CHAPTER 5
Design of the Scheme

The design of an MHP scheme involves specifying the locations, sizes, materials, and other parameters of all components that are to be constructed or installed at the site. An ideal design should be such that based on it an experienced installer should be able to construct the scheme independently of, or with only nominal assistance from, the designer.

The design of the scheme is based primarily on the information obtained during the various surveys. Therefore, it can only be as good as the results of these surveys. The design process is iterative since the dimensions and other parameters of the MHP components are interdependent.

5.1 Diversion Weir

A weir should be constructed to raise the water level in the river upstream if the river flow cannot be diverted naturally into the intake during the low flow period. In rare cases, it may be possible to locate a suitable natural weir of large boulders which requires only minimum additional construction work such as placing more boulders or gabions. However, the type of weir most commonly suitable for MHP plants is a temporary weir constructed or repaired every year after the annual floods. For schemes located in remote areas, this option is appropriate and economical since such a weir does not require cement or skilled labourers. There are two common types of temporary weir.

- A weir across the whole width of the stream, the main design parameter being the height (Figure 5.1).
- A diversion dam that extends along the length of the stream but gradually moves towards the centre to divert more water (Figure 3.10.b). This type of dam is suitable for low flows and for continuous demand at a particular time. It can be further extended if more river flow needs to be diverted.

More permanent weirs constructed from gabions, masonry, or concrete should only be considered if the river does not move boulders during the floods, the site is less than a day's walk from the roadhead, and there is a scarcity of water during the dry season. Permanent weirs can be constructed of plum concrete (1:3:6 with 40% plum), or stone masonry in 1:4 cement mortar. Gabion weirs are semi-permanent structures that usually require some annual maintenance.

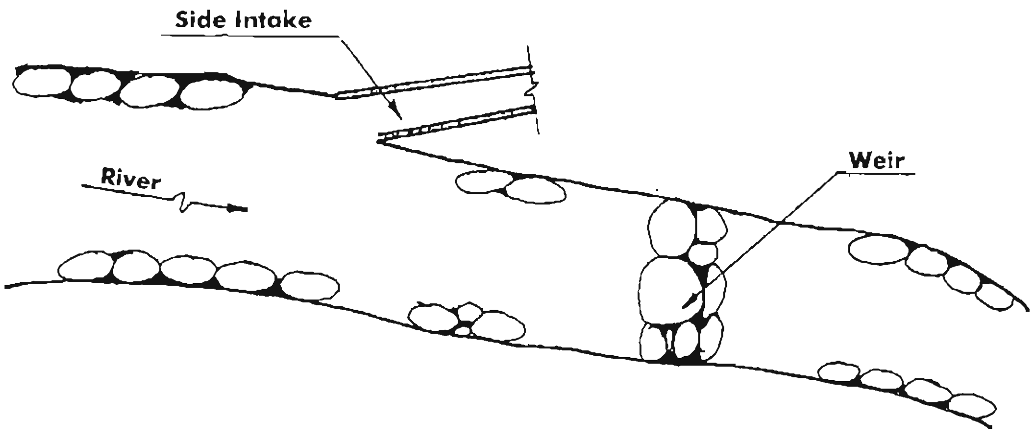

Figure 5.1: Side Intake with Temporary Diversion Weir

For both permanent and temporary weirs, the height should be kept as low as possible but enough to divert the required flow, and the slopes should be gradual so that boulders can roll over the weir. In order to determine the height of a temporary or permanent weir, the river depth/level during the dry season must be known together with the upper height of the orifice (the intake mouth, discussed in section 5.2). The intake height should be such that the water level rises above the upper edge of the orifice. The height of temporary weirs may have to be increased or decreased during the operation of the plant.

5.2 Intake

The intake should be so designed that the head loss is minimal and the entry of excessive flow (during floods) as well as bed load and other floating debris is minimised. Side intakes are most commonly used in MHP schemes since they are simple and less expensive than other types and most suitable for run-of-the-river type plants.

5.2.1 Design of Side Intake

A side intake can be designed either as an extension of the headrace canal (without any proper orifice) as shown in Figure 5.1, or as a rectangular orifice (Figure 3.9). If designed as an extension of the headrace canal, the procedure is to design a canal (discussed in section 5.3) capable of conveying the design flow and extend it to the side intake at the river bank. The initial length of the headrace that is in the flood zone of the stream is sometimes made wider to allow for seepage, especially if this section is temporary. A disadvantage of this type of intake is that it is not possible to automatically limit excess flows from entering the headrace during floods.

A rectangular orifice is a specially constructed opening in the side wall along the river bank, as shown in Figure 5.2, which allows the design flow to enter into the headrace but limits excess flows during floods. It should be sized such that it is submerged at the time of design flow during the low flow season, since this will limit excess flows during floods. An alternative to an orifice is to install a gate immediately downstream of the intake. This allows more control of the flow through varying the opening of the gate. However, such gates require more maintenance work and are also more vulnerable to flood damage.

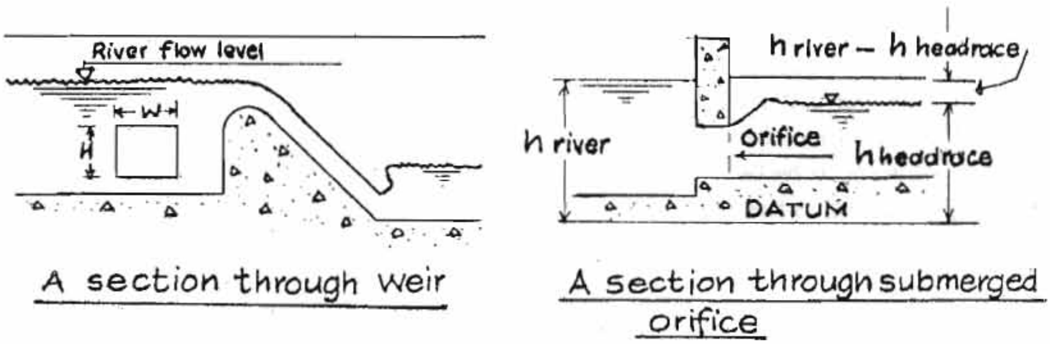

Figure 5.2: Sections through a Weir and a Submerged Orifice

The discharge through an orifice when submerged is given by:

$$Q = A \times V = AC\sqrt{2g(h_{river} - h_{headrace})} \qquad (1)$$

where

Q is the discharge through the orifice in m^3/s

V is the velocity through the orifice in m/s

A is the area of orifice in m^2

h_{river} - $h_{headrace}$ is the difference between the river and the headrace canal water levels (Figure 5.2).

C is the coefficient of discharge of the orifice. For a sharp edged and roughly finished, fully submerged concrete or masonry orifice structure this value can be as low as 0.6 and for a carefully finished and smooth opening it can be up to 0.8.

If it is economically feasible to construct such an orifice, then the excess flow can be minimised during the monsoons.

The size of the orifice is calculated as follows.

- Assume an initial velocity through the orifice of less than 3 m/s. Then calculate the required area of the orifice opening using $Q = V \times A$.
- For a rectangular opening, $A = W \times H$ where W is the width and H is the height of the orifice. Set H such that the water surface level at the headrace canal immediately downstream of it is slightly higher than the upper edge of the orifice. This ensures that the orifice is submerged. Calculations for the water level in the headrace are discussed in section 5.3.
- Now calculate h_{river} for the design flow conditions. This can be done by either rearranging the orifice equation (since all other parameters are known) or by trial and error. The h_{river} is the water depth that needs to be maintained in the river during normal conditions. If the actual level in the river is less during low flow, then the weir crest level will have to be taken to be equal to h_{river} (Figure 5.2).
- If the flood level in the river is known, the flow through the orifice for such conditions should be calculated using h_{river} = flood level (which can be determined by noting flood marks or information from the local community). The water level in the canal with a known flood flow in the headrace can be calculated as discussed in section 5.2. The excess flow will have to be spilled back into the river or nearby gullies. Note that if a weir is placed across the river, the flood level may be somewhat higher than before since the weir raises the water level. For temporary weirs this is not a problem since they normally get washed away. If a permanent weir is used, allowances should be made for this when calculating h_{river} (such as by adding the weir height above the measured flood level).

5.2.2 Trashracks for Side Intakes

A coarse trashrack should be placed at the intake mouth to prevent floating logs, cobbles, and boulders from entering the headrace canal and causing damage. Such a trashrack is fabricated using flat steel plates, angles, or bars (square or circular, about 25mm in diameter) that are welded together at fixed intervals. Since the coarse trashrack can get continuously impounded by cobblestones and other large particles, it needs to be strong. A bar spacing of between 50 and 200mm is generally adequate depending on the size of stones that the river carries during floods.

5.3 Headrace Canal

Many types of headrace canal made of different materials and using different methods of construction are used in MHP schemes. The types of canal and methods of designing the various components are described in the following sections.

5.3.1 Types of Canal

- Simple earthen or unlined canals

  These are constructed by simply excavating the ground to the required canal shape and are the cheapest type. Such canals can be used on stable ground where seepage (which usually exists) is not likely to cause instability problems such as landslides. Compaction of the earth and planting vegetation on the canal banks will increase stability and reduce seepage.

- Canals lined with stone masonry with mud mortar

  The second stronger option is to line the canal with stone masonry using mud mortar. There will be less seepage from this type of canal than from an earthen canal, but the construction will require more labour, materials, and funds. These canals should be used where a small amount of seepage will not cause slope instability, or where flow is limited, that is there is no extra flow that can be diverted into the canal to compensate for seepage.

- Canals lined with stone masonry with cement mortar

  The advantage of this type of canal is that seepage is minimal in comparison with earthen or stone-mud canals. However, this type of canal is significantly more expensive than the previous two types as a result of the cost of cement. A stone masonry in cement mortar canal is only needed along stretches where the soil is porous (causing high seepage losses) or where seepage is likely to trigger landslides.

- Canals lined with concrete

  Concrete canals are rarely used in MHP schemes except for short lengths at difficult locations, since they are more expensive than stone masonry with cement mortar canals. There is virtually no seepage from concrete canals. However, if the area is not stable then the whole structure can slide unless properly restrained or supported.

- Covered canals and pipes

  Where stones and other debris are likely to fall from above the headrace route, the canal can either be covered or pipes may be used. Flat stones are an economical way of covering canals; an expensive alternative is to use reinforced concrete slabs. Buried pipes made, for example, from HDPE also offer protection from falling debris. Another advantage of HDPE pipes is that they are flexible and can adjust to a certain amount of ground movement.

5.3.2 Design of the Headrace Canal

The canal dimensions and cross-section are governed by the following criteria.

- Flow and velocity

  The cross-sectional area should be such that the velocity is within the limits for the design flow (Table 5.1). Freeboard should also be allowed for excessive flow.

Table 5.1: Recommended side slopes and maximum velocities for headrace canals

Canal material	Side Slope (N = h/v)	Maximum recommended velocity for canals	
		less than 0.3m depth	less than 1m depth
Sandy loam	1.5 to 2.0	0.4	0.7
Loam	1.0 to 1.5	0.5	0.8
Clay loam	1.25	0.6	0.9
Clay	1.0	0.8	1.0
Stone masonry with mud mortar	0.5 to 1.0	1.0	1.0
Stone masonry with cement mortar	0 (i.e., vertical) to 1.5	1.5	1.5
Concrete	0 to 1.5	2.0	3.0

- Slope of the side

  The stability of the side walls of the canal depends on their slope. The effect of slope is different for the different types of material used in wall construction. For example, a vertical side wall will not be stable over a long period in a simple earthen canal, but will be stable if the walls are made of stone masonry in cement mortar or of concrete. In the following we define the slope of the side, N, as the ratio of the horizontal breadth, h, divided by the vertical height, v, of the canal wall, as shown in Figure 5.3. The recommended side slope for different types of canal is given in Table 5.1.

- Headloss and seepage

  Headloss can be minimised by routing the canal along even to gently sloping ground. Headloss is governed by the type of canal as well as the number and magnitude of bends. There are less frictional and head losses when the water flows over a smooth surface like that of cement plaster. The frictional losses are indicated by the roughness coefficient 'n', The values of n for different types of canal are given in Table 5.2.

- The Design Procedure for Canals

  1. Decide on the canal type according to the site conditions and taking into account stability and seepage (see above).

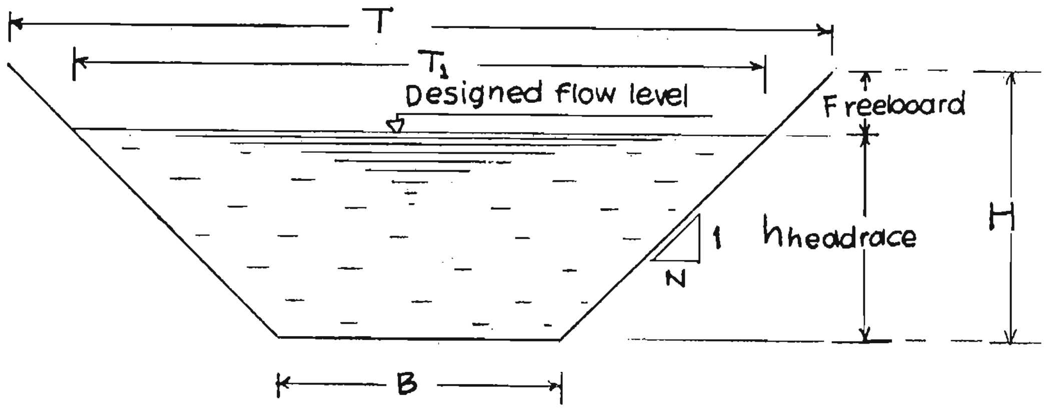

Figure 5.3: Headrace Canal with Trapezoidal Cross-Section

Table 5.2: Roughness Coefficients for Different Canals		
Canal type	**Description**	**Roughness coefficient 'n'**
Earthen canals	Clay, with stones and sand, after ageing	0.020
	Gravelly or sandy loam, maintained with minimum vegetation	0.030
	Lined with coarse stones, maintained with minimum vegetation	0.040
Rock canals	Medium coarse rock muck	0.037
	Rock muck from careful blasting	0.045
	Very coarse rock muck, large irregularities	0.059
	Rubble masonry with mud mortar	0.025
Masonry canals	Brickwork, bricks, and/or clinker with well-pointed cement mortar	0.015
	Normal masonry with cement mortar	0.017
	Coarse rubble masonry and coarsely hewn stones with cement mortar	0.020
Concrete canals	Smooth cement finish	0.010
	Concrete for which wood formwork was used, unplastered	0.015
	Tamped concrete with smooth surface	0.016
	Coarse concrete lining	0.018
	Irregular concrete surface	0.020

2. Choose a suitable velocity (V) for the type of canal selected by referring to Table 5.1; and find the roughness coefficient (n) from Table 5.2

3. Calculate the cross-sectional area (A) from the equation $A = \frac{Q}{V}$

Where Q is the design flow

4. Using Table 5.1 decide on the side slope (N)

5. Calculate the required dimensions using the following equations

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

X is the factor used to optimise the canal shape. For a rectangular canal, N = 0 and X = 2.

6. The desirable depth of water in the canal, $h_{headrace}$ (see Figure 5.3), is calculated using X.

$$h_{headrace} = \sqrt{\frac{A}{X + N}} \qquad (3)$$

The width of the bed B (see Figure 5.3) is given by
Finally the width of the top T (see Figure 5.3) is given by

$$B = X \times h_{headrace} \qquad (4)$$

$$= B + (2 \times h_{headrace} \times N) \qquad (5)$$

If because of conditions at the site it is not possible to construct the optimum shape (for example, the width available is too narrow), then either the width or the height should be selected to suit the site conditions.

7. The velocity must be less than 90 per cent of the 'critical velocity limit' to ensure uniform flow in the canal. The critical velocity limit, V_c, is given by

$$_c = \sqrt{\frac{A \times 9.81}{T}} \qquad (6)$$

If the canal velocity is greater than $0.9V_c^*$ then repeat the calculations using a lower velocity (i.e., start again from step 2).

8. Calculate the wetted perimeter, P, using

$$P = B + 2 \times h_{headrace} \times \sqrt{(1 + N^2)} \qquad (7)$$

9. Calculate the hydraulic radius, R, using

$$R = A/P \qquad (8)$$

10. The slope (S) can now be found from Manning's equation

$$S = \left[\frac{n\,V}{R^{0.667}}\right]^2 \qquad (9)$$

11. Now calculate the head loss in the canal from

$$Head\ loss = L \times S \qquad (10)$$

where L is the length of the canal section. If the slope of the canal varies along different sections, calculate the head loss for each section and add them up. If the loss is too high, or if the actual ground slope differs from the calculated canal slope, repeat the calculations using different velocities.

12. Allow a freeboard of about 300mm for flows up to 500 l/s.

5.3.3 Design of Spillways

Excess flow that enters into the intake during flood flows needs to be spilled as early as possible. This is achieved by incorporating a spillway close to the intake. If the headrace canal is long, another spillway may be required along the canal section so that the entire design flow can be diverted if the canal is blocked as a result of falling debris or landslides. Finally, a third spillway is almost always required at the forebay to spill the flow in case of sudden valve closure at the powerhouse as may occur during emergencies. The sizing of the spillway is based on the following equation.

$$L_{spillway} = \frac{Q_{flood} - Q_{design}}{1.6 \times (h_{flood} - h_{sp})^{1.5}} \qquad (11)$$

where,

Q_{flood} is the flood flow that enters the intake in m^3/s
Q_{design} is the design flow in the headrace canal
h_{flood} is the height of the flood level in the canal
h_{sp} is the height of the spillway crest from the canal bed

Figure 5.4 shows a section through a spillway.

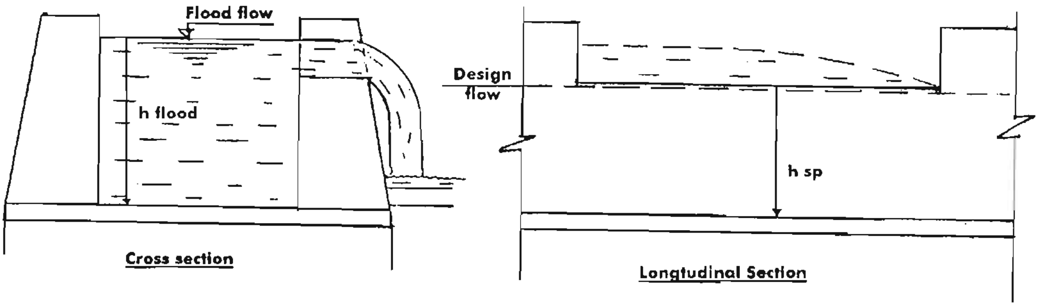

Figure 5.4: Spillway Design

The design procedure involves first calculating the maximum height of the water level in the canal during a flood (h_{flood}). Then the height of the spillway crest (h_{sp}) is set such that it is about 50mm higher than the design water level. This ensures that part of the design flow is not spilled, which would decrease the power output. '1.6' is a coefficient for a broad crested weir of a spillway with round edges which is easy to construct.

5.4 Settling Basins

Figure 5.5 shows the typical design of a settling basin. Settlling basins for an MHP scheme should have the following characteristics.

- The settling area should be large enough to reduce the velocity sufficiently to settle the sediments in the basin.
- It should be easy to flush the deposited silt.
- The basin should have a sufficient volume to store the settled particles until they are flushed.
- It should be possible to lead the discharge and sediments flushed from the basin safely into the river or a nearby gully without causing erosion or damage to other structures.
- Sharp bends should be avoided just before or within the basin since they cause turbulent flows which prevent the settling of particles.

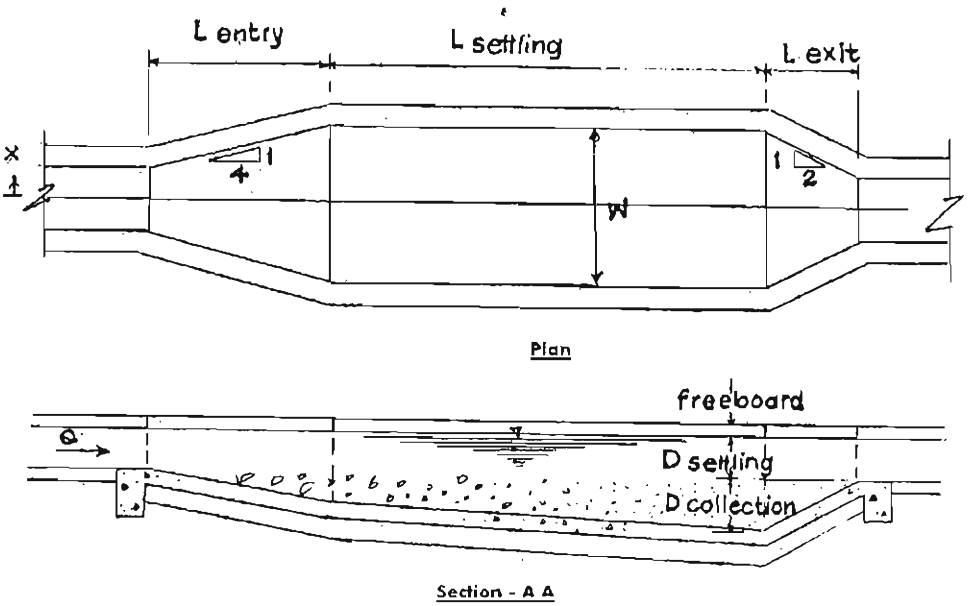

Figure 5.5: A Settling Basin

To reduce costs, one settling basin should be combined with the forebay, if possible.

5.4.1 Design of Settling Basins

The design of the settling basin is based on the following assumptions.

- Only particles with a diameter of 0.3mm or more are settled.
- A reasonable frequency for emptying the sediment from the basin during the monsoon period is twice a day (i.e., every 12 hours). It may not be convenient to empty the basin more often since this will interrupt power production. Similarly, if the flushing interval is increased a larger storage capacity will be required. Since the silt load in the river is less during the dry season, the emptying frequency could then be as low as twice a week.
- The sediments generally have a density of about 2,600kg/m^3 when dry. However, when submerged, they occupy more space and therefore the density decreases. This is measured in terms of the packing factor. The packing factor for submerged sediments is about 50 per cent, i.e., the density of sediment decreases by half when submerged.
- The concentration of suspended particles in the river flow varies seasonally and also depends on factors such as geology and vegetation cover of the catchment area. Therefore, it may be difficult to obtain data on sediment concentration. In this case, it

is reasonable to design the settling basin for 5kg/m^3 of sediment concentration for monsoon flows.

The design procedure is as follows.

1. Choose a suitable basin width, W, two to five times the width of the headrace canal, depending upon the available width at the site (the larger the better).
2. Calculate the settling length ($L_{settling}$) using the following equation.

$$L_{settling} = \frac{2Q}{W \times V_{vertical}} \qquad (12)$$

where, Q is the design flow in m^3/s,

$V_{vertical}$ = is the fall velocity, taken as 0.03 m/s, the value for 0.3mm particles.

Normally, the length of the settling basin should be four to 10 times the width.

3. Calculate the expected silt load, S_{load}, in the basin using the following equation.

$$S_{load} = Q \times T \times C \qquad (13)$$

where,

S_{load} = silt load in kg stored in the basin
Q = discharge in m^3/s
T = silt emptying frequency in seconds. Use 12 hours = 12 x 60 x 60 = 43,200 seconds
C = silt concentration of the incoming flow in kg/m^3, use 0.5kg/m^3 in the absence of actual silt concentration data.

4. Now calculate the volume of the silt load using the following equation:

$$O_{silt} = \frac{S_{load}}{S_{density} \times P_{factor}} \qquad (14)$$

where,

VO_{silt} = volume of silt stored in the basin in m^3.
$S_{density}$ = density of silt, use the value 2,600kg/m^3 unless other reliable data are available
P_{factor} = packing factor of sediments submerged in water = 0.5 (50%).

5. The settling zone should have the capacity to store the calculated value of VO_{silt}. This storage space is achieved by increasing the depth of the basin for the area calculated earlier.

Calculate the average collection depth required, $D_{collection}$

$$D_{collection} = \frac{VO_{silt}}{L_{settling} \times W} \qquad (15)$$

A tapered entry ensures that the incoming flow is evenly distributed in the basin (Figure 5.5). The entry length should have a slope of 1:4. The exit length can be shorter, with a slope of up to 1:2. Note that no exit length is required if the settling basin is combined with the forebay.

5.5 The Forebay

Figure 5.6 shows a section through a typical forebay. The function of the forebay is to provide adequate submergence for the penstock mouth so that the transition from an open channel to pressure flow in a pipe can occur smoothly. If an earthen canal is constructed between the settling basin and the forebay, unusual high velocity in the canal (such as during the monsoon) can cause erosion and carry sediments to the forebay. In such cases the forebay should also be designed to serve as a secondary settling basin. However, if the headrace upstream of the forebay consists of HDPE pipe or a cement masonry canal, and the settling basin is functioning well, there may be no need for a second settling.

The position of the submergence head (depth of water above the crown of the penstock pipe) is shown in Figure 5.6. If the head is too small, the pipe will draw in air and the flow in the penstock will fluctuate. The minimum submergence head required for the penstock pipe can be calculated as follows.

$$h_{submergence} = \frac{1.5V^2}{2g} \qquad (16)$$

where, V is the velocity in the penstock.

The minimum size of the forebay should be such that a person can get in and be able to clean it. Even if in general no sediment load is expected in the forebay, it may nevertheless occur, e.g., when the settling basin fills up quickly during the monsoon or rainwater enters the canal. If a person can get into the forebay and clean it occasionally, at least during the annual maintenance period, this will not be a problem. This is another reason for providing a gap between the bottom of the penstock pipe and the floor of the forebay as shown in Figure 5.6.

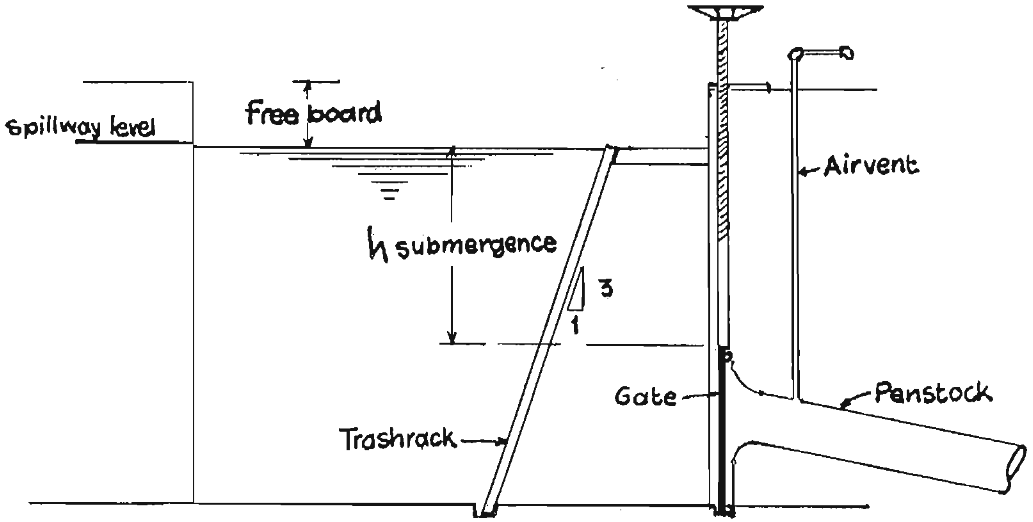

Figure 5.6: Submergence Head for Forebay

Incorporating a gate at the entrance of the penstock will make maintenance work on the turbine easier. The gate can be closed and the penstock emptied so that work can be carried out on the turbine. Rapid closure of the gate, however, could create negative pressure (i.e., a vacuum) inside the pipe and even cause it to collapse. Therefore an air vent should be placed as shown in Figure 5.6 to prevent such a situation. Air can then be drawn from the air vent pipe into the penstock.

The trashrack at the forebay should be placed at a slope of 1:3 both for efficient hydraulic performance and ease of cleaning (by raking, for example). The spacing between the trashrack bars should be about half the nozzle diameter for Pelton turbines and half the spacing between blades for crossflow turbines. This prevents the turbines from being obstructed by sediments and minimises the chances of surge.

5.6 The Penstock

Selecting the size of the penstock pipe requires selecting both the diameter and the wall thickness so that headloss is minimal and the pipe is strong enough to withstand any high pressure (surge) resulting from sudden blockage of the flow. The length of the pipe can be determined from the survey data.

Mild steel and HDPE pipes are the most common materials used for the penstock in MHP schemes. HDPE pipes are usually economical for low heads and flows and are easy to join and repair. They are also flexible enough to accommodate small angle bends or radial expansions resulting from pressure surges, and they are light. The disadvantage is that these pipes can degrade if exposed to ultra-violet rays (sunlight) and temperature variations and hence need to be buried.

5.6.1 Design of the Penstock Pipe

- The pipe diameter

  The following equation should be used to calculate the pipe diameter if the penstock length is less than 100m.

$$D = 41 \times Q^{0.38} \qquad (17)$$

where, D is the inside pipe diameter in mm and Q is the design flow in l/s.

If the penstock pipe is longer than 100m, a detailed analysis is required. Such an analysis is beyond the scope of this manual and other texts should be referred to.

- Pipe thickness

  The thickness of the pipe depends on the pipe diameter, the material, and the type of turbine selected. The surge effect is different for different types of turbine and hence the pipe thickness can differ even when the design flow, static head, and pipe materials are similar.

  The calculation of the mimimum wall thickness of the penstock for Pelton and crossflow turbines is as follows.

  Use the following method to calculate the surge head.

  1. Calculate the pressure wave velocity 'a' using

$$a = \frac{1400}{\sqrt{1 + \left(\frac{2.1 \times 10^9 \times d}{E \times t}\right)}} \qquad (18)$$

where,

the value of Young's Modulus (E) for mild steel is 210×10^9 N/m² and for HDPE is 0.2 to 0.8×10^9 N/m²

d is the pipe diameter in m

t is the wall thickness in m

2. Calculate velocity v in the penstock; $V = \frac{4Q}{\lambda d^2}$ (19)

3. Calculate the surge head (h_{surge}) from

$$h_{surge} = \frac{av}{g} \times \frac{1}{n} \quad (20)$$

where,

n is the number of nozzles in the turbine(s)

Note that in a Pelton turbine, which may have more than one nozzle, it is highly unlikely for more than one nozzle to be blocked (by silt/stones) simultaneously. Therefore, the surge head is also divided by the number of nozzles (n). For single jet Pelton and cross flow turbines n = 1.

4. Now calculate the total head

$$h_{total} = h_{grass} + h_{surge} \quad (21)$$

5. If the pipe is made of mild steel, it will be subject to corrosion and welding or rolling defects. Thus the effective thickness, $t_{effective}$, will be less than the original thickness, t. For mild steel, assume an initial thickness, t and calculate $t_{effective}$ using the following guidelines.

   a) If the pipes are joined by welding divide the initial thickness by 1.1.
   b) If the pipe is prepared by rolling flat sheets, divide the initial thickness by 1.2.
   c) Since mild steel pipe is subject to corrosion, subtract one mm for every 10 years of plant life or part thereof.

For example, the effective thickness of a four mm thick flat rolled and welded mild steel pipe designed for a 10-year life is

$$t_{effective} = \frac{4}{1.1 \times 1.2} - 1 = 2.03\,mm$$

Note that this does not apply to HDPE pipes where the effective thickness is the same as the original thickness of the pipe.

6. Calculate the safety factor (SF) using the following equation.

$$SF = \frac{t_{effective} \times S}{5 \times h_{total} \times 10^3 \times d} \qquad (22)$$

where,

S is the ultimate tensile strength of the pipe material in N/m^2. For mild steel S is usually taken as 350×10^6 N/m^2. For HDPE the value is between 6 and 9×10^6 N/m^2.

d is the internal diameter of the pipe in m

7. If SF <3.5, reject this penstock option and repeat the calculation for a greater thickness.

In a crossflow turbine, instantaneous blockage of water is not possible since there is no obstruction at the end of the manifold, unless an additional valve is also provided. Therefore surge pressure can only develop if the runner valve is closed rapidly.

A simple procedure to determine the thickness of the penstock pipe for a crossflow turbine is to add 20 per cent to the gross head to allow for surge pressure, i.e., $h_{total} = 1.2 \times h_{gross}$. This results in a more conservative value for the surge head, but its contribution to the increase in the thickness will anyway be insignificant since crossflow turbines are only used for low head schemes.

Once the total head is known, the procedure is the same as for the Pelton turbines (i.e., start calculations from step 4 for the Pelton turbine).

5.7 Anchor Blocks

An anchor block is a mass of concrete fixed into the ground that holds the penstock to restrain the pipe movement in all directions. It is constructed of plum concrete which is 1:3:6 (1 part cement, 3 parts sand, 6 parts aggregate) with about 40 per cent boulders placed evenly around the block as shown in Figure 5.7. The boulders add weight to the block and therefore increase stability while reducing the volume of cement required. For MHP schemes with a gross head of less than 60m and a design flow of less than 200 l/s, The following guidelines can be used to determine the size of an anchor block.

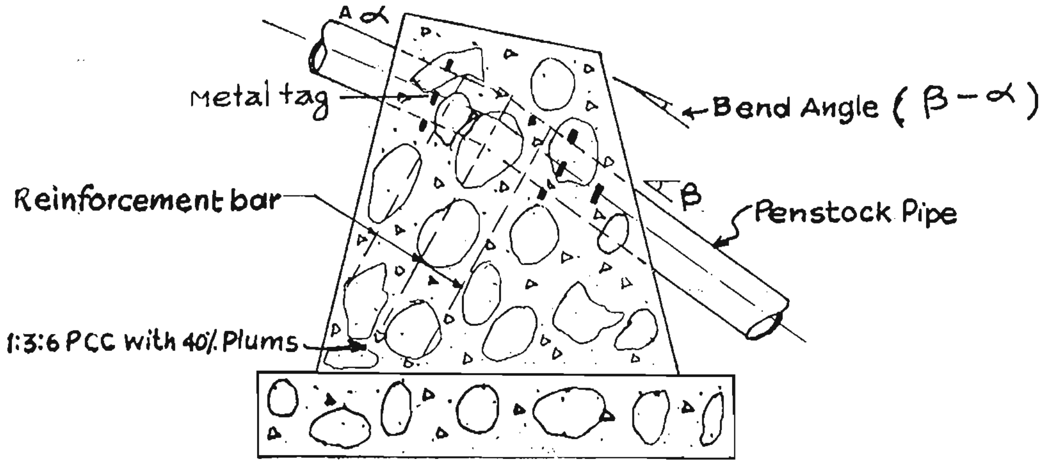

Figure 5.7 : An Anchor Block

- For straight sections, locate one anchor block every 30m along the length of the penstock. Use one m^3 of plum concrete for a pipe diameter of 300mm. If the pipe diameter is more or less, say 200mm, then adjust the amount proportionately. $(200/300) \times 1\ m^3 = 0.67\ m^3$.
- Always provide an anchor block at the bends in the penstock keeping a maximum distance of 30m between two blocks. For bends less than 45°, use double the concrete volume required for a straight section. For example, if the pipe diameter is 350mm and the bend is 20°, then use $(350/300) \times 2\ m^3 = 2.33\ m^3$ of concrete for the anchor block. If the bend angle is larger than 45°, then the required volume of concrete is three times that for a straight section.

For larger MHP schemes, more detailed calculations are required that are beyond the scope of this publication.

5.8 Support Piers

Support piers restrain the vertical forces of the penstock resulting from the weight of the pipe and water. However, they allow axial movement resulting from thermal expansion or contraction (Figure 5.8).

Support piers are generally constructed out of stone masonry in 1:6 cement–mortar. A metal plate should be placed on the support pier where the penstock pipe rests to minimise frictional effects and increase the useful life of the pipe. A further semi-circular strip of metal is usually laid around the penstock and bolted to the pier to provide some control over unwanted movements.

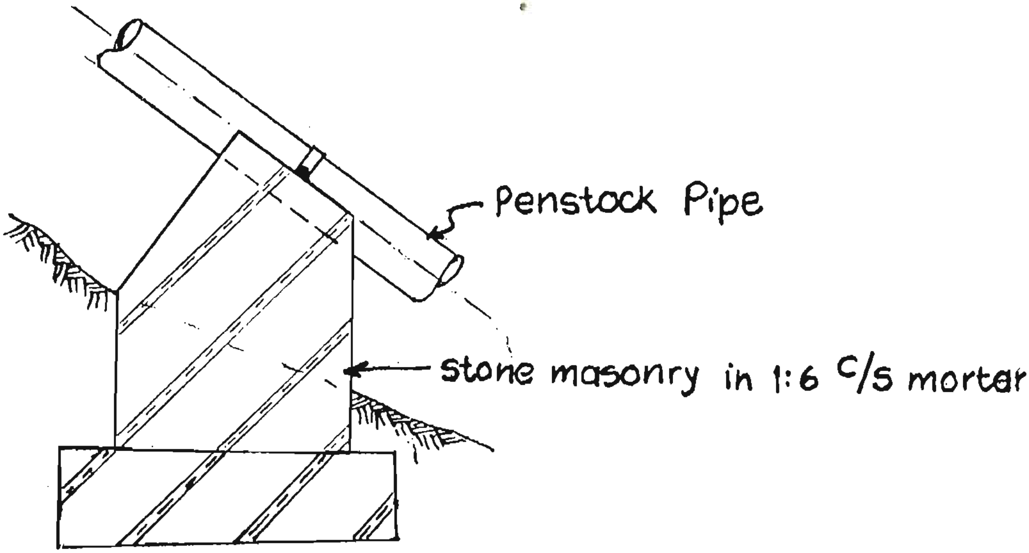

Figure 5.8: A Support Pier

Table 5.3 should be used to determine the spacing of support piers for mild steel pipes that are welded or connected by flanges according to British Standard specifications (minimum flange thickness = 16mm). (Support piers are not required for buried pipes.) Note that in Table 5.3 the support pier spacing is the horizontal (plan) length and not the sloping length of the pipe. For flanged mild steel pipes that do not meet British Standards, one support per individual pipe length should be used with the pier placed in the middle.

For small MHP schemes (h_{gross} <60m and Q <200 l/s) Figure 5.8 can be used as a guide for the shape of the support pier if the penstock pipe is less than one metre above the

Table 5.3: Spacing of support piers

Pipe diameter (mm) =>	100	200	300	400	500
Pipe thickness (mm)	Support pier spacing (horizontal), metres				
2	2	2	2.5	3	3
4	3	3	3	4	4
6	4	4.5	5	6	6

ground. The base of the pier(s) should be at least 1m x 1m. The size of the top piece on which the penstock is supported should be at least 0.5m along the direction of the penstock and about one metre width at right angles to the penstock. The uphill wall surface should be perpendicular to the penstock pipe. The required depth of foundation depends on the condition of the soil, but it should be at least 300mm.

For larger schemes, or if the penstock pipe is more than one metre above the ground, a more detailed calculation is required which is beyond the scope of this publication.

5.9 Expansion Joints

Above ground penstock pipes are subjected to expansion or contraction in length as a result of changes in the ambient temperature. The change depends on the change in temperature and the type of material used. Table 5.4 can be used to determine the changes in lengths for mild steel pipes of various lengths in different temperature ranges. The expansion lengths can be extrapolated to give intermediate values. Note that the maximum expected temperature variation should be used for the calculation (such as between when the pipe is empty during a mid-summer afternoon, and the lowest winter temperature).

Table 5.4: Thermal length change of mild steel pipe (mm)

Maximum temperature variation °C	Length between anchor blocks in m				
	10	20	30	40	50
Change in length (mm)					
25	3	6	9	12	15
35	4	8	13	17	21
45	5	11	16	22	27

A sliding type of expansion joint such as that shown in Figure 5.9 is commonly used in MHP schemes. It can be placed between two consecutive pipe lengths and can either be welded or bolted to the pipes. The stay rings are tightened to compress the packing and prevent leaking. Jute, rubber, or a similar type of fibre is used for packing. The penstock pipe expansion or contraction is accommodated by the gap in the expansion joint. Therefore, the gap in the expansion joint should be about twice the expansion length derived from Table 5.4.

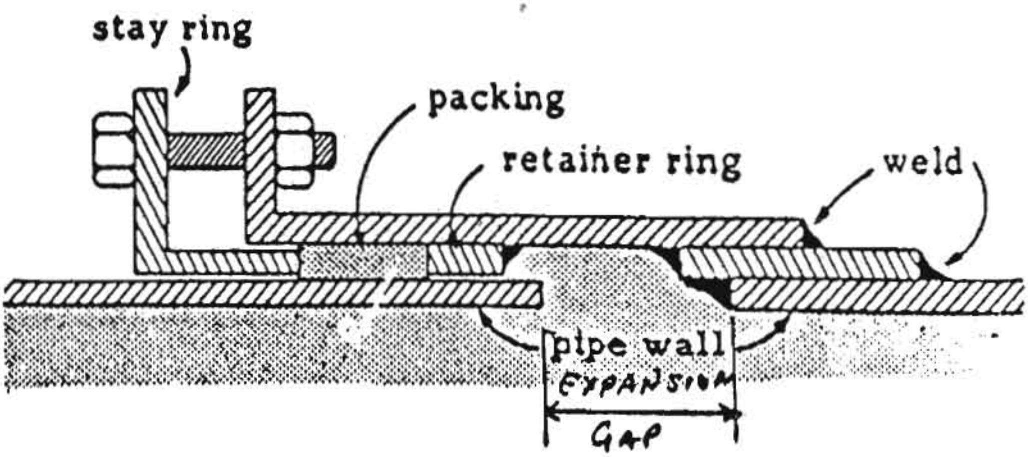

Figure 5.9: Sliding Expansion Joint

5.10 The Powerhouse

The main function of the powerhouse is to protect the electro-mechanical units from rain and other weather effects. Costs can be brought down if the construction is similar to that of other houses in the area. The powerhouse walls can be built of stone masonry in mud mortar with cement pointing on the surfaces. The roof can be covered with corrugated, galvanised iron (CGI) sheets. The powerhouse should be big enough that all the electro-mechanical equipment can fit in and be easily accessible for operation and repair work. If agro-processing units are also installed inside the powerhouse, additional space should be provided so that it is not overcrowded when people are working or delivering grain and so on. When planning the size of the powerhouse, all electro-mechanical units should be drawn to scale and laid out so that they fit in the plan area of the drawing.

The machine foundations should be constructed out of reinforced concrete so that all the loads, including the dynamic forces of the generator and the turbine, are properly supported and the alignment does not change over the years.

5.11 Turbine Type, Size and Accessories

Crossflow and Pelton turbines are the two most commonly used turbines in MHP schemes. Typical examples are illustrated in Figure 5.10. The type and size of the turbine to be selected for a particular site depends on the net head ($h_{net} = h_{gross}$ - losses) and the design flow.

Generally, crossflow turbines are used for high flow and low head schemes and are suitable for heads of less than 50m. Conversely, Pelton turbines are used for high head (50m

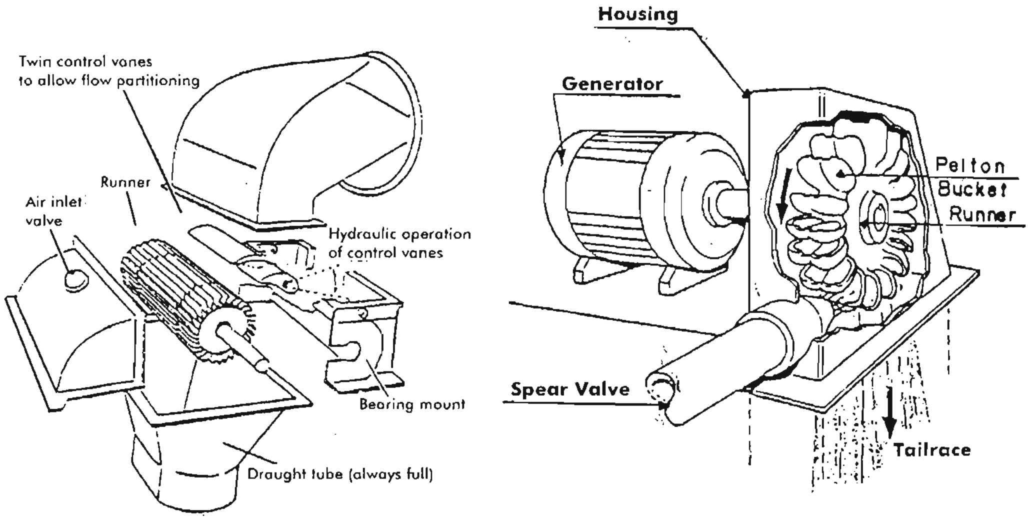

Figure 5.10: Crossflow and Pelton Turbines

and more), low flow schemes and are the only alternative if the net head exceeds 100m. A single jet Pelton turbine is usually less expensive than a multi jet one. However, a single jet may require a large runner and the penstock pipe may also have to be thicker to allow for the greater possible surge as discussed earlier. The Nomogram shown in Figure 5.11 can be used to select an appropriate turbine for the site conditions. The procedure is as follows.

- First draw a straight line connecting the turbine shaft power and the net head. Note that the turbine shaft power = 1.1 x power output. This allows for about 10 per cent loss in the generator.
- Now select a suitable turbine shaft speed (RPM) and extend a line from this point so that it is perpendicular to the previous line (turbine shaft power to head). This second line will point to either a single jet Pelton, multi-jet Pelton, or a crossflow turbine. If the line ends at the overlap region between turbine types, then both types are feasible. If the line ends beyond the crossflow range, try again with a different RPM.

Note that the suitable range of shaft speeds for a 1,500 RPM generator turbine lie between 400 RPM and 1,500 RPM. If possible for electrification schemes, the turbine shaft speed should be set at 1,500 RPM, which allows for direct coupling between the generator and the turbine since most MHP generators are designed to operate at 1,500 RPM. However, this is only possible with fast running Pelton turbines. For crossflow turbines, the usual rated speed is between 400 and 1,000 RPM. In such cases, a belt drive with the requisite speed ratio will be required. Once the turbine type and the RPM are known for a fixed head and flow, their price can be obtained from the manufacturer.

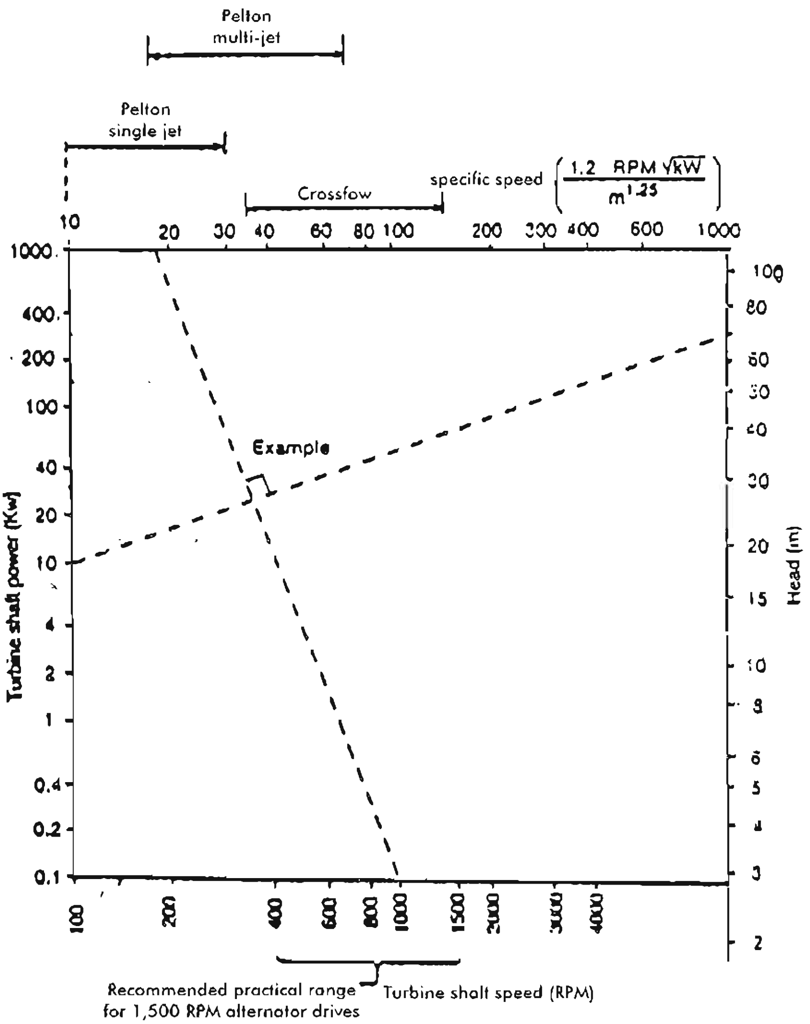

Figure 5.11: Turbine Selection Nomogram
(Source Harvey et al. 1993)

5.12 Generator Type and Size

Induction or synchronous type generators are mostly used in MHP schemes. Induction generators are inexpensive and appropriate for very small schemes of up to 10kW. For larger schemes, the induction generator load controller may not be very reliable. Table

5.5 below should be used to select the type of generator for the power output required. Once the generator type has been determined, the required nominal generator output power in kVA should be calculated using the following equation.

$$Generator\ no\min al\ range\ kVA = \frac{power\ output\ in\ kW}{(A \times B \times C \times D)} \qquad (23)$$

Note that the power output is the available power in kW calculated using the power equation. The factors A, B, C, and D can be found from Table 5.6.

Table 5.5: Selection of Generator Type

Type of scheme/generator	Very small schemes up to 10kW	Small schemes, 10kW to 20kW	Medium to large schemes, over 20kW
Type of generator	Induction, single or 3 phase, 2nd choice: synchronous	Synchronous, 3 phase only 2nd choice: induction, 3 phase only	Synchronous, 3 phase only

Table 5.6: Factors Affecting Generator Rating

	Ambient temperature in °C	20	25	30	35	40	45	50	55
A	Factor	1.10	1.08	1.06	1.03	1.00	0.96	0.92	0.88
	Altitude in m	1000	1500	2000	2500	3000	3500	4000	4500
B	Factor	1.00	0.96	0.93	0.90	0.86	0.83	0.80	0.77
C	ELC correction factor	Without electronic load controller (ELC)						1.00	
		With electronic load controller						0.83	
D	Power factor	When only light bulbs are used							
		When tube lights and other appliances are used							

Example

Determine the generator rating for an MHP scheme designed for a power output of 20kW. The site is located at 1,700m above sea level and the ambient temperature inside the powerhouse during midsummer afternoons can be up to 35°C. An ELC will be used in the plant and the villagers also plan to use tube lights.

Calculation

A = 1.03 for 35°C

$B = 0.96 - \left(\frac{0.03}{500} \times 200\right) = 0.95$ for 1,700m altitude.

C = 0.83 with ELC

D = 0.80 when tube lights are used

Required generator rating (kVA) = Power output inkW/(A x B x C x D)

= 20/(1.03 x 0.95 x 0.83 x 0.80)

= 31kVA

Note that if the calculated rating (kVA) of the generator is not available in the market, the next higher one should be selected. In the above example, a 35kVA generator may have to be purchased. However, since calculations like the above are not exact, a 30kW generator may also be considered; especially if the cost differences are significant.

5.13 Transmission Lines

The power generated in the plant needs to be distributed to the consumers via the main transmission line and secondary distribution lines.

- Selection of underground or overhead lines
  Transmission lines can either be buried (underground) or installed overhead on poles. Overhead lines are more common since they are less expensive and easier to install than underground lines. Overhead lines are also easy to repair and maintain. However, when houses are more densely located or heavy snowfall is expected during winter, underground transmission lines may be a viable alternative; if properly installed and protected, they should need very little maintenance.

- Selection of high voltage or low voltage
  If the powerhouse is far from the villages (load centres), high voltage transmission may be required together with step-up and step-down transformers to reduce line losses. However, this is an expensive option. In such a case the transmission voltage is stepped up to 11kV via the transformer at the powerhouse and stepped down to

400V at the village with another transformer. Then a low voltage (400V for three phase and 220V for single phase) distribution line is used. The general rule is to use a low voltage transmission line if the product of the power produced and the length of transmission line is less than 54kW.km (i.e., power output x transmission length < 54kW.km). If the installed capacity exceeds 10kW, a three phase transmission line should be used.

- Sizing of overhead transmission cables

  Aluminum conductor steel reinforced (ACSR) cables are generally used for overhead transmission lines in MHP schemes. These are available in various sizes and are named after certain animals (the larger the cable diameter the larger the type of animal whose name is used). The parameters required to size the commonly used ACSR cables are given below in Table 5.7.

Table 5.7: ACSR Transmission Line Parameters

Name	Equivalent aluminium area (mm^2)	Current rating in still air (amp)	Resistance (Ω/km)
Squirrel	20.7	76	1.374
Gopher	25.9	85	1.098
Weasel	31.2	95	0.9116
Rabbit	52.2	135	0.5449
Dog	103.6	205	0.2745

PVC (polyvinyl chloride) insulated armoured or unarmoured cables are used for underground transmission lines. These cables have insulated cores placed inside a PVC pipe for protection against moisture or other adverse effects when buried. The armoured PVC cables have an additional strip of steel strands inside the PVC pipe for protection against dynamic loads resulting, for example, from vehicular movement. More detailed design of underground cables is beyond the scope of this manual.

The following procedure is used to calculate the transmission line conductors for overhead lines.

- Determine the power factor (D in Table 5.6)
- Calculate the current, I, to be transmitted

Single phase; $$I = \frac{P}{V \times Power\ factor\ (D)} \qquad (24)$$

Three phase; $$I = \frac{P}{\sqrt{3} \times V \times Power\ fctor\ (D)} \qquad (25)$$

Where I = Current in amperes
P = Power in watts
V = Voltage, 220 V for single phase and 400 V for three phase

- Select a cable size from Table 5.7 so that the current rating is higher than the calculated current (I).
- Estimate the transmission line length (L) in km from the survey data.
- Determine the resistance R in W/km of the selected cable from Table 5.7 or from the manufacturer's catalogue. Note that the values may vary slightly between manufacturers.
- Calculate the voltage drop (V_{drop}) as follows.

Three phase: $$_{drop} = I \times L \times R \qquad (26)$$

Single phase: $$_{drop} = 2 \times I \times L \times R \qquad (27)$$

- If the calculated V_{drop} is higher than 13 per cent (preferably 10%) select the next higher size conductor and repeat the calculations.
- Calculate the loss of power (P_{loss}) in the transmission line as follows.

$$P_{loss} = I^2 \times L \times R \qquad (28)$$

This equation is used for determining the maximum power that can be used by the consumers (installed capacity - P_{loss}) and for which they can be billed.

Example

Determine the transmission line wire size for an MHP scheme that has an installed capacity of 20kW. The survey data show that the transmission line is 1.5 km long and the villagers may use bulbs, tube lights, and other appliances.

Calculation

Power out x transmission length = 20kW x 1.5 km = 30kW.km (< 54kW.km). Therefore, low voltage transmission is possible.

Since the installed capacity is higher than 10kW, a three-phase generation and transmission system will be required.
Power factor = 0.8

Transmitted current: $$I = \frac{P}{\sqrt{3 \times V \times Power\ factor\ (D)}} = \frac{20{,}000}{\sqrt{3 \times 400 \times 0.8}}$$

$$= 36\ A$$

For I = 36 A, choose SQUIRREL (Rated I = 76 A) which is the smallest size ACSR overhead transmission line.

From Table 5.7, R = 1.374 W /km

$$V_{drop} = I \times L \times R$$
$$= 36 \times 1.5 \times 1.374$$
$$= 74\ V$$

$\%\ V_{drop}$ = (74/400) x 100% = 18.5% > 13%, NOT ACCEPTABLE

Try WEASEL, R = 0.9116 W/km

$$V_{drop} = I \times L \times R$$
$$= 36 \times 1.5 \times 0.9116$$
$$= 49\ V$$

$\%\ V_{drop}$ = (49/400) x 100% = 12.3% < 13%, ACCEPTABLE

Check the power loss in the transmission line:

$$P_{loss} = I^2 \times L \times R$$
$$= 36^2 \times 1.5 \times 0.9116$$
$$= 1772\ W = 1.77kW$$

Therefore only 20 - 1.77 = 18.23kW are available for the consumers from this scheme.