatment methods

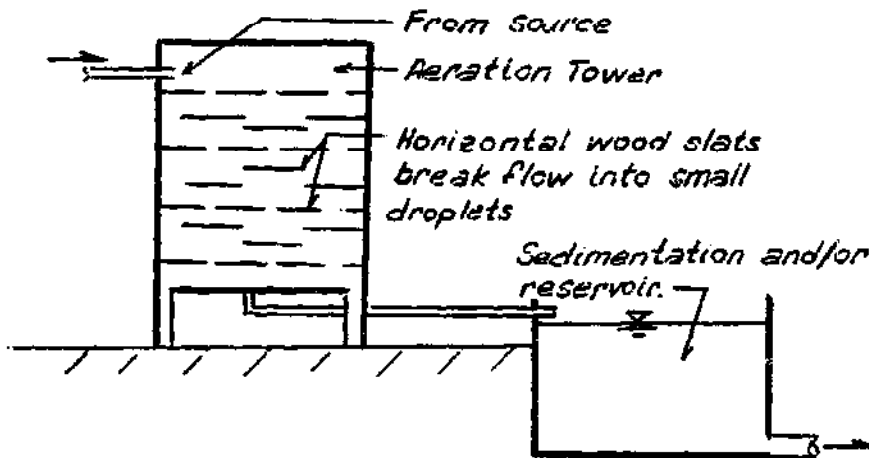
should be brought to the attention of an engineer for further investigation. Many references listed in the back of the text will provide the designer with detailed information on treatment schemes for rural water systems.

FIGURE 37. FEASIBLE TREATMENT SCHEMES FOR
REMOTE AREAS IN NEPAL



See Ref. # 10 for typical dimensions and design data

SAND FILTER



AERATION SCHEME

11. INTRODUCTION TO WINDMILLS AND HYDRAULIC RAMS
=====

Windmills and hydraulic rams, though presently not extensively used in the rural water supply program of RA & LDD, offer a welcomed freedom from the constraints of a gravity power water system. In the event that one or both of these pumping devices becomes readily available for use in rural water supply systems this section has been written. Presented below is some basic information on each which should provide a background for the further research required by the designer before attempting a design.

WINDMILLS

A windmill provides mechanical energy to pump water. The requirements for a satisfactory installation include situation requirements and windmill design requirements. The situation requirements are: a wind velocity of no less than 0.4 km/hr and at least 8 km/hr for 60% of the time; a water source of sufficient safe yield to accommodate sustained pumping over long time periods; and the economic feasibility of a large reservoir to provide for slack wind periods. The windmill design requirements are: the tower must be 4.5 to 6 meters above adjacent wind screening obstacles; the propeller should pivot into any direction of the wind; and a governing device which turns the propeller away from excessive winds should be incorporated. Excessive winds are those greater than 48 to 56 km/hr.

The power required to lift water is determined with the following equation:

$$\text{KILOWATTS OF POWER REQUIRED} = \frac{Q \times H}{102 \times e_p}$$

where:

- Q = desired water flow in LPS
- H = total dynamic head (static head + friction headloss + required residual head) in meters of water
- e_p = pump efficiency (usually 0.25 to 0.60)

The power supplied by the windmill to the pump is given by the approximate equation:

$$\text{KILOWATTS OF POWER SUPPLIED} = \frac{D^2 \times V^3 \times e_w \times e_m}{2000}$$

where:

- D = windmill diameter in meters
- V = wind velocity in meters per second
- e_w = efficiency of windmill (0.4 typically)
- e_m = efficiency of mechanical connection to pump

The preceding equation and efficiency of windmill implicitly account for an assumed average air density and maximum wind-to-blade energy conversion efficiency. Other equations may differ slightly due to different assumptions.

A rough estimate of the windmill size required for a particular situation can be obtained by combining the two equations and retaining the same system of units. Equating kilowatts required to kilowatts supplied yields:

$$\text{REQUIRED WINDMILL DIAMETER} = \sqrt[3]{\frac{20 \times Q \times H}{V \times e_p \times e_w \times e_m}}$$

(meters)

where Q is in LPS, H is in meters of water, V is in meters per second, and typical values for the respective efficiencies should be used.

The designer must provide the windmill manufacturer (who should also supply the pump) with the following information:

- Average wind velocity, velocity range, and duration
- Static pumping head (elevation difference from source to reservoir)
- Desired flow rate
- Residual pressure requirements

The typical range of standard windmill sizes is 1.5 to 9.0 meters in diameter.

HYDRAULIC RAMS

An hydraulic ram is a type of pump which uses the compressive properties of air for lifting water. Requirements for a successful installation include: a source which delivers 3 to 35 times the design peak water demands; suitable topography (see sketch in Figure 38); and the economic feasibility of a large sized reservoir. The diagram shows the important parameters for hydraulic ram design. Refer to Figure 38 for an explanation of the symbols encountered in the following paragraphs.

The hydraulic ram works on the following principle. A quantity of source water is released into the air compression chamber of the hydraulic ram unit. The energy for air compression derives solely from the driving head H_d , mitigated somewhat by friction losses and minor headlosses. The compressed air is stored energy, and this energy is used to lift a portion of the driving flow to the reservoir tank. The air is allowed to expand (decompress), and as it does so it releases its stored energy and displaces a flow of water Q_p into the reservoir. The ratio of driving flow to pumped flow is inversely related to the ratio of driving head to pumping head as shown in the table presented below.

Relation Between Head Ratio and Flow Ratio

(Values reflect typical ram installation as per Reference # 26, page 150)

$H_p/H_d :$	2	3	4	5	6	7	8	9	10	12	15	20
$Q_p/Q_d :$	0.37	0.24	0.17	0.13	0.11	0.10	0.83	0.077	0.067	0.056	0.045	0.029

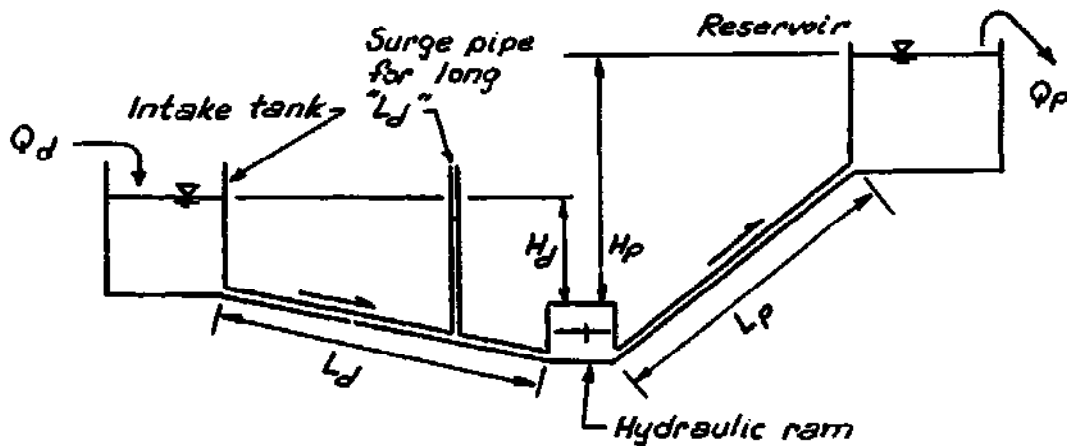
The process of compression and decompression of air in the ram and the accompanying pumping of water can continue unabated with little maintenance or attention for long periods of time. Hence, these units show great promise for use in rural water supplies. Similar units exist which employ the mechanical power of a moving stream of water to pump instead of relying on air compression. There are several innovations incorporated in recently designed hydraulic rams which greatly increase efficiency.

The designer must provide the hydraulic ram manufacturer with the following information when inquiring about an installation:

- Driving flow (Q_d)
- Driving head (H_d)
- Drive line length (L_d)
- Drive line material (whether steel, cast iron, or GI, etc.)
- Required pumped flow (Q_p)
- Available pumping head (H_p)
- Pumping line length (L_p)
- Pumping line material (whether HDP, PVC, or GI, etc.)

Further technical information on both windmills and hydraulic rams is available in the technical libraries of Peace Corps, RA & LDD, WHO, and UNICEF.

FIGURE 38. HYDRAULIC RAM DESIGN PARAMETERS



H_d - Driving Head

H_p - Pumping Head

Q_d - Driving Flow

Q_p - Pumping Flow

L_d - Drive Line Length

L_p - Pumping Length

See Appendix F, page 98, for additional information.

REFERENCES

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It is hoped that this manual will provide most of the information required to complete an acceptable survey and design for a Nepali village water system. Undoubtedly, however, there will be times when other references need to be consulted. Further reading may be of a complementary and/or supplementary nature. Complementary references will provide in depth discussions where the present manual treats a topic too superficially for the reader. Supplementary reference materials, on the other hand, should provide extra technical information being beyond the scope of this manual. Both complementary references, for example texts on hydraulic design and surveying, and supplementary references, like technical publications from pipe manufacturers, are readily available. One should never hesitate to investigate the technical sections in the libraries of WHO, UNICEF, UNDP, USIS, and Peace Corps, and the technical materials in the personal libraries of the engineers and WHO advisors at RA & LDD in Kathmandu.

The following list includes references which were consulted during the compilation of this manual plus other titles useful for survey, design, and construction work in a remote village setting. Not included in this list are the voluminous materials from manufacturers describing their respective products; products ranging from PVC and HDP pipe and fittings to FRP tanks, from windmills to hydraulic rams, from water testing kits to rural water treatment schemes. The reader is encouraged to keep abreast of the burgeoning files of such technical information for products and innovations applicable to particular village water system situations. Manufacturers' technical bulletins are available through the personal libraries of the Peace Corps CWS Program Officer and the engineers and WHO advisors at RA & LDD.

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Appendix A. A SAMPLE DESIGN FOR A RURAL WATER SUPPLY SYSTEM
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This design example is presented to fulfill two purposes. First, it details the step-by-step design procedure for gravity water supply design computations. In this respect it supplements the discussion in Chapter 6. Second, the format for the calculations, information, and the longitudinal profile exemplify the style, detail, and methods which are suggested for design reports submitted to the Rural Area and Local Development Department.

(Refer to longitudinal profile for elevations and pipe lengths)

23 April 1975

Ward : Number 8
Village Panchayat : Laalpaatipaani
District Panchayat : Salyan

Population

Present population = 1600
15 years hence population, i.e. design population = $1600 + \frac{25}{100} \times 1600$
= $1600 + 400 = 2000$

Note: 25% increase over a period of 15 years has been assumed.

Water Demand

(1) Domestic demand @ 45 LPCPD = $2000 \times 45 = 90,000$ liters
(2) Special demands
50 students school @ 20 liters per student
population = $50 \times 20 = 1,000$ liters
91,000 liters

Note: Design criteria -- domestic demand = 45 liters per capita.
Where source is not adequate capacity of system may be based on 230 liters per household per day.

Source

Source of supply : spring
Measured yield rate : 2.0 LPS (Feb. 1975)
Estimated safe yield rate : 1.6 LPS
Safe yield = $1.6 \times 60 \times 60 \times 24 = 138,000$ liters/day

The source chosen is adequate to meet the water needs of the community.

Note: Special care should be exercised in estimating safe yield. It is always safer to measure yield at the end of drought period.

Sizing of Supply Main & Reservoir

If the rate of draw off is greater than the rate of inflow into the system it will be necessary to provide storage to balance the flows. In the case where possible inflow rate is higher than the needed maximum draw off rate no storage will be needed. Furthermore, the supply main will be sized to deliver the needed maximum draw off rate and not the safe yield of source.

In the case of standpost supply the needed maximum draw off rate will be dependent on the number supply points (i.e.) taps.

Generally a flow of 0.225 LPS from a tap is considered as adequate rate of supply. The distribution system is designed for a flow of 0.225 LPS at each of the taps. However, if there are a large number of taps, principle of probability could be supplied say -

Up to 6 taps:	all taps open,	average rate of flow of 0.225 LPS per tap
7 - 9 taps:	85% open,	" " " " " 0.19 LPS " "
10 - 12 taps:	75% open,	" " " " " 0.17 LPS " "
Over 12 taps:	66% open,	" " " " " 0.15 LPS " "

In the system to be designed 10 taps are proposed.
Needed maximum rate of draw off - 10 x 0.17 LPS - 1.7 LPS

It will be seen that the safe yield rate of chosen source is not adequate to meet the needed maximum draw off rate. Therefore, a reservoir will have to be provided to balance rate of draw off against inflow rate. Based on the consumption habit curve presented in the text, the storage needed to balance rates of inflow and draw off can be computed from the schedule given below:

<u>Time needed to deliver the design days requirement</u>	<u>Storage needed to balance flows</u>
24 hours	50% day's demand
18 hours	33% " "
15 hours	25% " "
12 hours	15% " "
9 hours	10% " "

Storage Reservoir (Capacity)

Day's design demand	= 91,000 liters
Rate of inflow	= 1.6 LPS
Time needed to deliver day's demand =	
$91,000 \text{ liters} \times \frac{1 \text{ sec}}{1.6 \text{ liters}} \times \frac{1 \text{ hour}}{3600 \text{ sec}}$	= 15.8 hours

By interpolation in the preceding table, for 15.8 hours the storage requirement is 27.1% of the daily demand. 27.1% of 91,000 liters is 24,600 liters.

Required capacity of reservoir = 24,600 liters = 24.6 m³
say 25 m³

Supply Main

From the profile of the pipe trace it will be seen that a break pressure tank is needed at 1500 meters down slope of the source.

Section: Source to B.P. Tank #1.

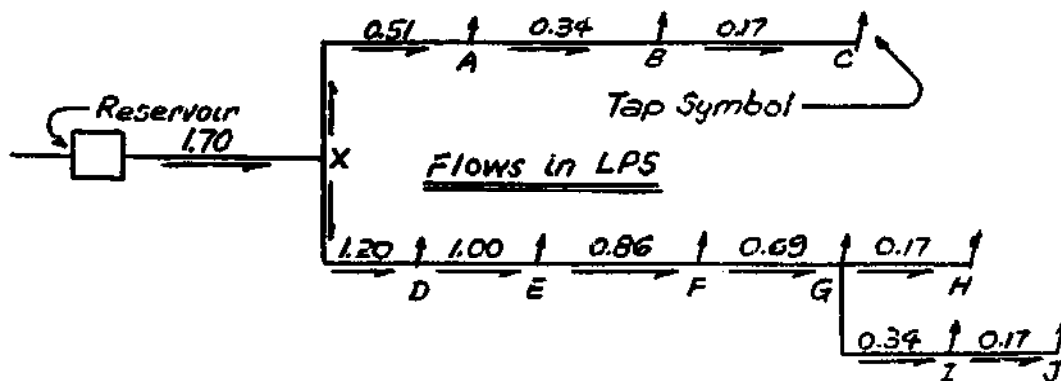
Ground level at source	= 2000 m
Ground level at B.P. Tank site	= 1960 m
Difference in level	= 40 m
Length of section	= 1500 m
Rate of flow	= 1.6 LPS
Head available per 100 m = 40/15	= 2.6 m
Try 63 m.m. pipe - head loss per 100 m	= 1.7 m
<u>Adopt 63 m.m. pipe</u>	

Section: B.P. Tank # 1 to Reservoir

Ground level at B.P. Tank #	= 1960 m
Reservoir level	= 1930 m
Difference in level	= 30 m
Length of section	= 560 m
Rate of flow	= 1.6 LPS
Head available per 100 m - 30/5.6	= 5.36 m
Try 63 m.m. pipe - head loss per 100 m	= 1.7 m
<u>Adopt 63 m.m. pipe</u>	

Note: The carrying capacity of supply main section below the break pressure tank is greater than that of the pipe feeding the break pressure tank. A control valve should be provided on the downstream side of break pressure tank to balance outflow with inflow.

FIGURE 39. SCHEMATIC FLOW DIAGRAM



Section: Reservoir to X

Water level of reservoir	= 1930 m
Level at branch point X	= 1910 m
Available head	= 20 m
Length of section	= 800 m
Head available per 100 m = 20/8	= 2.5 m
Rate of flow	= 1.7 LPS

Try 63 m.m. pipe - head loss per 100 m = 1.9 m
Loss of head in section = 1.9×8 = 15.2 m
Level of hydraulic grade at X = 1930 - 15.2 = 1914.8
Residual head at X = 1914.8 - 1910 = 4.8 m
Adopt 63 m.m. pipe for section Reservoir to X

Section: X to A (Hereafter written typically XA)

Level of hydraulic grade at X = 1914.8
Tap level at A = 1895.0
Available head = 19.8 m
Length of section = 475 m
Flow in XA = 0.51 LPS
Head available per 100 m = $19.8/4.75$ = 4.17 m
Try 50 m.m. pipe - head loss per 100 m = 0.63 m
Head loss in 475 m = $4.75 \times .63$ = 3.0 m
Level of hydraulic grade at A = 1914.8 - 3.0 = 1911.8 m
Residual head at A = 1911.8 - 1895.0 = 16.8 m
Adopt 50 m.m. pipe for section XA

Section: AB

Level of hydraulic grade at A = 1911.8
Tap level at B = 1865.0
Available head = 46.8 m
Length of section = 1850 m
Head available per 100 m = $46.8/18.5$ = 2.53 m
Flow in section = 0.34 LPS
Try 50 m.m. pipe - head loss per 100 m = 0.33 m
Head loss in 1850 m = $18.5 \times .33$ = 6.1 m
Level of hydraulic grade at B = 1911.8 - 6.1 = 1905.7 m
Residual head at B = 1905.7 - 1865.0 = 40.7 m
Adopt 50 m.m. pipe for section AB

Section: BC

Level of hydraulic grade at B = 1905.7
Tap level at C = 1855.0
Available head = 50.7 m
Length of section = 460 m
Head available per 100 m = $50.7/4.6$ = 11.0 m
Flow in section = 0.17 LPS
Try 32 m.m. pipe - head loss per 100 m = 1.0 m
Head loss in 460 m = 4.6×1.0 = 4.6 m
Level of hydraulic grade at C = 1905.7 - 4.6 = 1901.1 m
Residual head at C = 1901.1 - 1855.0 = 46.1 m
Adopt 32 m.m. pipe for section BC

Section: XD

Level of hydraulic grade at X = 1914.8 m
Tap level at D = 1880.0 m
Available head = 34.8 m
Length of section = 980 m

Head available per 100 m = 34.8/9.8	= 3.55 m
Flow in section XD	= 1.2 LPS
Try 63 m.m. pipe - head loss per 100 m	= 1.04 m
Head loss in 980 m = 9.8 x 1.04	= 10.2 m
Level of hydraulic grade at D = 1914.8 - 10.2	= 1904.6 m
Residual Head at D = 1904.6 - 1880.0	= 24.6 m

Adopt 63 m.m. pipe for section XD

Section: DE

Level of hydraulic grade at D	= 1904.6 m
Tap level at E	= 1875.0 m
Available head	= 29.6 m
Length of section	= 390 m
Head available per 100 m = 29.6/3.9	= 7.6 m
Flow in section	= 1.0 LPS
Try 50 m.m. pipe - head loss per 100 m	= 5.8 m
Head loss in 390 m = 3.9 x 5.8	= 22.6 m
Level of hydraulic grade at E = 1904.6 - 22.6 m	= 1882.0 m
Residual head at E = 1882.0 - 1875.0	= 7.0 m

Adopt 50 m.m. pipe for section DE

Note: This pipe size is revised to 63 m.m. below.

Section EF

Level of hydraulic grade at E	= 1882.0 m
Tap level at F	= 1870.0 m
Available head	= 12.0 m
Length of section	= 450 m
Head available per 100 m = 12.0/4.5	= 2.67 m
Flow in section EF	= 0.86 LPS
Try 50 m.m. pipe - head loss per 100 m	= 3.8 m

Note: As the available head per 100 m is not adequate it will be necessary to use larger diameter pipe. Instead of using it in this section try 63 m.m. pipe for section DE whereby residual head downstream of D can be raised. This will follow the section DX where 63 m.m. pipe has been adopted.

Section DE (Revised)

Flow in section DE	= 1.0 LPS
Try 63 m.m. pipe - head loss per 100 m	= .70 m
Head loss in 490 m = 3.9 x .7	= 2.7 m
Level of hydraulic grade at E = 1904.6 - 2.7 m	= 1901.9 m
Residual head at E = 1901.9 - 1875.0	= 26.9 m

Adopt 63 m.m. pipe for section DE

Section EF

Level of hydraulic grade at E	= 1901.9 m
Tap level at F	= 1870.00 m
Available head	= 31.9 m

Length of section	= 31.9 m
Head available per 100 m = $31.9/4.5$	= 7.1 m
Flow in section	= 0.86 LPS
Try 50 m.m. pipe - head loss per 100 m	= 3.5 m
Head loss in 450 m = 4.5×3.5	= 15.7 m
Level of hydraulic grade at F = $1901.9 - 15.7$	= 1886.2 m
Residual head at F = $1886.2 - 1870$	= 16.2 m
<u>Adopt 50 mm pipe for section EF</u>	

Section FG

Water level of reservoir	= 1930 m
--------------------------	----------

Note: It will be seen that pipe section beyond chainage 5750 meters will be subjected to static pressure higher than working pressure of pipe which is 6 kg/sq. cm (i.e.) 200 feet. Therefore, provide a break pressure tank at a suitable location. The most suitable location is at 5675 m downslope from the source.

Section: F to B.P. Tank # 2

Level of hydraulic grade at F	= 1886.2 m
Level at B.P. Tank # 2	= 1860.0 m
Available head	= 26.2 m
Length of section	= 1000 m
Head available per 100 m = $26.2/10$	= 2.62 m
Flow in section	= 0.69 LPS
Try 50 m.m. pipe - head loss in 100 m	= 1.2 m
Head loss in 1000 m = 1.2×10	= 12 m
Level of hydraulic grade at B.P. Tank = $1886.2 - 12$	= 1874.2 m
<u>Adopt 50 m.m. pipe for section F to B.P. Tank # 2</u>	

Note: It will be seen that a control valve should be provided upstream of B.P. Tank to provide the necessary head upstream to supply the taps D, E and F.

Section: B.P. Tank # 2 to G

Level of B.P. Tank # 2	= 1860.0
Tap level at G	= 1830.0
Available head	= 30.0
Length of section	= 450 m
Head available per 100 m = $30/4.5$	= 6.6 m
Flow in section	= 0.69 LPS
Try 50 m.m. pipe - head loss per 100 m	= 1.2 m
Head loss in 450 m = 4.5×1.2	= 5.4 m
Level of hydraulic grade at G = $1860.0 - 5.4$	= 1854.6 m
Residual head at G = $1854.6 - 1830$	= 24.6 m
<u>Adopt 50 m.m. pipe for section B.P. Tank # 2 to G</u>	

Section CH (Branch from G)

Level of hydraulic grade at G	= 1854.6 m
Tap level at H	= 1840.0 m
Available head	= 14.6 m

Length of section	=	410.	m
Head available per 100 m - 14.6/4.1	=	3.6	m
Flow in section	=	0.17	LPS
Try 32 m.m. pipe - head loss per 100 m	=	1.0	m
Head loss in 410 m = 4.1 x 1.0	=	4.1	m
Level of hydraulic grade at H = 1854.6 - 4.1	=	1850.5	m
Residual head at H = 1850.5 - 1840.0	=	10.5	m
<u>Adopt 32 m.m. pipe for section GH</u>			

Note: Flow from this branch will need to be controlled either with corporation cock or gate valve.

Section: GI (Branch from G)

Level of hydraulic grade at G	=	1854.6	m
Tap level at I	=	1835.0	m
Available head	=	19.6	m
Length of section	=	680	m
Head available per 100 m = 19.6/6.8	=	2.9	m
Flow in section	=	0.34	LPS
Try 50 m.m. pipe - head loss per 100 m	=	.30	m
Head loss in 680 m = 6.8 x .3	=	2.0	m
Level of hydraulic grade at I = 1854.6 - 2.0	=	1852.6	m
Residual head at I = 1852.6 - 1835.0	=	17.6	m
<u>Adopt 50 m.m. pipe for section GI</u>			

Section: IJ

Level of hydraulic grade at I	=	1852.6	m
Tap level at J	=	1840.0	m
Available head	=	12.6	m
Length of section	=	300	m
Head available per 100 m = 12.6/3	=	4.2	m
Flow in section	=	0.17	LPS
Try 32 m.m. pipe - head loss per 100 m	=	1.0	m
Head loss in 300 m = 3 x 1.0	=	3.0	m
Level of hydraulic grade at J = 1852.6 - 3.0	=	1849.6	m
Residual head at J = 1849.6 - 1840.0	=	9.6	m
<u>Adopt 32 m.m. pipe for section IJ</u>			

Summary of Size of Elements

1. Intake chamber - (Adopt typical design)
2. Supply Main -
 - (a) Intake chamber to B.P. Tank # 1 - 63 m.m. pipe
 - (b) B.P. Tank # 1 to Reservoir - 63 m.m. pipe
 - (c) B.P. Tank # 1 - No ball valve needed. Provided control valve on down stream side of B.P. Tank # 1.
 - (d) B.P. Tank # 2 - Self closing ball valve required
3. Storage reservoir - Provide tank of capacity 25 m³
4. Distribution system -
 - (a) Storage reservoir to junction X - 63 m.m. pipe
 - (b) Sections XA, AB - 50 m.m. pipe
 - (c) Section BC - 32 m.m. pipe
 - (d) Sections XD, DE - 63 m.m. pipe
 - (e) Sections EF, F to B.P. Tank # 2, B.P. Tank # 2 to G - 50 m.m. pipe
 - (f) Section GH - 32 m.m. pipe
 - (g) Section GI - 50 m.m. pipe
 - (h) Section IJ - 32 m.m. pipe.

FIGURE 40. KEY PLAN OF PIPE SIZES

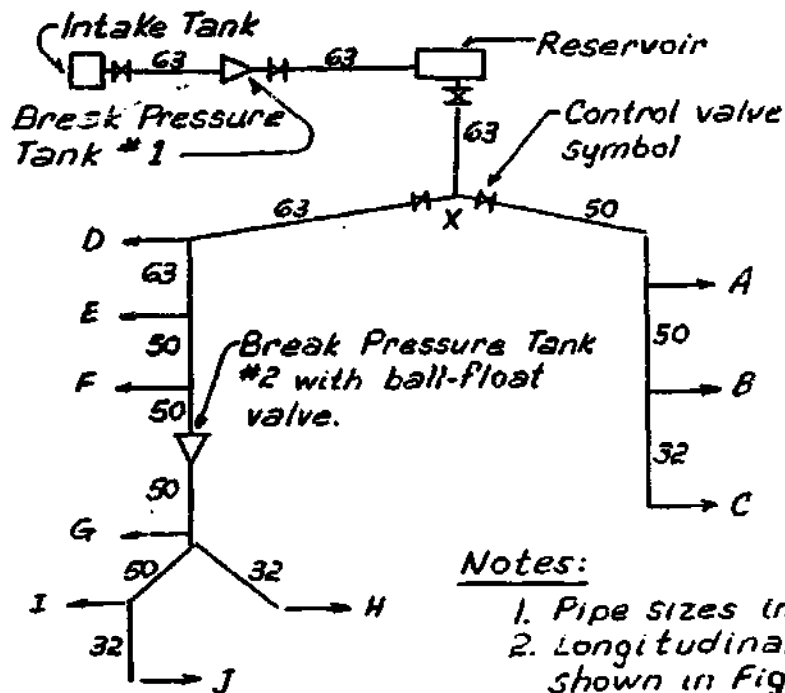
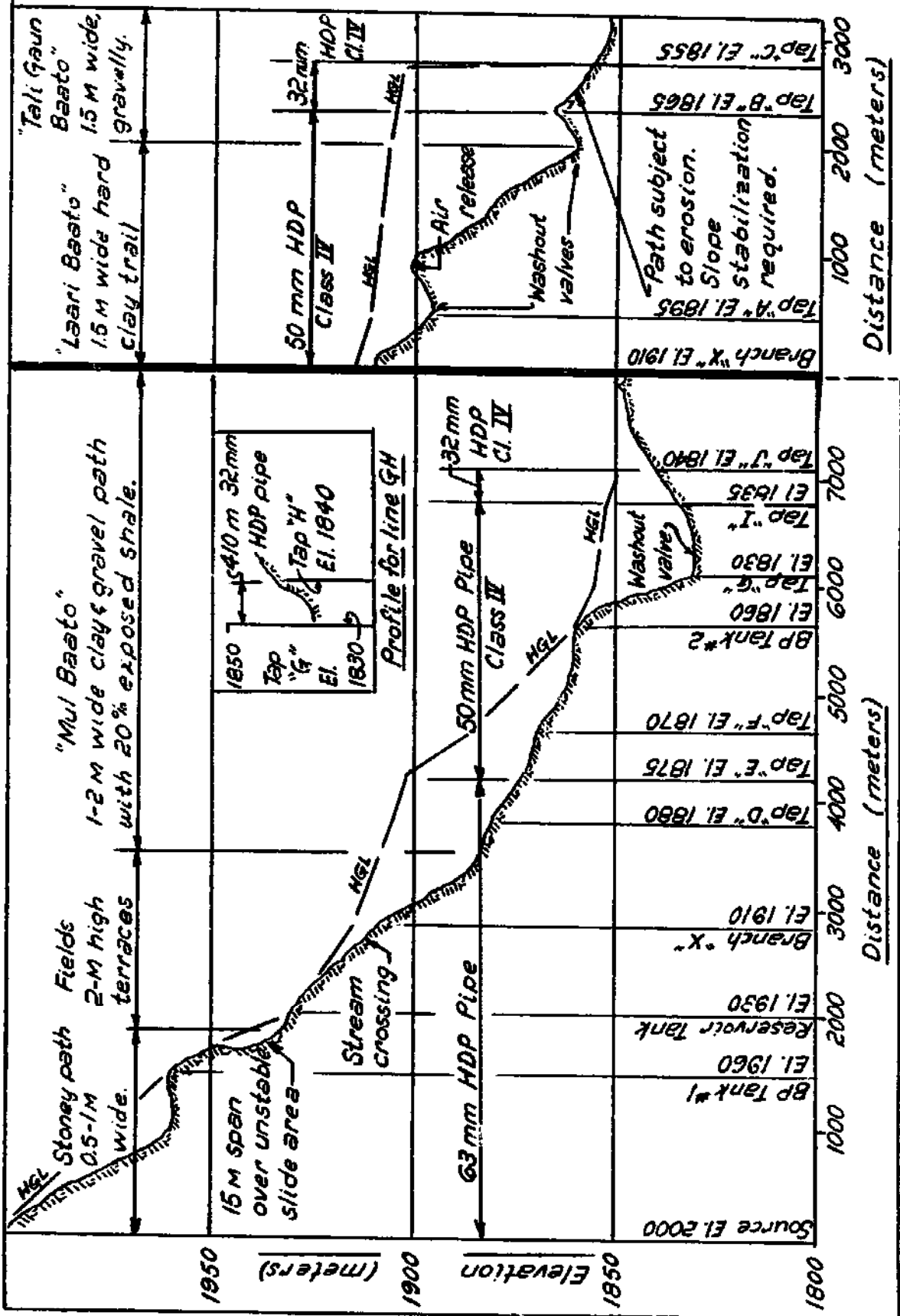


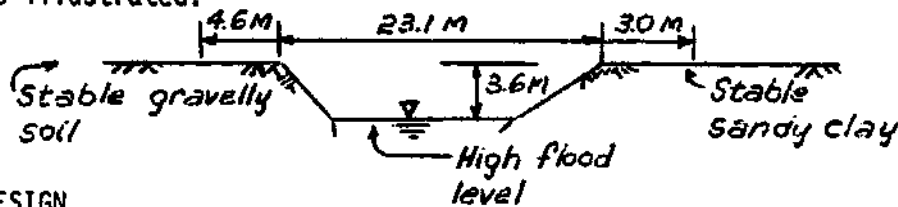
FIGURE 41. LONGITUDINAL PROFILE (FOR DESIGN EXAMPLE)



Appendix B. A SAMPLE DESIGN FOR A CABLE SUSPENSION

The following sample design of a cable suspension to support a flexible pipe is included due to frequent need for suspending pipelines in rural Nepal water systems. Landslides and unstable ground, stream crossings, cliff traverses, and monsoon floodways are often encountered in pipe routes and an overhead pipe suspension is generally a feasible solution for passing the area.

SITUATION - Design a cable to span a 23.1 meter stream with a 63 mm HDP pipe as illustrated.



DESIGN

1. Basics - Length of span = 4.6 + 23.1 + 3.0 = 30.7m Say 31.m
 - Right anchor in sandy clay, left anchor in gravel.
 - Must support full pipe, sheathing and binding wire.

2. Cable loading - (Load uniform throughout)

- . Pipe: 63 mm OD @ 0.80 kg/m ----- 0.80 kg/m
- . Bamboo Sheathing: ID = 70 mm
OD 76 mm

$$\text{Cross-sectional area} = \frac{2\pi (70 + 76) 1/2 \times (76-70)}{2}$$

$$= 688 \text{ mm}^2 = 6.88 \text{ cm}^2$$

Assume a specific weight for dry bamboo of 0.47 g/cm³.

$$\therefore \text{in 1 meter of sheath length the weight is } \frac{0.47 \text{ g}}{\text{cm}^3} \times 6.88 \text{ cm}^2 \times \frac{100 \text{ cm}}{\text{m}}$$

$$= 0.33 \text{ kg/m}$$

. Water:

$$\text{Cross-sectional area of pipe interior} = 24.4 \text{ cm}^2$$

$$\text{weight of water per meter is } 1.0 \text{ g/cm}^3 \times 24.4 \text{ cm}^2 \times \frac{100 \text{ cm}}{\text{m}}$$

$$= 2.44 \text{ kg/m}$$

- . Binding wire: Ref: "USS Tiger Brand Wire Rope Engineering Hand Book"

*7 wire (1/32"Ø) Corrosion Resisting Steel @ 25 LBS /1000 FT
Assume 3 wraps per foot as shown:



Wrapping circumference will be about 20 cm

$$9.3 \text{ wraps per meter @ } 20 \text{ cm/wrap} = 186 \text{ cm/m}$$

$$186 \text{ cm/m} \times \frac{25\#}{1000 \text{ Ft}} \times \frac{1 \text{ Ft}}{30.5 \text{ cm}} \times \frac{1 \text{ kg}}{2.2\#} = 0.006 \frac{\text{kg}}{\text{m}} \quad \underline{0.006 \text{ kg/m}}$$

CABLE DESIGN (Continued)

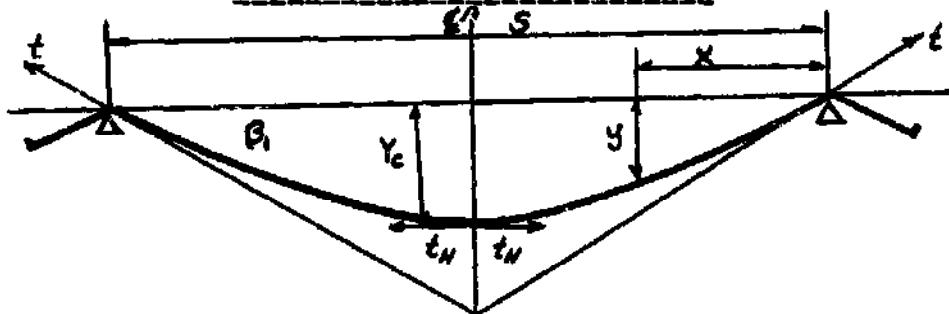
. Total cable loading (without cable weight)

Pipe	0.80
Bamboo sheath	0.33
Water	2.44
Binding wire	0.006
	<u>3.576 kg/m</u>

Add, say, 15% for wind. (No ice load expected)
 $3.576 \times 1.15 = 4.11 \text{ kg/m}$
 =====

. Cable weight: The unit weight of the cable must also be included, but not knowing the required size of cable a trial size of 3/8" ϕ Type 6x7 Wire Rope. From the above quoted reference, 6x7 - 3/8" cable weighs 0.21 pounds per foot and has a breaking straight of 5.10 (Plow Steel) tons.

FIGURE 42. RELATIONSHIPS FOR LEVEL,
 UNIFORMLY LOADED CABLE SPAN



W = weight of loading per unit length
 L = length of cable
 K = sag ratio Y_c/S

$$y_c = \frac{WS^2}{8t_H}$$

$$t = \frac{WS^2}{8Y_c}$$

$$t = t_H \sec(\theta)$$

$$\tan(\theta) = \frac{4Y_c}{S}$$

$$L = S \left(1 + \frac{8}{3} K^2\right)$$

Now a value for "W" can be used based on a trial size of 3/8" cable. 6x7 - 3/8" ϕ 0.21 #/FT = 0.29 kg/m
 Increase by 15% for wind = $0.29 \times 1.15 = 0.335 \text{ kg/m}$

"W" = $0.34 + 4.11 = 4.45 \text{ kg/m}$
 =====

CABLE DESIGN (Continued)

3. Cable size design -

The above relationships are now to be used. Water level in the stream rises to within 3.6 m of the bank. Therefore, be safe and allow a maximum sag (deflection) Y_c of 3.0 m

$$\begin{array}{l}
 Y_c = 3.0 \text{ m} \\
 S = 31 \text{ m} \\
 W = 4.45 \text{ kg/m}
 \end{array}
 \left| \rightarrow \right.
 \begin{array}{l}
 t_H = WS^2/8/Y_c^2 \\
 = \frac{4.45 (31)^2}{8 (3.0)} \text{ kg} = 178 \text{ kg}
 \end{array}$$

$$\tan(\beta) = \frac{4 Y_c}{S} = \frac{4(3.0)}{31} = 0.387 \quad \therefore \beta = 21^\circ 10'$$

$$\sec(\beta) = \sec(21^\circ 10') = 1.072$$

$$\text{Cable tension} = t = t_H \sec(\beta) = 178 (1.07) = 191 \text{ Kg}$$

=====

The 6x7 3/8" Plow Steel Wire Rope has a breaking strength of 5.10 tons = 4630 kg

For a safety factor of 4.0, the strength is $\frac{4630}{4} = 1155 \text{ kg}$

=====

The 3/8 cable is much stronger than needed. (i.e. LOAD = 191 Kg vs CAPACITY = 1155 kg)

Try a smaller cable, say 1/4" \emptyset

4. Cable size design -- Trail # 2.

From "USS Tiger Brand Wire Rope Engineering Handbook"

1/4" \emptyset 6 x 7 Monitor Steel Wire Rope

0.094 #/FT

2.64 tons breaking strength.

1/4" \emptyset 6 x 7 Mild Plow Steel

2.00 tons breaking strength

NOTE: be conservative and choose lower strength cable.

$$\text{Cable weight} = 0.094 \frac{\#}{\text{FT}} = 0.13 \text{ kg/m} \times 1.15 = 0.15 \text{ kg/m}$$

$$\text{New value for "W"} = 0.15 + 4.11 = 4.26 \text{ kg/m (wind included)}$$

Y_c and S remain unchanged

$$t_H = \frac{4.26(31.)^2}{8 (3.0)} = 170 \text{ kg}$$

$$t = 170(1.07) = 182 \text{ kg}$$

Cable capacity = 2 tons = 1818 kg (Breaking strength)

For safety factor of 4.0: Working Capacity = 455 kg

→ Adopt 1/4" \emptyset CABLE. This design will provide an adequate margin of safety for locally available cables.

5. Required cable length -- $L = \frac{S}{3} (1 + \frac{8 Y_c^2}{S^2})$; $K = \frac{Y_c}{S}$

$$L = 31 \left(1 + \frac{8 (3.0/31)^2}{3} \right) = 31.7 \text{ m}$$

Say 34 m to account for anchoring.

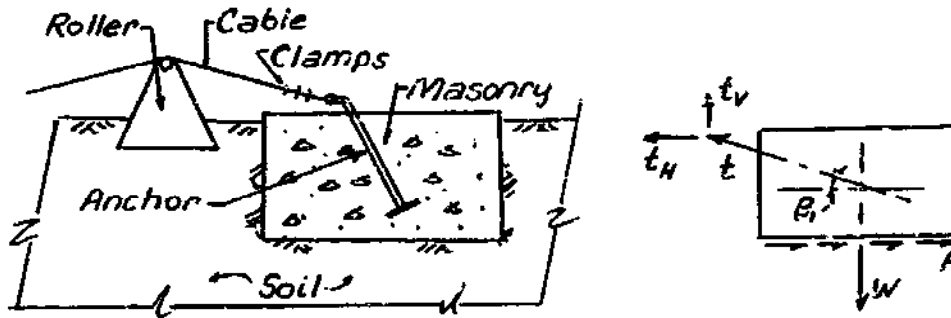
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CABLE DESIGN (Continued)

6. Anchorage design

The following detail and free body diagram illustrate the design considerations.

FIGURE 43. CABLE ANCHORAGE DESIGN



In this design it is assumed that only the frictional resistance between the anchor block and soil holds the cable. The line of tensional force from the cable must pass through the center of gravity of the block. $F = W$ times the coefficient of static friction, μ . From the "Indian Practical C.E. Handbook,"
 for masonry on gravel = 0.60 (Left Anchor)
 for masonry on sandy clay $0.4 + 0.5/2 = .45$ (right anchor)

The appropriate force relationship is $W = \frac{t_H (SF)}{\mu} + t_v$
 where "SF" is the safety factor (using SF=2.0). μ
 t_H and t_v are the horizontal and vertical components, respectively,
 of t , $t_v = 170$ kg (from preceding page)
 $t_H = t \times \sin(\beta) = 182 \sin(21^{\circ}10') = 182(0.345) = 63$ kg

Right anchor block: $W_R = \frac{170(2)}{0.45} + 63 = 755 + 63 = \underline{818}$ kg

Left anchor block: $W_L = \frac{170(2)}{0.6} + 63 = 567 + 63 = \underline{630}$ kg

The volume of each anchor is calculated with a density of 2.4 g/cc assumed for stone masonry.

Right anchor block: $\frac{1 \text{ cm}^3}{2.4 \text{ g}} \times 818000 \text{ g} = \underline{341000}$ cm³

Left anchor block: $\frac{1 \text{ cm}^3}{2.4 \text{ g}} \times 630000 \text{ g} = \underline{263000}$ cm³

Taking the cube root of each volume leads to final dimensions of 70 x 70 x 70 cm for the right block and 65 x 65 x 65 cm for the left block.

7. Summary of Design -

- Cable: 34 meters of 1/4" ϕ 6x7, 2.0 ton Capacity wire rope
- Right anchor: 70 cm cube of cement mortar and stone masonry.
- Left anchor: 65 cm cube " " " " " "

Appendix C. TECHNICAL MEMORANDUM ON SAFE YIELD

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DESIGN AND HYDRAULIC GRADE LINE CONCEPT

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The memorandum is reproduced in original form as an appendix to supplement the text discussion of the present topics. The design meeting which this memorandum summarizes was held to clarify the concepts, answer designers' questions, and promote a more uniform approach to design among those working for RA & LDD. Because this document explicitly addresses many of the designers' questions, the author feels its inclusion here, in original format, is warranted.

TECHNICAL MEMORANDUM

SUBJECT: Summary of American Peace Corps/Nepal - Community Water Supply Program Design Meeting.

The following information on safe yield design and hydraulic gradient summarizes the discussion in the meeting of 24 December 1974 which was held between Mr. Suphi (WHO), Paul Wishinski (Peace Corps Program Officer) and volunteers in order to clarify these concepts.

SAFE YIELD DESIGN

The safe yield of a water source is that flow rate which can be extracted over a long period of time without depleting the source. For CWS projects in Nepal the safe yield is essentially the minimum flow of the source in a typical year. A proper hydraulic design will take only the safe yield (or less) from the source. Designing the water system for the safe yield flow insures that:

- a) a constant supply of water is always delivered to the village;
- b) the pipelines will always flow full as designed; and
- c) the pipeline is economical because the smallest pipe size which can satisfy the water demand is used.

Designs for rural water supply systems in the CWS program should conform as closely as possible to this safe yield design method. There are many situations, however, when the safe yield is significantly less than the projected water demand at the end of the design period. A safe yield which supplies from 230 liters per household per day to 45 liters per person per day, plus any special water demands, (all computed for the end of the design period) is acceptable from a public health standpoint. A water system whose sources produce safe yields totalling less than an acceptable supply by public health standards should still be designed. The final decision for construction of such a system lies with RA & LDD officials.

In situations where the yearly source flow varies so that projected demand is met or exceeded for some time but the safe yield is less than projected demand, then consider the following criteria.

1. If source flow meets projected demand for most of the year (at least 8 months) yet drops to a safe yield which falls below an acceptable level,

then design for a flow somewhat less than that expected for the majority of the year.

2. If the source only produces enough water to meet an acceptable projected demand for a short time during the rainy season, then design for the safe yield flow. It is not justifiable to install a large pipe for the short term larger flow because water demands are always less than average during the rainy season.

The first situation necessitates a compromise on the safe yield design method. When this situation is encountered remember these three points while deciding how much water to deliver to the village.

1. Because designs are based on demand at the end of the design period (15, 20 or 25 years), then per capita supply will be larger in preceding years.
2. For a given pipe route, carrying capacity of pipes will increase approximately with the square of the diameter. Pipe cost, also increases rapidly; rising from 250-300% when the diameter is doubled. In addition, HDP pipe cost increases reflect petroleum price increases, as polyethylene is a petroleum-based material.
3. If a pipeline is designed for a flow in excess of the safe yield, then the designer must investigate two flow conditions. There will be a separate hydraulic gradient for both the design flow and the safe yield flow. Ensure that the hydraulic gradient does not pass below the pipeline and that fluid velocity at safe yield flow is not less than 0.6 meters per second. (At lower velocities sedimentation of entrained material occurs and the pipe will tend to clog.)

The safe yield design method should only be deviated from when it seems very unreasonable for the particular village circumstances. The design report, in any case, should explicitly state the basis for design and present justification for that basis.

HYDRAULIC GRADIENT

This information is meant to supplement and provide further explanation for the recent technical memorandum concerning hydraulic gradient. Refer to the schematic diagrams of pipelines and hydraulic gradients as shown at the end of this page. Hydraulic gradient and hydraulic grade line are synonymous and will be abbreviated as HGL, and referring to the diagrams: HGL-B, etc.

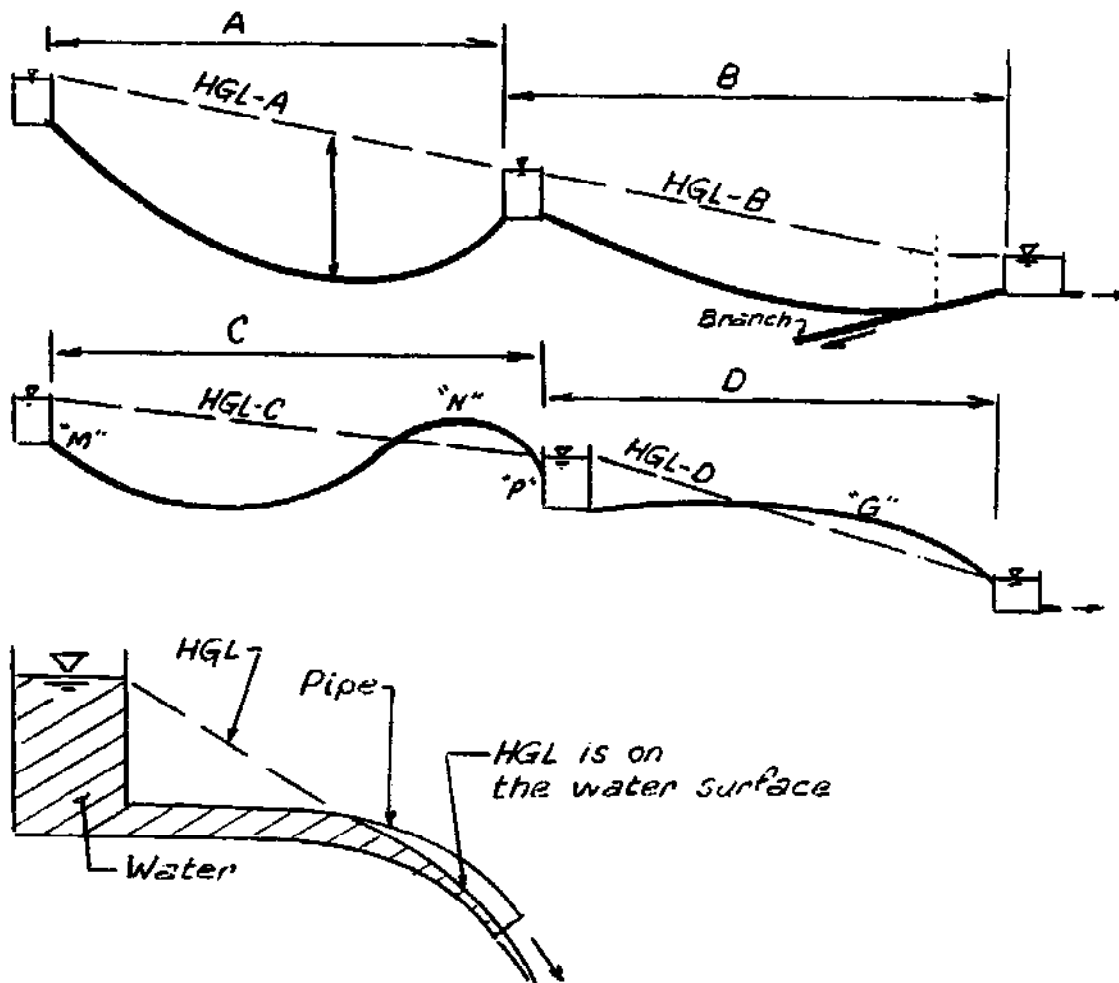
The HGL represents the pressure head in a pipeline. A water surface in contact with the atmosphere has zero pressure head, so the HGL is on the water surface. The pipe in section A is of uniform diameter and carries a constant flow, therefore the loss of pressure head is uniform along the pipe. HGL-A must connect the two water surfaces because the pressure head at each end is zero and the pressure head loss is uniform. Note that even though section A is classified as an "open-system", pressure head "H" can not exceed 200 feet with class III HDP pipe.

HGL-B has a change in slope because the flow is reduced at the branch. HGL slopes will also change when a different size of pipe is used or when a different type of pipe is used.

HGL-C in section C forebodes a problem because the pipe comes to a peak above the HGL. If the rise above the HGL is high enough the water will vaporize (boil) due to the lower than atmospheric pressure which exists in the pipe above the HGL-C. As the water boils and entrained air is sucked from the water an air pocket forms and the siphoning action, which was pulling the water over the peak, ceases to exist. At this time flow stops from point M to point N and water dribbles out of the pipe from N to point P. When air can enter the pipe at point P the pressure at point N becomes atmospheric and flow starts again from point M to point P. Flow continues until the water column breaks again and this cycle repeats. Thus, in section C, water will "glop" out of the pipe at a low flow rate. Locating a break tank at point N and increasing the pipe sizes as required will preclude this undesirable condition.

HGL-D in section D also passes above the pipe, and again represents a flow problem. In this case water will flow through the pipe at a constant rate, but the pipe will not flow full and the design flow (based on a full pipe condition) will not be achieved. Because the pipe becomes steeper as it runs along the convex land surface, the flow of water separates from the top of the pipe as the water velocity increases along the constantly steeper pipe slope.

FIGURE 44. HYDRAULIC GRADE LINE EXAMPLES



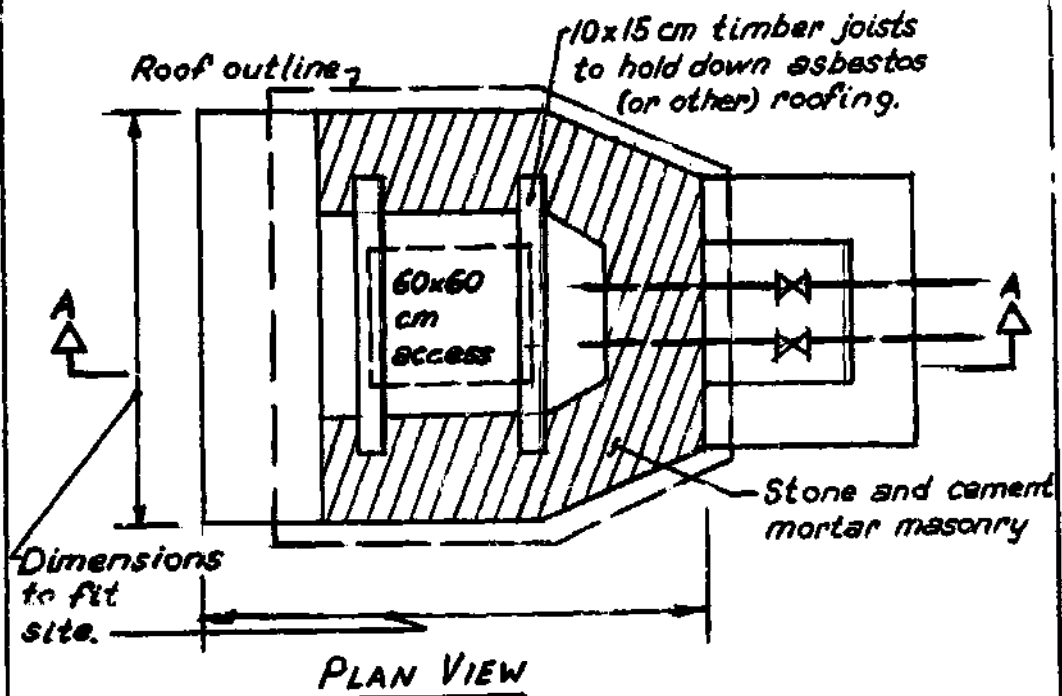
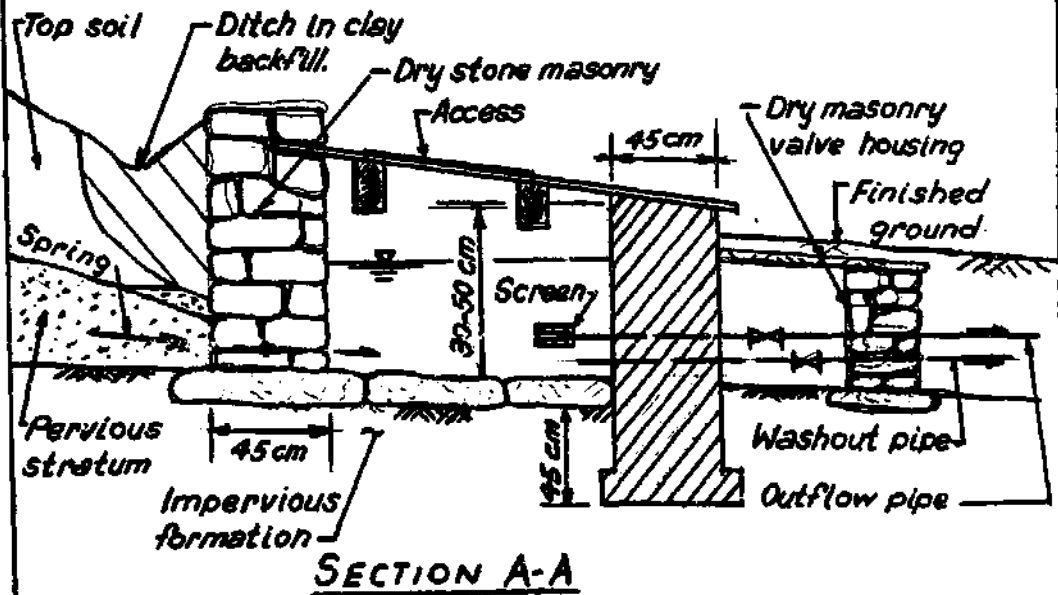
The figure at the bottom of the page shows how the depth of water diminishes in the pipe and how the HGL-D becomes coincident with the water surface in the pipe. While this condition is not as serious as the situation in section C, it should still be avoided in order to get full design flow and maximum benefit from the pipe. Place hydraulic gradient control structures (i.e. small tank or standpipe) at point "G" to resolve the problem.

NOTES:-

Appendix D. (Supplement to Third Revised Edition)

SPRING INTAKE

(Refer to Page 5)



NOTES:- Appendix E. (Supplement to Third Revised Edition)

DAILY WATER DEMAND PATTERN
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(Refer to page 48.)

The daily water demand pattern presented on pages 48 and 49 has been and continues to be used in village water supply design work in Nepal. However, an investigation by this writer ("Daily Water Demand in a Rural Nepalese Hill Village", July 1976, Peace Corps, Nepal) has shown one village in western Nepal to have a water demand scheme as follows:

5 am	-	7 am	:	10%	daily demand
7 am	-	11 am	:	25%	daily demand
11 am	-	1 pm	:	35%	daily demand
1 pm	-	5 pm	:	20%	daily demand
5 pm	-	7 pm	:	10%	daily demand

The village is typical of small, remote western hill region communities in its daily activities, agricultural practice, population density, housing, and land use. Other villages seem to conform, generally, to this single, mid-day peak water use pattern in the writer's experience, although only one village has been studied in any detail.

Appendix F. (Supplement to Third Revised Edition)

NOTES:-

FEASIBILITY SURVEY FOR HYDRAULIC RAMS

The hydraulic ram pump has great potential for use in village water systems. It is relatively easy to determine the feasibility of a particular site if one understands the basic types hydram installations and the fundamental techniques involved.

It is necessary to calculate the following variables:

1. Minimum daily quantity of water required.
2. Amount of water available to power the hydram (Qd).
3. Vertical fall from drive pipe intake to the hydram (Hd), which is between one and fifteen meters.
4. Drivepipe length (Ld), which is usually taken to be 4 or 5 times the vertical fall, or between 150 to 1000 times the pipe diameter, with an optimum ratio of length to diameter of 500.
5. Vertical lift (Hp) (should not exceed 100m)
6. Length of delivery pipe (Lp)

A useful formula for determining the output Qp is:
$$Qp = \frac{Hd \times Od \times 2}{Hp \times 3}$$

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

The water from the source to the hydram must travel through the drivepipe which must be: rigid (i.e. G.I. pipe), as straight as possible, securely anchored, water tight and without any air. This usually requires the construction of a small supply tank between the source and the pump.

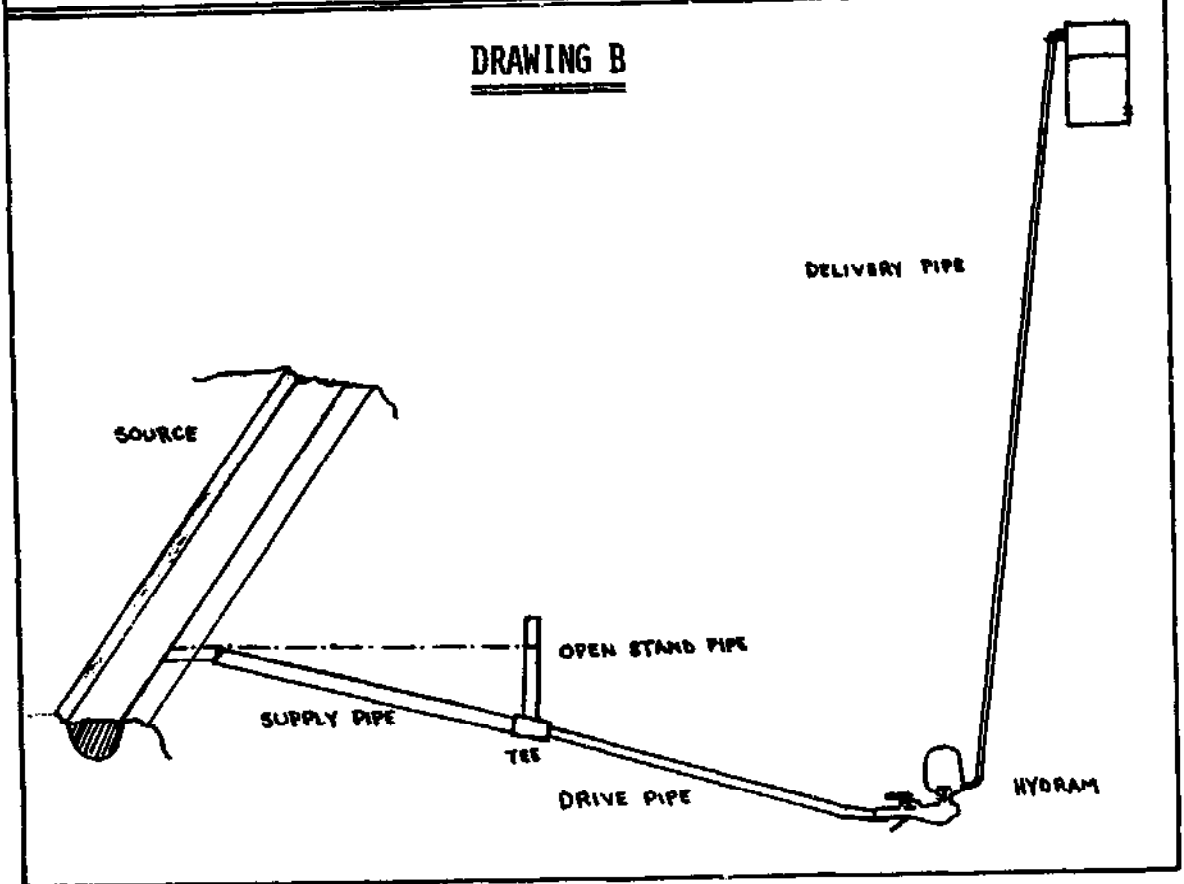
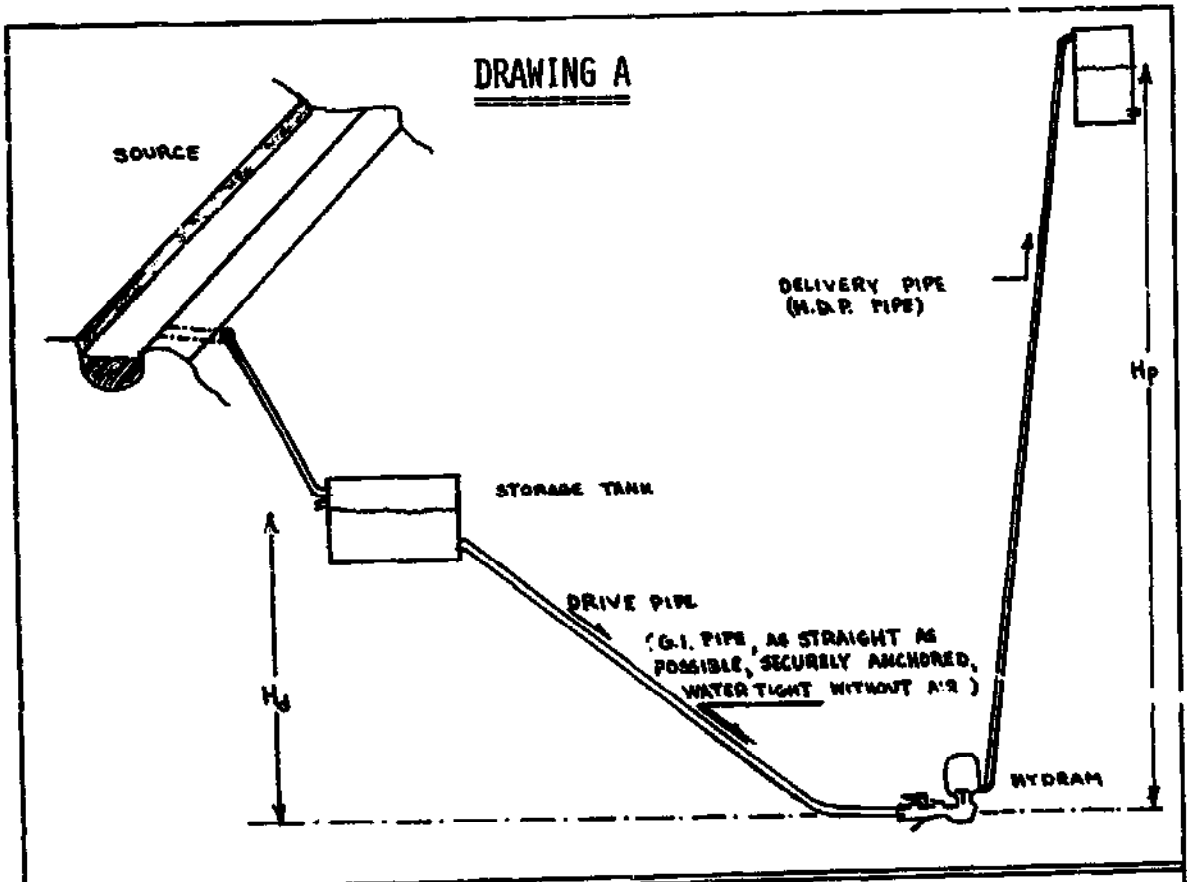
If the source is a river or stream it is almost always necessary to divert to a storage tank via a canal or pipe, see drawing A.

An alternative to a storage tank to ensure that the drivepipe has no air in it, is to take a pipe from the source and connect it to the drivepipe via a tee joint with an open standpipe rising to the hydraulic gradient. The supply pipe must be one size larger than the drivepipe to ensure that the standpipe is always full, preventing air from entering the drivepipe. The technique of using an open stand pipe is also useful when the drivepipe is excessively long. Correct placement of the standpipe will effectively shorten the drivepipe. See drawing B.

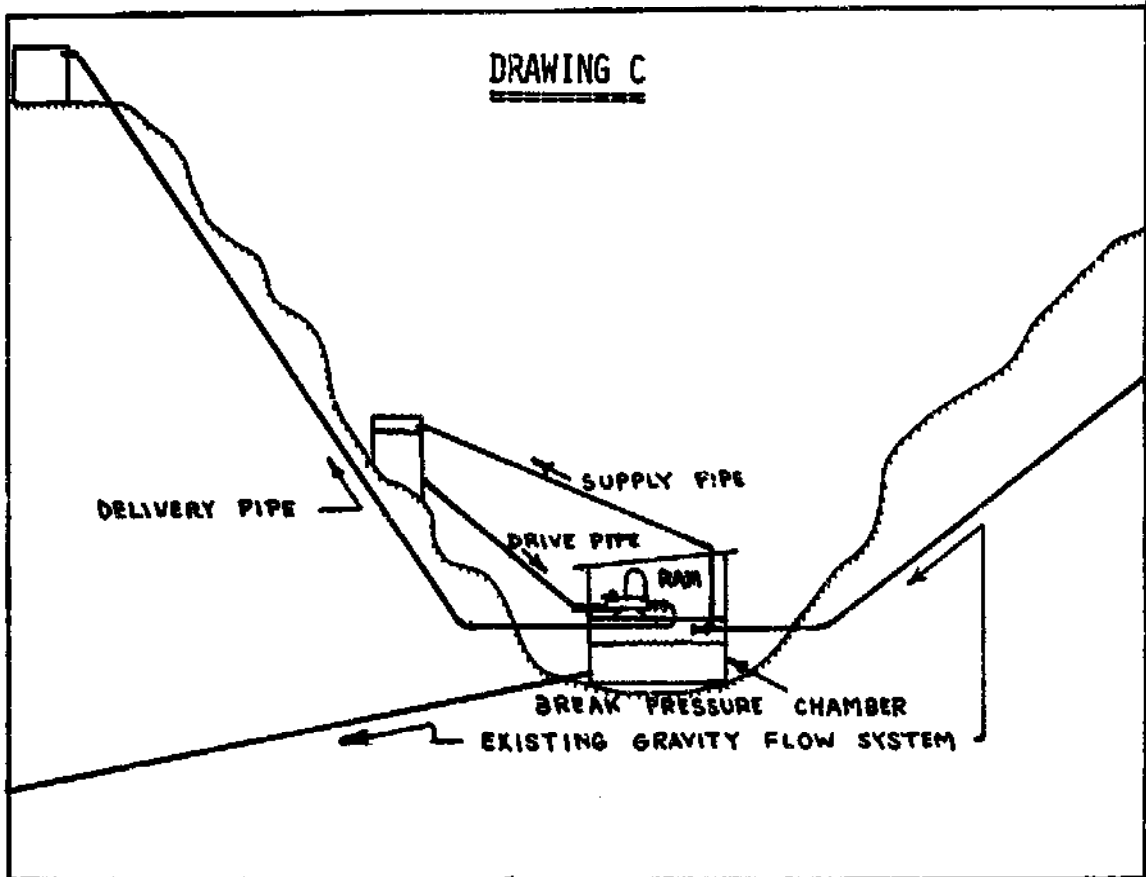
If the water source is a spring, a catchment tank must be constructed to ensure a sufficient level of water above the intake of the drive pipe.

If the hydram is to be used in conjunction with a gravity flow system it is advisable to incorporate the hydram in the break pressure chamber to avoid any waste. A separate head must be established to ensure that the drivepipe is properly designed with regard to diameter, length, angle, and airtightness. This requires another supply chamber above the break pressure chamber or the use of a larger supply pipe connected to a smaller drive pipe via an open tee joint which should be over-flowing. See drawing C.

NOTES:-



NOTES:-



If calculations show that one hydram is not sufficient to raise the required amount of water to the proper height, a series of pumps can be installed. There must be a sufficient supply of water, as each ram requires a separate drivepipe, but the pumps can share a common delivery pipe. It is not possible to power a higher hydram by the delivered water from a lower hydram, as the quantity of water will not be sufficient.

(Mitchell Silver)

