Handbook of Gravity-Flow Water Systems

by: Thomas Jordan

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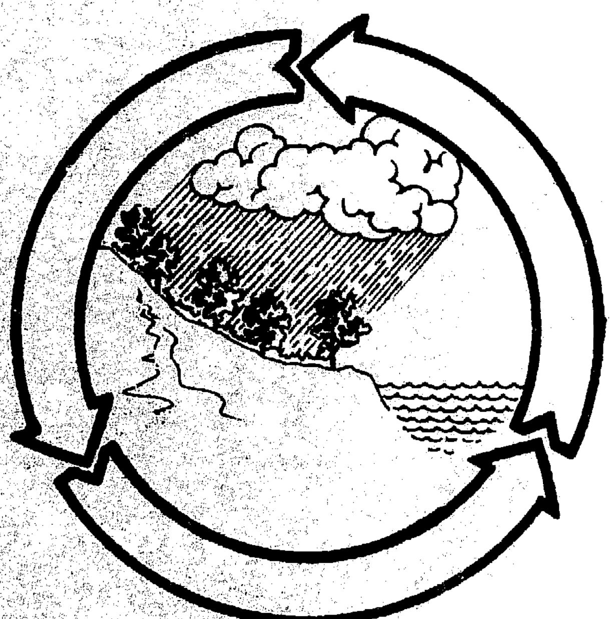

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HANDBOOK OF

GRAVITY-FLOW WATER SYSTEMS

FOR SMALL COMMUNITIES

BY:

THOMAS D. JORDAN JR.

A reference manual for surveying, designing, and constructing gravity-flow water systems, with special reference to such projects as implemented by the Local Development Department of His Majesty's Government of Nepal

United Nation's Children's Fund

Box 1187

Kathmandu, Nepal

HANDBOOK OF GRAVITY-FLOW WATER SYSTEMS

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ABBREVIATIONS AND SYMBOLS

cm = centimeters
cm^2 = square centimeters
cm^3 = cubic centimeters
kg = kilogram
kg/cm^2 = kilograms per square centimeter
LPS = liters per second (flow)
m/sec = meters per second (velocity)
LPCPD = liters per capita per day
GI = galvanised iron (pipe)
HDP = high-density polyethylene (pipe)
Q = flow
ID = inner diameter
OD = outer diameter
CGS = corrugated galvanised steel
RCC = reinforced concrete
RF = reinforcement

m = meters
m^2 = square meters
m^3 = cubic meters
H = head
@ = at
Ø = diamter
" = inches
$\sqrt{}$ = square-root
∇ = water surface of tank (@ atmospheric pressure)
HGL = Hydraulic Grade Line

IMPORTANT NOTICE

The pipeline problems worked out as examples in the text and figures of this handbook were made using a frictional headloss table for HDP pipe according to DIN (German) specifications. Since the examples were worked out, a new frictional headloss table was obtained for HDP pipe manufactured to ISI (Indian) specifications, which are the specifications adopted by UNICEF for Nepal. The new headloss tables have been included in the reference tables at the end of this handbook, but the original examples have not been re-worked.

PREFACE

This handbook is written with the intention of collecting together all the knowledge, theory, and practices necessary for the surveying, designing, and construction of gravity-flow drinking water systems for rural communities. Although the book is written specifically for the construction of such systems in Nepal, most of the principles presented herein are equally applicable in most locations around the world. Efforts have been made to organize the material for quick reference, and to present it in a manner that allows overseers of both engineering and non-engineering backgrounds to readily understand.

The stress of this handbook has been in presenting the fundamental principles of design, illustrated with several general examples of sucessfully constructed structures, rather than to present strict and unenlightened "standardized designs". This is because the author feels that, due to the typical ruggedness of the Nepali countryside, such standardized designs may not always be perfectly feasible. However, the overseer who understands the principles of the design can easily modify them to fit the locations.

This handbook will now replace the "Village Water Systems Technical Manual" written by Carl R. Johnson in 1975. In the years since that time, there have been several changes in design policies, new available materials, and much valuable field experience gained. New ideas, and better understanding of the principles and problems inherent in projects of this type have hopefully made their way into these pages. Johnson's original efforts paved the way, and this handbook is built upon the good foundations that he created.

Additionally, the author wishes to acknowledge those members of the Local Development Department, the American Peace Corps, the German Volunteer Service, the British Voluntary Service Overseas, the World Health Organization (WHO), and UNICEF who supported, encouraged, criticized, advised, and in many other ways helped in the making of this book.

However, this volume is by no means the final tome on this subject. Creative and inventive overseers will be constantly evolving new ideas, new uses for old materials, and new solutions for old problems. It is hoped that such resourceful persons will pass along their ideas to UNICEF, where they can be preserved and disseminated.

--Thomas D. Jordan, Jr.

Kathmandu, Nepal
March 1980

1. INTRODUCTION

1.1 DESCRIPTION OF SYSTEMS

This handbook concerns itself with all the knowledge, theory, and material necessary to survey, design, and construct a community water supply (CWS) system to meet the drinking water requirements for rural villages of small-to-moderate populations. The systems described herein are of the gravity-flow type; that is, the action of gravity is used to move the water downhill from a source to the village. This type of system is shown schematically in Figure 1-1: a suitable source is located at an elevation higher than the village. An intake structure is built to collect the water, which is then piped down to the village through a buried pipeline of High-Density Polyethylene (HDP) pipe. If needed, a reservoir tank is built above the village. From there, the water is distributed to several public tapstands that are scattered throughout the village, via the mainline, branchlines, and taplines. Where multiple sources are used, a collection tank may be built, and due to the topography of the land, at certain points break-pressure tanks may be required to prevent excessive pressures from bursting the HDP pipe. If the source water is carrying a lot of suspended particles, a sedimentation tank may be required to clean the flow of these.

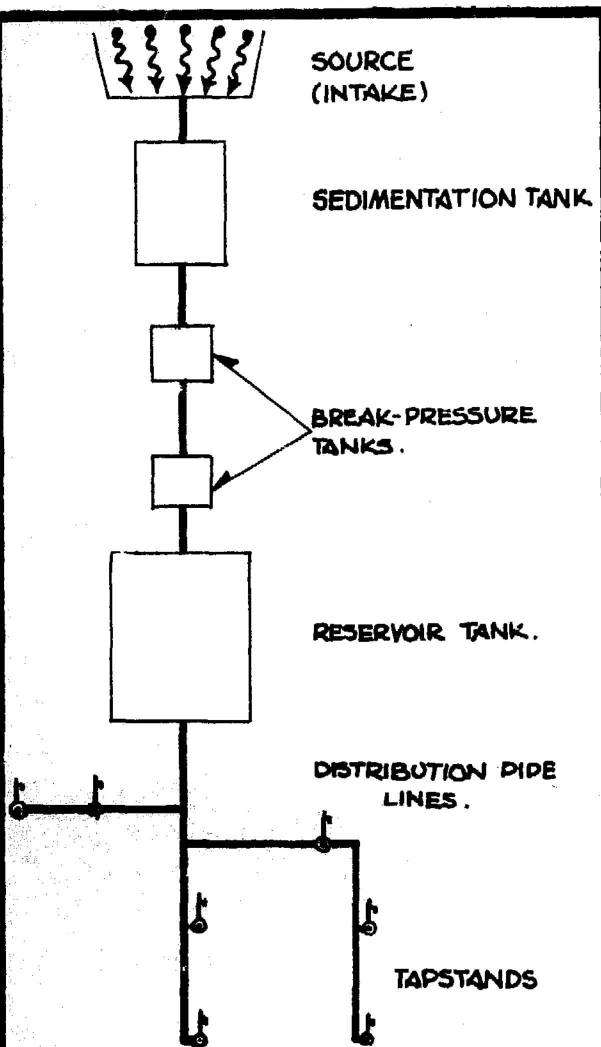


FIGURE 1-1
TYPICAL CWS SYSTEM

Each of these system components (intake works, pipeline, various tanks, tapstands, etc.) are discussed in this handbook.

1.2 FUNDAMENTAL PROBLEMS

The construction of a CWS system in rural villages is often beset by many problems which prolong, frustrate, or even cancel the project. And even when these initial problems are successtully circumvented and the project is completed, the system may soon be broken down due to misuse or unforeseen circumstances.

The technical problems encountered during construction may be such things as difficult terrain for pipelaying landslides or erosions which threaten to sweep away tanks, or water sources which unexpectedly yield less water than counted upon (or even dry up completely).

These are problems which are often clear and easily comprehended by the overseer in charge of construction, who can usually plan a strategy to overcome them.

Less obvious problems, however, arise from the "human factor." Because such projects are community efforts, often they become unavoidably embroiled in local disputes, arguments, politics, and economics. Here, the problems that arise are rarely clearly defined or easily solved. Water rights of the sources, locations of the tapstands, and division of labor are most often major problems, and until they are solved (if ever!) the future of the system, even if it is eventually completed, is dim.

Human problems that plague water systems are things such as neglect (inadequate or zero maintenance), abuse (from children and curious adults), or deliberate sabotage (by discontented villagers determined to punish the rest, or by selfish ones who cut open the pipeline to irrigate their fields). These are all very real problems that have effectively destroyed many water systems.

The technical problems can be grasped and dealt with by the overseer and consultant engineers, who are trained to recognize and solve these problems. The human problems, however, must be recognized and solved by the entire community, which is often the far more difficult task.

This hand book tries to address both types of problems: technical as well as human. Technical theory and construction practices alone will not build a successful water system. Many times, if the overseer underestimates the innocent destructiveness of children, or fails to understand some special needs of the villagers, there will be much lost time, damaged materials, repetition of labor, and general feelings of anger and frustration. Wherever possible, this handbook tries to mention these potential problems,and suggest possible means to minimize them.

1.3 ORGANIZATION OF THIS HANDBOOK

The material herein is roughly arranged in the order of the three phases of a CWS project: surveying, designing, and construction of the system. All dimensions and calculations are in the metric system of units, except for the GI pipe sizes which are given in inches (since this is how they are supplied in Nepal). The designs presented are carefully calculated to yield necessary structural strength and utility without using an excess of materials. Each system component is discussed in terms of function and good design characteristics, and several drawings are given of such structures that have been sucessfully built. The last pages of this handbook are a series of reference tables for fast reference and general information.

2. VILLAGE EVALUATION AND FEASIBILITY STUDY

2.1 INTRODUCTION

The first phase in creation of a CWS system is a visit to the village by a surveyor, for the purposes of determining the feasibility of the proposed project. Should he determine that the project is feasible, the surveyor must then conduct a topographic survey.

Evaluation of a village is both an objective and subjective process. Objectively, the surveyor determines facts: village population, locally-available materials, supply of skilled labor, logistical information, etc. Subjectively, the surveyor determines feelings: who are the influential people of the village, what are villager reactions and attitudes towards the project, do they realize the amount of work that they will be required to do, and will they do it? How real are the needs of the village, and who stands to benefit?

A project should be considered feasible only if both the technical factors and the human factors indicate success. To get accurate and reliable answers to the above questions, the surveyor must involve himself in discussion with as many villagers as possible. Relying only upon two or three persons for information is quite wrong. The surveyor must get out and walk around the village, meet the people in public places (such as in teashops, around temples, etc).

Once the surveyor is satisfied that the villagers are enthusiastic about the project, he should proceed with the technical aspects of surveying the system.

The entire village evaluation and topographic survey can rarely be done in less than two days. Time must be taken to ensure that a complete investigation has been made, and accurate results obtained. When the surveyor finally leaves the village, he should have resolved in his mind whether or not the project should be undertaken. If he feels that it should, then he must have all the necessary data for himself, or another person, to draw up the complete designs and estimates for the system.

This chapter will set forth guidelines for the surveyor to use when visiting a village on a feasibility study. The next chapter will present specific details on how to conduct the topographic survey.

2.2 POPULATION SURVEY

An accurate population survey of the village is absolutely necessary, since population determines water requirements. A village population, for the purposes of a water system, includes all persons who will depend upon it for their drinking water. Thus are included patients in health posts, students living in dormitories, employees in government offices, etc. Although most rural mountain villages will not usually have these special populations, they must be taken into account where they are found.

Field experience has shown that villagers usually have a very poor idea of their own population, and tend to grossly overestimate their real numbers. They also sometimes will slant their answers to suit their purposes. Care must be taken to obtain an accurate count. Three techniques have proven successful :

1) Making a written list of every household and the number of people living in it.

2) Walking around the village and counting houses, determining the number of people living in each one.

3) For very large villages, counting every single person in the village is not really feasible. Instead:

- At each tapstand site, determine the number of houses to be served;

- Personally survey about a dozen of those houses, and determine the average number of persons in each;

- Apply that average to determine the number of persons to be served by that tapstand;

- Total up the total population served by the tapstands.

In any case, discuss with as many villagers as possible these numbers, especially with those who live in that part of the village. To rely upon two or three persons alone is easier, but not as accurate.

2.3 VILLAGE ENTHUSIASM AND MOTIVATION

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. Better estimates of the "motivation factor" will come with experience. Never-the-less, it is necessary to get some idea of the motivation, cooperative ability, and potential social and political conflicts while visiting the village. Talking to people; examining past community construction efforts (such as schools, temples, etc); inspecting the conditions of paths, buildings, temples, and public areas (how well are they maintained?); and talking with local government officials, will all yield helpful clues as to the possible success or failure of the construction effort. The surveyor should strive to explain to the villagers, as clearly as possible, the role that they and the government will be expected to fill.

No matter how technically feasible the project may be, it can only succeed if the villagers are truly interested in it, and concerned enough to provide the long-term maintainence necessary to keep the system in working condition.

2.4 CURRENT, WATER SOURCES

One of the best ways of determining the likely motivation of the villagers is to examine their current sources of water, and deduce how helpful a CWS project would truly be. Those villagers living close to an adequate source of water will be less inclined to work on the project than those who are not so conveniently located. This reasoning can be applied to the whole village as well, and the surveyor can form a good idea how essential a new system will be.

2.5 SOURCE INVESTIGATION

Investigating a source for a water system should not be confined to only the most convenient source. At this time, water treatment techniques are not generally practical in Nepal, thus it is necessary to locate the cleanest source possible, even if it is not the closest one to the village. Rather than relying upon villager descriptions alone, the surveyor should personally visit <u>all</u> possible sources. Quantity and quality of the flows must be determined, means to develop the intake works must be studied, and water rights must be investigated.

Before beginning his examination of the sources, the surveyor should have completed his population survey. From the information presented in Chapter 4, he can calculate the daily water requirements of the village. No source is feasible if, in 24 hours, it cannot provide that much water.

Springs and small streams are the most common sources for water systems. Whenever possible, a spring should be the first considered, since springs are generally of better water quality, and easier to protect against further contamination.

Springs: Springs are points where water from an underground source is able to seep to the surface. Flows are typically less than 2 LPS, but some can be quite substantial. The flow of a spring is governed by several factors: watershed collection area, percolation rate of water through the ground, thickness of ground above the aquifer (ie- overburden), and the storage capacity of the soil. Springs are seasonally variable, tending to lag behind the seasonal rainfall patterns (ie- springs can give normal flows well into the dry season before tapering off, and may not resume full flow until after the rainy season is well under way). Due to ground percolation and filtration, most springs are quite free of the pathonogenic organisms that cause many health problems; however, some springs flow through limestone or geologic cracks and fissures in the rocks. In such cases, filtration effects are minimal, and the flow may still be contaminated. Also, it is possible that the source is not a true spring at all, but rather a stream that has gone underground for a short distance and is re-emerging. Investigation around the source will reveal the type of spring it is. Figure 2-1 shows the typical geology of a spring, showing the different levels of ground water during the dry - and rainy seasons.

Small streams: These are not as desirable sources, especially when there is human habitation and/or animal grazing areas further upstream. However, sometimes the water demand of the village cannot be met by other sources. Streams are very much seasonally variable, and also can react strongly (and quickly) to daily rainfall as well. When investigating a stream source, study the proposed intake site carefully, with an eye towards future erosion. Question the villagers closely about flood levels of the stream during heavy rainfall.

Big streams and rivers: These are the least-desirable sources, as they are sure to be grossly contaminated from points further upstream. They do offer, however, the best sources for hydraulic ram pumps (hydrams) which can supply villages at a higher elevation and for which there is absolutely no other feasible source. Refer to Chapter 18 for technical information on hydrams.

2.6 FLOW-MEASURING TECHNIQUES

In most investigations, accurate flow measurements of a source will require some earthwork, usually just a simple type of earth bank, dam, or drainage channel. Thus it is advisable to bring along one or more villagers with digging tools and a machete-type knife (for clearing away underbrush, etc). After the channels or dams have been constructed, wait a few minutes for the water to achieve steady, constant flow, before attempting any measurements.

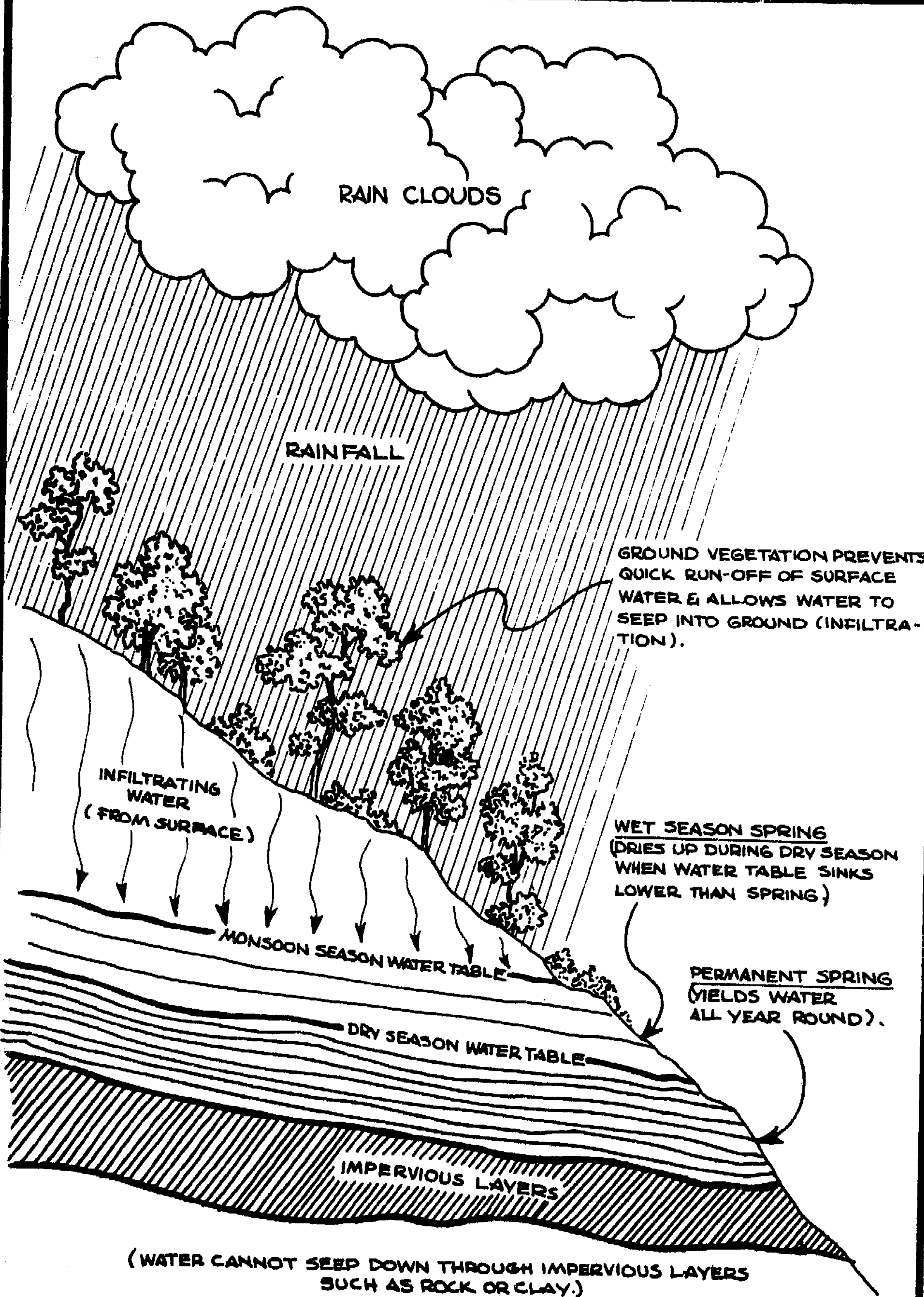


FIGURE 2.1 HYDROLOGIC DIAGRAM OF SPRINGS

Discussed below are three simple methods for measuring the flows of springs and streams. Always measure the flow several times, and calculate an average reading. Any measurements which are obviously deviant should be repeated. Question the villagers closely about seasonal variations in the flow.

Bucket and stopwatch: Spring flows are most conveniently measured by using a wide-mouthed container (of known capacity) and timing how long it takes to fill up. A large-size biscuit or kerosene container (capacities of about 18-20 liters), or a bucket, is usually available in the village. For the most accurate results, the capacity of the container should be such that it requires at least 15 seconds to fill (smaller containers, such as one-liter drinking canteens, should only be used if nothing larger is available). An ordinary wristwatch (that has a sweep-second hand) can be used for timings, but it is best in this case if two persons work together: one concentrating on the wristwatch, the other filling the container. The flow is calculated:

$$Q = \frac{C}{t}$$

where:

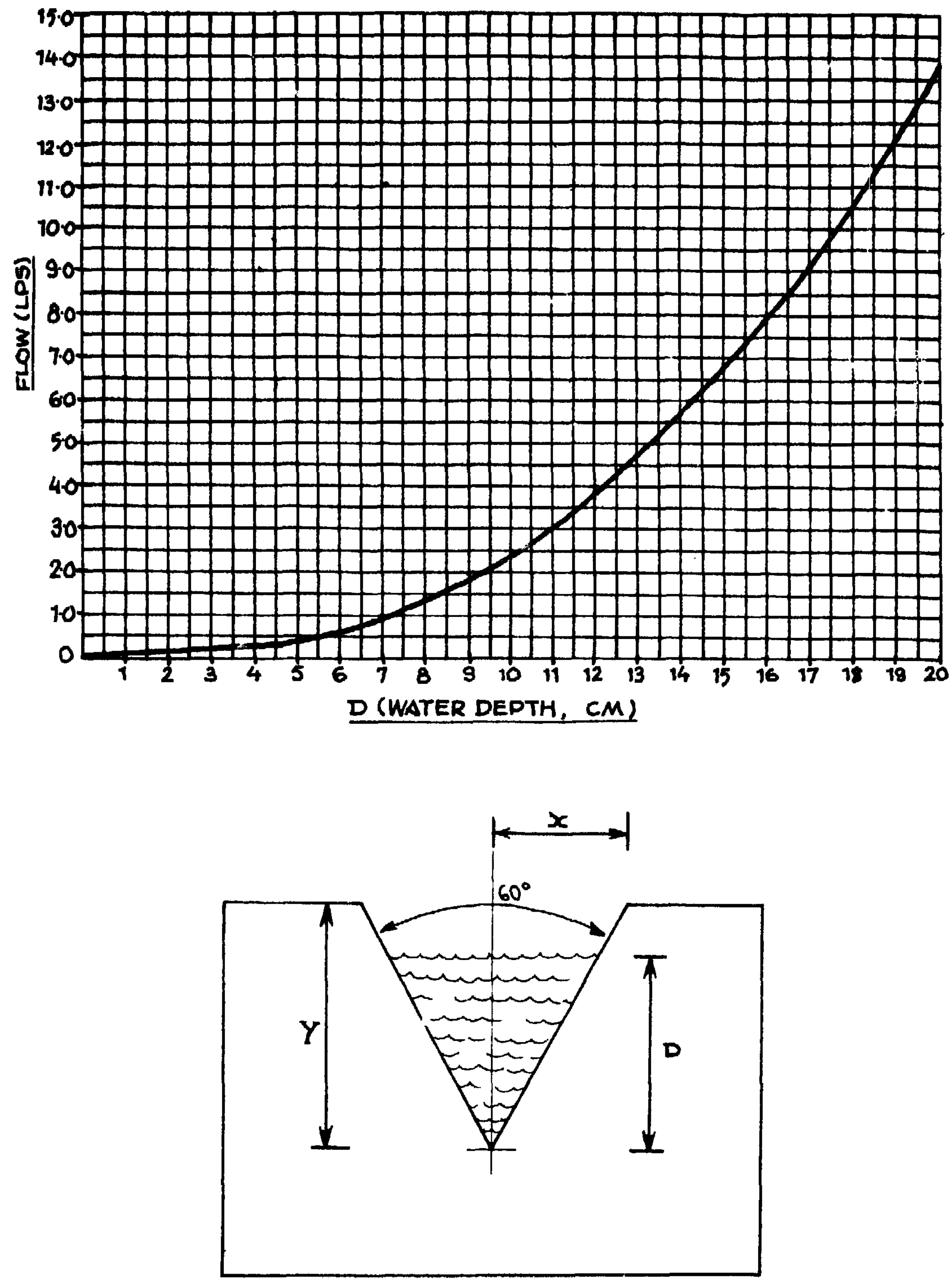
Q = flow (liters/second)

C = capacity of container (liters)

t = time to fill (seconds)

V-Notch weir: The V-notch weir can be used to measure the flow of large springs and small streams. This weir has a notch angle of 60°, and is recommended for the normal range of flows encountered in typical village sources. The surveyor may carry his own weir, or one can be easily made in the village from a wooden board or a sheet of tin. The weir is placed as a dam, perpendicular to the flow, with all the water overflowing in the notch. The stream or drainage channel above the weir should be straight and unobstructed for a distance of at least 2 meters. Flow is determined by using a ruler to measure the depth of water overflowing the notch (measured in centimeters), which is then read off of the calibration curve shown in Figure 2-2. The dimensions of a 60° V-notch weir are also shown in the figure.

Velocity-area method: This method requires more work and is not as accurate at the V-notch weir, yet for particularly wide streams it can be easier to use. Measure the surface water velocity of the stream by timing how long it takes a drifting surface float (such as a block of wood) to move down a measured length of the stream (this measured section must be fairly straight and free of obstacles, for a length of 6-10 times the average water depth). Measure the cross-sectional area of the stream. The measurements



$$\frac{X}{Y} = \frac{1}{1{\cdot}73}$$

FIGURE 2.2 V-NOTCH (60°) WEIR & FLOW GRAPH should be repeated several times, averaging the results together. The average stream velocity is 85% of the surface velocity, and the flow is calculated:

$$Q = 600 \times V \times A$$

<u>where:</u> Q = flow (LPS)

V = surface velocity (m/sec)

A = cross-sect'l area (m^2)

This method of flow measurement is applicable to streams of water depth of at least 30 centimeters. Figure 2-3 illustrates the velocity-area method of measurement.

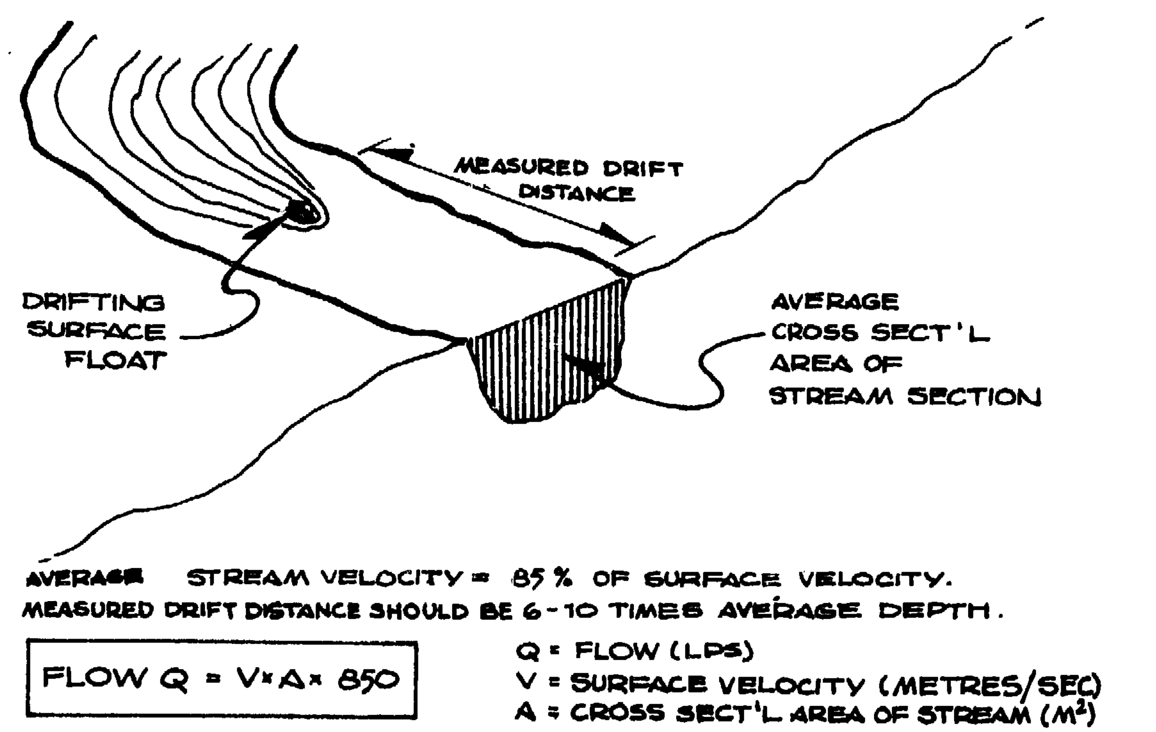


FIGURE 2-3 VELOCITY-AREA METHOD OF FLOW MEASUREMENT

2.7 SAFE YIELD

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

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

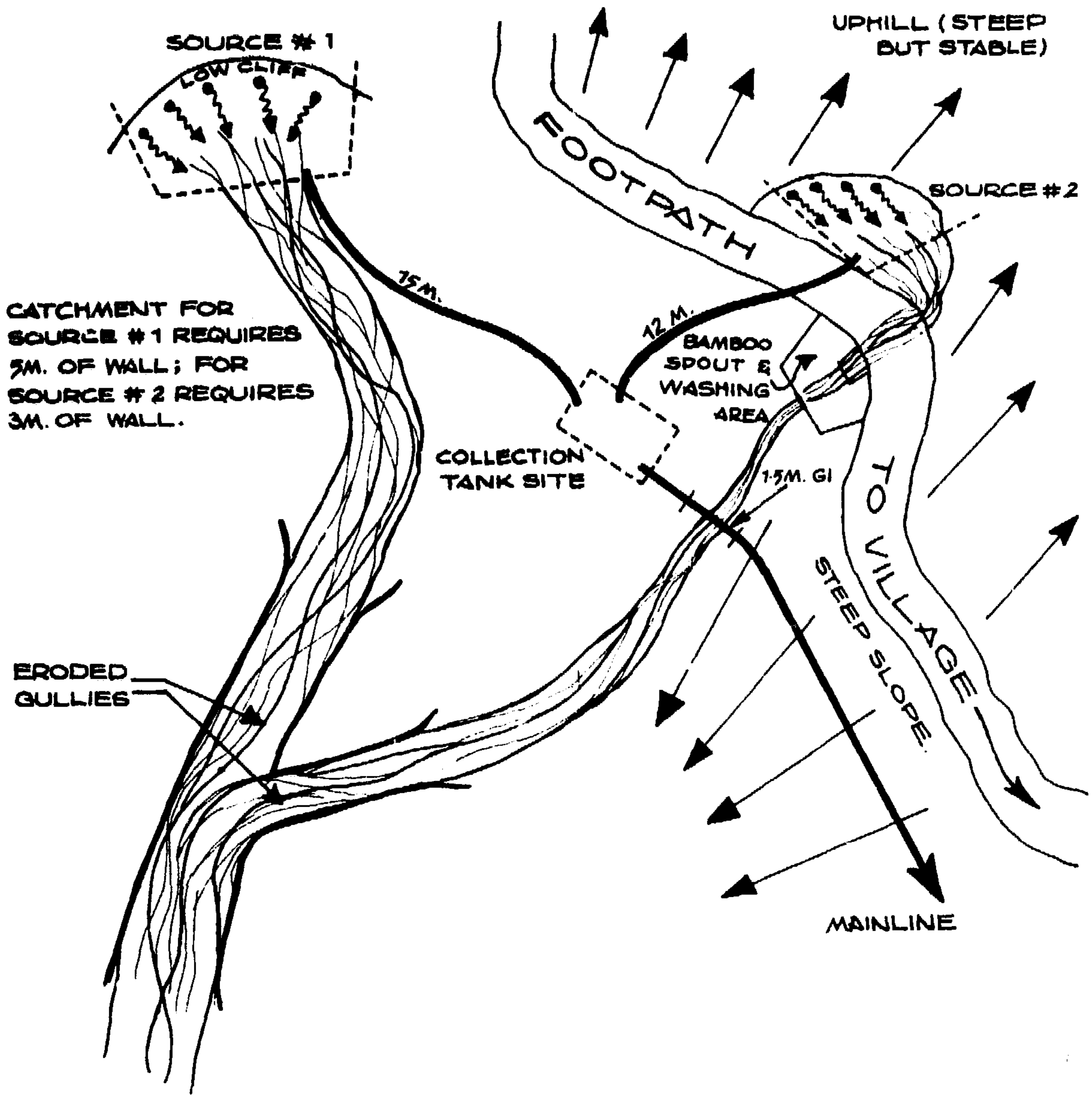
2.8 WATER QUALITY

At this time, practical water treatment schemes are not widely available, especially for remote project sites. However, if a source has turbidity (cloudiness),taste, and/or odor problems, these might be easily remedied by the simple treatment schemes discussed in Chapter 17.

In any case, the surveyor will have to use his own judgement about the suitability of a source. Villagers will know through experience if the water of a source is drinkable, therefore they should be consulted.

2.9 SOURCE DEVELOPMENT

When investigating a possible source, the surveyor must be developing an idea about how the intake works will be built. He should be thinking about methods of protecting the structure against erosion, floods, and contamination by surface run-off of rain. What amount of excavation will be called for, and will a dam or drainage channel be required? What further protection can be included to keep animals and curious villagers from tampering with the works? All these aspects should be carefully thought about, and measurements and sketches made in his field book, along with notes about topography, surrounding areas, etc. To depend upon memory alone when trying to design the intake works (which could be several weeks after having last seen the source) is unprofessional and unreliable Figure 2-4 is a sample page of field notes relating to a source.



NOTES:

1. SOURCE #1 = SPRING @ 0·35 LPS. ELEV = 1000M.
2. SOURCE #2 = SPRING @ 0·22 LPS. ELEV = 998 M.
3. COLLECTION TANK @ ELEV = 993 M.
4. SOURCE AREA JUNGLE W/LITTLE UPHILL CULTIVATION OR GRAZING.
5. MAINLINE NEEDS 1·5M. GI PIPE CROSSING GULLY.
6. SURVEY BEGUN @ SOURCE #2.

FIGURE 2.4 EXAMPLE SOURCE NOTES & SKETCH

2.10 WATER RIGHTS

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

2.11 LOCAL MATERIALS, LABOR, AND LOGISTICS

Before leaving the village, the surveyor should sit down and obtain all the information relevant to the following aspects:

- the full proper name of the village, ward number(s), panchayat, district, and zone;
- name of nearest roadhead, and distance;
- name of nearest airfield, and distance;
- portering time and fees between roadhead, airfield and village;
- supply of local skilled labor (masons, carpenters, etc) and wages;
- supply of local materials (slate, wood, bricks, etc) and costs;
- sources of sand and stone;
- schools (number of students) and health posts (number of beds);
- names of those villagers who were helpful and familiar with the surveyed route;
- accurate population count;
- other necessary information about special needs of the village.

The answers to these above queries should be carefully recorded in the surveyor's field book.

3. TOPOGRAPHIC SURVEYING

3.1 INTRODUCTION

This chapter shall present methods of conducting a topographic survey along a proposed pipeline route. Such a survey can be done using a theodolite, barometric altimeters, or an Abney hand level. Each of these methods will be discussed, although the main emphasis of the chapter shall be on surveying with the Abney level, since that is the easiest and most-widely used technique.

3.2 THEODOLITE SURVEYING

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

3.3 BAROMETRIC ALTIMETER SURVEYING

As altitude increases, the barometric pressure (ie- air pressure) of the atmosphere decreases. A barometric altimeter measures the atmospheric pressure, and the corresponding elevation is read directly off of the instrument.

Normal weather patterns cause the air pressure at any altitude to fluctuate slightly throughout the day. Thus, even if an altimeter is at a point, the elevation reading may increase and decrease by several meters throughout the day. Such pressure variations must be measured and accounted for when conducting a barometric altimeter survey.

Properly done, such a survey requires three persons, each with his own altimeter. The three altimeters are brought together and calibrated (for the same altitude reading) at the same time. One assistant then takes his altimeter to the highest point along the survey, while the other assistant takes his altimeter to the lowest point. They remain at those points during the entire time of the survey, and at regular intervals (such as every 15 or 30 minutes) they record the elevation readings of the altimeters, and the time. The surveyor takes the third altimeter along the route of the pipeline. Ground distances are measured with a tape measure, and at each station the surveyor records altitude and time. Later, the true elevations can be made by adjusting the surveyor's reading with the pressure changes recorded by the stationary altimeters.

Although not necessarily as accurate, this type of surveying can be done with two altimeters: one stationary at the mid-elevation of the route, while the surveyor carries the other.

This type of survey is the fastest to conduct, and accuracy limited only by the accuracy of the altimeters themselves. This method can be best used for feasibility surveying of a system, with a more careful survey conducted later.

3.4 ABNEY LEVEL SURVEYING

The standard method of conducting surveys for water system pipelines is using the Abney hand level (technically of the type known as a clinometer level). It is faster to use than the theodolite, and although not as accurate, it still yields results that are within acceptable limits needed for this type of survey. It is a lightweight instrument, easy to use, and of simple construction to make it rugged and easy to adjust. The remainder of this chapter shall deal with various aspects of using the Abney, adjusting it, and proper recording of survey field notes.

3.5 DESCRIPTION

The Abney level is basically a square tube (dimensions of about 16 x 1.5 x 1.5 cm) with an eyepiece at the observer's end and horizontal cross-hair at the objective end. Figure 3-1 shows a typical Abney level. Near the center of the tube is a 45° mirror, which reflects half of the line-of-sight upwards through an aperture in the tube. Mounted above the aperture is a bubble level with an index mark etched at its center. The bubble level is affixed to a movable index arm, which adjusts against scale graduations on a nickel-silver arc. Some types of Abney levels have interchangable arcs, offering different types of scales (such as degrees, percent, etc). For the purpose of this handbook, the degree arc is used.

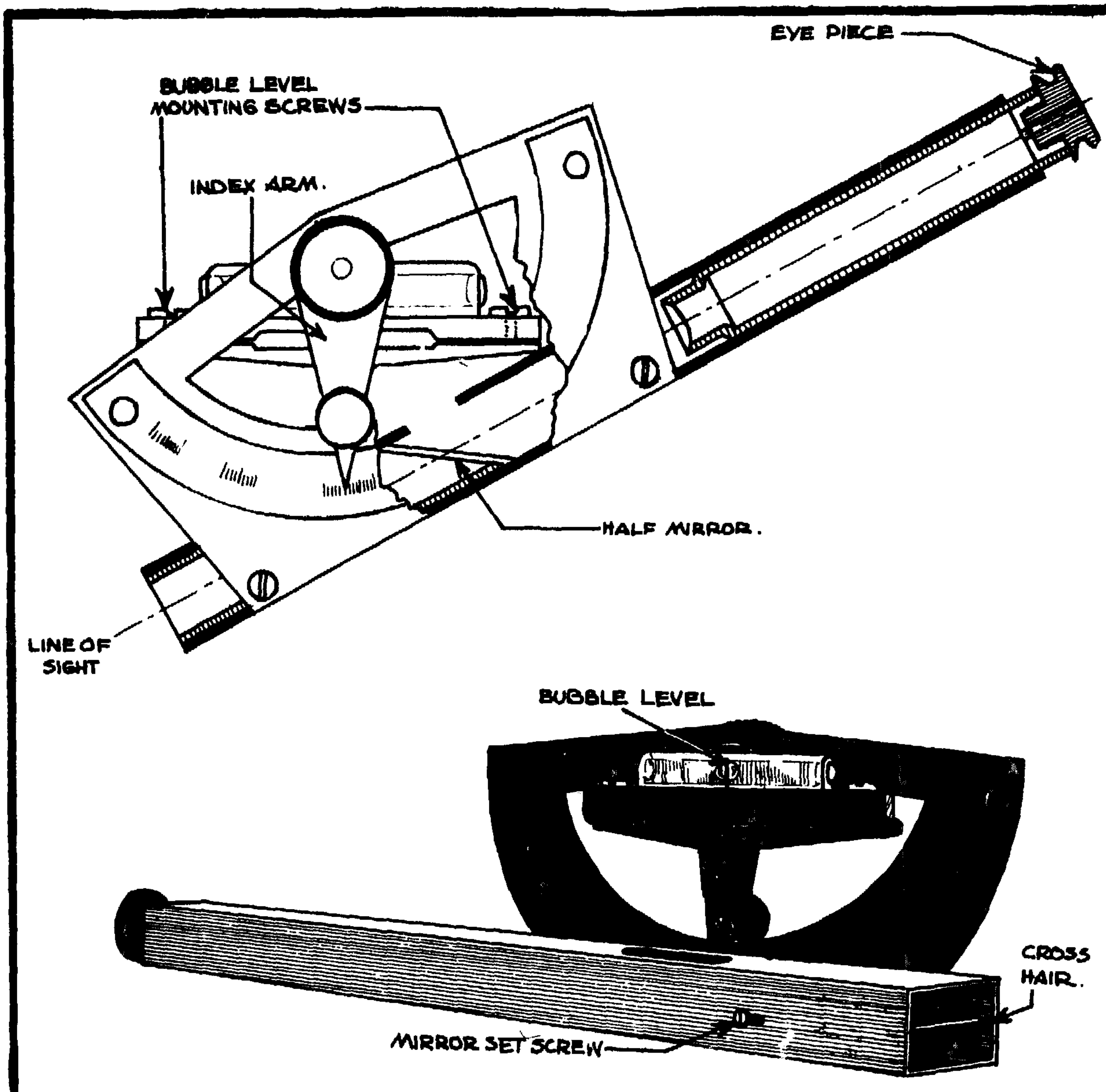


FIGURE 3-1 TYPICAL ABNEY LEVEL

3.6 SIGHTING WITH THE ABNEY

To use the Abney, the instrument is held to the eye and sighted on a target, centering the cross-hair against the target. The index arm is then adjusted until the bubble (visible in the right half of the field-of-view) is centered against the target and the cross-hair. When correctly adjusted, the target, the cross-hair, and the bubble are

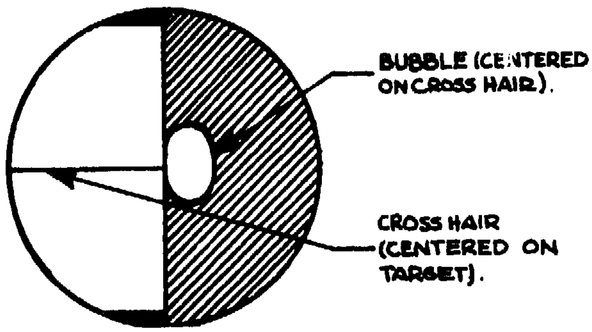


FIGURE 3-2
VIEW THROUGH ABNEY
WHEN PROPERLY LEVELED

all aligned horizontally, as shown in Figure 3-2. The angle of view (technically known as the vertical angle) is then read on the arc in degrees.

If the index is pre-set at exactly 0°, then the level may be used as a rough carpenter's level for construction of walls, beams etc.

3.7 ADJUSTMENT OF THE ABNEY

Like any precision instrument, the Abney level is liable to creep out of adjustment from time to time. When used in the field, it should be checked for levelness each day. No surveyor should ever begin a survey with an unfamiliar Abney, or one that has not been used for a long time, without first checking the adjustment. If the instrument is ever dropped, the adjustment should be checked before proceeding.

Adjusting the Abney is a quick and simple task. The various methods of adjustment are discussed below :

Two-post method: When checking the adjustment for levelness in the field, select two trees, posts, or building corners that are about 7-10 meters apart (refer to Figure 3-3). At Station A, the surveyor holds the Abney against a mark (located at approximately eye-level), and with the index of the Abney preset at 0°, he sights over to a Station B. An assistant, standing at Station B, moves a target (such as a pencil, his finger, stick, etc) up or down until it lies on the line-of-sight. At that point, he makes a mark. Then he and the surveyor exchange places, and the surveyor sights from the mark on Station B back towards Station A. The assistant sets a new mark on Station A that lies on this line of sight. If the two marks on Station A coincide, then the Abney is truly level, and no adjustments must be made. If they do not coincide, the assistant sets a third mark exactly halfway between the two other marks. Sighting on this mark, the surveyor then adjusts the bubble level until the bubble comes into alignment with the cross-hair and target mark.

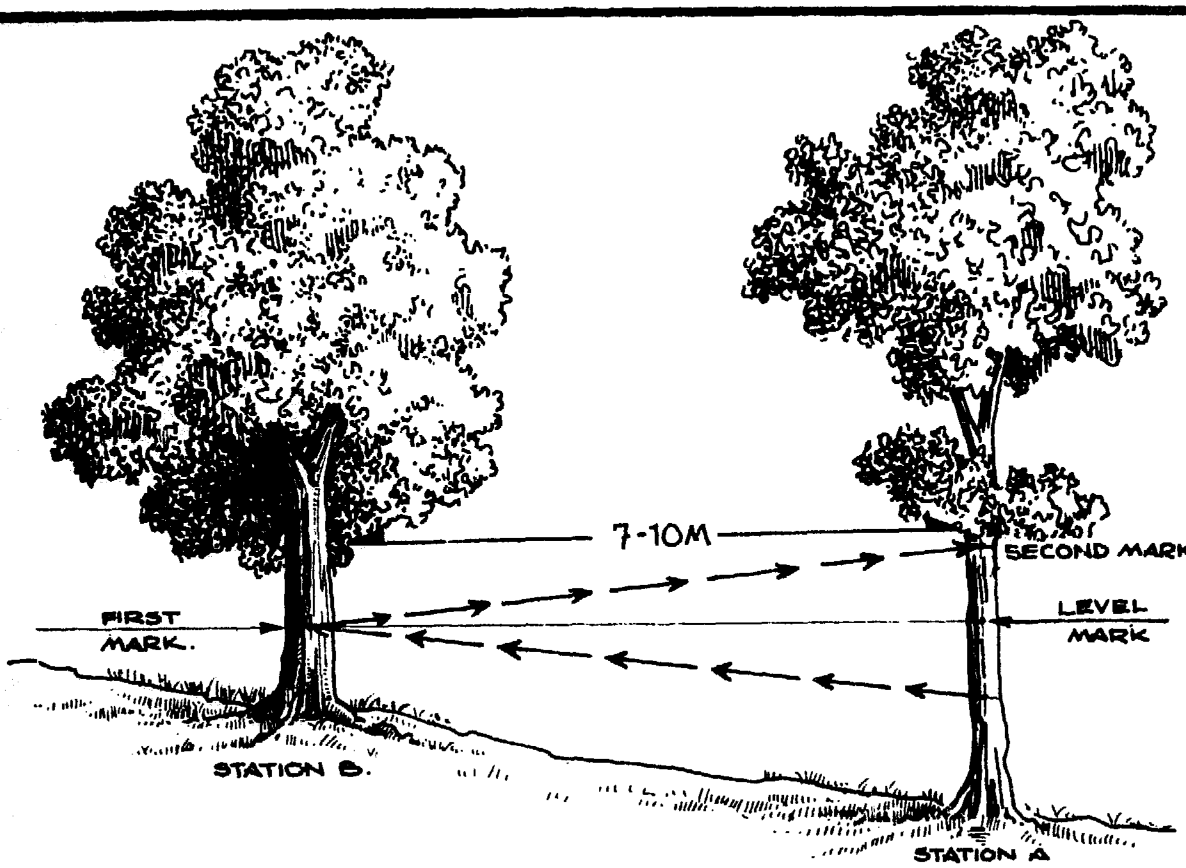


FIGURE 3-3 ADJUSTING THE ABNEY: TWO-POST METHOD

Many surveyors set up permanent level sight lines in their offices or on convenient outbuildings, so that it is only the work of a moment to check the Abney and adjust it themselves (not needing an assistant at all).

Flat surface method: In addition to adjusting for levelness, this method is also needed to make the further adjustments of the bubble level and mirror, described later.

A smooth, level surface is required. If nothing better is available, place a smooth board on a firm support, arranged so that it is possible to sight lengthwise along its surface. Place the Abney lengthwise on the board and outline its position with a pencil. Center the bubble against the etched index mark of the bubble level. Reverse the instrument end for end and place it within the penciled outline. The bubble should center. If it does not, move it half-way toward the etched mark (adjusting with the index arm), then wedge up the board until the bubble centers. Reverse the instrument end for end, repeating the whole procedure until the bubble centers when the instrument is in both positions. The penciled position on the board now provides the level surface required for making adjustments:

Setting the bubble: Set the index arm at 0° . Center the bubble against the etched mark on the bubble level by adjusting the mounting screws (turn the screws equal amounts in opposite directions).

Setting the mirror: With the instrument still in position, place a white surface a few centimeters in front of the tube so that the horizontal cross-hair can be clearly seen when sighting through the instrument. The bubble should center on the cross-hair when sighting through the instrument. If it does not, loosen the screw on the right-hand side of the tube and slide it backward or forward until the bubble centers properly against the cross-hair. Tighten the screw and check the setting.

Setting the height of the bubble level: Fix the instrument firmly to the edge of the board, angled downward about 30° (refer to Figure 3-4). Center the bubble against the etched index mark.

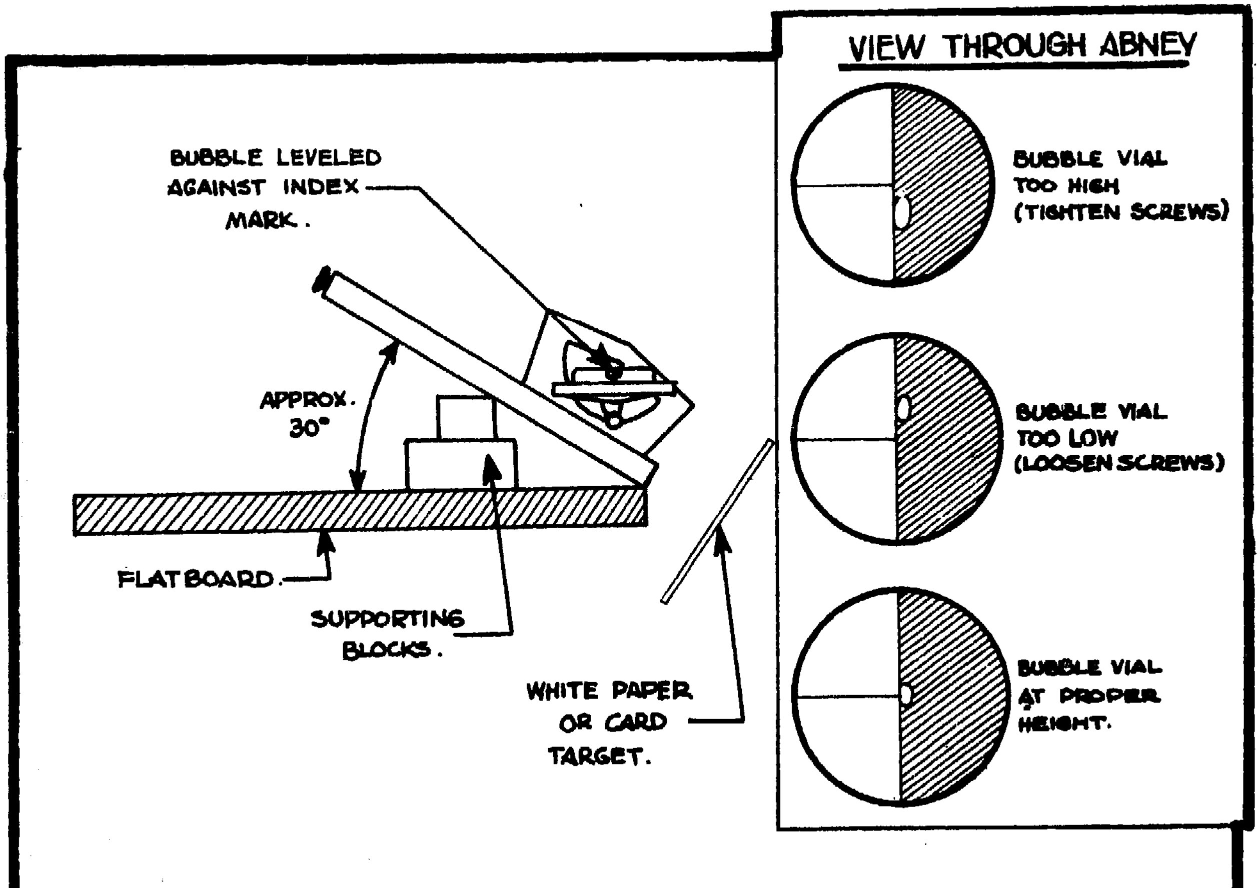


FIGURE 3-4 SETTING HEIGHT OF THE VIAL

Hold a white card in front of the tube and sight through the instrument. The bubble should be centered on the cross-hair. If it is not, adjust the height of the bubble level by tightening or loosening the mounting screws equal amounts in the same direction. If the bubble appears too low, then lower the level by tightening the screws, and vice versa.

The above information is taken from the Keuffel & Esser Co. manual no. 80 0204, "Topographic Abney Levels". Refer to this manual for further information concerning Abney levels.

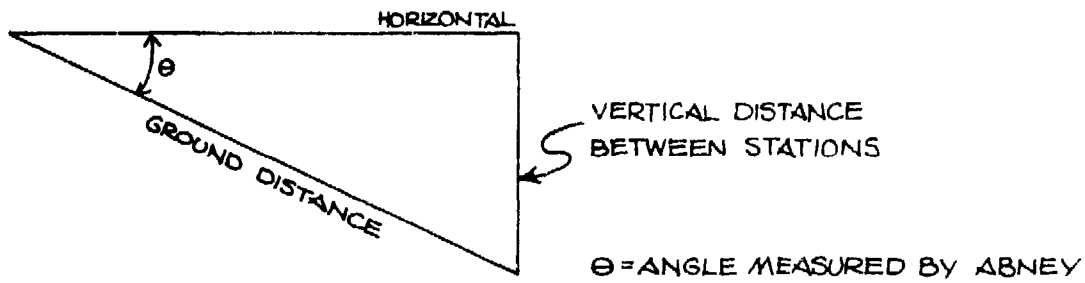
3.8 SURVEYING WITH THE ABNEY

Conducting a survey with the Abney requires minimally two persons, but additional ones are helpful, especially when surveying through terrain where underbrush must be cut away to provide clear lines-of-sight. An Abney level, a 30-meter tape measure, and a field book are necessary; a compass may be used if bearings are desired.

The survey is begun at some fixed reference point (such as the source, or some prominant landmark along the pipeline route) and proceeds upstream/downstream from that point, along the route of the proposed pipeline. Villagers will sometimes lead the survey team along convenient footpaths, when the actual trenchline will be dug along a different route; such a technique will create erroneous ground distances.

The surveying technique is simple: the surveyor sights through the Abney at a target held by his assistant, and the ground distance between them is measured. This distance, and the vertical angle (angle measured by the Abney) are recorded in the field book. It is important that the target which the surveyor sights upon must be the same height above the ground as the Abney, which is the same as the surveyor's eye-level. If the assistant is not as tall as the surveyor, then he sould carry a target stick cut exactly to the same length as the surveyor's eye-level. A red cloth can be tied to the top of the stick, or the assistant can place his hand over the end of it, to provide a clear target. It is also useful for the surveyor to use a forked stick as a stand to rest the Abney on, to provide a steadier reading (in this case, the target stick should be cut to the same length as this forked stick).

Figure 3-5 shows the basic arrangement and calculation used in trigonometric leveling with the Abney: the surveyor and his assistant are 28 meters apart (ground distance), and the vertical angle is -16° (the negative



VERTICAL DISTANCE = GROUND DISTANCE × SIN θ

EXAMPLE ILLUSTRATED BELOW :
$\theta = -16°$ (NEGATIVE SIGN INDICATES SIGHTING DOWNHILL)
$\sin\theta = 0.276$ (FROM TRIGONOMETRIC TABLE)
GROUND DISTANCE = 28 METERS
VERTICAL DISTANCE = $28 \times 0.276 = 7.73$ METERS.

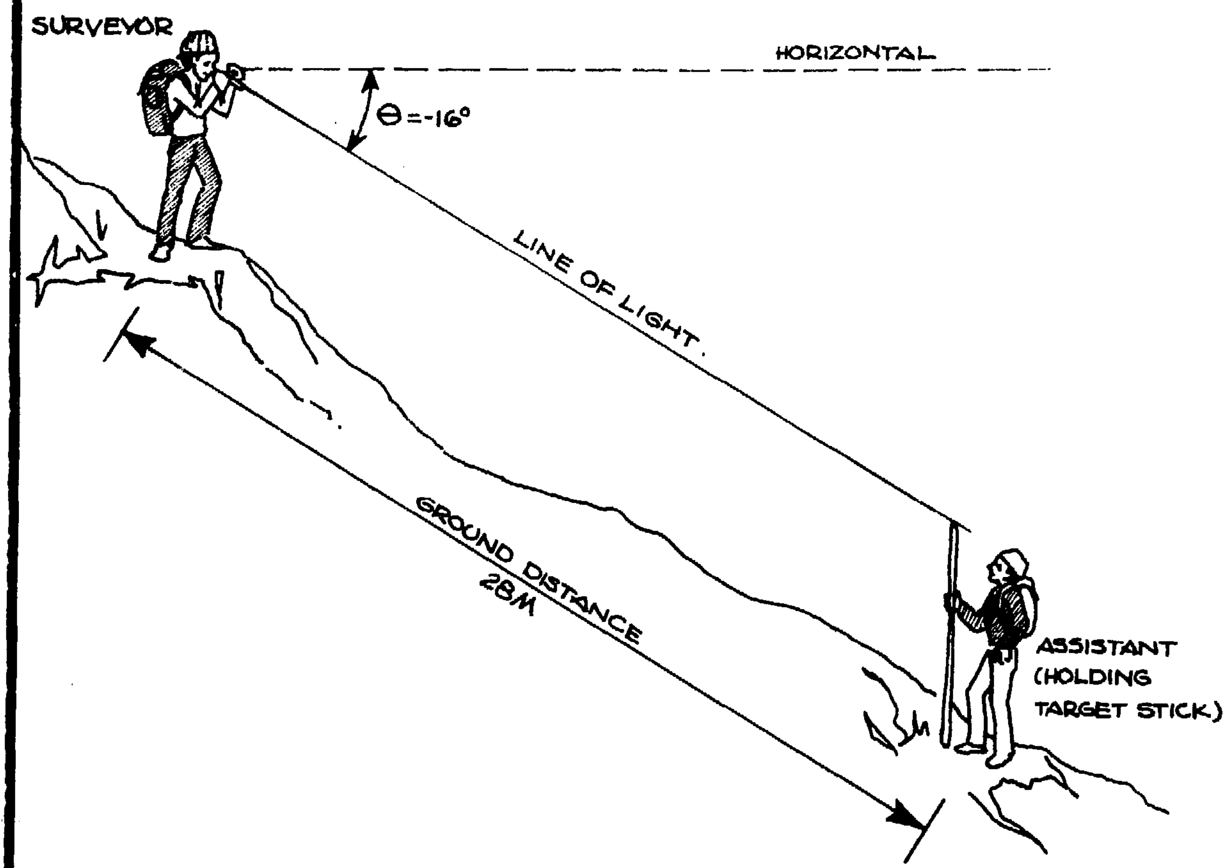


FIGURE 3-5 TRIGONOMETRIC SURVEYING WITH AN ABNEY LEVEL

angle indicates that the surveyor was sighting downhill). By consulting a table of natural sines and using trigonometry, it is possible to calculate that the vertical distance between them is 7.7 meters. The reference tables at the back of this handbook present the natural sines, and also the elevation changes for various ground distances and vertical angles.

3.9 FIELD METHODS

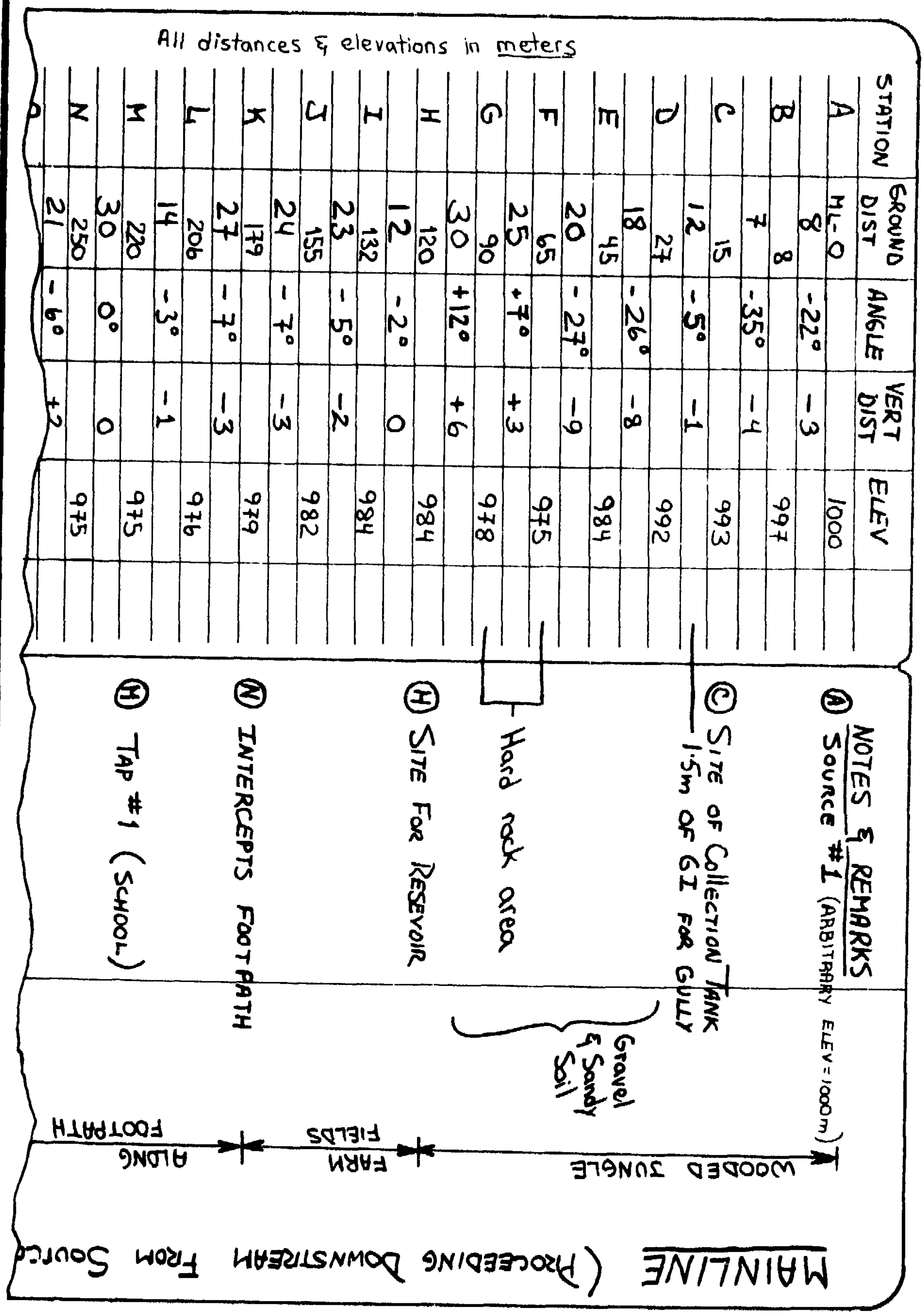
At the same time he is conducting the survey, the surveyor must also be observing the ground over which he walks. As he proceeds, he must constantly keep in mind that, at some later time, he or another person will actually have to dig a trenchline along that route. Thus, notes must be made about the type of terrain being traversed, such as stretches of jungle, cultivated fields, footpaths, gullies, soil conditions (ie- gravel, soft dirt, bare rock, etc). It is easy to survey across terrain which might be exceedingly difficult or impossible to lay a pipeline! The surveyor should make use of as many reference points as possible, so that if a section of the pipeline needs to be resurveyed at a later time, a convenient starting point can be found. Reference points should be permanent or semi-permanent; suitable examples are prominent trees, rock outcroppings, etc. If the surveyor carries one or two bottles of nailpolish, he can paint an identifying lable onto his landmarks.

Figure 3-6 shows a good, precise format for recording accurate and complete notes.

3.10 CLOSING THE SURVEY AND ACCURACY LIMITS

Closing the survey means tying the survey into two reference points of known elevations, thus providing a check on the surveyed elevations. For practical purposes of surveying in the hills of Nepal, closing a survey can only be done by repeating it entirely, beginning from the original endpoint and ending at the original starting point, but not necessarily along the same original route. Needless to say, this is a time-consuming process; however it is advisable to resurvey a few short sections of the pipeline, especially where elevation differences are critical (such as crossing over tops of ridges or bottoms of U-profiles).

An acceptable technique that allows a close check on the accuracy of readings requires a second assistant. One assistant is at the new station ahead of the surveyor, and second assistant is at the last station behind the



All distances & elevations in meters

STATION	GROUND DIST	ANGLE	VERT DIST	ELEV		NOTES & REMARKS
A	ML-0			1000		(A) SOURCE #1 (ARBITRARY ELEV = 1000m)
	8	-22°	-3			
B	8			997		
	7	-35°	-4			
C	15			993		(C) SITE OF COLLECTION TANK 1.5m OF GI FOR GULLY
	12	-5°	-1			
D	27			992		
	18	-26°	-8			
E	45			984		
	20	-27°	-9			
F	65			975		
	25	+7°	+3			Hard rock area
G	90			978		
	30	+12°	+6			
H	120			984		(H) SITE FOR RESEVOIR
	12	-2°	0			
I	132			984		
	23	-5°	-2			
J	155			982		
	24	-7°	-3			
K	179			979		
	27	-7°	-3			
L	206			976		
	14	-3°	-1			
M	220			975		
	30	0°	0			
N	250			975		(N) INTERCEPTS FOOTPATH
	21	-6°	+2			(H) TAP #1 (SCHOOL)
O						

Gravel & Sandy Soil

WOODED JUNGLE

FARM FIELDS

ALONG FOOTPATH

MAINLINE (PROCEEDING DOWNSTREAM FROM SOURCE)

FIGURE 3-6
EXAMPLE FIELD BOOK NOTES

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

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

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

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

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

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

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

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

4. DESIGN PERIOD, POPULATION, AND WATER DEMANDS

4.1 INTRODUCTION

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

4.2 DESIGN PERIOD

Community water supply systems should be designed and constructed for a 15-25 year lifespan. The choice of either a 15, 20, or 25-year design period is made by the surveyor, based upon the amount of potential change that he can foresee for the village. A remote area, far from future development efforts, might well be designed with a 25-year water demand projection. However, in an area where a new highway or airfield is slated for construction, a shorter design period should be considered, because the long-range water demands cannot be accurately forecasted.

4.3 POPULATION FORECAST

Selection of the design period leads directly to an estimate of the village population for the last year of that period. This design population is calculated using the current village population and the population growth factor for the design period, given in Figure 4-1.

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

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

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

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

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

FIGURE 4-1
POPULATION FORECAST TABLE

4.4 WATER DEMANDS

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

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

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

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

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

*Minimal figures are 65% of ideal

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

5. TYPES OF SYSTEMS

5.1 INTRODUCTION

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

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

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

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

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

5.2 OPEN SYSTEMS WITHOUT FAUCETS

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

5.3 OPEN SYSTEM WITH FAUCETS

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

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

5.4 CLOSED SYSTEM WITH RESERVOIR

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

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

5.5 CLOSED SYSTEM WITH INTERMITTENT SERVICE

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

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

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

5.6 CLOSED SYSTEM WITH FLOAT-VALVES

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

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

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

5.7 OPEN SYSTEM VS CLOSED SYSTEM

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

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

5.8 LIMITED EXPANSION

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

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

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

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

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

5.9 PHASED EXPANSION

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

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

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

6. HYDRAULIC THEORY

6.1 INTRODUCTION

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

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

6.2 ENERGY

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

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

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

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

6.3 HEAD: The Measure of Energy

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

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

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

By this practice, a water pressure of 1.4 kg/cm^2 is reported as 14 meters of head; a pressure of 4 kg/cm^2 is 40 meters of head; 5 kg/cm^2 is 50 meters of head, etc.*

6.4 FLUID STATICS: Water at Rest

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

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

Consider the system shown in Figure 6-1. The water pressure at point A is determined by the depth of water at that point. The pressures at points B and C are likewise determined by the height of the vertical distance from those points to the level of the water surface:

Point	Water Pressure	Head
A	1 kg/cm^2	10 meters
B	.2 kg/cm^2	20 meters
C	3.5 kg/cm^2	35 meters

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

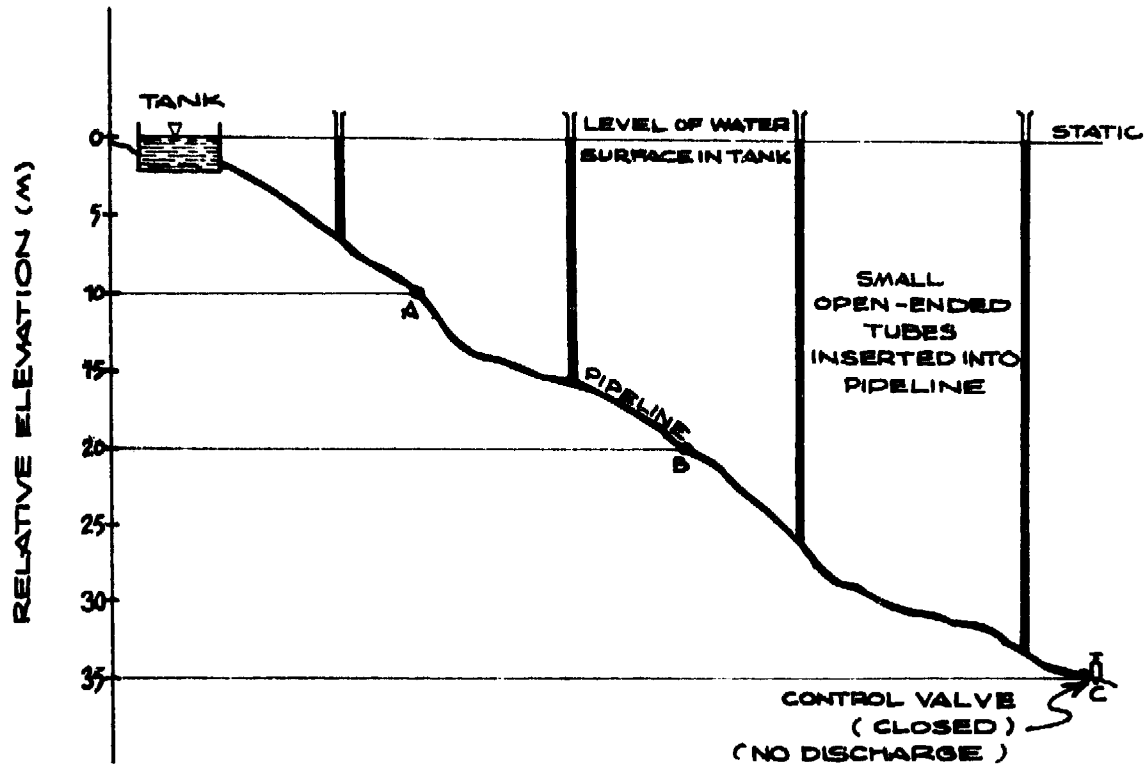


FIGURE 6-1 . STATIC EQUILIBRIUM

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

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

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

6.5 FLUID DYNAMICS: Water in Motion

Now suppose that the control valve at point C in Figure 6-1 is partially opened, allowing a small flow of water through the pipeline (and also assume that the tank refills as fast as it drains, so that the surface level remains constant). The water levels in each glass tube decrease a bit. As the valve is opened further and further to allow greater flows through the pipeline, the water levels in the tubes drop even lower, as shown in Figure 6-2.

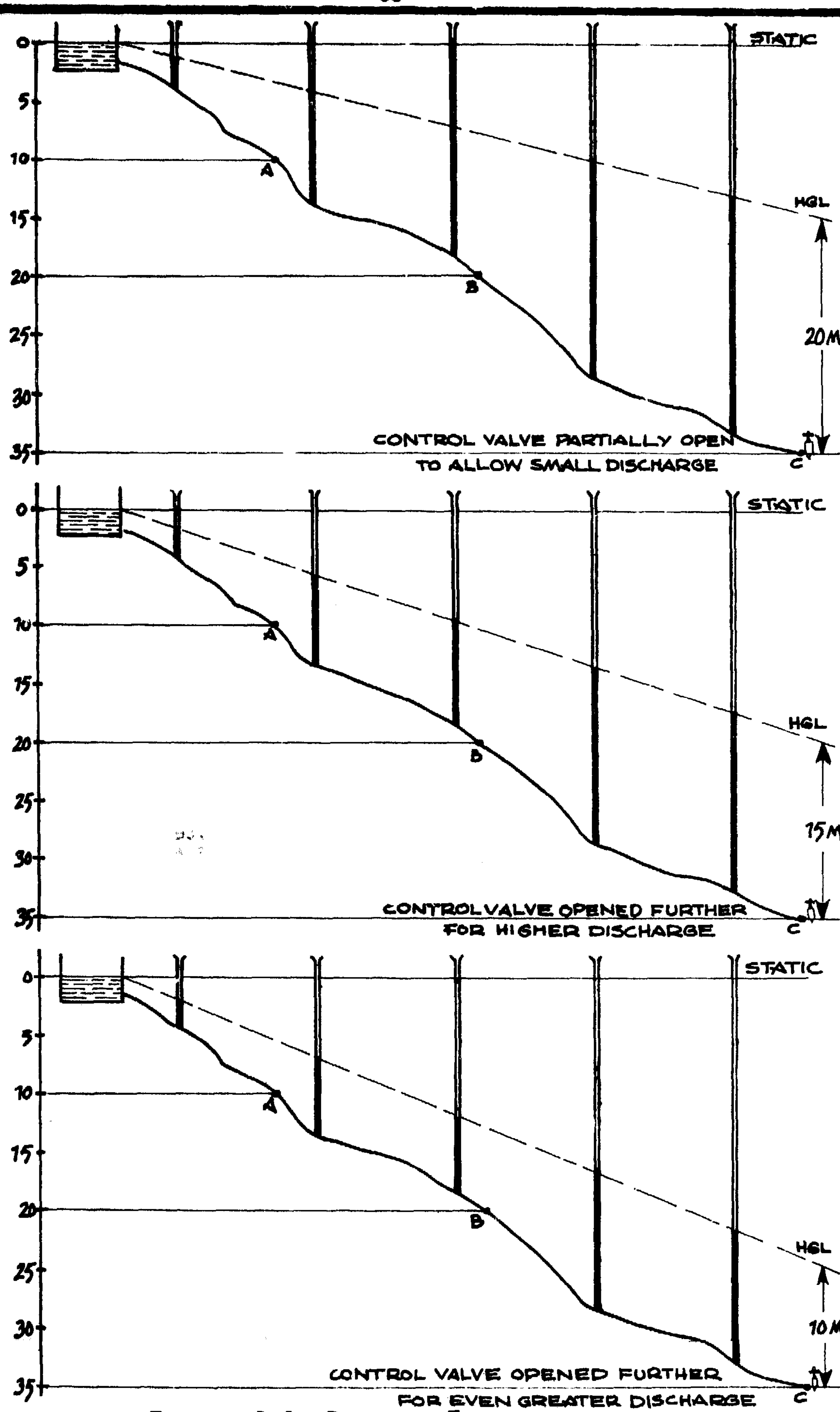


FIGURE 6-2 DYNAMIC EQUILIBRIUM

It can be seen that the water heights in these tubes form a new line for each new flow through the system. For a constant flow, the line formed by the water heights will remain steady. The system is now said to be in dynamic equilibrium. The line formed by the water levels in the tubes is called the hydraulic grade line, commonly abbreviated as HGL. A different flow establishes a different dynamic equilibrium, and a new HGL.

6.6 HYDRAULIC GRADE LINE (HGL)

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

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

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

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

6.7 FRICTION: Lost Energy

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

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

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

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

6.8 VALVES: Variable Friction Devices

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

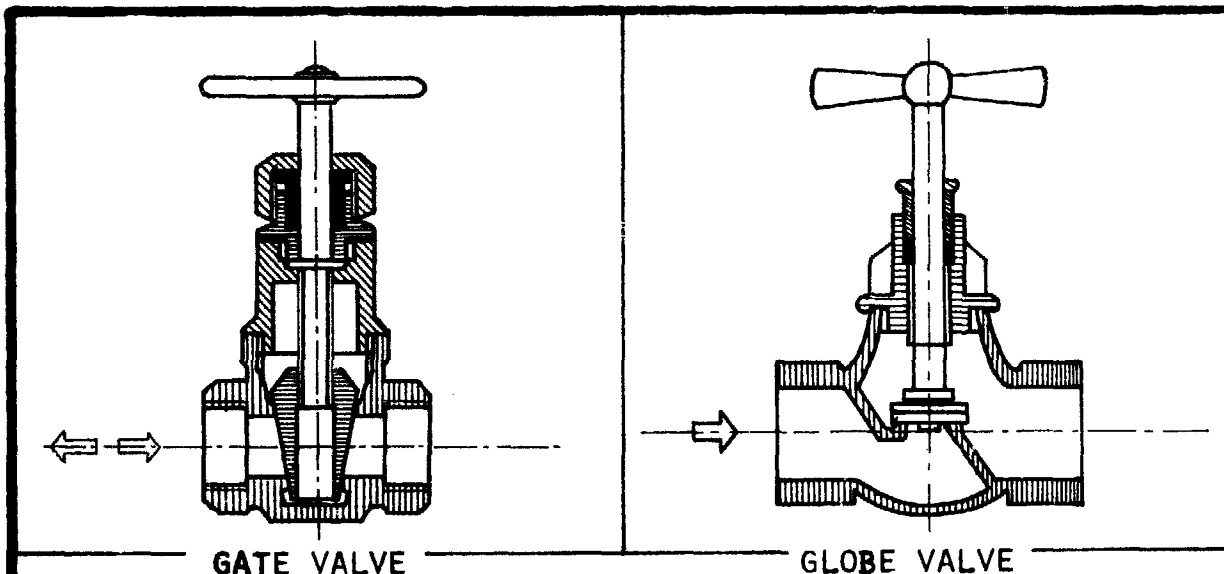


FIGURE 6-3 CONTROL VALVES

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

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

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

6.9 FRICTIONAL HEADLOSS FACTORS

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

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

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

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

Frict'l headloss factor = 2.56m/100m

1350 x 2.56/100 = 34.6 meters headloss

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

Frict'l headloss factor = 1.84%

730 x 1.84/100 = 13.4 meters headloss

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

Frict'l factor = 3.22%

2075 x 3.22/100 = 67 meters headloss

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

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

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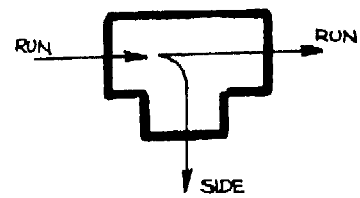
* the 1/2" globe valve used on tapstands is known in Nepal as a corporation cock.

6.10 EQUIVALENT PIPELENTHS OF FITTINGS

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

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



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

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

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

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

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

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

6.11 PLOTTING THE HGL

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

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

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

340m of 50mm HDP @ 0.45 LPS

Frict'l factor = 0.30%

340 x 0.30/100 = 1 meter headloss

This is plotted on the profile.

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

480m of 32mm HDP @ 0.45 LPS

Frict'l factor = 2.56%

480 x 2.56/100 = 12 meters headloss

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

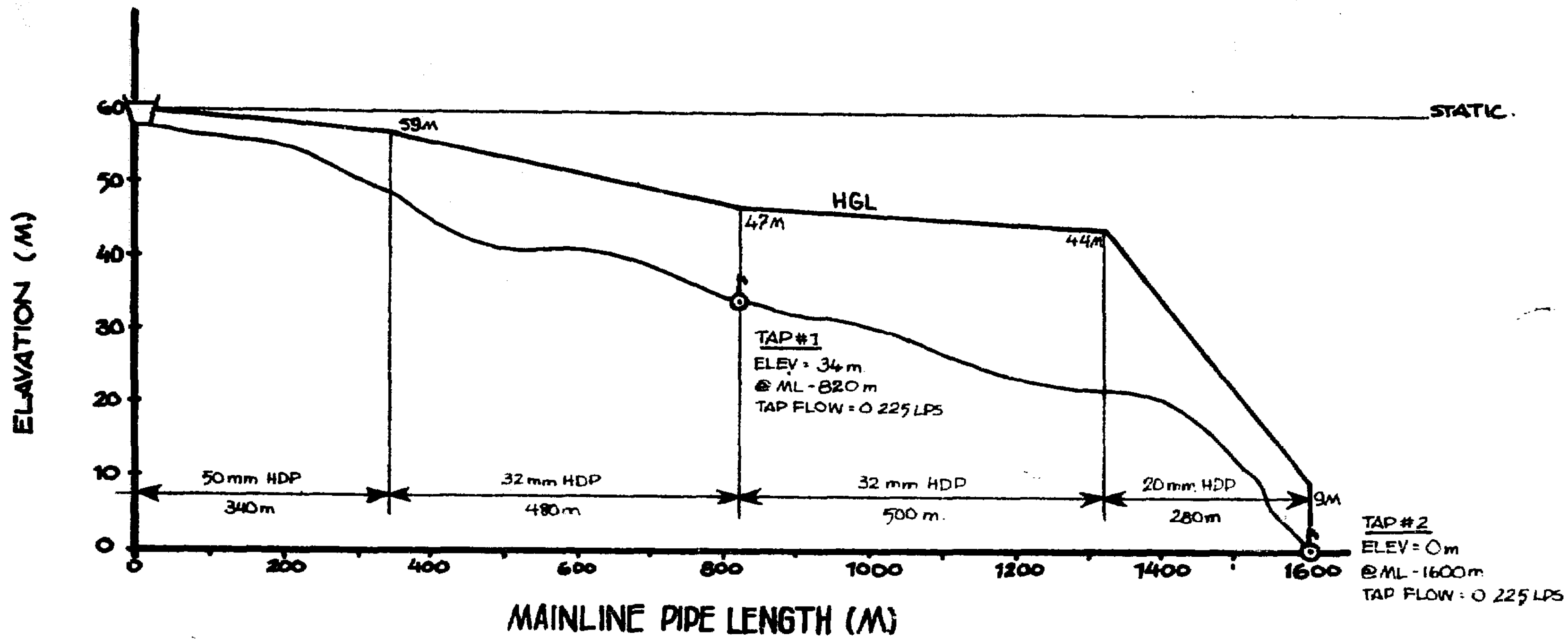
The third reach is from Tap # 1 to the end of the 32mm HDP pipe section: 500 meters of pipe. The desired flow in this section is now only 0.225 LPS (ie- flow for just the remaining single tap).

500m of 32mm HDP @ 0.225 LPS

frict'l factor = 0.78%

500 x 0.78/100 = 4 meters headloss

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



FRICTIONAL HEADLOSSES (M/100m)

	Q=0·225 LPS	0·450 LPS
20 MM.	5·12%	40·00%
32 MM.	0·78%	2·56%
50 MM.	—	0·30%

FIGURE 6-4 PLOTTING THE HGL (EXAMPLE SYSTEM)

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

280m of 20mm HDP @ 0.225 LPS

fric.'l factor = 12.0%

280 x 12/100 = 34 meters headloss

The residual head at Tap # 2 is 9 meters.

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

6.12 REQUIRED HGL PROFILES

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

6.13 RESIDUAL HEAD: Excess Energy

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

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

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

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

A more specific discussion on design residual heads follows:

When plotting the HGL for a flow which discharges freely into the atmosphere (such as into a tank or out of a tap), the residual head at the discharge point may turn out to be eitner positive or negative, as shown in Figure 6-5:

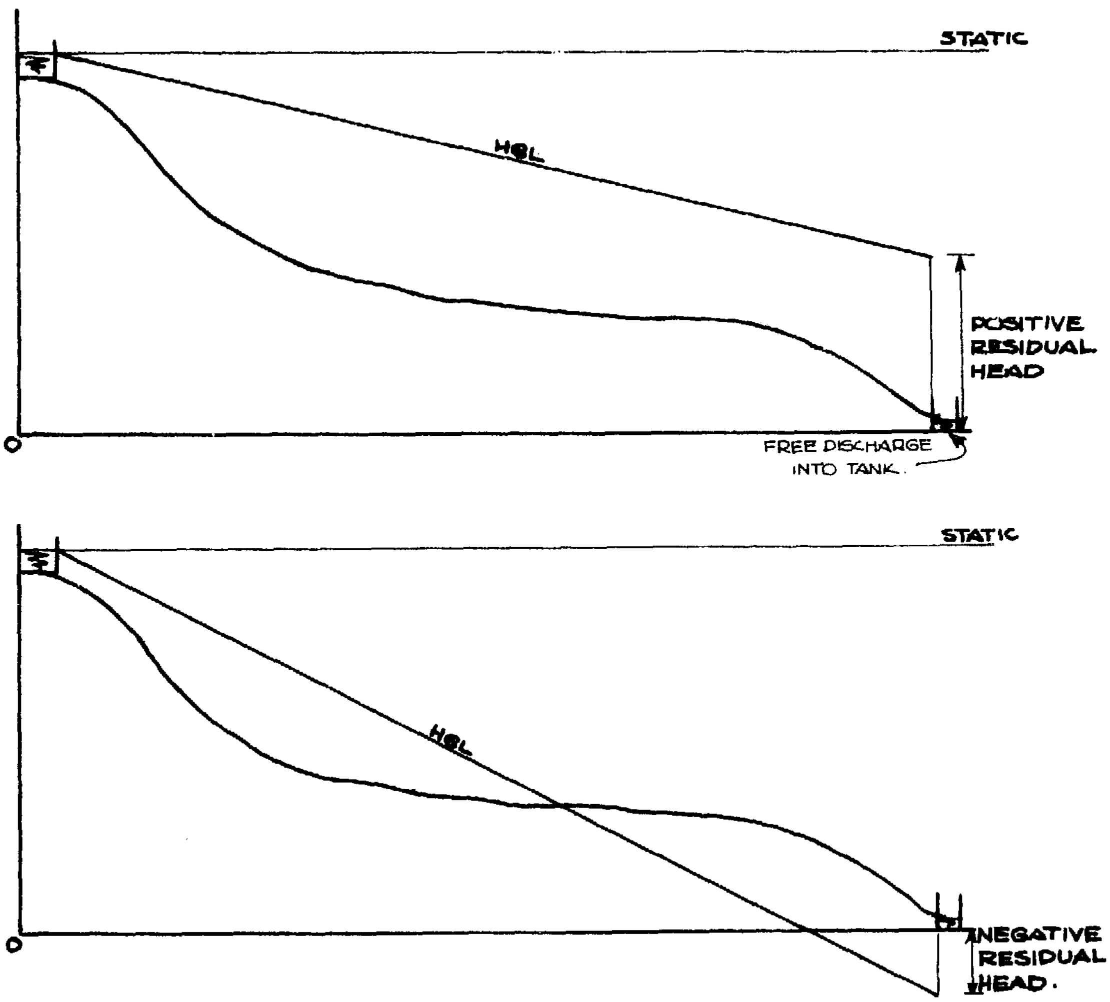


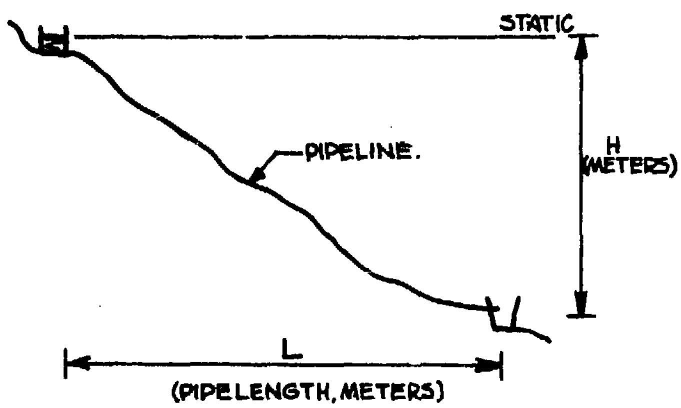
FIGURE 6-5 EXAMPLES OF POSITIVE & NEGATIVE RESIDUAL HEADS

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

Positive residual head: This indicates that there is an excess of gravitational energy; that is, there is enough energy to move an even greater flow through the pipeline. If allowed to discharge freely, a positive residual head means that gravity will try to increase the flow through the pipe; as flow increases, the frictional headlosses will decrease the residual head. The flow will increase until the residual head is reduced to zero.

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

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



NOTES:

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

2. FREE DISCHARGE OF FLOW INTO LOWER TANK.

EXAMPLE:

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

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

SOL'N

THE NATURAL FRICT'L FACTOR.

$FN = \frac{50(100)}{472} = 10.6\ m/100m$

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

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

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

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

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

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

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

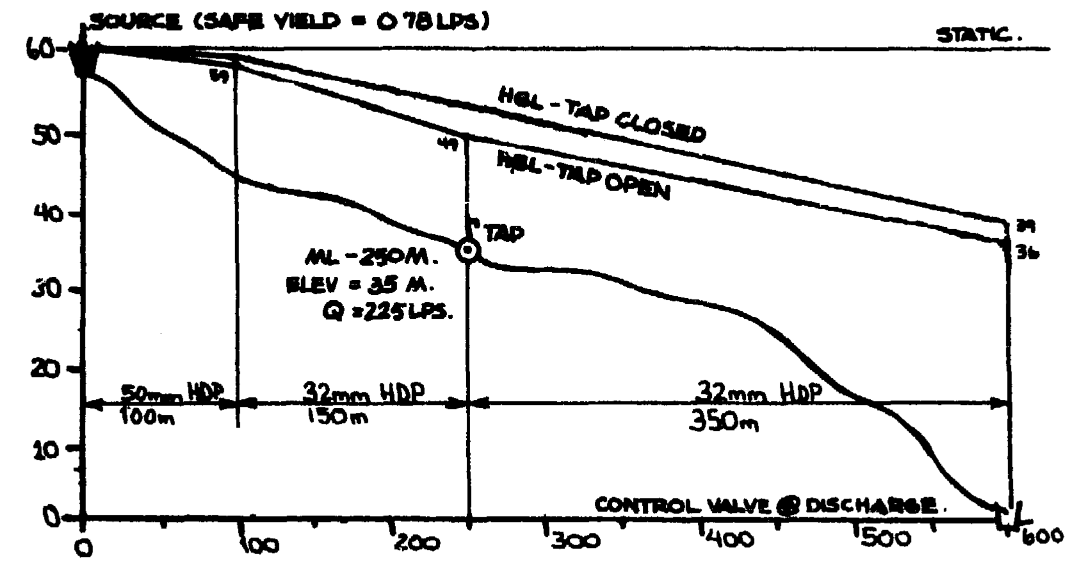
Since every tapstand requires some amount of residual head, then it is obvious that every tapstand requires a control valve. Control valves at discharges into reservoir or break-pressure tanks are only required where it is necessary to keep a specific flow in the pipeline, or to keep the upstream section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

6.14 MAXIMUM PRESSURE LIMITS

As discussed already, it is seen that pipe sizes are selected because of frictional headloss considerations. However, there is yet another consideration which determines what type of pipe must be selected. This consideration is pressure, and will dictate whether Class III HDP pipe, Class IV HDP pipe, or galvanized iron (GI) pipe must be used. The choice is determined by the maximum pressure that the pipe will be subjected to (these maximum pressures are always the result of static pressure levels). The maximum pressure limits for each of these pipes is discussed below:

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

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



TAP OPEN

First Reach (Source--Tap)

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

Second Reach (Tap--Tank)

Q= 0.55 LPS
L= 350m of 32mm HDP
Headloss= 13m
Residual head @ Tank discharge= 36m

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

TAP CLOSED

The equivalent pipeline of the system is:

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

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

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

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

FIGURE 6-7 EQUIVALENT PIPELENGTH

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

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

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

6.15 U-PROFILES & MULTIPLE PIPELINES

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

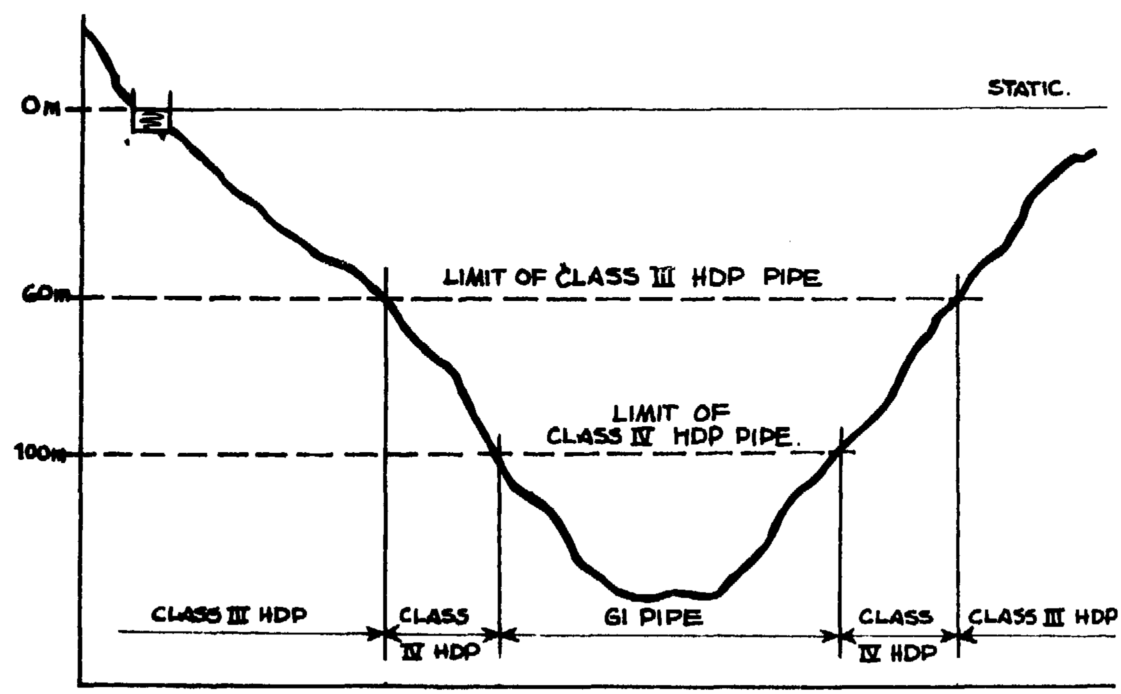
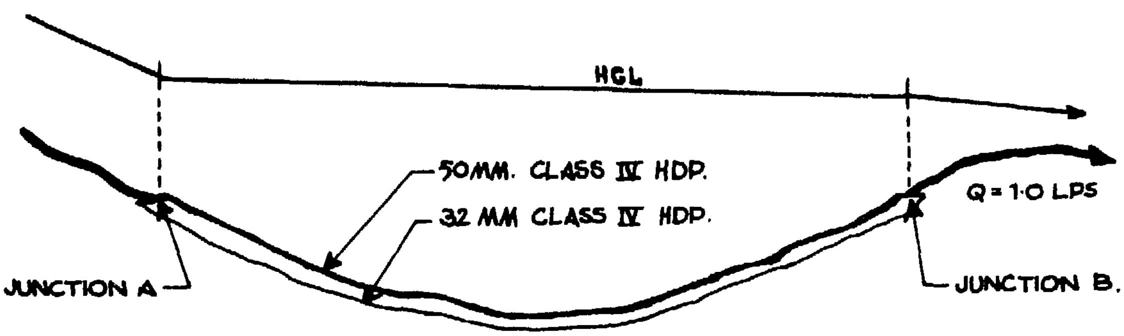


FIGURE 6-8 EXAMPLE OF U-PROFILE

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

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



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

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

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

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

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

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

6.16 MINIMUM PRESSURE LIMITS

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

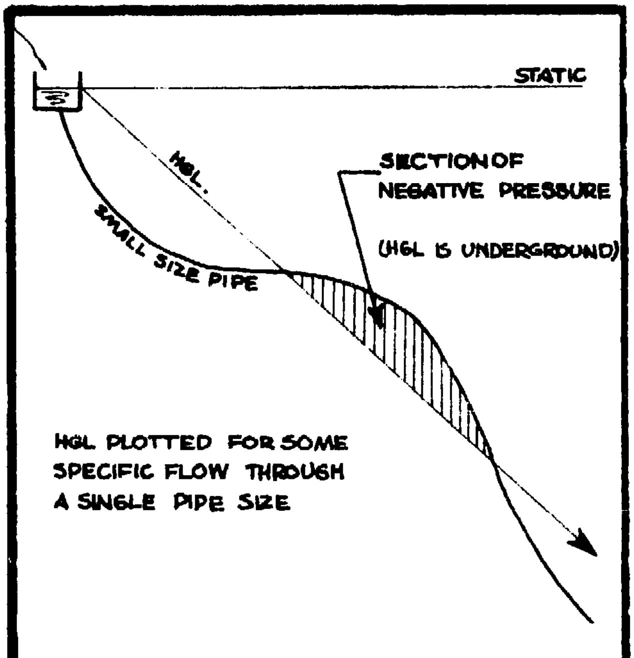


FIGURE 6-10
IMPROPERLY-DESIGNED HGL

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

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

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

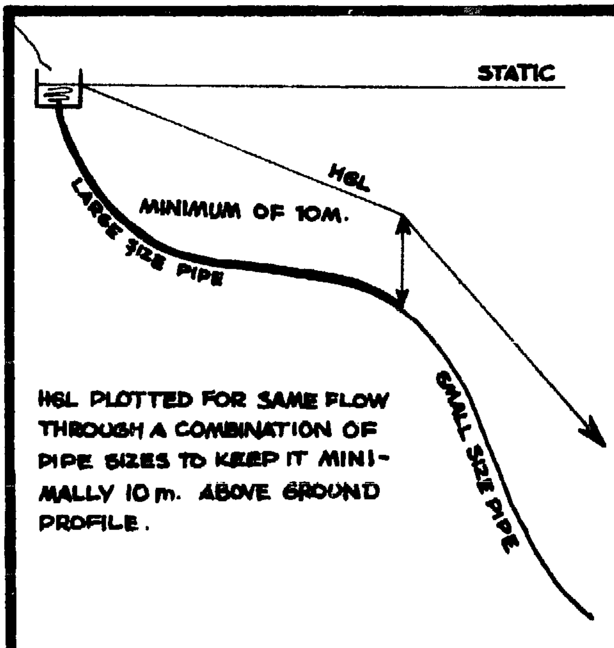


FIGURE 6-11
PROPERLY-DESIGNED HGL

6.17 VELOCITY LIMITS

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

maximum: 3.0 meters/second

minimum: 0.7 meters/second

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

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

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

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

6.18 SUMMATION

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

7. AIR-BLOCKS & WASHOUTS

7.1 INTRODUCTION

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

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

7.2 AIR-BLOCKS, Introduction

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

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

Generally, there will be no problems of air-blocks in a system where a tank is located at an elevation lower than the air-blocks, as long as the air-blocks are at least 10 meters below the static level. This is shown in Figure 7-1 below:

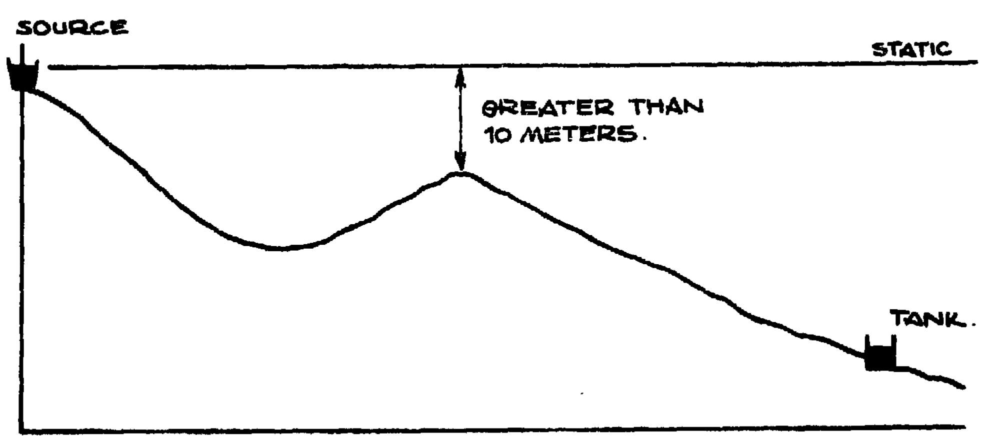


FIGURE 7-1 PROFILE WHERE AIR-BLOCKAGES WILL NOT INTERFERE WITH FLOW

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

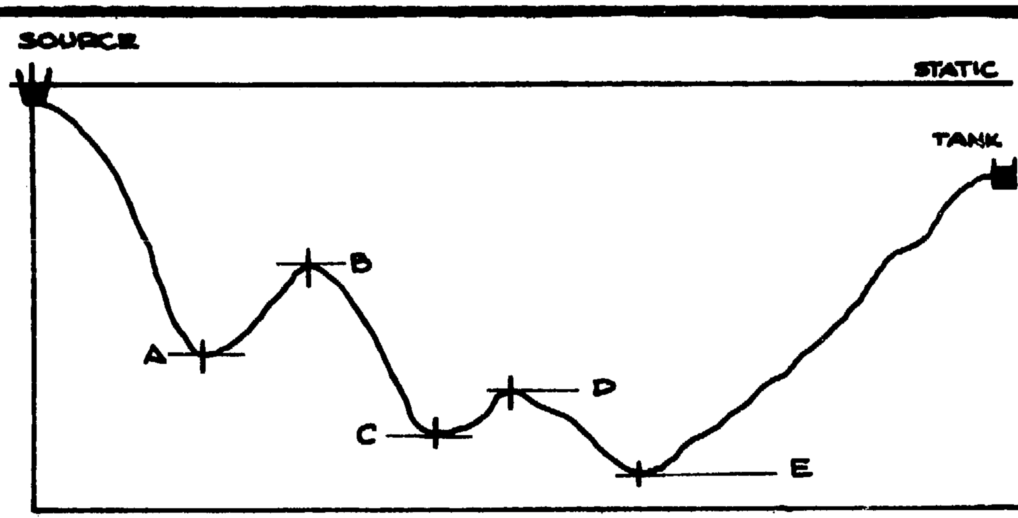


FIGURE 7-2 PROFILE WHERE AIR-BLOCKAGES MAY INTERFERE WITH FLOW

7.3 AIR-BLOCKS: Pipeline Design Practices

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

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

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

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

7.4 AIR-VALVES

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

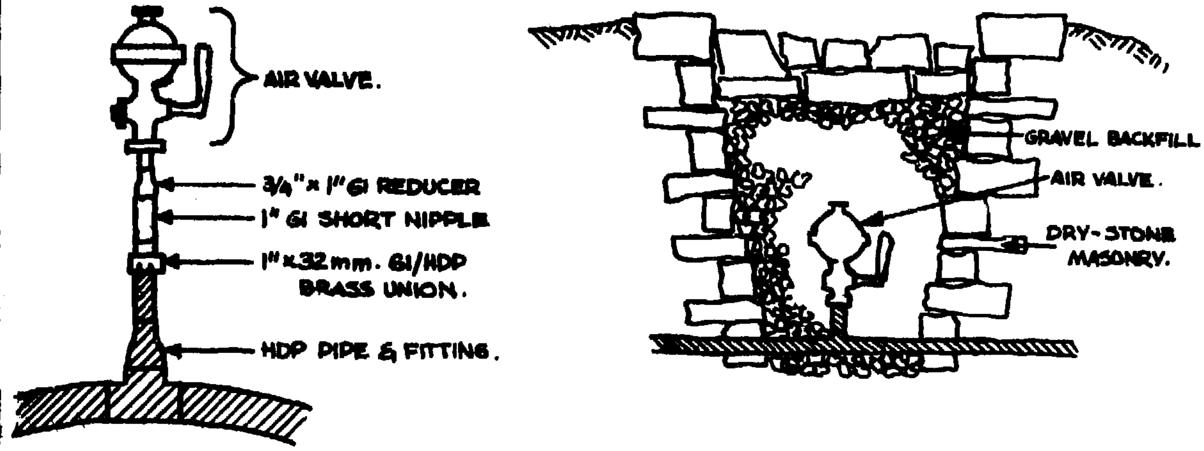


FIGURE 7-3
DETAILS OF AIR-VALVE & INSTALLATION

7.5 ALTERNATIVE AIR-RELEASES

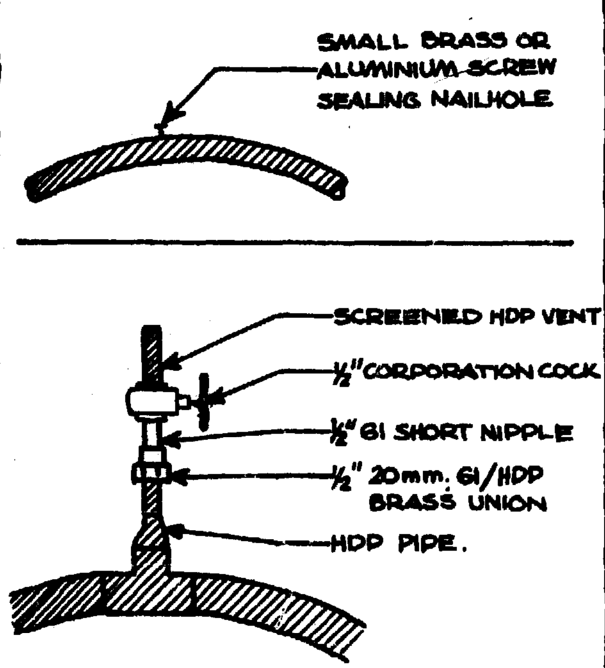


FIGURE 7-4
ALTERNATIVE AIR RELEASES

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

7.6 WASHOUTS

Over a period of time, suspended particles carried in the flow will tend to settle out, particularly at low points in the pipeline or where the flows are low enough so that the flow velocity drops below 0.7 meters/second. Reservoirs usually allow most of these particles to settle, but pipeline sections upstream from the reservoir do not benefit from this. Break-pressure tanks do not allow any sedimentation to occur, since flows through these are extremely turbulent.

Washouts should be located at the bottom points of major U-profiles, especially those upstream from the reservoir tank. The number of washouts in a system depends upon the type of source (a stream yields more suspended materials than a spring), whether or not there is a sedimentation tank and/or reservoir, and the velocity of flow through the pipeline.

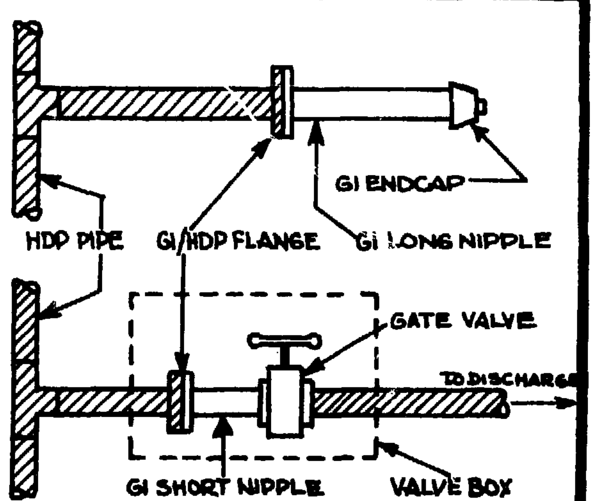


FIGURE 7-5
WASHOUT DESIGNS

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

8. PIPELINE DESIGN

8.1 INTRODUCTION

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

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

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

8.2 PIPELINE DRAWINGS

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

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

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

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

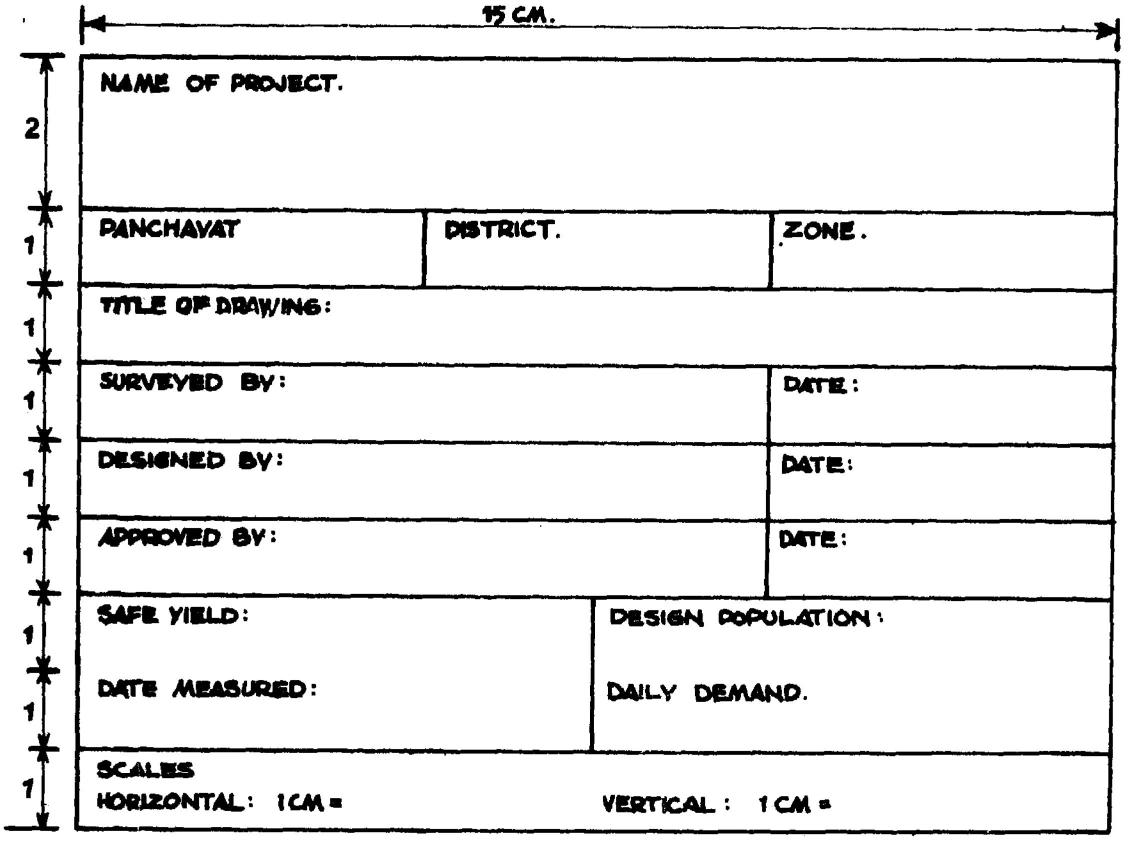


FIGURE 8-1 TITLE BLOCK

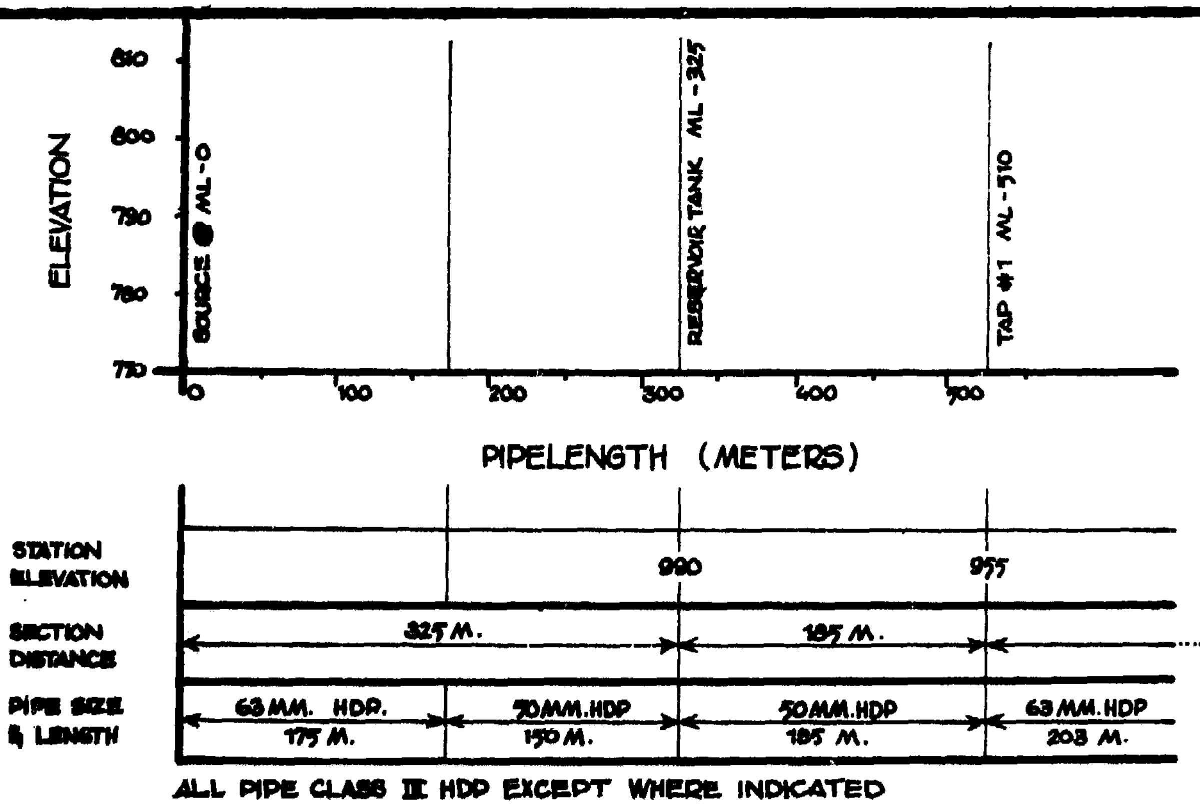


FIGURE 8.2
PORTION OF SAMPLE DESIGN SHOWING PROFILE AXES

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

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

of this book, and are not the ones referred to in the following examples.

8.3 DESIGN EXAMPLE: Mainline

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

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

Reservoir calculations:

Safe yield of source = 1.40 LPS

Demand by 6 taps @ 0.225 LPS = 1.35 LPS

Therefore reservoir tank not required

Preliminary pressure analysis:

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

Reference Table VIII (Cont'd)

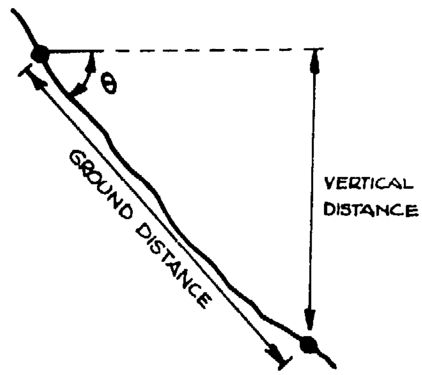
Angle	Sine	Cosine	Tangent	Cotan	
23°	.3907	.92050	.4245	2.356	67°
23°30'	.3987	.91706	.4348	2.300	66°30'
24°	.4067	.91355	.4452	2.246	66°
24°30'	.4147	.90996	.4557	2.194	65°30'
25°	.4226	.90631	.4663	2.145	65°
25°30'	.4305	.90259	.4770	2.097	64°30'
26°	.4384	.89879	.4877	2.050	64°
26°30'	.4462	.89493	.4986	2.006	63°30'
27°	.4540	.89101	.5095	1.963	63°
27°30'	.4617	.88701	.5206	1.921	62°30'
28°	.4695	.88295	.5317	1.881	62°
28°30'	.4772	.87882	.5430	1.842	61°30'
29°	.4848	.87462	.5543	1.804	61°
29°30'	.4942	.87063	.5658	1.767	60°30'
30°	.5000	.86603	.5774	1.732	60°
30°30'	.5075	.86163	.5890	1.698	59°30'
31°	.5150	.85717	.6009	1.664	59°
31°30'	.5225	.85264	.6128	1.632	58°30'
32°	.5299	.84805	.6249	1.600	58°
32°30'	.5373	.84339	.6371	1.570	57°30'
33°	.5446	.83867	.6494	1.540	57°
33°30'	.5519	.83389	.6619	1.511	56°30'
34°	.5592	.82904	.6745	1.483	56°
34°30'	.5664	.82413	.6873	1.455	55°30'
35°	.5736	.81915	.7002	1.428	55°
35°30'	.5807	.81412	.7133	1.402	54°30'
36°	.5878	.80902	.7265	1.376	54°
36°30'	.5948	.80386	.7400	1.351	53°30'
37°	.6018	.79864	.7536	1.327	53°
37°30'	.6088	.79335	.7673	1.303	52°30'
38°	.6157	.78801	.7813	1.280	52°
38°30'	.6225	.78261	.7954	1.257	51°30'
39°	.6293	.77715	.8098	1.235	51°
39°30'	.6361	.77162	.8243	1.213	50°30'
40°	.6428	.76604	.8391	1.192	50°
40°30'	.6494	.76041	.8541	1.171	49°30'
41°	.6561	.75471	.8693	1.150	49°
41°30'	.6626	.74896	.8847	1.130	48°30'
42°	.6691	.74314	.9004	1.111	48°
42°30'	.6756	.73728	.9163	1.091	47°30'
43°	.6820	.73135	.9325	1.072	47°
43°30'	.6884	.72537	.9490	1.054	46°30'
44°	.6947	.71934	.9657	1.036	46°
44°30'	.7009	.71325	.9827	1.018	45°30'
	.7071	.70711	1.	1.000	45°
	Cosine	Sine	Cotan	Tangent	Angle

<u>Reference Table IX</u>

VERTICAL DISTANCE SURVEY TABLE

The table below directly presents the vertical elevation difference between two stations, where the ground distance and vertical angle between the stations is known.

The distance may be either in feet or meters, and direct interpolation between two distances is allowable.



VERTICAL ANGLE θ

GROUND DISTANCE	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	11°	12°	13°
2	0	0	0	0	0	0	0	½	½	½	½	½	½
4	0	0	0	½	½	½	½	½	½	½	1	1	1
6	0	0	½	½	½	½	½	1	1	1	1	1	1½
8	0	½	½	½	½	1	1	1	1½	1½	1½	1½	2
10	0	½	½	½	1	1	1	1½	1½	1½	2	2	2
12	0	½	½	1	1	1	1½	1½	2	2	2½	2½	2½
14	0	½	½	1	1	1½	1½	2	2	2½	2½	3	3
16	½	½	1	1	1	1½	2	2	2½	3	3	3½	3½
18	½	½	1	1	1½	2	2	2½	3	3	3½	3½	4
20	½	½	1	1½	1½	2	2½	3	3	3½	4	4	4½
22	½	1	1	1½	2	2½	2½	3	3½	4	4	4½	5
24	½	1	1	1½	2	2½	3	3½	3½	4	4½	5	5½
26	½	1	1½	2	2½	2½	3	3½	4	4½	5	5½	6
28	½	1	1½	2	2½	3	3½	4	4½	5	5½	6	6½
30	½	1	1½	2	2½	3	3½	4	4½	5	5½	6	6½

GROUND DISTANCE	14°	15°	16°	17°	18°	19°	20°	21°	22°	23°	24°	25°
2	½	½	½	½	½	½	½	½	½	1	1	1
4	1	1	1	1	1	1½	1½	1½	1½	1½	1½	1½
6	1½	1½	1½	1½	2	2	2	2	2	2½	2½	2½
8	2	2	2	2½	2½	2½	2½	3	3	3	3½	3½
10	2½	2½	2½	3	3	3½	3½	3½	3½	4	4	4
12	3	3	3½	3½	3½	4	4	4½	4½	4½	5	5
14	3½	3½	4	4	4½	4½	5	5	5	5½	5½	6
16	4	4	4½	4½	5	5	5½	5½	6	6½	6½	7
18	4½	4½	5	5½	5½	6	6	6½	6½	7	7½	7½
20	5	5	5½	6	6	6½	7	7	7½	8	8	8½
22	5½	5½	6	6½	7	7	7½	8	8	8½	9	9½
24	6	6	6½	7	7½	8	8	8½	9	9½	10	10
26	6½	6½	7	7½	8	8½	9	9½	9½	10	10½	11
28	7	7	7½	8	8½	9	9½	10	10½	11	11½	12
30	7½	7½	8½	9	9½	10	10½	10½	11	11½	12	12½

Reference Table X

GI FRICTIONAL HEADLOSS FACTORS

These are the approximate headloss factors, in m/100m (%), for aged (10-15 years) GI pipe. Flows are in liters/second.

FLOW	½" GI	1" GI	1½" GI	2" GI	3" GI
0.10	5.87	0.38			
0.15	12.24	0.82			
0.20	21.43	1.33	0.20		
0.225	26.53	1.68	0.22		
0.25	31.63	2.04	0.24		
0.30	44.90	2.96	0.40		
0.35	58.16	3.82	0.52		
0.40	74.49	4.79	0.66	0.22	
0.45	91.84	6.02	0.83	0.27	
0.50		7.14	1.02	0.33	
0.55		8.67	1.19	0.39	
0.60		10.20	1.43	0.46	
0.65		11.73	1.63	0.53	
0.675		12.76	1.68	0.55	
0.70		13.27	1.73	0.58	
0.75		15.31	2.14	0.67	
0.80		17.35	2.35	0.77	
0.85		18.88	2.65	0.87	
0.90		21.43	2.86	0.92	
0.95		23.47	3.27	1.02	
1.00		25.51	3.57	1.12	
1.05		29.39	3.83	1.22	
1.10		30.61	4.18	1.33	
1.15		34.69	4.59	1.48	
1.20		35.71	4.92	1.58	
1.30		40.82	5.71	1.84	0.22
1.40		47.96	6.63	2.14	0.26
1.50		54.08	7.55	2.45	0.28
1.60		61.22	8.47	2.65	0.32
1.70		67.35	9.49	3.06	0.35
1.80		76.53	10.51	3.47	0.41
1.90			11.73	3.78	0.43
2.00			12.76	4.08	0.49
2.20			15.31	4.90	0.57
2.40			17.86	5.71	0.66
2.60			20.41	6.63	0.81
2.80			24.49	7.65	0.92
3.00			26.53	8.67	1.02
3.20			29.59	9.69	1.12
3.40			33.67	10.92	1.32

Reference Table X (Cont'd)

FLOW	½"GI	1" GI	1½" GI	2" GI	3" GI
3.60			37.76	12.24	1.43
3.80			40.82	13.27	1.58
4.00			45.92	14.79	1.73
4.50			56.12	17.86	2.09
5.00				21.43	2.55
5.50				26.53	3.06
6.00				30.61	3.67
6.50				35.71	4.18
7.00				40.82	4.85

Reference Table XI

HDP FRICTIONAL HEADLOSS FACTORS

These are the approximate headloss factors, in m/100m (%) of HDP pipe manufactured according to UNICEF specifications. Due to the variations in size and quality by different manufacturers, no exact headloss factors will ever be possible.

Headloss factors less than 0.20% are considered negligible. Factors are provided for up to the maximum recommended flow velocity of 3.0 m/sec; factors for flow velocities of less than 0.7 m/sec are flagged by an asterisk (*). Flows are for liters/second.

CLASS III (6 Kg./cm²)

FLOW	32mm	50mm	63mm	90mm
0.10	0.22*			
0.12	0.30*			
0.14	0.38*			
0.16	0.48*			
0.18	0.58*			
0.20	0.72*			
0.225	0.87*			
0.25	1.08*			
0.275	1.27*			
0.30	1.42*			
0.35	1.88*	0.22*		
0.40	2.44	0.28*		
0.45	2.87	0.34*		
0.50	3.70	0.40*		
0.55	4.1	0.47*		
0.60	4.9	0.56*		
0.65	5.6	0.63*		
0.675	5.9	0.67*		
0.70	6.3	0.72*	0.25*	
0.75	7.3	0.81*	0.28*	
0.80	8.2	0.90*	0.31*	
0.90	10.0	1.11*	0.37*	
1.00	11.9	1.34*	0.45*	
1.10	14.1	1.57	0.54*	
1.20	16.5	1.90	0.63*	
1.30	19.0	2.18	0.73*	
1.40	21.6	2.46	0.82*	
1.60	27.4	3.05	1.03*	
1.80	33.6	3.81	1.30	0.24*
2.00		4.6	1.55	0.28*
2.20		5.5	2.46	0.32*
2.50		6.7	2.24	0.40*
2.70		8.3	2.74	0.49*
3.00		9.5	3.08	0.56*
3.20		11.2	3.77	0.67*

FLOW	50mm	63mm	90mm
3.50	12.6	4.1	0.74
3.70	14.4	4.8	0.84
4.00	15.7	5.4	0.96
4.20	18.1	6.0	1.04
4.50	20.1	6.6	1.06
4.70		7.4	1.31
5.00		7.8	1.41
5.50		9.4	1.70
6.00		11.1	2.00
6.50		12.3	2.24
7.00		14.6	2.97

20mm HDP is only available in Class IV series

Reference Table XI (Cont'd)

CLASS IV (10 Kg./cm^2)

FLOW	20mm	32mm	50mm	63mm	90mm
0.10	3.1*	0.31*			
0.12	4.5*	0.40*			
0.14	5.6*	0.52*			
0.16	7.3	0.67*			
0.18	9.0	0.81*			
0.20	10.6	0.99*			
0.225	13.4	1.22*			
0.25	15.7	1.43*			
0.275	18.5	1.74*			
0.30	21.8	2.02*	0.25*		
0.35	28.[illegible]	2.71*	0.31*		
0.40	36.2	3.36	0.40*		
0.45	45	4.0	0.49*		
0.50	54	4.9	0.59*		
0.55	63	5.7	0.71*		
0.60	73	6.7	0.81*	0.27*	
0.65		7.8	0.94*	0.31*	
0.675		8.4	1.00*	0.34*	
0.70		8.8	1.06*	0.36*	
0.75		10.1	1.23*	0.40*	
0.80		11.2	1.34*	0.45*	
0.90		13.4	1.67*	0.54	
1.00		16.4	2.00*	0.66*	
1.10		19.8	2.37	0.78*	
1.20		22.6	2.77	0.92*	
1.30		26.4	3.19	1.08*	
1.40		30.2	3.61	1.23*	0.22*
1.60		37.5	4.5	1.52	0.28*
1.80			5.6	1.85	0.35*
2.00			6.7	2.24	0.41*
2.20			8.3	2.69	0.49*
2.50			10.1	3.25	0.60*
2.70			12.2	3.92	0.73*
3.00			13.9	4.6	0.84
3.20			17.4	5.5	0.99
3.50			18.9	6.2	1.11
3.70			21.3	6.9	1.29
4.00			23.7	7.7	1.43
4.20				8.7	1.57
4.50				9.5	1.78
4.70				10.8	1.97
5.00				11.8	2.13
5.50				13.4	2.46
6.00				15.9	2.91
6.50					3.36
7.00					3.89

Reference Table XII

FLOW NOMOGRAPH FOR SMOOTH PLASTIC PIPE AND IRON PIPE