Civil Works Guidelines for
MICRO-HYDROPOWER IN NEPAL

BPC Hydroconsult

PRACTICAL ACTION
Technology challenging poverty

1. Introduction

1.1 Hydropower and micro–hydropower

Hydropower is the generation of power (mechanical and/or electrical) using the fall of water. In the context of Nepal, a hydropower scheme with an installed capacity of less than 100 kW is classified as micro-hydro. Schemes in the range 100-1000 kW are classified as mini-hydro and share some of the characteristics of micro-hydropower schemes. Apart from the power output of schemes, some of the major differences between large and micro-hydro are shown below in Table 1.1.

SOME DEFINITIONS

- Civil engineering is the application of science to the practical building of safe and cost effective structures.
  A structure is an assembly of materials which serves the purpose for which it is designed (accommodate people, convey flow, traffic, etc.) and carries the associated loads. A civil engineering structure is specifically designed to fulfill a purpose and/or perform a function at an appropriate quality and to an acceptable time scale and cost.
- *Civil works* are all activities necessary for the building of structures.
  Storage schemes make use of a dam to stop river flow, building up a reservoir of water behind the dam. The water is then released through turbines when power is needed.
- *Run-of-river* schemes do not stop the river flow, but instead divert part of the flow via a headrace and penstock to a turbine. Therefore, the full power capacity (also referred to as the Installed Capacity) is only generated as long as the river flows permit. If the river flow is less than required for full power generation, the power output decreases proportionally. On the other hand when river flow is high, the excess flow (i.e., flow higher than required for full power generation) runs down the river without being utilised. Sometimes a small pond is also constructed in run-of-river schemes to store some water during off peak hours to generate full capacity during the peak hours. Micro-hydro schemes are almost always run-of-the-river type.

1.2 Aspects of civil engineering works

The design and construction of civil engineering works have some important characteristics:
- They are dependent on conditions at the site. No two sites are the same.
- They always involve structures that are in contact with the ground. Design engineers may have control over the materials used in construction, but have limited control over the ground on which the structure stands. They must therefore take into account the ground conditions, and may have to consider alternative sites to avoid problems with instability.
- They often involve a number of people working on design, supervision and construction at the site. Various skills and materials are involved, usually over a period of several months. Therefore planning, communication and accountability are very important factors.
- Failure of civil works can be very dangerous and expensive. Similarly, poor performance or over-design are uneconomical.

TABLE 1.1 **Differences between micro-hydro and large hydro schemes**

DESCRIPTION	MICRO-HYDRO SCHEME	LARGE HYDRO SCHEMES
Type	Mostly run-of-river	Both run-of-river and storage
Power generation	Electrical and/or mechanical	Electrical only
Nature of intake	Usually temporary or semi-permanent	Permanent
Tunnels & underground structures	Rare	Common
Penstock alignment	Vertical & horizontal bends	Fewer vertical and horizontal bends
Surge shaft	Rare, forebay acts as surge tank	Common
Distribution system	Isolated (i.e. not connected to the national electricity grid) but could be connected to the mini-grid.	Mostly grid connected
Unlined canal	Common	Rare

One important point should be recognised: complete standardisation of civil works is not possible due to the variation in site conditions. Instead, standard approaches to design are used, providing methods and criteria that enable a design to be adapted to conditions at a site.

Safe, accurate and economic design are fundamental in civil engineering but, because of site variations, a practical understanding of design is also crucial. Failures in civil engineering do not usually occur through an error in calculations but because of a seemingly minor event or circumstance which did not seem important. A thorough understanding of which elements are critical is required.

1.3 Components of micro-hydro schemes

Although no two micro-hydro sites are similar, all of them require specific common components of different dimensions to convey the stream water to the power generation units and back into the stream. These components are shown in Figure 1.1. The civil components are briefly discussed below:

HEADWORKS
Structures at the start of the scheme are collectively called the headworks. In micro-hydro schemes, the headworks always include the diversion weir, intake and gravel trap. A spillway and a settling basin are also usually at the headworks.

DIVERSION WEIR
A diversion weir is a low structure (small dam) placed across the river which diverts some of the river flow into the hydropower scheme. The weir can be of permanent, semi-permanent or temporary nature.

INTAKE
This is at the riverbank upstream from the diversion weir where water is initially drawn into a conduit (canal or a pipe). Usually a flow control structure and a coarse trashrack are incorporated at the intake.

INTAKE CANAL
Generally the gravel trap is sited away from the intake at some downstream location to protect it from flood and to provide sufficient flushing head. If such an arrangement is made in the project layout, then an intake canal will convey the flow that enters in the intake to the gravel trap. Whenever possible, provision for a spillway should be made immediately downstream of the intake so that excess flow during the flood season can be spilt back into the river. This will improve the settling efficiency of the gravel trap as only the desired flow reaches this structure. However, care should be taken to site the spillway such that flood flow will not re-enter to the waterways system via the spill way instead of spilling the excess water. Though the intake canal is steeper than the headrace canal, sometimes both types of canals are referred to as "headrace canal."

GRAVEL TRAP
This is a basin (pond) close to the intake where gravel and other coarse materials are trapped and then removed. In the absence of this structure gravel can settle along the gentler section of the headrace or in the settling basin.

SETTLING BASIN
This is also a basin where sand and other fine suspended particles present in the river water are settled and then removed. If allowed to enter the penstock, such particles would abrade the penstock pipe and the turbine and hence shorten their operational lives.

HEADRACE
This is a canal or a pipe that conveys the water from the headworks to the forebay structure. The headrace alignment is usually on even to gently sloping ground; a headrace pipe is generally not subject to significant hydraulic pressure. Sometimes the canal stretch from the intake structure to the gravel trap is also referred to as the 'Intake Canal". This section of the canal is generally steeper than the headrace canal downstream as it needs to convey the gravels along with discharge from the intake to the gravel trap. Similarly, the canal stretch from the gravel trap to the settling basin is also referred to as the "Approach Canal". This section of the canal is also steeper than the headrace downstream as it needs to convey sediments along with the discharge. However, the slope of the approach canal can be lower than that of the intake canal, i.e., a less sloped area is required to convey suspended sediments than gravels. In these guidelines, the entire canal (or pipe) stretch from the intake to the forebay is referred to as the headrace.

FOREBAY
This is a tank at the entrance to the penstock pipe. The forebay tank allows for flow transition from open channel to pressure flow, maintains submergence depth for the penstock pipe to avoid vortex formation and provides storage when there are flow fluctuations in the turbine. It can also serve as a final settling basin. In fact, sometimes the settling basin and the forebay structures are combined together. An overflow spillway should always be provided in the forebay structure to allow spilling of the entire flow in case of emergency plant closure and excess flow in case of excessive load fluctuations.

SPILLWAYS AND ESCAPES
Spillways are openings in headrace canals that divert excess flow and only allow the design flow downstream. Note that some literature may use the terms spill weir or overflow to refer to the spillway. Escapes are similar in structure but their function is to divert flows from the headrace canals in case the downstream sections get blocked (in case of a landslide).

CROSSINGS

These are structures that convey the flow over streams, gullies or across unstable terrain subject to landslides and erosion. Aqueducts, super passage, culverts and suspended crossings are examples of such structures.

PENSTOCK

This is a pipe that conveys water under pressure from the forebay to the turbine. The penstock pipe usually starts where the ground profile is steep. Some times a long penstock could be laid from headwork to the power house if the topography is suitable (e.g., alignment is suitable for a penstock alignment right from the intake). In such project layout a forebay is constructed at the headworks area and is combined with the settling basin.

ANCHOR BLOCK

An anchor block (thrust block) is an encasement of a penstock designed to constrain the pipe movement in all direction. Anchor blocks are placed at all sharp horizontal and vertical bends, since there are forces at such bends that will tend to move the pipe out of alignment. Anchor blocks are also required to resist axial forces in long straight sections of penstock.

SUPPORT PIER

Support piers (also called slide blocks or saddles) are structures that are used along straight runs of exposed penstock pipe (between anchor blocks), to prevent the pipe from sagging and becoming overstressed. They need to resist all vertical forces such as the weight of the penstock pipe and water. However, they should allow movement parallel to the penstock alignment which occurs during thermal expansion and contraction processes.

POWERHOUSE

This is a building that accommodates and protects the electro-mechanical equipment such as the turbine, generator and may include agro-processing units. The electro-mechanical equipment in the powerhouse converts the potential and kinetic energy of water into electrical energy.

TAILRACE

This is a channel or a pipe that conveys water from the turbine (after power generation) back into the stream; generally the same stream from which water was initially withdrawn.
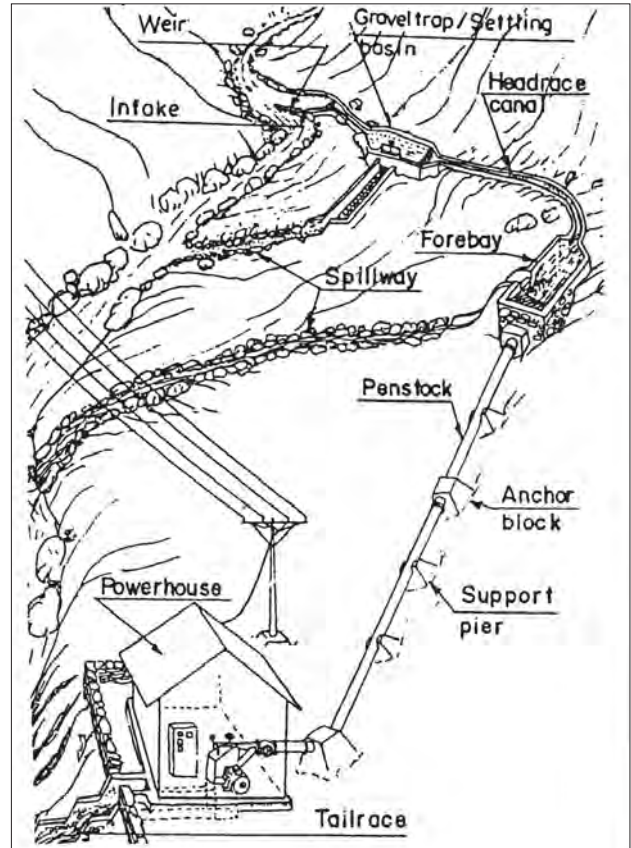
Detailed descriptions of these components including selection, design and construction methodology are discussed in subsequent chapters.

1.4 The power equation

The power available from a hydropower scheme is dependent on the volume flowing in the system and its drop in height. The relationship is expressed by the commonly used power

Figure 1.1 Components of a micro-hydro scheme

equation as follows:

$$P = Q \times g \times h_{gross} \times e_o$$

where:

P is the power produced in kW

Q is the flow in the penstock pipe in m^3/s

g is the acceleration due to gravity ($9.8\ m/s^2$)

h_{gross} is the gross head available in m

e_o is the overall system efficiency

Gross head, h_{gross}, is the difference between the water level at the forebay and the turbine centreline level (or tailrace water surface if a draft tube is used). This is shown schematically in Figure 1.2.
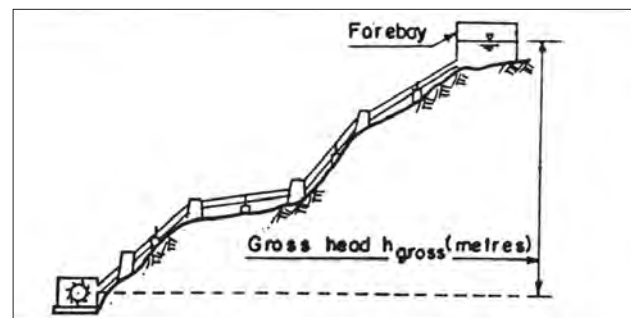

Figure 1.2 Head is the vertical height through which the water drops

However, based on first principles from physics, the power equation should be

$$P = e_o \times \gamma \times Q \times h_{gross}$$

where:

Other variables are same as above and

γ is the unit weight of water = 9.8 kN/m^3 Note that in the power equation based on first principles, γ is replaced by 'g' and since both have the same absolute value (9.81), the results are the same, i.e., same power output in kW for same flow (m^3/s), gross head (m) and overall system efficiency

Net head, h_n, is the pressure head at the entrance to the turbine. That is, the gross head minus conveyance losses in the penstock. For micro-hydropower schemes the penstock is generally designed such that the net head is 90-95% of the gross head measured from the forebay (refer to Section 6.4 for penstock sizing).The overall system efficiency, e_o, is the ratio of useful power output to hydraulic power input. It is the product of separate efficiencies for several components of the system, i.e.,

$$e = e_p \, e_t \, e_g \, e_{tr}$$

where,

e_p is the penstock efficiency, typically 0.90 - 0.95 ($h_n = h_{gross} e_p$)

e_t is the turbine efficiency, typically 0.65 - 0.80 depending on turbine type

e_g is the generator efficiency, typically 0.65 - 0.90 dependingon size

e_{tr} is the transmission efficiency including transformers if used, typically 0.85 - 0.90

For preliminary planning of micro-hydropower schemes in Nepal it is common to assume an overall system efficiency of 0.5 to 0.6. However, it may be as low as 0.3 for very small installations and as high as 0.7 for larger schemes. Therefore at detailed design stage it is important to recalculate the power output based on the actual design and manufacturers' data for the proposed equipment.

2. Site selection and planning
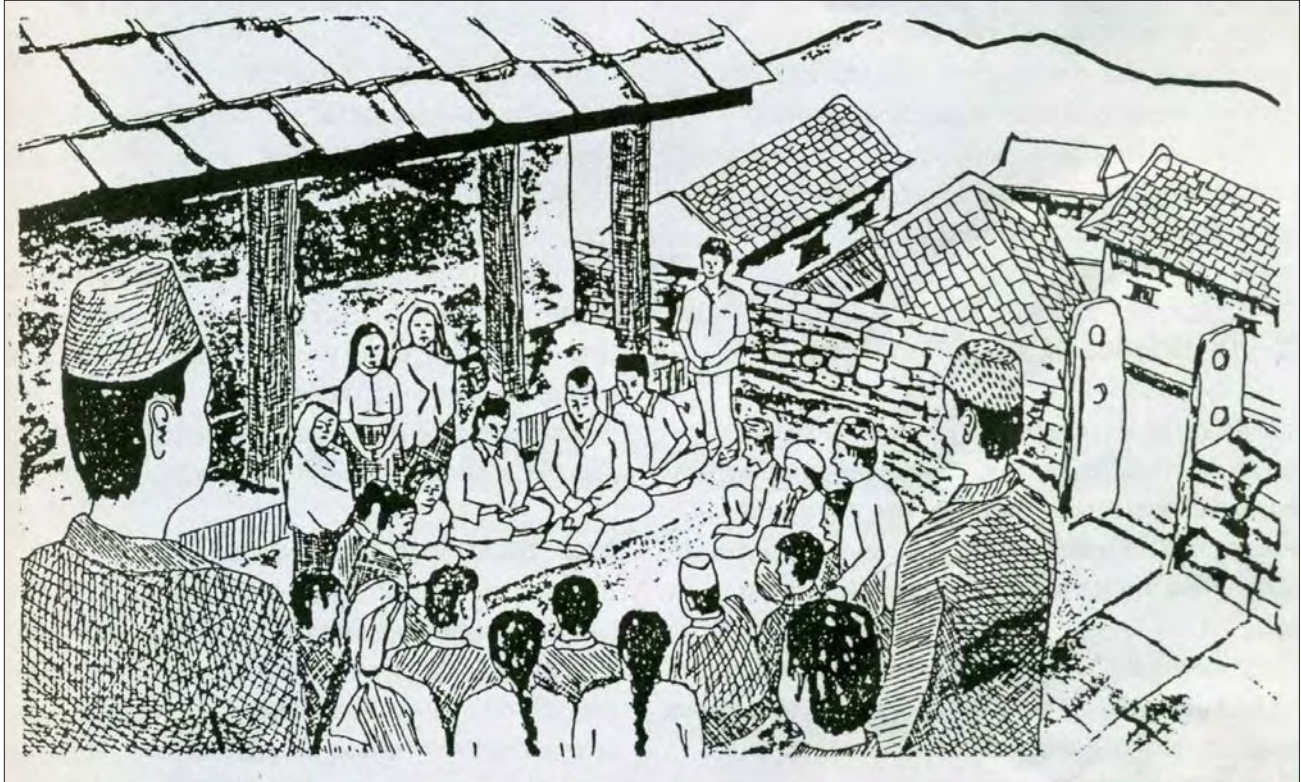
2.1 Overview

The selection of an appropriate site in most micro-hydropower schemes is an iterative process. In the Nepalese context, usually some community members who have had previous exposure to micro-hydropower technology approach funding agencies, consultants or manufacturers depending upon their financial resources and the size of the scheme. The technicians of the agency concerned undertake a site visit to assess whether the site is feasible for a micro-hydro installation. Based on the feasibility report submitted by these technicians, the community members and others involved in the process decide whether to proceed further with the development of the scheme.

Once the decision is made to proceed with the scheme, and if it is in the upper range of micro-hydro (say above 20 kW) then a detailed survey of the project area is undertaken and a detailed design report is prepared. The sanction of loans, subsidy and grants by funding agencies and banks are based on this report. There are frequent meetings between the concerned parties during this stage. For the lower end of micro-hydro (say less than 20 kW), usually the manufacturers undertake both the design and installation.

Apart from socio-economic factors such as the need for electricity, affordability, and supply and demand, technically the selection of an appropriate site depends on the following two factors:

- Stream flow
- Topography

As mentioned in Chapter 1, the power available from a micro-hydropower scheme is a function of both the flow and the head which depends on the topography. Micro-hydro becomes technically viable only if the combination of head and flow are such that the demand of the targeted community can be met. Under normal circumstances, the low season flow of the river should be used while calculating the power output. However if a micro–hydro is planned to be connected into a grid to be operated in a commercial basis, then the plant capacity should be optimised based on variations in river flow and the tariff offered. For example the optimum plant capacity for grid connected small hydropower plants in Nepal seem to correspond to 40 – 50 per cent flow exccedance given the hydrological characteristics of the streams in hills and mountains of the country and the buy back rate (tariff) currently offered by Nepal Electricity Authority at NRs 4.25/unit during 5 months of the dry season and NRs

3.00 during 7 months of the wet season along with provisions for 6 per cent price escalation during the first 5 years of operation.

It should be noted that designers have little control over the flow available in the stream. However, they have some control over the topography. They can choose different alignments for the intake, headrace and penstock. They can also modify the local topography through excavation, building of structures and by undertaking soil stability enhancement measures.

Although the planning of micro-hydro civil works does not require the detailed work of large projects, the principles are the same and care needs to be taken to follow some basic rules. Proper planning and co-ordination in the initial stage of the project will keep costs to a minimum and reduce delays.

Measurement of head and flow are beyond the scope of these guidelines but full descriptions of the methods used can be found in a number of texts including Ref1.

2.2   Principles of site investigation

Site investigation is the preliminary work carried out to establish the suitability for construction of the various options (or the most feasible option if it is apparent) through the investigation of soils, slope stability, flood levels, surface water movement and subsidence. This is discussed later in this chapter.

In most civil engineering work, the unexpected happens. Site investigation aims to predict what this might be so that the engineer can prepare a design that will deal with it.

It should be noted that the measurement of head and flow serves to establish the options available for development of the site for micro-hydro. The site investigation then assesses the suitability of the site for each alternative. The site investigation process helps to choose the optimum layout where more than one option appears to be feasible. Site conditions are also recorded during the site investigation stage so that there is adequate information for the detailed design phase.

There is usually a limit to the time and funds available for site investigation. It is always difficult to know when adequate work has been completed. The key is to work efficiently and to think carefully about where more thorough investigation is required. The principles of site investigation are:

- *Take adequate time and be thorough.* A return visit to collect information missed the first time is costly, and inadequate civil design even more so.
- *Visit all possible sites.* Gain a full appreciation of the options available.
- *Talk to local people,* especially those who have carried out construction work in the area. Since most of the rivers in

the mountains and the middle hills of Nepal have not been gauged, stream flow data are not usually available. Therefore, it is important to talk to local people to get a feel of the flood levels for rare flood events (say 20 years to 50 years return period).

- *Stay focused on the overall*, aim to raise the understanding and awareness of changes in the site over time.

2.3   Selection of alternative layouts

2.3.1   AN OVERVIEW OF THE SITE
This will involve viewing the site from a physical vantage point as shown in Figure 2.1, and taking time to consider the practical design and construction of the alternative layouts (i.e, selected potential sites). Each possible layout will require construction work on different parts of the potential site and the surveyor should therefore note on which part each component of the scheme will be located. The overview should note features that may affect the design of the scheme, such as slope stability, and land use and ownership. A sketch map of each site plan should be made as shown in Figure 2.2.

2.3.2 POSSIBLE PROJECT LAYOUT ARRANGEMENT
Different project layout arrangements are generally possible for a given site based on topography, flow availability, nature of the river and the demand form the load centres. It is important to select the optimum project layout arrangement based on site conditions, cost and expected performance of the micro–hydropower plant. Project layout arrangement also differs from project to project and not a single project layout arrangement will be similar to another. Thus the establishment of the optimum project layout needs experience as well as a number of iterations. Some typical project layout arrangements that maybe applicable for a micro hydro site in
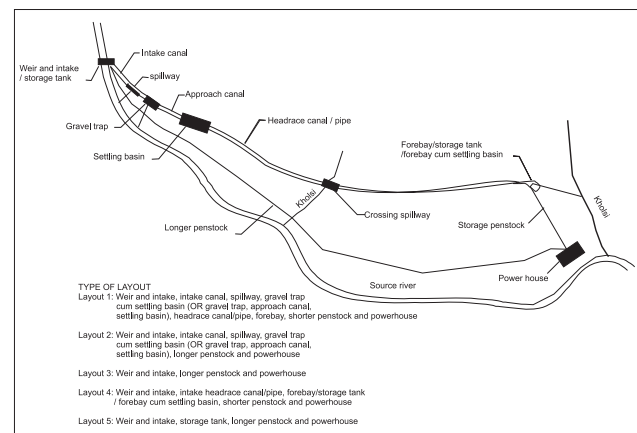

Figure 2.0: Types of project layouts

Nepal are shown in Figure 2.0.

From Figure 2.0, it can be seen that the weir, intake and powerhouse are the essential components for any micro–hydro project although their type and dimensions can differ from

Figure 2.1 Viewing the site from a vantage point gives the oppurtunity to assess the option for the layout of a scheme
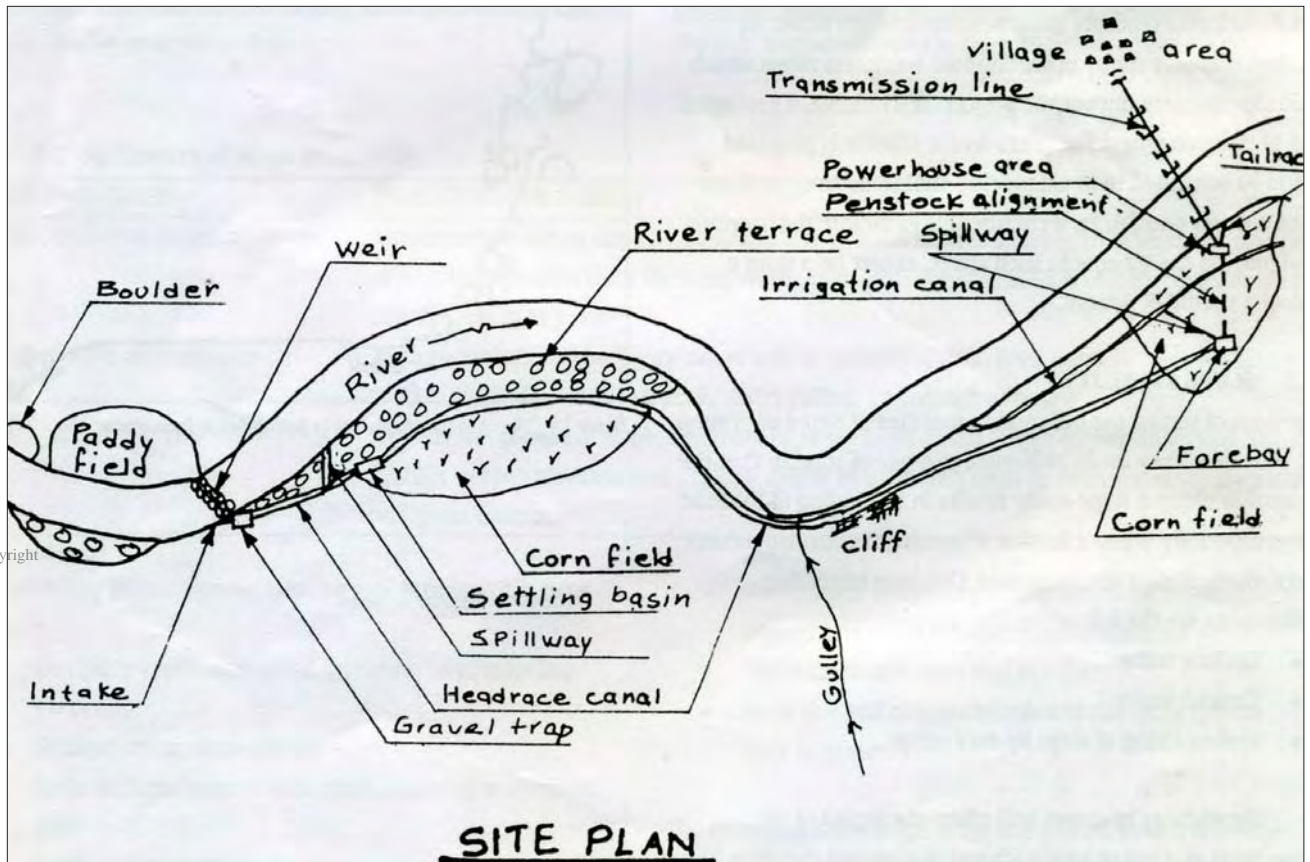

SITE PLAN

Figure 2.2 A typical site plan

site to site and project to project. Project layout arrangements shown in Figure 2.0 are combinations of weir and intake, intake canal, spillway, gravel trap, approach canal, settling basin, headrace canal/pipe, forebay, penstock and the powerhouse. The five layout arrangements shown in the figure are briefly discussed herein.

■ **Layout 1**
Layout 1 is a standard type of micro-hydro project layout arrangement with most components included. This type of project layout arrangement is used in case of higher design flows (300 l/s or above) from a relatively large river. Due to the large flow that can enter into the water conveyance system its management as well as sediment exclusion has higher importance. Thus, all standard structures of a micro–hydro project are required.

■ **Layout 2**
The difference in this layout compared to Layout 1 is that the headrace canal or pipe is replaced with a longer penstock pipe. Generally headrace canal or headrace pipe is preferred along gentle ground profile due to its low cost compared to loss in head (and therefore lower installed capacity). However, sometimes if a detailed financial analysis justifies a long penstock length with the initial alignment along gentler slopes, then the arrangement can be similar to Layout 2 in the figure above. One technical consideration required with longer penstock pipe is surge pressure due to water hammer which can be high and need to be addressed adequately.

■ **Layout 3**
If sufficient flow is available and river sediments is not a problem, such as in case of streams with very small catchment area or those originating from spring sources, then arrangement similar to Layout 3 maybe suitable. This arrangement requires only flow diversion and submergence condition at weir and intake location for the penstock. Similar to Layout 2, surge pressure may become higher in this arrangement.

■ **Layout 4**
In this arrangement, a separate settling basin is not incorporated; rather, it is combined with the forebay at the end of the headrace. This type of layout is possible in the case of small stream with low flow diversion and nominal sediment load (in the river) during flood. Furthermore, if the river flow is extremely low and provision for storage can be made at the forebay such as by constructing a larger tank, then the water can be stored during the off period (e.g., night time) for use during the peak hours (e.g., mornings and evenings). In such a case the forbay tank with storage will also take the function of a settling basin if flushing mechanism is also incorporated.

■ **Layout 5**
Though very rare, sometimes it may be feasible to construct a small storage pond on the river itself, especially if the source is a small (and clean) stream or a spring (instead of the forebay in

Layout 4). River water would be stored in this pond during off hours for use during the peak hours. This arrangement will avoid the need for longer headrace canal/pipe and the forebay, i.e., the penstock starts from the pond which also acts as a forebay.

2.4 Geotechnical considerations

2.4.1 GEOLOGY
The geology of the site is critical to the design, costs and future performance of the civil works of micro-hydro schemes. Geological maps of certain areas of Nepal are available at the Department of Mining and Geology or Tribhuvan University's geological library. It is worth checking whether such a map is available for the area of interest since this will indicate the general geological condition of the site.

Geological characteristics of a site can be grouped in the following ways:
■ Major weakness zones - Large areas of geological instability in the areas where the civil structures are to be located.
■ Slope stability - The degree of stability of the hillsides of the site.
■ Soil and rock types - Foundation conditions and liability to seepage undermining and subsidence around structures planned for the site.

2.4.2 MAJOR WEAKNESS ZONES
The main tectonic zones of the Himalayas generally correspond to the physiographic divisions of the country and run in northeast-southwest direction. Major weakness zones such as thrusts or faults separate these zones from each other. In addition there are many other "minor" weakness zones which
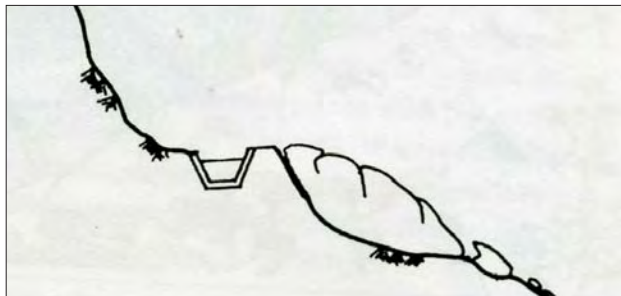

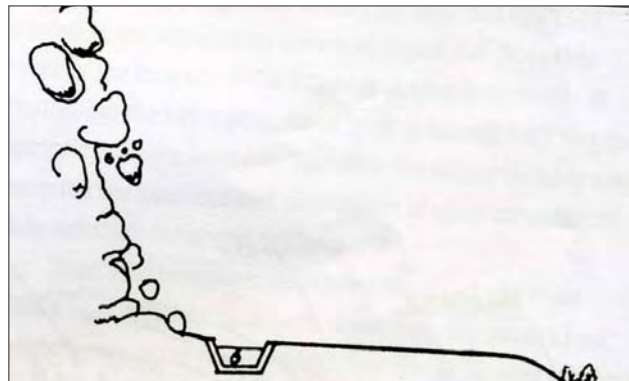*Figure 2.3 Threat to structures from below due to landslip.*

Figure 2.4 Threat to structures due to falling debris from above

Photo 2.1 Tension cracks along a slope

Photo 2.2 Obstruction caused by debris falling from on unstable slope

could significantly impact the project. If available, a geological map of the area where the micro-hydro scheme is proposed should be consulted to avoid placing civil structures on these major thrusts and faults. If circumstances dictate the inevitability of placing the scheme in such zones, expert help from a geologist should be sought.

2.4.3 SLOPE STABILITY
In geological terms, the hills and mountains of Nepal are young

and unstable. They could be likened to a pile of sand in that the excavation along a slope easily results in the sliding of the land above, especially when a further triggering mechanism occurs (particularly during the monsoon). Common triggering mechanisms are the following:
- Surface water
- Ground water
- Undercutting of slope by excavation

TABLE 2.1 **Indicators of slope instability**

SECTION OF SLOPE	INDICATOR OF INSTABILITY
Upper, middle or lower hillslopes	Tension cracks along slope (Photograph 2.1), ground shelves sharply, trees leaning downslope or bending upwards from the base, water springs or seepage at base of slope, displacement of paths, fence or posts.
Exposed faces of landslides	Fresh rock faces exposed, presence of soft, weatherable rock, open joints in rock, tension cracks, overhangs and loose rock, water springs or seepage at base of rock face.
Debris slopes	Fresh debris at base of slope, tree roots exposed, loose debris which moves underfoot, profiles steepen towards base of slope, debris littered with dead or overturned woody plants and grass clumps.

The stability of slopes will affect the design of all components of a micro-hydro scheme and should therefore be analysed thoroughly, particularly in the following key areas:

- Above and below proposed canal routes.
- Below the proposed location of a settling basin or a forebay tank.
- Along the proposed penstock alignment.
- Above and below the proposed location of the power-house.

Threats in these areas will either take the form of weakening of the support around the foundations through land slipping away or collapsing, or damage to structures through falling debris, as shown in Figures 2.3 and 2.4 and Photographs 2.1 and 2.2. Indicators of slope instability are presented in Table 2.1.

The following features of the slope or rock face indicate slope stability:

- Complete vegetation cover, including trees standing vertically
- Straight, even, slope profile
- Rock surfaces covered with moss, lichen or a weathere skin
- Hard, impermeable rock
- Rock with no or few joints
- Closed rock joints
- Well-packed debris, especially with fine material packed into voids between coarse material
- Well-established trees and shrubs
- No active gullying (although a stable gully system may be present)

The increased knowledge of the site gained from a thorough investigation of the slopes will influence the design of the whole scheme, particularly the location of principal structures. The recommendations from the investigation of slope stability should follow two basic rules:

- Never construct on fill, that is, land which has been built-up or filled using excavated material.
- Avoid the location of structures close to landslide zones.

2.4.4   SOIL AND ROCK TYPES

The surveyor should investigate what local construction materials such as soil and rock are available at site. Possible uses of such materials are presented in Table 2.2.

The type of soil or rock also affects the foundation of structures and the canal type. For example, if the soil type is sandy loam, a larger foundation depth is required. On the other hand, structures may be built directly on hard rock without any excavation. Similarly, lining may not be required for headrace canal if the soil type is clay. However, lining will be required if the alignment is through sandy soil.

Subsidence is caused by the location of acid substances in the local groundwater acting on soluble rocks such as limestone, by the presence of rocks which are liable to splitting and foliation, or by underground caverns which are prone to collapse. The presence of thick layers of loose sandy soil may also lead to subsidence.

These characteristics are identified by careful observation of the site. Limestone outcrops, sinkholes (holes of 2-10 m in diameter which form when the limestone beneath dissolves, causing the soil above to collapse), the appearance of streams or other seepage from depressions or cracks in the ground surface are examples of characteristics to look for.

Undermining refers to the action of surface water on the foundations of structures. The intake of the scheme and the penstock are particularly prone to undermining where surface water threatens the structures, but the headrace canal is also vulnerable.

2.5   Hydrology and water availability

2.5.1   PREDICTION METHODOLOGIES

Hydrology dictates the size of various micro-hydro components like the turbine, channel and the penstock. It also has great influence on the scheme being designed over or under capacity. The general practice in Nepal for micro-hydro schemes is to visit the site during dry season and measure the flow. The scheme is then designed based on

TABLE 2.2 **Possible use of soil and rock in micro-hydropower**

TYPE OF SOIL OR ROCK	POSSIBLE USE IN MICRO-HYDRO CIVIL WORKS	TREATMENT BEFORE USE
Sand	Concrete	Must be selected or thoroughly washed to be free from organic matter, fine particles, clay lumps and excessive mica.
Gravel	Aggregate in concrete	Must be carefully sorted and washed.
	Bed material in drains	Must be washed.
Rocks	Gabions and masonry	Must be selected for correct type of rock, density and size.
Clay	Canal lining	Testing of clay content required to ensure that it will serve as an impermeable layer.

this flow. This may lead to situations where the flow is less than the design flow and consequently turbines are producing less power than expected. The fact that many micro-hydro schemes in Nepal report generation way below the installed capacity is strong evidence of this. It is imperative to understand whether the flow was measured in a drier than average year or in an average year, because of the influence that this has on selecting the design flow. To be able to produce a design flow as accurately as possible, a prediction study must be undertaken.

Most potential micro-hydropower sites are located on ungauged catchments where site specific hydrologic data is lacking. To estimate yield from ungauged catchments, two techniques are currently available to predict flow. These are known as the WECS/DHM and the MIP methods, and are presented in the subsequent discussions to predict flow in ungauged catchments in Nepal.

However, at a regional training workshop on low flow measurement and analysis organised by ICIMOD in April 1999 in Kathmandu, Nepal, it was reported that both the WECS/DHM and MIP methods for estimating yield from ungauged catchments had major drawbacks, and use of these methods had to be exercised with extra caution. It was recommended that the WECS/DHM studies be reviewed, and estimation of the parameters be updated from time to time. In this regard, DHM is now collaborating with WECS to review the previous studies, and improve and update the parameters by using more stations with longer records of data. It was stressed that with Nepal facing big problems in estimating the design of low flow for a variety of applications including micro-hydropower, a reliable method was urgently required. In this context, the Institute of Hydrology, U.K. is undertaking a project titled "Regional Flow Regimes Estimation for small-scale Hydropower Assessment (REFRESHA)" in collaboration with ICIMOD and DHM from 1999, which aims to provide a reliable method for estimating the hydrological regime at ungauged sites in the Himalayan region of the country. REFRESHA is scheduled to be ready in about two years time.

WECS/DHM method
Department of Hydrology and Meteorology (DHM) method

The Water and Energy Commission Secretariat (WECS) and (DHM) (Ref. 4) method is based on a series of regression equations that are derived from analysis of all the hydrological records from Nepal. The findings of this regression analysis have been used to produce equations for predicting different hydrologic parameters such as the low flows, flood flows and flow duration curves. It is beyond the scope of this book to explain in detail the WECS/DHM method. Readers are advised to consult the reference, details of which are provided in Chapter 11. Appendix A describes this method with an example.

Medium Irrigation Project method (MIP)

The MIP method presents a technique for estimating the distribution of monthly flows throughout a year for ungauged locations. The MIP methodology uses a database consisting of DHM spot measurements. The occasional wading gaugings conducted by DHM include only low flow and these flows do not represent the natural conditions since they are the residual flows remaining after abstraction for different purposes like irrigation. MIP presents non-dimensional hydrographs of mean monthly flows for seven different physiographic regions. These hydrographs present monthly flows as a ratio of the flow in April (assumed lowest annual flow). For application to ungauged sites, it is necessary to obtain a low flow discharge estimate by gauging at a particular site. The measured flow is then used with the regional non-dimensional hydrograph to synthesise an annual hydrograph for the site. Appendix A describes this method with an example.

Comparisons of the WECS/DHM and the MIP approaches

WECS/DHM : Delineation of drainage basins and elevation contours are often distorted on the available maps; also regressions were derived on the basis of observed flows for catchments ranging in size from 4 up to $54,100$ km^2. Therefore, for flows in smaller catchments the results would prove to be unreliable.

MIP : The MIP method approach based on wading measurements taken on an intermittent basis cannot be expected to give a good estimation of total flow in the monsoon months. It can, however, give a reasonable approximation of the divertable flows in these months. In the wet season, MIP would be expected to underestimate WECS figures which should more accurately represent total flow. In the dry season MIP and WECS should both provide total flow estimates. The MIP procedure, which explicitly advocates the use of local data to adjust the regional hydrograph, should give reasonably accurate estimates through the dry season months that are critical in assessing micro-hydro projects.

Note that neither the WECS/DHM nor the MIP methods were derived from data for high altitude snow-fed catchments. For such catchments, more weight should be given to the results of site measurements.

It must be emphasised that one can get a feel for annual floods by measuring flood levels at site. Silt and debris deposited along the river banks or level just below the vegetation growth are indications of flood depths. By measuring flood depth, width and average gradient of the river at the intake area, it is possible to calculate the flood flow using Manning's equation, described in 4.3.2. It is beyond the scope of this book to describe the different methods of river gauging: please see Ref. 1 or Ref. 2 for guidance.

2.5.2 PROCEDURE TO ESTABLISH THE DESIGN FLOW

1. Conduct a flow measurement at site during the dry season (November-May). Preferably in February for snow-fed rivers and March/April for other rivers. Not if the year is drier than average, or wetter than average. This can be established by talking to the locals. Consider if there are significant abstractions by other water users, such as irrigation and drinking water schemes upstream of the point where the gauging was conducted.

2. Calculate:
   - Average monthly flows by using WECS/MIP methods.
   - Flow duration curve using WECS.
   - Instantaneous flood flows of different return periods using WECS.

3. Compare the dry season mean monthly flows obtained by WECS and MIP method. If the flow measured at site is above average according to the local people, compare the dry season mean monthly flow obtained by the WECS method with that obtained by the MIP method, and use the lower value. If the flow was measured at site during either a wetter than average year or a drier than average year, then use the value obtained by the WECS method.

4. Use the flow duration curve (FDC) to establish the probability of exceedance of the value from step **3**. The FDC is useful because the power equivalent of the flow can be superimposed onto it, so that it is possible to read off the amount of time each year that certain power levels can be obtained. This is a useful planning tool, allowing a choice of size of turbine to be made, together with an indication of required variable flow performance of turbine and an indication of the plant factor constraints which will result from any particular choice of turbine size. See Ref.1 for details.

5. Decide on what percentage of the flow established in step **3** can be diverted for power generation. If using a temporary weir assume that 50 percentage of the flow can be diverted. If the river presents formidable difficulties, assume less than 50 per centage. If using a permanent weir founded on bedrock assume 95 per centage and for weir based on alluvium foundation, allow for seepage losses and assume that 90 percentage of the flow can be diverted.

6. Calculate seepage losses for the water conveyance structures. These losses must be deducted from the flow established in step 5. Seepage calculation is covered in Section 2.5.3.

7. Consider if there are other water users such as irrigation and drinking water downstream of the diversion works. Establish the amount of flow that has to be released downstream and deduct this amount from the flow from step 6. *This is the design flow.*

A design example is included in Appendix A.

TABLE 2.3 **Canal seepage losses**

TYPE OF SOIL	SEEPAGE LOSS, (l/S/1000 m² OF WETTED AREA)
Rock	<0.5
Impervious clay loam	0.8 -1.2
Medium clay loam	1.2-1.7
Clay loam or silty soil	1.7-2.7
Gravelly clay loam or sandy clay or gravel cemented with clay	2.7 - 3.5
Sandy loam	3.5 - 5.2
Sandy soil	5.2 - 6.4
Sandy soil with gravel	6.4 - 8.6
Pervious gravelly soil	8.6 -10.4
Gravel with some earth	10.4 - 20.8

Example 2.1 Seepage Calculation

A 500 metre long unlined headrace canal is to be constructed in sandy clay to convey a design flow of 0.1m³/s. A standard trapezoidal section is proposed with a depth of 0.2 m, a bottom width of 0.6 m and side slopes of 1:2 (V:H). Calculate the seepage loss in the canal.

Solution:

The wetted perimeter (P) of the canal can be calculated using the following equation:

$$P = B + 2 \times H \times \sqrt{(1+N^2)} \text{ (see chapter 4)}$$
$$= 0.6 + 2 \times 0.2 \times \sqrt{(1 + 2^2)} = 1.49m$$

The wetted area
$$= P \times L$$
$$= 1.49 \times 500$$
$$= 747 \text{ m}^2$$

From Table 2.1, seepage loss in sandy clay is 3.5 l/s/1000 m² of wetted area. The seepage loss is given by:

$$Q_{seep} = 3.5 \times \text{wetted area}/1000$$
$$= 3.5 \times 747 /1000$$
$$= 2.6 \text{ l/s}$$

Which is 2.6% of the designed flow of the canal.

2.5.3 SEEPAGE

It is imperative to examine the soil along the route of the proposed canal and estimate the amount of seepage that a canal may suffer, an important issue that is often overlooked by micro-hydro designers. This is especially true for micro-hydropower schemes with long unlined canals. Table 2.3 gives canal seepage losses for different soil types. By calculating

the wetted area for a given cross-section of the canal, seepage can be calculated using data from the table. Example 2.1 illustrates this method.

2.6   Other considerations

2.6.1 FLOOD RISK

In site investigation, the concern is for the selection of the best option for the design of the scheme. Therefore knowledge of flood levels is important at the two extremes of the micro-hydro scheme, the intake and the powerhouse, or at other parts of the scheme that may be vulnerable to flood damage from the river.

Flood levels may be predicted by hydrological calculation from available data to give the 20 year or 50 year flood level, or by consulting local people. Ideally, both methods should be used to give a reliable estimate. Always allow a margin of error so that a rare flood event is allowed for, and think carefully about how the floods will affect each of the proposed layouts for the project. The location of the power-house higher on a slope will reduce the available head and therefore have an important impact on the capacity and the economics of the project.

2.6.2   CROSS DRAINAGE

Sometimes because of the nature of the topography, the headrace canal and/or the penstock alignments will need to cross gullies and small streams. Note that dry slopes are more stable than saturated slopes. Surface water can be diverted by constructing various types of cross drainage works. For example, catch drains can be constructed uphill from the micro-hydro alignment to divert the surface runoff. Catch drains are small channels that divert surface runoff (thus catch it) and divert it into nearby gullies or natural drainage.

Another example of cross drainage works is the use of a superpassage. This is a covered headrace canal arrangement such that the surface runoff flows over it whereas the design flow is safely conveyed in the canal.

2.6.3 WATER RIGHTS

Sometimes there can be regarding the water usage conflict between the proposed micro-hydro scheme and other prior uses of the source stream. For example, if there is an irrigation scheme downstream of the proposed micro-hydro intake that may receive less water (once the micro-hydro plant is commissioned), there will be conflicts. Such water rights issues should be resolved before implementing the micro-hydro scheme.

It should be noted that irrigation and micro-hydro can be co-ordinated if an agreement with all concerned parties is reached in the initial stage. This is because irrigation water is not required throughout the year and therefore water can

be used for power production at other times. This may result in less or even no power available during peak irrigation period. However, if the electricity users are also owners of the irrigated land, they can prioritise their needs, such as by irrigating in the afternoons and nights and producing power during mornings and evenings.

2.6.4 LAND OWNERSHIP AND LAND USE

The surveyor should note down the issues concerning land use and ownership. If the alignment traverses through a farmer's paddy field, the land may have to be bought by the project.

Another example is that an open channel headrace may be technically feasible but the designer may have to choose a buried pipe if the headrace alignment is along cultivated land. Similarly, sediment flushing and spillway flows need to be safely diverted away from cultivated land. It is important to note down land owners whose land will be used for structures, so that agreements such as lease arrangements can be negotiated. These factors will affect the design of the scheme.

2.6.5 HIGH ALTITUDE SITES

These guidelines are generally applicable to micro-hydropower in Nepal, but some particular measures need to be taken for high altitude sites to avoid ill-efffects from freezing temperatures. To avoid frost damage to concrete and masonry, the following measures are necessary:

- Keep the water to cement ratio as low as possible, preferably not more than 0.50.
- Avoid aggregate with a large maximum size, or a large proportion of flat particles.
- Use a water reducing air entraining agent (plasticiser).
- Ensure good compaction.
- Do not build while night temperatures are below freezing. Surfaces must be prevented from drying out for at least three weeks if the ambient temperature is on average 5°C or less.

To avoid ice damage to canals and structures, the water face of walls should be smooth concrete or plastered masonry, and inclined at approximately 1:1. The expanding ice can then rise between the walls, instead of pushing the walls apart.

Headrace canals should be designed for a minimum velocity of 0.6 m/s. Even though the surface may freeze, water will flow under the ice.

The top of trashracks should be below any expected ice level, to avoid ice forming around the trashrack bars. Timber trashrack bars may be less liable to icing than steel bars.

The foundation level of structures should be below the depth of ground freezing. This is likely to be about one metre depth.

2.7 Planning

The planning of civil engineering works for large projects is a complex process and the skills required are considered to be a separate discipline within the field of civil engineering. The reason that planning is given so much importance is that the project construction cost can be significantly brought down by efficient co-ordination of labour, equipment and materials. This ensures that the resources are used at their maximum productivity.

As mentioned earlier, the planning of micro-hydro civil works does not require the detailed work of large projects. However, the principles are the same and care needs to be taken to follow some basic rules. The process of constructing micro-hydro civil works has three parts:

- *Understanding* what has to be built
- *Establishing* the method, equipment and the people required
- *Carrying out the* work safely, economically and to the quality required to satisfy the client.

The *understanding* part of the process sounds straight forward, but it should not be overlooked. Given the likely number of people involved, effective communication and clear demarcation of responsibilities are essential in planning. Everyone needs to know what they are accountable for and to whom.

There are a number of factors affecting how, when and in what order the works can be carried out. A checklist of these factors is as follows:

- Performance of staff, equipment and materials.
- Availability of staff, equipment and materials.
- Holidays and festivals.
- Access to the site.
- Weather, seasons.
- Availability of funds.

- Site geology and topography.
- Existing use of the site and its boundaries.
- Public relations.

Preparing a "Project Implementation Schedule" in the initial stage is always helpful since it will indicate what activities are in the critical path and allow for planning ahead. This chart will be very useful in optimising the resources (labour, equipment and materials) by distributing them in a balanced way.

Furthermore, this will also be a key tool to monitor progress by tracking the planned versus actual milestones reached both for a particular activity and for the overall project implementation. Undoubtedly, such a chart will undergo frequent revision during the construction phase. However, it is still helpful to formulate a chart and make changes as necessary, since it can be used to monitor the progress of work and plan for future activities such as procurement of construction materials and labour arrangements. A typical project implementation chart is shown in Figure 2.5.

Apart from the implementation chart shown in Figure 2.5, a more elaborate schedule is also prepared, especially for larger projects (e.g., mini and small hydro) and could be useful for higher installed capacity micro hydro projects (~100 kW). Generally two types of schedule, one master schedule, and the other working schedule (prepared about two weeks in advance) are prepared. The master schedule contains major activities of a project, controls the overall project period and is also similar to chart shown in Figure 2.5. This is prepared before start of the project construction (at planning stage) and is continuously updated as construction activities commence. On the other hand, the working schedule is prepared during the construction period and incorporates day to day activities. A separate working schedule is prepared for each of the major activities of the project. A typical master implementation schedule is shown in Figure 2.6. Similarly, part of a working schedule is shown in Figure 2.7.

ACTIVITY	DURATION IN MONTHS									
	1	2	3	4	5	6	7	8	9	10
Public relations/ awareness raising						D T				
Site survey including low flow measurement and data processing						A I				
Detailed design						S H				
Financing						H A				
Tendering and award of construction contract @ 6 weeks						A R				
Fabrication and supply of steel parts (penstock, trashracks, flush pipes etc.)				M		I F				
Construction of headrace canal				o		N E				
Construction of settling basin and forebay				N		S				
Construction of masonry wall and gully crossing				S		F T				
Installation of penstock and construction of anchor blocks and support piers				o		E I				
Fabrication and supply of turbine, belt drive and other electro-mechanical equipment				o		S V				
Installation of transmission line				N		T A				
Powerhouse construction						I L				
Electro-mechanical installation at powerhouse						V				
Testing and commissioning						A				
Project handover to the client						L				

Figure 2.5 A Typical Project Implementation Chart

ABC Micro Hydro Project
MASTER IMPLEMENTATION SCHEDULE

ID	Task Name	Duration
1	TOTAL PROJECT	224 days
2	Pre-construction phase	133 days
3	Public reaction/awareness raising	7 days
4	Site survey including low flow measurement	14 days
5	Detailed design	28 days
6	Financial arrangement	42 days
7	Tendering and contract award	49 days
8	Construction phase	91 days
9	Fabrication and supply of steel parts	49 days
10	Penstock installation and supports construction	28 days
11	Headrace canal	49 days
12	Masonry wall and gulley crossing	28 days
13	Fabrication and supply of electro-mechanical equipments	49 days
14	Transmission line construction	21 days
15	Setting basin and forebay	28 days
16	Powerhouse construction	21 days
17	Electro-mechanical equipment installation	7 days
18	Testing and commissioning	7 days
19	Project handover	7 days

Legend:
- Task
- Critical Task
- Progress
- Milestone
- Summary
- Rolled Up Task
- Rolled Up Critical Task
- Rolled Up Milestone
- Rolled Up Progress
- Split
- External Tasks
- Project Summary
- Group By Summary
- Deadline

Project: master schedule
Date: Mar 17 '08

Page 1

Figure 2.6 A Typical Project Master Implementation

ABC Micro Hydro Project
WORKING SCHEDULE
(Powerhouse)

ID		Task Name	Duration	Start	Finish
1		POWERHOUSE	43 days	Mar 17 '08	Apr 28 '08
2		Tail pit	12 days	Mar 17 '08	Mar 28 '08
3		Base slab both unit	7 days	Mar 17 '08	Mar 23 '08
4		Rebar	3 days	Mar 17 '08	Mar 19 '08
5		Formwork	4 days	Mar 19 '08	Mar 22 '08
6		Concreting	1 day	Mar 23 '08	Mar 23 '08
7		Support wall both unit	8 days	Mar 24 '08	Mar 28 '08
8		Rebar	2 days	Mar 24 '08	Mar 25 '08
9		Formwork	2 days	Mar 26 '08	Mar 27 '08
10		Concreting	1 day	Mar 28 '08	Mar 28 '08
11		Plum vold filling	9 days	Mar 29 '08	Apr 6 '08
12		I pour formwork	1 day	Mar 29 '08	Mar 29 '08
13		I pour concreting	2 days	Mar 30 '08	Mar 31 '08
14		II pour formwork	3 days	Apr 2 '08	Apr 4 '08
15		II pour concreting	2 days	Apr 5 '08	Apr 6 '08
16		Valve base slab both units	8 days	Apr 7 '08	Apr 11 '08
17		Rebar	3 days	Apr 7 '08	Apr 9 '08
18		Formwork	1 day	Apr 10 '08	Apr 10 '08
19		Concreting	1 day	Apr 11 '08	Apr 11 '08
20		Anchor walls both units	9 days	Apr 12 '08	Apr 20 '08
21		Rebar	2 days	Apr 12 '08	Apr 13 '08
22		Formwork	4 days	Apr 14 '08	Apr 17 '08
23		Anchor bolt	2 days	Apr 18 '08	Apr 19 '08
24		Concreting	1 day	Apr 20 '08	Apr 20 '08
25		Valve side walls both units	4 days	Apr 21 '08	Apr 24 '08
26		Rebar	1 day	Apr 21 '08	Apr 21 '08
27		Formwork	2 days	Apr 22 '08	Apr 23 '08
28		Concreting	1 day	Apr 24 '08	Apr 24 '08
29		I layer earth filling	3 days	Apr 26 '08	Apr 28 '08

Figure 2.7 Typical Working Schedule

2.8 Checklist for site selection

LAND OWNERSHIP, LAND USE AND WATER RIGHTS
Have all issues concerning land use and ownership been duly recorded during the site visit? Does the alignment traverse through a farmer's paddy field or is it along barren land? Have water rights issues such as irrigation use been adequately addressed?

INTAKE
Make sure water can be diverted away from the river and towards the headrace. Does the river course appear stable or does it look like it will meander? Think about floods and flood levels. Does the river carry large boulders? If so, think about temporary diversion works rather than a permanent weir.

GRAVEL TRAP
Does the river carry a significant amount of gravel during the monsoon? If so provide a gravel trap as close to the headworks as possible. Can the gravel be easily flushed into the stream or a nearby gully from the gravel trap?

SETTLING BASIN
This structure should be located as close to the intake as possible. The earlier the sediment is removed the less the maintenance in the headrace. If the source river is not far away the sediment can be discharged back into it.

HEADRACE
In general the headrace alignment should be on level to slightly sloping ground. If the alignment is steep, consider using a headrace pipe instead of a canal. Try to get the alignment away from the river as early as possible to minimise flood damage. Provide escapes and spillways upstream of areas where the canal might be blocked by landslides. If seepage from the headrace canal can trigger landslides, think about lining the canal or using pipes.

Photo: 2.3 A straight penstock run keeps cost at a minimun

FOREBAY
Allowance should be made for final settling of sediments. Generally, this structure should be located just uphill of the transition area where the ground profile changes from gentler to steeper slope. Is there a possibility to safely discharge the entire flow from a spillway in case of system malfunction?

PENSTOCK
The penstock alignment should start where the ground profile gets steeper. An ideal ground slope would be between 1:1 and 1:2 (V:H). The flatter the ground slope, the less economic is the penstock. It is difficult to manually lay penstock, construct support piers and anchor blocks if the slope is greater than 1:1. Also try to minimise bends since these will require additional anchor blocks.

POWERHOUSE
Make sure that there is enough space for a powerhouse with the required dimensions (to fit the electro-mechanical equipment) at the location selected. Excavation can be minimised by locating the powerhouse on level ground. Think about where the tailwater can be discharged (i.e. tailrace alignment). Is the powerhouse high enough above the river to be safe from floods?

TAILRACE
Make sure that the tailrace is protected from the stream into which water emerging from the turbine is discharged. The tailrace should be oriented downstream to prevent floodwater, debris, and bed load from being funnelled into it toward the powerhouse.

TRANSMISSION LINES
Is the village situated away from the powerhouse site? If so transmission lines are required. The cost of transmission lines adds significantly to the overall cost of a scheme. Consult Ref.1 for details.

AVAILABILITY OF CONSTRUCTION MATERIALS AND LABOUR
Construction materials for micro-hydro schemes that may be found at site are sand, aggregate and stones. Are these materials easily available at site or brought from outside? Are skilled labourers such as masons and carpenters and unskilled labourers available at site? The unit rates for such construction materials and labour should be obtained while at site for estimating quantities and cost of the scheme during the design phase. It is more relevant to use prevailing rates rather than district rates, which are normally lower than the prevailing rates.

STABILITY
Apart from the above criteria, it is very important for the entire scheme to be on stable ground. If a small length of the alignment is unstable it may be possible to stabilise it. Refer to Section 2.4 and Chapter 9 to assess this issue.

3. Diversion works

3.1 Overview

The diversion works for a micro-hydropower scheme control the flow of water from the source river into the headrace. They comprise a diversion weir (usually), an intake, and sometimes river training works. The diversion works are part of the headworks, and serve the following functions:

- Maintain the design flow with nominal head losses during both monsoon and dry seasons.
- Revent, or at least minimise, the bed load and other floating materials (ice, timber, leaves etc.) entering the canal.
- Safely contain peak flows in the river and away from the micro-hydro system so that damage is minimum to the structures.

The principal maintenance task associated with civil works is often the removal of sediment and debris carried by the incoming water, which can cause damage to the turbines should it be permitted to enter the penstock. It is therefore essential that the adopted intake design prevent sand, bed load and debris from entering the intake as much as possible. One of the principal causes of problems in the operation of hydropower schemes is a poorly designed intake which may permit operation of the scheme in the short-term but beyond that, cause serious damage to the system. The design of an appropriate intake structure for micro-hydro in Nepal requires an adequate understanding of Himalayan rivers since they have some unique features.

3.2 General principles for selecting intake location

The major considerations required to select appropriate intake locations are discussed in this section. It is important for the design engineer to realise that much can be learned from observing the irrigation intake sites selected by local farmers. The farmers are familiar with the rivers and have the added advantage that they have observed them over a long period of time. In fact, some of the farmer managed irrigation schemes in Nepal are more than 100 years old and the intakes of such sites have faced most problems brought about by Himalayan rivers.

The following principles should be considered while selecting appropriate intake locations:

Minimal disturbance to the natural state of the river
Construction of high and permanent weirs (larger than 1-2 m) across the total width of the river is generally undesirable, because damming has the effect of rapid sediment deposition and change of present river course, leaving the intake dry and useless. The design and construction of weirs requires careful consideration to avoid presenting an obstacle to flood

flow in the rainy season. For diversion from a natural pool, no weir is required and water can be conveyed through HDPE pipes or a covered masonry flume to a headrace canal.

For this reason, attempts should be made to locate intakes such that the natural water level at low flow in the stream is suitable for the intake level of the canal. This will allow the canal intake structure to be built at stream level and the only measures necessary within the stream or river bed itself are measures for the stabilisation of the present state of the stream.

Location in an area which offers natural protection
When withdrawing water from a stream whose level may increase markedly during rainy periods, it is desirable to locate the intake behind or under large, permanently placed boulders or rock, these limit the water that can enter the intake, and deflect flood flows and river-borne debris away. Advantage can also be taken of stable banks and rock outcrops.

Location on the outside of a bend
There is a natural tendency of the river to deposit sediment on the inside of bends along the river. At bends, the direction of the flow closest to the river bed changes compared with the surface flow. A spiral flow forms, which transports the bed load to the inner side of the river bend. On all streams and rivers it can be observed that gravel and sand banks form at the inside bend, i.e., the bed load is diverted from the deflecting bank. As a result of this when the river flow decreases, the river width decreases from the inside of the bend. Therefore an intake should not be sited on the inside of a bend. To minimise sediment load and to ensure flow availability during the dry season, an intake should be sited on the outside of a bend. The best location is about 2/3 to 3/4 of the distance around the bend on the outside as shown in Figure 3.1. Sharper bends are more effective in preventing the entry of sediment, and the amount of bed load transported into the canal decreases as the diverted proportion of the total flow in the river decreases.
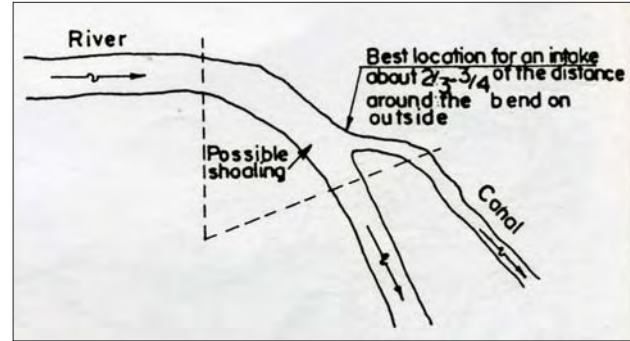
Figure 3.1 Locating intake around a bend

Other considerations
In straight sections of a river, the water flows parallel to the banks and the bed load is transported along the bottom. Therefore, in straight sections the location of the intake is governed by factors such as bank stability and headrace alignment.

The location of an intake structure must be so chosen that the largest possible portion of the bed load remains in the river and is not diverted into the headrace. However, even a good intake will not exclude all sediment; the gravel trap and settling basin further along the canal complete this.

3.3 Intake location in relation to river characteristics

3.3.1 CHARACTERISTICS OF HIMALAYAN RIVERS
In Nepal, most micro-hydropower schemes are located in the foothills of the Himalayan Range. This includes the High Mountains, Middle Mountains and the Siwaliks as shown in Figure 3.2. It is essential to have a clear understanding of the characteristics of these Himalayan rivers before approaching the design and construction aspects of diversion works. These rivers flow in geologically young mountain structures and can be characterised as follows:
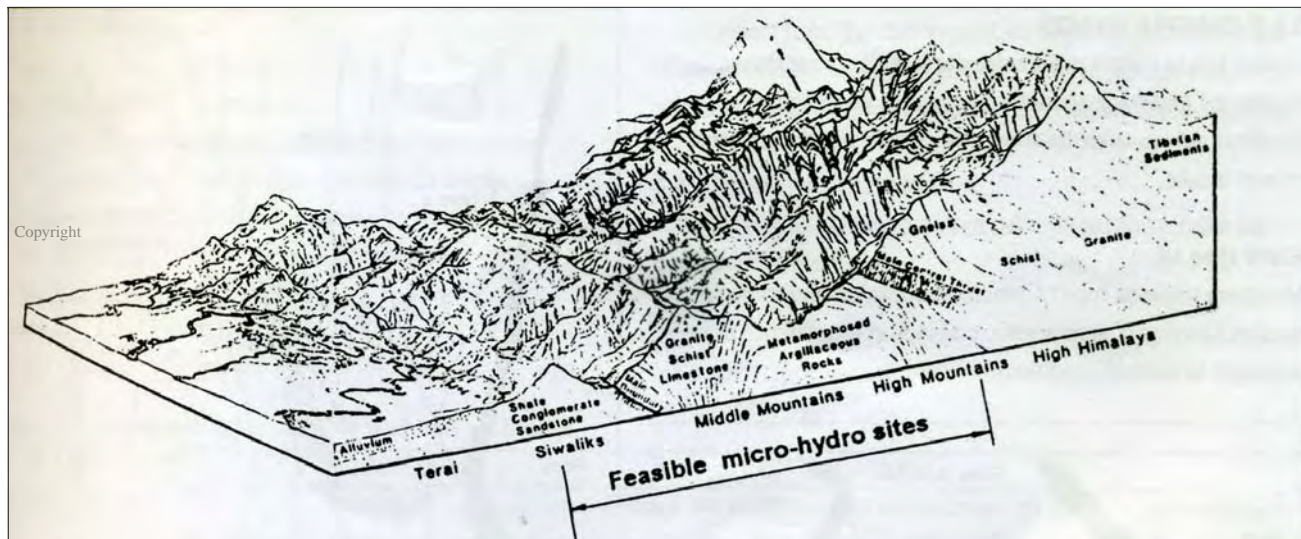
Figure 3.2 Feasible locations of micro-hydro schemes in Nepal

- Steep river gradient and steep slopes along both river banks.
- High degree of continuing erosion and sediment transport.
- Smaller streams of steep and unstable nature with a bouldery alluvial bed.
- Liable to transport considerable quantities of sediment including boulders during the monsoon.
- A significant flow and sediment increase in the rivers during the monsoon.

Due to these unique characteristics, development of hydro-power from the Himalayan rivers present great challenges. Design and construction of appropriate structures to cope with movement of large boulders and high sediment loads are two of these challenges. River intakes used elsewhere in relatively flat and stable rivers including the Terai are inappropriate in the case of mountain rivers of Nepal. Rivers in Nepal can be categorised according to the way in which they are influenced by various characteristics. The types of river that are mostly utilised in micro-hydro are shown in Table 3.1.

3.3.2 EXAMPLE INTAKES
Typical intake locations for some of these rivers are shown in Figures 3.3 and 3.4. It should be noted that these figures illustrate only possible locations for intakes, not the preferred type of intake.

River type IA
Mountain rivers of Type 1A provide favourable conditions for intakes in terms of permanence and lack of interference from sediment in normal conditions.

River type IB
Intakes on Type IB rivers can also be located similar to 1A. However, these rivers provide a greater choice of intake site, and permit more permanent intake structures, either from the side of the channel or as an angled or frontal intake built into the channel. It is often possible to protect the intake behind a rock outcrop.

TABLE 3.1 **Categories of Nepalese rivers**

TYPE	GRADIENT (MAIN LOCATION)	VALLEY SHAPE	BED MATERIAL	CHANNEL PATTERN	SEDIMENT MOVEMENT (N = NORMAL, F = FLOOD)
1A	Very steep (Mountains)	Narrow valley no flood plains	Rocks, very large boulders	Single	N - Sand in suspension gravel F -gravel, cobbles and boulders
1B	Step (Mountains and hill regions)	Narrow valley, irregular narrow flood plains	Rocks and boulders, gravel and cobbles in shoals	One main plus flood bypass channel	N - Sand and gravel F- Sand and gravel F - Includes cobbles and small boulders.
1C	Step (Mountains and hill regions)	Narrow valley, irregular narrow flood plains	Rocks and boulders, gravel and cobbles in shoals	Several active channels as well as floodways.	N - Sand and gravel F - Includes cobbles and small boulders.
2A	Intermediate (Hill regions)	Outwash river, confined by valley sides	Some boulders, mainly gravel and cobbles.	Single plus limited floodways	N - Sand and fine to medium gravel F - Includes coarse gravel, cobbles, perhaps small boulders.
2B	Intermediate (Hill regions)	Ditto but less confined valley	Some boulders, mainly gravel and cobbles	2-4 active channels with floodways	N - Sand and fine to medium gravel F - Includes coarse gravel, cobbles, perhaps small boulders.
2C	Intermediate (Hill regions)	Ditto but wider valley	Some boulders, mainly gravel and cobbles	Braided, several active floodways perhaps small boulders.	N - Sand and fine to medium gravel F - Includes coarse gravel, cobbles, perhaps small boulders.
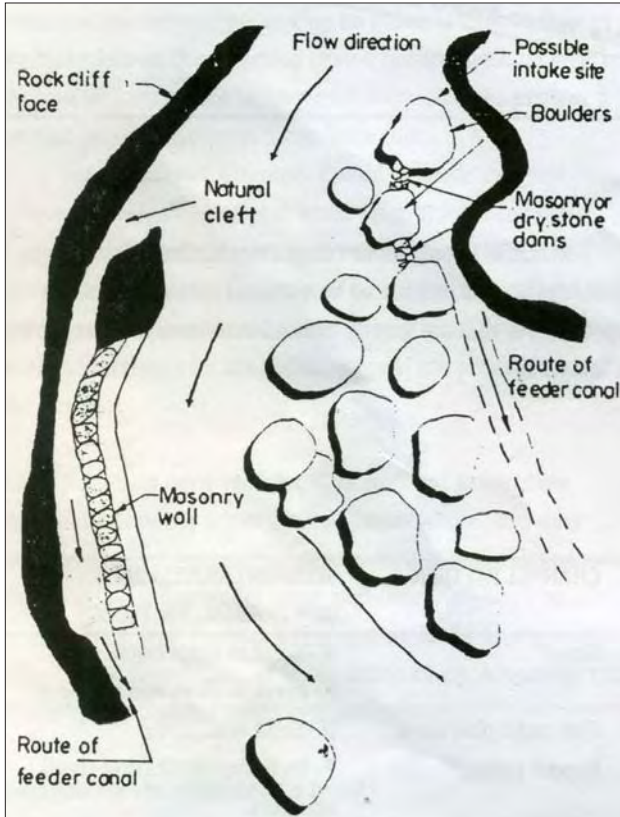
Figure 3.3 Siting of intakes in mountain rivers of Type 1A

Figure 3.4 Siting of intake in mountain or steep hill rivers of Type 1c. (Source: Ref. 5)

River type 1C

Intake selection in these rivers differs from 1A and 1B only in requiring control of one or more of the channels in order to ensure that sufficient flow reaches the intake. A possible arrangement is shown in Figure 3.4.

Gravel bed rivers

These are category 2 rivers (2A, 2B & 2C) which have less steep channels compared to category 1 types. The riverbeds are mainly of cobbles and gravel, together with some boulders. Intake sitting follows the same general principles as in category 1. However,

Photo 3.1 Area of intake in type 2B river (Dhading)

these rivers provide more flexibility. For example, it is possible to use more permanent river control structures such as concrete or masonry weirs and river training walls.

3.4 Intake types

3.4.1 DESCRIPTION
Types of intake structure are chiefly distinguished by the method used to divert water from the river. In micro-hydropower, mainly two types of intake are used which are as follows:

- Side intake
- Bottom intake

Apart from the above types, an innovative intake called "Coanda intake" has also been field tested in a micro-hydro scheme in the UK. This is discussed in Chapter 10 (Innovations).

3.4.2 SELECTION CRITERIA
Table 3.2 aids the choice between side and bottom intakes for given conditions.

TABLE 3.2 **Selection criteria for side and bottom intake**

SELECTION CRITERIA	SIDE INTAKE	BOTTOM INTAKE
Amount of water:	*Favourable site selection necessary (outside of a bend, or an artificial bend by groins) if the amount of diverted water is greater than 50% of the water supplied.*	*The bottom screen draws off the river water up to the capacity limit of the screen (i.e. all river flow if screen is large enough).*
Gradient of river:		
• very high (i > 10%) to high (10% > i > l%)	*Favourable: maintenance free operation of the intake structure should be ensured as far as possible.*	*Favourable for very high gradient; can be maintenance-free, if properly designed.*
• mean gradient (1% > i > 0.01%)	*Favourable*	*Unfavourable if i < 10%. Unfavourable: fine bed load into the initial headrace canal results in difficulty in flushing.*
Plan of river:		
• straight	*Possible*	*Very favourable, as bottom screen is uniformly loaded.*
• winding	*Very favourable if the river channel is stable; when arranged on the outside bend.*	*Unfavourable, as bottom screen is not uniformly loaded.*
• branched	*Unfavourable; damming of the river is required.*	*Unfavourable.*
Suspended sediment concentration:		
• high	Suitable in combination with very efficient settling basin.	Less suitable, Well suited
• low	*Well suited*	
Bed load transport:		
• strong	*Suitable as long as sufficient amount of water remains in the river for flushing.*	*Less suitable Well suited*
• weak	*Well suited*	

3.5 Side intake

3.5.1 DESCRIPTION

Side intakes are simple and less expensive than other types of intake. They are easy to build, operate and maintain. Side intakes are similar to farmers' traditional intakes for irrigation, and hence the farmers can quickly learn the principles of operation and maintenance of these intakes. The side intake of the 50 kW Galkot micro-hydro scheme can be seen in Photograph 3.2. Note that to minimise flood flows in the canal, the intake is designed as an extension of the headrace canal. It was felt that the intake could be vulnerable to flood damage and therefore the coarse trashrack is located further downstream.

Photo 3.2 Side intake of the Galkot micro-hydro scheme, Baglung, Nepal (see Appendix C for drawing)

Side intakes must be safe against boulder impact and floodwater entry. They are most effective when built on the outer bend of the river (to minimise the amount of sediment drawn into the intake), near natural pools or sited in such a way that they are protected as much as possible from river floods (e.g. behind a permanent rock outcrop). Side intakes can be used with or without a weir.

Side intake without weir (natural pond)

In some cases, an exposed intake structure may be avoided by extracting water from behind the shelter of a rock outcrop. In other cases, there may be a natural cleft in a rock spur, or an opening between a very large boulder and a rock wall, that can be used as a natural intake. Wherever possible, this type of intake should be preferred, since it is the most economical one, and safe from the damage of floods and debris materials. It also maintains the principle of "minimal disturbance to the natural state of river".

A side intake without a weir is unlikely to be suitable for river Types 1C, 2B and 2C, due to the strong possibility of the river course shifting in the future.

Side intake with weir

The function of a weir is to raise the water level in order to ensure a constant minimum depth of water upstream of the weir. This allows the required flow to be diverted to the headrace as long as there is sufficient water in the river. Types of weir are described in Section 3.6.

Side intake with and without gate

Gate(s) should be provided for a side intake if the river is relatively large and thus inflows into the intake need to be regulated, and at times stopped, such as during emergency or maintenance work. However if the river flow is low then gates may not be essential and only provision for a spillway downstream of the intake may be sufficient. One should note that provision of orifice like structures (can be as simple as a wall on top of intake canal) will help to control entry of excessive flows during floods. However, even with an orifice, some higher flows (than required for full power generation) will enter the intake during flood events. Thus, immediately downstream of the intake a spillway should be provided at an appropriate location so that only the desired flow is conveyed further downstream. This will both enhance the performance of the structures downstream (e.g., gravel trap and settling basin) and reduce risks due to uncontrolled overflows. In such a case the canal between the intake and the spillway should also be capable of handling excess water that enters during flood.

3.5.2 TRASHRACKS FOR SIDE INTAKES

The trashracks for side intakes can be manufactured from flat steel, angles, tees or round bars welded together at fixed intervals. The trashrack at the intake is also known as "coarse trashrack" since the bar spacing is wider here compared to

Photo 3.3 Where an intake is liable to attract floating debris a trashrack may be necessary (Sri Lanka)

the trashrack at the forebay. For side intakes, the function of the trashrack is to stop boulders, cobbles, floating logs and branches from entering the headrace. Coarse trashracks for side intakes are not designed to exclude gravel and sediment. This is the job of the gravel trap and the settling basin.

The size of the trashrack should be such that the water velocity is approximately 0.6 m/s (a lower velocity is uneconomic, whereas a high velocity tends to attract bedload and debris, and results in increased headloss).

Since boulders can frequently impact the coarse trashrack, it needs to be robust, i.e., thick steel sections should be used. Depending on the length and width of the opening, nature of the sediment load and the required flow, a clear spacing of 50 mm to 200 mm can be used. The side intake coarse trashrack of the Galkot Micro-hydro Scheme is shown in Photograph 3.4.

*Photo 3.4 Coarse trashrack for 50 kw Galkot MHP, Baglung, Nepal. 65 mm * 10 mm flats at 75 mm centre. (See Appendix C for the drawing)*

3.5.3   ORIFICE DESIGN

A side intake normally includes an orifice downstream of the trashrack at the riverbank, through which water is initially drawn into the headrace. Sometimes, the side intake is just a continuation of the headrace canal up to the riverbank. However, as far as practicable, an orifice should be incorporated to limit excessive flows during floods. With an intake that is just a continuation of the headrace canal to the riverbank, excess flow cannot be controlled during floods. Such excess flow can damage the headrace canal and other structures downstream. However, the orifice need not be at the intake area (i.e., at the riverbank). If it appears that the intake is at a flood plain or susceptible to damage from boulders, then the orifice can be located downstream. In such cases the canal upstream of the orifice and the intake would be temporary and may require repair after every monsoon. An orifice is an opening (Figure 3.5) in the intake from which the river water is conveyed towards the headrace. The orifice allows the design flow to pass through it under normal conditions (i.e., low flow) but restricts higher flows during floods. The discharge through an orifice for submerged condition is:

$$i = AC\sqrt{2g\,(h_r\text{-}h_h)}$$

$$V = C\sqrt{2g\,(h_r\text{-}h_h)}$$

where:
Q is the discharge through the orifice in m^3/s
V is the velocity through the orifice
A is the area of orifice in m^2
h_r is the water level in the river next to the orifice relative to a datum.
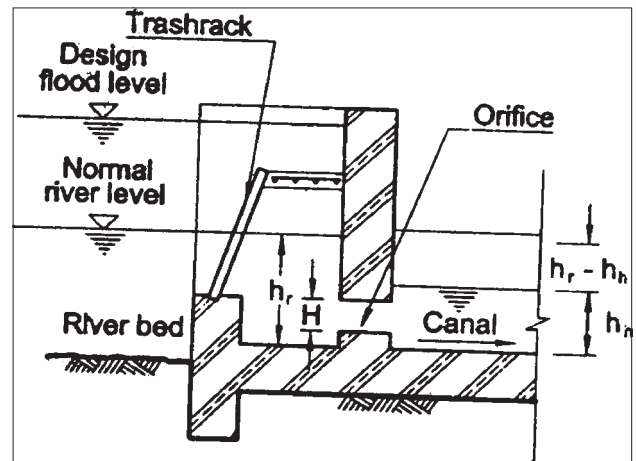
Figure 3.5 Slide intake

h_h is the water level in the headrace canal measured from the same datum as h_r.
g is the acceleration due to gravity = 9.8 m/s^2

C is the coefficient of discharge of the orifice and is dependent on the shape of orifice. The value of C decreases with the amount of turbulence induced by the intake. For a sharp edged and roughly finished concrete or masonry orifice structure this value is as low as 0.6 and for carefully finished aperture it can be up to 0.8.

(h$_r$ - h$_h$) will vary according to the discharge in the river since a higher water level in the river will produce a greater head at the orifice.

The maximum velocity for a well constructed concrete/ masonry orifice is 3 m/s: if the velocity exceeds this

value, the orifice surface will be scoured. For micro-hydro, the recommended velocity (V) through the orifice during normal flow is 1.0-1.5 m/s. Starting with a small orifice opening for normal flow (i.e., high velocity) will limit excess flow during floods, since the discharge through the orifice is proportional to the square root of the difference between the water level in the river and the headrace canal (h_r - h_h). However, if the orifice is directly at the river (without a trashrack) the velocity should be less than 1.0 m/s to avoid drawing bedload into the intake. The size of the orifice is calculated as follows:

- Assuming a maximum velocity of 1.5 m/s through the orifice, calculate the required area of the orifice opening using Q = V x A.
- For a rectangular opening, A = W x H
  where W is the width and H is the height of the orifice. Set H according to the river and ground conditions and calculate W.
- To ensure submerged condition, arrange the orifice opening such that the water surface level at the headrace canal is at or slightly higher (say up to 50 mm) than upper edge of the orifice. Note that the design of headrace canal is covered in Chapter 4. Hence the design of different micro-hydro components are interdependent. Now calculate h_r for the

design flow conditions. The h_r is the water level that needs to be maintained in the river during normal conditions. If the actual level in the river is less, repeat the calculations with larger width and smaller height of the orifice. If the actual river level is still less provide a weir with weir crest level at h_r

- Calculate the flow through the orifice for flood condition (h_r = design flood level). The excess flow (i.e., flow during flood less the design flow) will have to be spilled back into the river or nearby gullies in the initial reach of the headrace. This is discussed in the next Chapter.

An example of an orifice sizing is shown in Example 3.1

3.6 Diversion weirs

3.6.1 GENERAL

A weir is required if the flow cannot be diverted towards the side intake without raising the river water level, especially during the low flow season. The weir may be of temporary, semi-permanent or permanent construction. A temporary weir is the preferred option for micro-hydro schemes. In planning

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Example 3.1 Sizing of an orifice

Choose a suitable size of an orifice for a design flow of 250 l/s. The normal water level in the river is 0.8 m above the bed level. The design flood level is about 0.7 m above the normal water level. What is the discharge through the orifice during such a flood?

Q = 0.250 m³/s
Set V = 1.2 m/s

Orifice area (A) = $\dfrac{Q}{V}$ = $\dfrac{0.25}{1.2}$ = 0.21 m²

Set orifice height (H) = 0.20 m and Width of orifice (W)= $\dfrac{A}{H}$ = $\dfrac{0.21}{0.20}$ =1.05 m

Set bottom of orifice 0.2 m above the river bed level. This will minimise the bed load. Also, set the datum at the river bed level.

Set water level at headrace canal, h_h = 0.5 m with respect to the datum as shown in Figure 3.6 (i.e. 100 mm above the upper edge of orifice to ensure submerged condition). Note that later the headrace canal will have to be designed accordingly.

Q = $AC \sqrt{2g\ (h_r\text{-}h_h)}$
Assume C = 0.6 for roughly finished masonry orifice.
Q = 0.21 x 0.6$\sqrt{2 \times 9.8 \times (0.8\text{-}0.5)}$
= 0.31 m³/s or 310 l/s
$Q_{required}$ = 250 l/s: Therefore orifice size is OK.
Discharge through the orifice during flood flow:
h_r - h_h = 0.8 + 0.7 - 0.5 = 1.0 m
Q_{flood} = 0.21 x 0.6$\sqrt{2 \times 9.8 \times 1.0}$ = 0.56 m³/s
Q_{flood} = 560 l/s
The dimension of the orifice and the levels are shown in Figure 3.6. Note that the excess flood flow can be discharged via a spillway at the gravel trap or another suitable location. A second option is to install another orifice (double orifice system) downstream.
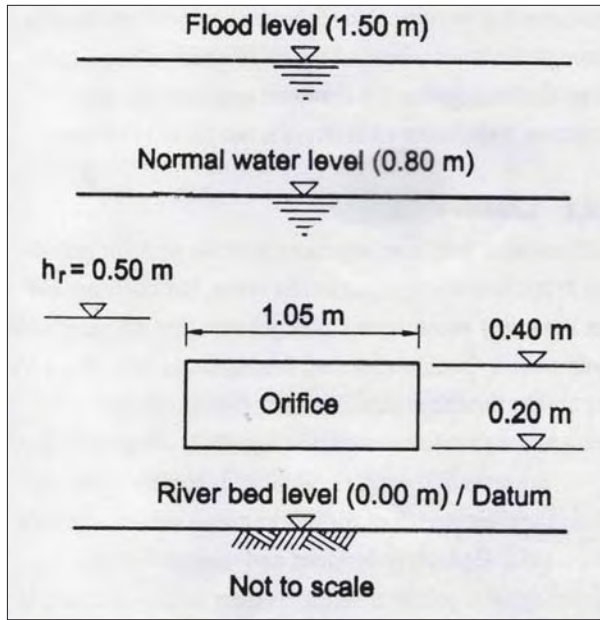
Figure 3.6 Dimensions of the orifice and levels

Photo 3.5 Timber planks placed horizontally in grooves provide a low-cast gate in a channel. In this case, the timbers have been lifted to act as a restricting orifice (Mhapung)

a weir, attention must be given to the geomorphology of the river, and any changes that may be taking place. Always consult with local people to establish whether the river is eroding (a general lowering of the river-bed), aggrading (a general building up of the riverbed), or shifting its course. The headworks design, and in particular the choice of weir, must take account of possible future changes. When it becomes apparent that a weir is required, the following factors should be considered for both permanent and temporary weirs:

- If a weir across part of the river length is sufficient, then it should not be extended across the entire width. Apart from adding extra cost it also encourages sediment deposition upstream of the weir.
- The weir height should be as low as possible (i.e., weir crest level = h_r, just sufficient to maintain the water level in the

intake). This makes the structure more stable, less susceptible to flood damage and also minimises sediment deposition.

3.6.2 TEMPORARY WEIRS

A temporary weir is typically constructed using boulders placed across part or all of the river width. A diagonal alignment may reduce the required height of the weir above the riverbed. This is the traditional method used by Nepali farmers to feed irrigation canals or water mills (ghattas), and is used quite extensively in micro-hydro schemes in Nepal. For micro-hydro schemes in the lower range such as those used for agroprocessing only, this type of weir is often appropriate.

Though a temporary weir is simple and low cost, it has a few limitations: for example it is not possible to divert all of the river flow, even in the dry season. Therefore this type of weir is best suited to situations where the dry season flow of the river exceeds the plant's design discharge.

During the high flows, even if the weir is washed away, it may still be possible to divert the required flow towards the headrace since the water level in the river is high.

A proposal for a temporary weir constructed of stone masonry in mud mortar is shown in Figure 3.7. This is intended to allow diversion of a higher proportion of the dry season flow, but would still get washed away during the annual floods. Constructing temporary weirs with boulders as large as can be handled manually and including cut-off walls and riprap can minimise flood damage. As can be seen in Figure 3.7, cut-off walls are downward extensions of the weir at the upstream and downstream faces that reduce seepage past the weir. Riprap is an engineering term used to denote the placing of a layer of boulders for scour protection. The omission of scour protection would result in scouring of the riverbed, eventually leading to the failure of the weir itself.

Photo 3.6 A temporary weir in Type 2B river (Dhading)

Once the river discharge decreases a temporary weir can be usually reconstructed at little cost. The repair and maintenance work on a temporary weir can be minimised by building the weir using rock outcrops, large boulders and other natural protection of the river. Good management of cash for the annual weir "rebuild" is required. In most cases, a temporary weir is suitable only for the diversion of flows below 1 m³/s. This fits well into the micro-hydro discharge range, since the maximum flow in micro-hydro schemes rarely exceeds 500l/s. For micro-hydro schemes, a temporary weir is the preferred option over more permanent structures. This is because most rivers flowing through the mountains and the middle hills of Nepal carry large boulders during the monsoon and therefore any structure built across such rivers is not likely to survive.

3.6.3 GABION WEIR

Gabions have been used extensively in the past, for both micro-hydro and irrigation intake weirs, but the result has not been very encouraging. The gabion wires are vulnerable to damage by boulders moving during floods, and after a few are broken the entire gabion structure may collapse. Gabions are therefore unsuitable in river Types IB and 1C.

However, if there is no significant boulder movement along the river stretch at the intake area a gabion weir may be possible. If properly designed and constructed, the advantage of a gabion structure is that, unlike concrete and masonry structures, it can tolerate some ground movement without significant damage. The Jhankre mini-hydro weir is an example of a gabion weir structure (see Figure 3.8 and Box 3.1). The weir design should include checking:

- Safety against scour (by founding on rock or large boulders, or by constructing a 'counterweir* downstream to form a stilling pool)
- Seepage control (by using an impermeable membrane)
- Stability against overturning and sliding.
- Safety on bearing capacity of the foundation.

Note: All dimensions are in mm.
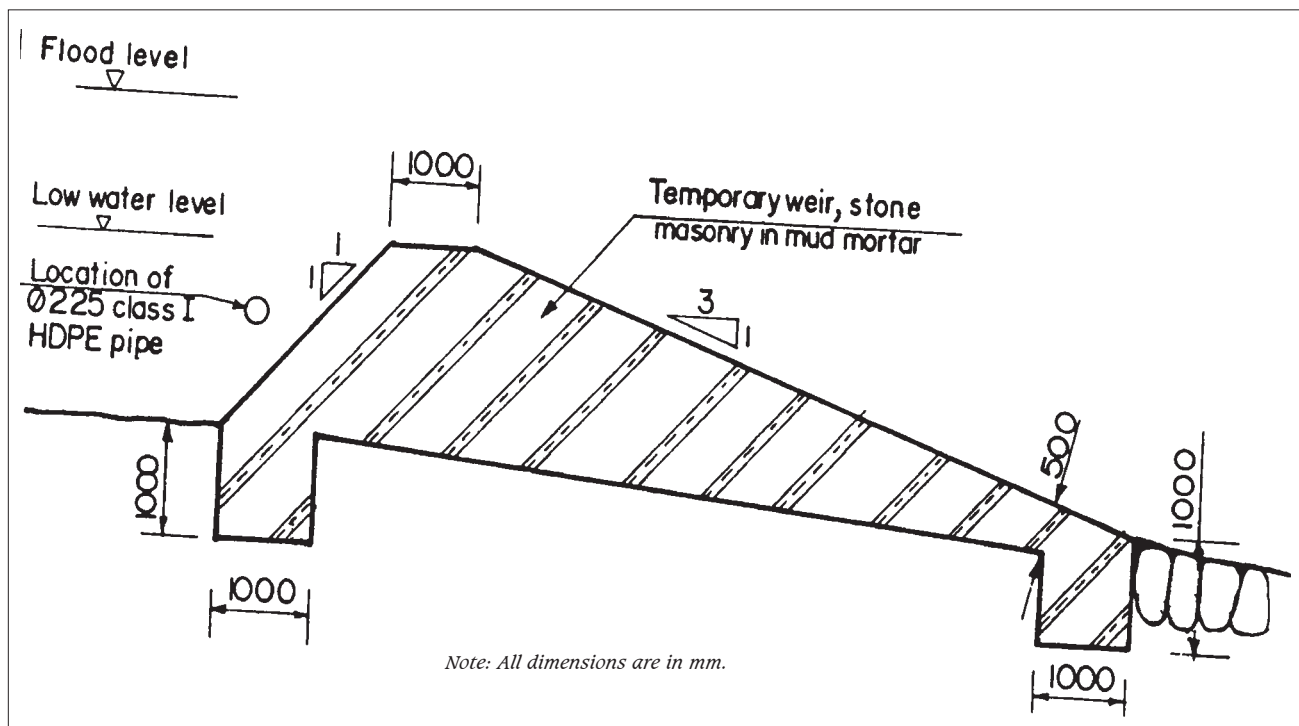
Figure 3.7 A temporary weir proposed for the kw Thorong Phedi micro-hydro scheme, Morang, Nepal. Note the cut-off walls and riprap at downstream face.
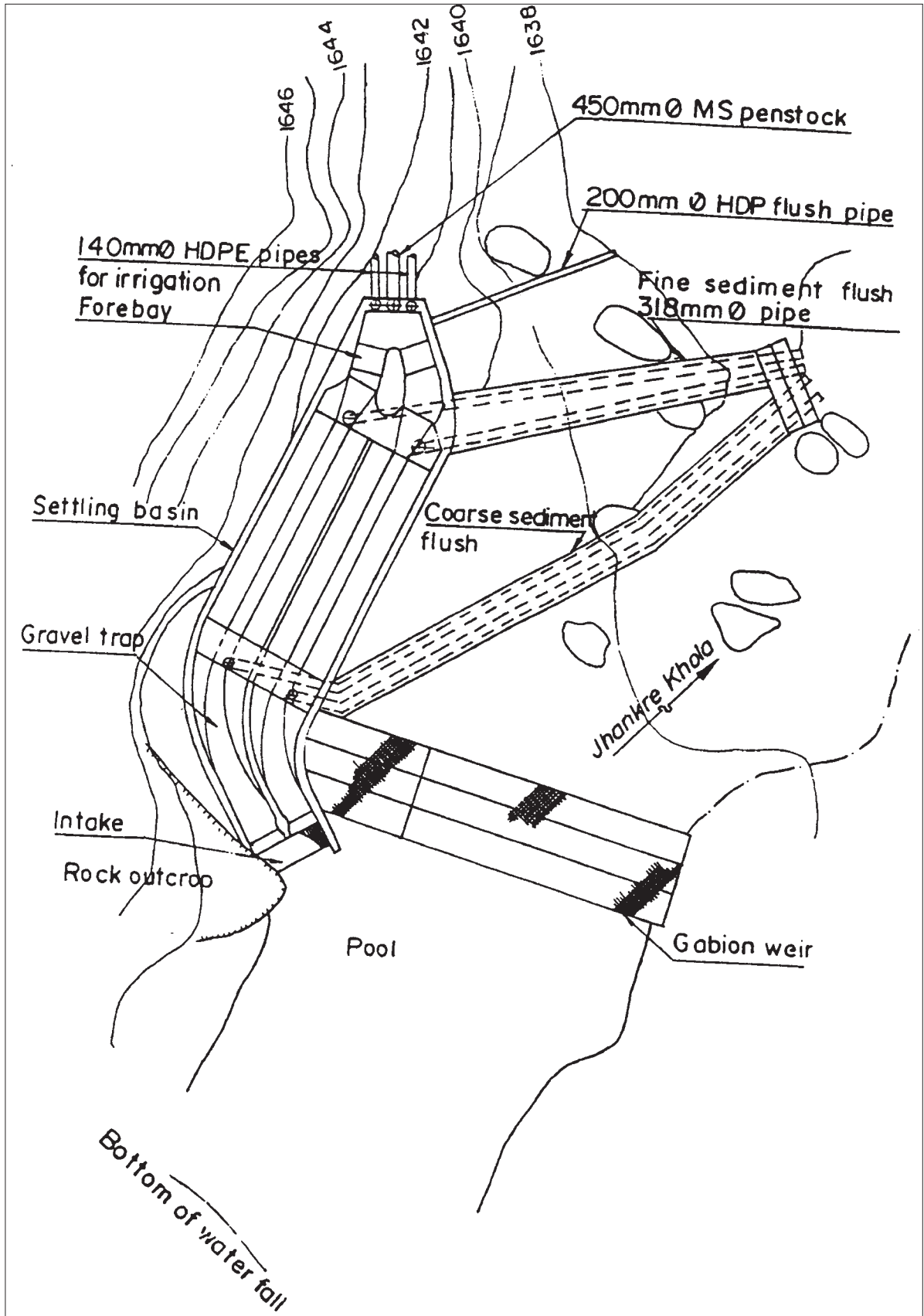
Figure 3.8 Headworks arangement of the 500kw Jhankre mini-hydro scheme, Nepal. Note that the intake, setting basin and the forebay are one combined structure and there is no headrace. The topography is such that a penstock alingment could be started right at the headworks.

Box 3.1 Composite gabion concrete PVC sheet weir Jhankre Nepal
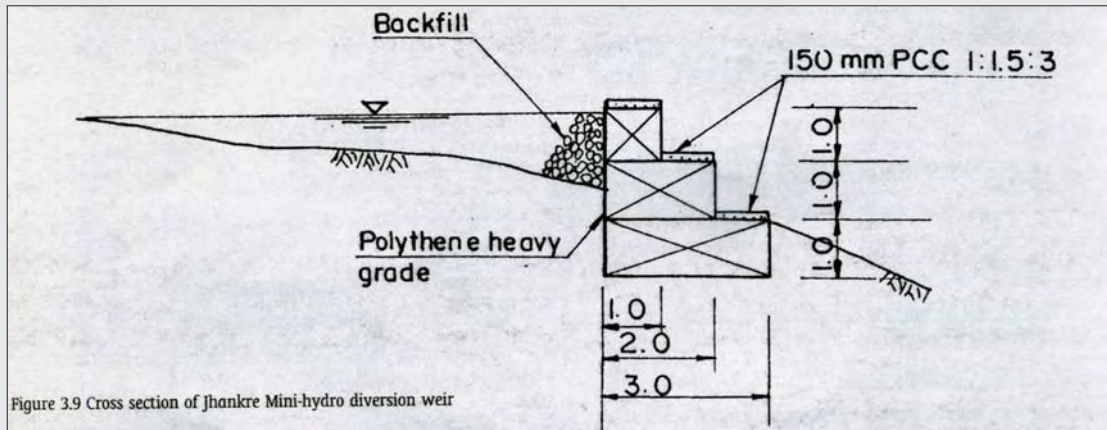
Figure3.9 Cross section of Jhankre Mini-hydro diversion weir

Photo 3.7 Diversion weir of the Jhankre Mini-Hydro Scheme

The 500 kW Jhankre Mini-hydropower Scheme, located on the Jhankre river, Dolakha, Nepal, was designed jointly by BPC Hydro consult and Development and Consulting Services (DCS). The construction was undertaken by DCS. As can be seen in the section through the weir, to minimise seepage in the dry season, a heavy grade polythene sheet has been fixed at the upstream face of the gabion weir. To prevent the sheet from being punctured by boulders and other debris, stone backfill has been incorporated in front of it. Also to prevent the ga-bion wires from being nicked by rolling boulders, 150 mm of plain concrete is placed along all exposed surface of the weir.

During a 1995 monsoon flash flood (estimated to be a 1:30 year return period flood), this weir was partially damaged. It was then repaired. Since then, the weir has faced two annual floods without any repairs. Occasional repair of the concrete topping and the gabion wires are expected (i.e. during the annual maintenance period).

3.6.4 PERMANENT WEIR

Sometimes if there is a scarcity of water, especially during the low flow season, and the river does not carry large boulders, a permanent weir may be built across the river. Micro-hydro schemes in the higher range (50 kW or above) and mini-hydro schemes (100 kW to 1000 kW) often have permanent weirs. Permanent weirs are generally constructed of mass concrete (1:1.5:3 with 40% plums), or stone masonry in 1:4 cement mortar. A reinforced concrete surface layer may be considered to protect the weir from damage by boulders moving in flood. A more recent experience in Nepal in small hydropower plans has been to line the weir surface using 300 mm hard stone in rich mortar (1 cement to 2 sand ratio) instead of reinforced concrete, especially if large boulders and sediment movement occur during floods. This arrangement has more abrasion resistance capacity than that of reinforced concrete and is relatively easy to repair, i.e., replace the abraded boulders with new ones using rich cement mortar, whereas replacing reinforcement bars can require welding works .

For micro-hydro scheme, generally a permanent weir should only be considered if all of the following conditions are met:

- Large boulders do not move in the river at the weir site.
- The river bed is stable (not eroding, aggrading or shifting course)
- There is a scarcity of flow, especially in the dry season.
- Skilled masons are locally available for both construction and maintenance.
- There are sufficient funds for both construction and future maintenance work.

Even if all of the above conditions are met, further consideration should be given for remote sites (3-4 days walk) because of the cost and difficulty involved in transporting cement and reinforcing steel. Besides considering the factors mentioned earlier (use of large boulders, weir across part of the river if possible and low weir height), for permanent weirs scour protection should also be provided. The toe of the weir (i.e., downstream face of the weir) is most susceptible to scouring since there is a drop from the crest of the weir. Protection against scour is provided by cut-off walls and placing boulders (such as stone soling) or riprap downstream of the weir as shown in Figure 3.10. The cut-off walls also reduce seepage under the weir, which can increase the flow available to the intake during the dry season. Figure 3.11 shows another

Note: All dimension are in mm, levels are in m.
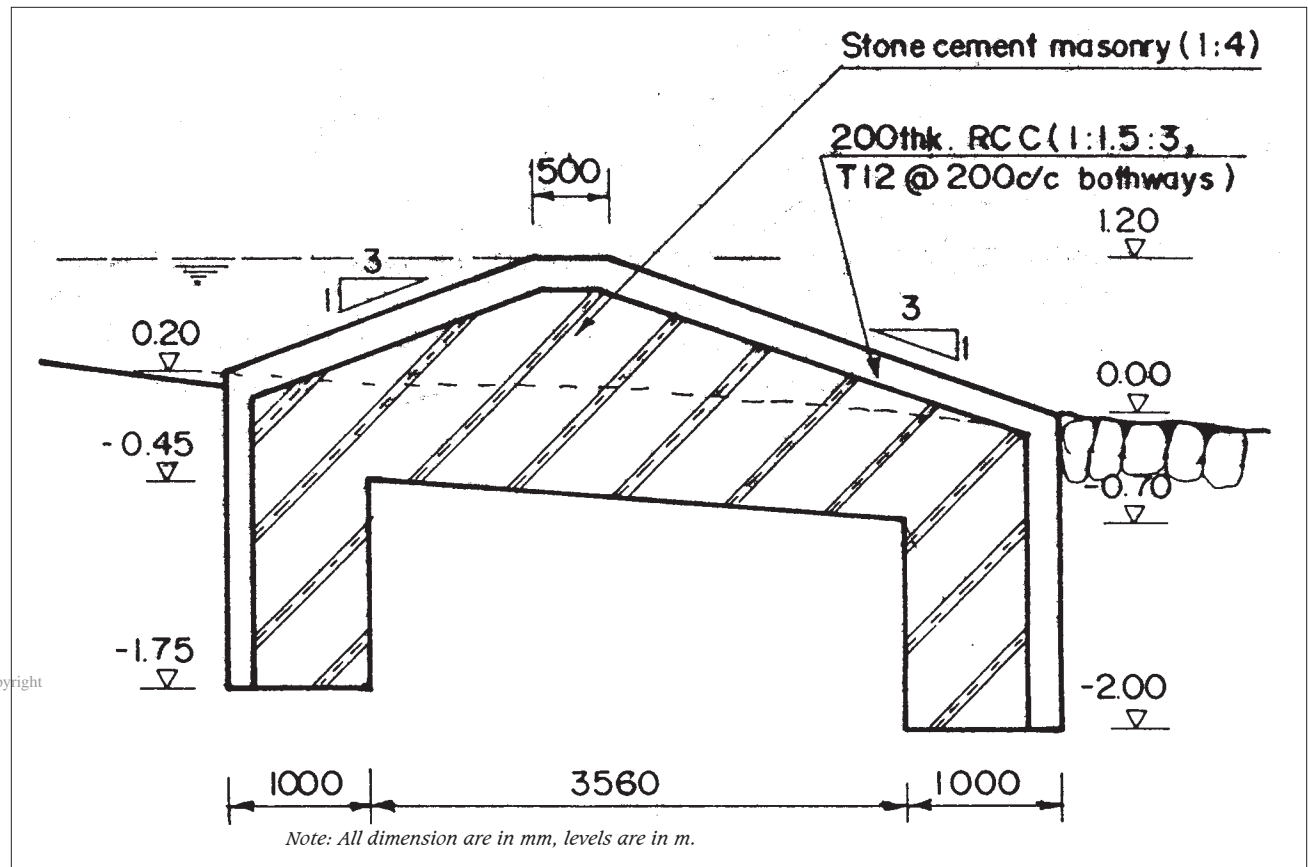
Figure 3.10 A stone masonary parmanent weir proposed for Ghami micro-hydro scheme, Mustang, Nepal. The 1:3 slope allows rolling boulders to travel downstream through the weir and the RCC blanket helps to strengthen the weir surface against abrasion due to rolling boulders. Also note the cut-off wall and riprap at downstream face

option for a permanent weir in case bedrock is found at the proposed site. When the weir is on bedrock, deep cut off walls and riprap are unnecessary. As shown in Figure 3.11, shallow cut off walls and anchor rods can be used to fix the weir on the rock surface: the anchor rods should be grouted into the rock.

As mentioned earlier, boulder lined weirs are becoming popular in small and larger hydro (even up to 60 MW) sector of Nepal as they are able to better resist the impacts of rolling boulders during flood events. Furthermore, boulders are also widely available around river banks. Although heavy earth moving equipment (e.g., excavators) are used to lay boulders (sometimes up to 2.5 m size) in larger hydropower projects, the boulder sizes that could be laid manually (say around 300 mm) could be considered in micro-hydro projects.

High sediment concentration and boulder movement during flood along with debris flow are common in many small rivers in the hills and mountains of Nepal. In such conditions, boulder lined weir appears to be superior to other types. This type of weir for larger hydro projects mainly consists of central RCC cut off wall to control seepage and boulder lining on graded filter (well graded gravel, pebble and small boulders). At the downstream end of the weir, a random reinforced plum concrete with 300 mm size boulder armoured surface is also sometimes incorporated for scour protection. Clay blanket or impervious membrane could be placed at the upstream end of the weir if the seepage control cannot be achieved only with a central cut off wall. The clay blanket needs to be placed beneath the graded filter on a geotextile wrapper. One important point to be considered is that the downstream side slope of the weir should not be steeper than 1 vertical to 6 horizontal to minimise the impact of rolling boulders. However the upstream side slope could be as high as 1 vertical to 2 horizontal. In this type of weir, sediment deposition at the upstream side of the weir crest will occur with time. Thus intake orifice sizing and siting are some of the challenges here and special attention in designing to ensure flow availability all times together with minimal sediment entry. A typical boulder line weir section is shown in Figure 3.11

3.6.5 HEAD OVER WEIR

As stated earlier, placing a weir across the river raises the water level. Any excess flow that is not withdrawn into the intake flows over the weir. The discharge over the weir is given by the following equation:

$$Q = C_w \times L_{weir} \times (h_{overtop})^{1.5}$$
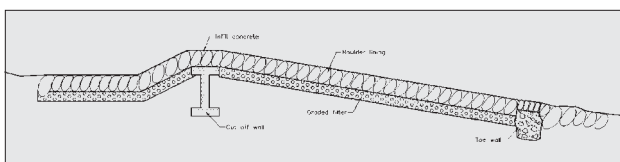
Figure 3.11: Boulder lined weir

where:
Q = Discharge over the weir in m^3
L_{weir} = Length of weir in m
$h_{overtop}$ = Head over the weir crest level in m
C_w = Weir coefficient which varies according to the weir profile. C_w for different weir profiles is shown in Table 3.3. In micro-hydro, the weir is usually broad with round edges and therefore C_w is l.6 .
Profile of crest of weir C_w

Profile of crest of weir	C_w
broad; sharp edges	1.5
broad; round edges	1.6
round overfall	2.1
sharp-edged	1.9
rounded	2.2
roof-shaped	2.3

Table 3.2: Profile of crest of weir

The weir equation is also useful in calculating the flood levels at the intake if the flood discharge is known or can be calculated based on the river hydrology. Once the flood levels are known, the flood protection walls at the riverbank can be designed. For known discharge over the weir, the head over the weir (and hence the water level at the intake) can be calculated by rewriting the weir equation as follows:

$$h_{overtop} = [Q/(C_w \times L_{weir})]^{0.667}$$

Calculation of head over a weir can be seen in Example 3.2

3.7 Bottom intakes

3.7.1 DESCRIPTION

The bottom intake, also known as a Tyrolean or trench intake, is a grille-like opening that captures water from the bed of the river and drops it directly into the headrace. The flow generally passes through an opening in a wing wall of the intake structure and away from the river. In some cases the grille may cover a small chamber, but generally the bottom intake is designed as a trench, perpendicular to the direction of the river flow.

The bottom intake is most appropriate in locations where there is no appreciable sediment movement along the riverbed, because it withdraws bottom water in preference to surface water. This type of intake was first used for small hydro and irrigation systems early this century in alpine areas of Europe.

Worldwide practice shows that it is applicable in small rivers in mountainous and hilly regions, where the following conditions exist:

- Steep river bed of bare rock or boulders which rarely move (they are suitable for flow velocities exceeding 3 m/s);
- Minimal bed load of sand and gravel;
- Surplus flow available for continual flushing. To date only a few bottom intakes have been constructed in Nepal, so Nepalese farmers are not familiar with them. The design of bottom intakes must be done carefully to avoid becoming blocked with sediment. Bottom intakes for Thame and Jhong micro-hydro scheme are shown in Photograph 3.9 and 3.10.

3.7.2 TRASHRACKS FOR BOTTOM INTAKE

Similar to side intakes, the trashracks of bottom intakes can be manufactured from flat steel, angles, tees or round bars welded together at intervals. The section chosen must be strong enough to withstand impact by any bed load moving during floods. Its shape is also very important, since this affects the chances of clogging. Round bars, for example, are more prone to clogging, because the opening in the middle is smaller than on the top. From the point of view of clogging, the sections listed below are arranged in the order of best to worst:

- Tees
- Angles
- Channels
- Flats
- Round bars

The recommended clear spacing between these flats, angles or bars is 6 to 15 mm and a commonly used spacing is 12 mm.

Box 3.2 Slotted concrete piers, Atahualpa and Tambomayo, Peru.

In Peru, Practical Action has been using an approach to design of intakes which uses planks slotted in piers perpendicular to the direction of flow of the river. Short reinforced concrete piers are constructed at a spacing of 2 metres. Each pier has vertical grooves along its full depth. Then timber planks are inserted between the piers by inserting the ends of planks onto the slots. During the rainy season, one or more spans can be removed to regulate the flow at the intake orifice and to allow river-borne debris to flow along it without causing damage to the whole structure.

Photo 3.8 Slotted concrete piers

Plum concrete (1:3:6, 40% Plums)
200 thk RCC (1:1.5:3)
T12 @ 200c/c bothways)
1.20
500
3
3
1
300 min.
0.20
650 200
Bed rock
0.00
T16 anchor rod 1200 long @ 300c/c, grouted with 1:1:0.4(cement:sand: water) grout.

Note: All dimensions are in mm, levels are in m.
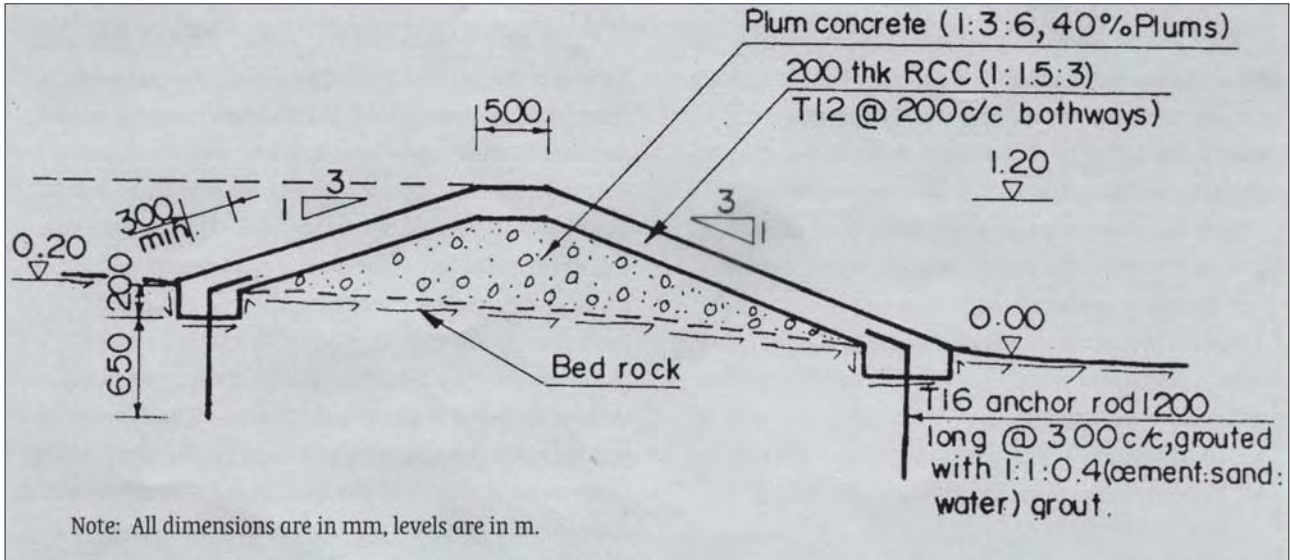
Figure 3.12 A plum concrete permanent weir proposal for Ghami micro-hydro scheme. The 1:3 slope allows rolling boulders to travel downstream through the weir and the RCC blanket helps to strengthen the weir surface against abrasion due to rolling boulders.

Photo 3.9 Bottom intake of Theme micro-hydro scheme, Nepal .

Photo 3.10 Bottom intake with the grill removed (Jhong)

Example 3.2 Head over weir calculation

A broad crested weir has been placed across a river for a micro-hydro intake as shown in Figure 3.12. The weir height is 0.5 m and the length 5 m. How high should the flood protection wall be for a 20 year return flood of 11 m³/s?

C_w = 1.6 for broad crested weir :
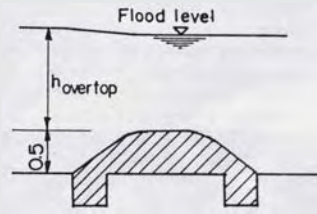
Figure 3.12 Broad crested weir

$$h_{overtop} = \left(\frac{Q}{C_w \times L_{weir}} \right)^{0.667}$$

Note that C_w is 1.6 for broad crested weir

$$h_{overtop} = \left(\frac{11}{1.6 \times 5} \right)^{0.667} = 1.24 \text{ cm}$$

Height of flood protection walls from river bed level = 0.5 m m + $h_{overtop}$ + 0.3 m (allow 300 mm of freeboard) = 2.04 m.

The reason why these bars are closer than those of the side intake trashrack is that gravel also needs to be excluded from the bottom intake. Since the initial headrace for this type of intake is covered, it would be difficult to remove any gravel that obstructs the flow. It should therefore be excluded. The spacing of the flats or angles depends on the predominant particle size of the sediments carried by the river flow (i.e. bed load) and the provision for a settling basin in the canal system. The larger the spacing (opening), the larger the particles that will enter the headrace. On the other hand, if the openings are too narrow, there is a high chance of clogging necessitating frequent cleaning of the trashrack. It is also important to place the trashracks such that the bars are along the direction of flow. This minimises the risk of clogging.

One of the drawbacks of the bottom intake is the clogging of trashrack by pebbles and dry leaves. Especially during the dry season, the river may carry a lot of leaves, which become trapped in the trashrack and reduce the flow through it. Therefore the trashrack needs to be cleaned periodically during the dry season. During monsoon, this is not a problem; the river flow sweeps the gravel and leaves before they can clog the trashracks.

3.7.3 DESIGN OF BOTTOM INTAKE

The following equation is used for the design of a bottom intake: $Q_A = 2/3 \, c \, \mu \, b \, L\sqrt{2gh}$

TABLE 3.4 x values for ß

ß	x	ß	x
0°	1.000	14°	0.879
2°	0.980	16°	0.865
4°	0.961	18°	0.851
6°	0.944	20°	0.837
8°	0.927	22°	0.825
10°	0.910	24°	0.812
12°	0.894	26°	0.800

where:
Q_A = design discharge into the intake in m^3/s
b = width of the bottom intake in m
L = length of the trashrack in m. In practice, it is recommended that the trashrack length (L) be increased by 20%, i.e., L= 1.2 x $L_{calculated}$. This will ensure that there will be adequate flow when the trashrack is partially blocked by wedged stones and branches.

$h = 2/3 \times h_E$
h_o = Initial water depth in m in the river upstream of the intake.
$h_E = h_o + v_o^2/2g$.

Note that as can be seen in Figure 3.13 h_E is actually the initial water depth in the river plus the velocity head of the
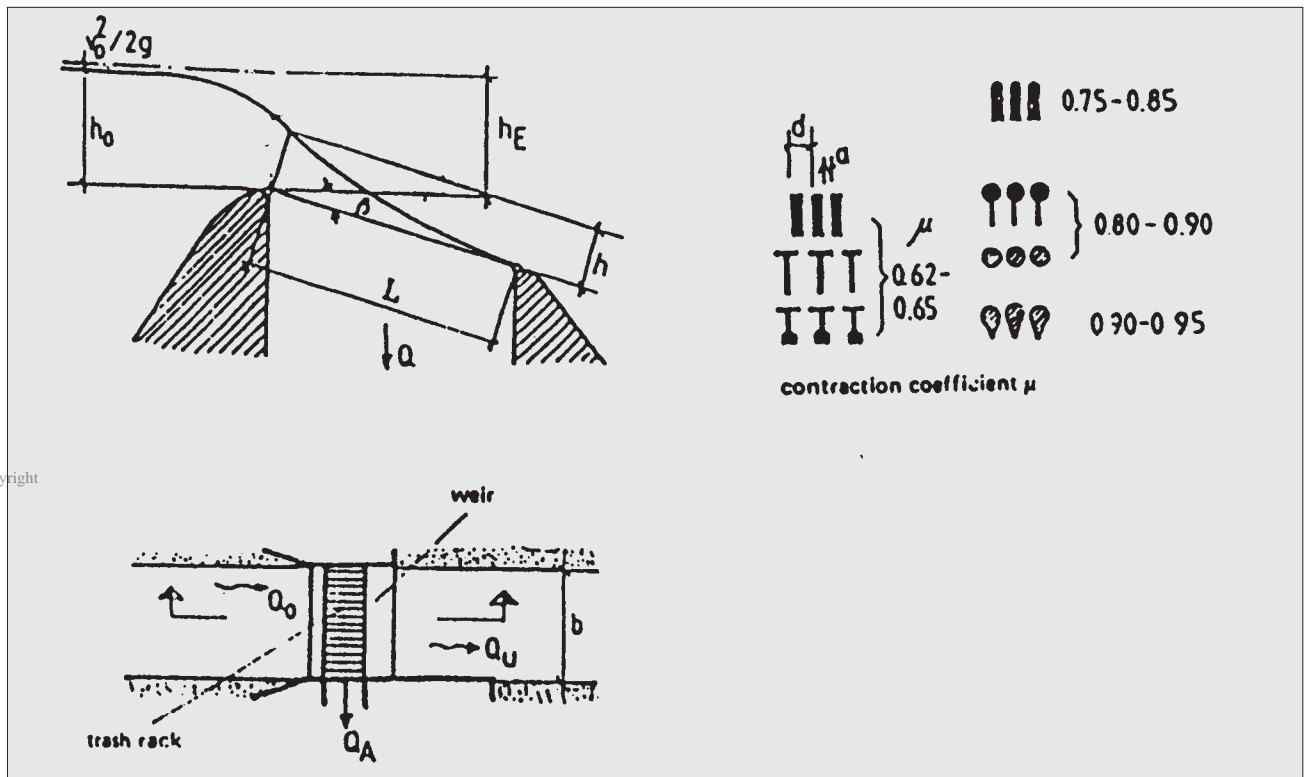
Figure 3.13 Symbols used in the bottom intake equation.

Example 3.3 Sizing of a bottom intake

A suitable site has been located for a bottom intake. The river width at this area is 5 m and the depth is 0.5 m (i.e. h_o = 0.5 m). A velocity of 3 m/s was measured at the intake site. The design flow (Q_A) required for power generation is 0.40 m³/s. Select an appropriate size for the bottom intake.

Design calculations

Choose 20 mm diameter round bars for the trashrack.

μ = 0.85 for round bars (from Figure 3.12) Set the clear spacing between the bars,

a = 12mm

Centre to centre distance between bars,

d= 32mm

Set the inclination of the trashrack ß = 8° (The inclination of the trashrack should be equal to or slightly greater than the river gradient)

For ß = 8° , X = 0.927

$h = 2/3 X . h_E$

$h_E = 0.5 + 3^2/2g = 0.96$ m

Or h = 2/3 x 0.927 x 0.96 = 0.59 m

$c = 0.6$ a/b $\cos^{3/2}$ß

c = 0.6 x (0.012 / 0.032) x $\cos^{3/2}$ (ß 8°) = 0.22

Now use the bottom intake equation:

$Q_A = 2/3$ cμbL $\sqrt{2gh}$

$Q_A = 2/3$ x 0.22 x 0.85x b x L$\sqrt{2 \times 9.8 \times 0.59}$

Q_A = 0.42 x b x L

With Q_A = 0.40 m³/s:

b x L = 0.40 / 0.42 = 0.95 m²

or L = 0.95 / b
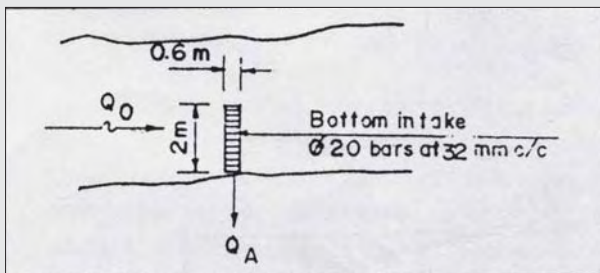
Figure 3.14 Dimension for the bottom of Example 3.3

Select the width of the trashrack, b = 2 m L = 0.95/2 = 0.48 m. Increase the length by 20%: L= 0.48 x 1.2 = 0.57 m. The proposed dimensions of the bottom intake are as follows:
Width of the opening, b = 2.0 m (at right angles to the flow)
Length of the opening, L = 0.6 m (parallel to the river flow)
Trashrack bar size = 20 mm diameter round bars
Bar spacing = 32 mm centre to centre

The plan of this proposed bottom intake is shown in Figure 3.14.

river ($v_o^2/2g$). For steep rivers, the flow velocity should be measured since the velocity head can be high.
X = a function of the inclination of the trashrack (P) as shown in Table 3.4.
C = Correction factor for submerged overfall,
    = 0.6 x 4- a/d $\cos^{3/2}$ß
a = clear spacing of the trashrack bars in m.
d = centre to centre distance between the trashrack bars in m,
ß = angle of inclination of the trashrack with respect to the horizontal in degrees.
μ = contraction coefficient for the trashrack, which depends on the shape of the bars as shown in Figure 3.12.

Also in the figure, Q_o is the river flow upstream of the intake and Q_u is the excess flow in the river downstream of the intake.
Note that to solve the bottom intake equation, either the length or the width of the intake opening needs to be set and the other dimension can then be calculated. The selection of one of these dimensions depends on the site conditions. For example, if the length of the trashrack is too small, the headrace canal will require deeper excavation in the riverbed, which may be difficult. Generally, the length of the bottom intake should be equal to the width of the headrace canal, and the width should match the river channel.

Photo 3.11 Gabion walls at the headworks of the 30kw Jharkot micro-hydro scheme, Mustang, Nepal.

It is important that the culvert beneath the trash rack is steep enough to convey the maximum conceivable sediment load to the gravel trap: a gradient of at least 1:20 is recommended. The gravel trap may require continuous flushing, which means that sufficient head and surplus flow has to be available. The design must be able to carry and spill back to the river the maximum flow entering the intake under flood conditions. Engineers designing a bottom intake should refer to References 3 and 5 for further information.

3.8 River training works

A flood protection wall along the riverbank may be required if there is a high probability of flood damage to the initial headrace and other structures such as the gravel trap and settling basin. Such walls are also called river training structures since they confine the river channel. The wall height should be greater than or at least equal to the design flood level.

The foundation of any river training walls must be protected from undermining by the river. This can be done by one of the following methods.

- Founding the wall on rock or large boulders. For gabion walls it may be necessary to first build up a level base using stone masonry or mass concrete.
- Founding the wall below possible scour depth.
- Using a gabion mattress along the river side of the wall. This method is not appropriate in rivers carrying a heavy

bed load, because the gabion wires will be damaged by boulders moving during floods.

On alluvial rivers (i.e. deep deposition of sand and cobbles), gabion flood protection walls are usually more appropriate for micro-hydro scheme. This is because the ground of alluvial rivers tends to change and flexible structures can cope better in such conditions. Gabion walls may require annual maintenance (especially after monsoon) therefore skilled manpower should either be available at site or some local people should be trained during the construction phase. Gabion walls can also serve the function of retaining walls and stabilise the slopes behind it. If slopes at the alluvial riverbank are unstable, then gabion walls can also be designed as retaining walls. Photograph 3.11 shows the use of a gabion wall to stabilise the bank slope. Refer to Section 9.4 for retaining wall design. On stable riverbanks, such as exposed bedrock, a masonry wall can be built provided that the river does not carry large boulders that could damage masonry structures. In large hydropower and irrigation projects even concrete flood barrier walls are used but usually such solutions are economically unjustifiable for micro-hydro schemes. Figure 3.15 shows the use of a gabion wall to prevent the river bypassing the diversion weir and damaging the headrace pipe during floods.

3.9   Checklist for diversion works

- Refer to Table 3.1 and find out what category the source river falls in.
- Refer to Table 3.2 and decide on whether a side intake or a bottom intake is suitable.
- Is a weir required or is it possible to divert the river water
- without one? Remember the concept of "minimal disturbance to the natural state of the water".
- Does the river course appear stable or does it look like it will meander? Think about flood and flood levels. Also, if the river carries large boulders during the floods, and a weir is required, think about temporary diversion works rather than a permanent weir.
- To minimise flood damage the intake location should be such that it is possible to set the headrace alignment immediately away from the river course.
- If a side intake has been selected along a river bend, remember to locate it on the outside of the bend.
- Have the flood levels and history of the river course been discussed with the local community members?
- Finally, consider the cost of different options. Is it more economic to construct temporary diversion works and incur some annual labour charges or to choose more permanent diversion works?
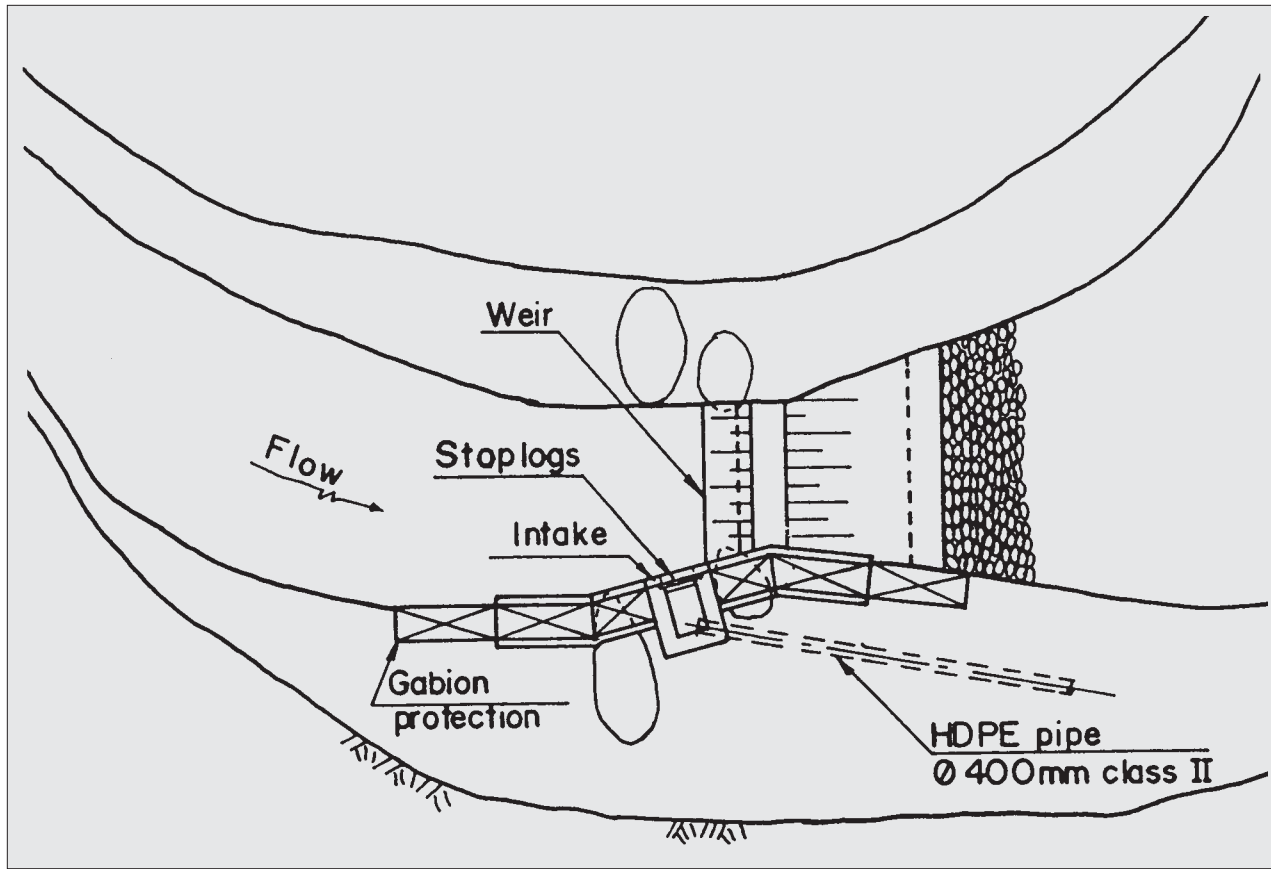
Figure 3.15 Headworkss arrangement of the 80kw Bhujung MHP, under construction in Lamjung, Nepal, Notice the flood protection gabion walls and riprap at downstream face of the weir.

4. Headrace

4.1 Overview

The headrace of a micro-hydropower scheme is a canal or a pipe that conveys water from the intake to the forebay. The headrace alignment is usually on even to gently sloping ground and the flow is caused by gravity. A headrace pipe is generally not subjected to significant hydraulic pressure.

Since canals are generally less expensive than pipes, they are used more often for headraces in micro-hydro schemes. The general rule is to use canals as often as possible and to use pipes only for the difficult stretch of the headrace alignment, such as to negotiate cliffs or unstable areas.

Micro-hydro headrace canals are similar to farmer managed small irrigation canals in that they are designed to keep seepage, friction and erosion to a minimum. However, there are also some basic differences as follows:

- Irrigation canals are used only 3-6 months in a year whereas micro-hydro schemes require water throughout the year.
- In irrigation canals, some variation in flows does not create much problem, and temporary repairs (e.g. placing of branches and leaves at a leaking section of a canal) can be made. The headrace canal in a micro-hydro scheme needs to be more reliable.
- The loss of head over the length of the headrace should be minimised so that power output can be optimised.

Some micro-hydro texts use the term power canal/ conduit for either the length between the intake and the settling basin (when this structure is separate from the forebay) or for the entire headrace. In this text the term headrace is used in all cases.

The velocity in the initial headrace length needs to be high enough to carry gravel and sediment up to the gravel trap and settling basin respectively. Where there is a separate settling basin and forebay, the velocity in the headrace between these structures can be lowered since it will carry sediment free flow.

4.2 Canal types

Headrace canals can be classified according to the materials used to construct them. Various types of headrace canal used in micro-hydro schemes are as follows:

4.2.1 EARTH CANALS

These are constructed by simply excavating the ground to the required shape. Such canals are used on stable and gently sloping ground. Seepage can be high in such canals depending on soil type. If there are signs of instability in a headrace section, or if seepage from the canal is likely to contribute to slope instability such as landslides, this type of canal should not be selected. However, for headrace alignments on stable ground where seepage is not likely to cause instability, earth canals are the most economic option.

Where clay is locally available it could be considered as a lining to reduce seepage from earth canals. However, a scheme

Photo 4.1 Earth Channel (Dhading)

in Syangja successfully used a clay lining protected by stone pitching for the peaking reservoir (which was fenced off) but in the headrace canal the lining was destroyed by cattle.

4.2.2 STONE MASONRY IN MUD MORTAR CANALS

If an earth canal does not appear to be feasible, the second option to be considered is stone masonry in mud mortar type. Compared to an earth canal, there will be less seepage from this type of canal. For similar flows, the cross section of this type of canal can be smaller than the earth canal because a higher velocity is acceptable (without causing erosion) as will be discussed later. An example of a stone masonry canal in mud mortar can be seen in Photograph 4.2.

4.2.3 STONE MASONRY IN CEMENT MORTAR CANALS

In terms of cost, this is usually the least preferable option for a headrace canal. The advantage with this type of canal is that seepage is minimal (i.e. significantly less than stone

Photo 4.2 A raised masonary channel supplying a mill (Mustang).

masonry in mud mortar canals). A stone masonry in cement mortar canal should be used at locations where the soil type is porous (leading to losses of unacceptable amounts of flow) and/or seepage is likely to trigger landslides. For micro-hydro sites located 3-4 days walk from the roadhead, the need for a long cement mortar canal can make a micro-hydro scheme uneconomic due to the high cost of cement.

An example of a stone masonry in cement mortar headrace canal can be seen below in Photograph 4.3.

Photo 4.3 Stone masanory in cement mortar headrace canal of the 50kw Galkot MHP, Galkot, Baglung, Nepal.
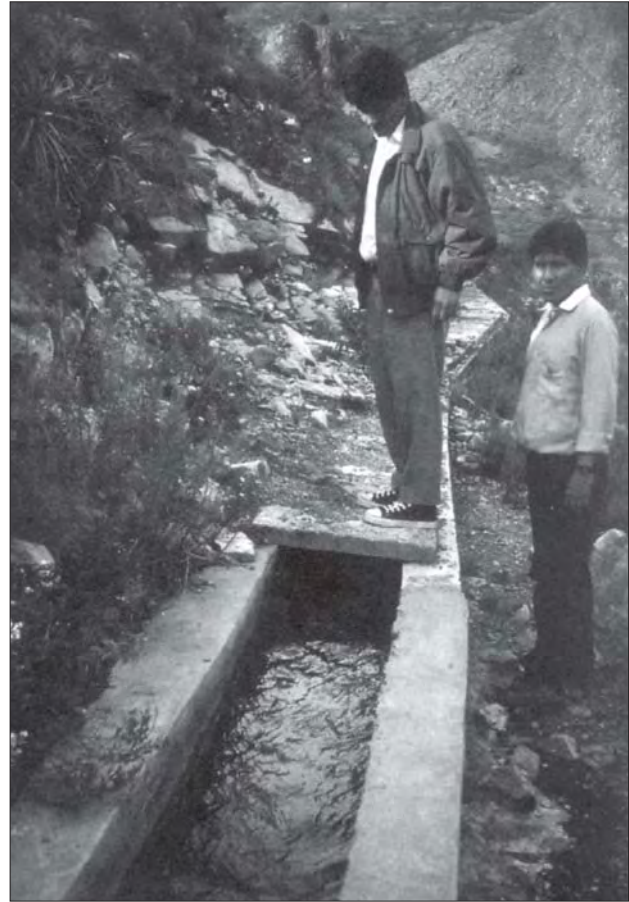
Photo 4.4 Reinforced concrete cover slabs provide protection from falling debris (Bolivia).

4.2.4 CONCRETE CANALS
Most micro-hydro schemes do not have headrace canals constructed of concrete since they are very expensive. There is virtually no seepage through such canals. Sometimes, re-inforced concrete canals are used for short crossings.

Generally, HDPE headrace pipes are more economic than concrete canals.

4.2.5 OTHER TYPES OF CANAL
In certain areas there may also be other types of canal than those mentioned above. For example, an irrigation canal in Ecuador constructed of used oil drums cut into two semi-circular halves can be seen in Photograph 4.5. Such a canal may be useful for short and difficult sections or for aqueducts where used drums are easily available and economical.

Another example is the use of timber canal as can be seen in Photograph 4.6. This requires the use of hardwood and skilled labour. Similar to oil drums, timber canals can be possible for short crossings and aqueducts or where timber is abundant and inexpensive.

Examples of other types of canal are presented in Boxes 4.1 to 4.5.
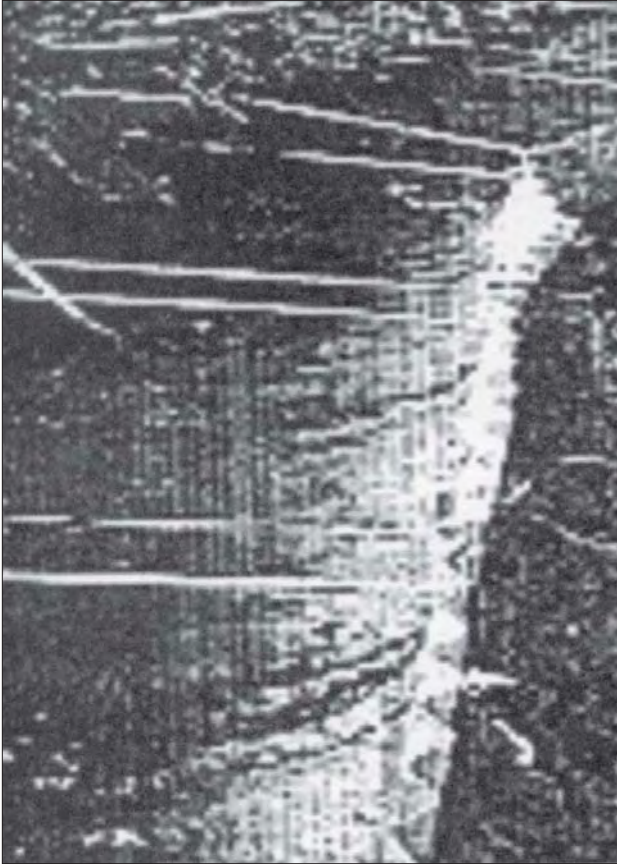
Photo 4.5 An oil drum irrigation canal, Ecuador.

Photo 4.6 Timber canal with bracing supporting the sides, Thuptenchuling.

Box 4.1 Canals with semi-circular concrete sections

Practical Action has developed semi-circular formers in Peru as a means of reducing costs of materials for concrete canals. These are fabricated from 50 mm x 50 mm x 6 mm steel angles with transverse flats welded at the top to provide stability as shown in the photograph. These are placed at certain intervals as guides for the shape of the canal. When compared to trapezoidal sections designed for similar flows (801/s in this case) and installed adjacent to the semi-circular section, the savings in materials were calculated to be around 20 per cent in cement, sand and gravel. However, labour costs were about 50 per cent higher due to the greater demand of excavating, placing and finishing concrete.
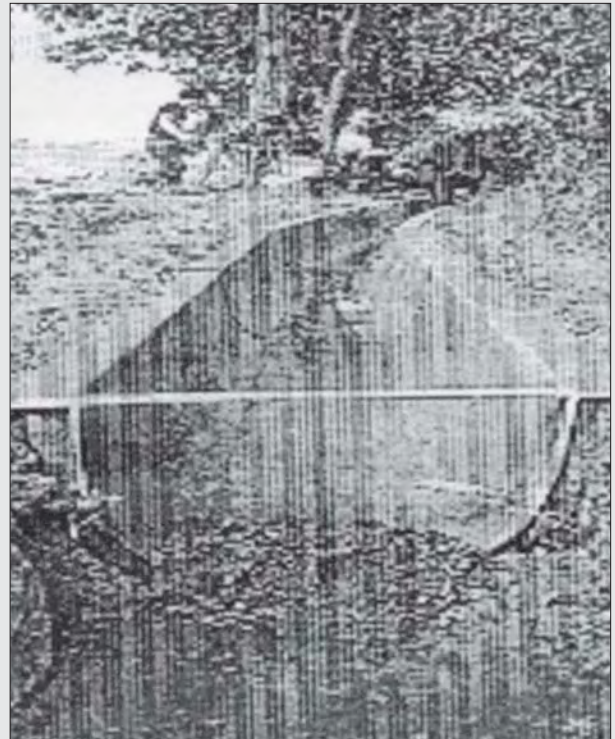
Photo 4.7 A semi- ciruclar concrete canal section, El Tinte, Peru

Box 4.2 The experience with ferrocement in Nepal

Ferrocement pilot projects have been promoted by the *Andhi Khola Irrigation Project (AKIP)* and the International Labour Organisation (ILO). Ferrocement structures are made of thin cement sand mortar (1:2 to 1:3) with thin steel mesh as reinforcement. ILO has used ferrocement for lining irrigation canals in the Sindhuli Flood Rehabilitation Project.

The ILO ferrocement canal cost was US$ 31 per linear metre and the cement masonry design of similar capacity was US$ 28 (1989 prices). The ILO justifies the additional cost by attributing it to better durability and little maintenance which ferrocement canals require. Other advantages are smooth finishing which reduces head losses, resistance to abrasion, and very low seepage losses.

The ferrocement flume used in AKIP (designed by BPC Hydroconsult) is shown in Photograph 4.6 and Figure 4.1. Galvanised sheets with intermediate steel frames were used for the formwork. Multiple layers of 10 mm to 15 mm thick, 1:3 cement sand mortar were placed on the formwork. The final inside layer (i.e. water retaining surface) was prepared using a mix of 1:2 cement sand mortar. Galvanised thin wire mesh (also known as chicken wire mesh) was placed between each layer as reinforcement.

The *Andhi Khola* ferrocement flume has been functioning well since its commissioning in 1993. This design was more economical than the conventional stone masonry in cement mortar canal with drop structures. However, it should be noted that the construction of ferrocement canals requires skilled and well trained manpower (masons) to achieve the required quality of work and therefore may only be justified where a very long canal is to be installed in poor soils. Furthermore, if skilled labour is expensive, ferrocement canals may cost more than the conventional design, as in the case of Sindhuli Flood Rehabilitation Project.

Photo 4.8 Andhi Khola Project ferrocement flume
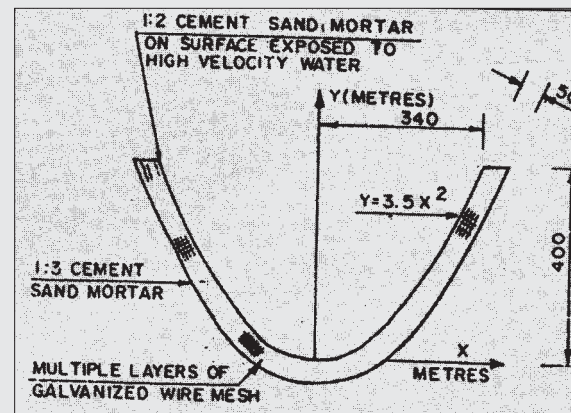
Figure 4.1 Andhi Khola Project ferrocement flume

Box 4.3 Trials of HDPE linings at El Tinte, Peru

The installation of HDPE lining at El Tinte was carried out in a trapezoidal excavation of 1.2 m width. The material was laid in a single sheet of 25 m length in a straight section of the canal as shown in Photograph 4.9. The thickness of the HDPE sheet was 6 mm. The sheet was anchored by excavating 150 mm square anchor trenches parallel to the main canal on each side and then ramming earth into the anchors, as illustrated in Figure 4.2 below.

PVC and butyl/ rubber are some of the membrane types commonly used. Note the differences in this design with the El Tinte HDPE lining. For example, this design has gentler side slopes and the lining is covered with sieved soil, which protects it from being damaged by people and cattle. This type of plastic lining minimises seepage loss but requires more labour time for placing it. Laying the plastic sheet needs careful attention to ensure that it is not punctured.
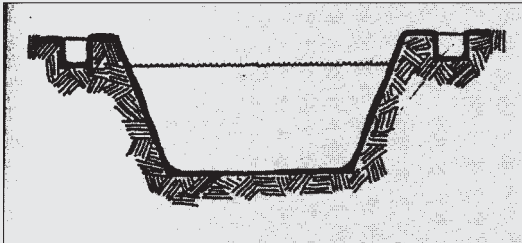
Figure 4.2 HDPE Lining at El Tinte, peru.

The labour required for this operation was one supervisor and one assistant for one day, although a longer section or one involving curves, would have required greater time in jointing and cutting the material.

An analysis of costs shows the HDPE-lined canal to be US$ 10 per m^2 for materials delivered to site, which results in $20 per linear metre of completed canal including labour costs. The additional labour cost for a canal with curves and joints is estimated to be US$ 0.30 per linear metre.

An estimate of the cost of a concrete canal of the same dimensions would be around US$ 12 per linear metre. An alternative buried membrane lining method is proposed in Ref. 5. This is shown below in Figure 4.3. HDPE,

Photo 4.9 HDPE lining at El Tinte, Peru

Layer of 0.10 m gravel on 0.10 m sieved soil laid on formed section

0.30 x 0.30 m anchorage trench

0.20 m thick dry
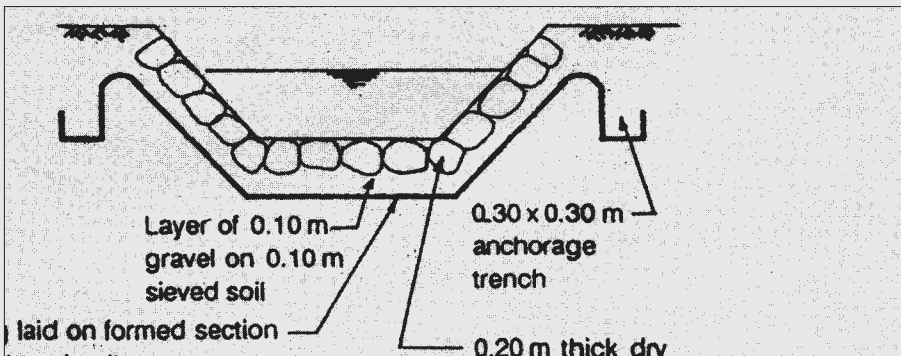
Figure 4.3 Buried membrane lining porposed in design manual's for Irritation Projects in Nepal.

Box 4.4 Soil cement canal in *Andhi Khola*, Nepal

In the *Andhi Khola Irrigation Project (AKIP)* designed by BPC Hydroconsult, soil-cement was tested as an option for irrigation canals. The soil-cement was prepared using a mix of one part cement and one part sand to ten parts of local red coloured clayey silt soil. The red colour of the soil indicates a high iron content, which reacts with cement to form a hard layer on the excavated surface of the canal.

Two applications of soil-cement were tested in 1990 and 1991. The first test used soil which was graded using a 4-mm sieve, with larger lumps and soil broken up with a tamper. After mixing the dry ingredients of cement, sand and soil, water was added and mixed thoroughly until the mortar reached the desired consistency for plastering. The excavated surface of the canal which contained permeable soil and gravel was first made moist by sprinkling water and

of the *Andhi Khola* soil cement canal can be seen in Photograph 4.10.

To date, the performance of the soil-cement lining at *Andhi Khola* has been good. The 1990 test section developed some cracks after a week, apparently due to an excess of water in the mix, which then caused cracking as the soil-cement dried up and hardened. These cracks have not worsened. The 1991 section has not shown any cracking. The lining installed in the steep canal section has also been performing well.

The advantage of this technique is that it is low cost. Between 20% and 40% of the cement required for a conventional concrete mix is replaced by soil. The procedures are easy to learn and are similar to those used for traditional

Photo 4.10 Soil cement lined canal section at Andhi Khola, Nepal.

then the mix was applied firmly to a thickness of 40 mm and packed tightly to eliminate air pockets. The surface was trowelled smooth and then cured for a week.

In 1991, a 15 mm sieve was used to grade the soil over a test section of 140 m². Later more demanding conditions were used for a further test over a section of 25 m length, a gradient of 1:20 and a velocity of approx. 1.3 m/s. A section

houses constructed in the Andhi Khola area. However, one prerequisite for this type of canal is the need for high iron content in the soil used to prepare the mix. Other soils will not perform well. Another conclusion that has been drawn from the Andhi Khola experience is that soil-cement canals are not appropriate for turbulent flows. They are suitable where seepage control is required and the gradient is gentle (velocity limited to ~1.0 m/s.)

Box 4.5 The formers method for lining canals

The method developed by Practical Action in Peru and described here is generally known as the 'formers method' for constructing canals. The formers method permits savings in time and material in constructing concrete channels by reducing the need for pouring concrete into conventional formwork. Precise placement of formers and lines tied between them enable the concrete to be plastered to the insides of the trench and finished with a trowel (see Figure 4.4).

The method involves placing a layer of concrete on the bottom and sides of the canal to form a uniform thickness and a smooth finish. Levelling and finishing the surface is done according to the former.

Procedure

- *Setting out for the formers.* Locate pegs every 10 metres in straight sections and every 5 metres in curved sections, taking into account the slope of the design. It is preferable to use a builder's level to achieve the required precision.
- *Fixing the formers.* Locate formers on each peg at right angles to the centreline of the canal, vertical and exactly in line. They are fixed to the pegs using No. 16 gauge wire and nails, after which intermediate formers are located every 2.5 metres in straight sections, the required slope being checked with a pipe level to give 5 mm drop every 2.50 metres (a slope of 2 in 1000). Each former is checked for line, level, that it is perpendicular to the canal centreline and fixed firmly.
- *Lining the canal.* Prepare a 1:1.5:3 concrete mix
  After making the dry mix, turning the mix a minimum of three times to mix thoroughly, add water, which should have a quantity no greater than one-half of the total weight of the cement (i.e. for mix with 1 kg of cement put ½ litre of water). Next the sides of the canal are plastered and compacted. The pegs are taken out after the finish is completed. Then fine sand is sprinkled with cement to give a mix of 1:3 and a plastering board is used to give a smooth, impermeable finish to the lining. When the sides of the canal are completed the same procedure is followed for the bottom. To finish the edges, care is required to ensure that the formers remain in line. They should be checked using a cord or rule.
- *Extracting the formers.* Formers are taken out after 24 hours in cold climates. To make extraction easy, a layer

of oil or petrol is placed on the formers before carrying out the lining. This also assists with the preservation of the formers. Care should be taken to avoid damage to the edges of the lining when the formers are taken out.

- *Curing the concrete.* To reach the required strength and durability, fresh concrete should be cured. This is achieved by filling the surrounds with water so that
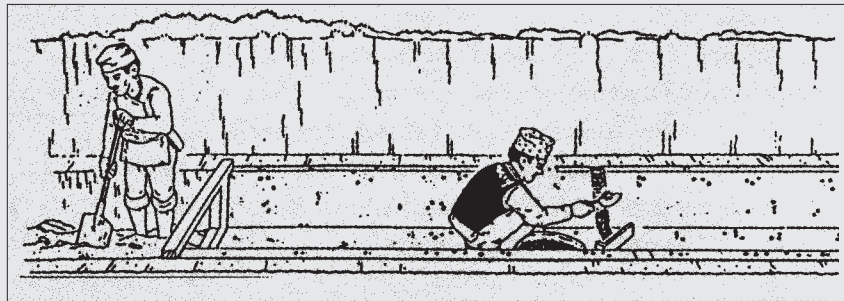
Figure 4.4 Formers method of lining canals.

the linings remain soaked for a period of a minimum of 10 days. This is easy to carry out by locating turns or earth banks at each end, which retain the water. During rainy periods a spillway can be formed to allow excess water to escape, which will also offer a check on the slope. The curing of concrete is very important and should not be overlooked.

- *Expansion joints.* Expansion joints are required in the spaces that are left when the formers are taken out every 2.5 metres in straight sections and variable in curved sections. These permit the concrete to expand and contract without cracking the linings. To fill the joints the following work is required:

a) Clean the joints of debris and unwanted materials with an angular palette whose dimensions are suitable for the width of the joint.

b) Prime the inside surface of the joint with a solution of tar with kerosene in proportions 1:3 so that it has the viscosity of paint. This solution should be applied with a brush.

c) Place a hot mix of tar with fine sand, in proportions of 1 can of tar to 4 cans of sand. First heat the tar and then gradually add the sand while mixing until it has the consistency of black sugar. This mixture is placed first at the sloping sides of the channel and then at the bottom. It is placed in layers and compacted with the angular palette. The finished level of the joint should not exceed the level of the canal lining.

4.3 Canal design

4.3.1 DESIGN CRITERIA

The following criteria are used for the design of headrace canals:

Capacity

The headrace canal should be able to carry the design flow with adequate freeboard. Freeboard is the difference in elevation between the canal bank top and the design water level. During monsoon, the river water level is high and therefore flows higher than the design flow can enter the intake. Spillways and escapes are required to discharge the excess flows. Similarly if falling debris or other obstructions block the canal, the entire flow needs to be safely discharged into a nearby gully or stream before it induces further instability problems.

Velocity

The velocity should be low enough to ensure that the bed and the walls of the canal are not eroded. The recommended maximum velocity for different types of canal is shown in Table 4.1. If the velocity is too low, aquatic plants and moss will start to grow on the canal and reduce the cross sectional area. A minimum velocity of 0.4 m/s should be maintained to prevent the growth of aquatic plants. Also, the velocity in the headrace canal up to the settling basin needs to be high enough to prevent sediment deposition.

Headless and seepage

As mentioned earlier headloss and seepage need to be minimised. Headloss is governed by the canal slope. Seepage can be controlled by choosing the construction materials (earth, mud or cement mortar canals etc.) appropriate for the ground conditions.

Side slopes

Theoretically, the optimum cross sectional shape for a canal is a semi-circle, since it can convey the maximum flow for a given cross sectional area. Since it is difficult to construct a semi-circular canal, in practice, a trapezoidal shape (which is close to a semi-circle) is used. For masonry canals in cement mortar or plain concrete canals that are continuous, rectangular shapes (i.e., vertical walls) are recommended unless the backfill can be well compacted or excavating the required trapezoidal shape is possible. This is because trapezoidal cement masonry and plain concrete canals' side walls will have to depend on the backfill for support. The walls may crack at the canal bed level (causing seepage) since it may be difficult to compact the backfill properly behind the walls, as shown in Figure 4.5. Recommended side slopes for different canal types are shown in Table 4.2.

Stability

Not only should the canal be on stable ground but the areas above and below the alignment also need to be stable. When
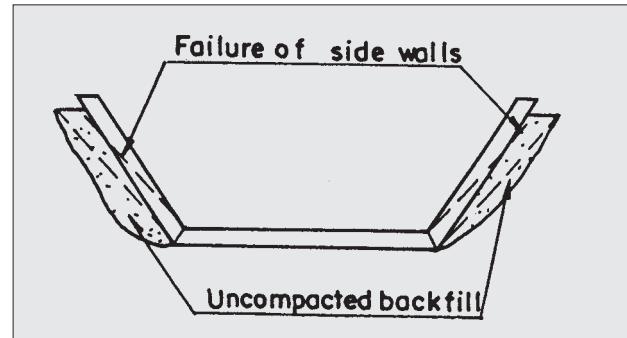
Figure 4.5 Failure of side walls for rigid trapezoidal canals

determining the canal route at site, the signs of stability and instability discussed in Chapter 2 should be referred to.

The canal design should address stability issues such as protection against rockfalls, landslides and storm runoff. Covering canals by placing concrete slabs (or flat stones) and some soil cover (to absorb the impact of falling rocks) can be an appropriate solution if a small length of the canal is vulnerable to rockfalls. Examples of concrete slabs can be seen in the superpassage drawings of the Galkot scheme in Appendix C.

Economics

Similar to any other engineering structure, the design of the canal should be such that the cost is minimised. This is especially important in the case of a long headrace canal since optimising the design will result in substantial saving in the total project cost. Design optimisation or minimising costs requires keeping the canal alignment as short as possible (unless longer lengths are needed to avoid unstable areas and crossings) as well as minimising excavation and the use of construction materials, especially cement and stones. For example, in a micro-hydro scheme, cement masonry canal could be used only at sections where the soil is porous and/or seepage is likely to trigger landslides. In the same scheme, earth and stone masonry in mud mortar canals could be used at sections where problems associated with seepage are not expected. Where the headrace canal constitutes a significant portion of the total project cost, it would be worthwhile to optimise the canal dimensions.

Optimisation of canal may also be worthwhile if the design flow is large, the length is long or expensive canal lining is required. For optimisation of the canal, the least cost method is generally used. A schematic diagram of the canal optimisation process is presented in Figure 4.6. Then cost of 1m long canal for a given design flow and different longitudinal slopes should be calculated. Note that the canal dimensions (depth and width) are primarily dependent on the longitudinal slope and hence accounted once the design flow and the lining type is finalised. Generally the costs involved are: excavation and lining costs. For simple cost comparison only lining cost in case of lined canal and excavation cost in case of unlined canal should is considered

in the example. Then head loss in 1m length of canal for each case should be calculated. The corresponding energy loss (or power loss in most cases of micro-hydro) due to the head loss over a year should be calculated. Then, the cost corresponding to the energy loss should be calculated and the sum of energy or power loss cost discounted over the plant's economic life (generally 15 years) for a discount rate (generally 10 %) is determined. Note that the discount rate is the opportunity cost of investment in the prevailing market and should be greater than or at least equal to the prevailing lending rates for infrastructure projects by the commercial banks of the country. The longitudinal canal slope corresponding to the minimum of sum of canal cost and energy loss cost as shown in Figure 4.6 will be the optimum canal slope which determines the canal size (depth and width).
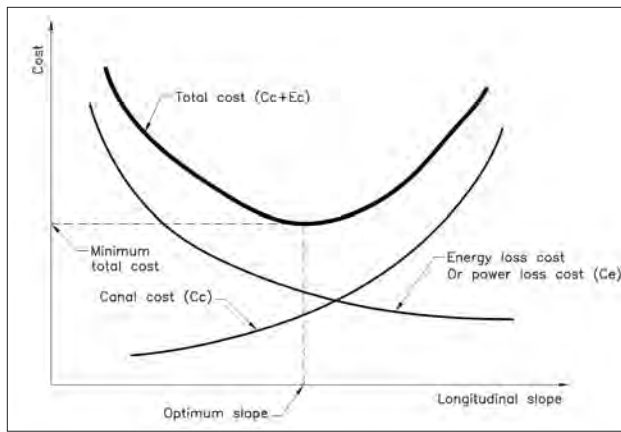
Figure 4.6 Canal optimization

- Canal optimization example

Design flow which was fixed earlier based on hydrology and electricity demand , Q= 300 l/s =0.3 m³/s

Decide canal shape and lining method (according to site condition): Rectangular, stone masonry (1:4) with base slab and side walls, t=300 mm=0.3 m thick and free board, F= 300 mm=0.3 m

Manning's constant from table 4.1, n= 0.02

Assume an appropriate longitudinal slope of canal, S=1/500 (1 in 500) in this case

Select a width to depth ratio, B/D=r=2, (generally 2 is optimum if site condition allows and should be lowered if width is a constraint)

$$ D = \left[\frac{Q \times n \times (r+2)^{2/3}}{r^{5/3} \times S^{1/2}} \right]^{3/8} = \left[\frac{0.3 \times 0.02 \times (2+2)^{2/3}}{2^{5/3} \times (1/500)^{1/2}} \right]^{3/8} = 0.43\text{m} $$

Note that the above equation is derived from Manning's equation. Sizing and quantity calculations can be done similar to this procedure for unlined or trapezoidal canal using Manning's equation.

Considering length of canal, L=1 m
Base slab:
B=r*D=2*0.43=0.86 m
Width, W=B+2*t=0.86+2*0.3=1.46 m
Quantity, V_b=W*t*L=1.46*0.3*1=0.44 m³

Walls:
Wall height, H=D+F=0.43+0.3=0.73 m
Quantity, V_w=2*t*H*L=2*0.3*0.73*1=0.44 m³
Total quantity, V_t=V_b+V_w=0.44+0.44=0.88 m³
Stone masonry rate, say R_m=4000 Rs/m³
Cost of canal, C_c=V_t*R_m=0.88*4000=Rs 3200
Head loss, H_L=L*S=1*1/500=0.002 m
Overall efficiency, e=0.5, generally for micro-hydro
Power loss based on the power equation discussed in Chapter 1, P_L=QgH_Le=0.5*9.81*0.3*0.002=0.00294 kW=2.94 W
Power tariff, R_p =Rs 1.0 per kW/month (say)=NRs 12 per kW/year
Energy/Power loss cost per annum, C=P_L*R_p=2.94*12=Rs 35.28

If the tariff is based on energy i.e., Per kilo Watt Per hour basis, then energy loss (E_L, kWh) all around the year should be calculated considering the plant factor (P) (generally 0.2 to 0.5 for micro hydro). Then energy loss should be multiplied by energy rate (R_e, Rs/kWh) to determine cost of loss over a year (C).Thus, E_L=8760*P*P_L C=E_L*R_e

Assuming the following parameters for the financial analysis:
Plant's economic life N=15 years
Discount rate, i=10%
Discount factor of annuity

$$ DF_A = \frac{(1+i)^N - i}{i \times (1+i)^N} = \frac{(1+10\%)^{15} - 10\%}{10\% \times (1+10\%)^{15}} = 9.76 $$

Therefore power loss cost, C_e=C*DF_A=35.28*9.76= Rs. 344.33

Now, total cost, C_t=C_c+C_e=3200+344.33=Rs. 3544.33 Similarly costs for various canal slopes should be calculated and plotted to find the minimum total cost (as shown in Figure 4.6). The corresponding canal longitudinal slope for the minimum total cost will be the optimum slope.

4.3.2 MANNING'S EQUATION

The design of a headrace canal is based on Manning's equation. Manning's equations for flow and velocity are as follows:

$$ Q = \frac{AR^{2/3}\sqrt{S}}{n} $$

$$ V = \frac{R^{2/3}\sqrt{S}}{n} $$

where:
Q   is the flow in the canal in m3/s
V   is velocity in the canal in m/s

TABLE 4.1 **Roughness coefficient and allowable maximum velocity**
See note below for advice on channels where the water depth is less than one metre.

CHANNEL TYPE	DESCRIPTION	n	MAX. VELOCITY(M/S)
Earth channel	Clay, with stones and sand, after ageing	0.020	0.8
	Gravelly or sandy loams, maintained with minimum vegetation	0.030	0.4
	Lined with coarse stones, maintained with minimum vegetation	0.040	1.0
	For canals less than 1 metre deep, use the equation in Note 4.1 for n e.g.:		
	Vegetated (useful to stabilise soil); water depth 0.7 m	0.050	0.8
	water depth 0.3 m	0.070	0.8
	Heavily overgrown, water depth 0.3 metres	0.150	1.0
Rock cut	Smooth and uniform	0.035	1.5
	Jagged and irregular	0.045	1.5
	Very jagged and irregular	0.060	1.5
Masonry and concrete	Stone masonry in mud mortar, dry stone masonry	0.035	1.0
	Stone masonry in cement mortar using rounded stones	0.030	
	1:4 cement sand mortar		1.5
	1:3 cement sand mortar		2.5
	Stone masonry in cement mortar using split stones (dressed)	0.020	
	1:4 cement sand mortar		2.0
	1:3 cement sand mortar x		3.0
	with 1:2 pointing		5.0
	Concrete (according to finish)	0.013 - 0.017	
	1:3:6 plain concrete		1.5
	1:2:4 plain concrete		2.0
	1:1.5:3 reinforced concrete		3.0
	1:1:2 reinforced concrete		5.0
Cement plaster		0.013	3.0
	1:3		
	1:2		5.0
Wooden canals	Planed, well-jointed boards	0.011	3.0
	Unplaned boards	0.012	3.0
	Older wooden canals	0.015	3.0
Metal canals	All types	0.020	3.0
Mountain streams	Dominant bed material :		
	Gravel (up to 60 mm)	0.03	
	Cobbles (up to 200 mm)	0.04	
	Boulders (up to 600 mm)	0.05	
	Large boulders (> 600 mm)	0.07	

Note *4.1 Roughness effect for shallow channels*

n is the roughness coefficient of the canal (also called Manning's n) which is dependent on the materials of the canal. The value of n for different types of canal is given in Table 4.1.

A is the cross sectional area up to the water surface level in m2.

S is the slope of the energy grade line. The invert slope of the canal is used for S since it is parallel to the energy grade line at longer lengths. For example 1:500 (1 in 500) invert slope is 1 m of drop in level in 500 m of horizontal canal length.

Sometimes percentage (%) or fractions are also used to denote the slopes. For example a slope of 1% means that there will be a difference in level of 1 m every 100 m of horizontal distance. The equivalents of the slope in fractions or decimals are given by the following examples;

2% = 2/100 = 0.02 = 1 in 50
2 in 1000 = 2/1000 = 0.002 = 1 in 500
1.5% = 1.5/100 = 0.015 = 1 in 67
3.5 in 1000 = 3.5/1000 = 0.0035 = 1 in 286

R is the hydraulic radius. R = A/P

P is the wetted perimeter in m. This is the total length of the bottom and the two sides of the canal up to the water surface level.

4.3.3 SEDIMENT DEPOSITION IN CANALS

The velocity in each section of the headrace canal should be high enough to transport any sediment entering that section. Between the intake and the gravel trap a velocity of 1.5 - 2.0 m/s is recommended. Between the gravel trap and the settling basin a lesser velocity is possible, but the sediment transport capability should be checked using a simplified version of Shield's formula: $d = 11RS$

where:

d is the size of particle transported in a canal, in m

R is the hydraulic radius, in m

S is the canal bed slope.

If the gravel trap is designed to settle particles larger than 2 mm, then the canal downstream of the gravel trap must be able to transport particles up to 2 mm.

Research at Wageningen University in the Netherlands demonstrated that the roughness is increased for channels under 1 metre in depth, because of the turbulence created by the side and bed surfaces. The research showed that the following equations can be used to find the roughness coefficient. H is the depth of water.

Well maintained channels with little vegetation:

$n = 0.03/H$ $\sqrt{H} < 1$ m

Channels with short vegetation:

$n = 0.04/H$ $\sqrt{H} < 1$ m

Heavily overgrown channels:

$n = 0.08/H$ $\sqrt{H} < 1$ m

In practice it is sensible to maintain short vegetation in order to protect the banks of canals.

TABLE 4.2 **Recommended side slopes for headrace canals**

CANAL MATERIAL	SIDE SLOPE (N = hfv)
Rock/conglomerate (hard to loose)	0 (vertical) to 0.5
Firm clay	0.25 to 0.5
Loam	1.0 to 1.5
Sandy clay, sandy loam	1.5 to 2.0
Silty sand, sandy earth	2.0 to 2.5
Loose sandy earth, porous earth	2.5 to 3
Gravely earth, stiff or loose conglomerate	0.5 to 1
Gravel and boulder mixed with earth (soft and loose)	1.5 to 2
Stone masonry in mud mortar	See Note 2
Stone masonry in cement mortar	See Note 2
Plain concrete	See Note 2

Note :

1 These values are for canals excavated in soil of low moisture content with water table below canal bed. Slopes need to be flattened if these conditions are not achieved.

2 The sides of lined canals may be vertical (designed as retaining walls) or at the slope recommended for the underlying soil.

4.3.4 DESIGN PROCEDURE

The headrace canal design procedure is as follows:

1. Decide on canal type as per site conditions (e.g. earth canal, stone masonry in mud mortar or stone masonry in cement mortar).

2. Choose a suitable velocity (V) such that it is less than the maximum velocity given in Table 4.1. Note that unacceptable headloss may result if chosen velocities are close to maximum velocity. Also choose the corresponding roughness coefficient (n) from Table 4.1. Then calculate cross-sectional area (A) from the following equation:
   $A = Q/V$

3. Using Table 4.2 decide on the side slope (N). Note that N is the ratio of the horizontal length divided by the vertical height of the side wall (i.e., N = h/v as shown in Figure 4.6).

   $X = 2\sqrt{(1+N^2)} - 2N$

   $H = \sqrt{\dfrac{A}{X+N}}$

   $B = HX$

   $T = B + (2HN)$

4. Calculate the optimum canal height (H), canal bed width (B), and the canal top width (T) using the following equations:
   Note that in case of a rectangular canal, N = 0 and X = 2, so:
   $H = A/2$ and $T = B = 2xH$

Hence, for a rectangular canal the hydraulically optimum shape is when the width is twice the height. These symbols are schematically shown in Figure 4.6.

If an optimum canal shape is not possible due to site specific conditions (such as narrow width along a cliff) then either the width or the height should be selected to suit the conditions. Then the other dimension can be calculated.

5. To ensure stable and uniform flow in a the velocity must be less than 80% of the " critical velocity limit" $V_c = \sqrt{Ag/2}$ where V_c is the critical velocity. Note that for a rectangular canal $V_c = \sqrt{Hg}$

6. Calculate the wetted perimeter (P) using the following equation
   $P = B + 2H\sqrt{(1 + N^2)}$ note that for a rectangular canal,
   $P = B + 2H$

7. Calculate the hydraulic radius (R) as follows: $R = A/P$

8. The slope (S) can now be found from Manning's equation:
   $s = (nV/R^{0.667})^2$
   Now all dimensions required for the construction of the canal are known.

9. Headless = L S (L is the length of the canal section). Sometimes S is fixed by the canal route, which has already been decided and surveyed. Another example of fixed slope (S) situation is when an existing irrigation canal is proposed to be used for a micro-hydro scheme (and higher flows as well as less leakage are required). In such situations different cross sectional area should be assumed (i.e., trial and error) such that the velocity is less than the allowable maximum velocity for the design flow and the type of canal proposed. This can be done by rewriting Manning's equation as follows:
   $$Q = \frac{(BH + NH^2)^{5/3}\sqrt{S}}{n[\,B + 2H\sqrt{(1+n^2)}\,]^{2/3}}$$
   With a known design flow (Q), select the appropriate side slope (N) according to the type of canal chosen. Then fix either B or H and calculate the other using the above equation. Finally from Table4.1, check that the velocity for the canal type.

10. Calculate the size of the largest particle that will be transported in the canal:
    $d = 11\,R\,S$
    • Water level being above the design level due to obstruction that for a rectangular canal $V = Hg$
    If this is less than the possible size in the canal, repeat the design using a higher velocity.

11. Allow a freeboard as follows:
    300mm for Q<500 1/s
    400mm for 500 1/s < Q < 1000 1/s (such flows are unusual for micro-hydro shcemes).
    Such freeboard allows for:
    ■ Uncertainties in the design (e.g. the value of *n* may differ by 5% to 10% from estimate).
    ■ Water level being above the design level due to obstruction in the canal or during emergencies.
    ■ Deterioration of the canal embankment.

12. Check that possible flood flow in canal can be accommodated without using more than 50% of the freeboard.

13. Find the total head loss. If this is too high or too small, repeat the calculations with a different velocity. Consider using different types of canal keeping the overall cost in mind.

Avoid a canal width of less than 300 mm as narrow canals can be easily blocked. Also for stone masonary canals, smaller sizes are difficult to construct.

Example 4.1 Headrace sizing for the *Galkot* **MHP**

The existing irrigation canal at *Galkot* needs to be modified as a headrace canal for a power output of 50 kw. The existing irrigation canal's command area is 20 hectares. The community has requested that the canal be sized such that it would be possible to irrigate the fields and produce 50 sectional areas should be assumed (i.e. trial and error) kW simultaneously.

The following information was collected through site investigation and detailed survey:
Gross head (h) = 22 m (forebay to powerhouse)
Intake to 130 m downstream: 1:50 slope (S) with one drop structure.
131 m to 231 m downstream: 1:92 slope.
232 m to 405 m downstream: 1:365 slope.
406 m to 730 m downstream: 1:975 slope with 3 crossings.
731 m to 1119 m downstream (forebay): 1:400 slope with one crossing.
Site conditions dictate that the entire headrace alignment be constructed out of stone masonry in cement mortar.
Note that 1 m gap has been provided at change of slopes. This is the transitional length that connects the two different slopes.
In this example, the required design flow will be calculated and the headrace canal from chainage 406 m to 730 m will be sized.

Design flow calculations:
Assume 55% overall efficiency (e_o = 0.55)

$\quad\quad$ P $\quad$ = Q g h e_o (power equation)
$\quad\quad$ Q $\quad$ = P/(ghe_o)
$\quad\quad\quad\quad$ = 50/(9.8 x 22 x 0.55) = 0.421 m³/s
Therefore flow required for power generation is 4211/s
Assume an irrigation requirement of 1.5 l/s/ha for existing irrigated land.
Irrigation demand (Q) $\quad$ = 1.5 l/s/ha x 20 ha
$\quad\quad\quad\quad\quad\quad\quad$ = 30 l/s
Total design flow for the headrace canal = 421 l/s + 30 l/s = 451 l/s. **Therefore use a design flow of 455 l/s to size the headrace canal.**

Canal sizing

Canal type: stone masonry in cement mortar
n = 0.020 for dressed stone masonry (from Table 4.1)
Q = 0.455 m³/s
s = 1/975
From Table 4.2 choose N = 0.5 (lh/2v)
Set the bottom width (B) = 0.450 m which is the size of the existing irrigation canal. This minimises excavation works.
Now use the following form of the Manning's equation where only the water depth, H, is unknown:

$$Q = \frac{(BH+NH^2)^{5/3} \ \sqrt{s}}{n[(B + 2H \sqrt{(1+N^2)}]^{2/3}}$$

$$0.455 = \frac{(0.450H+0.5H^2)^{5/3} + \sqrt{1/975}}{0.020[0.450+2H \sqrt{(1+0.5^2)} \ ]^{2/3}}$$

By trial and error method, the above equation is balanced when H = 0.768 m for a flow of 4551/s.
Therefore, the water depth will be about 768 mm. Now check that the velocity is less than the maximum allowable velocity of 2.0 m/s from Table 4.1.
$\quad$ V = Q/A
or V = 0.455 / (BH+NH²)
or V = 0.455 / (0.450x0.768+0.5x0.768²)
or V = 0.7 m/s < 2.0 m/s OK.

The drawing and dimensions for this canal section can be seen in Drawing 420/04/2A01 (Canal type B) of Appendix C. Note that the original design was based on an assumed value of 0.017 for Manning's n, giving a water depth of 705 mm, therefore actual freeboard may be less than recommended. The other canal sections of this scheme can be verified by similar calculations.

Note that when the canal alignment has already been fixed (i.e, fixed canal bed slope, S) as in the case of the *Galkot* scheme, there is little control in the velocity. The velocity can be slightly modified by changing the cross sectional area but it will be difficult to make significant changes. For example, the velocity in the 1/92 slope canal section of the *Galkot* scheme is about 2 m/s. However, this was found to be more economical (i.e., reshaping the existing canal) than realigning the canal alignment to reduce the slope. This implies that the steeper sections of the canal may require more maintenance work than the gentler sections. This is one reason for plastering the inside surfaces of the canal.

Also, note that earth irrigation canals are generally steeper since 'n' (roughness coefficient) is higher. When such canals are modified as headrace, they become smoother (use of cement mortar, plaster, better shape, etc.) and hence can have higher velocities.

3.4 Spillways

3.4.1 LOCATION OF SPILLWAYS

As mentioned earlier, spillways are required in headrace canals to spill excess flows during the monsoon and in case of obstruction in the canals. Similarly, spillways are also required at the forebay to spill the entire design flow in case of sudden valve closure at the powerhouse .

The excess flows that are discharged via a spillway should be safely diverted into the stream or nearby gully such that they do not cause any erosion or damage to other structures. Sometimes, this may require the construction of a channel to the natural water course. Locating spillways close to a gully will save the cost of channel construction as can be seen in Photographs 4.11 and 4.12. .

4.4.2 SPILLWAY DESIGN

Where water is ponding at a downstream regulator such as in a forebay, the design of spillways can be based on the weir equation discussed in Chapter 3.

$$Q_{spillway} = C_w L_{spillway} (h_{overtop})^{1.5}$$

where:

$Q_{spillway}$ = discharge over the spillway in m^3/s
$L_{spillway}$ = length of the spillway in m
$h_{overtop}$ = head over the spillway in m (i.e. height of water over the spillway)
C_w = a coefficient (similar to weir coefficient)

which varies according to the spillway profile. C_w for different weir profiles is shown in Table 3.3 (Chapter 3).

The design steps are as follows:
- Calculate the flow through the intake during floods as

Photo 4.11 Overflow from the forebay over a rock cliff, Dhading micro-hydro Nepal.

Photo 4.12 Spillway on a crossing where the excess flow is discharged into a gully, Galkot micro-hydro scheme, Baglung, Nepal.

discussed in Chapter 3. The spillway should be sized such that the entire flood flow can be diverted away from the canal. This is because the micro-hydro system could be closed during flood or there could be an obstruction in the canal.

- Choose a spillway profile and determine C_w. In the Nepalese context, a broad, round edged profile ($C_w = 1.6$) is suitable since it is easy to construct.

- Spillway crest level should be 0.05 m above normal canal water level. No more than 50% of the freeboard should be used. Therefore, with a generally used freeboard of 300 mm, the available $h_{overtop}$ is 0.5 x 030 - 0.05 = 0.10 m. The

required length can then be calculated for the chosen $h_{overtop}$ and flood flow.

Where there is no ponding immediately downstream, such as in the headrace canal, the spillway length calculated using the weir equation should be multiplied by 2: this accounts for the gradual decrease in head over the spillway, until the required level is reached at the downstream end of the spillway. In this case only the excess flow ($Q_{flood} - Q_{design}$) should be used for ($Q_{spillway}$) Note that in such cases, locating the spillway immediately upstream of an orifice will increase the flow through the weir. The design of a spillway is presented in Example 4.2.

Example 4.2 Design of a headrace canal and a spillway

Design a headrace canal to convey a flow of 285 l/s. Site conditions indicate that the canal would be stable if stone masonry in mud mortar is used. The expected flow through the intake during a 20-year return flood is about 480 l/s. Design an adequate spillway.

Design procedure:
Canal type: stone masonry in mud mortar

Q = 0.285 m³/s

From Table 4.1:

Roughness coefficient n = 0.035

choose V = 1.0 m/s

From Table 4.2, for gravelly earth, select side slope, N = 0.5, (1h/2v).

Cross sectional area, A = 0.285/1.0 = 0.285 m²

$X = 2\sqrt{(1+N^2)} - 2N$

$X = 2\sqrt{(1+0.5^2)} - 2 \times 0.5$

X = 1.236

Calculate the water depth in the canal, H:

$$H = \sqrt{\frac{A}{(X+N)}}$$

$$H = \sqrt{\frac{0.285}{1.236+0.5}}$$

H = 0.405 m   Calculate the bed width,   B = HX

B = 0.405 x 1.236

B = 0.50 m

Calculate the top width up to the design water level, T:

T = B + (2HN)

T = 0.50 + (2 x 0.405 x 0.5)

T = 0.905 m

Check if V< 0.8 V_c

$V_c = \sqrt{Ag/T} = \sqrt{0.285 \times 9.8 / 0.905}$

V_c = 1.76m/s

0.8 V_c = 1.41 m/s > V = 1.0 m/s OK

Calculate the wetted perimeter, P:

$P = B + 2xH\sqrt{(1+N^2)}$

$P = 0.5 + 2 \times 0.405\sqrt{(1+0.5^2)}$

P = 1.406m

Calculate the hydraulic radius, R:

R = A/P = 0285/1.406 = 0.230 m

Figure 4.7 Proposed internal canal dimensions for Example 4.1

Calculate the required canal bed slope, S:

$S = \sqrt{\left(\dfrac{nV}{R^{0.667}}\right)2}$

$S = (0.035 \times 1/0.203^{0.667})^2$

S = 0.0103 or 1:97 (i.e. 1 m of drop in 97 m of horizontal canal length)

Finally allow 300 mm of freeboard. The canal dimensions can be seen in Figure 4.7.

Check the flow depth for maximum flood flow in the canal.

$$Q = \frac{(BH+NH^2)^{5/3}\sqrt{S}}{n[(B + 2H\sqrt{(1+N^2)}]^{2/3}}$$

$$0.480 = \frac{(0.5H+0.5^2)^{5/3} \times \sqrt{0.0103}}{0035[(0.5+2H\sqrt{(1+0.5^2)}]^{2/3}}$$

By trial and error method, the above equation is balanced when H = 0.55 m. Therefore, the flood flow occupies 50% of the freeboard (the maximum allowed, as discussed earlier) and the head on the spillway ($h_{overtop}$) will be 100 mm.

Check the size of particle that will settle in the canal at a velocity of 1.0 m/s.

D   = 11RS

      = 11 x 0.203 x 0.0103

      = 23 mm

i.e., particles larger than 23 mm would settle in this headrace canal. Therefore, to avoid deposition upstream of the settling basin, the gravel trap must be designed to remove all particles greater than 23 mm.

Design of spillway

Note that 2 conditions need to be checked as follows:

1. The spillway must be able to convey the entire flood flow of 48 l/s in case the headrace canal downstream gets obstructed (ponding case).

2. The spillway should be able to spill the excess flow (480 l/s - 285 l/s) when there is no obstruction downstream.

The calculated maximum spillway length should be used in the design.

Calculations

Choose a broad crested weir with round edges profile, so $C_w = 1.6$

case 1:

$Q_{spillway}$ = 480 l/s
$h_{overtop}$ = 100 mm calculated earlier.

Now calculate the length of the spillway,

$$L_{spillway} = Q_{spillway} / C_w x (h_{overtop})^{1.5}$$

$$L_{spillway} = 0.480/1.6 x (0.1)^{1.5} = 9.5 \text{ m}$$

Case 2:

$$Q_{spillway} = 480-285 = 0.1951/s$$

$$L_{spillway} = 2Q_{spillway} / C_w x (h_{overtop})^{1.5}$$

$$L_{spillway} = 2x0.195 /1.6 x (0.1)^{1.5} = 7.7m$$

Therefore a spillway length of 9.5 m is required for the above canal (Case1)

4.5 Crossings

Sometimes the headrace or the penstock alignment may need to cross gullies and small streams. Crossings are such structures that convey the flow over streams, gullies or across unstable terrain subject to landslides and erosion.

The Galkot crossing with a spillway was shown in Photograph 4.10. This is a 1.2 m long aqueduct that is constructed from

Photo 4.13 HDPE pipes provide an overflow from a timber channel (Mhapung)

reinforced concrete. Its size and slope are similar to the upstream headrace canal. In micro-hydro schemes, reinforced concrete crossings may be feasible if the length is short. Such structures are expensive and complicated for longer lengths.

The Jhankre mini-hydro penstock crossing can be seen in Photograph 4.14. In this case the penstock alignment had to traverse a 12.6 m wide gully. This gully is active only in the monsoon and at other times it is dry. A series of masonry walls were designed to support the penstock (similar to the support piers) along the gully. All of these walls rest on a continuous foundation pad. During the monsoon, the surface runoff flows between the walls.

Photograph 4.15 shows the Ghandruk crossing. The 50 kW Ghandruk micro-hydro scheme has a long HDPE headrace pipe (see Box 4.6) and at one location, the alignment had to cross a gully. As can be seen in the photograph, a mild steel pipe was used for the crossing with another vertical pipe supporting it.

Apart from the types of crossings discussed above, inverted siphons are also sometimes used across gullies. Inverted

Photo 4.14 Jhankre mini-hydro penstock crossing, Dolakha, Nepal.

Photo 4.15 Jhankre mini-hydro headrace crossing Ghandruk, Nepal.

siphons are pipes that are buried across the gully. They traverse down to the lowest point of the gully and then come up at the other side (hence the name inverted siphon). As long as there is sufficient head and the pipe is below the hydraulic grade line, the flow can be conveyed through such siphons. A flush out valve must be incorporated at the low point of the siphon (since sediment can be deposited at the low point). Such siphons can also reduce costs of crossings as well as risks due possible damages from floods. Sufficient protection against scour and damages due to rolling boulders should be provided for inverted siphons. This can be achieved by encasing the pipe in concrete (generally nominally reinforced) along with stone pitching on the top of the pipe as shown in Figure 4.8. Inverted siphons across larger streams will also require some bank protection structures based on site conditions. A typical section of an inverted siphon under a stream or a gulley is shown in Figure 4.8.

4.6 Headrace pipe

4.6.1 GENERAL

Pipes may be required along the headrace alignment where slopes are unstable and where landslides may occur. Although masonry and concrete canals can minimise seepage induced landslides, they are rigid structures and in the event of slope failures, such canals can be swept away. These canals will also crack if there are small slope movements. Where soil instability problems are expected, flexible pipes may be an appropriate solution provided that the required pipe length is not too long (see Box 4.6). Another case for the use of flexible pipes is when the entire hillside is slowly sliding (i.e. mass movement is occurring) and part of the headrace alignment needs to traverse it.

In Nepal HDPE pipes are often used to address the above problems. These pipes are flexible enough to accommodate some ground movement and can be joined by heat welding, which is described in Box 4.7. HDPE pipes should be buried to protect them from sunlight, cattle and vandalism.

The reason why PVC pipes have not been used for headrace in Nepal is because although they are easy to join (with a PVC cement solution), they are also very rigid. Therefore, they cannot accommodate ground movement.

Appendix B includes data on standard pipe sizes available in Nepal.

4.6.2 DESIGN CRITERIA

The design criteria for headrace pipes are similar to those of headrace canals. Specifically, the design should address the following issues:

- The pipe diameter should be such that for the ground slope of the alignment, it should be able to convey the design flow. If there is a possibility of flood flows entering into the pipe, make provision for spilling such excess flows.
- The inlet to each section of headrace pipe should be protected with a trashrack, so that debris does not get in and block the pipe. The spacing of the trashrack bars should be no more than one third of the pipe diameter, and the velocity through the trashrack should not exceed 1 m/s.
- Where a section of headrace pipe ends in an unlined canal, a masonry transition structure is recommended, to avoid scour by the high velocity flow.
- Headrace pipes are efficient when they are flowing full, but if the head on the pipe exceeds the rated pipe head (i.e. allowable head on the pipe) break pressure tanks need to be provided. Such tanks dissipate the head over the pipe and avoid the need to use a higher pipe rating. However, in practice, repeated use of break pressure tanks has sometimes induced cyclic surge (i.e. periodic change in head and hence the flow). Another option in such cases is to select a larger pipe diameter such that open flow condition prevails. Break pressure tanks should be provided with lockable covers, so

that debris cannot get in and block the pipe.

- As far as possible, the pipe alignment should be such that it is always sloping downhill. This ensures that there is always a positive head over the pipe and the chance of it being blocked is also reduced.
- If there is a need for inverted siphons (or the pipe needs to go uphill for some length due to the ground profile), air release valves should be provided at high points along the alignment. Similarly, flush valves should also be provided at low points to flush sediment from the pipes and hence prevent them from being clogged.

Note that the setting out and preparation of the bench for headrace pipe is similar to the headrace canal discussed in Section 4.4. As mentioned earlier, HDPE pipes should always be buried. A minimum buried depth of 1 m with sieved soil 150 mm to 300 mm around the pipe is recommended as shown in Figure 4.8. The use of sieved soil ensures that the pipe is not punctured by pointed rocks during compaction, distributes the loads evenly and prevents future differential settlements above the pipe. The 1 m depth minimises the overburden loads over the pipe such as when people or cattle walk over it. Also, in areas where freezing is expected during mid-winter, 1 m is usually sufficient to be below the frost line.

At inlet and outlet sections of a headrace pipe, it is recommended to provide inlet and outlet structures of stone masonry or concrete.

Photo 4.16 Break-pressure tank, with ice around the walls (Jhong)

Photo 4.17 Flush-outs should be provided at low points in pipelines so that heavy debris can be periodically removed (Siklis)

Box 4.6   HDPE headrace pipes, the Ghandruk experience

The 50 kW Ghandruk micro-hydro scheme was one of the first micro-hydropower projects that Practical Action Nepal was involved in. HDPE pipe has been used successfully for the long headrace through forest, but lessons should be learned from the problems experienced:

- Sticks and leaves entering the pipe at the headworks get wedged at the weld beads, causing pipe blockage.
- Vandals throwing stones into the break pressure tanks.
- Pipe collapse due to negative pressure at a high point (where the pipe is below the hydraulic grade line).
- Surging flow due to air being drawn into the pipe at break pressure tanks.

At one short location, the hillside was not very stable and the HDPE pipe has been supported by galvanised wires tied to trees as can be seen in Photograph 4.18. Also notice that the HDPE pipe can be bent when the bend radius is large. However, it would have been technically sounder if a gabion wall had been built downhill of the pipe alignment and the pipe covered with soil as shown in Figure 4.8 Photograph 4.19 shows a mitred bend on the Ghandruk HDPE headrace pipe. This was made by cutting pipe sections at an angle and them by heat welding. It has started leaking at the bend and the villagers have wrapped it with plastic sheets and galvanised wire. Bends that are constructed by cutting and welding pipe sections require care during the joining process (i.e., the more joints, the higher the likelihood of leakage). If there is some head over the headrace pipe, then there can be significant forces at the bend as discussed in Chapter 7. Such forces can weaken the joints and cause leakage. Also note that the pipe section shown in the photograph should have been buried.

Photo 4.18 HDPE headrace pipe along unstable alignment, Ghandruk micro-hydro scheme, Nepal.

Photo 4.19 Bend prepared by cutting and welding the HDPE headrace pipe at Ghandruk

Box 4.7   Joining of HDPE pipes by heat welding

HDPE pipes are available in the market in fixed lengths (e.g. 4-6 m pieces) and need to be joined at site. Unlike PVC pipes, there is not a liquid solution that can be used to join HDPE pipes. The only economical method of joining these pipes is by heat welding them. This involves heating the ends (that need to be joined) such that they become soft and malleable and then joining them by applying force from close to both ends of the pipes. This joining temperature is reached at about 200℃ The following steps are recommended when joining HDPE pipes at site:

- First heat the welding plate until the required temperature is reached. The welding plate (also known as the heating plate) is a mild steel disc with a rod welded at the edge and a wooden grip at the end of the rod. Heating the welding plate can be done by either using a kerosene burner (as shown in Photograph 4.20) or by heating the plate over a charcoal fire. A special chalk called thermo-crayon can be used to ensure that the plate has reached the required joining temperature. A few lines should be marked on the plate while it is

Photo 4.20 Heating the welding plate

Photo 4.21 Joining HDPE pipes by pushing them while hot. Notice the Teflon bag.

being heated. When the plate reaches the joining temperature the chalk colour turns blue to black within one second.

- The welding plate should then be removed and placed inside a Teflon bag (the bag can be made by stapling Teflon fabric). The Teflon bag ensures that the heated HDPE pipe ends do not stick to the heating plate and distort the shape of the pipe. Teflon is a special fabric that can withstand higher temperature.
- Then with the heating plate inside the Teflon bag, the pipes should be pushed together until there is a uniform bead around the outside joint surface. The heating plate along with bag should then be removed and the pipes quickly pushed against each other. This requires at least three people (one to hold the plate and two to push the pipes.) as shown in Photograph 4.21. Once the plate is

placed between the pipes, the entire process should be completed within 3-5 minutes since the plate temperature will start decreasing. One problem in this method is that the two pipes may not be straight since it will be difficult to apply uniform forces around the pipe circumference manually. An alternative is to use collar flanges as shown in Photograph 4.22 These flanges are made in two halves such that they fit on the outside circumference of the pipe. In this method, the collars are fitted about 50 mm to 100 mm from the heating ends of the pipes, before inserting the heating plate. As soon as the required temperature is met, the heating plate is taken out, and bolts are inserted along the flanges and tightened evenly. This ensures that the two pipes are straight. Once the joint cools, the collars are removed by unbolting them. As an alternative to the above methods, a mechanical jig is commonly used, which serves both to align the pipe ends and to apply the required joining force. The pipe manufacturers can advise the required force for different pipe diameters and grades.

Refer to Chapter 10 (Innovations) Section 10.3 concerning a de-beader tool to remove the circumferential bead caused by heat welding.
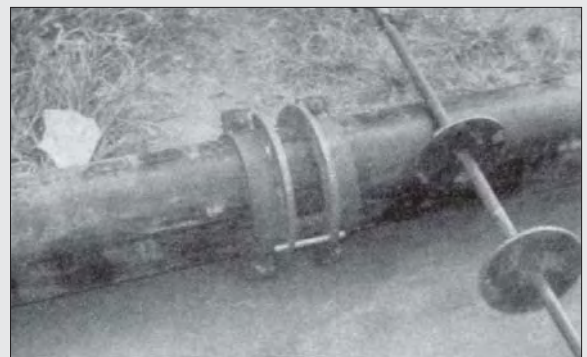
Photo 4.22 Collars used to join HDPE pipes.

General backfill compacted in 250 mm layers

1000 min

300 min

150 min

Sieved backfill (maximum size 5mm) compacted in 150 mm layers

Pipe

150 min.

Drystone masonry to retain backfill

General backfill

Sieved backfill

Pipe

1000 min.

300 min

150 min.

150 min.

150 min.
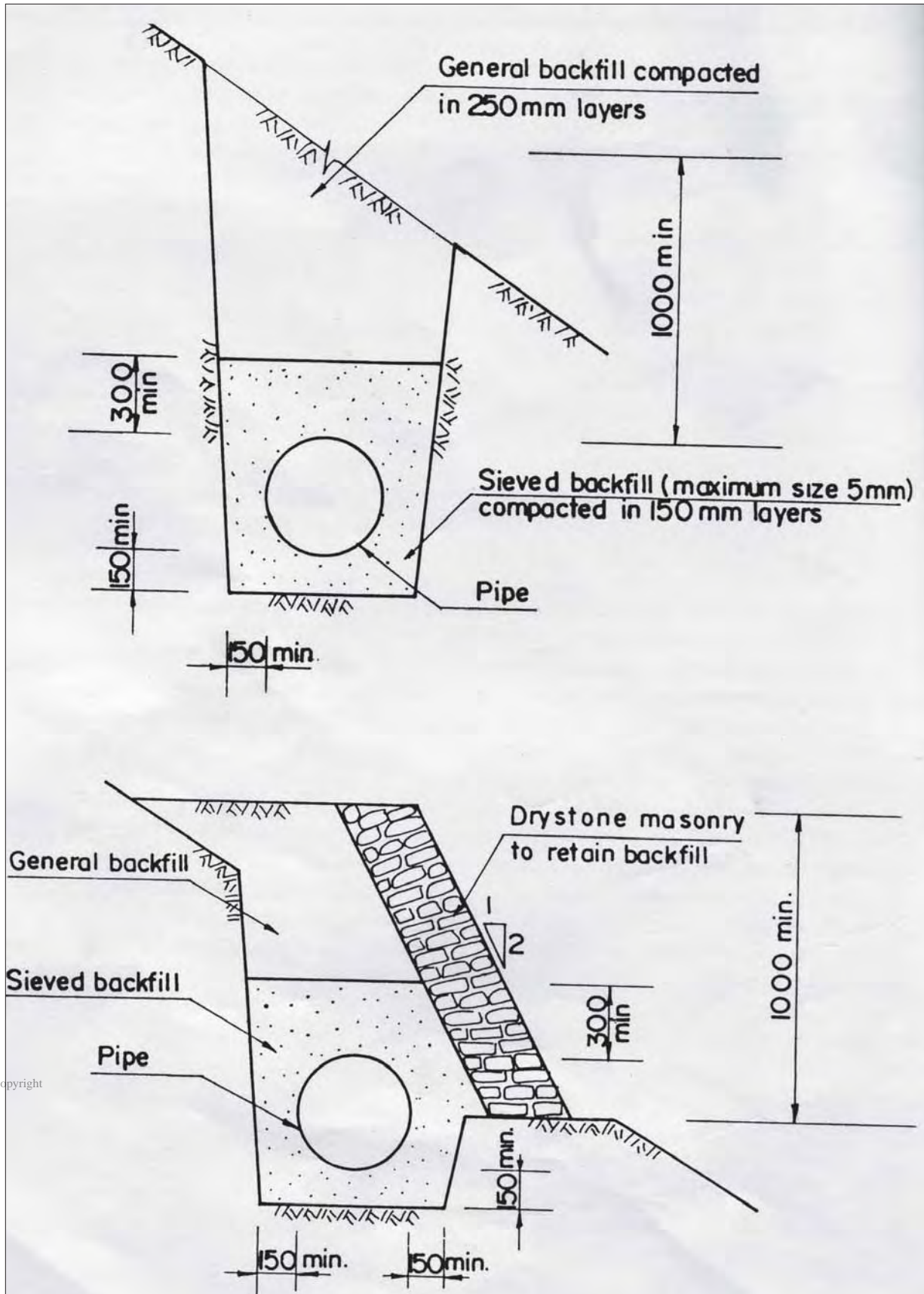
Figure 4.8 Typical pipe burial details

4.6.3 DESIGN PROCEDURE

The design procedure (i.e. selection of an appropriate pipe diameter) for a headrace pipe is as follows:

1. Choose a standard pipe size from Appendix B, such that the velocity V is less than 3 m/s (to minimise wall abrasion and to avoid excessive headloss) and greater than 0.6 m/s (to avoid sediment being deposited in the pipe). In general, for HDPE pipes a velocity of 2.5 m/s to 3.0 m/s is found to be economical.

2. Calculate the actual velocity:
   $V = 4Q / \Pi d^2$
   where:
   V is velocity in m/s
   Q is design flow in m³/s
   d is the pipe internal diameter in m.

3. At the entrance of the headrace pipe set the submergence head as follows:
   $h_s \geq 1.5 V^2 / 2g$
   where h_s is the submergence head in m as shown in Figure 4.9. Note that this is the head from the crown of the pipe. If the submergence head is less than required, then the pipe will not be able to convey the design flow (Q) because air will be drawn into the pipe.

4. Calculate the headloss in the pipe length based on the inlet, wall friction, bends, valves and exit losses as follows:
   Total head loss = wall loss + turbulence losses

   The wall losses result from the friction between the flow and the pipe wall. Wall losses are calculated as follows:

First determine the roughness value, k in mm from Table 4.3. Note that the values of k in this table are based on normal age (5-15 years) or condition.

Then use the Moody Chart in Figure 4.10 to find the corresponding friction factor f for the selected pipe material, diameter and the design flow.
The wall loss can now be calculated from the following equation:

$$h_{wall\ loss} = f\ (LV^2 / dx2g)$$

In terms of the flow, diameter and length, this equation can also be rewritten as:
$$h_{wall\ loss} = fLQ^2 / 12d^5$$

Turbulence losses are calculated as follows:
$$h_{turb\ loss} = V^2 / 2g\ (K_{entrance} + K_{bend} + K_{contraction} + K_{valve})$$

where head loss coefficients, K, are as shown in Table 4.4.

Note that HDPE pipes can be bent (by hand) without causing any damage if the bend radius is at least 50 times the pipe diameter. This should be done wherever possible, because:
a) it avoids the need for mitred bends;
b) it avoids the need for anchor blocks to restrain bend forces (discussed in Chapter 7); and
c) at such large radius, K_{bend} becomes negligible.
Where a long radius bend is not possible, a sharper bend is required, and the value of K_{bend} should be taken from Table 4.4. Mitred bends will normally be used for steel and HDPE pipelines: these are fabricated by cutting the pipe at an angle (maximum 15°) and then welding the ends together to create a bend of up to 30°. For bends of more than 30°, two or more mitre joints are required.
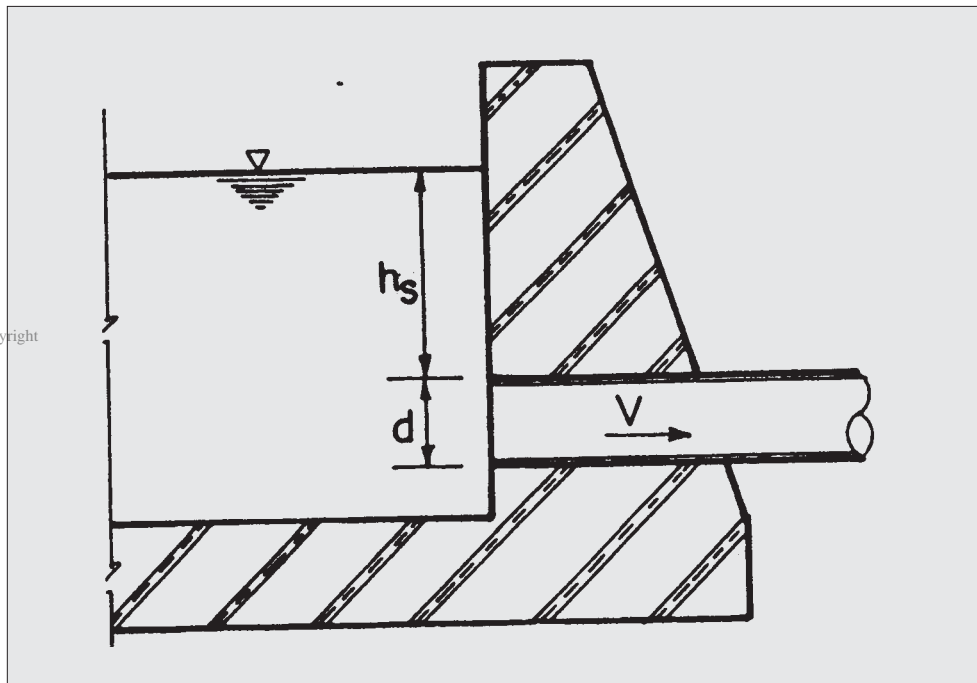
Figure 4.9 Submergence head for a pipe

TABLE 4.3 **Roughness value for different pipe materials**

MATERIAL	ROUGHNESS VALUE, k (mm)
Smooth pipes *PVC.HDPE, MDPE, Glass fibre*	0.06
Concrete	0.15
Mild steel - Uncoated - Galvanised	0.06 0.15

5. Check if the total head loss for the design flow is less than the loss in head due to the pipe gradient (S) and that the pipe profile is below the hydraulic grade line everywhere.

If not, repeat calculation with larger pipe diameter.

6. Determine the water level at the control structure at the end of pipe such as the break pressure tank, gravel trap or the settling basin. Allow 10% margin by assuming that the total head loss is 10% higher than calculated (i.e. water level is 10% lower than calculated). This is to allow for uncertainties such as the wall losses being higher than assumed.

7. Repeat calculations with higher submergence head due to flood flows and calculate the corresponding losses and pipe flow. The excess flow will have to be spilled from a control structure (gravel trap, settling basin etc.) at the end of the pipe.
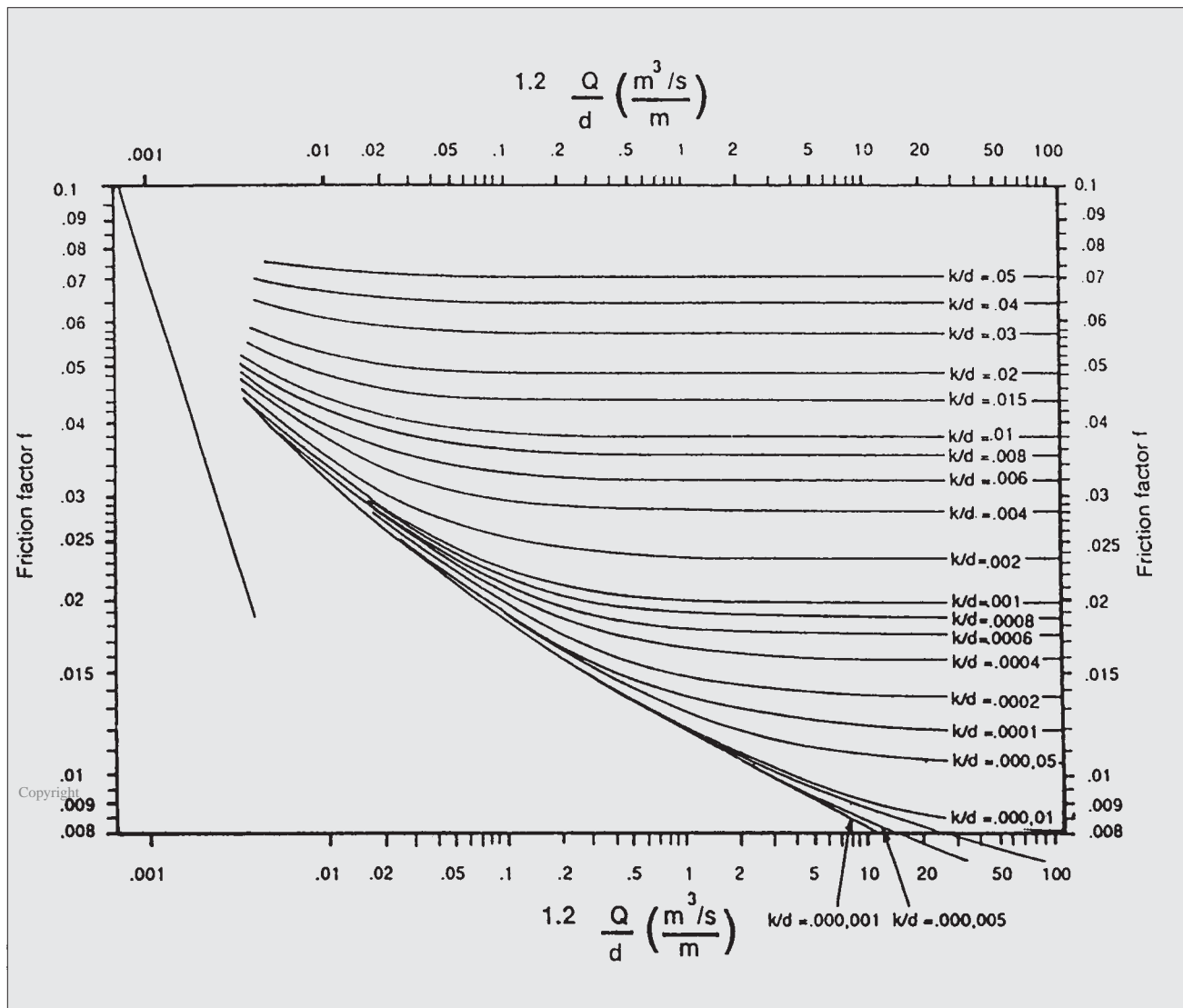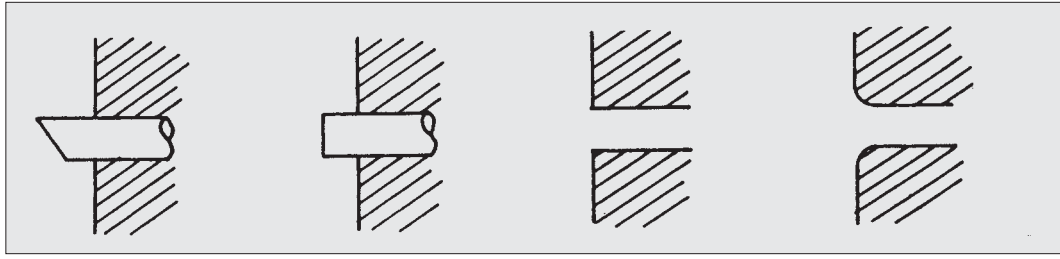
Figure 4.10 Moody chart

TABLE 4.4 Turbulence losses in pipes

Head loss coefficient for intakes ($K_{entrance}$)
Entrance profile

$K_{Entrance}$	1.0	0.8	0.5	0.2

Head loss coefficients for bends (K_{bend})
Bend Profile
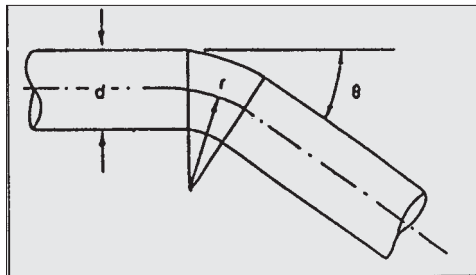
r/d	1	2	3	5	MITRED*
$K_{bend}(\theta = 20°)$	0.20	0.15	0.12	0.10	0.10
$K_{bend}(\theta = 45°)$	0.40	0.30	0.25	0.20	0.22
$K_{bend}(\theta = 90°)$	0.75	0.50	0.25	0.20	0.45

Mitred bends with r/d = 1.5, maximum *30° per mitred joint.*

Head loss coefficients for sudden contractions ($K_{contraction}$)
Contraction profile
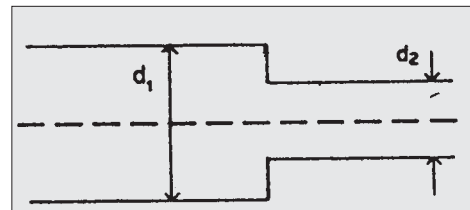
d_1/d_2	1.0	1.5	2.0	2.5	5.0
$K_{contraction}$	0	0.25	0.35	0.40	0.50

Head loss coefficient for valves (K_{valve})

TYPE OF VALVE	SPEHRICAL	GATE	BUTTERFLY
K_{valve}	0	0.1	0.3

Example 4.3 Siklis headworks
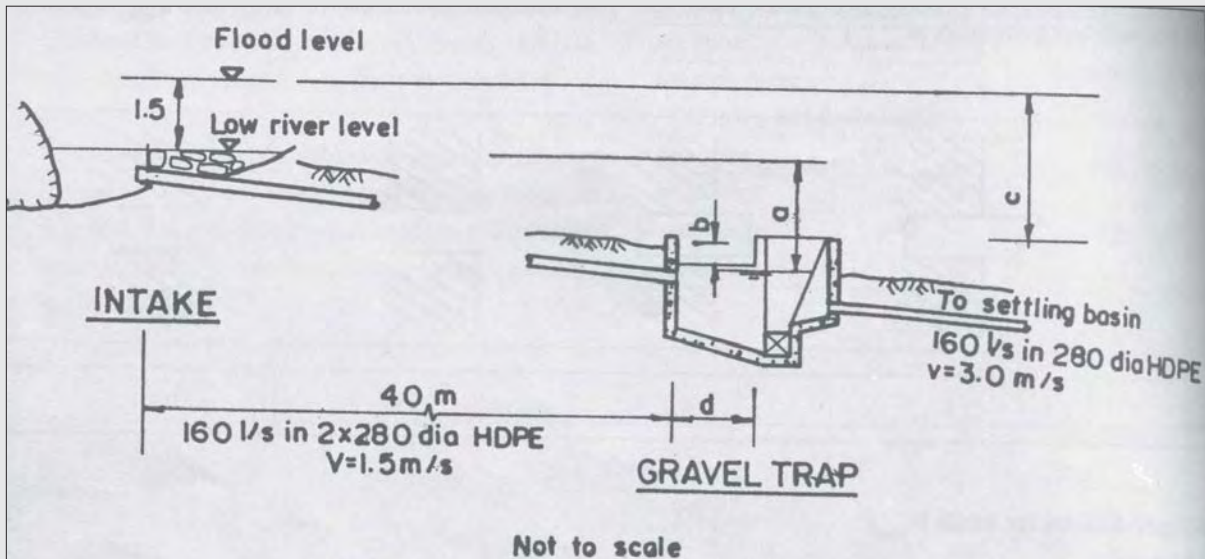
Figure 4.11 Headworks of the 100kw Siklis micro-hydro scheme (Siklis, Nepal)

A sketch of the headworks of the existing 100 kW Siklis micro-hydropower scheme is shown above in Figure 4.11. Calculate the following:

1. Headloss in intake pipes
2. Design water level in gravel trap
3. Flood flow through intake pipes
4. Spillway length at gravel trap

1. Headloss in intake pipes at design flow

a) Pipe friction

For 280 mm diameter. Class II HDPE pipe manufactured in Nepal, internal diameter = 262 mm

Flow per pipe (Q) = 160/2 = 80 1/s

From Table 4.3, assume k = 0.06 mm

   or, k/d = 0.06mm / 262mm = 0.00023

   or, $1.2Q/d$ = 1.2x0.08 / 0.262 =0.366

   f   = 0.0166 (from Moody Chart, Figure 4.10)

   $\therefore h_{wall\ loss} = fLQ^2 / i2d^5$

          = 0.0166x40x.0.08^2 / 12x0.262^5 = **0.29m**

b) Inlet headloss

   $K_{entrance}$ = 0.8 for this case (Table 4.4}

   $h_{inlet\ loss}$ = $K_{entrance}$ x V^2 / 2g

Check velocity;

   V = 4Q / IId^2 = 4x0.08 / IIx0.262^2 = 1.5 m/s

   $h_{inlet\ loss}$ = 0.8x1.5^2 / 2x9.8 = **0.09m**

c) Exit headless

   $h_{exit\ toss} = K_{exit}$ x V^2/2g

   1.0x1.5^2 / 2x9.8 = **0.11m** Total headloss in intake pipes = 0.29 m + 0.09 m + 0.11 m = **0.49 m**

2. Design water level in gravel trap

This should be at least 0.49 m + 10% (safety margin) i.e. 0.54 m below the low river level at the intake, or "a" = 0.54 m in Figure 4.11.

3. Flood flow through intake pipes

Flood level is 1.5 m above low river level.

Allow 0.2 m head over gravel trap spillweir (i.e. b = 0.2 m in Figure 4.11).

Then net head on intake pipes = 0.54 + 1.5 - 0.2 = **1.84 m.**, or "c" = 1.84 in Figure 4.11.

Trial and error solution for flow:

a) Try   Q = 200 1/s in each pipe

or, $1.2Q / d = 1.2 \times 0.2 / 0.262 = 0.916$

or, $f = 0.0152$ (from Moody Chart)

or, $h_{wall\ loss} = 0.0152 \times 40 \times 0.20^2 / 12 \times 0.262^2 = $ **1.64m**

$V = 4Q / \Pi d^2 = 4 \times 0.2 / \Pi \times 0.262^2 = 3.71$ m/s

Inlet & exit losses $(0.8 + 1.0)\ V^2 / 2g = $ **1.26m**

Total head loss = 2.90m

This is more than 1.84 m (net head on intake pipes for flood level), ∴ assumed flow is too high.

Try interpolating : 200 1/s $(1.84/2.90)^{1/2} = $ 159 1/s

b)   **Check for Q = 159 1/s in each pipe.**

or, $1.2Q / d = 1.2 \times 0.159 / 0.262 = 0.730$

or, $f = 0.0154$ (from Moody Chart)

or, $h_{wall\ loss} = 0.0154 \times 40 \times 0.159^2 = 1.05$m

Inlet and exit losses $(0.8+1.0)\ V^2 / 2g = 0.80$m

Total head loss = 1.85m

This is near enough to 1.84 m

Therefore flood flow through the two intake pipes is: 2 x 159 1/s = 318 1/s

4. Spillway length at gravel trap

Weir equation:

$$Q_{spillway} = C_w \times L_{spillway} \times (h_{overtop})^{1.5}$$

∴   Required spillway weir length,

$$L_{spillway} = Q_{spillway} / C_w \times (h_{overtop})^{1.5}$$

Take $C_w = 1.6$ and $Q_{spillway} = 318$ 1/s to allow the entire flood flow to be spilled over in case the turbine needs to be closed during such floods.

$h_{overtop} = 0.20$ m (head over weir, assumed earlier)

∴ $L_{spillway} = 0.318 / 1.6 \times 0.20^{1.5} = 2.22$m

or 'd' = 2.22 m in Figure 4.11.

Note the following:

1. To limit the inlet velocity, two parallel pipes have been used from the intake to the gravel trap. Downstream of the gravel trap there is only one pipe and the velocity is doubled. The reason for using a lower velocity in the initial pipe length was to lower the submergence head so that excavation work at the intake could be minimised. With lower inlet velocity there less chance of attracting floating debris which could block the inlet.

2. The spillway weir length is reasonable, but note that an alternative assumption for head over the weir would give a different answer. The walls around the gravel trap must be high enough to contain the flood flow over the spillweir.

3. The gravel trap has to be located sufficiently far downstream of the intake to ensure that the gravel trap spillweir is above the maximum river flood level at that point: in this case the 40 m intake pipe length is more than enough to meet this criterion.

4.7 Construction of canals

4.7.1 DESCRIPTION
Once the canal type has been selected and the design carried out, there are four stages in the actual construction as follows;
- setting out of the course of the canal,
- preparing the bench for the canal,
- excavating the canal, and
- lining the canal.

These sections describe a general method of canal construction and offer examples of other proven methods that may be suitable under certain conditions.

4.7.2 SETTING OUT
Setting out the canal requires the following equipment and staff:
- **Basic equipment:**
  - Level machine (or Dumpy level)
  - Measuring tape
  - Tripod
  - Wooden pegs
  - Machetes
  - Mallet
  - Pick
  - Hoe
  - Paint
  - Paintbrush
- **Staff:**
  - Surveyor
  - Chainperson (assistant to surveyor)
  - Helper to clear vegetation and prepare pegs

The setting out of the canal is done by placing pegs along the alignment. Depending on the topography, such pegs should generally be placed at 5 to 20 m intervals along the alignment. Pegs should also be placed at bends, structures such as drops. and the beginning and end of crossings and superpasses.

Some intermediate pegs or reference pegs should be placed just outside the canal alignment using a level machine (or a Dumpy level). With the use of the level machine, the difference in levels between these pegs can be calculated. Such pegs will serve as reference levels for the excavation work. An alternative to this is to paint marks at exposed rocks just outside the alignment and calculate their levels.

4.7.3 BENCH CUT
The bench of a canal is like a road of uniform width and slope, see Figure 4.12. The bench is prepared by excavating a strip of land of even width along the pegs placed earlier on the canal alignment.

The bench width should be the top width of the canal plus an allowance for berms on each side of the canal. On the hill side, a berm of 300 mm is recommended, so that material washed down by rain from the slope above is not deposited directly in the canal. A 1.0 m wide berm is recommended on the outside of the bench, to reduce seepage through the canal bank, and to provide access for construction and maintenance. A lesser berm should only be used in conjunction with vertical cement masonry walls founded on rock. Note that a berm width less than 500 mm is difficult to walk along.

The slope of the bench should be the same as the slope (S) of the canal section. Therefore, where there is a change in the canal slope (in the design) the bench slope should also change accordingly. The levels of the canal and the bench at different locations can be verified using a theodolite or a level machine and the intermediate pegs that were placed outside the canal alignment earlier.

Once the initial level at the intake is fixed, the subsequent levels can be calculated based on the slopes. The initial level can be estimated based on the contour maps of the area or by an altimeter. Another method is to use the trigonometric points established by the survey department, but this may take longer and require more resources. The initial level does not have to be very accurate (i.e. the exact elevation from the sea level) but the differences between intermediate pegs should be accurate, since it is these differences that determine the slope of the canal.

An example of a level calculation is presented below:

The designer has recommended a slope of 1.5% for a certain canal section. The topographic map of the area indicates that the elevation at the intake is around 1600 m above mean sea level (MSL).

In this case the first peg that is placed at the intake area can be assumed to be at a level of 1600 m above MSL. If the second peg is to be placed 20 m (horizontal length) downstream, the bench level here should be: 20 m x 1.5/100 = 0.30 m down from the intake or 1600 m - 0.30 m = 1599.70 m above MSL

The subsequent readings between intermediate pegs (i.e. reference points) can be noted in sequence with similar calculations.
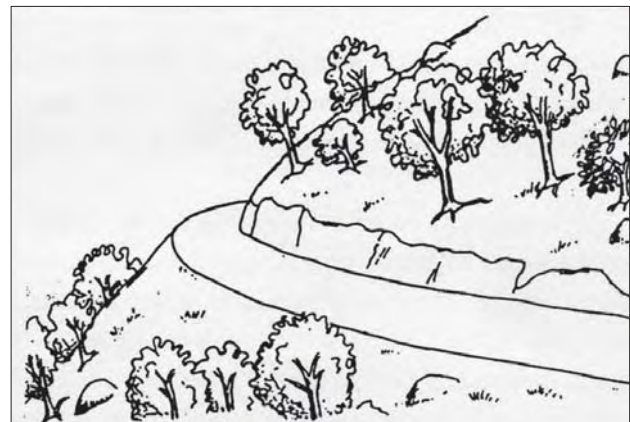
Figure 4.12 Canal bench

4.7.4 CANAL EXCAVATION

Once the canal bench has been prepared, the excavation lines need to be set out as follows:

- Place pegs along the centreline and the top and bottom widths of the canal. The centreline is an imaginary line that passes through the centre of the canal and parallel to the sides. Note that the top and bottom widths should include the side wall thickness as well (i.e. outside edges of the finished canal).

- Join the pegs using thin ropes. Then mark separate lines for the top and bottom widths (for trapezoidal sections) using powdered lime or ashes so that they are indicated on the ground. Note that for rectangular sections, the top and bottom widths are equal and two parallel lines are sufficient for the excavation work.

- Check the dimensions against the design specifications. Once the excavation lines have been prepared the canal should be excavated to the required shape and slope as per the design. For a rectangular canal, the excavation should start from the sides down to the required depth. For trapezoidal sections, the excavation should start at the central part without exceeding the bottom width lines vertically down to the required depth. Then the sides should be excavated without exceeding the top width and meeting the bottom width at the required depth. Thus, the required trapezoidal shape can be arrived at. This method of excavation minimises the use of construction materials and the need to backfill. Note that, as mentioned earlier, the side walls of a trapezoidal cement masonry canal are more likely to crack if constructed on backfill.

It is also helpful to prepare a wooden "former" to check the cross sectional shape of the canal for trapezoidal shapes. This involves constructing a wooden frame (using rectangular sticks) of the required trapezoidal shape.

The canal invert slope should be constantly checked using a level machine. Note that an inaccurate slope can be very costly: if the slope is less than required, the canal will not have the capacity to convey the design flow; if the slope is steeper than required, the velocity may exceed the maximum value for the canal type and start eroding it.

4.7.5 CANAL LINING

Once the excavation work has been completed, the actual construction of the canal can commence. The construction of the canal depends on the type that has been chosen. For example, if an earth canal has been chosen, all that is required is to trim the side walls and bottom width at some places where the excavation work has been poor. However, if a masonry canal has been chosen, then this will require collecting stones, dressing/sizing them and then placing them at the excavated surface according to the design. For stone masonry in cement mortar, the following is recommended: For stone masonry in cement mortar, the following is recommended:

- The minimum thickness for bed and side walls should be 300 mm, since thinner walls require more stone work (dressing and sizing) and may not have the required strength. This also applies to stone masonry in mud mortar canals. Recommended designs are shown in Figure 4.13.

- Sand used for preparation of the mortar should be clean and free from organic materials and fine particles. The sand particles should be granular (like ordinary sugar) and not flaky. Sand mixed with fine particles should be thoroughly washed before use. The ratio of the mortar should not be less than 1 part cement to 4 parts sand by volume (1:4 cement/sand mortar). This applies to all water retaining structures such as the settling basin and the forebay tank. The stones should be wetted before construction (dry stones absorb water from the cement mortar, stopping it reach full strength).

- 1:3 cement sand mortar may be used for plasterwork in the headrace canal and other water retaining structures such as the settling basin and the forebay. Plastering the canal lining is normally unnecessary, but could be used to reduce hydraulic losses (refer to Table 4.1) or where seepage is occurring in badly constructed masonry. The thickness of the plaster should be about 12 mm (1/2 inch).

- Immediately after the construction of a cement masonry portion of the canal length, it should be kept moist for at least four days. This is called curing and is done by gently pouring water on the walls of the canal. The use of hessian
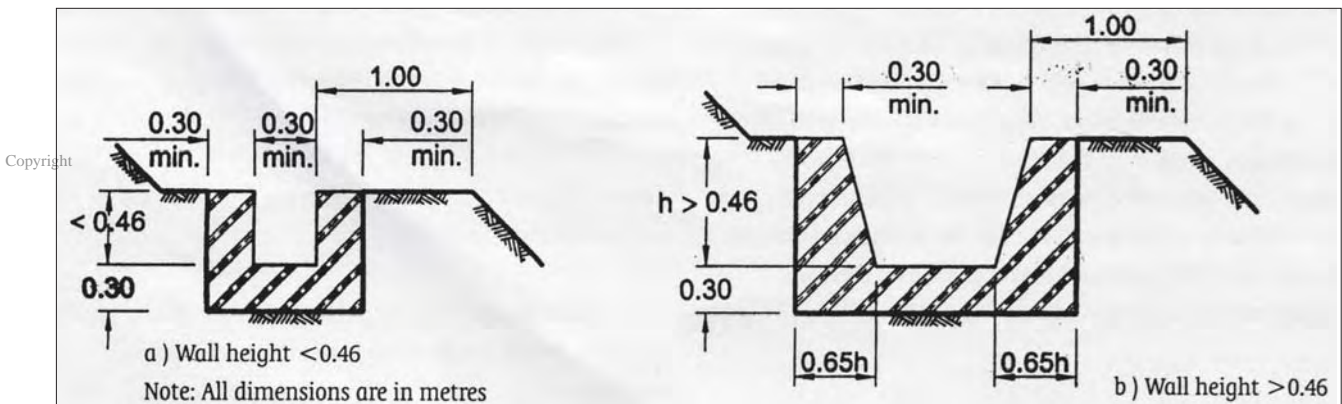
Figure 4.13 Canal lining with stone masonary in cement mortar

(wet sacks) to cover the masonry helps to retain moisture and cures the canal structure better. An uncured canal will not gain full strength. During hot and dry weather pouring water on the masonry canal will be required frequently to ensure that the canal walls remain moist and reach their full strength. Curing is more important for plasters since they are thin surfaces and can easily crack if they dry up quickly. Note that if the plaster is done at a later stage (and not immediately after the masonry work), it will require further curing for at least another 4 days. Also, the masonry should be wetted before applying the plaster.

- For plain concrete lining, use 80 mm thickness. A minimum curing period of 7 days is recommended.
- For reinforced concrete see Section 8.5.2.

4.8 Checklist for headrace works

Check the headrace alignment for stability. Is the area above and below the headrace alignment stable? Refer to Chapter 2 for signs of instability. Remember that earth canals are the most economic option where the headrace alignment is on stable ground and seepage is not likely to contribute to slope instability.

- To minimise costs, stone masonry in cement mortar canals and headrace pipes should only be used along the difficult stretches of the alignment.
- While fixing the headrace alignment do not make the invert slope steeper than necessary, since any loss in head here leaves less head for power generation. Minimise the length unless longer length is required to avoid costly crossings. Try different options so the design is economical and the construction is practical.
- Has the headrace canal or the pipe size been calculated based on the available site data? If an irrigation canal is being refurbished into a headrace, note that there may not much control over the invert slope. Decide on the type of canal by trying different cross sections and calculating the corresponding velocities such that they are within the limit shown in Table 4.1.
- For HDPE headrace pipes, be sure to follow Figure 4.8 for pipe burial details. HDPE pipes should not be exposed.
- For the construction work refer to Section 4.7.

5. Gravel trap, settling basin and forebay

5.1 Overview

5.1.1 THE SEDIMENT PROBLEM

Most rivers carry a substantial quantity of sediment in the form of gravel, sand or finer material depending on the river characteristics, geology of the catchment area and the discharge. Steeper rivers such as those that originate from the Himalayas carry cobbles and even move large boulders during annual floods. Intakes are located and designed to limit the amount of sediment entering the micro-hydro system, but such sediment cannot be entirely eliminated. Intakes can only prevent boulders and cobbles from entering into the system and minimise the influx of gravel and finer sediment. Large particles can block the headrace and reduce its capacity. Suspended sediment can cause severe wear on the turbine runner, seals and bearings, since the flow velocity at the runner is high. Such wear causes a reduction in the efficiency of the turbine and eventually leads to its complete failure. In either case, maintenance is necessary, requiring high expenditure in terms of replaced parts, man-hours and in loss of power production. There are abundant examples of turbine runners completely destroyed within a few years after installation at micro-hydro plants that lacked settling basins. The rate of wear of turbine parts due to sediment abrasion is governed by the following factors:

- Concentration of suspended particles
- Hardness of particles i Size of particles
- Shape of particles
- Resistance of turbine runner
- Turbine head

It is not necessary to exclude all sediment at the settling basin. This is virtually impossible and would not be economically viable, especially for micro-hydro schemes. A small concentration of fine sediment is often permissible as will be discussed later. The design should be such that the size and concentration of sediment passing the settling basin are within acceptable limits.

5.1.2 FUNCTION OF THE STRUCTURES

Gravel traps, as the name denotes are designed to trap gravel that enters the intake along with the diverted flow. If a river only carries fine sediment and not gravel (even during floods), then this structure is not required. However, most mountain rivers in Nepal carry gravel, especially during floods. In the absence of a gravel trap, gravel will settle along the gentler sections of the headrace or in the settling basin, where it is difficult to flush out.

A settling basin is a basin whose function is to settle the suspended particles present in the diverted river flow. Since rivers are never free from sediment, all micro-hydro schemes should have a settling basin. For small schemes, this may simply be a widened section of the canal. The flushing mechanism may be rudimentary, which is acceptable provided that damaging sediment does not reach the turbine.

A forebay is a tank located at the end of the headrace and the beginning of the penstock pipe. It is a structure that allows for the transition from open channel to pressure flow condi-tions. The water level at the forebay determines the operational head of the micro-hydro scheme.

5.1.3 LOCATION OF THE STRUCTURES

Whenever possible the gravel trap, settling basin and forebay should be combined. This minimises the construction cost. Sometimes, either the gravel trap and the settling basin or the settling basin and the forebay are combined, but the topographic conditions are rarely appropriate to be able to combine all three structures. The Jhankre mini-hydro is a rare example where it was possible to combine all three structures as described in Box 5.1. Selection of an appropriate settling basin site is governed by the following criteria:

- The location should be such that it is possible to flush the sediment and spill excess flow from the basin without causing erosion problems or damage to other structures. There must be sufficient head to flush the sediment and drain the basin.
- The settling basin should be located as close to the headworks as possible, especially if it is separate from the forebay. The earlier the sediment is removed the less the maintenance of the headrace. Furthermore, the headrace alignment downstream of the settling basin can be gentler (hence less loss in the available head) since the flow will be sediment free. A location close to the intake allows easy discharge of sediment back to the river. From an operational viewpoint, it will also be easier for the operator/helper to combine work at the intake, such as cleaning the coarse trashrack, and flushing of the settling basin.
- There needs to be adequate space to construct this structure as designed. Note that it can be a relatively wide and long structure. Therefore, locating this structure on fairly level ground minimises the excavation costs. The forebay is located immediately uphill of the transition area where the ground profile changes from level to steep. The following additional factors should be considered before deciding whether a site is suitable for a forebay:
- It should be possible to spill the entire design flow from the

forebay without causing erosion or instability problems. Ideally if this structure can be located close to a gully, it may be possible to safely divert the spillway flows into it.

- Similar to the settling basin there needs to be adequate space to construct this structure as designed. However, the forebay is usually smaller in size.

5.2 Gravel trap

A gravel trap is recommended for all micro-hydro schemes in Nepal. In the absence of a gravel trap, the settling basin must be close to the intake and able to flush the gravel that enters the basin. Gravel traps differ from settling basins in that they handle coarse material that enters near the bed, rather than suspended material that needs to be settled. The main design principle for a gravel trap is that the velocity through it should be less than required to move the smallest size of gravel to be removed. The largest size allowed to enter into the intake can be controlled by the spacing of the coarse trashrack bars. In general gravel traps should settle particles larger than 2 mm diameter. Smaller sized particles will be settled and removed in the settling basin. The following criteria should be used for the design of the gravel trap:

- To be able to trap particles down to 2 mm diameter, the velocity in the gravel trap should be limited to 0.6 m/s.
- If the gravel trap is hopper shaped, the floor slopes should be about 30° (1:1.7). Such an arrangement will facilitate easy flushing of gravel. If it is not possible to construct such a shape, the floor should slope towards the flushing end, with a longitudinal slope of 2-5%.
- The length of the gravel trap should be at least three times the width of the headrace canal or 2 m, whichever is larger. With this fixed length and a velocity of 0.6 m/s, the required width of the trap can now be determined. Note that this is a general rule of thumb, but if a significant bed load can enter the intake, then a longer length may be required. Since studies regarding the movement of gravel in rivers are rare (rarer than sediment studies), it is usually difficult to estimate the storage required in a gravel trap. Note that the storage must be provided below the normal flow depth.
- To minimise blockage of the headrace or damage due to abrasion in the headrace, gravel traps should be located as close to the intake as possible.

Gravel traps can be emptied via flushing gates or by lifting stoplogs (i.e. wooden planks). Since gravel enters the intake only during high flows, incorporating stoplogs is generally more convenient and economic.

The Galkot gravel trap is shown in Photograph 5.1 and in Drawing 420/04/2C02 of Appendix C. Although a gravel trap, this structure has also been designed as a primary settling basin. This is because the headrace canal is long (1.1 km) and if significant sediment load can be trapped in the gravel trap, the maintenance requirement will be less far downstream in the headrace canal. Furthermore, once the sediment is removed, the headrace canal slopes can be gentler as discussed in Chapter 4. Since this is a combined structure, the calculations are presented after the discussion on settling basins.

Note that as can be seen in Drawing 420/04/2A01 (Appendix C), the Galkot gravel trap is located 35 m down-stream from the intake. This is because the initial length of the intake was felt to be vulnerable to flood damage. For the same reason the coarse trashrack is placed at the end of the gravel trap. In the Galkot micro-hydro scheme, significant gravel load is not expected for the following reasons:

- The diversion weir is of a temporary nature and does not extend throughout the river width.
- The intake is located on the outside of a bend.

Photo 5.1 Galkot gravel trap

5.3 Settling basin

5.3.1 DESIGN CRITERIA

Suspended sediment that is not settled in the gravel trap is trapped in the settling basin. The basic principle of settling is that the greater the basin surface area and the lower the through velocity, the smaller the particles that can settle. A settling basin has a significantly larger cross sectional area than the headrace canal and therefore the flow velocity is lower which allows the settling of the suspended materials. A settling basin must satisfy the following three criteria:

Settling capacity
The length and width of the basin must be large enough to allow a large percentage of the fine sediment to fall out of suspension and be deposited on the bed. The sediment concentration passing the basin should be within acceptable limits. The geometry of the inlet, the width of the basin and any curvature must be such as to cause minimum turbulence, which might impair the efficiency.

Storage capacity

The basin should be able to store the settled particles for some time unless it is designed for continuous flushing. Continuous flushing mechanisms are however not incorporated in micro-hydro schemes due to the complexity of the design and the scarcity of water during the low flow season. Hence, the storage capacity must be sufficiently large that the basin does not require frequent flushing.

Flushing capacity

The basin should be able to be operated so as to remove the stored particles from it. This is done by opening gates or valves and flushing the sediment along with the incoming flow in the basin. The bed gradient must be steep enough to create velocities capable of removing all the sediment during flushing.

5.3.2 THE IDEAL SETTLING BASIN

The theory behind the design of a settling basin is derived on the basis of an ideal basin. Therefore, before proceeding to the design phase, the concept of the ideal basin needs to be under-stood. Such an ideal basin is shown in Figure 5.1. Consider a particle entering the "ideal settling basin" on the water surface at point X (i.e. beginning of the settling zone) as shown in Figure 5.1. In this figure:
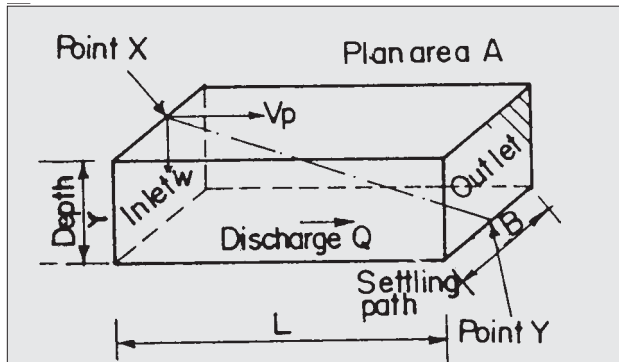
Figure 5.1 An ideal settling basin

L	=	length of the settling zone (m)
B	=	width of the settling zone (m)
Y	=	mean water depth in the settling zone (m), aslo called hydraulic depth
t	=	time for particle to travel the length L (s)
V_p	=	horizontal velocity component of the particle (m/s)
W	=	vertical velocity component of the particle (ms), i.e., "fall velocity" which is discussed later
Q	=	discharge (m³/s)

Then the following equations must hold for the particle to reach the end of the settling basin (point Y):

$$y = w\,t \quad (a)$$
$$L = v_p t \quad (b)$$
$$Q = Bv_p y \,(c)$$

Substituting for y, v_p and t from (a) and (b) into (c) results in:
$$Q = BLw$$
Therefore, for a given discharge Q, the plan area of the settling basin can theoretically be determined for sedimentation of a particle with fall velocity w. However, in practice, a larger basin area is required because of the following factors:
- the turbulence of the water in the basin;
- imperfect flow distribution at the entrance; and
- the need to converge (sometimes curve) the flow towards the exit. Therefore in "real basins" the through velocity is limited, to reduce turbulence, and the required plan area is about twice the area calculated for the "ideal basin".

5.3.3 FALL VELOCITY OF SEDIMENT AND PARTICLE SIZE

The fall velocity, w, characterises the ability of particles of various sizes to settle out under gravity. For a discrete particle, this value depends on its size, density, and shape, as well as the temperature of water.

Figure 5.2 shows the fall velocity in water, w, as a function of the particle diameter for reference quartz spheres. This figure can be used to estimate w for the calculations required in the design of the basin. Note that the temperature effect becomes less for larger diameter particles.

In micro-hydropower schemes, the settling basin is designed to trap 100% of particles greater than a certain size, d_{limit} Only a proportion of smaller particles will be trapped, but d_{limit} is set so that the smaller particles passing though the basin will not cause significant abrasion damage to the turbine.

For micro-hydro schemes the following procedure is recommended for the selection of d_{limit} :
- Low head schemes, h≤10 m: d_{limit} = 0.2 mm to 0.5 mm
- Medium head schemes, 10 m < h≤100 m : d_{limit} = 0.2 to 0.3 mm
- High head schemes, h > 100 m : d_{limit} = 0.1 to 0.2 mm
where h is the gross head.

The current practice in Nepal is to use d_{limit} of 0.3 mm regardless of the head of the scheme, which is somewhat arbitrary. The approach outlined in this section is more logical. This is because for a given particle *size,* the higher the head, the more the damage is to the turbine.
The d_{limit} range given above as a function of head and flow allows the designer some flexibility in deciding the particle size to be settled.

The following factors should be used while deciding on the value of d_{limit}:
- If most particles are highly abrasive (quartz sand or minerals), then the lower limiting values should be used. If the particles are softer less abrasive substances, then the higher limiting values may be acceptable.
- Crossflow turbines are relatively less sensitive to soft impurities such as silt and clays. Other types such as the Francis turbines are more sensitive to any kind of suspended matter. Pelton turbines are intermediate.
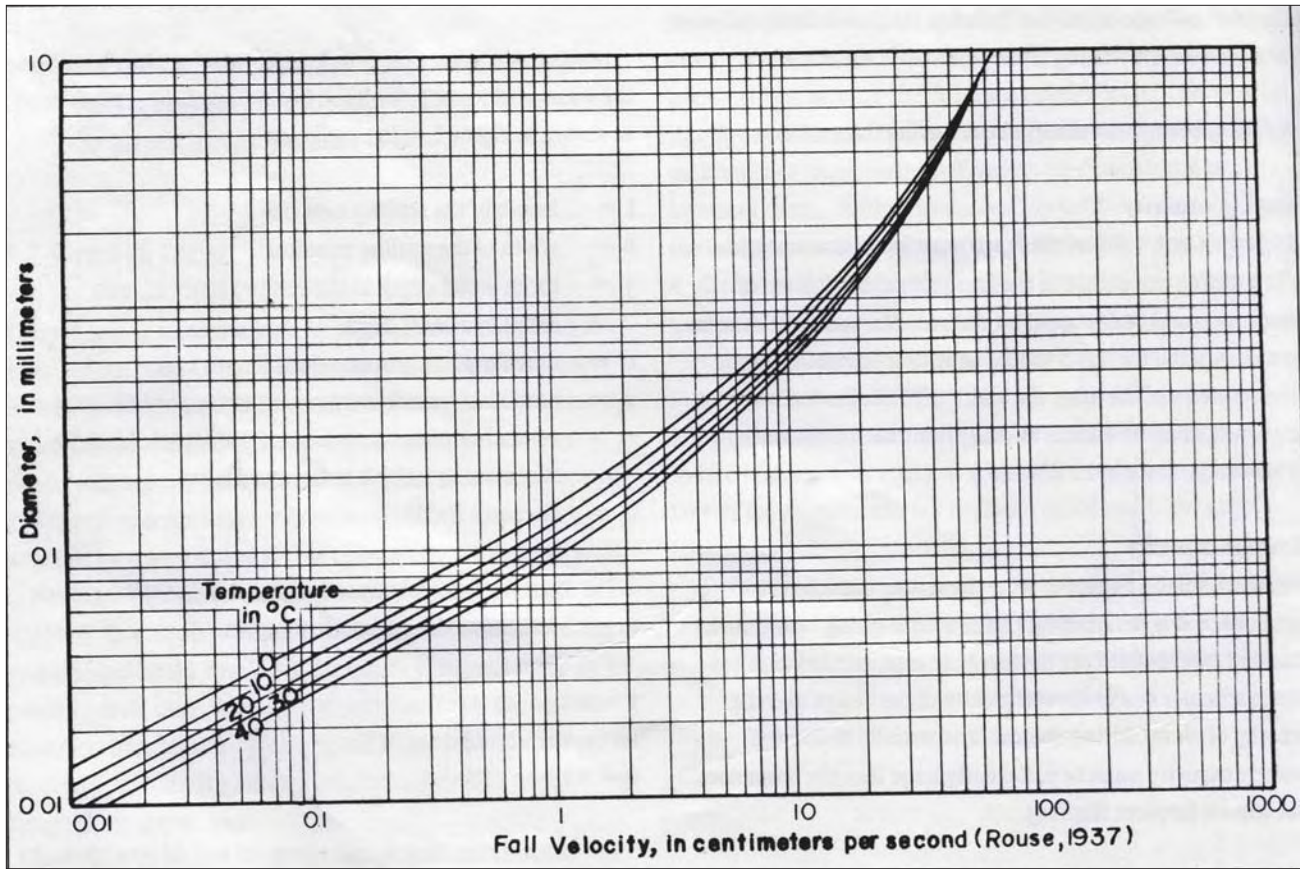
Figure 5.2 Fall velocity of quartz spheres in water

- For example, d_{limit} = 0.2 mm should be selected in a case where: h = 50 m, suspended particles are mostly pure quartz or similar minerals, and a Francis turbine is used.

5.3.4 SETTLING DESIGN

The area required for the settling basin and its plan shape are calculated as follows:

1. Using the criteria discussed in Section 5.3.3, determine what the range of the scheme is (i.e., low, medium or high head) and decide on the corresponding minimum particle size to be settled, i.e. d_{limit} .
2. Using Figure 5.2, for the selected d_{limit} , determine the fall velocity, w.
3. Calculate the required basin surface area (A) using the following equation: A = 2Q / W Note that a factor of 2 has been used to allow for turbulence in the basin.
4. With the basin area calculated above, fix either the length, L, or the width, B, according to site conditions and calculate the other dimension such that $4 \leq L/B \leq 10$.
5. Check that the horizontal velocity (V = Q/By) is less than $0.44\sqrt{d_{limit}}$, i.e. V < 0.24 m/s

where d_{limit} = 0.3 mm. If not, increase the cross sectional area (B or y) to meet this condition.
Alternatively Vetter's equation which is commonly used in design of small hydro settling basin in Nepal and gives reasonable estimates of the dimensions required is also worth

mentioning here. The basic philosophy in this method is to calculate settling efficiency for the particle size considered. The vetter's equation to determine settling efficiency is:
Where, $\eta = 1\text{-}e\text{-}\left[\frac{w*As}{Q}\right]$
η = settling efficiency of a settling basin (the result is a ratio and could be converted into a percentage by multiplying by 100)
w = settling velocity of considered particle size, m/s
A_s = surface area of the settling basin settling zone, m²
Q = design flow, m³/s
A 90% settling efficiency for particle size based on head as discussed earlier would be appropriate when using Vetter's equation to size settling basin in micro hydro schemes. The desired efficiency of the settling basin is achieved by finding the surface area of the settling zone and then dimensioning the area considering the criteria mentioned in previous method.
A schematic diagram of a typical settling basin is shown in Figure 5.1. Double chamber will not be required for small flows but this arrangement will enhance the settling efficiency and another advantage is that while sediments are being flushed in one chamber, the other could continue conveying flows downstream and thus half of the possible power output can be generated instead of complete plant shutdown.

5.3.5 STORAGE DESIGN

The concentration of suspended particles in the flow can be expressed as follows:
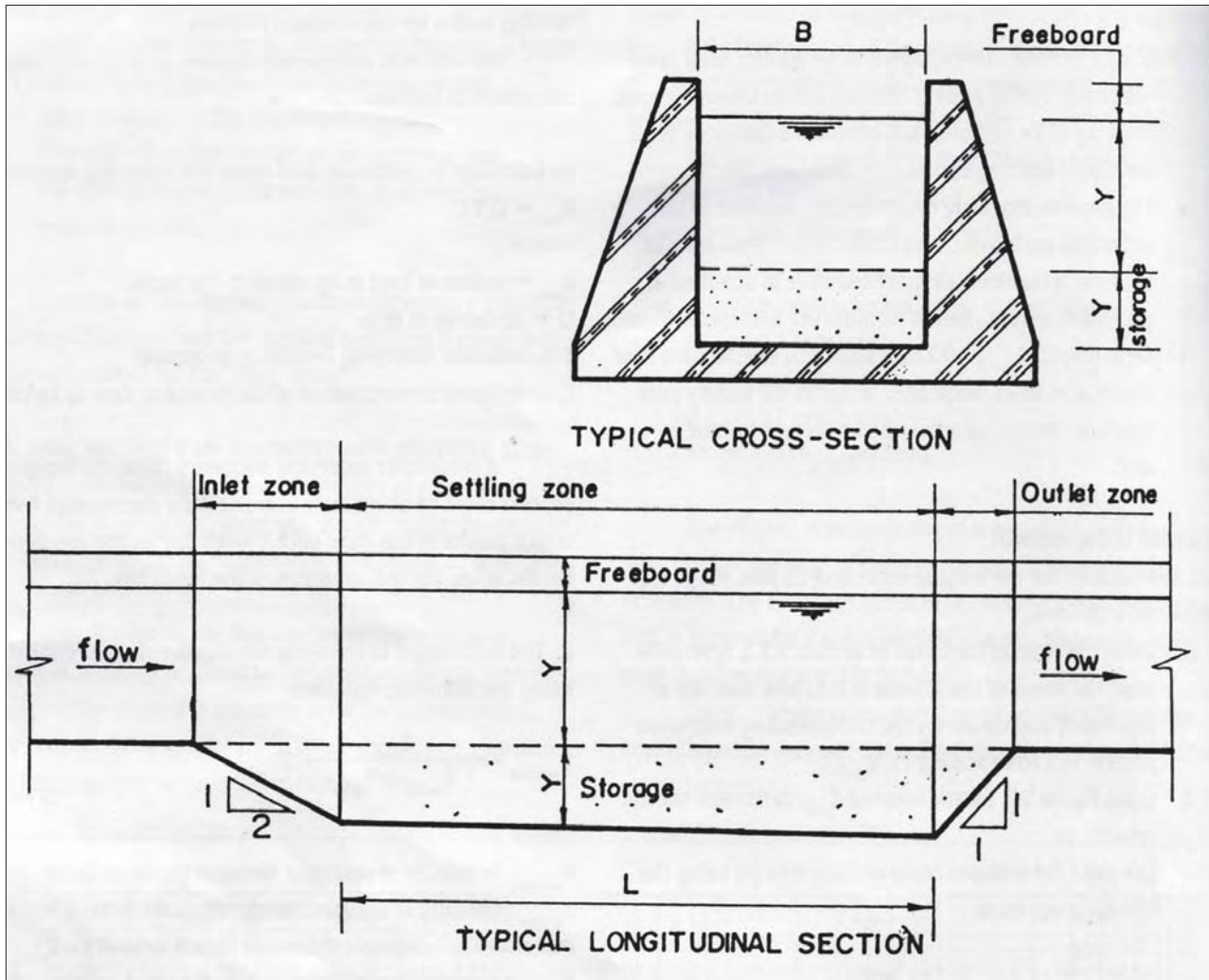
Figure 5.3 A typical settling basin and its components

Concentration (C) = kg of suspended matter / m³ of water
Unfortunately, there have not been any studies regarding the concentration of sediment in small mountain rivers of Nepal that are appropriate for micro-hydro installation. Therefore, in the Nepalese context, the designer has to rely on data available from large hydropower projects. The recommended practice in Nepal is to use C = 2 kg/m³ for the design of settling basins for micro-hydro schemes .

The sediment storage requirement in a settling basin is calculated as follows:

1. Calculate the sediment load using the following equation:

S_{load} = Q T C

where:

S_{load} = sediment load in kg stored in the basin
Q   = discharge in m³/s
T   = sediment emptying frequency in seconds
C   = sediment concentration of the incoming flow in kg/m³ .

A reasonable emptying frequency (T) in the Nepalese context could be about once to twice daily during high flow, which results in less than once a week during the low flow season when the sediment concentration is low.

2. The next step is to calculate the volume of the sediment. using the following equation:

$V_{sediment}$ = S_{load} / $S_{density}$. S_{factor}

where:

$V_{sediment}$ = volume of sediment stored in the basin in m³.
$S_{density}$ = density of sediment in kg/m³, about 2600 kg/m³. Unless other data are available this value should be used for $S_{density}$.
P_{factor} = packing factor of sediment submerged in water.

When submerged in water, particles occupy more space than when dry. This is measured in terms of packing factor, which is the ratio of unit volume of dry sediment to unit volume of wet sediment (i.e. volume of 1 m³ of dry sediment divided by the volume of this sediment when submerged). Packing factor for submerged sediment is about 0.5 (i.e. the volume of dry sediment is doubled when submerged).

Below the settling zone must be the capacity to store the calculated volume of sediment, $V_{sediment}$. This storage space is achieved by increasing the depth of the basin as follows:

$$Y_{storage} = V_{sediment} / A$$

Where Y_{storae} is the storage depth in the settling basin below the hydraulic depth (y) discussed earlier, and A is the plan area. The hydraulic depth and the storage depth are also shown in Figure 5.3.

5.3.6 COMPONENTS OF A SETTLING BASIN

The settling basin has three distinct zones; the inlet, settling and outlet zones. These are discussed below and shown in Figures 5.3 and 5.4.

Inlet zone

This is the initial zone where the transition from the headrace to the settling basin occurs and there is a gradual expansion in the basin width.

The design of the inlet is important to the efficiency of the basin. For high hydraulic efficiency and effective use of the basin, the inlet should distribute the inflow and suspended sediment over the full cross sectional area of the settling zone. Various research data show that horizontal velocity variations across the width of a rectangular tank affect the hydraulic efficiency considerably more than velocity variations in depth. Therefore, attention needs to be given to uniform flow distribution in the horizontal plane. The following methods are used in the inlet zone to achieve a good flow distribution:
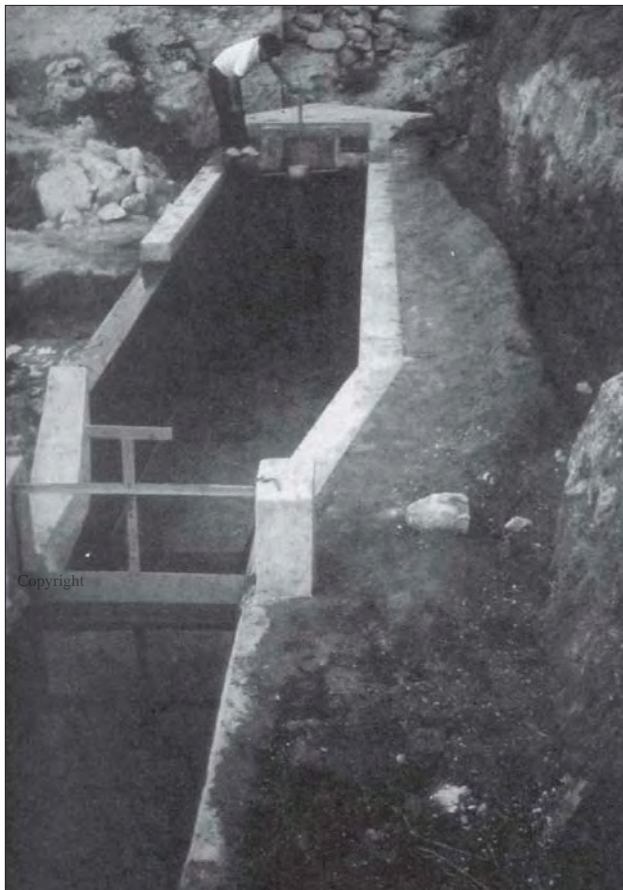
Photo 5.2 Sliding gate at settling basin entrance, Peru
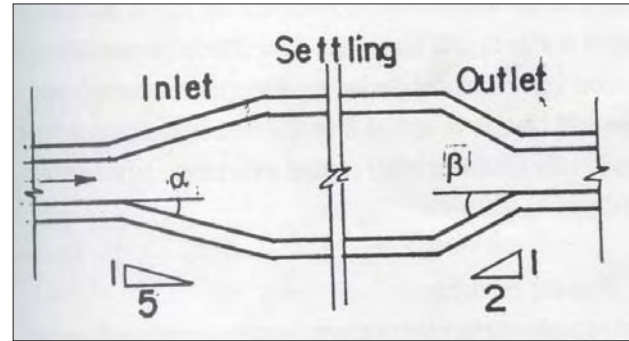
Figure 5.4 Expansion and contraction ration in settling basin

- Gradual expansion of the inlet channel. This is the most commonly used method in micro-hydro schemes. To determine the length of the inlet zone, set the horizontal expansion ratio at about 1:5 (a = 11°) as shown in Figure 5.4. This will allow an even flow distribution at the beginning of the settling zone. The vertical expan-sion ratio can be higher at about 1:2 (a = 27°) as shown in Figure 5.3.
- Another option is to incorporate a weir as can be seen in the Galkot settling basin (Drawing 420/04/3C01).
- Troughs with slots or orifices in walls or bottom.
- Baffle walls

Note that orifices or baffle walls are often used in water treatment facilities where extremely low velocity is required but these methods are rarely used in micro-hydro schemes. In some schemes in Peru, a sliding gate is installed in front of the settling basin as shown in Photograph 5.2. During flushing, the gate is initially closed, impounding water behind it. When the settling basin is emptied, the gate is opened and the sudden rush of the impounded water flushes out any sediment that has remained inside the basin.

Settling zone

The basin reaches the required width at the beginning of this zone. Particles are settled, stored and flushed in this zone. The length of this zone is longer than the inlet or the outlet zones. It should be noted that long narrow basins perform better than short wide basins. A range of 4 to 10 is recommended for the ratio of the length to width (L/B). Basin shape can also be improved by subdivision with a longitudinal divide wall, since this doubles the L/B ratio for a given basin length. Also, the longitudinal divide wall can assist in the operation of the scheme. For example, the sediment in one sub-basin can be flushed while the other is in operation, producing half the power output. Without the subdivision, the plant would have to be closed during flushing.

Provision for flushing the stored sediment should be at the end of the settling basin. A floor slope of 1:20 to 1:51 in the settling zone facilitates flushing.

Outlet zone

This forms the transition from the settling zone to the headrace. The transition can be more abrupt than the inlet
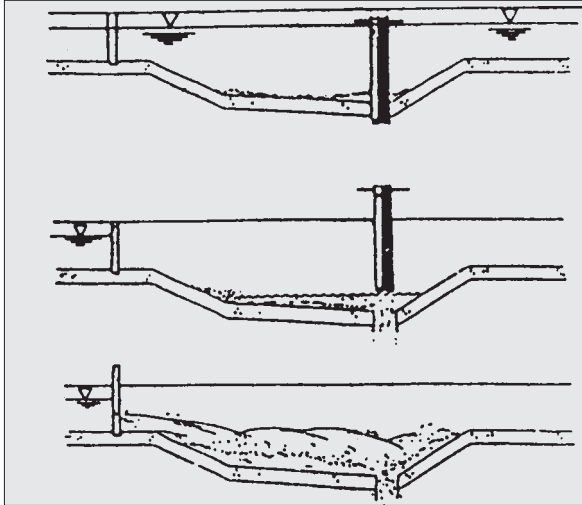
Figure 5.5 Flushing settling basin using the vertical flush pipe method

Photo 5.4 Jharkot settling basin

expansion (i.e., horizontally 1:2 or p = 26.5° as shown in Figure 5.4, and vertically 1:1 as shown in Figure 5.3). Note that if the settling basin is combined with the forebay, then this zone is not necessary: the forebay structure can be directly downstream of the settling zone.

The operating water level of the settling basin is generally controlled at the outlet, sometimes by a weir which may be designed to operate as submerged in order to conserve head.

5.3.7 FLUSHING ARRANGEMENTS
Vertical Hush pipe method

There are various ways of removing the stored sediment from the settling basin. An appropriate method for micro-hydro settling basins is the "vertical flush pipe". This uses a detachable vertical mild steel pipe over a hole in the basin floor. A drain pipe is fixed below the basin floor to convey the flow out of the basin. When the vertical flush pipe is lifted, the water stored in the basin and the incoming flow along with the sediment are drained through the hole. Apart from being simple, the other advantage of this system is that it can spill

Photo 5.3 Lifting the flushing mechanism (Jhong)

some excess flow such as during floods when the water level in the basin is above the normal level. This vertical flush method is shown schematically in Figure 5.5. The Galkot settling basin is based on this method as can be seen in Drawing 420/04/3C01 of Appendix C. Similarly, the jharkot micro-hydro scheme also uses this method for flushing as can be seen in Photograph 5.3. The diameter of the flush pipe is governed by the following criteria:

a) Overflow capacity

It needs to spill the excess flood flow that enters the basin as shown in Figure 5.6. This is governed by the weir equation, where the perimeter of the pipe is used for the length as follows:

$$Q_{flood} = \Pi d C_w h_{flood}^{3/2}$$

where:

Q_{flood} is the expected flood flow in the basin

h_{flood} is the depth of water above the vertical pipe during Q_{flood} This is the height between the top of the vertical flush pipe and the top of the settling basin wall.

C_w is the weir coefficient for a sharp edged weir, which is 1.9 (see Table 3.3, Chapter 3). The reason for using the sharp edged weir coefficient is because the pipe thickness is small compared to the head.

In terms of the pipe diameter, the above equation can be rewritten as follows:

$$d = Q_{flood} / 1.9 \Pi h_{flood}^{3/2}$$

To ensure draining of excess flow and to prevent spilling of the design flow the height of the vertical flush pipe should be such that the top level is 50mm above the design water level. Also note that if the settling basin is combined with the forebay, it may be more important to size the flush pipe diameter such that it is able to spill the design flow. This is because if the turbine valve is closed during emergencies, the entire design flow will have to be spilled from the forebay until the operator reaches the intake or other control structures upstream of the forebay.

The pipe should be able to divert both the incoming flow and the water volume in the basin, thus emptying it. This is based on the following equations:

$$1.5Q_{design} = CA \sqrt{h_{basin} + h_{flush}}$$

$$\text{or, } Q_{design} = CA \sqrt{h_{flush}}$$

where:

Q_{design} is the design flow. Q_{design} is multiplied by 1.5 in the first equation to ensure that there is a draw down in the water level inside the basin during flushing (i.e., both the incoming flow and the flow in the basin can be drained).

C is the orifice coefficient = 2.76 (applies only where the total pipe length is less than 6 m).

h_{basin} is the depth of water in the basin during the design flow prior to flushing.

h_{flush} is the flushing head when basin is empty. This is the difference in level between the floor of the basin and the flush pipe outlet as can be seen in Figure 5.6

A is the area of the pipe section.

The second equation ensures that the design flow can be discharged through the system when the basin is empty. It is important to check this condition especially if the h_{flush} is low.

$$d = (\ 6Q_{design} / \Pi C \sqrt{h_{basin} + h_{flush}}\ )$$

$$\text{or, } d = (4Q_{design} / \Pi \sqrt{h_{flush}})$$

In terms of the pipe diameter, the above two equations can be rewritten as follows:

Note that these equations assume that there is free pipe flow at the outlet and the pipe diameter is constant (vertical and horizontal pipes of the system have the same diameter).

All of the above three equations should be used to size the diameter of the flush pipe. The pipe should be sized using the equation that results in the largest diameter. If the total pipe length is more than 6 m, the flow should be calculated using the guidelines given in Chapter 4.

5.3.8 OTHER CONSIDERATIONS

Spillway requirement

If excess flows can not be spilled from the upstream headrace portion such as due to lack of a suitable area (or if a pipe is used), a spillway should also be incorporated at the settling basin. The spillway should be sized to spill the entire flow expected during the high flow season. This is because the plant may need to be shut during high flows for repair work. The spillway should be located upstream of the basin to avoid excess flow (and sediment) through the basin. Note that the design of spillways was covered in Chapter 4.

However, in the case where there is a vertical flush pipe sized to divert the expected high flows, a separate spillway is not necessary.

Cover

Sometimes there can also be site specific considerations that need to be addressed during the design of the settling basin.

Photo 5.5 Settling basin at Jharkot
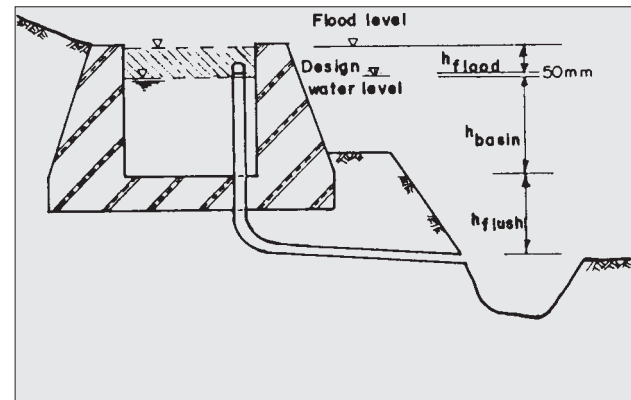
Figure 5.6 Vertical flush pipe section in a settling basin
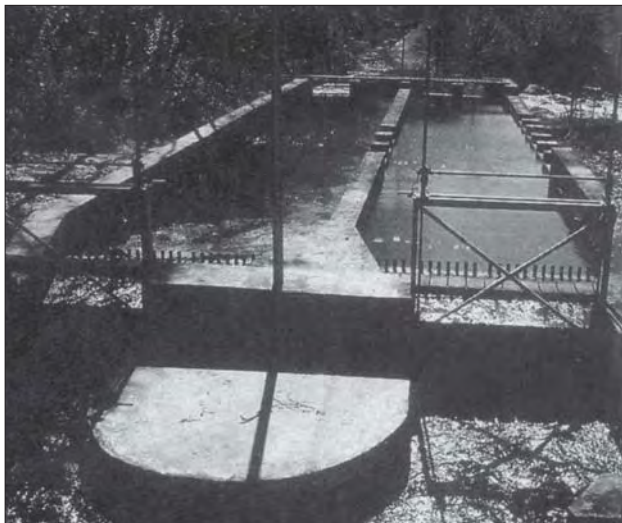
Photo 5.6 Salleri Chialsa settling basin

For example, the Ghandruk settling basin is located in a forest area and large tree leaves were constantly blocking the trashrack. This problem was overcome by placing wire mesh over the basin as can be seen in Photograph 5.7.

Sluice gate

Another conventional method of flushing includes the use of sluicing gates. This is more common in mini- and large hydropower schemes. In this system, gates are lifted either manually or mechanically, to drain the basin. The Salleri

Photo 5.7 Ghandruk settling basin

Example 5.1 Galkot gravel trap

The Galkot gravel trap/primary settling basin is designed for a flow of 4551/s. Note that the gross head of the scheme (h) is 20 m.

As gravel trap only, the minimum dimension should be as follows:

Cross sectional area required for V = 0.6 m/s to trap particle size down to 2 mm.

A = Q / V = 0.455 / 0.6   = 0.758 m^2
With a flow depth   = 0.5 m:
Width B   = 0.758/0.5   = 1.52 m, say 1.5 m,
Using L   = 3 x headrace canal width (B' = 0.6 m)
          = 1.8 m. Use 2.0 m minimum length.

Therefore if this structure were designed as a gravel trap only, the dimensions would have been as follows:

        B = 1.5m
        L = 2.0 m

As a settling basin, the dimensions required are as follows:

Since 10 m < h < 100 m, the scheme is classified as medium head. Recall that the turbine is a crossflow type.

Therefore, d_{limit} = 0.3 mm.

With a mean river temperature of 15 °C during the high flow season, from Figure 5.2, the fall velocity w = 0.037 m/s for

d_{limit} = 0.3 mm
LB = 2Q/w = 2 x 0.455/0.037=24.6 m^2
Set B = 2.5 m
L = 24.6/2.5 = 9.8 m. Set L= 10.0 m
L/B = 4

Inlet profile: due to space constraints set á = 26°,

L_{inlet} = B-B' / 2tan(á)
             = 2.5 - 0.6 / (2 tan 26°)
             = 1.94 m. Set L_{inlet} = 2.0 m

Check required depth of settling zone, y: Maximum horizontal velocity

V   = $0.44\sqrt{d_{limit}}$
           = $0.44\sqrt{0.3}$
           = 0.24 m/s
Y   = Q /BV =0.445 / 2.5 x 0.24 = 0.76m

Sediment storage requirement:

Assume sediment concentration, C = 2 kg/m^3

Flushing frequency, T = 8 hours was chosen for this scheme (but 12 hours is recommended). 8 hours = 28,800 s

$S_{density}$ = 2600 kg / m^2 p_{factor} = 0.5
S_{load} = Q xT x C
               = 0.455 x 28,800 x 2
               = 26,208 kg
$V_{sediment}$ = S_{load} / $S_{density}$ xS_{factor}
               = 26,208 / (2600 x 0.5) =20.16 m^3

Actual basin area = LB = 10 x 2.5 = 25 m^2

Required storage depth, $Y_{storage}$ = 20.16/25
                                    =0.81 m

Required depth of basin   = Freeboard + y + $Y_{storage}$
                               = 0.3 + 0.76 + 0.81
                               = 1.87 m

Therefore, as can be seen in Drawing 420/04/2C02 in Appendix C, the actual internal dimensions of the basin are as follows:

$L_{settling}$ = 10.0 m
$B_{settling}$ = 2.5 m

Depth of basin   = 1.9 m

L_{inlet} = 2.0 m
L_{outlet} = 1.25 m

Since the headrace canal is long, a secondary settling basin has been incorporated upstream of the forebay as can be seen in Drawing 420/04/3C01 in Appendix C. As can be seen in this drawing, the settling basin has the following dimensions:

$L_{settling}$ = 7.8 m
$B_{settling}$ = 2.4 m

Depth of basin = 1.7 m (av.)

L_{inlet} = 4.4 m including the weir length
L_{outlet} = 0 m (outlet into forebay)

Check settling basin efficiency in settling particle diameters of 0.3 mm, i.e., selected d_{limit}

Recall Vetter's equation

$$\eta = 1 - e^{-\left[\frac{w*A_s}{Q}\right]}$$

Where for the above design: w = 0.037 m/s for d_{limit} = 0.3 mm and As the surface area of settling basin = 25 m2.

Or: $\eta = 1 - e^{-\left[\frac{0.037*25}{0.455}\right]}$ = 0.87 or the settling basin has 87% efficiency in settling particle size of 0.3 mm. Note that the efficiency is close to the desired value of 90% and thus the basin dimensions derived from the first method is in agreement with Vetter's equation.

Note the following:

Q_{design} = 4211/s in the secondary settling basin/forebay. Recall from Chapter 3 that the flow required for power production was 4211/s and the headrace canal was designed for 4551/s to provide extra irrigation water.)

As discussed earlier, a weir has been incorporated at the inlet zone to ensure an even distribution of flow. Since the settling basin is combined with the forebay, the outlet zone has been omitted. Also notice the submerged weir at the end of the basin which controls the velocity through the basin and provides sediment storage depth. Since the gravel trap is expected to settle most of the sediment, there has been a compromise in the ratio of L/B (3.25) in the settling basin: in this case the weir and parallel sides upstream of the settling zone give a good flow distribution across the basin.

As discussed earlier, note the flushing arrangement where sediment is flushed by removing the vertical pipe. The sizing of this pipe is as follows:

Since there are a number of spillways along the headrace canal (upstream of the settling basin/forebay), flood flow is not expected. Therefore, the criterion is to ensure that the flush pipe is able to divert both the incoming design flow and the water volume in the basin, thus emptying it during flushing

Check the flush pipe diameter using the following two equations:

Note that as can be seen in the Galkot drawing (420/04/3C01) h_{basin} = 1.3 m and h_{flush}=1.7 m

$$d = \left(\frac{6Q_{design}}{Ð\,c\sqrt{h_{basin} + h_{flush}}} \right) = 0.410 \text{ m or } 410 \text{ mm}$$

$$d = \left(\frac{4Q_{design}}{Ð\,c\sqrt{h_{flush}}} \right) = 0.386 \text{ m or } 386 \text{ mm}$$

Terefore, a diameter of 410 mm is required for the flush pipe.

The actual flush pipe diameter that was used for the Galkot scheme is 400 mm (slightly smaller than calculated). This was accepted because it matches the penstock diameter and was easier to fabricate.

Box 5.1 Jhankre settling basin

The Jhankre mini-hydro scheme is a rare example where it was possible to combine the gravel trap, settling basin and forebay into one structure as can be seen in Photograph 5.8 and Figure 3.8 (Chapter 3). The intake is located at the left bank of a wide pool and there was adequate space to combine all three structures here. Also, the topography was such that it was possible to start the penstock alignment right at the intake.

The gravel trap is placed at the curved section immediately behind the intake. The straight section behind the gravel trap works as a settling basin and the tank behind this is the forebay. Since the initial penstock alignment is buried it cannot be seen in the photograph. Also note that as discussed earlier, the structure has been divided into two basins so that the plant need not be completely shut during flushing.

Photo 5.8 Gravel trap, settling basin and forebay of Jhankre mini-hydro

Platforms have been incorporated at the end of the gravel trap, settling basin and above the forebay to facilitate flushing as well as cleaning of the trashrack. The flushing system for both gravel and sediment is a modification of the "vertical flush pipe" method. As shown in Figure 5.7, instead of a flush pipe a cylinder (tapered at the bottom) is used which fits snugly into the hole on the basin floor. Since the water depth is high, the cylinders are lifted with spindle wheels (thus gaining mechanical advantage) that rest on the slabs. However, note that with this system, the excess flow cannot be diverted via the cylinders since they are closed on top.

The trashrack at the forebay is cleaned by raking through the bars (flat plates) with a special steel rake. To prevent vandalism the spindle wheels are stored separately and installed only during flushing. In the photograph the threaded ends (where the spindles are inserted) can be seen on the gravel trap and settling basin platforms.
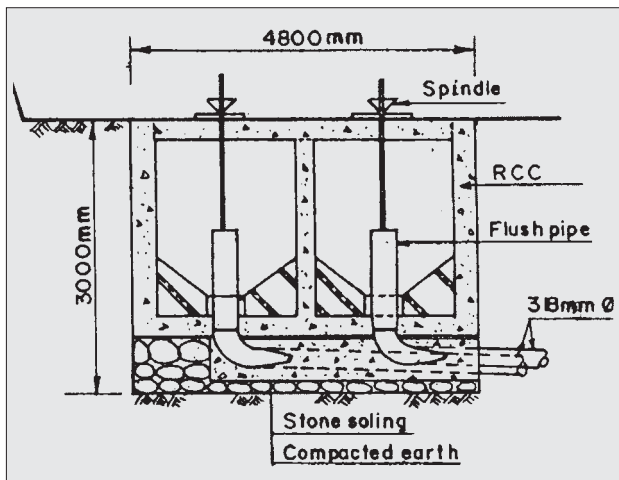
Figure 5.7 Flushing system of Jhankre mini-hydro

Photo 5.9 A dry stone masonry forebay showing the connection with the penstock

Chialsa mini-hydro scheme is based on such conventional flushing system as can be seen in Photograph 5.6.

5.4 Forebay

5.4.1 GENERAL
The forebay of a micro-hydro scheme serves the following functions:
- It allows for the transition from open channel to pressure flow conditions.
- It regulates the flow into the penstock, particularly through the release of excess water into a spillway.
- It releases the surge pressure as the wave travels out of the penstock pipe.
- It can also serve as a secondary/final settling basin and trap some particles that enter the headrace downstream of the settling basin.
- Although very rare in micro-hydro schemes, the forebay can also provide water storage for use during peak power demand period as discussed in Box 5.2.

Structurally, the forebay tank is similar to the settling basin

except that the outlet transition is replaced by a trashrack and the entrance into the penstock pipe.

5.4.2 PIPE LEVEL
The forebay allows for the transition from open channel to pipe flow by providing adequate submergence for the penstock pipe. As discussed in Chapter 4, if the submergence head is not sufficient, the pipe will draw in air and be unable to convey the design flow. Similarly, recall from Chapter 4 that the minimum submergence head required for the penstock pipe is as follows: $h_s > 1.5 V^2 / 2g$ where: h_s is the submergence head, and V is the velocity in the penstock.

5.4.3 DESIGN OF A FOREBAY
If the length of the headrace canal between the settling basin and the forebay is long, then sediment can enter the canal, such as when debris falls from uphill of the headrace alignment. Similarly if an earth canal (or stone masonry in mud mortar) is constructed between the settling basin and the forebay, sometimes high velocity in the canal, such as during the monsoon, can cause erosion and carry sediment to the forebay. In such cases the forebay should also be designed to serve as a secondary settling basin and the design method used for sizing a settling basin should be used. However, a lower sediment concentration (say C = 0.5 kg/m³) can be used since only particles that have escaped the settling basin or those that have been eroded from the headrace canal are expected in the forebay. If the headrace upstream of the forebay consists of HDPE pipe or of cement masonry canal and the settling basin is functioning well, there will not be any need for secondary settling. However, as a precaution, some storage depth below the pipe invert should be allowed for. A depth of 300 mm or equal to the pipe diameter, whichever is larger is recommended for this purpose.

5.4.4 FOREBAY SIZE
The minimum size of the forebay should be such that a person can get in and clean it. The minimum clear width required for this is 1 m. Even if the sediment load is not expected in the forebay, it may sometimes reach this structure such as when the settling basin is filled in quickly during the monsoon or there is a small landslide. If a person can get into the forebay and clean it occasionally or during the annual maintenance period, limited sediment accumulation will not be a problem. A storage depth below the invert of the penstock should be provided for this, as can be seen in Figure 5.8.

If possible, the forebay should also be sized such that 15 seconds of the design flow can be safely stored in the tank above the minimum pipe submergence level. This is more important if the scheme consists of a headrace pipe instead of a canal. There can be small transient surges in the headrace pipe which result in uneven flow. The 15 second storage capacity helps to balance such uneven flows.

5.4.5 TRASHRACK

The trashrack at the forebay should be placed at 1:3 slope for both efficient hydraulic performance and ease of cleaning (such as by raking) as shown in Figure 5.8. To minimise headless and blockage, the recommended velocity through the trashrack is 0.6 m/s. but a maximum of 1 m/s could be used.

It should also be noted that the trashrack bars should be placed vertically since horizontal bars are difficult to clean as shown in Photograph 5.11. The spacing between the trashrack bars should be about half the nozzle diameter for Pelton turbines and half the runner width for crossflow turbines. This prevents the turbines from being obstructed by debris and minimises the chances of surge.

Photo 5.10 A cement masonry channel and forebay at the top of a steep slope (Dhading)

Cleaning of the trashrack can be minimised by fixing it such that it is submerged during the design flow as in the case of the Salleri Chialsa mini-hydro (Photograph 5.12.). Here the top level of the trashrack is below the design water level. Any floating debris such as leaves are washed by the flow above the trashrack and spilled via the spillway. Although this type of arrangement facilitates self-cleaning of the trashrack, some additional flow (than the design flow) will be constantly required.

5.4.6 SPILLWAY

As discussed earlier, a spillway should also be incorporated at the forebay. The spillway should be sized such that it can release the entire design flow when required. This is because if the turbine valve is closed during emergencies, the entire design flow will have to be spilled from the forebay until the operator reaches the intake or other control structures upstream of the forebay.

Figure 5.8 Forebay

Photo 5.11 Cleaning can be difficult with horizontal bars
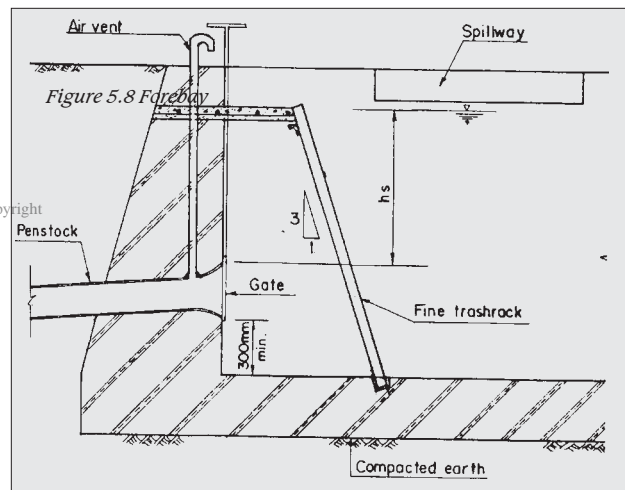
5.4.7 GATE AND VENT

Incorporating a gate at the entrance of the penstock will make the maintenance work of the turbine easy. The gate can be closed, thus emptying the penstock and work can be done in the turbine. However, a rapid closure of this gate will create a negative pressure (i.e. vacuum) inside the pipe and could cause it to collapse. Placing an air intake pipe as shown in Figure 5.8 will prevent such a situation since air can be drawn

Photo 5.12 Submerged trashrack, Salleri Chialsa mini-hydro scheme

through the air intake pipe and into the penstock. The required size of the air vent is given by:

$$d^2 = Q \text{ or}$$

$$F/E \left(D / t_{effective} \right)^3$$

where:

d = internal diameter of air vent (mm)l

Q = maximum flow of air through vent (l/s)

  = maximum flow of water through turbine

E = Young's Modulus for the penstock (N/mm², see Table 6.2)

D = Penstock diameter(mm)

$t_{effective}$ = effective penstock wall thickness at upper end (mm) (refer to Section 6.6)

F = safety factor, 5 for buried pipe or 10 for exposed pipe.

$$d^r = Q \sqrt{\frac{F}{E} \left(\frac{D}{t_{effective}} \right)^3}$$

5.5 Construction of water retaining structures

Once the size of the gravel trap, settling basin and forebay have been calculated, the type and dimensions of the walls and floors need to be determined. For micro-hydro schemes, stone masonry in cement mortar is generally the most appropriate and economic option. The construction details and procedures for this type of structure are as follows:

- The ground should first be excavated according to the basin shape and then be well compacted using a manual ram.
- Since these are water retaining structures, 1:4 cement sand mortar should be used for the walls and floors as discussed

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Example 5.2 Sizing of air vent

Consider a 300 mm steel penstock of 3 mm wall thickness connected to a turbine that can take 2501/s. The penstock is above ground.

Q = 250 1/s

E = 2.0 x 10⁵ N/mm² (from Table 6.2)

D = 300 mm

t = 3mm

$t_{effective}$ = 1.27 mm (from Section 6.6)

F = 10 for above ground pipe

Then $d^2 = 250 \sqrt{10 / 2\text{x}10^5(300 / 1.27)^3}$

Or, d = 80 mm, i.e. the minimum internal diameter of the air vent should be 80 mm.

---

in Chapter 4.

- The walls should be built such that they are a minimum of 300 mm thick at the top and increase with depth as shown in Figure 5.9. Note that in this figure, the wall surface on the water retaining side is vertical. This increases the stability of the structure since for a constant depth the water pressure is larger than the soil pressure.

Box 5.2 Ghandruk peaking reservoir

The design flow of the 50 kW Ghandruk micro-hydro scheme is 35 1/s but the dry season flow is only 20 1/s. Hence during the dry season the power output was about 39 kW until a peaking reservoir was built in 1994. Such a peaking reservoir became necessary because of the demand for full power (50 kW) in the village during the mornings and evenings. There is nominal power demand in the afternoon and even less during night time.

A suitable site for the reservoir was located on the terrace immediately downstream of the old forebay. Photographs of the peaking reservoir during and after construction can be seen below.

Photo 5.15 Ghandruk peaking reservoir after construction

Photo 5.14 Ghandruk peaking reservoir during construction

The walls and the floor of the reservoir were constructed of stone masonry in cement mortar. The reservoir has been sized such that it can provide the design flow of 35 1/s for at least six hours (3 hours in the morning and 3 hours in the evening). The calculations are as follows:

Storage volume required = (Design flow - dry season flow) x 6 (hr) x 60 (min/hr) x 60(s/min) = (35 - 20) x 6 x 60 x 60 = 324,000 litres or 324 m^3

Hence a reservoir with a minimum storage capacity of 324 m^3 is required to be able to provide 50 kW for six hours. The actual dimensions of the reservoir are as follows:

- Length = 20 m .   Width = 20 m
- Depth = 1.2 m

Hence, the storage volume is 480 m^3, about 48% larger than the minimum required volume. Note that the old forebay has now become redundant. Also, since the peaking reservoir is downstream of the old forebay, a few metres of head are lost. This is compensated by slightly increasing the design flow.

Any sediment deposited in the reservoir is manually cleaned during the annual maintenance period. A flushing facility was not installed because of potential erosion and landslide problems due to the large volume of water involved.

At present the power plant is shut when there is no demand for power, then the reservoir is allowed to fill.
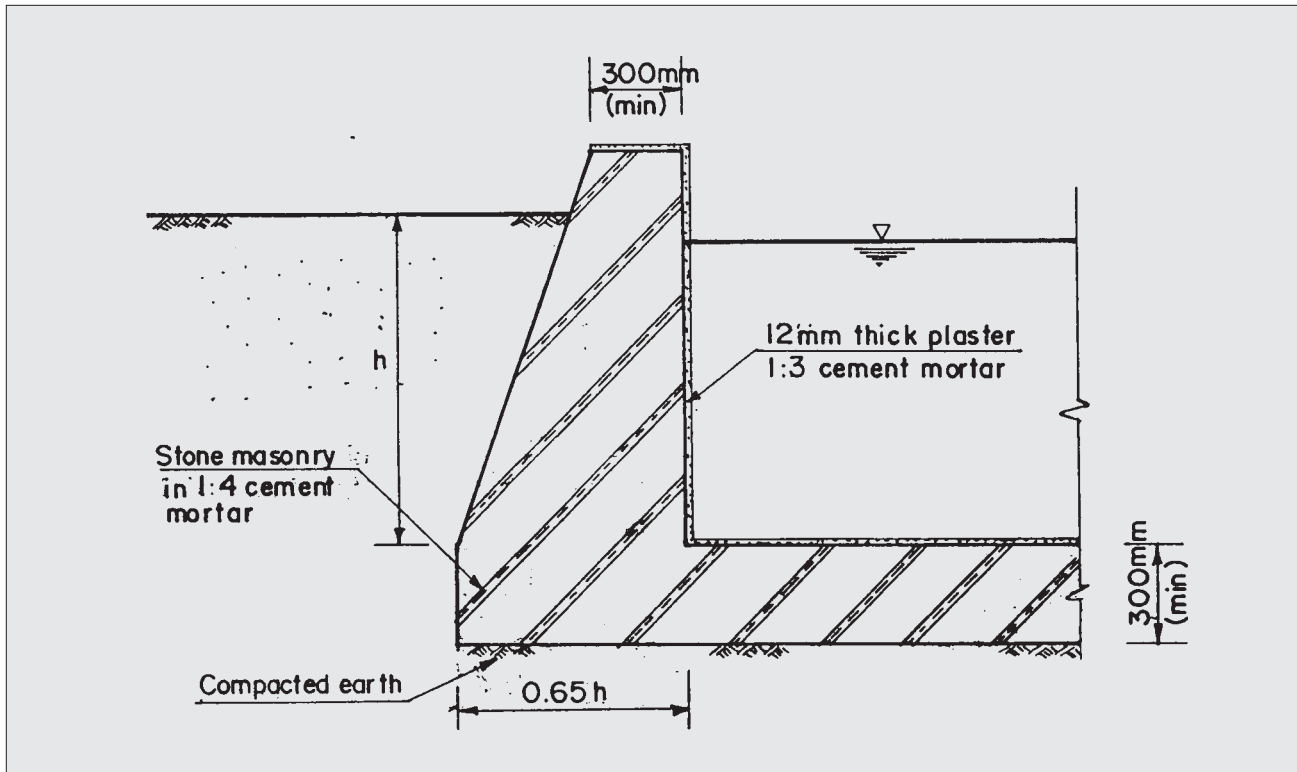
Figure 5.9 Wills and floors of water retaining

If the vertical face is towards the soil, the water volume in the basin will increase but structural stability will be slightly reduced.

- The internal surface (water retaining surface) of the walls and the surface of the floor should be plastered using 1:3 cement sand mortar to a thickness of 12 mm. This significantly minimises the likelihood of seepage.
- Finally, the walls and the floor should be cured as discussed in Chapter 4. Another option is to use reinforced concrete for the floor and walls. However this is more expensive and also requires skilled labour, so is not generally recommended for micro-hydro schemes.

5.6 Checklist for gravel trap, settling basin and forebay work

- Can the gravel trap, settling basin and the forebay or at least two of these structures be combined together?
- Are these structures located such that excess water can be spilled safely, without causing erosion or stability problems?

- Is the settling basin sized such that the emptying frequency is once to twice daily during high flows? Also, does d_{limit} correspond to the head and turbine type? Refer to Section 5.3.3.
- Is secondary settling required at the forebay? Is the forebay large enough for manual cleaning and is a spillway incorporated in this structure? Has the submergence head been checked?
- Once these structures have been sized, refer to Section 5.5 for the construction details

6. Penstocks

6.1 Overview

A penstock is a pipe that conveys the flow from the forebay to the turbine. The penstock pipe starts where the ground profile is steep as shown in Photograph 6.1. The penstock pipe usually constitutes a significant portion of the total micro-hydro construction cost. Therefore it is worthwhile optimising the design. This involves a careful choice of: pipe material, such as mild steel or HDPE; an economical diameter such that the head loss is within acceptable limits; and wall thickness so the pipe is safe for the design head and any surge effect that may result from sudden blockage of the flow. The potential energy of the flow at the forebay is converted into kinetic energy at the turbine via the penstock pipe. Since the flow is conveyed under pressure it is important for the pipe design to be safe. Cases have been reported where the penstock pipes have burst. Since the penstock is on steep ground slopes, such pipe burst can instantaneously cause landslides and other stability problems. Furthermore, penstock installation is often challenging and requires safe and careful work as shown in Photograph 6.2.

Photo 6.1 Penstock alignment of the 36 kW Jharkot micro-hydro scheme, Mustang. Nepal

6.2 Selection of the penstock alignment

6.2.1 SITE WORK
Selection of the penstock alignment at site should be based on the following criteria:

Forebay location
The penstock starts at the forebay, for which location criteria are given in Section 5.1.3. In addition, the forebay location should be chosen to optimise the lengths of headrace and penstock whilst achieving the required power output from the scheme. Penstock pipe is generally more expensive than headrace canal, therefore in most cases the forebay location should be chosen to give the minimum penstock length. However, sometimes a longer penstock may be economic, to avoid the need for the headrace to cross an unstable slope.

Practical ground slope
An ideal ground slope for the penstock alignment is between 1:1 and 1:2 (V:H). The flatter the ground slope the less economic is the penstock since a longer pipe length is required for a lower head.

Photo 6.2 Penstock alignment of the 50 KW Barpak micro-hydro scheme, Gorkha, Nepal

Although a steep slope minimises the penstock length, it will be difficult to manually lay the penstock, construct support piers and anchor blocks if the slope is greater than 1:1. Therefore, for penstock alignments on slopes steeper than 1:1, the added site installation cost may outweigh the savings made on the pipe costs.

Avoid a penstock profile that starts at a gentle slope and then becomes steeper, because of the risks of negative surge pressures causing sub-atmospheric pressure. See Section 6.5. For micro-hydro schemes with less than 20 kW of installed capacity, the ground profile of the penstock alignment can be measured using an Abney Level as discussed in Micro-hydro Design Manual (Ref. 1). For larger micro-hydro schemes, the use of a theodolite and a professional surveyor is recommended. This is because if the prefabricated bends do not fit at site due to survey errors, additional cost and time will be required to amend these, especially if the site is located far from the roadhead and the pipes are flanged. Note that some slight adjustment can be made if the pipes are welded at site.

Errors in the design head calculation (due to survey errors) will result in either oversizing or undersizing the electro-

Photo 6.4 penstock installation is often challenging and requires safe and careful work.

mechanical units, which will also increase the project cost, either in terms of lost power production or in extra cost for the oversized units.

Minimum number of bends
Bends increase the headless and require additional anchor blocks. Therefore the selected alignment should be as straight as possible, both in plan and elevation. Note that small bends can be avoided by varying the support pier heights for the exposed section and the trench depth for the buried section.

Space for powerhouse area
The chosen alignment should be such that it is possible to construct a powerhouse at the end of the penstock. A river terrace well above the flood level is ideal for the powerhouse area. A route that is otherwise suitable for the penstock alignment but does not allow for the construction of the powerhouse is inappropriate.

Stability
Since the penstock alignment is on steep ground slopes and the pipe is under pressure, it is important for the alignment to be on stable ground. Any ground movement can damage the

Photo 6.3 Excavation to reduce the cost of the penstock and reduce the need for anchors (Siklis)

pipe, support piers and anchor blocks, and in case of pipe bursts unstable slopes will cause further erosion and landslides. Slope stability is discussed in detail in Chapter 9.

Other site specific conditions
Apart from the above criteria, there may be other site specific conditions that dictate the penstock alignment. For example, if the alignment crosses a local trail, this section should either be buried or high enough above the ground such that people and cattle can walk underneath. The Jhankre mini-hydro penstock alignment is an example where a site specific

Photo 6.5 Penstock alignment high above the ground to allow access for people and cattle, Jhankre mini-hydro

condition governed the penstock alignment. There is cultivated land between the intake and powerhouse of this scheme, so the penstock was aligned mostly along the edge of the cultivated land. At one section this was not possible and the alignment had to traverse the cultivated fields. Since it was not possible to bury the pipes at this section (due to downstream alignment), a few of the support piers were sized to be 2 m high as shown in Photograph 6.5. This resulted in a dear space of about 2.5 m under the penstock, which allows farmers and cattle to walk underneath.

6.2.2 PROFILE OF THE SELECTED ALIGNMENT
Based on the site survey, a plan and profile of the penstock alignment should be prepared at the design office as follows:
- The ground profile should first be drawn using an appropriate scale. Same scale should be used for both horizontal and vertical lengths so that the bend angles are true angles, which minimises the likelihood of errors. If the alignment also has horizontal bends, then a plan view should also be prepared to show horizontal bend angles.
- Once the ground profile has been prepared, the penstock pipe should be drawn on it such that the number of bends is kept to a minimum. In general for above ground alignment the support pier height should be minimised unless some of them need to be increased to avoid small angle bends. Similarly, excavation should be minimised for the buried section unless deeper trenches are required at short sections to avoid small angle bends. Optimising the alignment will require some iterations. An example of a penstock profile is shown in Figure 6.1.
- For above ground penstock sections, a minimum ground clearance of 300 mm is recommended to keep the pipe dry and for ease of maintenance such as painting.
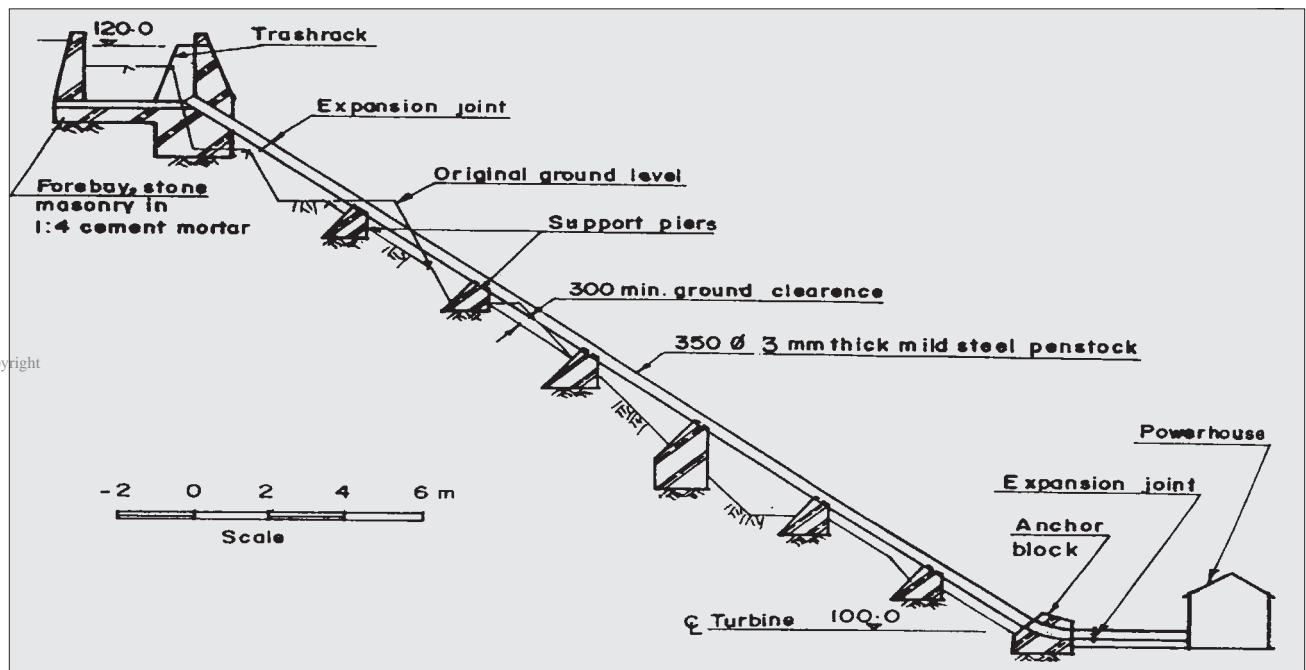
Figure 6.1 Typical penstock profile

TABLE 6.1 **Advantages and disadvantages of different penstock materials**

MATERIAL	OCCURRENCE	ADVANTAGES	DISADVANTAGES
Steel	Most common	• Very widely available	• Heavy, transport cost can be high
		• Pipes can be rolled to almost any size	• Rigid, bends have to be specially fabricated at work shop
		• Can go up to almost any thickness	• Has corrosion problems
		• Easy to join and can with stand high pressure	• Can be expensive
			• Can have high surge requirement
HDPE	Fairly common	• Does not corrode	• Difficult to join (solvent not available)
eters, in		• Light and hence easy to transport	• Available in discrete diam- maximum diameter available Nepal is 315 mm OD
		• Flexible (accommodates small bends)	• Must be buried (due to ultra violet (UV) and thermal degradation) and carefully backfilled.
		• Low surge requirement	• Limited pressure ratings available in Nepal (up to 10 kg/cm^2 which is 100 m head)
			• More expensive than mild steel for large diameters and high pressures
PVC	Common in other countries but not yet used in Nepal	• Does not corrode	• Brittle, can be damaged during transportation
		• Light	• Must be buried (due to UV and thermal degradation) and carefully backfilled.
		• Easy to install (solution available)	• Unsuitable in freezing condi tions to join pipes)
			• Not available in diameters larger than 250 mm in Nepal
			• Limited pressure ratings available in Nepal (up to 10 kg/cm^2 = 100 m head)
			• More expensive than mild steel at high pressures

■ For buried penstock sections, a minimum soil cover of 1 m is recommended as in the case of HDPE headrace pipe, and the trench details should be similar to those shown in Figure 4.8 (Chapter 4).

6.3 Pipe materials

In Nepal the most commonly used penstock pipe materials are mild steel and HDPE. Rigid or unplasticised PVC (uPVC) is another option that has been used in other countries such as Peru (see Chapter 10, Innovations) and Sri Lanka, but has not yet been used in Nepal. Table 6.1

The decision as to which pipe material to use for the penstock can be based on Table 6.1, especially in Nepal. When in doubt, it is recommended that the designer undertake preliminary designs for all pipe materials available and compare the costs.

To minimise costs, for long penstock alignments HDPE pipes can be used for the upstream length where the head is relatively low (see Box 6.1). Standard couplings are available to join HDPE and mild steel pipes as shown in Photograph 6.6 and Figure 6.2.

Although steel pipe for micro-hydro in Nepal has normally been specially manufactured locally, standard steel pipes may be cheaper in some cases. Details of such pipes are given in Appendix B.

6.4 Pipe diameter

Once the penstock alignment and pipe material have been decided on, the design involves choosing the diameter and pipe thickness. Selecting an appropriate pipe diameter is discussed in this section and the wall thickness is discussed in Section 6.6.

Note that with a few exceptions the sizing of the penstock diameter is similar to that of a headrace pipe discussed in Chapter 4. For simplicity, the entire penstock diameter selection process is included in this section.

1. Choose a pipe size such that the velocity, V, is between 2.5 m/s and 3.5 m/s. In general, a velocity lower than 2.5 m/s results in an uneconomically large diameter. Similarly, if the velocity exceeds 3.5 m/s, the headless can be excessive and hence uneconomical in the long run due to loss in power output. Furthermore, higher velocities in the penstock will result in high surge pressure as will be discussed later.

Note that compared to the headrace pipe, higher velocities can be allowed in the penstock pipe since it conveys sediment free water.

For steel penstocks, it may be economical to choose the diameter so that there is no wastage from standard size steel sheets. For HDPE or PVC, available sizes must be selected. Pipes are normally specified by outside diameter, so two times wall thickness must be subtracted to obtain the internal diameter. Standard pipe sizes are given in Appendix B.

2. Calculate the actual velocity:
$V = 4Q / \Pi d^2$
where:
V is velocity in m/s.
Q is design flow in m^3/s.
d is the pipe internal diameter in m.

3. Calculate the headloss in the pipe length based on the inlet, wall friction, bends, valves and exit loss as follows: Total head loss = wall loss + turbulence losses / Wall losses are calculated as follows: First determine the roughness value, 'k' in mm from Table 4.3 (Chapter 4). Then use the Moody Chart in Figure 4.10 (Chapter 4)
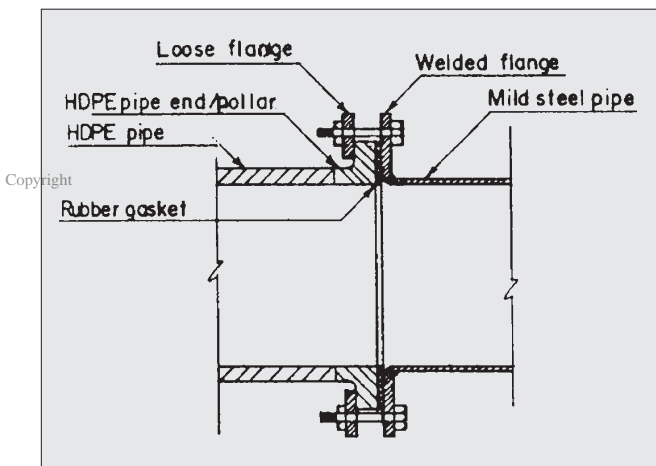
Figure 6.2 Typical HPDE-mild pipe coupling
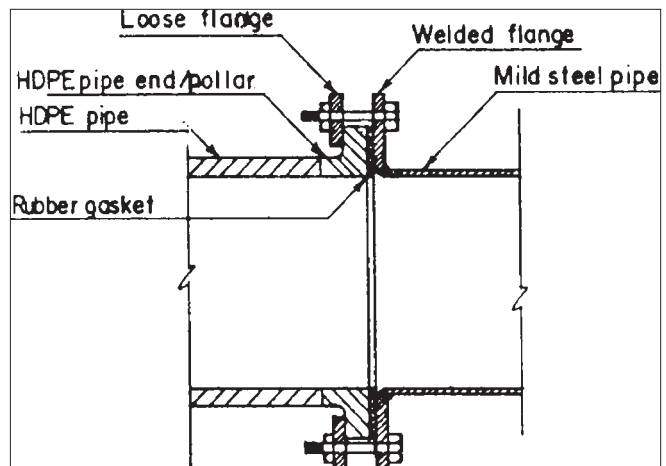
Photo 6.6 HDPE-mild steel coupling, Note that except for the final length, the HDPE pipe is buried, Jhong micro-hydro scheme, Mustang, Nepal

to find the corresponding friction factor f for the selected pipe material, diameter and the design flow. The wall loss can now be calculated from either of the following equations: ,

$$h_{wall\ loss} = f \frac{LV^2}{d \times 2G}$$

$$h_{wall\ loss} = \frac{fL \times Q2}{12d^2}$$

Turbulence losses are calculated as follows:

$$h_{turb\ loss} = V^2 /29\ (K_{entrance} + K_{bend} + K_{contraction} + K_{valve})$$

where loss coefficients, K, are as shown in Table 4.4.

4. In general ensure that total head loss for the design flow is between 5% and 10% of the gross head, i.e. 95% to 90% penstock efficiency.

5. If the head loss is higher than 10% of the gross head, repeat calculations with larger diameter. Similarly, if the head loss is less than 5% the pipe diameter may be uneconomic, therefore repeat calculations using smaller diameters.

Note that in exceptional cases a less efficient penstock may be more economic, such as when the power demand is limited, the penstock pipe is long and there is abundant flow in the river even during the low flow season. In such cases, a higher flow can be allowed in a smaller diameter pipe allowing a higher head loss, which is compensated by the flow. Hence savings can be made in the cost of pipes as discussed in Box 6.1. However, this approach should be justified by a detailed economic analysis.

On the other hand for larger hydropower projects (mini/small or large) the diameter of the penstock pipe is selected on the basis of a detailed financial analysis. The process involves calculating pipe costs and head losses for a range of diameters and comparing these with the present value of energy losses within the economic life of the power plant. As long as the net present value remains significant (i.e., benefits outweighs costs) the diameter is increased. Alternatively, for a given diameter both the penstock pipe cost (and hence the total project cost) and the cost of energy lost due to head loss can be calculated. Then, the optimum penstock diameter would be the diameter corresponding to minimum total costs as shown graphically in Figure 6.2. It should also be noted that there are generally other criteria as well, such as availability of steel plate widths in the market, local fabricating capabilities

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BOX 6.1 HDPE-mild steel penstock, Bhujung micro-hydro scheme, Nepal

The 80 kW Bhujung micro-hydro scheme (Bhujung, Lamjung, Nepal) designed by BPC Hydroconsult for Annapurna Conservation Area Project (ACAP) is currently under construction. The design discharge of the scheme is 150 1/s, the total penstock length is 860 m and the gross head is 104 m. Since it is a long penstock, HDPE pipes are used for the initial length and mild steel pipe for the downstream end as follows:

PENSTOCK TYPE (m)	THICKNESS (mm)	GROSS HEAD AT d/s END (m)	LENGTH (m)	HEADLOSS(m)
315 mm diameter, Class II HDPE	10.3	26.8	310	4.6
315 mm diameter, Class III HDPE	16.1	41.8	180	3.2
300 mm diameter, mild steel	3	64.8	260	3.5
300 mm diameter, mild steel	4	103.9	110	1.5
Total headloss				12.8
% headloss				12.3%

Note that the total head loss is about 12%: though more than 10%, in this case a higher headloss was found to be economic. This is because the estimated power demand for the village is 80 kW and the source river (Midim Khola) of the scheme has significantly higher flows, even during the low flow season (minimum flow of 0.5 m³/s in April) than the required design discharge. If the penstock pipe diameter was sized for less than 10% headloss (i.e. by increasing the diameters and decreasing the design discharge such that power output = 80 kW), the pipe cost including laying and transportation would have increased by 15%. Similarly if mild steel was used for the entire alignment (for headloss = 12%), the pipe cost would have increased by about 30%. Note that settling basin and headrace costs need to be included in the optimisation.

limiting diameters beyond a certain range which could affect the optimisation process. An example of Penstock Optimisation is shown in Example 6.2

6.5 Surge calculation

6.5.1 GENERAL
The thickness of the penstock pipe is determined by the gross and surge heads of the scheme. It is therefore important to have some understanding of the concept of surge before calculating the pipe wall thickness.

A sudden blockage of water or rapid change in velocity in the penstock (or any pipe that has pressure flow) results in very high instantaneous pressure. This high pressure is known as 'surge' pressure or often referred to as "waterhammer". Surge pressure travels as positive and negative waves throughout the length of the penstock pipe.

Water hammer occurs as the surge wave travels from the source or the origin of the disturbance, along the pipeline until it strikes some boundary condition (such as a valve or other obstruction) and is then reflected or refracted. If the pipe is strong enough to withstand the initial surge effect, the pressure will ultimately dissipate through friction losses in the water and pipe wall as well as through the forebay. The speed of the surge wave (wave velocity) is dependent on such factors as the bulk modulus of water, flexibility of the pipe and the ratio of pipe diameter to wall thickness.

In hydropower schemes, positive surge characteristics are different for different types of turbines. Surge head calculations for the two most common turbines used in micro-hydro schemes are discussed here. Note that these calculations are based on the initial (i.e. undampened) positive surge head. In practice there will be some damping of the surge pressure as the wave travels along the pipe, and whilst the pressure fluctuation is uniform in the lower portion it diminishes gradually to zero at the forebay, as shown in Figure 6.3. However, the pipe is normally designed for static head plus constant positive surge over the full penstock length.

Note that the negative surge can produce dangerous negative (sub-atmospheric) pressure in a penstock if the profile is as in Figure 6.3. Once the negative pressure reaches 10 metres the water column separates, and subsequent rejoining will cause high positive surge pressure sufficient to burst the penstock. Sub-atmospheric pressures less than 10 metres can cause inward collapse of the pipe wall, so should also be avoided. If there is any possibility of negative pressure the pipe wall thickness must be checked for buckling (see Section 6.6.2).

To avoid negative pressure, move the forebay to Point A in Figure 6.3. Alternatively take measures to reduce the surge pressure.
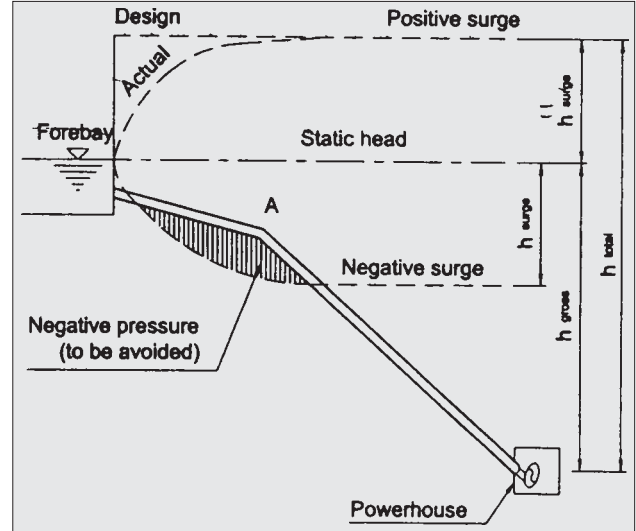
Figure 6.3 Surge pressures

"Bursting disc" technology could provide a reliable means of safely releasing excess head in case of surge pressure without increasing the pipe thickness (which is the convention). This is discussed in Chapter 10.

6.5.2 PELTON TURBINE
For a Pelton turbine use the following method to calculate the surge head:

1. First calculate the pressure wave velocity 'a' using the equation below.
   $$a = 1440 / \sqrt{1 + (2150 \times d / E \times t)} \text{ m/s}$$

where:
E is Young's modulus in N/mm^2. The value of Young's modulus for mild steel, PVC and HDPE can be seen in Table 6.2.

d is the pipe diameter (mm) t is the nominal wall thickness (mm), not $t_{effective}$

2. Then calculate the surge head (h_{surge}), using the following equation: $h_{surge} = av/g \times 1/n$ where: n is the total no. of nozzles in the turbine(s).

Note that in a Pelton turbine it is highly unlikely for more than one nozzle to be blocked instantaneously. Therefore, the surge head is divided by the number of nozzles (n). For example if a penstock empties into two Pelton turbines with two nozzles on each turbine, n = 4
The velocity in the penstock (V) is: $V = \dfrac{4Q}{\pi d^2}$

3. Now calculate the total head: $h_{total} = h_{gross} + h_{surge}$

4. As a precaution, calculate the critical time, T_c, from the following equation: $T_c = (2L)/a$
where:
T_c is the critical time in seconds,
L is the length of penstock in m,
'a' is the wave velocity calculated earlier.

If the turbine valve closure time, T, is less than T_c, then the surge pressure wave is significantly high. Similarly, the longer T is compared to T_c, the lower the surge effect.

Note that this calculation is based on the assumption that the penstock diameter, material and wall thickness are uniform. If any of these parameters vary, then separate calculations should be done for each section.

Also note that when the $T = T_c$, the peak surge pressure is felt by the valve at the end of the penstock. If a pressure gauge is not installed upstream of the valve, a valve closure time of at twice the critical time (i.e., $T \geq 2T_c$) is recommended.

The design engineer should inform the turbine manufacturer of the closure time (T) so that if possible the manufacturer can choose the thread size and shaft diameter such that it will be difficult to close the valve in less than twice the calculated closure time. The operator at the powerhouse should be made aware of this closure time and the consequences of rapid valve closure.

If the gross head of the scheme is more than 50 m, it is recommended that a pressure gauge be placed just upstream of the valve. Compared to the cost of the turbine and the penstock, the cost of such a device is low (about US$50 in Nepal) and is worth the investment. When the operator closes or opens the valve, his speed should be such that there is no observable change in the pressure gauge reading

6.5.3 CROSSFLOW TURBINE

In a crossflow turbine, instantaneous blockage of water is not possible since there is no obstruction at the end of the manifold (i.e. crossflow turbine has a rectangular bore opening instead of a nozzle). Therefore, surge pressure can develop only if the runner valve is closed rapidly. For a crossflow turbine use the following method to calculate the surge head:

1. Calculate the pressure wave velocity 'a' (using the same equation as for Pelton turbine).
2. Now calculate the critical time T_c, similar to the Pelton turbine case: $T_c = (2L)/a$
3. Choose a closure time, T (in seconds), such that: $T > 2T_c$
   Similar to the Pelton turbine case, the design engineer should inform the turbine manufacturer of the closure time (T) and the operator at the powerhouse should be made aware of this closure time.
4. Now calculate the parameter 'K[1]' using the following equation:
   $K = [L \times V/g \times h_{gross} \times T]^2$
5. Calculate surge head by substituting the value of 'K' in the equation below:
   $h_{surge} = [K/2 \pm \sqrt{K + k^2/4}] h_{gross}$

If 'K' is less than 0.01 [i.e. closure time (T) is long enough], then the following simplified equation can also be used:

$h_{surge} = h_{gross} \sqrt{K}$

Note that if the valve is closed instantaneously, the entire length of the penstock will experience a peak pressure as follows:

$h_{surge} = av/g$ (i.e., same as in the case of Pelton turbine with one nozzle.)

However, in practice it will take at least five to ten seconds for the operator to close the valve, therefore in a crossflow turbine instantaneous surge pressure is not a problem.

If the gross head of the scheme is more than 50 m, a pressure gauge should be placed upstream of the valve to control its closing/opening speed, as in the case of a Pelton turbine.

6.5.4 QUICK METHOD FOR SMALL SCHEMES WITH CROSSFLOW TURBINES

For small micro-hydro schemes using crossflow turbines (such as milling schemes) where the power output is less than 20 kW and the gross head is less than 20 m, this quick method may be used. Add 20% to the gross head to allow for surge head,

i.e. $h_{surge} = 1.2 \times h_{gross}$. This results in a more conservative value for the surge head but its contribution to the increase in the thickness would be insignificant since the h_{gross} is low.

6.6 Pipe wall thickness

6.6.1 POSITIVE INTERNAL PRESSURE

Once the surge head has been determined, the nominal wall thickness (t) can be calculated as follows:

1. If the pipe is mild steel, it is subject to corrosion and welding or rolling defects. Its effective thickness ($t_{effective}$ will therefore be less than the nominal thickness. Therefore, for mild steel, assume a nominal thickness (t) and to calculate $t_{effective}$ use the following guidelines:
a) Divide the nominal wall thickness by 1.1 to allow for welding defects.
b) Divide the nominal wall thickness by 1.2 to allow for rolling inaccuracy of the flat sheets.
c) Since mild steel pipe is subject to corrosion,

The recommended penstock design life is 10 years for schemes up to 20 kW, 15 years for schemes of 20-50 kW, and 20 years for schemes of 50-100 kW. These figures may be adjusted on the basis of a financial analysis. For example the effective thickness of a 3 mm thick mild steel pipe designed for a 10 years life is:

$$t_{effective} = \frac{3}{1.1 \times 1.2} - 1 = 1.27 \text{ mm}$$

From this example it is clear that if a mild steel pipe used, the nominal wall thickness (t) should be at least 3 mm.

Note that this does not apply for HOPE pipes: their effective thickness is the nominal wall thickness of the pipe. A low temperature correction factor may apply to PVC pipes, refer to the pipe manufacturer: if the temperatures are sub-zero, $t_{effective}$ may be as low as 0.5t. Apart from protection from ultraviolet degradation, this is another reason to bury PVC pipes at high altitude.

2. Now calculate the safety factor (SF) from the following equation:

$$SF = -\frac{200 \times t_{effective} \times S}{h_{total} \times d}$$

where:

$t_{effective}$ is the effective thickness and d is the internal diameter of the pipe. Note that same units (m or mm) should be used for both $t_{effective}$ and d since they cancel out in the above equation. S is the ultimate tensile strength of the pipe material in N/mm². Values of S and other useful parameters are shown in Table 6.2.

h_{total} is the total head on the penstock as follows:
$h_{total} = h_{gross} + h_{surge}$

3. For mild steel or PVC pipes:

If SF < 3.5, reject this penstock option and repeat calculation for thicker walled option. However, SF ≥ 2.5 can be accepted for steel pipes if the surge head has been calculated accurately and all of the following conditions are met:

a) There are experienced staff at site who have installed penstock pipes of similar pressures and materials.
b) Slow closing valves are incorporated at the powerhouse and the design is such that a sudden stoppage of the entire flow is not possible.
c) Damage & safety risks are minimal. For example even if the pipe bursts, it will not cause landslides or other instability problems in the short run.
d) Careful pressure testing to total head has been performed before commissioning.

For HDPE pipes:
HDPE pipes are available in discrete thicknesses based on the pressure ratings (kg/cm²) or static heads. The designer should set SF ≥. 1.5 and calculate $t_{effective}$ (note that t = $t_{effective}$ for HDPE).

Then from the manufacturer's catalogue the actual thickness should be chosen such that it is equal to or larger than the calculated $t_{effective}$. The Safety Factor should then be checked using the actual thickness. For HDPE pipes, it is recommended that the Safety Factor always be at least equal to 1.5.

6.6.2 NEGATIVE INTERNAL PRESSURE
Check the pipe wall thickness for buckling if the negative surge can produce negative internal pressure in the pipe. Note that the negative pressure must not exceed 10 metres head, see Section 6.5.1. The shape of the negative surge pressure profile cannot be accurately determined: assume it is horizontal in the lower half of the penstock, and diminishes gradually in the upper half to zero at the forebay, see Figure 6.3.

In order to provide an adequate factor of safety against buckling, the minimum pipe wall thickness is given by:
$t_{effective} \geq d(FP / 2E)^{0.33}$

where:
$t_{effective}$ is the effective pipe wall thickness, mm
d is the pipe internal diameter, mm
F is factor of safety against buckling (2 for buried penstock and 4 for exposed penstock)
P is the negative pressure, N/mm² (10 m head ≅ 0.1 N/mm²)
E is Young's modulus for the pipe material, N/mm² (from Table 6.2).

If the steel quality is uncertain it is best to ask for samples and have them independently tested at laboratories. Properties of PVC and HDPE vary considerably; they should be confirmed from manufactures' catalogues or by laboratory tests.

6.7 Pipe jointing

6.7.1 GENERAL
Individual mild steel penstock pipes can be joined at site by two conventional methods, namely site welding and via flanges. Each of these methods has its own advantages and disadvan-tages as discussed in Table 6.3.

6.7.2 SITE WELDING
This involves transporting a welding machine and diesel or petrol to site, then joining the pipes by welding together the ends as shown in Photograph 6.10.

TABLE 6.2 **Physical characteristics of common materials**

MATERIAL	YOUNG'S MODULUS (E) N/MM²	COEFFICIENT OF LINEAR EXPANSION (á) °C	ULTIMATE TENSILE STRENGTH (S) N/mm²	UNIT WEIGHT (Y) KN/m³
Steel (ungraded)	2.0×10^5	12.5×10^{-6}	320	77
Steel to IS 226 / 75 or IS 2062/84	2.0×10^5	12.5×10^{-6}	410	77
PVC	2750	$(20 - 60) \times 10^{-6}$	35-55	14
HDPE	1000	$(140 - 240) \times 10^{-6}$	20-35	9.3

Example 6.1 Sizing of Galkot penstock pipe

The required data for the design of the Galkot penstock are as follows:

- Q = 42 l/s (calculated in Example 4.2)
- h_{gross} = 22m
- two vertical bends, Þ = 20° & 42°, both mitred.
- Penstock material: uncoated mild steel, 35 m long and flange connected.
- Turbine type: crossflow

Refer to Drawing 420/04/3A01 in Appendix C for the ground profile and bend angles.

Pipe diameter calculation

Set V = 3.5 m/s
Calculate the internal pipe diameter:

$$d = \sqrt{4Q / \Pi V}$$
$$d = \sqrt{(4 \times 0.421)/(\Pi \times 3.5)}$$
$$= 0.391 \text{ m}$$

Calculate wall loss:
From Table 4.3 choose k = 0.06 mm for uncoated mild steel.

$$k/d = 0.06 /391$$
$$= 0.00015$$
$$1.2Q/d = (1.2 \times 0.421) / 0.391$$
$$= 1.29 \text{ m}$$

f = 0.013 (Moody Chart, Chapter 4)

$$h_{wall\,loss} = f (LV^2 / d \times 2g) = 0.013 (35 \times 3.5^2 / 0.391 \times 2g)$$
$$h_{wall\,loss} = 0.73 \text{ m}$$

Inlet loss:
$$K_{entrance} = 0.5 \text{ for this case (Table 4.4)}$$
$$h_{Inlet\,loss} = K_{entrance} \times V^2 / 2g$$
$$= 0.5 \times 3.5^2 /2g$$
$$= 0.31 \text{ m}$$

Note that Exit loss = 0 since the flow at the end of the penstock is converted into mechanical power by rotating the turbine runner.

For mitred bends, from Table 4.4

$$è = 22°, K_{bend} = 0.11$$
$$è = 42°, K_{bend} = 0.21$$

Bend losses $= (0.11 + 0.21) \times (3.5^2 / 2g)= 0.20\text{m}$
Total head loss $= 0.73 \text{ m} + 0.31 \text{ m} + 0.20 \text{ m} = 1.24\text{m}$
% head loss $= 1.24 / 22 \times 100 \% = 5.6\% < 10\%$ OK.

Although the calculated pipe diameter was 391 mm internal, Drawing 420/04/3C02 in Appendix C shows the outside pipe diameter to be 388 mm, i.e. internal diameter 388 - 2 x 3 = 382 mm. This was chosen to correspond to a 1200 mm standard plate size (1200/Π = 382). In practice, however, the manufacturer proposed 400 mm internal diameter pipes for the same cost, because this corresponded to the plate size that he had..

Pipe wall thickness calculations

Calcutate the pressure wave velocity 'a'

$$a = 1440 / \sqrt{1 + [(2150 \text{ xd}) / (E \times t)]}$$
E = 2.0 x 10^5 N/mm^2 for mild steel (Table 6.2)
d = 400 mm
t = 3 mm
$$\text{or, } a = 1440 /\sqrt{1 + [(2150 \times 400) /(2.0 \times 10^5 \times 3)]}$$
or, a = 923 m/s

Now calculate the critical time:

$$T_c = 2L / a$$
$$= 2 \times 35 / 923$$
$$= 0.08 \text{ s .}$$

Note that it would be impossible to close the valve in the powerhouse in 0.08 seconds!
Choose closure time T = 10 s > 2 T_c = 0.16 s

$$K = [LV / gh_{gross}T]^2$$
$$= [35 \times 3.5 / g \times 22 \times 10]^2$$
or, K = 0.003. Since K is less than 0.01
$$\text{or, } h_{surge} = h_{gross} \times ök = 22 \times ö 0.003$$
$$\text{or, } h_{surge} = 1.20 \text{ m}$$

$$\text{or, } h_{total} = h_{surge} + h_{gross} = 23.2 \text{ m}$$

The pipes were manufactured by welding (1.1) rolled flat steel plates (1.2). 1.5 mm has been subtracted to allow for at least 15 years of life.
$$t_{effective} \quad 3 / (1.1 \times 1.2) -1.5 = 0.77 \text{ mm}$$

Now check the safety factor:
$$SF = (200 \times t_{effective} \times S) / (h_{total} \times d)$$
S = 320 N/mm^2 for ungraded mild steel (Table 6.2)
$$SF = (200 \times 0.77 \times 320) / (23.2 \times 400)$$
$$SF = 5.3 > 3.5 \text{ OK .}$$

Note that the safety factor is higher than required but the minimum recommended thickness for flat rolled mild steel pipe is 3 mm.
The Galkot penstock alignment before and after pipe installation can be seen in Photographs 6.7 and 6.8 respectively.

Photo 6.7 Excavation for penstock alignment, Galkot

Photo 6.8 Penstock alignment, Galkot

Example 6.2 Sizing of Jhankre Penstock

The required data for the design of the Jhankre penstock are as follows:
- Q = 450 1/s
- h_{gross} =   180 m
- ten vertical bends, è = 69° , 23°, 26°, 37°, 40°, 2°, 3°, 12°, 8° & 3°, all mitred.
- Penstock material: mild steel, flat rolled and site welded, 550 m long. High quality steel plates were bought and tested for tensile strength at the laboratory. Minimum tensile strength, S = 400 N/mm^2 was ensured through the tests.
- Turbine type: 3 Pelton turbines with 2 nozzles in each turbine, therefore n = 3 x 2 = 6.

Calculate the required pipe diameter and wall thickness. Note that since the penstock is long, it will be economic to decrease the thickness at lower heads.

Pipe diameter calculation

Try V = 3.5 m/s.

Calculate the internal pipe diameter:

d $= \sqrt{4Q / \Pi V}$ $= \sqrt{(4 \times 0.450) / (\Pi \times 2.5)}$ =   0.405 m

Calculate wall loss:

From Table 4.3 choose k = 0.06 mm for uncoated mild steel.

   k /d   = 0.06 / 405   = 0.000148

   1.2Q / d   = 1.2 (0.450 /0.405)   = 1.33

   f = 0.0145 (Moody Chart, Figure 4.7)

   $h_{wall loss}$ $= f (LV^2 / d \times 2g)$ = 0.0145 (550 x 3.5^2) / 0.405 x 2g

   $h_{wall loss}$ = 12.29 m

Inlet loss:

$K_{entrance}$ = 0.2 for this case (similar to fourth entrance profile in Table 4.4)

$h_{inlet loss}$ $= K_{entrance}$ x V^2 /2g   = 0.2 x 3.5^2 / 2g

$h_{inlet loss}$ = 0.12 m

Exit loss = 0.

For mitered bends, interpolate from Table 4.4 :

     for θ   = 69°, K_{bend} = 0.34

     for θ   = 23°, K_{bend} = 0.11

     for θ   = 26°, K_{bend} = 0.13

     for θ   = 37°, K_{bend} = 0.18

     for θ   = 40°, K_{bend} = 0.20

for $\theta = 2°$, $K_{bend} = 0.02$
for $\theta = 3°$, $K_{bend} = 0.02$
for $\theta = 12°$, $K_{bend} = 0.06$
for $\theta = 8°$, $K_{bend} = 0.04$
for $\theta = 3°$, $K_{bend} = 0.02$

Bend losses = (0.34 + 0.11+ 0.13 + 0.18 + 0.20 + 0.02 + 0.02 + 0.06 + 0.04 + 0.02) $\times (3.5^2 / 2g)$ = 0.70 m

Total head loss = 12.29 m + 0.12 m + 0.70 m   = 13.11m

% head loss = (13.11 / 180) x 100% = 7.28%,

Since the head loss is between 5% to 10%, in case of micro-hydropower plant, this could have been accepted. However, since this is a mini-hydropower plant (500kW installed capacity), a 450mm diameter was adopted which gives a head loss of 7.61m (4.2%) based on 'optimisation' which is explained below:

The power gained by increasing the pipe diameter from 405 mm to 450 mm at a conservative
60% efficiency   =(12.69 -7.38) x 0.450x9.81x0.6
                            = 14.1 kW
Annual energy gained assuming an overall plant factor of 40%
                            = 14.1 x 24 x 364 x 0.4
                            = 49,271 kWh
Expected annual incremental income at Rs 4.0 per kWh of electricity generated
                            = Rs. 197,084
Note: The Jhankre power plant was initially implemented to provide partial construction power at the intake site for the then under construction 60 MW Khimti Small hydropower plant. The alternative source of energy was diesel that had a cost of generation of Rs 4.0 per kWh.

Now the question is, "Is this gain of Rs 197,084 worth the increase in pipe diameter?
The next step is to calculate the Net Present Value of Benefits and compare it with the incremental costs.

NPV of Benefits assuing:
            -          discount rate i = 10%,
            -          Economic live, n = 15 years
            -          Annual income, A = Rs. 197,084

The equation for calculating NPV is: $NPV_{benefit} = A \left[\dfrac{(1+i)^n-1}{i(1+i)^n} \right]$

Or NPV benefits = Rs.1,499,336

Now the incremental cost in increasing the pipe diameter should be considered:

Assume average pipe thickness = 4.5 mm:
Additional weight (recall pipe length = 550 m)
= P(0.450 –0.405)x4.5 x7.85x550= 2747 kg
Note: A 1 m2 with 1 mm thickness steel plate weighs 7.85 kg and thus 4.5 is multiplied by 7.85 above to calculate the weight
AT Rs 200/kg market price including fabrication transport, site welding & installation the
NPV costs = NRs. 200/kg x 2747 kg = NRs 549,400

Thus, the NPV of incremental costs in increasing the pipe diameter is 36% of NPV of benefit which justifies the decision to adopt 450 mm diameter. However, the following should be noted:
1. Increasing the diameter also somewhat increases the transport and installation costs. The anchor blocks volume would also increase. As these factors have not been taken into account, if the NPV of costs were close to NPV of benefits it would not have been worth increasing the pipe diameter. However, at NPV costs = 36% of NPV benefits the decision is well justified as differences are high

2. The pipe fabricator company proposed to roll the penstock pipes at 455 mm diameter for the price of 450 mm! This was because the plate width that the fabricator was able to obtain in the market was just right for a 455 mm diameter (1430 mm width). Thus, it was cost effective for the fabricator to compensate extra 5 mm diameter than the additional work required to cut plate width to make them right for 450 mm diameter. The point to be noted here is that along with optimization calculations, practical considerations need to be taken into account as well.

Pipe wall thickness calculations

First calculate the thickness required at the downstream end of the penstock (i.e. $h_{static} = h_{gross} = 180$ m) using d = 450 mm. Try t = 6 mm .

$$a \quad = 1440\sqrt{1 + [(2150 \times d) / (E \times t)]}$$
$$a \quad = 1440\sqrt{1 + [(2150 \times 450) / (2.0 \times 6)]}$$
or, $\quad a \quad = 1071$ m/s
$$V \quad = 4Q / \pi d^2 \quad = 4 \times 0.450 / (0.450)^2 \pi \quad = 2.83 \text{ m/s}$$
$$h_{surge} \quad = (av / g) \times (1 / n)$$
$$h_{surge} \quad = [(1071 \times 2.83) / 9.8] \times [1 / 6] \quad = 52 \text{ m}$$
$$h_{total} = h_{gross} + h_{surge} \quad = 180 + 52 \quad = 232 \text{ m}$$
$$t_{effective} \quad = [6 / (1.1 \times 1.2)] - 1.0 \quad = 3.55 \text{ m}$$

(1.1 for welding, 1.2 for flat rolled, and 1 mm for corrosion allowance: the corrosion allowance is less than previously recom-mended for larger schemes because Jhankre was designed to provide construction power to a larger project). Calculate the safety factor:

$$SF \quad = (200 \times t_{effective} \times S) / (h_{total} \times d)$$
$$= (200 \times 3.55 \times 400) / (232 \times 450)$$

$$SF \quad = 2.72 > 2.5, \text{ although SF is less than 3.5, it}$$
is acceptable in this case since:
1. There were experienced staff at site. The site staff had installed penstock pipes in various other micro-hydro projects.
2. The valves at the powerhouse are of slow closing type.
3. The pipes were pressure tested as follows:
   Tensile test of steel plates was performed at a laboratory and an ultimate tensile strength of 400 N/mm² was ensured as mentioned earlier.

Rolled pipes were pressure tested at the workshop at h_{total} using a hydraulic pump.

Finally, the pipes were also pressure tested on site after installation by simulating h_{surge} at the entrance (forebay) using a hydraulic pump.
4. The alignment was assessed to be fairly stable. In case of pipe burst it was not expected to instantaneously cause landslide.

Since the Jhankre penstock is long, to optimise the design, it was decided to decrease the pipe thickness at lower heads (i.e.. upstream) using the same safety factor (SF).

Calculations of the static head at which the penstock thickness can be decreased by 1 mm (i.e. thickness = 5 mm) using the same safety factor (SF = 2.72) are as follows:

$$a \quad = \frac{1440}{\sqrt{1 + [(2150 \times 450) / (2.0 \times 10^5 \times 5)]}} = 1027 \text{m/s}$$

Note that t = 5 mm in this case

$$V \quad = 2.83 \text{ m/s, (same Q \& d)}$$
$$h_{surge} \quad = (av / g) \times (1 / n)$$
$$= [(1027 \times 2.83) / 9.8] \times [1 / 6]$$
$$= 49 \text{ m}$$

$$t_{effective} = [5 / (1.1 \times 1.2)] - 1.0 \quad = 2.79 \text{ mm}$$

$$SF \quad = [200 \times t_{effective} \times S] / [h_{total} \times d]$$

or rewriting this equation in terms of h_{total} :

$$h_{total} \quad = [200 \times 2.79 \times 400] / [2.72 \times 450]$$
$$= 182 \text{ m}$$

$$h_{gross} \quad = h_{totalt} - h_{surge}$$
$$= 182 - 49 = 133 \text{ m}$$

Therefore the pipe thickness was reduced to 5 mm at a static head of 133 m in Jhankre, keeping the same factor of safety (i.e. 2.72) as shown in Figure 6.4. The Jhankre penstock alignment for the last section can also be seen in Photograph 6.7. These calculations were repeated for lower static heads and a wall thickness down to 3 mm has been used to reduce the cost.

6.73 FLANGE CONNECTION

This involves welding flanges (that have bolt holes) at both ends of the pipes in the workshop, then joining them at site by bolting them together. A rubber gasket should be placed between the flanges for tightness and to prevent leakage. The Galkot penstock (Photograph 6.8) is of flange connected type. A comparison of these two methods along with general

6.7.4 HDPE AND PVC PIPES

For HDPE pipes the best method of joining them is by heat welding as described in Chapter 4 (Box 4.7). Although special flanges are available to connect HDPE pipes, they are generally more expensive than the cost incurred in heat welding them. HDPE pipes are available in rolls for small diameter (up to 50 mm) and for larger diameter they are available in discrete lengths (3 m to 6 m in Nepal)

PVC pipes with small diameter (up to 200 mm) have socket at one end such that another pipe can be inserted inside after applying the solution at the ends. Larger diameter PVC pipes are joined with a coupler, which is a short pipe section with inside diameter equal to the outside diameter of the pipes to be joined. The solution is applied on the connecting surfaces of both the coupler and the pipes and then joined together.

6.8 Pipe lengths

Mild steel pipes can be manufactured at the workshop in almost any length required. PVC and HDPE pipes are available in fixed lengths (3 m to 6 m in Nepal). Although, shorter pipes are easy to transport, additional costs will have to be incurred in joining them at site (more flanges or welding work). It should be noted that, unlike cement bags, animals (mules and yaks) do not usually carry penstock pipes because of the shapes and lengths involved.

Sometimes, due to the weight involved the only option for transporting the generator and turbine to remote site is by a helicopter. In such cases, it may be possible to transport the penstock pipes in the same trip because the current transport helicopter available in Nepal can carry up to three tons (depending on altitude). The combined weight of the generator charge is dependent on the flying hours and not on loads. When this is the case longer pipes (up to 6 m lengths) can be transported to site and hence joints can be minimised. Recommendations for pipe lengths under various conditions are discussed below:

Mild steel pipe

The following factors should be considered while sizing mild steel pipes.

1. In general pipes longer than 6 m should not be manufactured since they will be difficult to transport on trucks.

2. If the pipes need to be carried by porters from the roadhead, the weight should be such that an individual length can be carried by 1-2 porters. For example, if the pipe weight is about 50 kg, usually one porter can carry it. Similarly two porters may be to able carry up to 90 kg. Therefore, it is optimum to size pipes accordingly, especially if the penstock length is long and the site is located more than a day's walk from the roadhead.

3. For flat rolled pipes the manufacturing costs will be less if the pipe length is a multiple of the available steel plate width. For example if pipes are rolled from 1.2 m wide plates, lengths of 1.2 m, 2.4 m or 3.6 m etc. will lower manufacturing costs.

HDPE and PVC pipes

As mentioned earlier, these pipes are manufactured in the factory at fixed lengths, but they can be cut in half or one third of the length for ease of transportation. However, a PVC pipe with a socket at one end should not be cut since rejoining will not be possible without a special collar. The following factors should be considered while determining the length of HDPE and PVC pipes.

1. In Nepal the maximum available length of these pipes is 6 m. Even if longer pipes become available, such lengths should not be used for micro-hydro schemes. Generally it is easier to carry two 3 m pipes rather than one 6 m pipe of the same type.

2. If the pipes need to be carried by porters from the roadhead, then the criteria outlined in No. 2 for mild steel also apply.

6.9 Exposed versus buried penstock

HDPE and PVC pipes should always be buried. This minimises thermal movement and protects the pipe against impact, vandalism and ultra-violet degradation. Flanged steel pipe should be above ground. This is because the gaskets may need to be replaced during the life of the scheme. Mild steel penstock with welded joints can be either buried or above ground. However, maintenance of buried pipe is difficult, therefore the original painting and backfilling must be carefully supervised to ensure that corrosion does not reduce the life of the penstock. Sometimes part of the penstock alignment may be above ground and part buried. In such cases, it is best to make the transition at an anchor block, otherwise careful detailing is required. An example of such detailing at the transition is the use of a retaining masonry wall with a larger diameter mild steel pipe through which the penstock comes out and can accommodate thermal expansion and contraction, see Figure 6.5. An expansion joint should normally be used immediately downstream of the retaining wall. Note that the design of anchor blocks is covered in Chapter 7. Table 6.4 compares the advantages and disadvantages of buried penstock pipes.

For buried pipes, a minimum cover of 1 m should be provided in all cases (i.e. HDPE, PVC and mild steel pipes). See Figure 4.8 for trench details. Buried pipes do not require support

piers, and the savings made on the piers may equal or even exceed the cost of excava-tion and backfill. Since this is a site-specific case, a cost calculation should be done if buried mild steel pipe is being considered. Both exposed and buried penstock pipes require anchor blocks at significant bends. However, for relatively low head and flow, as well as small bend angles, the 1 m depth of well compacted soil cover on buried pipe may be adequate (see Chapter 7, Anchor blocks and support piers). The nature of the terrain and the soil depth may also govern whether to bury or expose the penstock pipe. Buried penstock is not practicable on routes steeper than 30° because the backfill will be unstable. Where top soil is thin or rock is exposed, the costs involved in excavating the rock may make burial of the pipe impossible.

6.10 Expansion joints

Penstock pipes are subjected to temperature variations due to changes in the ambient temperature. When the ambient temperature is high the pipes will expand and when it drops,

the pipes will contract. Such thermal expansion causes stresses in the pipes if they are not free to expand.

An above ground penstock is subjected to greater temperature variations resulting in higher thermal expansion. The thermal expansion or contraction is highest when the penstock is empty, such as during installation or repair work. The temperature variation is relatively low when the pipe is full since the flow of water with fairly constant temperature prevents the pipe from rapidly heating up. As long as pipes are free to move at one end, thermal expansion does not cause additional stresses. However, a penstock pipe section between two anchor blocks is kept fixed at both ends. In such a case thermal expansion could cause additional stresses and the pipe can even buckle. Therefore, provision must be made for the penstock pipe to expand and contract, by installing an expansion joint in a penstock pipe section between two anchor blocks. The most common type of expansion joint used in Nepal is of sliding type. This is shown in Figure 6.6 and Photograph 6.13. Such an expansion joint is placed between two consecutive pipe lengths and bolted to them. The stay

TABLE 6.3 **Comparison between site welding and flange connection**

PIPE JOINING METHOD	ADVANTAGE	DISADVANTAGE	GENERAL RECOMMENDATIONS AND COMMENTS
Site welding	Easy to fabricate at workshop since flanges do not have to be welded at pipe ends. A properly welded pipe will not leak and requires less maintenance.	Higher degree of precision work required at site to weld the pipe ends. Improper welds can cause leaks and pipe can burst at high heads. Need to transport a welding machine and a generator at site. Also requires supply of petrol/ diesel to site.	Difficult logistics if the site is more than a day's walk from the road head. Generally not economic for small schemes and/or short penstock lengths. Select this option only if the site staff are experienced, site is less than a day's walk from the roadhead and penstock length is more than 50 m.
Flange connection	Easy to install at site. Site installa-tion work involves placing a gasket between the flanges and bolting them.	Fabrication cost is high since flanges need to be welded at ends. Also there is some wastage since the flange is prepared by removing the central disc of a diameter equal to the external pipe diameter. The pipe alignment and the bends can not be adjusted at site. Can leak if the bolts are not well tightened or if gaskets are of poor quality. Higher risk of vandalism since the bolts can be removed.	Flange connection is appropriate for schemes that are located more than a day's walk from the roadhead and/or have a relatively short penstock length. Minimum flange thickness should be at least twice the penstock wall thickness or 8 mm whichever is larger. A minimum bolt diameter of 12 mm is recommended. A minimum gasket thickness of 5 mm is recommended. Should be above ground.

TABLE 6.4 **Advantages and disadvantages of buried penstock pipe**

ADVANTAGES	DISADVANTAGES
■ Protects the penstock against adverse effects of temperature variations.	■ Pipes are less accessible for inspection, and fault finding becomes difficult.
■ Protects the water from freezing due to low air temperatures.	■ Repair and maintenance of the pipes is difficult.
■ Protects the pipe from falling debris and trees.	■ Installation is expensive in rock and where soil cover is thin.
■ Protects the pipe from tampering and vandalism.	■ Impracticable on steep slopes (>30°).
■ Eliminates support piers.	
■ Small bends do not need anchor blocks.	
■ Does not change the landscape.	

rings are tightened which compresses the packing and prevents leaking. Jute or other similar type of fibre is used for packing. When the pipes expand or contract, the change in lengths is accommo dated inside the joint section since there is a gap between the pipes. An advantage of an expansion joint is that it reduces the size of the anchor blocks since they will not need to withstand forces due to pipe expansion. Another advantage is that they can accommodate slight angular pipe misalignment. Expansion joint requirements for various penstock conditions are discussed below.

Mild steel pipes
An expansion joint should always be incorporated immediately downstream of the forebay and immediately downstream of each anchor block, for both above ground and buried steel pipe. One is also recommended immediately downstream of a transition from buried to above ground pipe.

HDPE pipes
Expansion joints are not necessary for HDPE penstock pipes provided that they are buried (which should always be the case). This is because HDPE pipes are flexible and can bend to accommodate the expansion effects due to the differences in temperature between installation and operational phases.

Photo 6.9 Jhankre mini-hydro penstock alignment
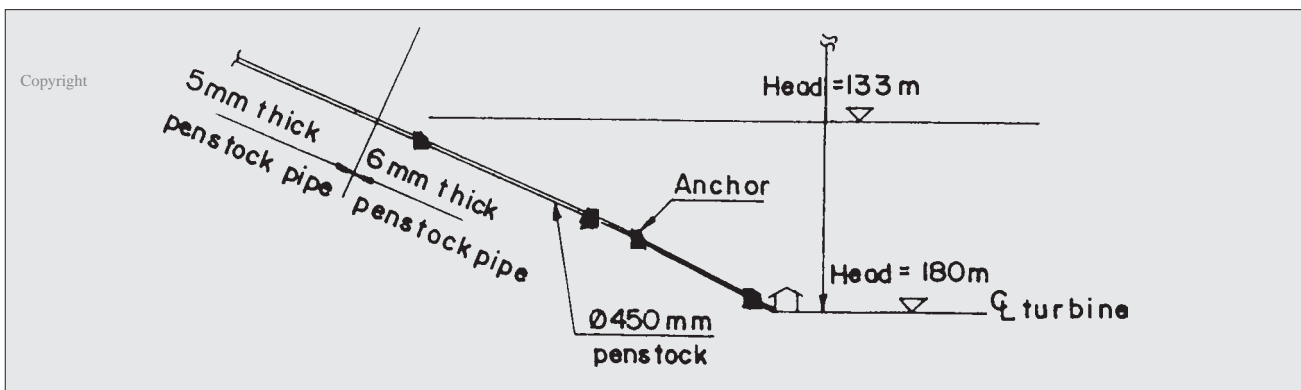
Figure 6.4 Jhankre mini-hydro penstock alignment at downstream end
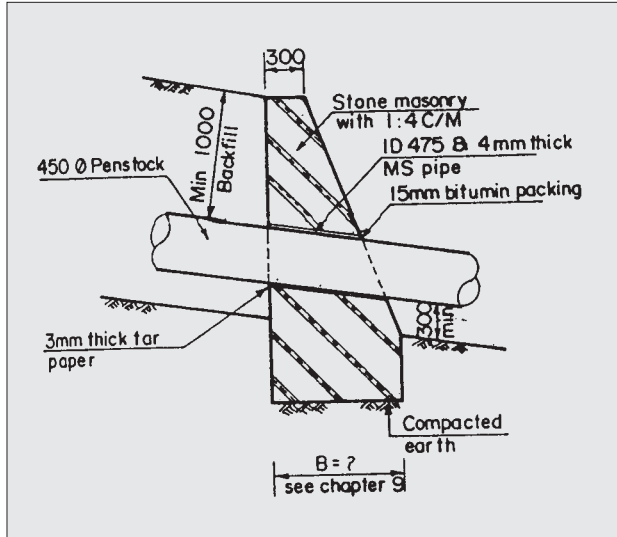
Figure 6.5 Transition from buried to exposed penstock, Jhankre mini-hydro

Photo 6.10 Site welding of penstock pipes, Jhankre mini-hydro

PVC pipes

PVC pipes with glued joints require provision for expansion, at the same locations as for steel pipes.

Sizing of expansion joints

The sliding surface of the expansion joints should be machine finished (such as in a lathe machine) to a tolerance of about 0.1 mm. The recommended thickness of the steel parts (retainer and stay ring) is about twice the thickness of a well-designed penstock pipe.

Photo 6.11 Penstock, Purung

The gap in the expansion joint should be about twice the calculated maximum pipe expansion length.

The maximum expansion length is calculated using the following equation:

where:

$$= \alpha \, (T_{hot} - T_{cold}) \, L$$

L = pipe expansion length in m as shown in Figure 6.7. α = coefficient of linear expansion in m/m °C of the pipe, which depends on the pipe material. This coefficient relates to the length that a material will expand per 1°C increase of
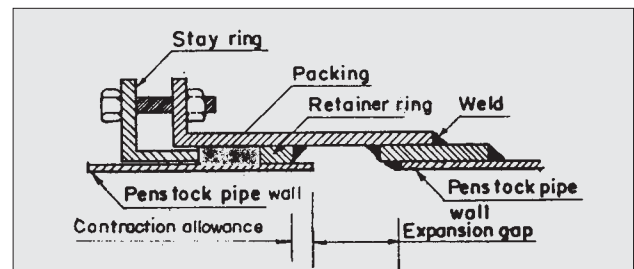
Figure 6.6 Sliding type of expansion joint
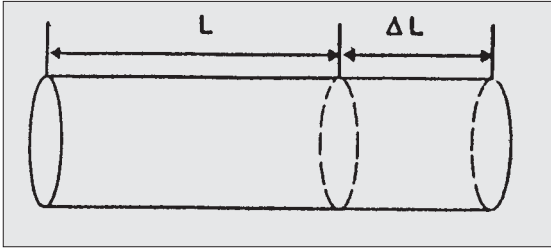
Figure 6.7 Thermal expansion of a penstock pipe

temperature. Different materials expand at different rates. The values of this coefficient for mild steel, HDPE and PVC are shown in Table 6.2.

T_{hot} = highest temperature in °C that the pipe will experience. Note that this can even be during mid-summer afternoon when the pipe is empty (either during installation or repair work).

T_{cold} = lowest temperature in °C that the pipe will experience. This can be during winter when the water temperature is just above the freezing point. Note that if freezing temperatures are expected, the pipe should either be emptied or provision

Photo 6.12 An expansion joint should be located just below an anchor block to protect the block from forces which it may not be designed to resist (Siklis)

Example 6.3 Calculating penstock pipe expansion length

A mild steel penstock pipe is 45 m long between the forebay and the first anchor block. The steel temperature during installation was 40°C, and the expected lowest temperature during the operational phase is 4°C during winter. What expansion gap should be recommended to the manufacturer? Also, if the temperature during installation is 20°C, what gap should be provided?

$$\alpha = 12 \times 10^{-6} \text{ m/m °C}$$
$$T_{hot} = 40 \text{ ° C}$$
$$T_{cold} = 4 \text{ ° C}$$
$$L = 45m$$
$$\triangle L = a(T_{hot} - T_{cold})L$$
$$\text{or } \triangle L = 12 \times 10^{-6} (40 - 4) \times 45$$
$$\text{or } \triangle L = 0.019m$$
$$\text{or } \triangle L = 19 \text{ mm}$$

Therefore minimum recommended expansion gap

= 19.4 x 2 = 38.8 mm, say 40 mm.

If the temperature during installation is 20°C

$$\triangle L = 12 \times 10^{-6}(40-20) \times 45$$
$$\text{or} \triangle L = 0.011 \text{ m}$$
$$\text{or} \triangle L = 11 \text{ mm}$$

Therefore, during installation an expansion gap of 11 mm x 2 = 22 mm should be provided.

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should be made for constant flow. If the water in the penstock stagnates during freezing temperature, ice will form inside the pipe and could burst it, because when water freezes, the volume expands. L = pipe length in m. Since it may be difficult to determine when the expansion joint will be installed at site, the manufacturer should be asked to allow an expansion gap of 2 L Then, during installation, the temperature should be noted and the gap left accordingly.

6.11 Painting

Since mild steel pipes are subjected to corrosion, appropriate coats of paint should be applied before dispatching them to site. Proper painting of mild steel pipes significantly increases their useful lives.

The pipes should be sand blasted if possible, otherwise they should be thoroughly cleaned using a wire brush and a piece of cloth. Prior to painting, the pipe surface should be clean from oil, dust and other particles. When applying subsequent coats of paint, the previous coat must be dry.

Photo 6.13 Sliding expansion joint, Jhankre mini-hydro

Photo 6.14 Penstock at Ghandruk with village in the background

The following coats of paint are recommended:

Outside surface of above ground mild steel pipes

First two coats of primer should be applied on the pipe surface. Red Oxide Zinc Chromate primer is appropriate for this purpose. Then another two coats of high quality polyurethane enamel paint should be applied on top of the primer.

Outside surface pipe of which will be buried or cast into anchor blocks

Two coats of primer similar to above ground pipe should be applied. Then, another two coats of high-build bituminous paint should be applied over the primer. Provide an extra coat of bituminous paint at transition areas, which are more prone to corrosion (see Figure 7.1).

Inside surface of pipes

For small diameter pipes it may not be possible to paint the inside surface. However whenever possible, the inside surface should be painted with two coats of good quality red lead primer. If there is a doubt about the quality of paint, the supplier's specifications should be checked prior to its use. Note that paintwork is not required for HDPE or PVC pipes. Any paintwork damaged during transport and installation must be made good, so that the full number of coats is present everywhere. This is especially important for buried pipes.

6.12 Installation

The following procedure should be used:
- The centreline of the penstock should be set out using a cord and pegs along the selected route as shown in Figure 6.8. For micro-hydro schemes above 20 kW of installed capacity, a theodolite should also be used to ensure that the bend angles correspond to the fabricated pipe bends.
- A line should be marked by spreading lime on the surface of the ground to replace the cord. Then the positions of anchor blocks and support piers should be marked to the required spacing for exposed pipes and excavation carried out along this line as required.
- For buried pipes, the penstock is installed in the excavated trench and backfilled as shown in Figure 4.8. The backfill should be rammed in layers and a slight hump above the level of the ground helps to keep the alignment dry. An improperly backfilled penstock alignment can quickly become the route for drainage water down the hillside. However, note that backfill should be completed only after the pipe has been pressure tested.
- For exposed pipes, the anchors and supports should be constructed as will be discussed in Chapter 7. The pipe should be cast into the anchors and placed on one support pier at a time. No further supports or anchors should be built until the pipe is secured to the previous anchor block or support pier. For both site welded and flange connected pipes, the end should protrude from the last support block with adequate margin (~ 300 mm) so that either the flange or the weld line does not lie on the support pier during thermal expansion or contraction. If more than one pipe section needs to be welded between the support piers, temporary supports should be used as shown in Photograph 6.15. Flange connected pipes should be joined and the bolts tightened as the installation progresses.

The installation of the penstock should start from the machine foundation and proceed upstream. This avoids any misalignment between the penstock and the turbine housing. Since the turbine needs to be firmly fixed to the machine foundation, there is almost no tolerance at this end after the machine foundation has been constructed. Furthermore, the pipe sections below the expansion joints can slide down if installation proceeds downstream from the forebay. Minor pipe deviation can be adjusted at the forebay wall, but such adjustment is
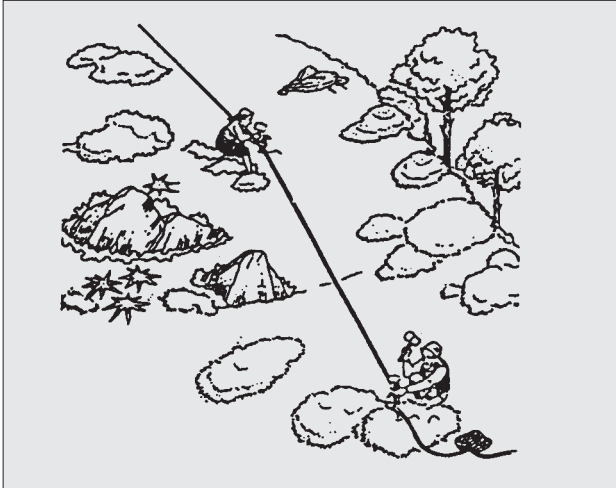
Figure 6.8 Setting out the centreline of the penstock alignment

Photo 6.15 Temporary support for site welding work, jhankre mini-hydro, Nepal

not feasible at the machine foundation. For micro-hydro schemes, laying of penstock in discrete lengths is not recommended since this can lead to misalignments of the pipes.

- Penstock pipes should be pressure tested at the factory before transport to site. For schemes where the head is more than 15 m the completed penstock should also be pressure tested during the commissioning phase. If feasible such pressure test should include the surge head (i.e., pressure test at h_{total}). This can be done by simulating the expected surge head at the forebay using a manual pressure pump. If any leakage is noticed, the section should be repaired such as by tightening the bolts, changing faulty gaskets or welding. For buried pipe alignment, the backfill should only be completed after successful pressure testing; however, if there are any minor bends without anchor blocks, these must be backfilled before pressure testing. Once the pipe trench is backfilled, it will be difficult and time consuming to reexcavate and identify the leaking section.

6.13 Maintenance

Above ground mild steel penstocks should be repainted every 3 to 4 years depending on the conditions. Nuts, bolts and gaskets of flange connected mild steel pipes should be checked annually, loose bolts should be tightened and damaged gaskets should be replaced. A visual check for flange leaks should be carried out monthly. For buried penstock sections, signs of leakage such as the sudden appearance of springs along the alignment (especially during winter) and moist ground where the area was previously dry should be checked. If any leakage is noticed, the penstock should be drained and carefully excavated for repair of the leaking section

6.14 Checklist for penstock work

- Refer to Table 6.1 and decide on the penstock material. When in doubt compare the costs of all available options.
- Is the alignment on practical ground slope? Is there adequate space for the powerhouse area at the end of the penstock alignment? Have the bends been minimised?
- For mild steel pipes refer to Table 6.3 to decide on flange connection or site welding. Also be sure to specify appropriate coats of paint.
- If a buried penstock alignment is being considered, refer to Table 6.4 to compare the advantages and disadvantages, and Figure 4.8 for the trench details.
- Is the pipe diameter such that the headloss is between 5% and 10%?
- Has allowance been made for surge effects while sizing the penstock wall thickness?
- Is the safety factor sufficient as discussed in Section 6.6? Are the pipe lengths and weights such that they are transportable and porterable?

Refer to Section 6.12 for pipe installation at site.

7. Anchor blocks and support piers

7.1 Overview

An anchor block is an encasement of a penstock designed to restrain the pipe movement in all directions. Anchor blocks should be placed at all sharp horizontal and vertical bends, since there are forces at such bends which will tend to move the pipe out of alignment. Anchor blocks are also required to resist axial forces in long straight sections of penstock. Support piers are short columns that are placed between anchor blocks along straight sections of exposed penstock pipe. These structures prevent the pipe from sagging and becoming overstressed. However, the support piers need to allow pipe movement parallel to the penstock alignment which occurs due to thermal expansion and contraction.

7.2 Anchor blocks

7.2.1 GENERAL
Locations at which anchor blocks are required and their construction are described in this section.

7.2.2 LOCATION OF ANCHOR BLOCKS
Anchor blocks are required at the following locations:
- At vertical or horizontal bends of the penstock as shown in Photograph 7.2. A filled penstock exerts forces at such bends and the pipe needs to be properly 'anchored'.
- Immediately upstream of the powerhouse. This minimises forces on the turbine housing.
- At sections of the penstock where the straight pipe length exceeds 30 m. This is to limit the thermal expansion of the pipe since an expansion joint will be placed downstream of the anchor block.

7.2.3 CONSTRUCTION OF ANCHOR BLOCKS
Anchor blocks should normally be constructed of 1:3:6 concrete (1 part cement, 3 parts sand, 6 parts aggregate) with 40% plums and nominal reinforcement. Plums are boulders that are distributed evenly around the block such that they occupy about 40% of the block volume. The boulders add weight to the block and therefore increase stability while decreasing the cement volume required. Hoop reinforcement is required around the pipe to resist cracking of the concrete due to tensile forces from the pipe. Three 10 mm bars are generally sufficient, as shown in Figure 7.1

Photo 7.1 A straight penstock with four supports and an anchor block before the power house

Photo 7.2 Jhankre mini-hydro anchor block for an upward vertical bend

The hoop bars should be approximately 150 mm clear of the pipe, and should extend to 100 mm from the base, so that the whole weight of the block can be mobilised without cracking. If the reinforcement is inadequate, the block can crack, as shown in Photograph 7.3. A collar or metal tags may be welded to the pipe to ensure that the pipe does not slide within the anchor block. For downward bends, the anchor block is mainly in compression, therefore a stone masonry structure (1:4 cement mortar) can be considered if costs can be brought down. Composite anchor blocks can also be considered to save cost as shown in Figure 7.2. Foundation parts and central portion of the block can be made of 1:1.5:3 reinforced concrete and outer portion of the block can be made of stone masonry in 1:4 (cement: sand) mortar.

The cement requirements for plum concrete and cement masonry are as follows:
- 1:3:6 concrete with 40% plums: 132 kg of cement per m³ of block volume.
- 1:1.5:3 concrete: 400 kg of cement per m³.
- Stone masonry in 1:4 cement mortar: 159 kg of cement per m³.

Although more cement is required for cement masonry blocks, savings may be made by avoiding the cost of form work (where wood is expensive) and crushing of stone to prepare aggregates. Therefore, whether plum concrete or cement masonry is economical is site specific but this issue should be investigated if there are a number of downward vertical bends. Cost can also be reduced by using permanent dry stone walls as formwork for the buried portion of the anchor block as shown in Photograph 7.4. At sites where wood is expensive this approach is worth considering. Both plain concrete or stone masonry in cement mortar blocks should be cured as discussed in Chapter 3 by keeping them moist for at least a week. The design of anchor blocks is covered in Section 7.4.

Photo 7.3 Cracking of the upper surface of an unreinforced anchor block

Photo 7.4 Use of dry stone wall for forework, Jhankre mini-hydro
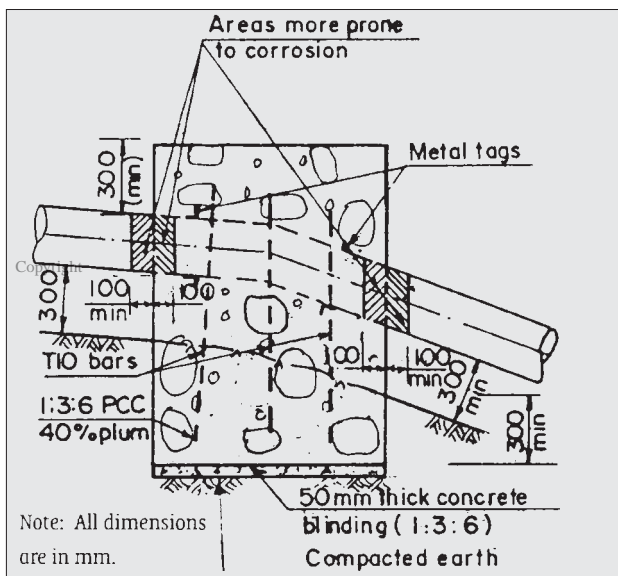
Figure 7.1 Anchor Block Section
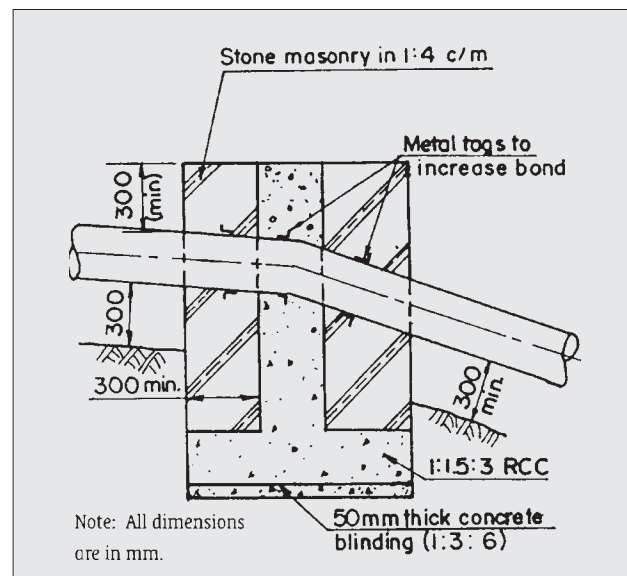
Figure 7.2 Composite anchor block section

7.3 Support piers

73.1 GENERAL
Locations at which support piers are required and their construction are described in this section.

73.2 LOCATION OF SUPPORT PIERS
Support piers are required along the straight sections of exposed penstock between anchor blocks. The maximum spacing of support piers to avoid overstressing the pipe is given in Table 7.1. Please read the notes under the table. Thin-walled plain pipe can buckle at the support piers with relatively short spans. In this case the permissible span can be increased by welding a wear plate to the pipe at each support, see Figure 7.3. This may be economical for pipes larger than 300 mm diameter. Corners of wear plates should be cut with a radius, to avoid stress concentrations. Note that a wear plate is also required where the pipe leaves an anchor block, if the span to the first support pier exceeds that allowed for plain pipe. It is usually not economical to increase the pipe wall thickness in order to increase the support pier spacing, but this should be considered where the cost of support piers is significant.

TABLE 7.1 Support pier spacing (centre to centre horizontal length in metres)

EFFECTIVE PIPE WALL	PLAIN PIPE				PIPE WITH WEAR PLATES			
THICKNESS, $T_{effective}$ (mm):								
320 N/mm² STEEL	1.3	1.9	2.6	3.9	1.3	1.9	2.6	3.9
410 N/mm² STEEL	1.0	1.5	2.0	3.0	1.0	1.5	2.0	3.0
(a) Total head h_{total} < 100 m								
100 mm dia	2.0	3.2	4.0	4.7	2.0	3.2	4.0	4.7
200 mm dia	4.1	6.0	7.3	8.3	4.1	6.0	7.3	8.3
300 mm dia	2.5	5.7	8.6	10.5	4.9	7.0	8.6	10.5
400 mm dia	1.4	2.6	5.8	11.7	5.1	7.4	9.1	11.7
500 mm dia	-	2.1	3.7	8.7	4.1	7.5	9.3	12.0
(b) 100 < h_{total} < 150 m								
100 mm dia	1.9	3.1	4.0	4.7	1.9	3.1	4.0	4.7
200 mm dia	3.9	5.8	7.1	8.3	3.9	5.8	7.1	8.3
300 mm dia	2.5	5.7	8.2	10.5	4.4	6.7	8.2	10.5
400 mm dia	1.4	2.6	5.8	11.2	4.4	6.9	8.6	11.2
500 mm dia	-	2.1	3.7	8.7	4.0	6.8	8.7	11.5
(c) 150 < h_{total} < 200 m								
100 mm dia		2.3	3.5	4.7		2.3	3.5	4.7
200 mm dia	2.7	5.0	6.5	8.3	2.7	5.0	6.5	8.3
300 mm dia	2.5	5.7	7.6	10.1	3.2	5.8	7.6	10.1
400 mm dia	1.4	2.6	5.8	10.6	3.0	6.0	7.9	10.6
500 mm dia	_	2.1	3.7	8.7	2.0	5.8	7.9	10.7
(d) 200 < h_{total} < 250 m								
100 mm dia	_	1.8	3.1	4.7		1.8	3.1	4.7
200 mm dia	1.8	4.4	6.0	8.3	1.8	4.4	6.0	8.3
300 mm dia	2.1	5.2	7.0	9.7	2.1	5.2	7.0	9.7
400 mm dia		2.6	5.8	10.1	-	5.1	7.2	10.1
500 mm dia	_	2.1	3.7	8.7	-	4.7	7.1	10.1

Notes:

1. Applies only to steel penstocks welded or flanged to British Standard (minimum flange thickness = 16 mm). In other cases use one support pier for each individual pipe length, with the pier in the middle.
2. Wear plates to be of some thickness as pipe wall , and weilded an all edges, covering bottom 180° of pipe. The length should be enough to extend at least 0.5 times the pipe diameter beyond each side of the support pier. See figure 7.3
3. For the calculation of $t_{effective}$ refer to Section 6.6.
4. Interpalate between the above values for intermediate pipe diameters, wall thickness or steel grades.

7.3.2 CONSTRUCTION OF SUPPORT PIERS

Support piers are generally constructed out of stone masonry in 1:4 cement mortar. Dressed stone should be used for the outside surfaces of the pier. A 140° bearing area from the centre of the penstock diameter should be provided to support the penstock pipe as shown in Figure 7.4. Placing a steel saddle plate above the support pier where the penstock pipe rests along with a 3 mm thick tar paper as shown in Figure 7.4 minimises frictional effects and increases the useful life of the pipe. C-clamps may also be provided to protect the pipe from vandalism and a sideways movement, but there must be a gap between the surface of the pipe and the C-clamp, so that axial forces are not transferred to the support pier. Stone masonry support piers with C-clamps can be seen in Photograph 7.5.Wooden support piers have occasionally been used in micro-hydro schemes, as can be seen in Photograph 7.7. However, wood is generally expensive and also requires frequent maintenance such as painting.

Steel support piers can also be used as an alternative to stone masonry, especially at sites where cement is expensive or the soil is weak in bearing. An example of steel support piers is included in Chapter 10 (Innovations).
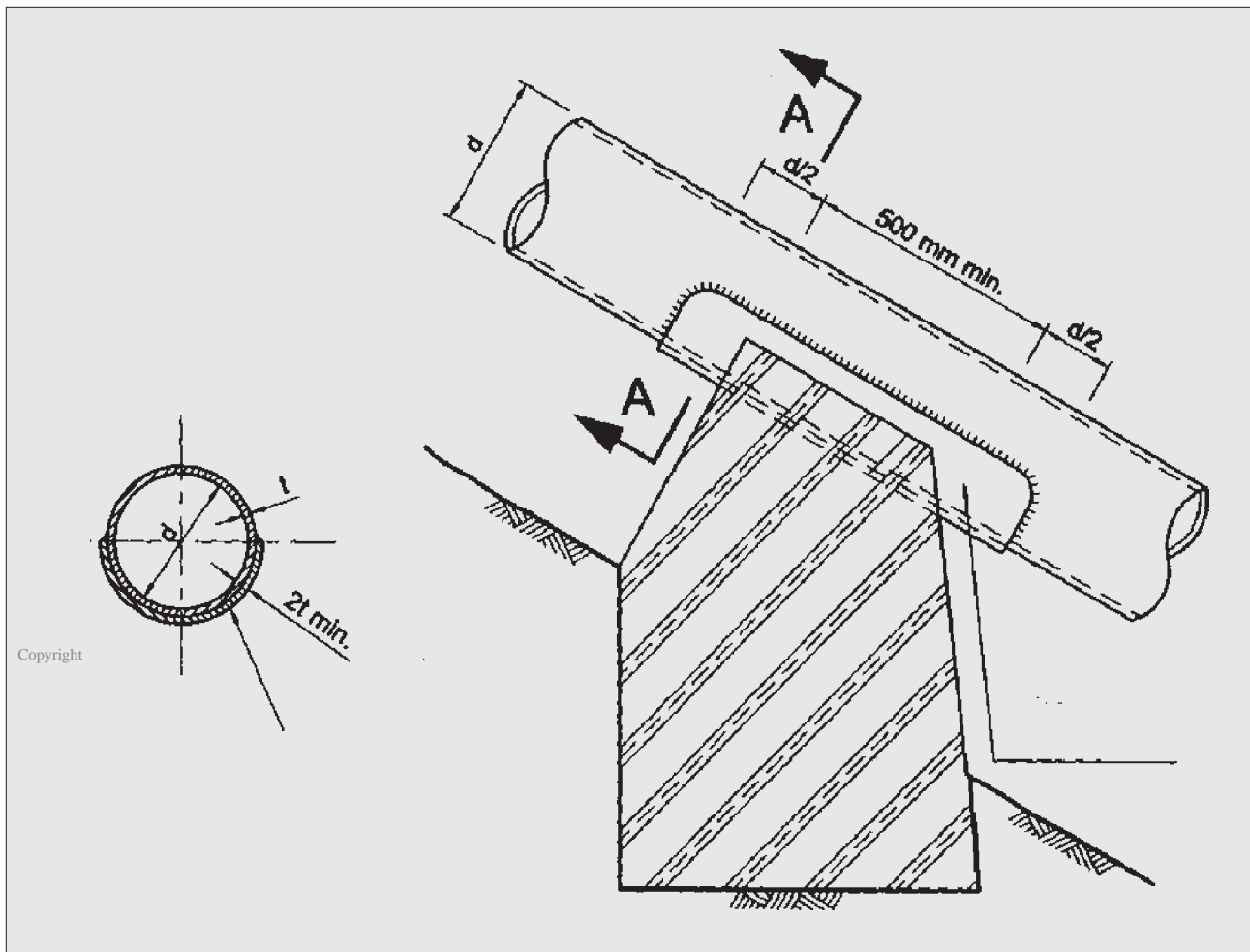
Figure 7.3 Arrangement of wear plate at support pier

Photo 7.5 Stone masonry support piers, Jhankre mini-hydro scheme

Figure 7.4 Typical section through a support pier

7.4 Design of anchor blocks and support piers
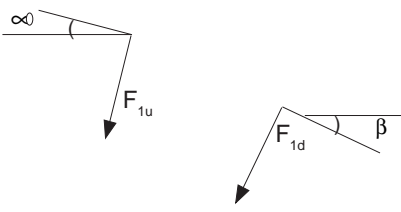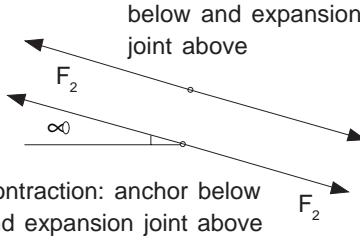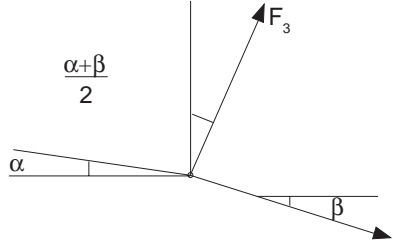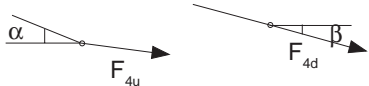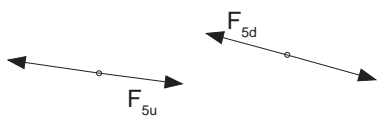
7.4.1 GENERAL
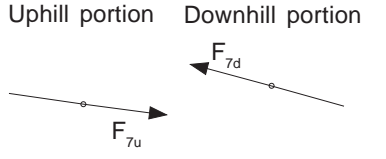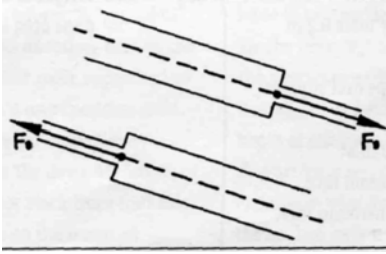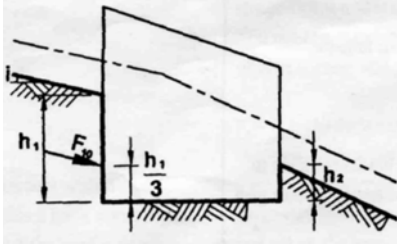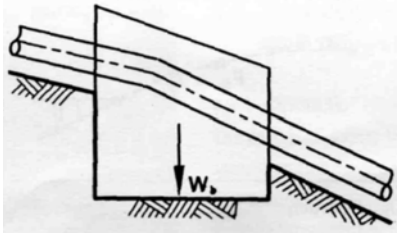
The design of anchor blocks and support piers requires resolving some common forces, which are therefore discussed together in this section. First the structures are tentatively sized and the various forces that act on them are resolved. The minimum calculated block size that is safe against bearing, sliding and overturning is accepted. It should be noted that the design process involves a few iterations. Various forces that can act on anchor blocks and support piers are summarised in Table 7.2 and discussed thereafter.

Photo 7.6 The use of an extension to the concrete support lifts the pipe clear of an block, allowing drainage and preventing corrosion between the block and the pipe (Khamche)

Photo 7.7 Wooden support piers at Kamche micro-hydro, Nepal

TABLE 7.2 **Forces on anchor and slide blocks**

FORCES (kN)	DIRECTION OF POTENTIAL MOVEMENT OF ANCHOR BLOCK OR SUPPORT PIER	SYMBOLS ARE DEFINED AT THE END OF THIS TABLE
F_1 F_1 = combination of F_{lu} and F_{ld} $F_{lu} = (W_p + W_w)L_{lu}\ \cos \alpha$ $F_{ld} = (W_p + W_w)L_{ld}\ \cos \beta$ If pipe is straight, $F_1 = (W_p + W_w)(L_{lu}+L_{ld})\cos\alpha$	Uphill portion Downhill portion F_{1u} F_{1d} β	F_1 is the component of weight of pipe and water perpendicular to the pipe. Applies to both support piers and anchor blocks
F_2 $F_{2u} = f(W_p+W_w)L_{2u}\cos\alpha$ $F_{2d} = f(W_p+W_w)L_{2d}\cos\alpha$	Expansion: anchor below and expansion joint above F_2 α Contraction: anchor below and expansion joint above F_2 Directions for forces on support pier	F_2 is the frictional force due to the pipe sliding on the support piers. Applies to support piers and anchor blocks. The force acting at an anchor block is the sum of forces acting on the support blocks between the anchor block and expansion joints, but opposite in direction.
F_3 $F_3 = 2y_{water}\, h_{total} x(\Pi d^2/4)$ $\quad \sin(\beta - \alpha)/2$ $= 15.4h_{total}\, d_2 \sin(\beta - \alpha)/2$	F_3 $\dfrac{\alpha+\beta}{2}$ α β	F_3 is the hydrostatic force on bends that acts along the bisector of the bend. Only applies to anchor blocks that have horizontal and/or vertical bends
F_4 F_4 = combination of F_{4u} and F_{4d} $F_4 = W_p L_{4u} \sin\alpha$ $F_{4d} = W_p L_{4d} \sin\beta$	Uphill portion Downhill portion α β F_{4u} F_{4d}	F_4 is the component of pipe weight acting parallel to pipe. Applies to anchor blocks only. Calculate only if the angles (α or β) are larger than 20°.
F_5 $F_5 = 1000Ea\, T\Pi(d+t)t$ See Table 6.2 in Chapter 6 for values of E and a	Uphill portion Downhill portion F_{5d} F_{5u}	F_5 is the thermally induced force restrained by the anchor block in the absence of an expansion joint. Applies to anchor blocks only. Calculate only if expansion joints are not installed between anchor blocks.
F_6 $F_6 = 100d$	F_6 direction as F_5	F_6 is the factional force in the expansion joint. The F_6 force is felt because the joint will resist sliding. Applies to anchor blocks only.

F_7 $F_7 = \gamma_{water} \, h_{total} \, \Pi(d+t) \, t$ $\quad = 31 h_{total}(d+t)t$ Usually insignificant	Uphill portion Downhill portion F_{7d} F_{7u}	F_7 is the hydrostatic force on bends that acts along the bisector of the bend. Applies to anchor blocks only.
F_8 $F_8 = [2Q^2/(1/4) \, \Pi \, d^2] \sin(\beta - \alpha)/2$ $\quad = 2.5 \, (Q^2/d^2) \sin(\beta - \alpha)/2$ Usually insignificant	F_8 direction as F_3	F_8 is the dynamic force at a bend due to change in direction of moving water. Velocities are usually low in penstocks so this force is small. Applies to anchor blocks only.
F_9 $F_9 = \gamma_{water} \, h_{total} \times \Pi/4(d_{big}^2 - d_{small}^2)$ $\quad = 7.7 h_{total}(d_{big}^2 - d_{small}^2)$		F_9 is the force due to reduction in pipe diameter from d_{big} to d_{small} Applies to anchor blocks only.
F_{10} $F_{10} = \dfrac{\gamma_{soil} h_1^2}{2} \cos i \times K_a \times w$		F_{10} is the force due to soil pressure upstream of the block. Applies to both anchor blocks and support piers Calculate F_{10} if $(h_1 - h_2)$ is more than 1 m. The force acts at 1/3 of the height (h_1) from the base of the block.
W_b $W_b = Vol._{black} \times \gamma_{black}$		W_b is the weight of block. Applies to anchor blocks and support piers.

DEFINITION OF SYMBOLS USED IN TABLE 7.2

α = Upstream penstock angle with respect to the horizontal.

β = Downstream penstock angle with respect to the horizontal.

γ = Unit weight in kN/m^3

γ_{water} = 9.8 kN/m^3

$\gamma_{concrete}$ = 22 kN/m^3

$\gamma_{masonary}$ = 20 kN/m^3

$\gamma_{pipe\ material}$ = See Table 6.2

γ_{soil} = see Table 7.3

ϕ = Soil angle of friction, see Table 7.3

a = Coefficient of linear expansion of pipe ($^{\circ}c^{-1}$), see Table 6.2 in Chapter 6 where is the symbol used

d = Pipe internal diameter (m)

d_{big} = Internal diameter of larger pipe in case of reduction in pipe diameter.

d_{small} = Internal diameter of smaller pipe in case of reduction in pipe diameter.

E = Young's modulus of elasticity, see Table 6.2 in Chapter 6.

f = Coefficient of friction between pipe and support piers.

g = acceleration due to gravity = 9.8 m/s^2

h_1 = Buried depth of block at the upstream face.

h_2 = Buried depth of block at the downstream face.

h_{total} = Total head including surge.

i = Uphill ground slope (Figure 7.5). Note that

i = may not always be equal to á

K_a = Active soil pressure coefficient as follows:

K_a = $\dfrac{\cos i - \sqrt{\cos^2 i - \cos^2 \theta}}{\cos i + \sqrt{\cos^2 i - \cos^2 \theta}}$

L_{1d} = Half the distance from anchor block centreline to the centreline of the first downstream support pier (Figure 7.5).

L_{1u} = Half the distance from anchor block centreline to the centreline of the first upstream support pier (Figure 7.5).

L_{2d} = Distance between two consecutive support piers downstream of the anchor block.

L_{2u} = Distance between two consecutive support piers upstream of the anchor block.

L_{4d} = Distance from the anchor block centreline to the downstream expansion joint (Figure 7.5).

L_{4u} = Distance from the anchor block centreline to the upstream expansion joint (Figure 7.5).

Q = Flow in the penstock pipe (m^3/s).

t = Wall thickness of penstock (m).

t = Maximum temperature change ($^{\circ}C$) that the pipe will experience after being fixed at anchor blocks.

W = Width of the anchor block in m.

W_p = Weight of pipe in kN/m

= $\Pi(d+t)t\ \gamma_{pipe\ material}$

W_w = Weight of water in kN/m

= (Pipe area in m^2) x γ_{water}
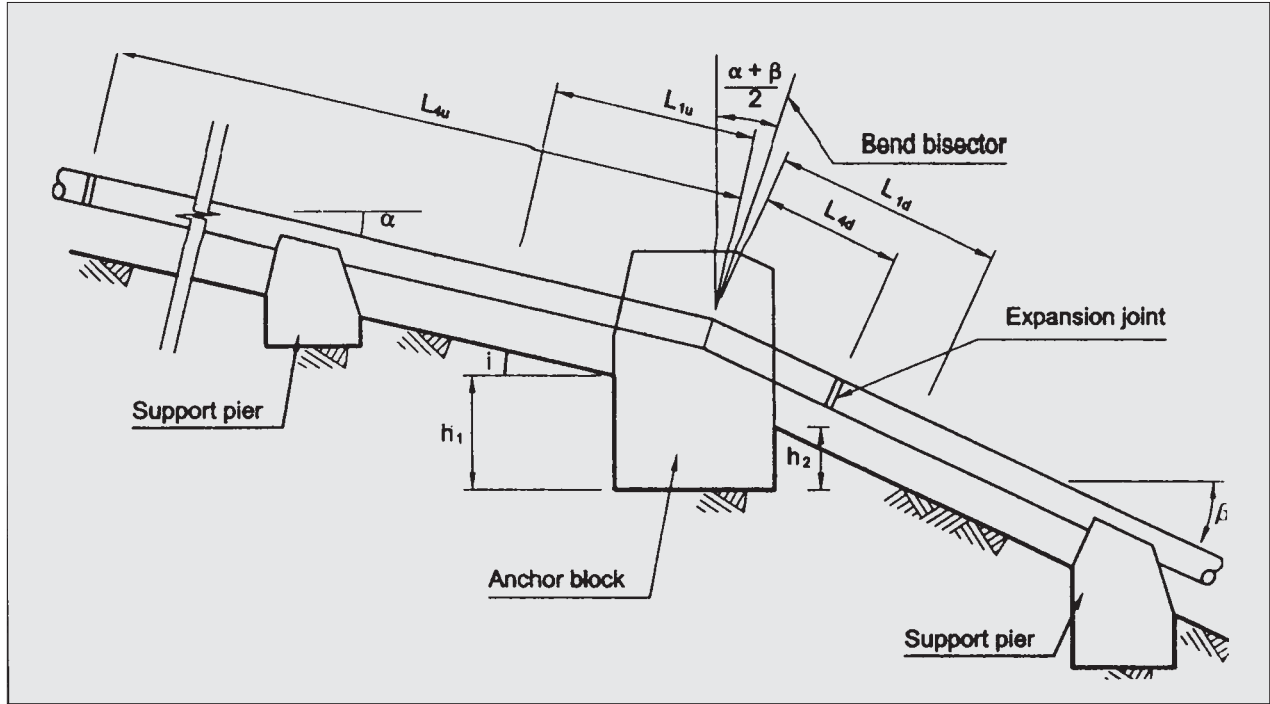
Figure 7.5 Distances and angles used in anchor block and support pier equation

7.4.2 DESCRIPTION OF FORCES

$F_1 - F_1$ is the component of the weight of pipe and enclosed water perpendicular to the pipe alignment. If there is a bend at the anchor, however, both the upstream and downstream lengths of pipe contribute separately, each force perpendicular to the centreline of the pipe segment which contributes to it.

$F_2 - F_2$ is the frictional force of pipe on support piers. If the penstock moves longitudinally over support piers, a friction force on the pipe is created at each pier. A force "F_2", equal to the sum of all these forces but opposite in direction, acts on the anchor. This force exists only where one or more support piers are located between the anchor block and an expansion joint. For example, if an expansion joint is located immediately downhill of the anchor, friction forces on the downhill length of pipe will not be transmitted to the anchor block from that side. The friction coefficient, f, depends on the material against which the penstock slides and is as follows:
steel on concrete, f = 0.60
steel on steel, rusty plates, f = 0.50
steel on steel, greased plates or tar paper in between, f = 0.25

$F_3 - F_3$ is the force due to hydrostatic pressure within a bend. The hydrostatic pressure at a bend creates a force which acts outward for upward bends and inward if the bend is downward. This is a major force which must be considered in designing anchor blocks. However, the block size can be significantly reduced if the bend angle ($\beta - \alpha$) can be minimised while fixing the penstock alignment.

$F_4 - F_4$ is the force due to the component of the weight of pipe parallel to the pipe alignment. On a slope, the component of the weight of the pipe which is parallel to the pipe tends to

pull it downhill and exerts a force on an anchor block. The sections of pipe both upstream and downstream of an anchor block may have to be considered. The lengths 'L_{4u}' and 'L_{4d}' in the equation for the force "F_4" acting on an anchor block are the lengths of the upstream or downstream section of the penstock which is actually to be held by that block. The upstream section may begin at the forebay or, more usually, at an expansion joint. The downstream section usually ends at an expansion joint. If the expansion joint downstream of an anchor block is located near the anchor, as it usually is, the force arising from the weight of the downhill section of pipe between the anchor and the joint is insignificant and is usually neglected. Also, the anchor block will not experience this force if the penstock is buried since the ground friction will resist this force.

$F_5 - F_5$ is the force that is transmitted to the anchor block due to thermally induced stresses in the absence of an expansion joint. If an exposed section of a rigid pipe does not incorporate an expansion joint, thermally induced stresses build up in the pipe and act on the anchor block. The associated force "F_5" may push against the anchor block (with increasing temperature) or pull the anchor block (with decreasing temperature).

$F_6 - F_6$ is the force due to friction within the expansion joint. To prevent leaking, the packing within an expansion joint must be tightened sufficiently. However, this tightening also makes it more difficult for the joint to accept any longitudinal movement of the pipe. Friction between the packing and the concentric sleeves in the expansion joint creates a force "F_6" which opposes any expansion or contraction of the pipe. This force is dependent on pipe diameter, tightness of the packing

gland and smoothness of sliding surfaces. If there is not a change in the pipe direction ($\alpha = \beta$) upstream and downstream of the anchor block, the forces (from upstream and downstream expansion joints) cancel out.

F_7 - F_7 is the hydrostatic force on exposed ends of pipe in expansion joints. The two sections of penstock pipe entering an expansion joint terminate inside the joint; therefore, their ends are exposed to hydrostatic pressure, resulting in a force "F_7" which pushes against the anchors upstream and downstream of the joint. This force usually contributes minimally to the total forces on an anchor since the ratio of pipe thickness to the diameter is low. However, this force can be significant at mild steel-HDPE joint section (since HDPE pipes are thicker). Note that h_{total} is the total head at the expansion joint.

F_8 - F_8 is the dynamic force at the pipe bend. At the bend, the water changes the direction of its velocity and therefore the direction of its momentum. This requires that the bend exert a force on the water. Consequently, an equal but opposite reaction force "F_8" acts on the bend; it acts in the direction which bisects the exterior angle of the bend (same as F_5). Since velocities in penstocks are relatively low (< 5 m/s), the magnitude of this force is usually insignificant.

F_9 - F_9 is the force exerted due to the reduction of pipe diameter. If there is a change in the diameter of the penstock, the hydro-static pressure acting on the exposed area creates a force "F" which acts in the direction of the smaller diameter pipe. If the penstock length is long (as in the case of Jhankre mini-hydro), then the pipe thickness is increased with increasing head. However, the effect of changing the diameter by a few mm does not contribute significant forces and can be ignored.

F_{10} - F_{10} is the force on the anchor blocks or support piers due to the soil pressure acting on the upstream face. If there is a significant difference between the upstream and downstream buried depth (h_1 - h_2 > 1 m) of the block then a force will be exerted on the anchor block due to soil pressure. In such cases, this force should be considered since it has a destabilising effect. Note that the resultant of this force acts at 1/3 h_1.

7.4.3 DESIGN PROCEDURE

Once all of the above relevant forces have been calculated the design procedure for anchor blocks and support piers requires checking the three conditions of stability as follows:

Safety against overturning

The forces acting on the structure should not overturn the block. For structures that have rectangular bases, this condition is met if the resultant acts within the middle third of the base. This is checked as follows:

- First take moments about one point of the block along the face parallel to the penstock alignment.
- Find the resultant distance at which the sum of vertical forces act using the following equation:

$$d = \left(\frac{\Sigma M}{\Sigma V} \right)$$

where:

d is the distance at which the resultant acts.
ΣM is the sum of moments about the chosen point of the block.
ΣV is the sum of vertical forces on the block.

- Now calculate the eccentricity of the block using the following equation:

$$e = \left| (L_{base} / 2) - d \right|$$

- For the resultant to be in the middle third of the block, the eccentricity must be less than 1/6 of the base length as follows:

$$e_{allowable} = L_{base} / 6$$

Finally check that $e < e_{allowable}$

Safety on bearing

The load transmitted to the foundation must be within the safe bearing capacity limit of the foundation material. If the transmitted load exceeds the bearing capacity limit of the foundation, the structure will sink. The bearing pressure at the base is checked using the following equations:

where:

$$P_{base} = \frac{\Sigma v}{A_{base}} \left(1 + \frac{6e}{L_{base}} \right)$$

P_{base} = maximum pressure transmitted to the foundation.
ΣV_{base} = the sum of vertical forces acting on the block.
L_{base} = length of the base.
A_{base} = the base area of the block eccentricity calculated earlier.
e   = eccentricity calculated earlier.

The calculated P_{base} must be less than the allowable bearing pressure ($P_{allowable}$) for the type of soil at the foundation level. Allowable bearing pressure for different types of soil is shown in Table 7.3.

Unit weight (γ), angle of friction (Ø) and allowable bearing pressure for different soil types

SOIL TYPE	UNIT WEIGHT, Y (KN/m³)	FRICTION ANGLE (Ø°)	ALLOWABLE BEARING PRESSURE (KN/m²)
Soft clays and silts	16	22	50
Firm clays and firm sandy clays	17	25	100
Stiff clays and stiff sandy clays	20	30	200
Very stiff boulder clays	20	32	350
Loose well graded sands and sand /gravel mixture	18	31	100

Safety against sliding

The structure should not slide over its foundation. The following equation is used to check this condition:

$\mu \, \Sigma V / \Sigma H \geq 1.5$;

where:

$\Sigma H =$ sum of horizontal forces

$\mu \quad =$ Friction coefficient between the block and the foundation.

A value of $\mu = \tan$ Ø, but not exceeding 0.5, is recommended for friction between masonry or concrete on soil.

$\Sigma V =$ the Sum of vertical forces acting on the block

$L_{base} =$ length of the base.

$A_{base} =$ the base area of the block.

Example 7.1 Design of an anchor block

Design for one of the jhankre mini-hydro anchor blocks. The following information is provided:

Pipe diameter = 450 mm,

Pipe thickness = 4 mm.

$h_{gross} \quad$ = 60 m,

$h_{surge} \quad$ = 48 m,

$\alpha = i = 13°$, $\beta = 25°$

Distance to upstream support pier = 4 m

∴ L_{1u} = 2m

Distance to downstream support pier = 4 m

∴ L_{1d} = 2m

Distance to upstream expansion joint = 30 m.

∴ L_{4u} =30m

There are 8 support piers at 4 m centre to centre spacing (L_{2u} = 4 m) up to the upstream anchor block. To reduce friction, all support piers are provided with steel shaddle plates and tar paper on top of the plates as in Figure 7.4. An expansion joint will be located just downstream of the block. The soil type is stiff clay.

CALCULATIONS

$H_{total} \quad = \quad h_{gross} + h_{surge}$

$\quad = \quad$ 60m + 48m

$\quad = \quad$ 108

Consider the block shape shown in Figure 7.6.

Block volume excluding volume of the pipe

$= \{(2.25 \times 3) + (1/2 \times 3 \times 1.05)\} \times 2 - 1 \times \Pi \times 0.458^2$

$/4 \cos 13° - 2 \times \Pi \times 0.458^2 / 4 \cos 25° = 16.12 \text{ m}^3$

Unit weight of concrete, $(\gamma_{concrete})$ = 22 kN /m³

Weight of block,
$W_B = 16.12 \times 22 = 354.64$ kN

Weight of pipe,

$W_p \quad = \Pi(d + t) t \, \gamma_{steel}$
$\quad = \Pi \times 0.454 \times 0.004 \times 77$
$\quad = 0.44$ kN/m

$W_w \quad = \dfrac{\Pi(0.450^2)}{4} \times 9.8$

$\quad = 1.56$ kN/m

$W_p + W_w \quad = \quad 2.00$ kN / m

Calculate the relevant forces:

1. $F_{1u} \quad = (W_p + W_w) \, L_{1u} \, COS \, \alpha$
$\quad = (2.00) \times 2 \times \cos 13° = 3.90$ kN

2. $F_{1d} \quad = (W_p + W_w) \, L_{1d} \, COS \, \beta$
$\quad = (2.00) \times 2 \times \cos 25° = 3.63$ kN

3. Frictional force per support pier:
$\quad = \pm f \, (W_p + W_w) \, L_{2u} \, COS \, \alpha$
$f \quad = 0.25$ for steel on steel with tar paper in between,
$\quad = \pm 0.25(2.00) \times 4 \cos 13°$
$\quad = \pm 1.95$ kN per support pier

Since there are 8 support piers
F_{2u} on Anchor block $= \pm 1.95 \times 8 = \pm 15.6$ kN
Note that F_{2d} is zero since an expansion joint is located immediately downstream of the anchor block.

4. $F_3 \quad = 15.4 \, h_{total} \, d^2 \sin[(\beta - \alpha) \, /2]$
$\quad = 15.4 \times 108 \times (0.450)^2 \sin\left(\dfrac{25° - 13°}{2} \right)$
$\quad = 35.20$ kN

5. $F_{4u} \quad = W_p L_{4u} \sin\alpha$
$\quad = 0.44 \times 30 \times \sin 13°$
$\quad = 2.97$ kN

Figure 7.6 Proposed anchor block shape

Note that F_{4u} is insignificant since α is less than 20° and could have been ignored as discussed in Table 7.2. F_{4u} has been calculated here only to show how it is done. F_{4d} is negligible since an expansion joint is placed immediately downstream of the anchor block., i.e., $L_{4d} \approx 0$ and therefore

6. $F_6 \quad = 100 \times d = 100 \times 0.450 = 45$ kN

7. $\quad F_7 \quad = 31 h_{total} \, (d + t)t$
$\quad F_{7u} \quad = 31 \times (108 - 30 \sin\alpha) \times 0.454 \times 0.004$
$\quad\quad = 5.70$ kN
$\quad F_{7d} \quad = 31 \times 108 \times 0.454 \times 0.004 = 6.08$ kN

Note that as discussed earlier the resultant of these forces is insignificant.

8 $\quad F_8 \quad = 2.5 \, (Q^2/d^2) \, \sin[(\beta-\alpha)/2]$
$\quad\quad = 2.5(0.450^2/0.450^2) \, \sin[(25°-13°)/2]$
$\quad\quad = 0.26$ kN

Note that as discussed earlier, this force is insignificant.

9. $F_9 = 0$ since the pipe diameter does not change.

10. Soil force, F_{10}
From Table 7.3, $(\gamma_{soil} = 20$ kN/m^3 and $\varnothing = 30°$ for stiff clay.
Recall that i = 13°

$K_a \quad = \dfrac{\sqrt{\cos i - \ \cos^2 i - \cos^2\varnothing}}{\cos i + \sqrt{\cos^2 i - \cos^2\varnothing}} = 0.371$

$F_{10} \quad = \dfrac{\gamma_{soil} h_1^{\,2}}{2} \, \cos i \times k_0 \times w$

$\quad = \dfrac{20 \times 1.8^2}{2} \cos 13° \times 0.371 \times 2$

$\quad = 23.45$ kN

This force acts at 1/3 of the buried depth at upstream face of anchor block from point 0 as shown in Figure 7.7, which is (1/3 × 1.8) = 0.6 m.

Resolution of forces
$\alpha = 13°$, $\beta = 25°$

Forces (kN)	X-component (kN)→ +	Y-component (kN)↓ +
$F_{1u} = 3.90$	$- F_{1u} \sin\alpha$ $= - 0.88$	$+ F_{1u} \cos\alpha$ $= + 3.80$
$F_{1d} = 3.63$	$- F_{1d} \sin\beta$ $= -1.53$	$+ F_{1d} \cos\beta$ $= +3.29$
$F_{2u} = \pm 15.6$	$\pm F_{2u} \cos\alpha$ ± 15.20 Positive during expansion 1 Negative during contraction 2	$\pm F_{2u} \sin\alpha$ ± 3.51 Positive during expansion 1 Negative during contraction 2
$F_3 = 35.20$	$+ F_3 \sin[(\beta + \alpha) /2]$ $= + 11.46$	$- F_3 \cos[(\beta + \alpha) / 2]$ $= - 33.29$
$F_{4u} = 2.97$	$+ F_{4u} \cos\alpha$ $= +2.97\cos 13°$ $+2.89$	$+ F_{4u} \sin\alpha$ $= - 2.97 \sin 13°$ $= 0.67$
$F_6 = 45$	$\pm F_6 (\cos\alpha - \cos\beta)$ $\pm 45 (\cos 13° - \cos 25°)$ ± 3.06 Positive during expansion 1 Negative during contraction 2	$\pm F_6 (\sin\beta - \sin\alpha)$ $\pm 45 (\sin 25° - \sin 13°)$ ± 8.90 Positive during expansion 1 Negative during contraction 2
$F_{7u} = 5.70$	$+ F_{7u} \cos\alpha$ $= 5.70 \cos 13°$ $= + 5.55$	$+ F_{7u} \sin\alpha$ $5.70 \sin 13°$ $= 1.28$
$F_{7d} = 6.08$	$= - F_{7d} \cos\beta$ $= - 6.08 \cos 25°$ $= - 5.51$	$- F_{7d} \sin\beta$ $= - 6.08 \sin 25°$ $= - 2.57$
$F_8 = 0.26$	$+ F_8 \sin [(\beta + \alpha) /2]$ $= 0.26 \sin [(25° + 13°)/2]$ $= + 0.09$	$+ F_8 \cos [(\beta + \alpha) /2]$ $= 0.26 \cos [(25° + 13°)/2]$ $= - 0.25$
$F_{10} = 23.45$	$+ F_{10} \cos i$ $= 23.45 \cos 13°$ $= + 22.85$	$+ F_{10} \sin i$ $23.45 \sin 13°$ $= + 5.28$
$W_B = 354.64$	0	$+354.64$
SUM	$\Sigma H = +53.18$ 1) Expansion $\Sigma H = + 16.66$ 2) Contraction	$\Sigma V = 345.26$ 1) Expansion $\Sigma V = 320.44$ 2) Contraction

Note that forces are positive in X-direction is towards the right and Y-direction downwards.

Calculate the centre of gravity of the block from the upstream face of the block taking the moment of mass.

The effect of the pipe passing through the block is negligible, so need not be calculated.

$[\{(3 \times 2.5)3/2\} + \{(1/2) \times 3 \times 1.05 (1/3)3\}2] / [\{(3 \times 2.25) + (1/2)3 \times 1.05\}2]$ $= 1.41$ m

$\therefore$ The weight of the block W_B acts 1.41 m from point O .

Sum of horizontal forces that act at the bend

$\Sigma H - F_{10X}$

1. Expansion case 53.18 - 22.85 = 30.33 kN→
2. Contraction case 16.66 - 22.85 = - 6.19 kN←

Sum of vertical forces that act at the bend

$\Sigma v - F_{10y} - W_B$
1. Expansion case
345.26 - 5.28 - 354.64 = 14.66 kN↑
2. Contraction case
320.44 - 5.28 - 354.64 = 39.48 kN↑

Now draw a force diagram on the block as shown in Figure 7.7.

Check if structure is safe against overturning:

1. → Expansion case

Take sum of moments about point 0 with clockwise moments as positive:
$\Sigma M@0$ = 30.33x2.15 + 22.85x0.6 + 354.64x1.41-14.66x1.0 = 564.30 kN-m

$d = \left(\frac{\Sigma V}{\Sigma M}\right) = 564.30/345.26 = 1.63$ m

$e = |3/2 - 1.63| = 0.13$m

$e_{allowable} = L_{base}/6 = 3/6 = 0.5$ m

$\therefore e < e_{allowable}$

2. → Contraction case

Take sum of moments about point 0 with clockwise moments as positive:
$\Sigma M@0$ = -6.19x2.15 + 22.85x0.6 + 354.64x1.41-39.48x1.0 = 460.96 kN-m = 460.96

$d = \left(\frac{\Sigma V}{\Sigma M}\right) = 460.96/320.44 = 1.44$ m

$e = |3/2 - 1.44| = 0.06$ m

Recall that $e_{allowable} = 0.5$

$\therefore e < e_{allowable}$

Since $< e_{allowable}$ for both cases, the structure is safe against over turning.

Check if the structure is safe on bearing capacity:
Note that for stiff clay allowable bearing pressure is 200kN/m² (Table7.3).
1. → Expansion case:
$P_{base} = \left(\frac{\Sigma V}{A_{base}}\right) 1 + 6e/L_{base}$

P_{base} = 345.26/6 [1 + (6x0.13/3)] = 72.5 N/m²

2. → Contraction case:
$P_{base} = \left(\frac{\Sigma V}{A_{base}}\right) 1 + 6e/L_{base}$

P_{base} = 320.44/6 [1 + (6x0.06/3)] = 59.8 N/m²
In both case $P_{base} < P_{allowable}$ = 200kN/m²
∴ the structure is safe against sinking.

Check if the block is safe against sliding:

1. → Expansion case
$\Sigma H < \mu\Sigma V$
μ = 0.5 for concrete/masonary on oil
53.18 kN < 0.5x.45.26 kN
53.18 kN < 160.22 kN   OK.

2. → Contraction case
$\Sigma H < \mu\Sigma V$

Since $\Sigma H < \mu\Sigma V$ in both cases the structure is safe against sliding.
∴. The anchor block is stable. The calculations could be repeated to justify using a smaller, more economical block.
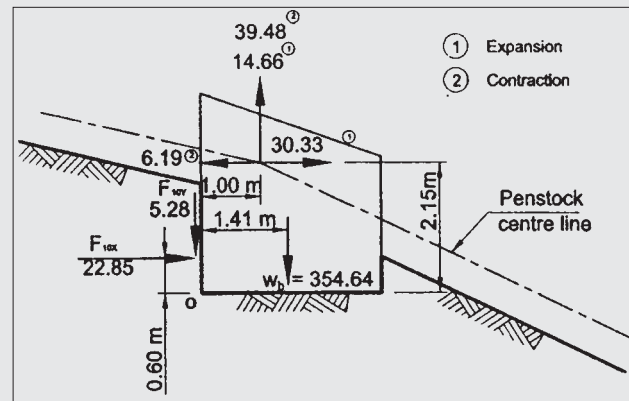
Figure 7.7 Force diagram on the anchor block

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7.4.4 SIZING OF ANCHOR BLOCKS FOR SMALL SCHEMES

For micro-hydro schemes with a gross head less than 60m and an installed power capacity less than or equal to 20 kW, the following guidelines can be used to determine the size of an anchor block:

- At a straight section, locate one anchor block after every 30 m distance (as discussed earlier) by placing 1m³ of plum concrete for each 300mm of pipe diameter. For example, if the pipe diameter is 200 mm, then:

  1 x (200 / 300) =0.67 m³ of concrete volume is required .

- At a penstock bend, where the bend angle is less than 45° i.e. (β-α), double the concrete volume than what is required for a straight section. For example if the pipe diameter is 200mm and the bend is 20°, then:

2x(200 / 300) = 1.33m³ of concrete is required for the anchor block.

- Similarly, if the bend angle is larger than 45°, then the required concrete volume should be three times that for a straight section. For example if the pipe diameter is 350 mm and the bend is 50°, then:

  3x (350/300) = 3.5m³ of concrete is required for the anchor block.

John Bywater has developed a more sophisticated method of sizing anchor blocks for small schemes. This is available through Practical Action.
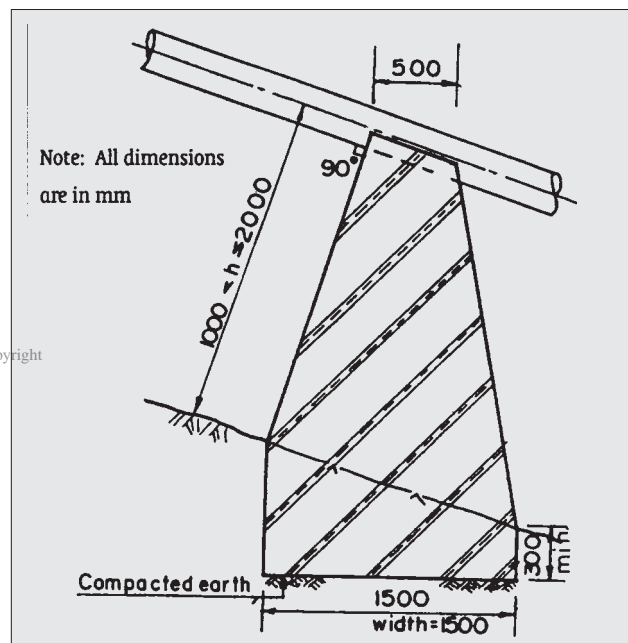
Figure 7.9 Support pier for small schemes with ground height of 1 m to 2 m

7.4.5 SIZING OF SUPPORT PIERS FOR SMALL SCHEMES

For small schemes (gross head less than 60 m and power capacity limited to 20 kW) Figures 7.8 and 7.9 can be used as guidelines to size for support piers.

If the penstock alignment is less than 1 m above the ground, Figure 7.8 can be used as a guide for the shape of the support pier. The minimum length and width at the base should be 1 m x 1 m and the top width parallel to the penstock alignment should be 0.5m. The width at the top perpendicular to the penstock pipe route should be kept 1 m and the uphill wall surface should be perpendicular to the penstock pipe. A minimum foundation depth of 300 mm should be provided. Similarly, if the penstock pipe is 1-2m above the ground, Figure 7.9 can be use as a guideline. Note that the structure is similar to Figure 7.8 except that the base length and width are 1.5m x 1.5m.

For larger schemes all relevant forces should be resolved and conditions of stability should be checked as discussed earlier.

7.4.5 Checklist for anchor block and support pier works

- Have anchor blocks been located at exposed penstock length intervals exceeding 30 m even when there are no bends?
- For anchor blocks, has a minimum cover of 300 mm around the pipe been provided? Is adequate reinforcement included?
- If there are a lot of downwards bends and wood is expensive at site consider using masonry anchor blocks. Also for the buried sections, dry stone walls can be used as permanent formwork.
- Has adequate foundation depth been provided for both support piers and anchor blocks? Be sure to include steel plates and tar paper on support piers to minimise friction.
- Have all relevant forces on both support piers and anchor blocks been checked as discussed in Section 7.4?
- Finally refer to Chapter 9 for issues concerning stability.
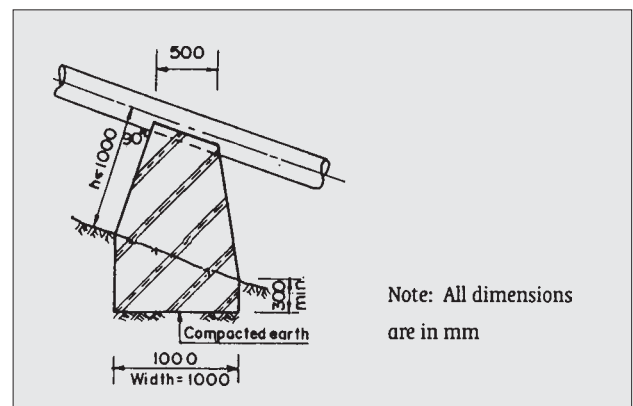
Figure 7.8 Support pier for small schemes with ground height of less than 1 m

8. Powerhouse and tailrace
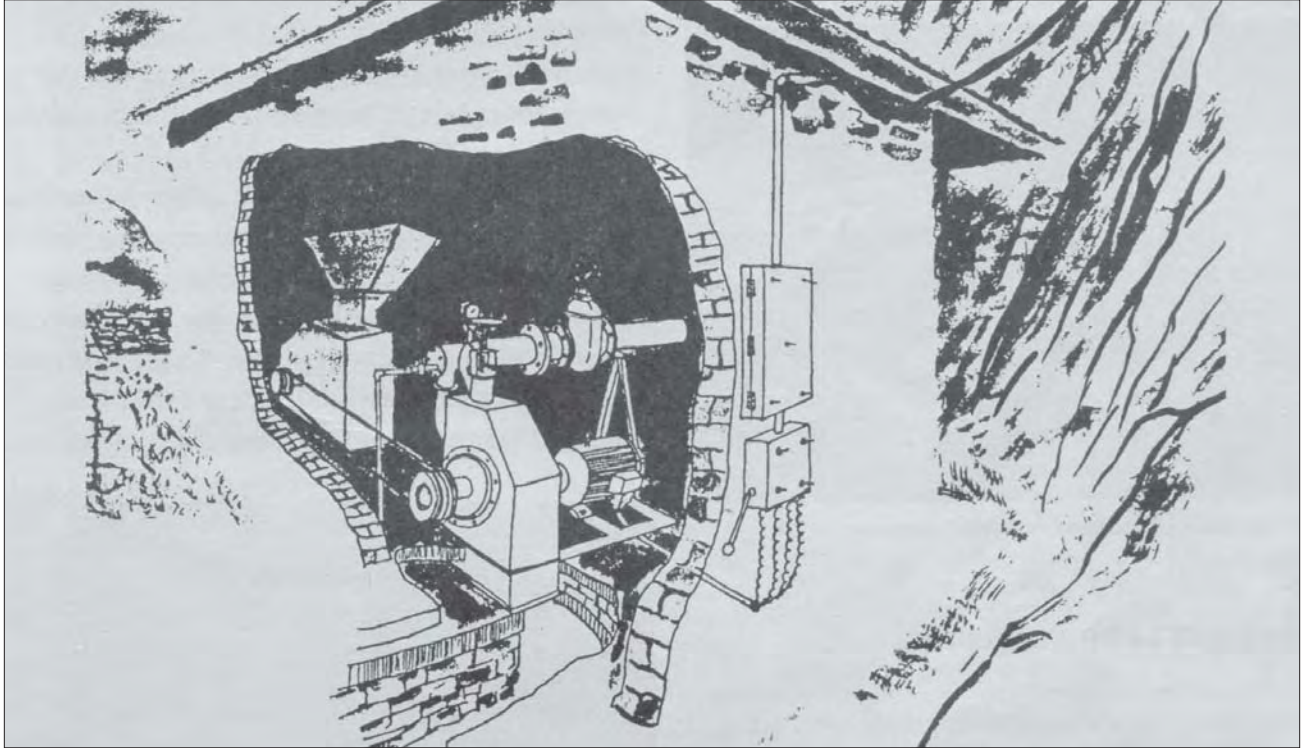
8.1 Overview

The powerhouse accommodates electro-mechanical equipment such as the turbine, generator, agro-processing units and control panels. The main function of this building is to protect the electro-mechanical equipment from adverse weather as well as possible mishandling by unauthorised persons. The powerhouse should have adequate space such that all equipment can fit in and be accessed without difficulty. Cost can be brought down if the construction is similar to other houses in the community. The powerhouse of the Barpak micro-hydro scheme cun be seen in Photograph 8.1. This building is similar to other local houses in the community. Note that the transformer is fenced to prevent accidents due to unauthorised access. The generators, turbines and the belt drives need to be securely fixed on the machine foundation in the powerhouse. This requires a careful design since the equipment generates dynamic forces and even a slight displacement can cause excessive stresses on various parts of the equipment and lead to equipment malfunction. The tailrace is a channel or a pipe that conveys water from the turbine (after power generation) back into the stream; generally the same stream from which the water was initially withdrawn. The powerhouse and tailrace of the Salleri Chialsa mini-hydro scheme can be seen in Photograph 8.2
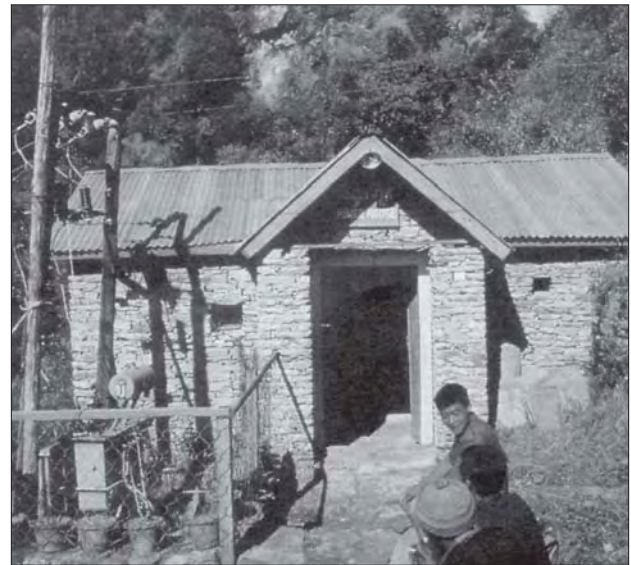
Photo 8.1 Powerhouse of the Barpak mini- hydro scheme

Photo 8.2 Powerhouse and tailrace of the Salleri Chialsa mini-hydro scheme

Photo 8.3 Powerhouse of the Jhankre mini-hydro

8.2 Location of powerhouse

The location of the powerhouse is governed by the penstock alignment since this building must be located at the end of the penstock. Apart from this, the following criteria are recommended for locating the powerhouse:

- The powerhouse should be safe from not only annual floods but also rare flood events. Discussions should be held with the local community members to ensure that floodwaters have not reached the proposed powerhouse site within at least the past 20 years. For micro-hydro schemes of 50-100 kW it is recommended that the powerhouse be above the 50-year flood level.
- It should also be possible to discharge the tailwater safely from the powerhouse back to the stream.
- If possible the powerhouse should be located on level ground to minimise excavation work.
- The proposed location should be accessible throughout the year. At some places this may require constructing a new foot trail.
- The powerhouse should be located close to the community that it serves, provided that the penstock alignment and other parameters are feasible and economical. This will reduce the transmission line cost, and if agro-processing units are also installed in the powerhouse, the community will not have to carry their grain for a long distance.

Other stability issues discussed in Chapter 9 should also be addressed.

8.3 Design of powerhouse

8.3.1 GENERAL
A powerhouse that is similar to other local houses in the community is generally economical and appropriate. The community members will be able to construct such a building with nominal supervision.

If a decision is made to construct the powerhouse similar to other local houses, then the civil design input required is to size the plan area of the building and design the machine foundation. The area inside the powerhouse should be well lit and ventilated with sufficient windows. Placing a few transparent fibreglass sheets (skylight) in the roof will provide additional illumination as can be seen in Photograph 8.3.
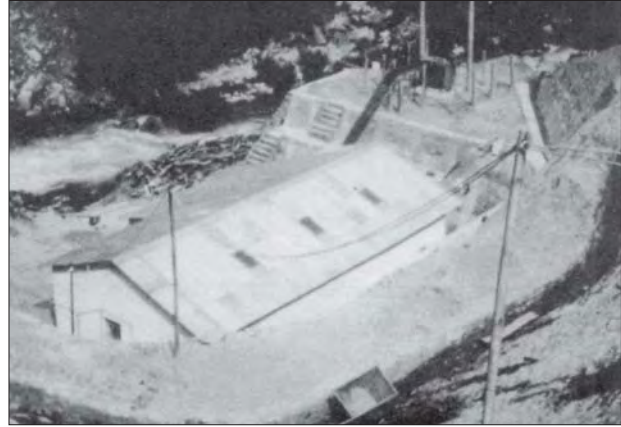
8.3.2 SIZE OF THE POWERHOUSE
The plan area of the powerhouse should be determined as follows:

- The size of the electro-mechanical equipment should be obtained from the manufacturer.
- All required equipment should be drawn to scale and placed on the proposed powerhouse plan area. This may require a few trials to determine the optimum layout.
- Adequate space should be provided such that all equipment is easily accessible. There should be a clear spacing of at least 1 m around each item of equipment that has moving parts (such as the generator, turbine and the belt drive). Noted that if agro-processing equipment is installed, the community members will regularly visit the powerhouse (to process their grain). Therefore additional space is required so that the powerhouse does not become overcrowded and a potential area for accidents. It is recommended that such additional space is provided as a lobby at the entrance and the equipment is placed beyond it. A lobby large enough for five people to wait with their grain (about 3m x 3m) may be adequate in most cases.
- Adequate windows should be provided for lighting and ventilation. Note that the door and windows need to be located such that they do not obstruct access to the equipment. This requires co-ordinating the locations of the equipment, windows and the door.

The powerhouse layout of the Jhankre mini-hydro scheme is shown in Figure 8.1.

8.3.3 POWERHOUSE WALL AND OTHER DETAILS
Most houses in the hills of Nepal are constructed of stone masonry in mud mortar. Wooden truss with corrugated iron sheet (CGI), slates or straw thatching are used for the roofs. A similar building is recommended for the powerhouse structure with the following considerations:
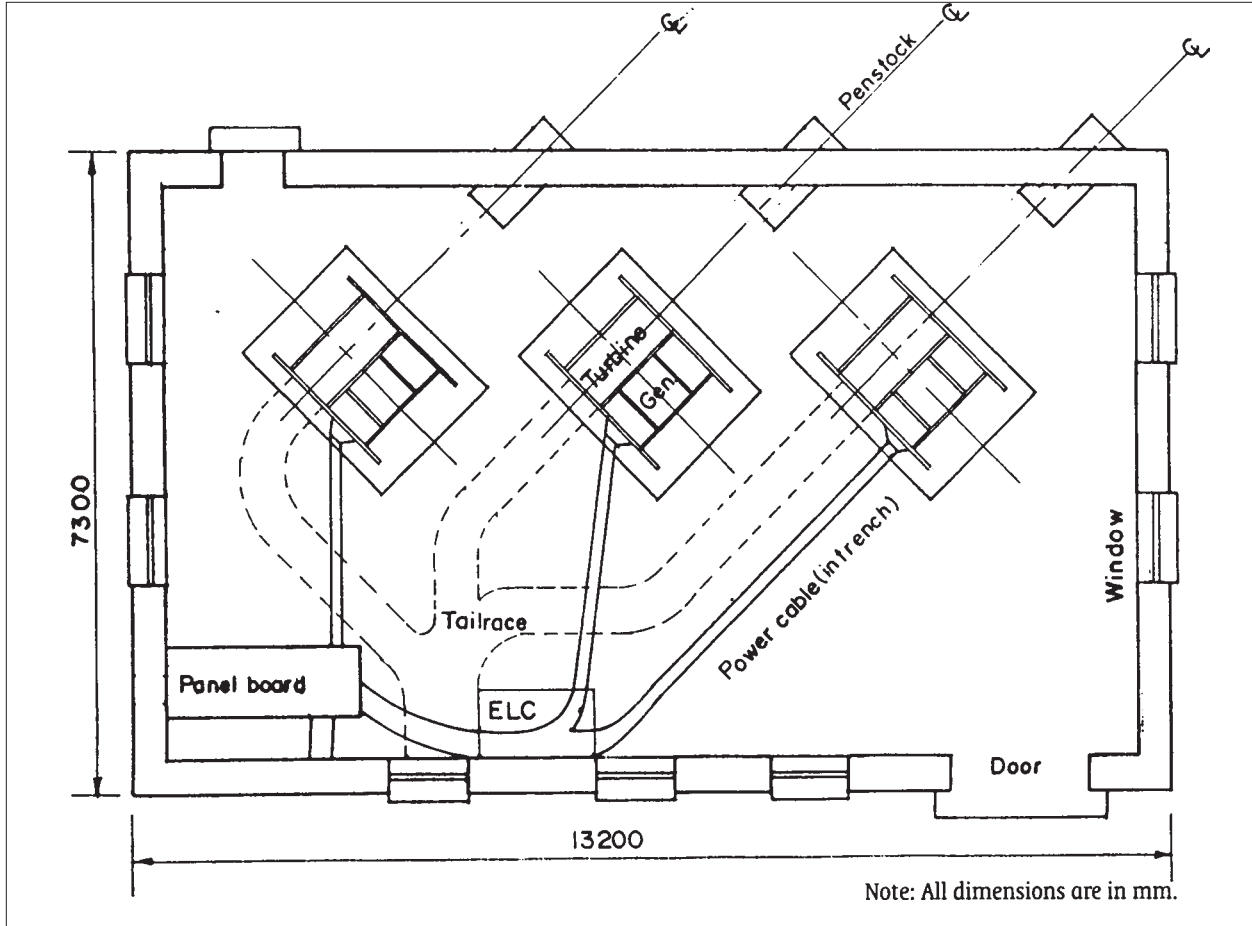
Figure 8.1 Powerhouse floor plan at the Jhankre mini-hydro scheme

- The minimum wall thickness of the stone masonry walls (mud mortar) should be 450mm. If funds are available at least the external surface of the walls should be plastered in cement-lime mortar (12mm thick, 1:1:6 mix) to keep away moisture from rain.
- The penstock pipe should not normally be built into the powerhouse wall, otherwise the wall could be damaged by vibration from the turbine. The recommended solution is to leave an oversized opening in the wall; an alternative is to place a bigger pipe outside the penstock pipe where it passes through the wall. At Jhankre mini-hydro the wall was locally thickened to act as a support pier for the penstock pipes entering the powerhouse.
- The clear height of the building should be 2.5m to 3m.
- For larger schemes, provision must be made for lifting equipment that cannot be lifted by hand. A crane is expensive; more appropriate is a block and tackle supported by a beam or temporary A-frame.
- CGI sheets should be used for the roofing, since they are relatively fire resistant and leak proof.
- The floor of the powerhouse should be 300mm to 500mm above the outside ground surface to prevent dampness and rainwater entering. Drains should also be provided outside the powerhouse.
- Doors and windows should open outwards for safety in case of fire or flooding.

8.4 Design of machine foundation

The design of the machine foundation is similar to that of an anchor block, but simpler. The most significant forces that the machine foundation can experience are as follows:

Photo 8.4 Construction of Jhankre mini-hydro machine foundation.

- The thrust due to hydrostatic force when the valve at the powerhouse is closed or the total head including surge due to sudden blockage of flow. If there is an expansion joint upstream of the valve, the entire head will be transferred to the machine foundation from the turbine housing.
- The vertical force due to the weight of the foundation block, the turbine and the generator.

Soil forces do not need to be considered because they are balanced on each side of the foundation.

Since a homogeneous and rigid structure is required the machine foundation should be constructed of reinforced concrete. The design process is to tentatively size the machine foundation and then check the structure against overturning, sliding and sinking as in the case of anchor blocks.

The Jhankre mini-hydro machine foundation pits can be seen under construction in Photograph 8.4. The machine foundation of the Galkot scheme can be seen in Drawing 420/C/ 3C03 of Appendix C. Placing the turbine pit floor 0.3 m below tailrace invert level helps to reduce abrasion by the water leaving the turbine.

Example 8.1 illustrates the design principles of a machine foundation.

Example 8.1 Design of machine foundation

Design a machine foundation to support a directly coupled turbine and generator. The following information has been provided:
Penstock pipe diameter = 300 mm, mild steel.
The pipe centreline is 300 mm above the powerhouse floor.

Q_{design} = 150 1/s
Gross head (h_{gross}) = 51 m
Expected maximum surge head (h_{surge}) = 50 m
Water level in the tailrace channel = 0.25 m
Weight of turbine (W_T) = 300 kg
Weight of generator (W_G) = 350 kg

Site conditions reveal that the foundation needs to be constructed on soil.

Calculations:

Try a reinforced concrete structure with dimensions as shown in Figures 8.2 and 8.3.

h_{total} = h_{gross} + h_{surge} = 51m + 50m = 101m

Force due to h_{total}, (F_H) = (Pipe area) x 101 m x unit weight of water
= Ð 0.3^2/4 m^2 x 101m x 9.8 kN/m³
or F_H = 69.960 N = 70.0 kN

Weight of turbine (W_T) = 300 kg = 300 x 9.8 = 2940 N = 2.94 kN

Weight of generator(W_G) = 350 kg = 350 x 9.8 = 3430 N = 3.43 kN

Place all forces on the machine foundation and divide the block in three sections W_1 , W_2 and W_3 as follows:

Calculate the weight of three sections of the block using 22 kN/m³ for unit weight of concrete.

W_1 = 0.4x1.5m x 2.5m x22kN/m³ =33.00 kN
W_2 = [(0.45 x 1.5 x 2.5)-(0.45 x 1 x 0.5)-(0.45 x 0.5 x 1)]x 22
= 27.23 kN
W_3 = 2.35 x 1.5 x 2.5 x 22 = 193.88 kN

Check whether the block is safe against overturning:

Take sum of moments about point B (counter clockwise moments as positive):

$\Sigma M@b$
= W_1[(0.4/2) + 0.45 + 2.35] + (W_2 + W_T)x[(0.45/2 + 2.35] + (W_G + W_3) (2.35/2) - F_H x 1.8
= 33.00(3.0) + (27.23+2.94)(2.575) 4- (3.43 + 193.88) (1.175)- 70 xl.8
or $\Sigma M@B$ = 282.5 kNm

Sum of vertical forces,
V = W_1+ W_2+ W_3+ W_T+ W_G
= 33.00 + 27.23 + 193.88 + 2.94 + 3.43
or, ΣV = 260.5 kN

Note: All dimensions are in n
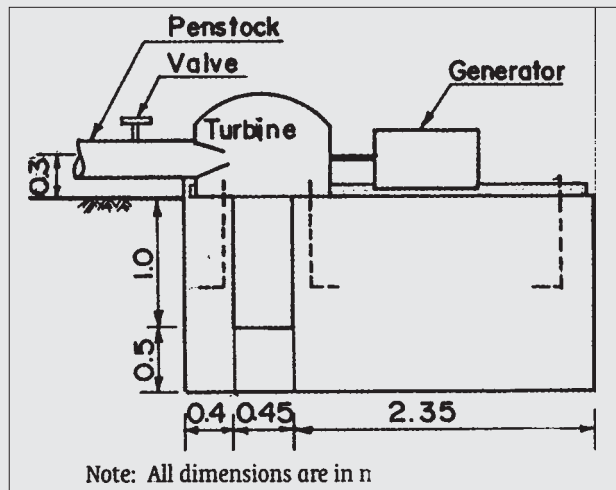
Figure 8.2 Machine foundation section

Equivalent distance at which ΣV acts from point B:

$d = \Sigma M / \Sigma V = 282.5 / 260.5 = 1.08 m$
eccentricity, $\qquad e = (L_{base} / 2) - d$
$\qquad\qquad\qquad e = (3.2 / 2) - 1.08$
$\qquad\qquad\qquad e = 0.25 m$
$\qquad e_{allowable} \qquad = L_{base} / 6 \quad = 3.2 / 6 = 0.53 m$

Since e is less than $e_{allowable}$, eccentricity is in the middle third.
$\therefore$ The structure is safe against sinking.
Check bearing pressure :

$P_{base} \qquad = \Sigma V / A_{base} [1 + 6e/L_{base}]$
$\qquad\qquad = [260.5 / (3.2 \times 2.5)][1+ (6 \times 0.52)/3.2]$
$\qquad\qquad = 64.3 \ kNm^{-2} < 180 < kNm^{-2}$ (max. allowed for soil)
$\therefore$ The structure is safe against sinking.

Check sliding:

Assume that the friction coefficient between block and soil, $\mu = 0.5$
$\Sigma H = F_H = 70 \ kN$
$\mu\Sigma V = 0.5 \times 260.5 = 130.2 \ kN$
Factor of safety against sliding :
$\qquad\qquad = \mu \ \Sigma V / \Sigma H = 1.86 > 1.5$
$\therefore$ The structure is safe against sliding.
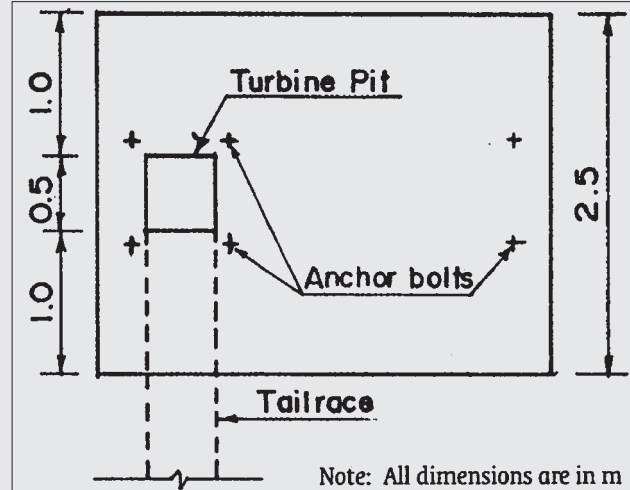
Figure 8.3 Machine foundation plan
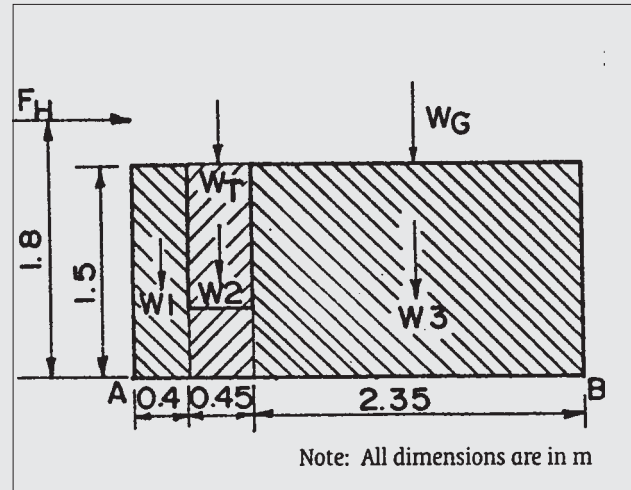
Figure 8.4 Resolution of forces on the machine foundation

Hence, the structure as designed is adequate. The final design including the reinforcement bars can be seen in Figure 8.5. A 1:1.5:3 mix concrete with reinforcement pattern as shown in the figure is recommended for the machine foundation since the structure must be rigid and strong enough to withstand the forces. A 50 mm cover (clear spacing between the bars and the edge of the concrete surface) should be provided for tine reinforcement bars. Such cover provides protection for the reinforcement bars against corrosion and other adverse effects.

Note that as can be seen in Figure 8.5, a 100 mm width of sand and gravel has been placed at the periphery of the machine foundation down to the depth of the powerhouse floor. This will structurally isolate the machine foundation from the power-

house floor so that the dynamic forces (such as vibrations) are not transferred to the floor and walls. Cracks along the powerhouse floor and walls have been observed where the machine foundations have not been structurally isolated. The 50 mm thick bituminous surface prevents the gravel and sand from being compacted (and hence the possibility of transferring forces to the powerhouse floor). This is done by pouring hot bitumen (as used in black topped roads). The 50mm thick concrete blinding provides an even surface for reinforced concrete work of the machine foundation. Also note that, if a belt drive system is required, the machine foundation should be extended to cover it. However, the depth of foundation for the belt drive can be lowered to 300 mm but with similar reinforcement pattern.
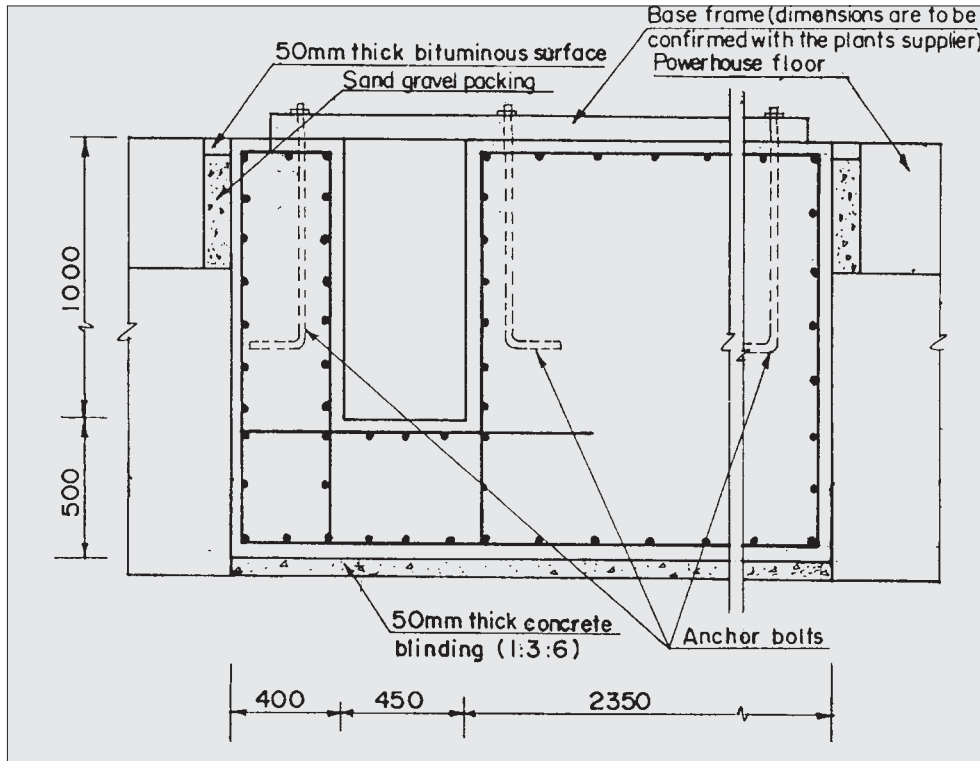
Figure 8.5 Proposed machine foundation section for Example 8.1

Notes
1. All dimensions are in mm.
2. A minimum of six 20mm diameter, 700mm long anchor bars are to be used to fix the base frame to the machine foundation.
3. 10 mm diameter Tor steel bars are to be used for reinforcement. Maximum spacing to be 150mm in turbine pit and 200mm on other faces. Lap length shall be 400 mm minimum.
4. Minimum reinforcement cover shall be 50mm.
5. Structural concrete shall be 1:1.5:3 mix.

8.5 Tailrace

8.5.1 GENERAL

The tailrace is the final civil structure that conveys the design flow. Similar to the headrace, open channel or pipes can be used for the tailrace section. Tailrace channels of the Jhankre mini-hydro and Salleri Chialsa schemes are shown in Photographs 8.5 and 8.6 respectively.

Often, inadequate attention is given to the design and construction of the tailrace since the flow at this stage does not contribute towards power production. However, such a practice can result in inadequate depth of the tailrace pit or erosion of slopes, which could threaten the powerhouse structure.

8.5.2 DESIGN OF THE TAILRACE CHANNEL

Design of the tailrace channel is similar to that of the headrace canal discussed in Chapter 4. However, since headloss does not need to be minimised a higher velocity can usually be

Photo 8.5 Tailrace channel of the Salleri Chialsa mini-hydro scheme

Photo 8.6 Tailrace channel of the Jhankre mini-hydro scheme
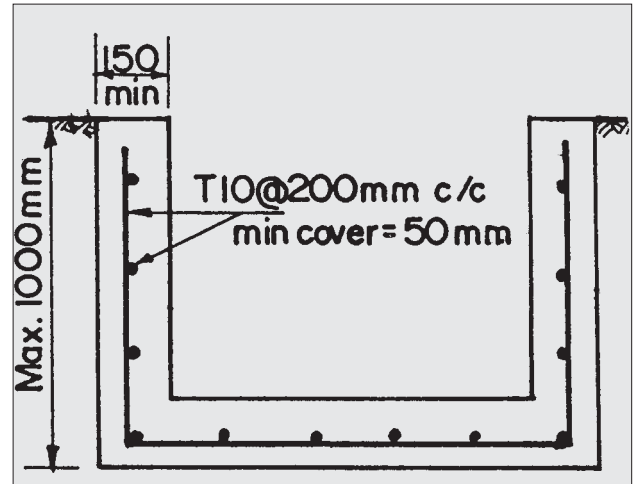
Figure 8.6 Reinforced concrete tailrace channel

Chapter 4) should be used to size the pipe. Similar to a tailrace channel, a higher headloss can be allowed for the pipe.

If HDPE pipe is used, the velocity should be limited to 3m/s and the pipe should be laid to a uniform gradient. Higher velocities or non-uniform gradient can result in air entertainment and surge problems.

If possible, the tailrace should empty onto large rocks at the riverbank so that there is no erosion at the confluence.

8.6 Checklist for powerhouse and tailrace

Is the powerhouse located above the appropriate flood level?(Refer to Section 8.2) Is the powerhouse area stable? Refer to Chapter 9 for further details on stability.

Has adequate space been allowed inside the powerhouse such that all equipment can fit in and permit access without difficulty? Has the machine foundation been sized such that it is safe against overturning, bearing and sliding? Also, be sure to structurally isolate the machine foundation.
- Is a channel or a pipe adequate for the tailrace?
- Have the velocity limits been checked?
- Is the tailwater likely to cause erosion at the riverbank?

allowed, within the limits given in Table 4.1. Note that at higher velocities a stronger grade of mortar or concrete is required to resist erosion. Reinforced concrete may be economic for a steep channel, as shown in Figure 8.6.

Note that the downstream end of the tailrace must be arranged so that there is no danger of erosion either by the river or by the flow from the tailrace. Ideally the discharge point should be onto rock or large boulders. In erodible material a stilling basin may be required to dissipate the energy from a steep tailrace channel.

8.5.3 DESIGN OF TAILRACE PIPE
If due to site conditions, a pipe is required for the tailrace, the design procedure discussed in Section 4.5 (Headrace pipe,

9. Slope stabilisation

9.1 Overview

Nepal's mountain slopes, particularly the slopes of the Middle Mountains, are undergoing rapid changes due to river cutting, weathering, and soil erosion. The rate of soil erosion is very intense in the Middle Mountains because of the subtropical climate and intense rainfall (2000 to 2500 mm per year falling within 3 to 4 months). This area is also widely cultivated using irrigated terraces and heavily deforested due to population pressures. Poor water management and forest mismanagement in this area have led to further decline of the hill slopes. The natural processes coupled with man's influence have led to landslides, and degradation of hill slopes affecting the sustainability and durability of irrigation channels, water supply systems, micro-hydro schemes and other development work.

It is not possible to completely check these natural processes, however it is possible to control them by appropriate choice of management, design and construction practices. The underlying principle behind slope stabilisation measures is to stabilise hill slopes and river banks so as to protect the micro-hydro schemes.

Most slope stabilisation problems can be effectively tackled by making sure that the hill slopes are dry (by diverting the surface water from the slopes), constructing retaining walls as well as undertaking bio-engineering measures such as planting appropriate vegetation. Note that dry slopes are more stable than saturated ones and landslides generally occur on wet slopes.

Photo 9.1 Unstable slopes are a threat to schemes. This powerhouse was destroyed by a landslide

Retaining structures such as dry stone masonry walls, gabions and terracing are the most common method used to stabilise slopes in micro-hydro schemes. In most micro-hydro schemes constructing reinforced concrete retaining walls is not feasible due to their cost. In the long term, preventive bio-engineering measures would be more effective, sustainable and cheaper than remedial works. These measures will often need the continued maintenance commitment of the community.

9.2 Implications of natural geological processes

Soil erosion, river cutting, weathering, and slope failures have implications for design, construction, operation and maintenance of micro-hydro schemes. River cutting can affect intakes in several ways, besides triggering slope failures, that may damage a portion of headrace canal, foundations of settling basin, crossings and powerhouse. For example, meandering rivers can leave intakes high and dry. Similarly, degrading rivers can render intakes useless. When headrace canals are built on hill slopes where surface erosion has advanced to a stage where gullies have already formed, there are greater risks of canal failure due to deepening and enlargement of the gullies.

The main cause of gully formation is excessive run-off due to deforestation, overgrazing and burning of the vegetation.

Excavation work can also trigger soil erosion. The following is recommended to reduce the risk of slope failure due to excavation work:
- Catch drains can be constructed above the top of an excavation, diverting surface water to a safe area.
- When excavating for canal construction, to prevent surface erosion fresh hill cuts and exposed slopes of channel banks must be quickly covered with topsoil so that vegetation can be re-established.
- Spoil from excavations should be carefully disposed of so that soil erosion is not initiated.
- Wherever possible canals should have balanced cut and fill sections to avoid too much excavation and exposure of fragile layers.
- Provide adequate berm width on the hillside of headrace canals, to stop shallow landslips blocking the flow and causing overtopping, which leads to erosion of downhill slopes.

9.3 Bio-engineering works

All engineering measures such as retaining walls and check dams should be well supplemented with bio-engineering measures as far as practicable.

Planting grass or shrubs on the freshly cut hill or the landslide area are examples of bio-engineering measures. Fast growing, deep rooted and dense cover type of vegetation that is appropriate to the local environment should be used for such purposes.

Only deep-rooted trees should be used for bio-engineering purposes, and they should not be planted so close to canals or structures that their roots could cause piping or structural damage. At least 3 metres clearance is recommended. Fast growing trees that do not have intense root systems should be avoided since they may fall due to their own weight during storms.

Once the slopes have been stabilised, care should be taken to ensure that there is no further overgrazing.

9.4 Retaining structures

Retaining walls are structures that support the backfill and surcharge load from the additional canal width or platform over the walls in hill sections. Though the per metre cost of canal construction requiring retaining walls is more than constructing the same length by cutting inside the hill, the use of retaining walls sometimes becomes essential.

The most common types of retaining wall used in micro-hydro schemes are gravity walls of gabions or cement masonry. These depend on the mass of the structure to resist overturning. Their design depends on the wall density, soil parameters, drainage and loading conditions, typically resulting in a base width of 0.40 to 0.65 times the height. The designs shown in Figures 9.1 and 9.2 are therefore safe, but conservative in many conditions.

For high or long walls it will be economical to design for the specific site conditions. Site specific designs should also be made where the backfill is inclined rather than horizontal.

The walls should be checked for overturning, sliding and bearing pressure, as described for anchor blocks in Chapter 7. Alternatively refer to standard civil engineering texts such as Ref. 7. (The unit weight of filled gabions is 14-18 kN/m^3, depending on the unit weight of the rock fill and assuming 30-35% voids).

Wall foundations must be deep enough to be safe against erosion: normally at least 0.5 m below ground level, but see Section 3.8 for river works. Lined toe drains may be

*Photo 9.2 M*asonry steps for energy dissipation and control of spillway water. Although costly, elaborate control is essential where slopes are vulnerable t*o erosion (Siklis)*

used in erodible areas to carry seepage water safely away from the wall foundation.

Gabion retaining walls should be constructed with an inclination of 10%, see Figure 9.1. Where gabions are to be built on sand or fine soils, a layer of filter cloth should be placed between the foundation and the gabions. The gabion boxes should be laced together along all edges and stretched before filling with rock. The rock should be packed with the minimum of voids.

Stone masonry walls can be constructed in 1:4 cement/ sand mortar as shown in Figure 9.2. Such walls are suitable for retained heights of up to 2 to 3 m. The slope of the front face may be steepened if necessary, provided that the base width is maintained. The rear face of the wall should be left rough to increase friction with the backfill. Weepholes must be provided to relieve water pressure behind the wall, and their mouths should be protected with carefully placed stones. Backfill behind the wall should be freedraining gravel or stones; if the retained soil is fine, a filter cloth should be placed as shown to prevent the soil particles blocking the drainage.

Photo 9.3 Gabion at the toe of an unstable slope

9.5 Terracing and dry stone wall

Lack of effort in looking for alternate wall types, such as terracing and dry stone walls often rules out the development of other techniques that are more economical and durable. Minor landslide areas can be stabilised by constructing dry stone terraces as can be seen in Figure 9.3. The overall slope of such terraces should be limited to 30° (i.e. terrace width should be twice its height). 500 mm thick dry stone walls should be used for the vertical face of the terraces. Such dry stone walls retain the soil behind and allow the surface water to drain out. Constructing small catch drains on the terraces helps to reduce soil erosion by draining the surface water. An alternative method used to stabilise the steeper Jhankre mini-hydro powerhouse slope is discussed in Box 9.1.

Photo 9.4 Stone masonry can provide slope stabilisation along the route of the penstock (Barpak)

h , m	B, m
1.0	1.0
2.0	1.5
3.0	2.0
4.0	2.5
5.0	3.5
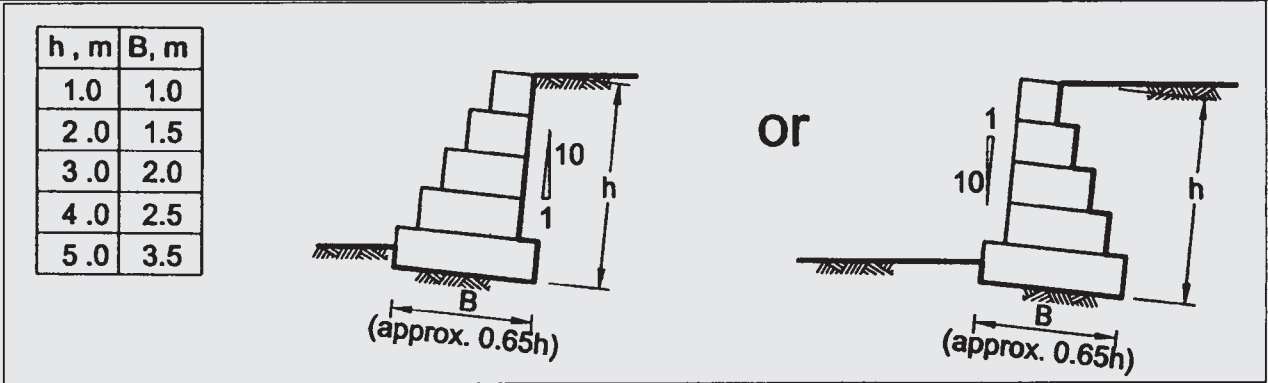
Figure 9.1 Gabion retaining wall
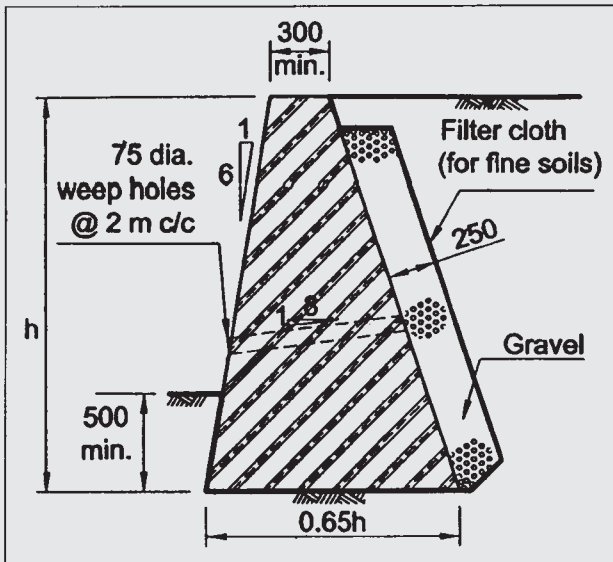
Figure 9.2 Stone masonry retaining wall

1.   Catch drains often lined with impermeable lining materials (i.e. stone masonry) to avoid infiltration. Water collected from catch drains needs to be drained to nearest natural drain.
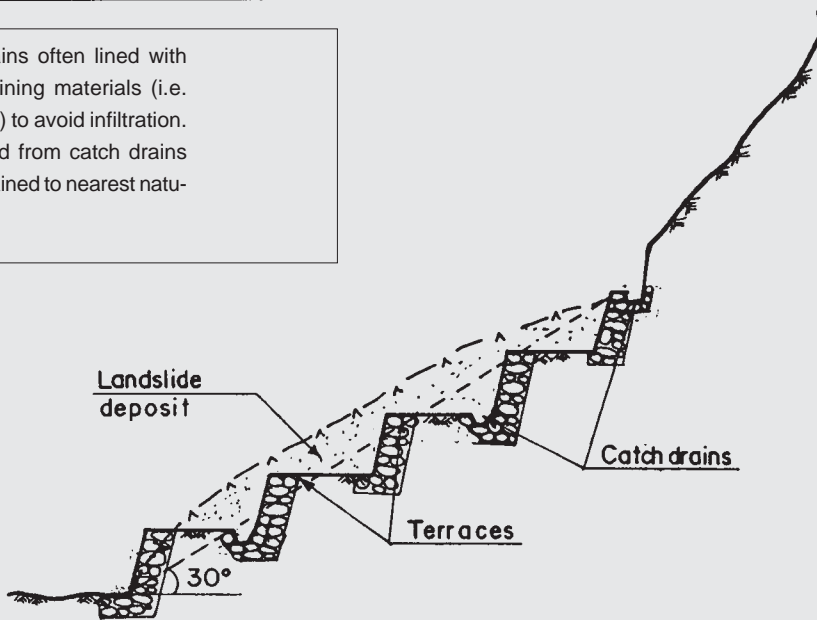
Figure 9.3 Terracing and dry stone walls to retain slopes

Box 9.1   Use of masonry grid to stabilise the jhankre mini-hydro powerhouse area slope

In order to increase the gross head of the Jhankre mini-hydro scheme, it was decided to excavate a 20 m depth at the power-house area. This required stabilising the hill slope behind the powerhouse area. This area also had to drain ground water due to seepage from the cultivated terraces (paddy fields) above. When irrigation water was provided for the paddy fields significant seepage was observed at the powerhouse area. After considering various alternatives, it was decided to use a grid of masonry beams and columns infilled with dry stone panels. Photographs 9.5 and 9.6 show the hillside during excavation and after the construction of the masonry grid.

The hill slopes were first excavated at 2:1 to 3:1 slopes (V:H) with two intermediate berms along the hillslope and one at the sides. Then the grid of stone masonry (in 1:4 cement: sand mortar) beams and columns with dry stone masonry infill panels was constructed along the excavated slopes. The beams and columns are 500 mm wide and 300 mm deep. The distance between the columns is 2 m and the vertical distance between the beams is 1.5 m (maximum). The dry stone infill saved the cost of cement and facilitates drainage. Catch drains have been provided at the berm levels.

To date this 16 - 20 m high structure is stable. During the monsoon ground water that has seeped from the paddy fields above can be seen draining out from the weep holes and the dry stone panels.

Photo 9.5 Excavated slope at powerhouse
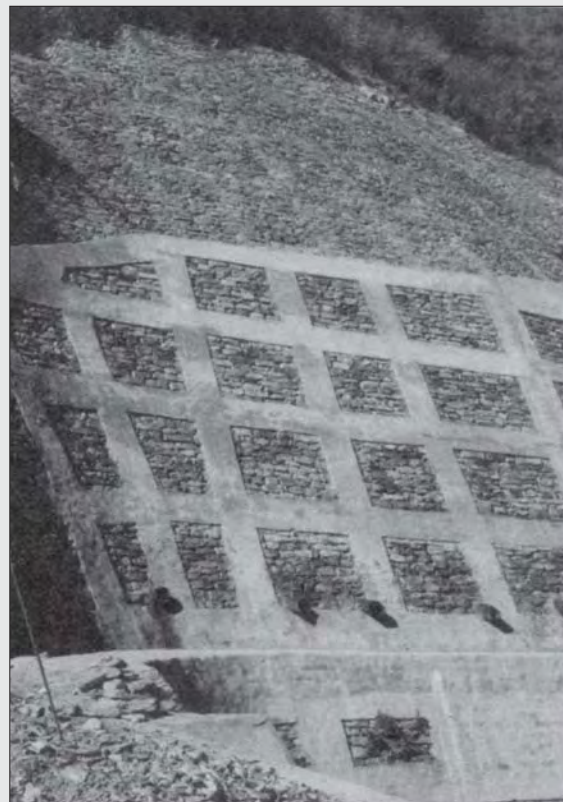
Photo 9.6 Masonry grid at powerhouse slope

9.6 Check dams and gully control

Gullies that are active or small streams where scouring of the riverbed is prominent can be controlled by constructing check dams. Check dams are small walls that prevent further erosion on the watercourse and also allow deposition of bed load upstream of it at a stable gradient. For small gullies that are only active during the monsoon, the check dam could consist of a simple dry stone wall. For small streams, bed erosion can be controlled by using gabion check dams.

Gabion check dams have been used to control riverbed scouring at the Jharkot micro-hydro scheme. The riverbed at the Jharkot intake area had been scoured by more than 3 m at some places and the scour depth was getting deeper. It was felt that further scouring along the riverbed would cause total failure of the existing gabion wall along the left bank of the river. A series of gabion check dams was constructed at the intake area to prevent further scouring and to facilitate the deposition of bed load. The first check dam is shown in Fig-ure 9.4. Note that to prevent the gabion wires from being broken
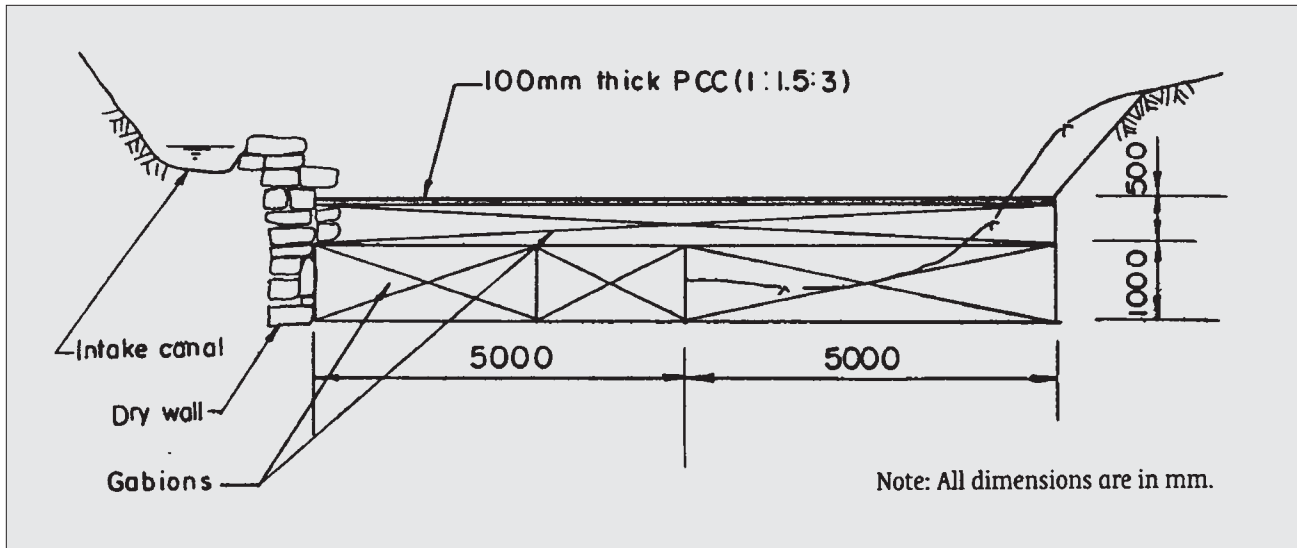
Figure 9.4 A gabion check dam at the intake of Jharkot micro-hydro scheme

by rolling boulders, 100 mm thick plain concrete was provided at the top surface of the gabion wall. A minimum foundation depth of 1 m from the lowest streambed level has been provided for all check dams.

These check dams were constructed by the end of June 1998. They have survived the first monsoon floods and currently their conditions are being monitored.

9.7 Maintenance

Retaining walls, check dams and other slope stabilisation structures should be inspected regularly; specifically before and after every monsoon. Remedial works should be done as soon as any problems are noticed. For example, the gabion crates, if broken, should be repaired soon. Similarly, if stones are missing from the dry stone masonry retaining walls or check dams, they should be replaced. The drainage system of the stabilised slope should be well maintained. Deposition of boulders, gravel or soil in the drain should be removed. If the plaster or masonry is broken, it should be repaired. Note that stability problems can occur in even a well stabilised slope if the drainage system stops functioning. It is important to note that some structures such as gabion walls are not meant to be permanent on their own. They may deteriorate and collapse. However, once appropriate vegetation over these structures has taken root and has matured repair of these structures may not be necessary, since the roots of the vegetation will stabilise the soil mass. In the case of bio-engineering measures, any plants that are missing should be replaced. If possible, newly planted areas should be fenced to prevent grazing of animals.

10. Innovations

10.1 General

A number of innovative ideas, research, applications and pilot projects relevant to micro-hydro technology that have not yet been fully field tested, especially in the Nepalese context, are discussed in this chapter. Some applications are in the research and development stage, others have been successfully implemented in other countries or carried out as "pilot projects" in Nepal.

10.2 Coanda intake

A Coanda intake has a special screen that utilises the tendency of fluids to follow a surface. This is known as the "Coanda effect". As shown in Figure 10.1, the Coanda screen is installed along the crest of the diversion weir and is shaped in the ogee curve configuration. A curve acceleration plate at the top of the screen stabilises and accelerates the flow. As the flow passes over the screen surface, the shearing action of the bars combined with the Coanda effect separates the flow. Clean water passes down through the screen whereas sediment and debris pass over the screen to rejoin the water course below the weir.

On rivers carrying cobbles and boulders during flood, the Coanda intake must be carefully located so that heavy bedload does not pass over the screen and damage it.

The potential advantages of the Coanda intake are on particular sites which suffer from exposure to high silt load or which offer scope for cost savings in the headrace. In the first case, the intake can reduce the need for large or multiple settling basins. In the second case, where a site layout is suitable, it may be possible to commence the penstock run directly from the Coanda, gaining head and avoiding the need for a headrace canal. Of course, this might imply that the penstock is longer than other potential layouts or that it runs close to the river. A thorough financial and technical analysis of the options is required before making a decision on the suitability of the Coanda for a particular site.

The Coanda screens are fabricated to a high tolerance from stainless steel. The supplier of the Coanda screens (also called "Aqua Shear Screens") in Europe, Dulas Limited, Wales UK, claims that screens can be produced with 0.5 mm to even 0.2 mm clear spacings, which eliminate 90% of 0.25 mm and 0.1 mm particles respectively. Both types of screen eliminate all 1 mm particles. In most micro-hydro systems this would also eliminate the need for a settling basin.

The flow capacity of these screens is 140 1/s per metre of weir length. A screen with a flow capacity of 40 1/s (0.3 m width) costs about US$ 1380 (1997 price).

A Coanda intake has been tested at a micro-hydro site in Wales by Dulas Ltd., in conjunction with Practical Action, UK.
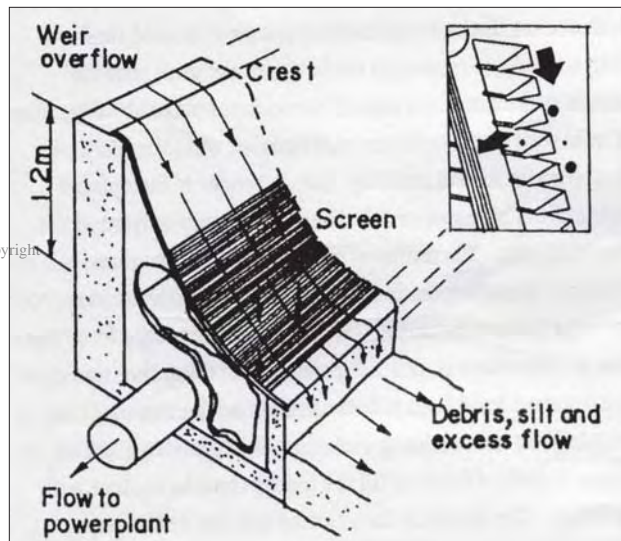
Figure 10.1 A Coanda intake screen
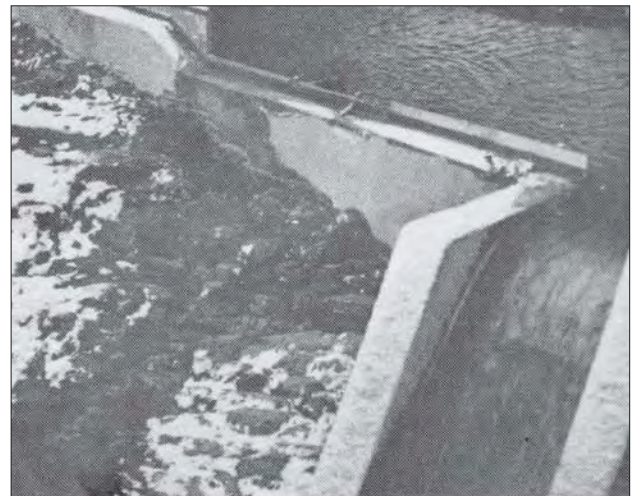
Photo 10.1 Coanda intake of a micro-hydro scheme in Wales, UK

The design flow of this micro-hydro scheme is 401/s and the intake is shown in Photograph 10.1.

Over a six month period, the performance of the screen has shown the following characteristics:
- Around 90% of sediment between 0.5 mm and 1 mm diameter was excluded.
- Some build up of algae was noted, but this did not inhibit flow during the trial period.
- The effect on performance due to ice was not noticeable, even at temperatures 12°C below freezing point.

Tests are required over a longer period to check for corrosion of the screens and effect of continued algae growth, particularly in warmer temperatures.
More information on Coanda screens can be obtained from:
DULAS Limited,
Machynlleth, Powys SY20 8SX, Wales, UK
Fax: +44(0)1654781390
e-mail: dulas@gn.apc.org

A pilot project in Nepal, a Coanda screen has been retrofitted in an existing micro hydropower plant with joint efforts by Small Hydropower Promotion Project (SHPP/GTZ), Energy Systems and Rural Energy Development Program (REDP) of United Nation Development Program (see Photograph 10.3).

Dulas Engineering provided the Coanda Screen at free of cost for this pilot project.

A 16 kW Cha Khola micro-hydropower plant at Singe VDC, Kavrepalanchowk district, Nepal was identified to be suitable for retrofitting the Coanda screen. It was agreed that the ideal location to retrofit a Coanda screen would be at the forebay of this micro-hydropower plant. Major specifications of the Cha

Photo10.3 De-beader tool

Khola micro hydropower plant are as follows:
Design Flow: 50 l/s
Gross Head: 55 m
Installed Capacity: 16 kW
Turbine: 3-jets Pelton
Beneficiaries: 125 Households

Screen commissioning
During commissioning in January 2004, the screen operation was observed until a steady condition was reached. Once, steady state was reached, the turbine nozzles were closed to observe the overflow/spilling mechanism from the screen. Some leaves, stems and twigs were introduced into the flow to observe the self-cleaning mechanism of the screen. Also, a handful of sand particles were placed at the weir crest of the screen during the steady state flow condition to visually observe the sediment exclusion effects.

The findings during the commissioning phase were as follows:

i.   When leaves, stems and twigs were introduced with the flow, they initially blocked part of the screen openings. However, the entire diverted discharge was able to pass through the screen even when the flow was partially obstructed. When the turbine valves were closed, the water flowing through the screen pushed the leaves out (of the screen), demonstrating the self-cleaning mechanism.
ii. Although it was not possible to visually quantify the sediment volume that passed over the screen when such sediments were introduced at the weir section, the larger once could be seen rolling down the screen.
iii. The discharge entering into the screen passed through the first 250 mm to 300 mm length (the screen length is about 645 mm).

Photo 10.2 Weld beads on HDPE surface

iv. During steady state when the turbine was under operation, the water depth in the chamber remained below the toe of the screen. Thus, the submergence depth set initially seemed to be adequate.

v. The water level inside the forebay chamber decreased by 400 mm compared to the previous case, thus decreasing the gross head by about 0.7 percent.

Although, the Cha Khola plant has faced intermittent problems with the generating equipment, to date the screen is performing satisfactorily.

10.3 De-beader for HDPE pipes

As discussed in Chapter 4 (Box 4.7), HDPE pipes are joined by heat welding which melts and fuses the ends together. This leads to raised "beads" on the inside and outside of the pipe as shown in Photograph 10.2. The external bead is not a problem but the internal bead promotes blockages and significant head loss. An effective debeader would reduce the roughness value for HDPE from 0.06 mm (Table 4.3) to 0.03 mm.

A "debeader" tool has been designed (for IT Nepal) to remove the internal beads from HDPE pipes while the joints are still hot (i.e., hot debeading). This equipment has been designed to remove beads for pipe diameters up to 250 mm. It is still in the experimental phase and requires some development for use in the field.

As shown in Photograph 10.3, the debeader consists of a mild steel shaft with a sleeve on which a bush spring loaded to a locking collar (via an Alien key) is placed. There are three hardened steel blades (cutter arms) that are pin connected by rectangular steel bars to the bush. By unlocking the bush the connecting pin can slid up the sleeve to increase the cutting diameter.
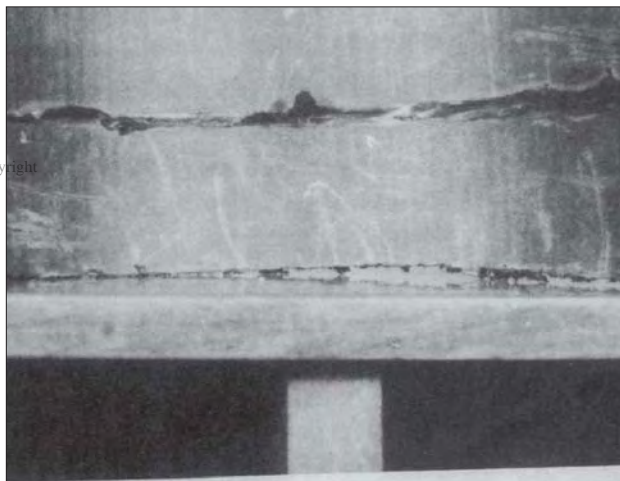
Debeading with this tool is done by first unlocking the bush so that the tool can fit inside the pipe. A mild steel rod with a handle is connected to the debeader such that the handle sticks out of the pipe. The rod is supported by a number of mild steel discs inside the pipe (smaller than the HDPE pipe diameter) for lateral stability. The de-beader is then placed inside of the pipe such that it is about 100 mm in front of the proposed joint. The radius of the cutting arms are then arranged by sliding the bush such that the blades are in contact with the inside pipe surface. Once the blades snugly fit on the pipe surface, there is a clicking sound indicating that the cutting arms have been locked. As soon as the two pipe ends are joined by heat welding as discussed in Chapter 4, the debeader is pushed forward till the blades come in contact with the beads. The handle is then turned and the debeader is pushed forward which removes the bead.

This de-beader was tested at Nepal Yantra Shala Energy, Kathmandu, on a 200 mm pipe as shown in Photograph 10.4. Debeading was tried on a joint about 2 m from one end of the pipe. The test was partially successful. It was not possible to remove the entire strip of the bead. Part of the bead and some thin strands were left on the joint, as can be seen in Photograph 10.4.

The following observations were made in the workshop:
- The major constraint was that the radius of the cutter blades is fixed and the blades do not work equally well on all pipe diameters within the range.
- If debeading is not started immediately after the pipes are joined (i.e. within 30 seconds), the beads cannot be removed.
- The turning of the handle and pushing of the de-beader has to be controlled. A sudden jerk pushes the de-beader beyond the joint.
- The debeading process is also hampered if the pipe ends are not totally circular.

Photo10.4 HDPE pipe joint de-beaded using the de-beader

Photo10.5 HDPE pipe joint de-beaded using a commercial de-beader. The small ring in front of the pipe section is the bead.

Hence, design improvements are required before this debeader and can be used in the field.

Commercial debeaders are also available but they are expensive. Some such commercial debeaders can also remove beads after the joints have cooled (i.e. cold debeading). Photograph 10.5 shows a section of an HDPE pipe which was cold de-beaded using a commercial debeader. Further information on commercial debeaders can be obtained from:
Fusion Group PLC
Chesterfield Trading Estate,
Chesterfield S419PZ,
England,
UK Fax: +44(0)1246450472

10.4 Bursting disc

As discussed in Chapter 6, penstock pipes for micro-hydro schemes are designed to accommodate the surge head when setting the pipe thickness. An increase in the pipe thickness also increases the cost of the pipe. Furthermore, depending on the location of the site, the transportation cost also in-creases. In a high head scheme with a long penstock alignment, the increase in cost to accommodate the surge head can be significant.

There are many ways to guard against surge damage but most involve significant cost (where, for safety reasons the flow has to be constrained) or involve great care in installation and maintenance.

The "bursting disc" technology may provide a reliable means of safely releasing the excess head in case of surge pressure. A "bursting disc" is a commercially available over-pressure safety device made from a brittle material such as graphite or an appropriate metal, or a suitable metal which is *designed to rupture extremely quickly once a* critical pressure is exceeded, such as the surge head induced inside the penstock pipe in the event of a jet blockage. Such discs are commercially used in the chemical industry to protect pipelines and pressure vessels (that convey gas and petroleum fluids) from high surge pressure. Pipes that have bursting discs do not need to be designed to accommodate surge pressures. Photograph 10.6 shows a commercially available bursting disc (including the burst plate). Note that scratch lines are made in the plate during manufacture to introduce weaknesses in the plate such that it bursts according to the pattern shown in Photo-graph 10.6 once the prescribed pressure is reached.

Most graphite discs are flat, deform very slightly under a pressure differential and because of their physical properties, at the set pressure shear instantaneously around the periphery of the disc active area giving immediately full bore venting.

Such discs are suitable for venting of both liquids and gases. The disc is manufactured to burst within its tolerance only when installed in a suitably designed and manufactured holder. Some of the main advantages of graphite bursting discs are that they are not adversely affected by misaligned pipe work or overtorquing of pipe flange bolts. Due to the sufficiently high burst pressure, this type of disc does not require backpressure support to withstand full vacuum pressure. The discs have an operating ratio of 90% and are guaranteed to rupture within a maximum of 30 milliseconds. The discs are also inexpensive and in case of rupture due to surge pressure, all that is required to recommission the pipe
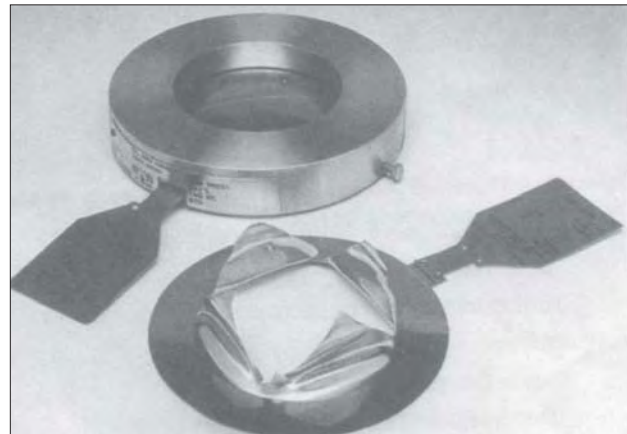
Photo 10.6 A commercially available bursting disc

is to replace the graphite plate. Hence, this technology could be highly suitable for micro-hydro schemes including those located in remote areas. Theoretical research on the applicability of bursting discs for micro-hydro schemes has been undertaken by Dulas in conjunction with Warwick University. The bursting disc arrangement proposed by the study is shown below in Figure 10.2.

Note that in case of the rupture of the disc, the flow would discharge inside the turbine casing. Such an arrangement is well suited in micro-hydro schemes since a separate flow control structure is not required.

The conclusions of the above study were as follows:
- The disc could reduce surge pressure by 60% to 70% if the subsequent flow rate through the branching arrangement is only moderately reduced. The reason why the entire surge head cannot be eliminated is because the diameter disc is usually smaller than the penstock diameter and hence the flow is reduced.
- The penstock safety factor could be reduced from 3.5 to 2.5.

These theoretical findings need to be thoroughly verified by actually installing the discs in existing micro-hydro schemes and monitoring the results. Practical Action in Nepal has plans to field test the bursting discs in some existing micro-hydro schemes.
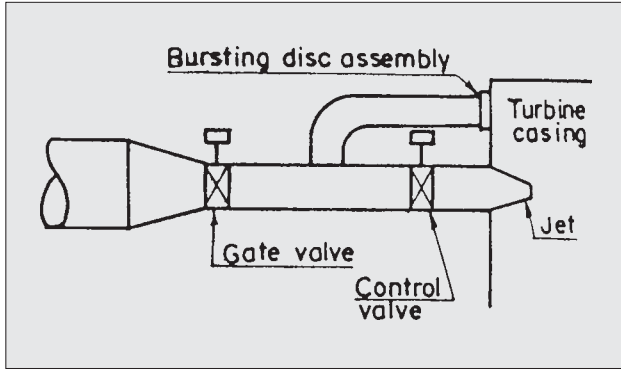
Figure 10.2 Proposed arrangement for bursting disc installation in micro-hydro schemes

More information on the applicability of bursting discs for micro-hydro schemes can be obtained from Dulas Limited (same address as above).

Information on commercial bursting discs can also be obtained from the following manufacturer:
IMI Marston Limited
Wobaston Road, Fordhouses,
Wolverhampton WV106QJ
England, UK
Fax: +44 (0) 1902 397792

10.5 Flexible steel support pier for Jharkot micro-hydro

The 36 kW Jharkot micro-hydro scheme is located in Mustang District, Nepal. This is a community owned scheme and is managed by the Jharkot Electrification Committee. Practical Action in Nepal has been involved in providing technical support for refurbishment work of this scheme for some time.

Similar to other areas of Mustang, the topography of the project area consists of fragile and unstable slopes and is prone to landslides. The intake and the initial headrace canal have been damaged frequently by landslides and floods. Although the slope along the penstock alignment is relatively stable compared to the intake area, it is weak and also prone to landslides. The existing masonry support piers started sinking due to their own weight as well as the weight of the penstock pipe and the water inside it. Hence, the penstock (flange connected) started to sag at various places. As part of the preparation of this text, a pilot project was carried out to design and install steel support piers for the Jharkot scheme with assistance from Mr. Shyam Raj Pradhan of NYSE. The design criteria were as follows:

- The support pier had to be light, to minimise self weight.
- It had to be fabricated using parts that could be carried by porters or mules. Jharkot is about half a day's walk from the district airport and 5 days' walk from the nearest roadhead.

- The design had to allow for the sinking of the foundation. In case of sinking of the ground below the foundations, the piers should not pull the penstock pipe down along with it.

The design of the support pier and the foundation are shown in Figures 10.3 and 10.4 respectively. The total weight of a 2 m support pier is 60 kg (excluding the foundation work) whereas a masonry pier of similar height would weigh 4000 kg. Note that such support piers should be installed perpendicular to the penstock alignment (not vertically) since they are only resisting force Fj (see Chapter 7). The top section of the pier consists of a channel which is pin connected to two legs that have turnbuckles. The penstock pipe rests on the channel and the pin connection allows the channel some rotation such that it is perpendicular to the penstock alignment. Two holes have been provided on the channel to clamp the penstock with a 12 mm diameter bar. The turnbuckles can be adjusted to fine tune the height of the support piers (up to 300 mm) during installation and in case the foundation sinks in the future. The bottom of the turnbuckles (40 mm rods) fit inside a hollow pipe as shown in Figure 10.3. In case the ground beneath the foundation sinks, the support pier structure below the turnbuckles drops down along with the foundation and only the top part (up to the turnbuckle legs) hangs with the penstock. Hence the penstock pipe is not dragged down with the pier in case of sinking. The bottom part of the pier consists of angles which are bolted back to back (Figure 10.3). Bolt holes at a distance of 150 mm are provided for coarse adjustment of the pier. The bottom angles are pin connected to the foundation so that the moments due to thermal expansion of the penstock pipe are not taken by the support pier or the foundation.

During installation as well as later in case the foundation sinks, coarse adjustment can be made using the bolt holes of the bottom angles and then fine tuned using the turnbuckles at the top. The top and bottom parts of the pier have fixed heights. The length of the middle portion (angles bolted to channels) is varied such that the total pier height is equal to the required height. Note that this support pier can be dismantled such that there are 12 individual pieces (including the 12 mm stirrup bar to connect to the penstock). 18 support piers ranging from 1.0 m to 2.6 m height have been fabricated based on this design. To ensure that the support pier would function well, one (2.5 m total height) was tested at the manufacturer's workshop (NYSE) as shown in Photographs 10.7 and 10.8. About 500 kg of axial load (maximum compressive load expected on the pier) was applied on the pier. There was no observable effect on the pier (deformation or deflection of angles) during the test of about 2 hours. It was even possible to raise the height of the pier by rotating the turnbuckles with the full test load of 500 kg.

As of July 1998, all 18 support piers have been installed at the Jharkot scheme. Their performance is currently being monitored.
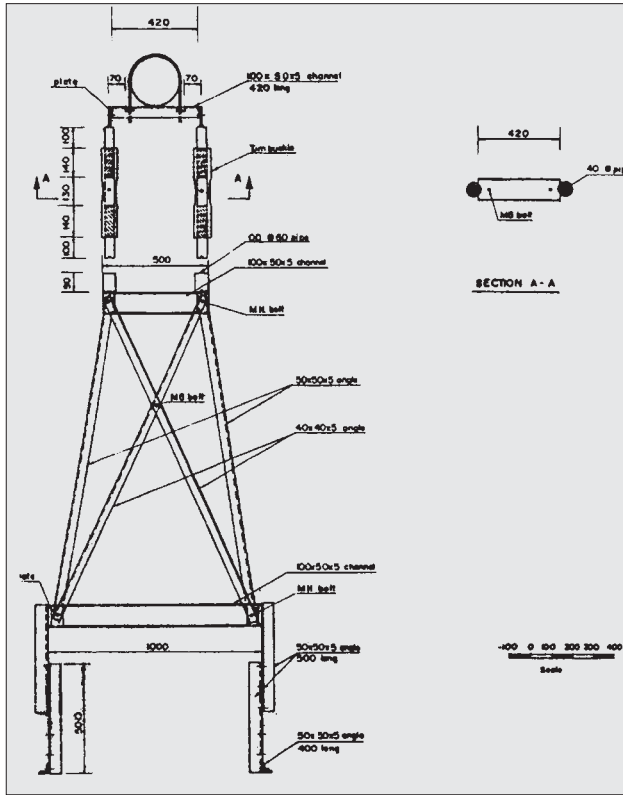
Figure 10.3

10.6 PVC pipes

PVC pipes are frequently used by Intermediate Technology for penstocks in its micro-hydro programme in the northern Andes. One of the first schemes to benefit was Chalan. This project has a capacity of 25 kW, and a head of 96 metres. The penstock diameter is 200 mm. Connection of pipe lengths was through glued spigot and socket joints. On commissioning, it was found that pinhole leaks appeared in the joints, a problem that was addressed through the application of additional resin. The whole length of the penstock was buried for protection from sunlight, animals and other potential sources of damage.

10.7 Anchor block design

The stability calculations for anchor block design are time consuming. A spreadsheet program for the stability analyses has been written by John Bywater to speed up the process. To date there is no manual to accompany the program, nor evidence that it has been verified. However, the program was used to verify the guidelines given in Section 7.4.4. for sizing anchor blocks for small schemes.

Photo 10.7 Load test of steel support pier (side elevation)

Photo 10.8 Load test of steel support pier (front elevation)
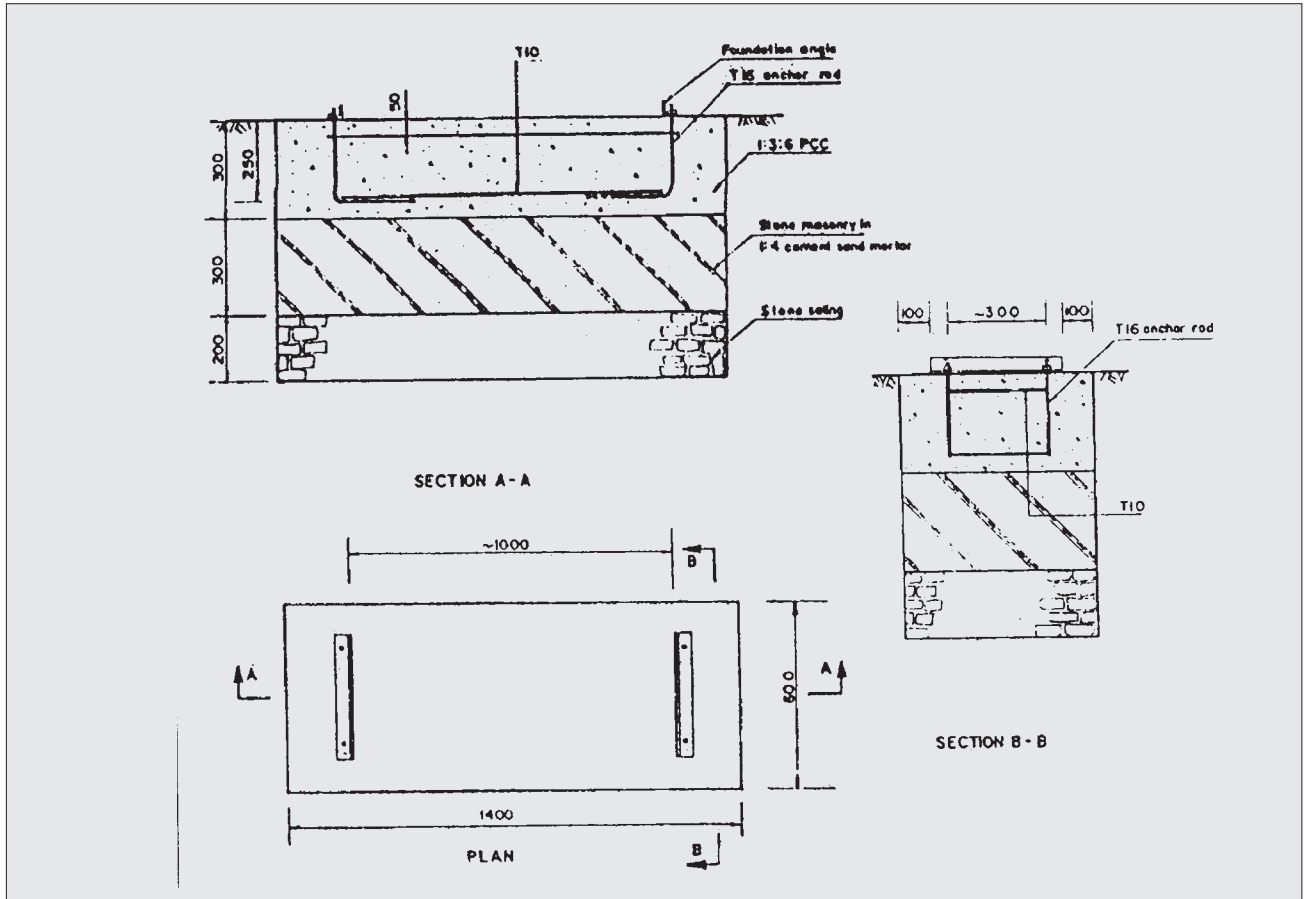
Figure 10.4 Foundation for the Jhankre flexible support pier

11. References

1 Adam Harvey et.al. (1993), Micro-Hydro Design Manual, A guide to small-scale water power schemes. Practical Action Publishing, ISBN 1 85339103 4.

2 Alien R. Inversin (1986), Micro-Hydropower Sourcebook, A Practical Guide to Design and Implementation in Developing Countries, NRECA International Foundation, 1800 Massachusetts Avenue N. W., Washington, DC 20036.

3 Helmut Lauterjung/Gangolf Schmidt (1989), Planning of Intake Structures, GATE/GTZ, Vieweg.

4 Methodologies for estimating hydrologic characteristics of ungauged locations in Nepal (1990), Nepal Government, Ministry of Water Resources, Water and Energy Commission Secretariat, Department of Hydrology and Meteorology.

1 Design Manuals for Irrigation Projects in Nepal (1990), Planning and Design Strengthening Project (PDSP), Nepal Government, Ministry of Water Resources, Department of Irrigation. United Nations Development Programme (NEP/85/013)/World Bank.

2   Salleri Chialsa Small Hydel Project (1983), Technical Report, DEH/SATA. ITECO.

3 P.N. Khanna (1996), Indian Practical Civil Engineer's Handbook, 15th Edition, Engineer's Publishers, Post Box 725, New Delhi-110001.

Appendix A - Flow estimation

A.1   WECS/Department of Hydrology and Meteorology (DHM) method

A.1.1   PROCEDURE FOR ESTIMATING INSTANTANEOUS FLOOD PEAK

1. From available topographic maps, find out the catchment area (km²) below 3000 m elevation.
2. In the following equation, input coefficients from Table Al.

Q_0 $=$ α(Area below 3000m + 1) m³/s where subscript a
        is either 2 year or 100 year return period.

TABLE A1 PREDICTION COEFFICIENTS FOR INSTANTANEOUS FLOOD FLOWS

RETURN (YEARS)	PERIOD CONSTANT COEFFICIENT (α)	POWER (β)
2	1.8767	0.8783
100	14.630	0.7342

3. Flood peak discharge, Q_F, for any other return period can be calculated using:

$Q_F = e^{(lnQ2 + s.\delta_{lnQF})} = e^{(lnQ_2 + s.\delta_{lnQF})}$

where S is the standard normal variate for the chosen return period, from Table A2, and

δ_{lnQF} $=$ $ln(Q_{100} / Q_2) / 2.326$

TABLE A2 Values of standard normal variate for various return periods

RETURN PERIOD (T) (YRS)	STANDARD NORMAL VARIATE (S)
2	0
5	0.842
10	1.282
20	1.645
50	2.054
100	2.326
200	2.576
500	2.878
1000	3.090
5000	3.540
10000	3.719

A.1.2   PROCEDURE FOR ESTIMATING FLOW DURATION CURVE

1. From available topographic maps, find out the catchment area below 5000 m elevation.

2. Use the following equations to calculate the flows. The values of monsoon wetness index can be read from Figure A3. $Q_\%$ is the dischagre(m³/s) for the specific probability of exceedence.

In $Q_{0\%} =$ -3.5346 + 0.9398. In (Area below 5000 m + 1)
       + 0.3739. In (Monsoon wetness index)

In $Q_{5\%} =$ -3.4978 + 0.9814. In (Area below 5000 m + 1)
       + 0.2670 In (Monsoon wetness index)

In $Q_{20\%} =$ -5.4357 + 0.9824. In (Area of basin) + 0.4408.
       In (Monsoon wetness index)

In $Q_{60\%} =$ -6.4846 + 1.0004 In (Area of basin) + 0.3016.
       In (Monsoon wetness index)

In $Q_{80\%} =$ -4.8508 + 1.0375. In (Area of basin) + 0.3016.
       In (Area below 5000 m + 1)

In $Q_{95\%} =$ -5.4716 + 1.0776. In (Area below 5000 m + 1)

$\sqrt{Q_{100\%}} =$ -0.09892 + 0.08149$\sqrt{}$ (Area below 5000 m + 1)

A.13   PROCEDURE FOR ESTIMATING LONG TERM AVERAGE MONTHLY FLOWS

1. From available topographic maps, find out the catchment area below 5000 m elevation.
2. In the following equation, input coefficients from Table A3 and the values of monsoon wetness index from Figure A3.

$Q_{mean\ mouth} =$ C.(Area of basin)^{A1}. (Area below 5000$_m$ +
       1)$_{A2}$ (Monsoon wetness index)$_{A3}$

where subscript month denotes one of the months from January to December. A power of 0 indicates that particular parameter does not enter into the equation for that month.

A.2 Medium Irrigation Project method (MIP)

Procedure for estimating mean monthly flows of a selected catchment.

1 In the low flow period from November to April, visit the catchment in question and make one flow measurement. Ensure that there has been no heavy rainfall during the preceding few days and that the water level is not fluctuating rapidly.

TABLE A3 Prediction coefficients for long term average monthly flows

MONTH	CONSTANT COEFFICIENT C	POWER, AREA OF BASIN (KM²) A_1	POWER, AREA OF BASIN BELOW 5000 M+1 (KM²) A_2	POWER OF MONSOON WETNESS INDEX $A3$
January	0.01423	0	0.9777	0
February	0.01219	0	0.9766	0
March	0.009988	0	0.9948	0
April	0.007974	0	1.0435	0
May	0.008434	0	1.0898	0
June	0.006943	0.9968	0	0.2610
July	0.02123	0	1.0093	0.2523
August	0.02548	0	0.9963	0.2620
September	0.01677	0	0.9894	0.2878
October	0.009724	0	0.9880	0.2508
November	0.001760	0.9605	0	0.3910
December	0.001485	0.9536	0	0.3607

Note : unit of flow are m³/s

2 Ascertain if there are significant upstream abstractions, attempt to quantify them and add this amount to the measured flow.

3 Establish in which hydrological region the catchment lies, from Figure A2. Divide the measured flow by the non-dimensional hydrograph ordinate (Table A4) for the appropriate month and region. If the flow measurement was conducted at the beginning or the end of the month, it may be necessary to interpolate between the two relevant ordinates from Table A4. The result represents the mean April flow to be expected in that catchment.

4 Take the April flow calculated in step 3 and multiply it by each non-dimensional ordinate from Table A4. The result is the hydrograph of mean monthly flows.

5 It is useful to compare the hydrograph calculated in step 4 with the appropriate regional hydrograph depicted among Figures A4 to A10. To do this, divide each ordinate of the catchment hydrograph by the catchment area. Normally, the calculated hydrograph will correspond to the regional hydrograph within the limits indicated. The limits may be used as a rough guide to the reliability of flow in the catchment. If the hydrograph lies outside the limits then it is not typical, due perhaps to unusual land use or a typical detail of topography and geology.

TABLE A4 **Non-dimensional regional hydrographs**

MONTH	REGION						
	1	2	3	4	5	6	7
January	2.40	2.24	2.71	2.59	2.42	2.03	3.30
February	1.80	1.70	1.88	1.88	1.82	1.62	2.20
March	1.30	1.33	1.38	1.38	1.36	1.27	1.40
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	2.30	1.21	1.88	2.19	0.91	2.57	3.50
June	6.00	7.27	3.13	3.75	2.73	6.08	6.00
July	14.50	18.18	13.54	6.89	11.21	24.32	14.00
August	25.00	27.27	25.00	27.27	13.94	33.78	35.00
September	16.50	20.91	20.83	20.91	10.00	27.03	24.00
October	8.00	9.09	10.42	6.89	6.52	6.08	12.00
November	4.10	3.94	5.00	5.00	4.55	3.38	7.50
December	3.10	3.03	3.75	3.44	3.33	2.57	5.00

A3   Design example

Catchment name	Solu Khola
Catchment location	Solukhumbu
Hydrological region	3
Basin area	330 km²
Area below 5000 m	308.5 km²
Area below 3000 m	97.7 km²
Monsoon wetness index	1500
Month of gauging	April
Flow measured	2.8 m3/s

Figure A1 shows the catchment map.

A 3.1   WECS/DHM PROCEDURE

a) Flood flows

RETURN PERIOD (YRS)	INSTANTANEOUS FLOOD DISCHARGE (M³/S)
2	106
5	175
10	228
20	283
50	362
100	426

example:

$$Q_2 = 1.8767 (97.7+1)^{0.8783} = 106 m^3/s$$

$$Q_{100} = 14.63 (97.7+1)^{0.7342} = 426 m^3/s$$

$$Q_{100} = e \left[\ln 106 + 2.054 \frac{\ln(426/106)}{2.326} \right] = 326 m^3/s$$

b)   Flow duration curve

PROBABILITY OF EXCEEDENCE (%)	DISCHARGE (M³/S)
0	98.45
5	59.33
20	32.62
40	9.47
60	4.58
80	3.00
95	2.03
100	1.78

example:

$$\ln Q_{5\%} = -3.4978 + 0.9814 \ln (308.5 + l) + 0.2670 \ln 1500$$
$$= 4.083 \rightarrow Q_5 = e^{4083} = 59.33$$

$$\ln Q_{5\%} = -5.9543 + 1.0070 \ln (330) + 0.3231 \ln 1500$$
$$= 2.248 \rightarrow Q_{40} = e^{2.248} = 9.47$$

$$\ln Q_{95\%} = -5.4716 + 1.0776 \ln (308.5 + l)$$
$$= 0.708 \rightarrow Q_{95} = e^{0.708} = 2.03$$
$$Q_{100\%} = [-0.09892 + 0.08149 \times \sqrt{(308.5 + 1)}]^2 = 1.78$$

c) Long term average discharges

MONTH	LONG TERM AVERAGE DISCHARGE (M³/S)
January	3.88
February	3.30
March	3.00
April	3.17
May	4.37
June	15.17
July	43.86
August	52.45
September	40.07
October	17.59
November	8.06
December	5.24
Annual	16.68

example:

$$Q_{mean\ July} = 0.02123.(330)°.(308.5+1)^{1.0093}.(1500)^{0.2523}$$
$$= 43.86\ m^3/s$$

A.3.2 MIP PROCEDURE

Estimating the hydrograph of mean monthly flows

MONTH	NON-DIMENSIONAL HYDROGRAPH	MEASURED FLOW (M³/S)	PREDICTED APRIL FLOW (M³/S)	PREDICTED HYDROGRAPH (M'/S)
January	2.71			2.71 x 2.8 = 7.59
February	1.88			5.26
March	1.38			3.86
April	1.00	2.8	2.8/1.00 = 2.8	2.80
May	1.88			5.26
June	3.13			8.76
July	13.54			37.91
August	25.00			70.00
September	20.83			58.32
October	10.42			29.18
November	5.00			14.00
December	3.75			10.50

A.3.3 RESULTS

1. The dry season mean monthly flows calculated by the WECS and MIP methods are presented in the table and figure below. WECS shows a slightly higher figure than MIP for the month of April. By definition MIP shows the measured flow. Experience shows that results obtained by WECS and MIP methods vary for different catchments and it may not be always true that MIP yields lower value than WECS. It is worth mentioning that Salleri Chialsa mini-hydro scheme uses a design flow of 2.5 m³/s (Ref.6).

MONTH	WECS	MIP	MEASURED FLOW
October	17.59	29.18	
November	8.06	14.00	
December	5.24	10.50	
January	3.88	7.59	
February	3.30	5.26	
March	3.00	3.86	
April	3.17	2.80	2.80
May	4.37	5.26	

HYDROGRAPH OF MEAN MONTHLY FLOWS
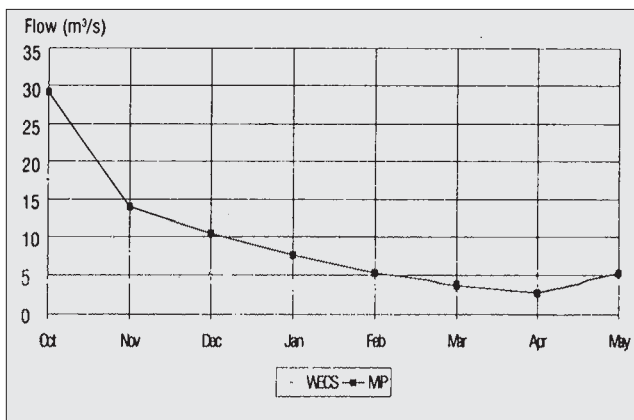
2. The design flow of 2.5 m³/s is exceed 85% of the time, according to the WECS flow duration curve .
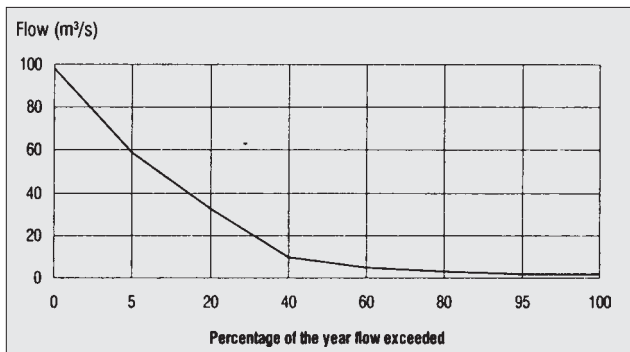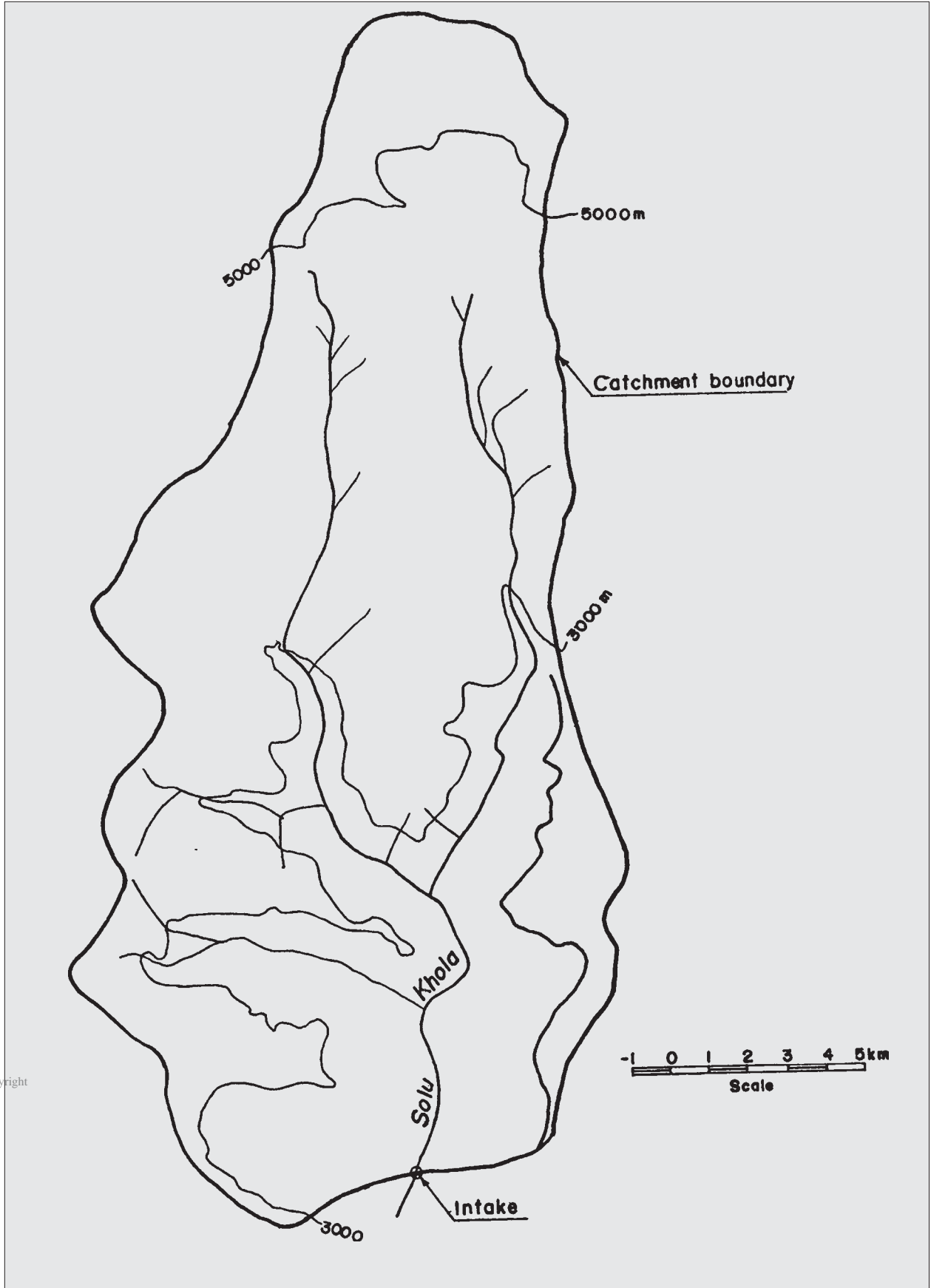
Figure A1 CATCHMENT OF SOLUKHOLA ABOVE THE SALLERI CHIALSA INTAKE
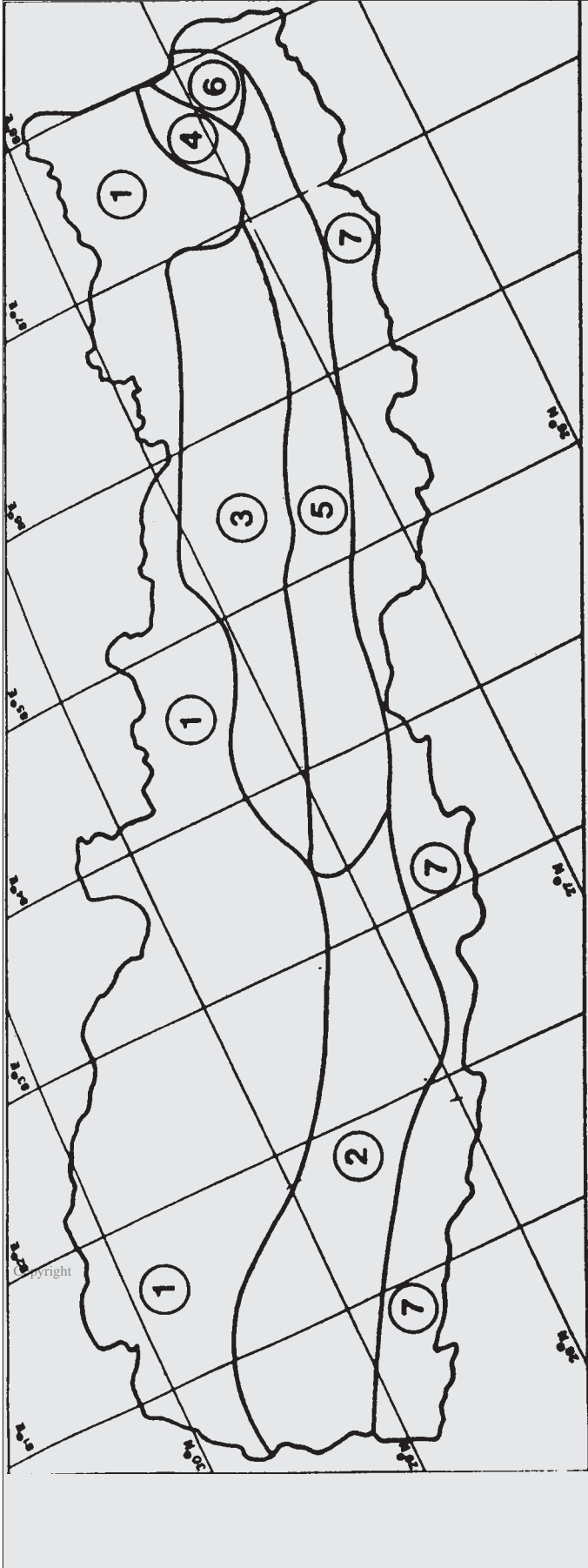
Figure A2 The hydrological regions of Nepal.

Key:
1. Mountain catchments
2. Hills to north of Mahabharats
3. Pokhara, Nuwakot, Kathmandu, Sun Koshi tributaries
4. Lower Tamur Valley
5. River draining Mahabharats
6. Kankai Mai basin
7. Rivers draining from Churia range to the Terai
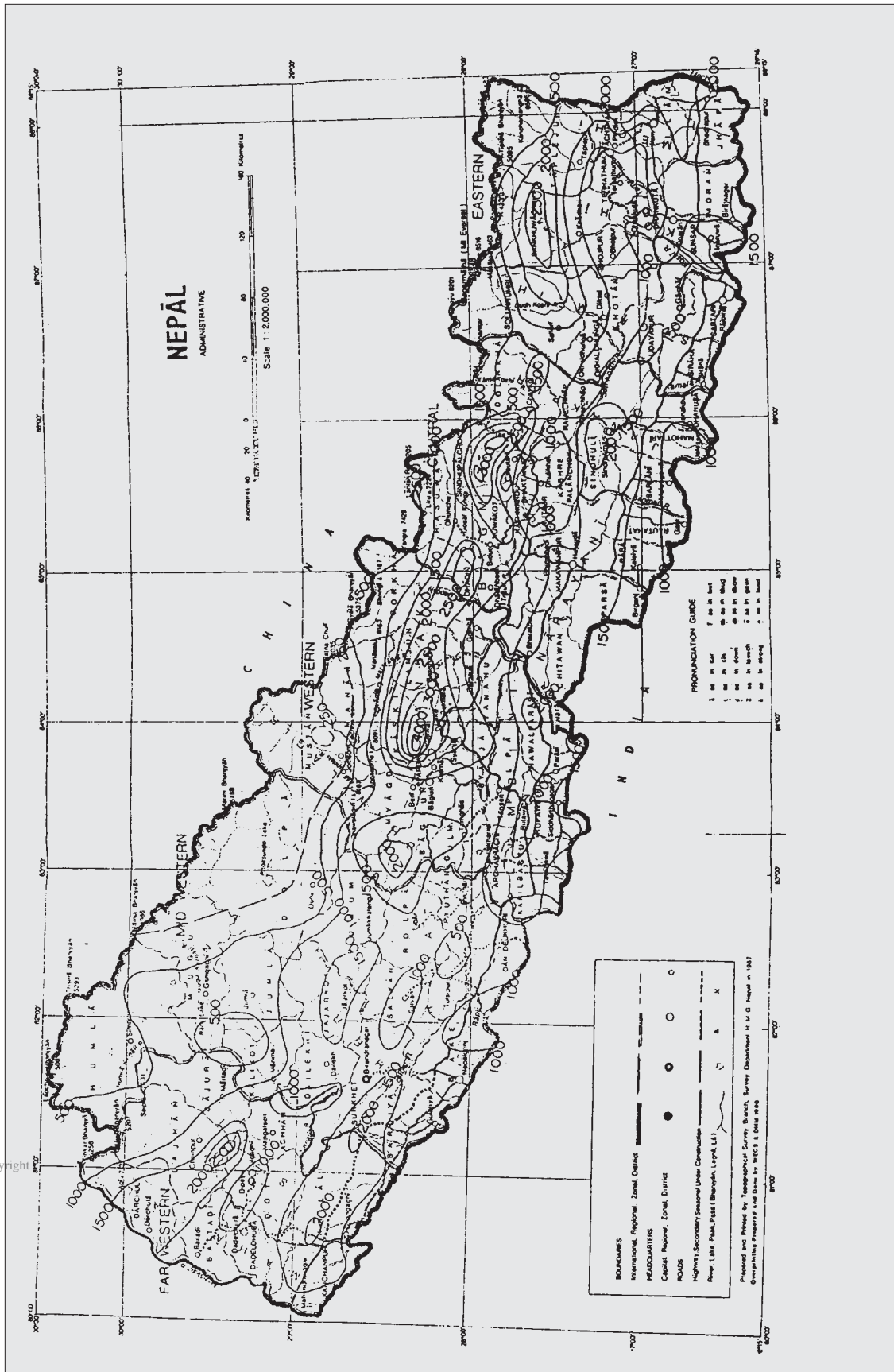
Figure A3 Monsoon wetness index isolines
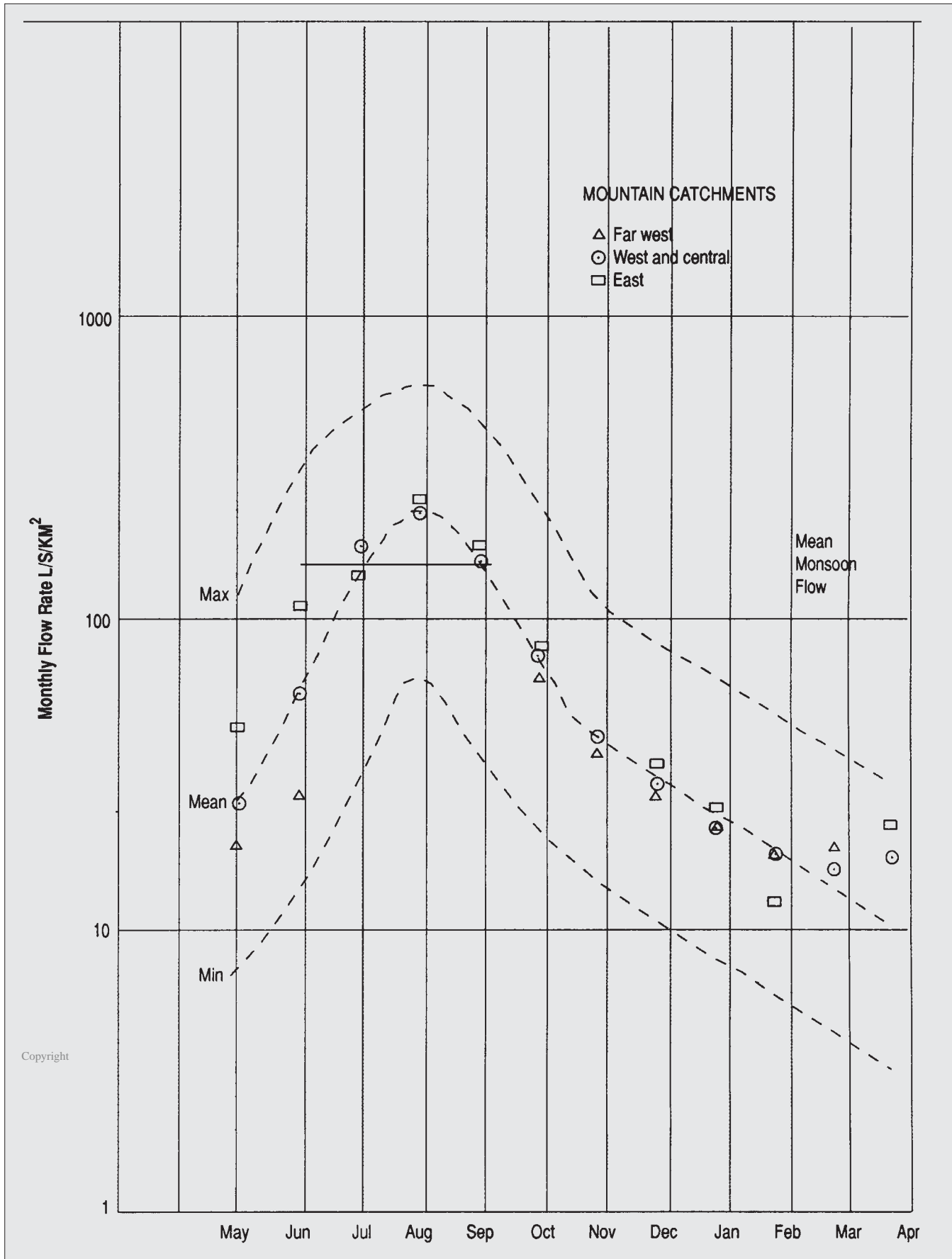
Figure A4 Mean Monthly Hdydrograph - Region 1 (Source Ref. 5)
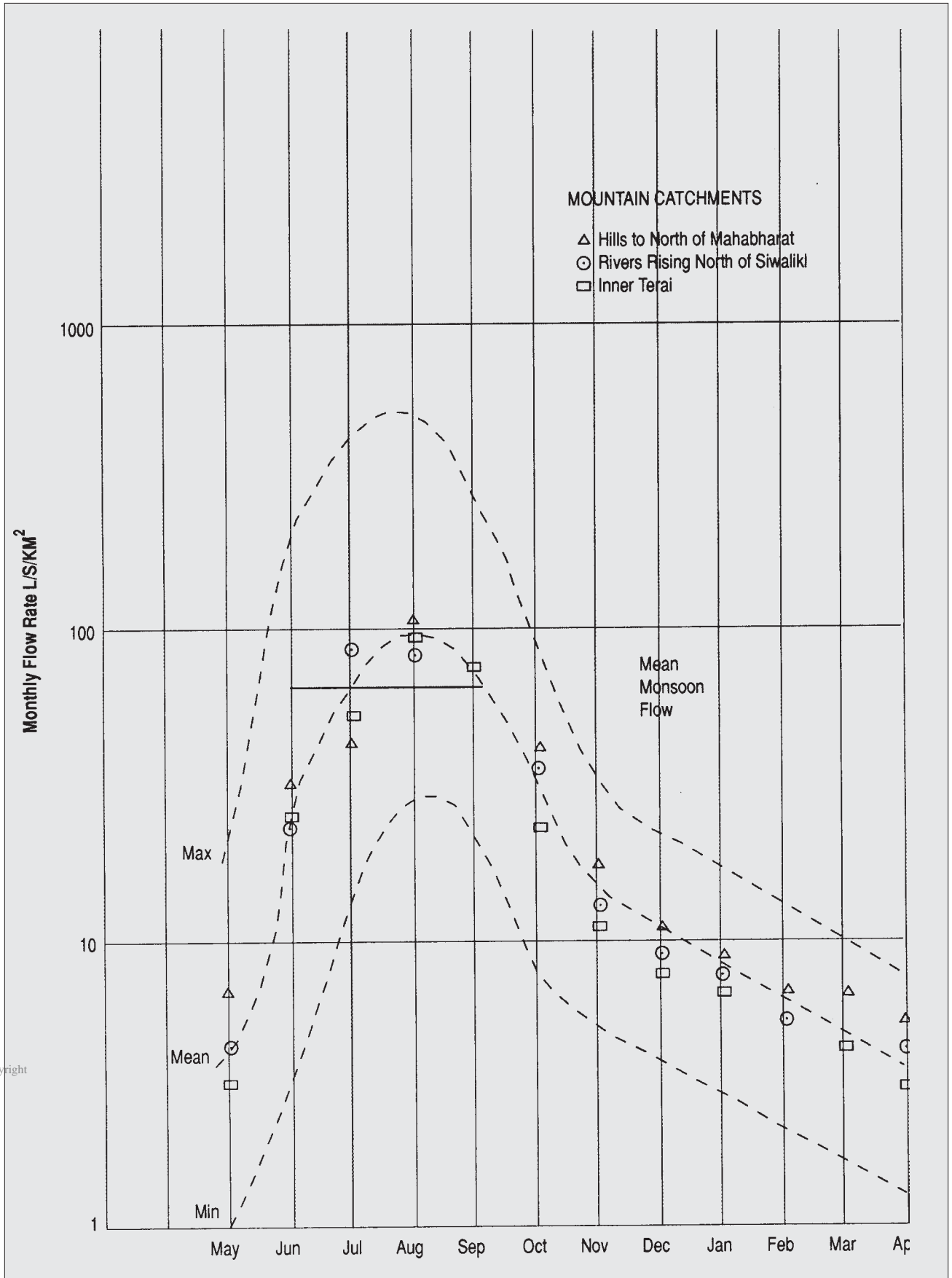
Figure A6 Mean Monthly Hydrograph - Region 2 (Source Ref.5)
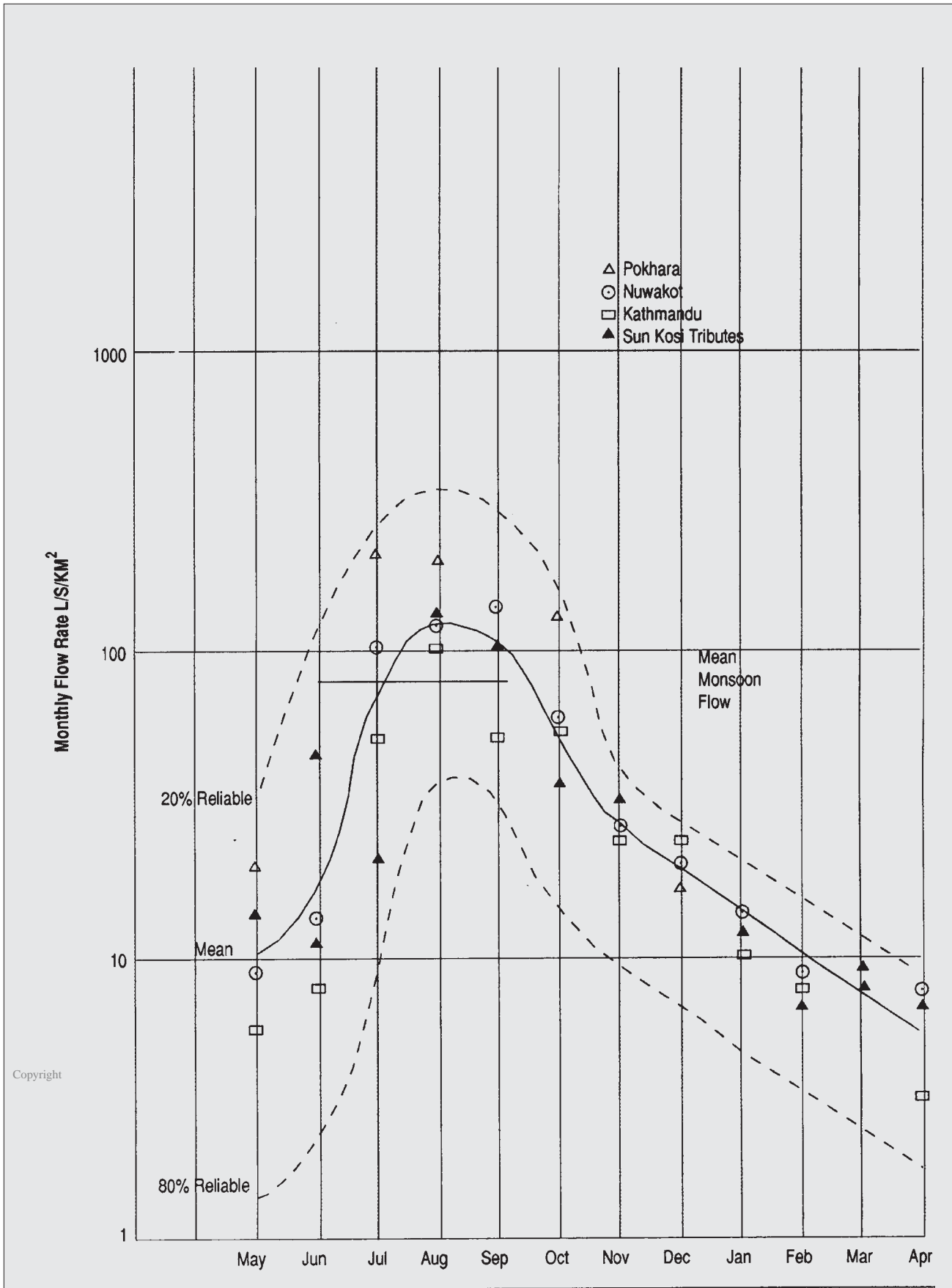
Figure A6 Mean Monthly Hydrograph - Region 3 (Source Ref.5)
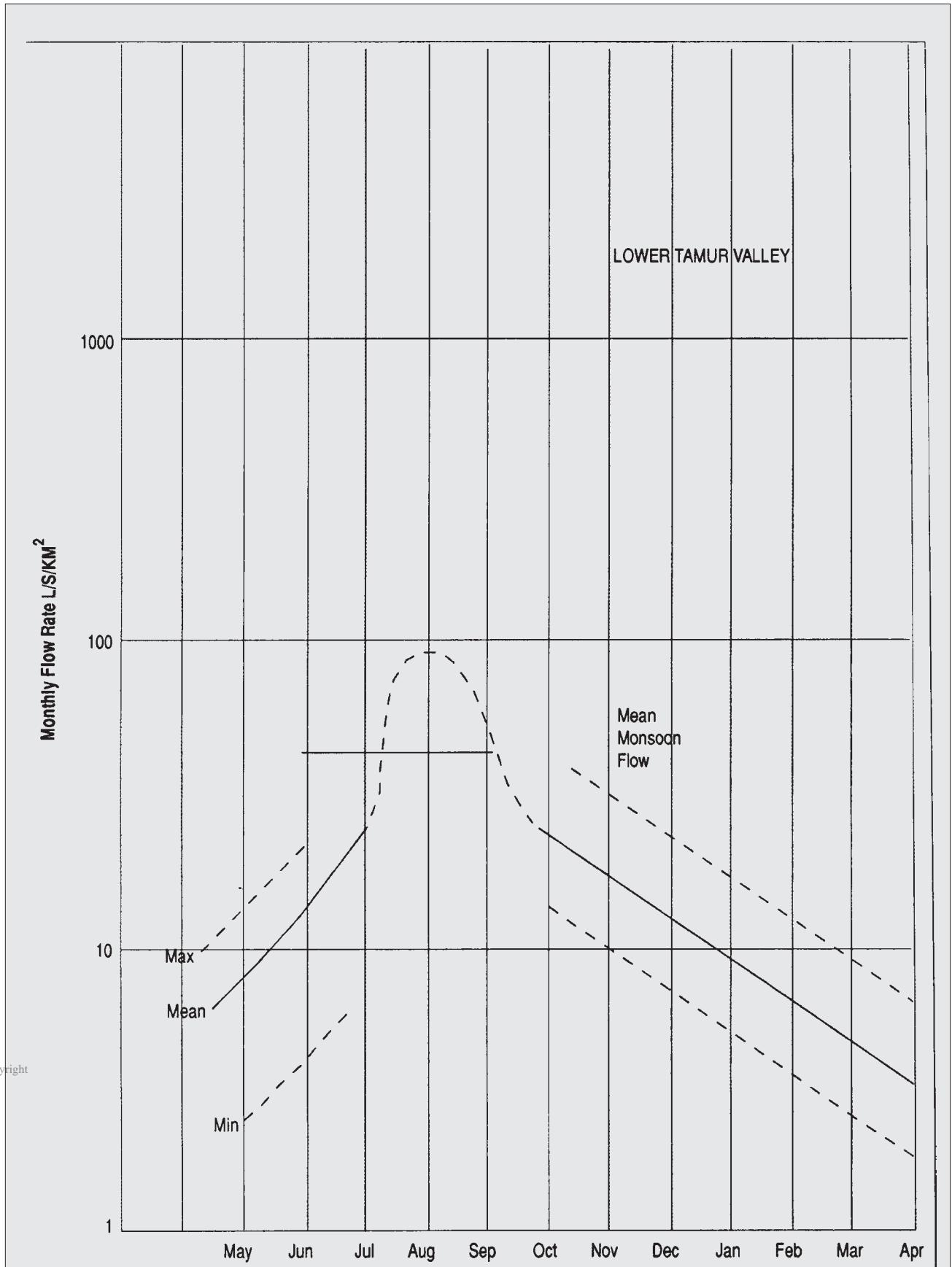
Figure A7 Mean Monthly Hydrograph - Region 4 (Sources Ref.5)
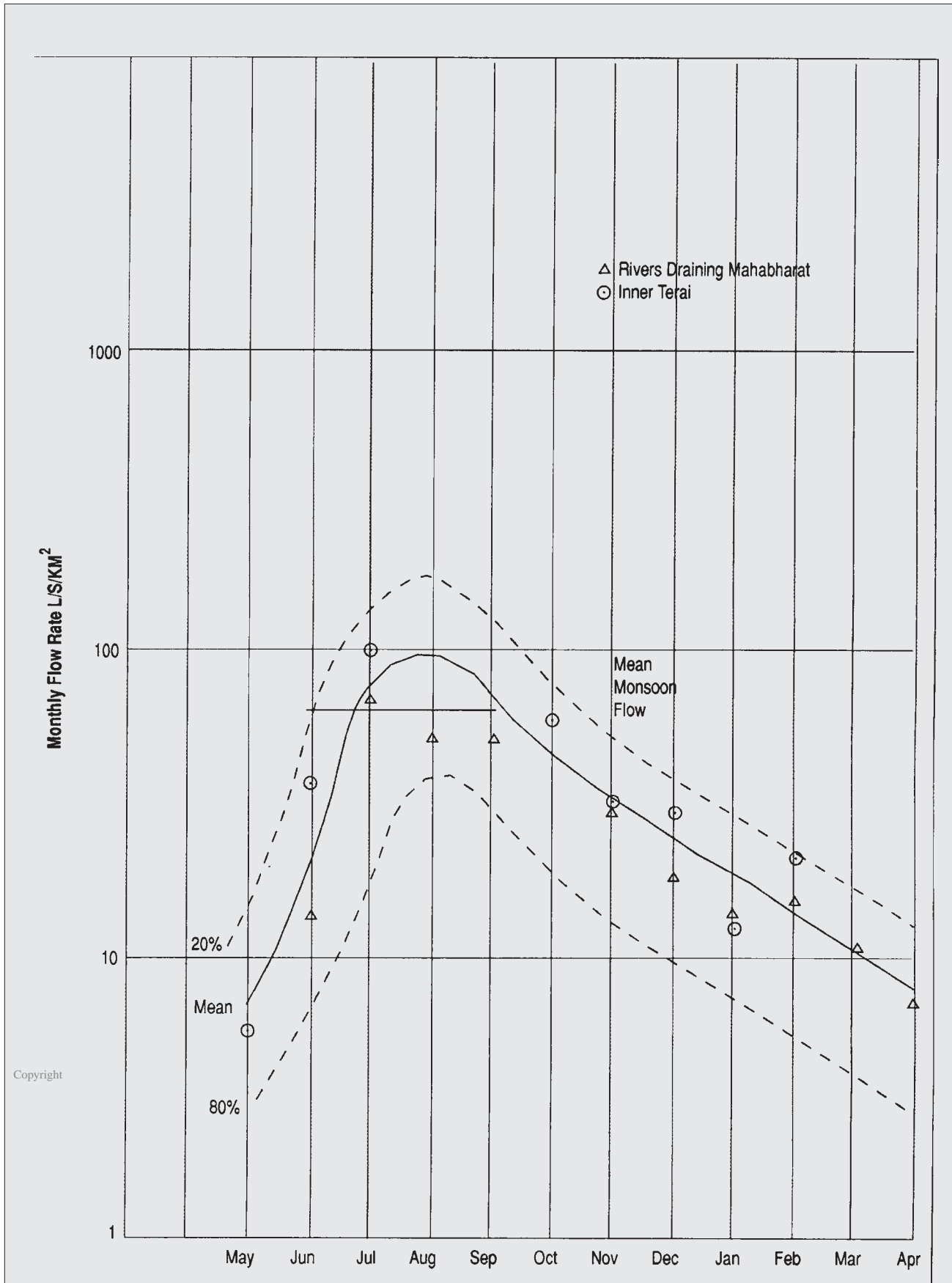
Figure A7 Mean Monthly Hydrograph - Region 5 (Sources Ref.5)
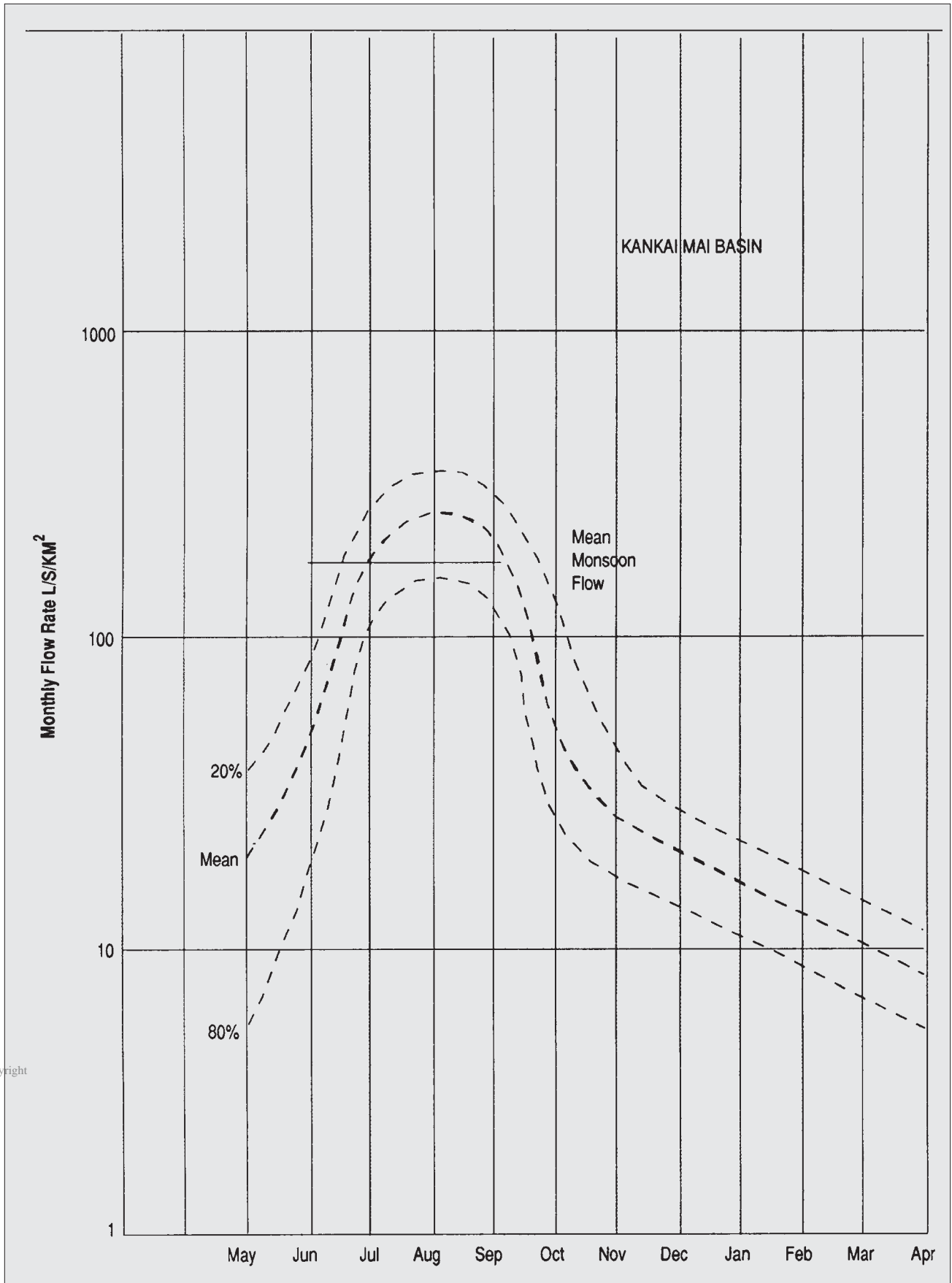
Figure A7 Mean Monthly Hydrograph - Region 6 (Sources Ref.5)
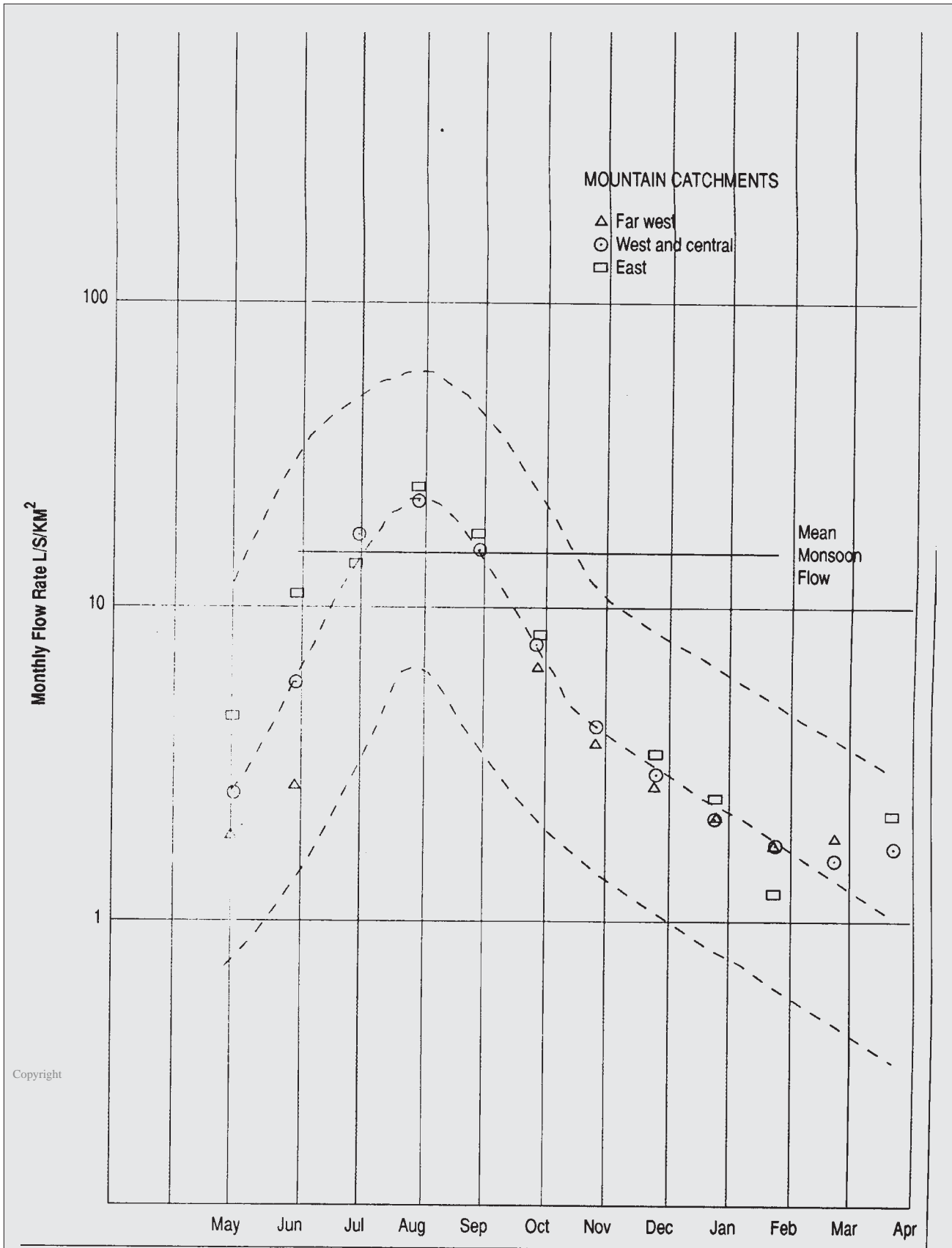
Figure A7 Mean Monthly Hydrograph - Region 7 (Sources Ref.5)

Appendix B -Standard pipe sizes manufacturers and suppliers

List of Tables (as of March 1999)

NEPAL POLYTHENE & PLASTIC INDUSTRIES (Pᴠᴛ.) Lᴛᴅ.
Manufacture of HOPE Pipes

FACTORY; **Balaju Industrial District :**
Balaju, Kathmaadu,Nepal.
Phone : 350091

HEAD OFFICE :   Post Box No. 1015
Tripureswor, Kathmandu
Phone : 261501, 261749
Fax No. 00977-1-261828
Telax No. 2365 KHANAL NP

Nepothene High Densily Polyetbene Pipe Manufactured as per NS 40/040
Price for H. D. P. E Pipes Manufactured as per NS 40/040
PRICE LIST

Outside Diameter mm	SERIES II Working Pressure 2.5 Kgf/cm sq. Wall Thickness			SERIES III Working Pressure 4Kgf/cm sq. Wall Thickness			SERIES IV Working Pressure 6 Kgf/cm sq. Wall Thickness			SERIES V Working Pressure 10 Kgf/cm sq. Wall Thickness		
	Min.	Max.	Price(Rs.)	Min.	Max.	Price(Rs.)	Min.	Max.	Price(Rs.)	Min.	Max.	Price(Rs.)
16mm										2.0	2.4	11.13
20mm1/2"										2.3	2.8	16.21
25mm 3/4										2.8	3.3	24.44
32mm 1"							2.3	2.8	27.35	3.6	4.3	40.41
4Cmm1.25"				2.0	2.4	30.37	2.9	3.4	42.35	4.5	5.2	62.19
50mm 1.5"				2.4	2.9	45.74	3.6	4.2	65.58	5.6	6.4	96.32
63mm 2"	2.0	2.4	48.76	3 0	3.5	70.79	4.5	5.2	102.8.5	7.1	8.1	153.55
75mm 2.5"	2.3	2.8	67.40	3.6	4.3	102.37	5.3	6.1	144.11	8.4	9.5	215.62
9Omm 3"	2.8	3.3	96.68	4.4	5.2	147.62	6.4	7.3	207.76	10.1	11.4	310.73
110mm 4"	3.4	4.0	143.39	5.0	5.8	20606	7.8	8.8	301.95	12.4	13.9	459.92
125mm 4.5"	3.9	4.5	185.13	6.0	6.8	276.97	8.9	10.8	398.45	14.1	15.7	600.40
140mm 5"	4.3	5.0	229.54	6.8	7.7	351.02	9.9	11.1	502.15	15.8	17.6	715.29
160mm 6"	4.9	5.6	296.81	7.8	8.8	456.53	11.3	12.7	647.96	18.0	20.0	977.56
180mm 7"	5.6	6.4	380.91	8.7	9 8	576.20	12.8	14.3	822.80	20.3	22.6	1240.98
200mm 8"	6.2	7.1	468.88	9.7	10.9	712.69	14.2	15.9	1015.31	225	25.0	1527.02
225mm 9"	6.9	7.8	583.46	10.9	12.2	900.85	15.9	17.7	1275.32	25.4	28.2	1937.69
250mm 10"	7.7	8.8	727.45	12.1	13.6	1111.63	17.7	19.7	1577.96	28.2	31.3	2390.60
280mm 11"	8.6	9.7	903.99	13.5	15.1	1386.06	19.8	22.0	1975.57	31.5	34.5	2989.67
315mm 12"	9.7	10.8	1139.46	15.2	17.0	1755.47	22.3	24.8	2503.97	36.4	39.2.	3770.36
355mm 14"	10.9	12.2	1446.80	17.1	19.1	2224.22	25.1	27.9	3175.40	39.9	44.1	4795.84
'00mm 16"	12.3	13.8	1841.74	19.3	21.5	2824.50	28.3	31.4	4030.39	45.0	40.7	6091.62

Terms & Conditions:

1.   10% Tax will be charge extra.
2.   The price are ex-factory price excluding sales tax & contract tax.
3.   Subject to Usual'Force Major condition.
4.   Price are Subject to change with out notice, 25% advance should be paid at the time of ordering of goods and balance should be paid before delivery.

* The quoted prices are from 2002. The prices may vary as of 2009

PANCHAKANYA PLASTIC IND. [P] LTD.
Pipes Manufacturered as per NS 40/040
[Manufacturerhes of HDPE & Accessories]

Outside Diameter mm	SERIES Working Pressure 2.5 Kgf/cm sq. Wall Thickness			SERIES Working Pressure 4Kgf/cm sq. Wall Thickness			SERIES Working Pressure 6 Kgf/cm sq. Wall Thickness			SERIES Working Pressure 10 Kgf/cm sq. Wall Thickness		
	Min.	Max.	Weight	Min.	Max.	Weight	Min.	Max.	Weight	Min.	Max.	Weight
16 mm										2.0	2.4	0.092
20mm										2.3	2.8	0.134
25 mm										2.8	3.3	0.202
32 mm							2.3	2.8	0.226	3.6	4.3	0.334
40 mm				2.0	2.4	0.251	2.9	3.4	0.350	4.5	5.2	0.514
50mm				2.4	2.9	0.378	3.6	4.2	0.542	5.6	6.4	0.796
63 mm	2.0	2.4	0.403	3.0	3.5	0.585	4.5	5.2	0.850	7.1	8.1	1.269
75mm	2.3	2.8	0.557	3.6	4.3	0.846	5.3	6.1	1.191	8.4	9.5	1.782
90mm	2.8	3.3	0.799	4.4	5.2	1.220	6.4	7.3	1.717	10.1	11.4	2.568
110mm	3.4	4.0	1.185	5.0	5.8	1.703	7.8	8.8	2.545	12.4	13.9	3.801
125mm	3.9	4.5	1.530	6.0	6.8	2.289	8.9	10.8	3.293	14.1	15.7	4.962
140mm	4.3	5.0	1.897	6.8	7.7	2.901	9.9	11.1	4.150	15.8	17.6	6.209
160mm	4.9	5.6	2.453	7.8	8.8	3.773	11.3	12.7	5.355	18.0	20.0	8.079
180mm	5.6	6.4	3.148	8.7	9.8	4.762	12.8	14.3	6,800	20.3	22.6	10.256
200mm	6.2	7.1	3.875	9.7	10.9	5.890	14.2	15.9	8.391	22.5	25.0	12.620
225mm	6.9	7.8	4.822	10.9	12.2	7.445	15.9	17.7	10.544	25.4	28.2	16.014
250mm	7.7	8.8	6.012	12.1	13.6	9.187	17.7	19.7	13.041	28.2	31.3	19.757

HEAD OFFICE: J3/42 Krishna Galli, Lalitpur, P.O. Box. No. 2743 Kathmandu, Nepal. Phone 526357, 525172. Fax No. 526529. E-mail: steel@pnchknya.mos.com.np

FACTORY: Lumbini Zone, Kotihawa, Bhairahawa, Nepal. Phone (071) 60368, Fax: 60574.

PANCHAKANYA ROTOMOULDS [P] LTD.
Pipes Manufacturered as per NS 206/046
[Manufacturer of uPVC Pipes & Accessories]

Outside Diameter mm	CLASS - 1 Working Pressure 2.5 Kgf/cm sq. Wall Thickness			CLASS - 2 Working Pressure 4Kgf/cm sq. Wall Thickness			CLASS - 3 Working Pressure 6 Kgf/cm sq. Wall Thickness			CLASS - 4 Working Pressure 10 Kgf/cm sq. Wall Thickness		
	Min.	Max.	Weight	Min.	Max.	Weight	Min.	Max.	Weight	Min.	Max.	Weight
20mm										1.1	1.5	0.111
25 mm										1.4	1.8	0.170
32mm										1.8	2.2	0.270
40 mm							1.4	1.8	0.281	2.2	2.7	0.416
50 mm							1.7	2.1	0.418	2.8	3.3	0.647
63 mm				1.5	1.9	0.474	2.2	2.7	0.668	3.5	4.1	1.010
75mm				1.8	2.2	0.660	2.6	3.1	0.931	4.2	4.9	1.443
90mm	1.3	1.7	0.606	2.1	2.6	0.944	3.1	3.7	1.334	5.0	5.7	2.848
110mm	1.6	2.0	0.894	2.5	3.0	1.369	3.7	4.3	1.938	6.1	7.0	3.077
140mm	2.0	2.4	1.413	3.2	3.8	2.223	4.8	5.5	3.178	7.7	8.7	4.955
160mm	2.3	2.8	1.884	3.7	4.3	2.947	5.4	6.2	4.139	8.8	9.9	6.456
180mm	2.6	3.1	2.333	4.2	4.9	3.474	6.1	7.0	4.961	9.9	11.1	7.830
200mm	2.9	3.4	2.945	4.6	5.3	4.563	6.8	7.7	6.464	11.0	12.3	10.185

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ΗULΑΣ Steel Industries P. Ltd.

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Fax: 977-1-220612 Factory : Simra. Bara. Tel: 977-53-20075, 20078, 20170. Fax: 977-53-20160

GALVANISED AND BLACK STEEL PIPES FOR ORDINARY USES IN WATER, GAS AIR & STEAM LINES

TVPE	NOMINAL BORE		WALL THICKNESS	APPOX OD	WEIGHT OF BLACK PIPE PLAIN END		WEIGHT OF GALVANISED PIPE THREADED & SOCKETED	
CLASS	MM	INCH	MM	MM	KG/M	MTR/M.1	KG/M	MTR/M.T
LIGHT' 'A'	15	1/2	2.00	21.30	0.92	1050	1.01	990
	20	3/4	2.35	26.90	1.41	709	1.48	676
	25	1	2.65	33.70	2.01	498	2.11	474
	32	1 1/4	2.65	42.40	2.58	388	2.72	355
	40	1 1/2	2.90	48.30	3.25	308	3.41	281
	50	2	2.90	60.30	4.11	243	4.33	222
	65	2 1/2	3.25	76.20	5.80	172	6.11	156
	80	3	3.25	88.90	6.81	147	7.21	133
	100	4	3.65	114.30	9.89	101	10.49	91
MEDIUM 'B'	15	1/2	2.65	21.30	1.22	820	1.28	781
	20	3/4	2.65	26.90	1.58	633	1.65	606
	25	1	3.25	33.70	2.44	410	2.54	394
	32	1 1/4	3.25	42.40	3.14	318	3.27	292
	40	1 1/2	3.25	48.30	3.61	277	3.77	254
	50	2	3.65	60.30	5.10	196	5.32	179
	65	2 1/2	3.65	76.20	•.51	154	6.82	140
	80	3	4.05	88.90	8.47	118	8.87	107
	100	4	4.50	114.30	12.10	83	12.69	75
	125	5	4.85	139.70	16.20	62	16.95	55
	150	6	4.85	165.10	19.20	52	20.00	47
HEAVY 'c'	20	3/4	3.25	26.90	1.90	526	1.97	508
	25	1	405	3370	297	336	3.07	326
	32	1 1/4	4.05	42.40	3.84	260	3.97	239
	40	1 1/2	4.05	48.30	4.43	226	459	207
	50	2	4.50	60.30	6.17	162	6.39	148
	65	2 1/2	4.50	76.20	7.90	127	8.21	115
	80	3	4.85	88.90	10.10	99	10.52	90
	100	4	5.40	114.30	14.40	69	15.44	63
	125	5	5.40	139.70	17.80	56	18.52	51
	150	6	5.40	165.10	21.20	47	22.22	42
200mm GALVANISED AND BLACK STEEL PIPES (Detailed specification on request)								
	200	8	5.20	219.10	27.71	36	29.20	32
	200	8	6.00	219.10	31.82	31	33.20	28

Notes:

1) Tensile Strength: Tensile strength for water tubes when tested from strips, cut out from selected tubes shall be more than 320 n/mm².

2) Tolerances: a) Thickness: Light tubes: {+} not limited, (-) 8% max. Medium & Heavy pipes: (+) not limited, (-) 10% max.

b) Weight: Single tubes: (+) 10%, (-) 8%.

c) Length: Unless otherwise specified 4 to 7 meters.

3) Pipes of higher tensile strength and bigger diameter can also be manufactured as per request.

A JOINT VENTURE BETWEEN

HULAS Steel Industries P. Ltd.

HIPCO STRUCTURAL STEEL PIPES

Structural steel pipes are manufactured in the same process as water pipes. However, the steel used is of higher tensile/yield strength. We normally use high grade steel with tensile strength of 42/55 kgs/cm as per customers requirements. We normally use the steel as per JIS: G-3132, SPHT2,3or4, or IS: 11513/1985 for the pur-poses. Structural pipes are basically used for manufactur-ing poles and welded structures. Use of high grade struc-tural steel reduces steel consumption drastically Pipes of structural grade normally conform to IS: 1161/1979, JIS: G-3444/1993, BS: 1387/1985.

HIPCO STRUCTURAL TUBES

NOMINAL BORE		OUTSIDE DIAMETER	CLASS	THICKNESS	WEIGHT (BLACK PIPES)	AREA OF CR. SECTION	MOMENT OF INERTIA	SECTION MODULUS	RADIUS OF GYRATION
mm	inch	mm		mm	kg/m	Cm	Cm	Cm	Cm
15	1/2	21.30	L	2.00	0.96	1.21	0.57	0.54	0.69
			M	2.65	.22	1 55	0.69	0.65	0.67
20	3/4	26.90	H	3.25	.45	1.84	0.77	073	0.65
			L	2.35	.42	1.81	1 38	1.02	0.87
			M	2.65	.58	2.02	1.50	1.12	086
25	1	33.70	H	3.25	.90	2.41	1.72	1.28	084
			L	2.65	2.04	2.58	3.14	1.86	.10
			M	3.25	2.46	3.11	365	2.16	.08
32	1¼	42.40	H	4.05	2.99	3 77	4.22	2.51	.06
			L	2.65	2.61	331	6.57	3.10	41
			M	3.25	3.15	4.00	7.71	3.64	39
40	1½	48.30	H	4.05	3.86	4.88	9.07	4.28	.36
			L	2.90	3.27	4.14	10.70	4.43	.61
			M	3.25	3.61	460	11.73	4.86	.60
			H	4.05	4.43	5.63	13.90	5,75	.57
50	2	60.30	LI	2.90	4.14	523	21.59	7.16	2.03
			L2	3.25	4.57	5.82	23.77	7.89	2.02
			M	3.65	5.10	6.50	26 17	8.68	201
			H	4.50	6.17	7.89	30.90	1020	1.98
65	2½	76.10	L	3.25	5,84	7.44	49.44	13.00	2.58
			M	3.65	653	8.31	5465	14.40	2.56
			H	4.50	7.92	10.10	65.12	17 10	254
80	3	8890	L	3.25	6.86	8.74	80.31	18.07	3.03
			M	4.05	8.48	10.80	97.38	21.91	3.00
			H	4.85	10.01	1280	11346	25.53	2.98
100	4	114.30	L	3.65	9.97	12.70	194.39	34.01	3.91
			M	4.50	12.10	15.50	234.32	41.00	3.89
			H	5.40	14.50	18.50	274.54	48.04	386
125	5	139.70	L	4.50	14.90	19.10	437.20	62.59	4.78
			M	4.85	16.20	20.50	467.64	66.95	4.77
			H	5.40	17.90	22.80	514.49	73.66	4.75
150	6	165.10	L	4.50	17.80	22.70	732.57	88.74	5.68
			M	4.85	19.20	24.40	784.49	95.03	5.67
			H	5.40	21.20	27.10	864.69	104.75	5.65
175	7	193.70	L	4.85	22.60	28.70	1284.00	133.00	6.68
			M	5.40	25.00	31.90	1416.96	146.30	6.66
			H	5.90	27.30	34.80	1536.11	159.00	6.64
200	8	219.10	L	4.85	25.70	32.60	1874.06	171.07	7.58
			M	5.60	29.40	37.60	2141.59	195.49	7.55
			H	5.90	31.00	39.50	2247.00	205.11	7.54

Tolerance:
a) Thickness: (+) not limited (-) 10%   b) Weight: (+) 10% (-) 8%
c) Outside Daimeter: Upto 48.3 mm, (+) 0.4 mm (-) 0.8 mm, (+) 1%

A JOINT VENTURE BETWEEN

Appendix C Galkot drawings

List of drawings

PLAN

SECTION C-C
Scale B

SECTION B-B
Scale B

SECTION D-D
Scale B

SECTION E-E
Scale B

SECTION A-A
Scale B

PLAN
Scale D

Drawing 6 headwork and headrace, gravel trap/overflow

SECTION D-D

SECTION C-C

PLAN
(Crossing no. 1 Aqueduct)

C - 7

SECTION A - A

PLAN

Drawing 11 penstock area, anchor block and support details

SECTION C-C
Scale B

PLAN
PIPE SUPPORT AND FILLING CHANNEL

SECTION B-B
Scale C

PLAN
ANCHOR BLOCK NO 2
Scale C

SECTION A-A
Scale C

PLAN
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SECTION B - B

SECTION A - A
(POWERHOUSE WALL & FOUNDATION NOT SHOWN)

SECTION D - D

SECTION C - C

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In 1979, Practical Action started its work in Nepal, initially concentrating on the development and transfers of micro-hydro technologies, building the capacities of local manufacturers and rural entrepreneurs, and advocating for appropriate policies and institutions in the micro-hydro sector. After establishment of the Country Office in 1998, Practical Action diversified its activities into other forms of renewable energy, expanding into agro processing, rural transport and disaster management. Since 2003, Practical Action Nepal Office is directed by four International Programme Aims (IPAs): Reducing vulnerability; Markets and livelihoods; Improving access to useful services, systems and structures; and Responding to new technologies. Within these Aims, Practical Action focuses its work mainly on six broad priority areas in Nepal - 1) securing food for the poor, 2) reducing risk from disaster and climate change, 3) minimising impacts of conflict through improved access to market, 4) increasing rural productivity, 5) sustainable urban environment and 6) healthy homes.

Practical Action believes that the right intervention – however small – can create jobs, improve health and livelihoods, give access to services and help people lead better lives. In its every effort, Practical Action aims to bring about positive and lasting changes in people's lives. Practical Action's programmes are driven by the needs of both the rural and urban poor, and are launched through partnership with government, non government (I/NGOs) and private sector stakeholders. In Nepal, Practical Action is operating through a General Agreement and separate Project Agreements with the Social Welfare Council of the Government of Nepal.

PRACTICAL ACTION
Technology challenging poverty

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