Collecting Data on the Site 4

Before visiting the site and in addition to having a map, it is important to obtain the minimum equipment needed for the most accurate evaluations, as well as information on the access and other characteristics of the area. It is recommended that the visit should take place during the month when the river is at its lowest stage.

4.1 Measuring Heads

There are various ways of measuring heads, the most popular being the altimeter and the level hose, recommended for small heads. A carpenter's level is appropriate for intermediate slopes and short lengths; the inclinometer and tape measure are quick and advisable for all slopes and lengths; the topographic level is quick but the most expensive. Of all these, the topographic level is the most accurate. The accuracy of the inclinometer can vary up to 5%, depending on the skills and ability of the operator; its advantage is that it takes less time, is inexpensive and quite acceptable for a pre-feasibility study.

The inclinometer method will be explained, as apart from obtaining the head with the data collected in the field, the ground profile can be drawn so that the penstock can be designed.

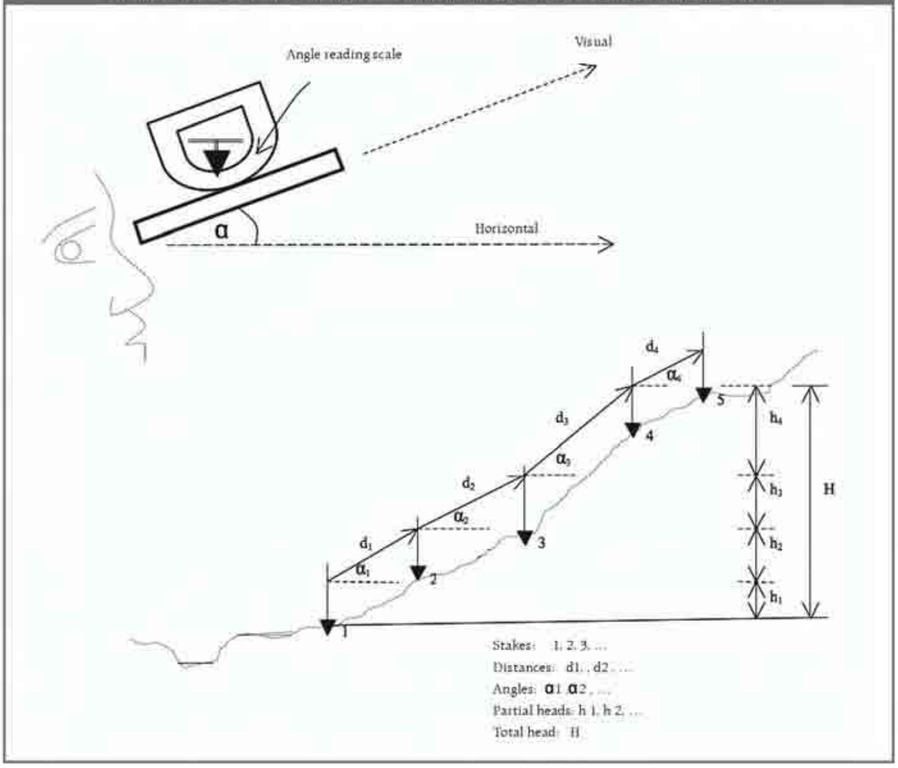
Inclinometer method

The following is required:

- 1 inclinometer
- 1 30 to 50 m tape measure
- 1 calculator
- 1 notebook
- 1 pen
- 1 chopper
- Wooden stakes
- At least 3 people, including the operator

The method consists of measuring the vertical angles with the inclinometer and the distances with the tape measure to obtain partial heads. The total head is equivalent to the sum of all partial heads. The measured angle is formed by the horizontal and the visual in the direction of the measurement. The reading of the angle is obtained in the angular scale in sexagesimal degrees on the inclinometer. Begin from the powerhouse towards the head tank or vice-versa. For recording data, it is advisable to have the following table prepared and make a note of each section in the ground profile.

FIGURE 24. MEASURING THE HEAD WITH AN INCLINOMETER AND A TAPE MEASURE



For a definitive study, the topographic level or theodolite method should be employed, in order to obtain detailed level curves for the exact location of the head tank, the powerhouse, the tailrace channel and the complete design of the penstock.

Numerical example:

Stakes	Distance d (m)	Ángle α	Partial head (m) h = d x Sen	Accumulated head (m)
2	19.50	38	12.05	12.05
2-3	25.75	36.5	15.31	27.36
3-4	20.44	42.5	13.51	40.87
4-5	15.80	29	7.65	**48.52**

The total head is equivalent to the sum of the partial heads or the last accumulated head: 48.52 m.

4.2 Measuring the Flow

The volume of the river is measured upstream from the site of the intake weir, preferably during the dry season. There are various methods of measuring the flow, such as the current meter, the salt solution method, the spillway, the floater method, the container, etc. The application of one method or another depends on the equipment available, the ability and experience of the people involved and the technical conditions recommended for each method.

Container method. This is very useful for small flows of less than 30 or 40 L/s. It involves creating a jet in the course of the river. The entire flow must go though the jet into a container with a specific capacity in litres. The time it takes to fill the container is registered with the stop watch. This should be repeated at least three times. The measured flow will be the capacity of the container in litres divided by the average time in seconds. To create the jet, advantage can be taken of waterfalls in the river or watercourse, if it has sharp slopes. Otherwise, another method must be applied.

The following equipment is required:

- 1 container with a specific capacity in litres
- 1 stop watch
- 1 plastic sheet 1 × 2 m
- Basic tools (pick and shovel)

Floater method. This consists of measuring the superficial speed of the water in m/s in a straight and uniform section of the river, and determining the area of the cross-section in m^2. The flow is equivalent to the result of the section area times the superficial speed and a speed correction coefficient. The coefficient is generally 0,60.

The following equipment is required:

- 1 pick
- 1 shovel
- 6 stakes
- 30 m of rope
- 1 graduated wood or metal 1.20 m ruler
- 1 20 m tape measure
- 1 notebook
- 1 floater (empty bottle)
- 1 stop watch
- 1 pair of rubber boots
- 1 calculator
- At least three people
- 1 chopper

Procedure:

a) Select a straight and uniform section of the river.

b) Clear the river bed of any brush and removable stones.

c) Take a distance L and mark it with stakes along the river bank.

d) Determine the section of the river and the wet area: measure the width of the river at the level of the water surface and several depths at equal distances. Then take an equivalent area of a familiar shape, like the one shown in the following figure, to make it easier to use the area formula.

e) Measure the superficial speed of the river water in the considered section and calculate the time it takes the floater to travel along the selected distance (at least three times).

f) The measured flow is calculated with the following formula:

$$Q = A \times V \times K$$

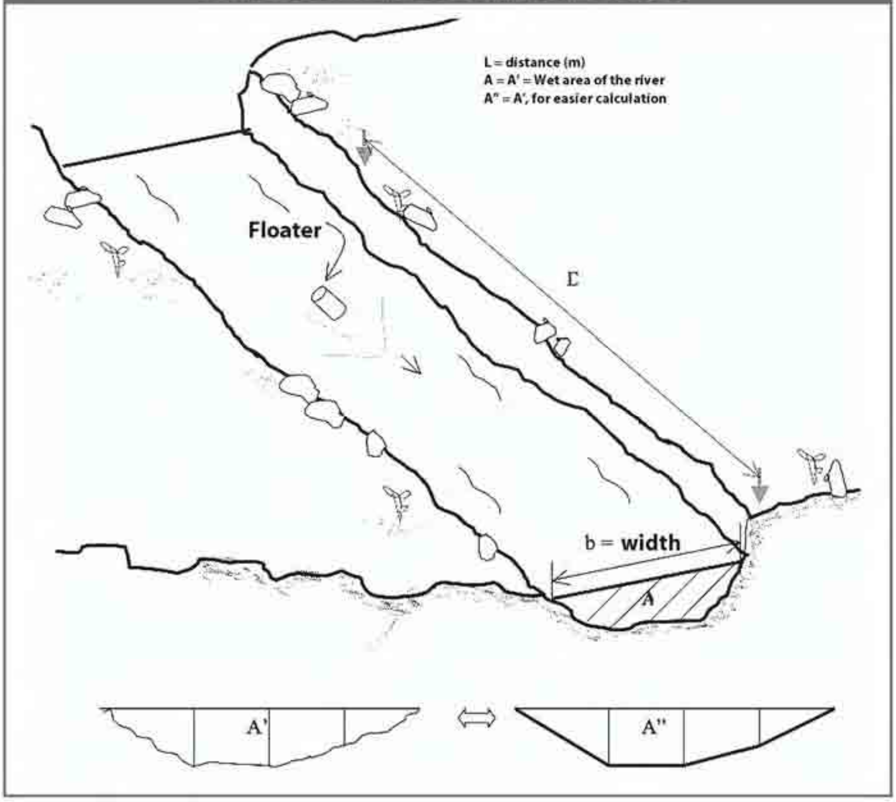
Where:

A = wet area of the river

V = superficial speed of the river water

K = speed correction factor = 0.60

FIGURE 25. FLOATER METHOD FOR MEASURING THE FLOW



The calculation process is explained in the following figure.

Numerical example:

D = 10 m

Travelling times to distance L:

T_1 = 15.44 s

T_2 = 16.10 s

T_3 = 15.92 s

Average time T = 15.82 s

Superficial speed = 0.63 m/s

b = 6.80 m

Depths:

d_1 = 0.00 m

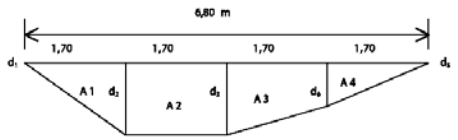
d_2 = 0.5 m

d_3 = 0.26 m

d_4 = 0.18 m

d_5 = 0.00 m

Calculation of the area:



A1 = (1.70 x 0.25)/2 = 0.21 m^2

A2 = ((0.25 + 0.26)/2)1.70 = 0.43 m^2

A3 = ((0.26 + 0.18)/2)1.70 = 0.37 m^2

A4 = (0.18 x 1.70)/2 = 0.15 m^2

A = 1.16 m^2

Flow Q = 1.6 m^2 x 0.63 m/s x 0.80 = 0.438 m^3/s

This flow corresponds to the river during the low stage, measured on the site.

Estimating the maximum flow. Other parameters must be evaluated in the same place, such as the slope of the river and the depth of the water surface and the level it reaches during the rainy season. This can be obtained by observing the track left by the water on both river banks or by talking to people who live in and are familiar with the area. Among other data, a note should be made of the quality of the soil in terms of its stability for designing the intake weir.

Follow the procedure below:

a) Calculate the river slope with an eclimiter and a measuring tape.

b) Calculate the area of the river section based on the new water surface and the recorded depths.

c) Apply the Manning formula to find out the speed, bearing in mind the parameters involved: area, perimeter, hydraulic radius (Rh), river slope (s) and the rugosity coefficient (n) , in accordance with the material comprising the river.

$$V = \frac{1}{n} \; R_h^{\frac{2}{3}} \; s^{\frac{1}{2}}$$

d) Apply the flow formula as in the previous case.

$$Q = A \times V \times K$$

4.3 Calculating the Power Generated (Installed Capacity)

The power generated is calculated with the actual head and flow data obtained in the field. To this end, preliminary sizing should have been done, to give an idea of the type of turbine to be used. Also, data on the demand should be available for comparison purposes. The power capacity can be obtained with the following formula:

$$P_E = \eta . \eta_{TR} . \eta_G . \gamma . QH$$

Where:

PE	=	Electric power in the generator terminals
P	=	Power in the turbine hub, kW.
Q	=	Turbine flow, m^3/s
H	=	Net head in m
γ	=	Specific weight of the water, 1.000 kg/m^3
η	=	Efficiency of the turbine, non-dimensional
η_{TR}	=	Efficiency of the transmission, non-dimensional
η_G	=	Efficiency of the generator, non-dimensional

4.4 On-Site Survey for the Distribution Line

The survey must consist of a study of the loads and the estimates for establishing the electric power demand. The objective is to analyze the consumption and then the anticipated demand in order to find a fair balance between the supply and the demand. It is not the idea to accurately establish what will happen in the future, but to establish a reasonable working theory for calculating the values that will probably be adopted in the area.

4.4.1 Study of Loads

Divide the existing loads by sectors, in accordance with the domestic, commercial, non-industrial and industrial consumption, and the special loads to be pinpointed in the respective map.

4.4.2 Estimating the Electric Power Demand

Estimates should be made for a period of 20 years.

a) Based on data from the census, estimate the number of people in order to obtain the growth rates, which may be obtained from the statistical entity.

Use the following formula for the estimates:

$$P_f = P_o(1+t)^n$$

Where:

P_f = Final population

P_o = Population in the first year

t = Growth rate

n = Number of years from the beginning

b) With these statistics, calculate the average number of people per home and then the number of homes in the area.

c) The number of subscribers is calculated taking into consideration the ratio between the number of subscribers and the number of homes

$$Electrification\ coefficient = \frac{N°\ of\ subscribers}{N°\ of\ houses}$$

In an area where there is an electricity service, the initial electrification coefficient can be determined by dividing the number of current subscribers by the number of homes obtained in the survey. Where there is no electricity service, an electrification coefficient of 0.10 and 0.20 can be estimated, providing it is justified.

d) Special loads. These are determined based on the survey of medical posts, nursery schools, primary and secondary schools, police stations, community centres, municipalities, churches, etc.

e) To determine the domestic consumption, use table 8. It may be higher or lower depending on the consumer habits of users and the actual time the appliances are used.

f) The net load is the sum of the loads obtained in the previous points.

g) The gross load is the sum of the total net load and the losses in the distribution system, estimated at 10% of the total net consumption.

h) The maximum demand, the value of which is determined by dividing the gross consumption by the number of hours of maximum use, is estimated considering a continuous service.

i) The total estimated consumption of the area is obtained by multiplying the number of estimated subscribers for each year times the unit consumption per subscriber.

4.4.3 Methodology for Estimating the Demand

The methodology is based on the estimates of the power consumption and the maximum demand, which is small and medium sized towns are based on establishing a growing functional relationship between the power consumption per domestic subscriber (kWh/subscriber) and the number of subscribers estimated for each year. This ratio takes into consideration that the urban expansion resulting from the population growth rate is closely linked to the development of productive activities that lead to better income levels and, therefore, a per capital increase in the consumption of electricity.

Main factors to be considered:

a) Percentage growth rate
b) Initial electrification coefficient
c) Final electrification coefficient
d) Initial domestic consumption per unit (kWh-month).
e) Consumption growth rate per year
f) Consumption growth rate per year
g) Consumption growth rate per year
h) AP factor in kWh/user-month
i) Percentage consumption for general use
j) Percentage of losses
k) HGEU (Hours of gross energy use)
l) Increase of HGEU

Table 8-1 Form 1

Page N°		Produced by	
Project		Date	

INFORMATION ON EXISTING SYSTEMS

LOCATION:

Department		Province	
District		Area	

TECHNICAL DATA:

	Service		Type of consumption
	Public		Private
	Domestic		Commercial
			Industrial

PRIMARY GENERATOR:

Hydraulic / turbine	
Other	

DATA:

Number plate	
No number plate	

Make		Type	
Year of manufacture		Nominal power	
RPM		m.a.s.l	

State		Good		Fair		Poor

GENERATOR:

Number plate	
No number plate	

Brand		Series		Number	

Year of manufacture		Nominal power	
N.° of phases:		Power factor:	
Nominal tension		Power factor	
RPM		m.a.s.l	

State		Good		Fair		Poor

Table 8-2 Form 2

Page N°		Produced by	
Project		Date	

DEMAND FORECAST

LOCATION:

Department		Province	
District		Locality	
Population		Nº of families	

N° of inhabited homes	Uninhabited	Total
Estimated length of streets	Principal (KM)	

N° of blocks	
Estimated total area of village Has	
N° shops	
Small manufacturing	
Institutions	

	N° of institutions	N° of rooms	N° of hours of use
Municipality			
Public Schools			
Private Schools			
Preschools			
Medical posts			
Community centre			
Courts			
Technical institutes			
Police Station			

UNUSUAL LOADS

TECHNICAL DATA:

	Service		Type of consumption
	Public		Domestic
	Private		Commercial
			Industrial

Preliminary Design 5

5.1 Civil Works Design

5.1.1 Design of the Intake

The intake should be located on a straight stretch of the river. This section should be as uniform as possible, with uniform width and slope, and both banks should be stable. A curved section should not be chosen because the flow rate in the convex part is higher and rises during the rainy season, while the concave part fills with sediment, making it unsuitable for a water intake.

The design of the intake consists of choosing dimensions for each element in accordance with the design flow rate and project characteristics. When the topographical conditions and characteristics of the river are favourable, the type of intake most often used in mini/micro hydropower projects with an intake flow rate of 0,600 m^3/s or less is the mixed barrage.

(1) Retaining walls

The conventional material for the training walls is simple concrete with up to 40% large stone. The design must meet three conditions for it to be stable, as shown in table 9.

TABLE 9. DESIGN OF THE RETAINING WALLS	
STABILITY	**STABILITY CONDITIONS**
1. Overturning	$\frac{\sum Me}{\sum Mv} \geq 2$ $\sum Me$ = Sum of moments of stabilising forces $\sum Mv$ = Sum of moments of forces causing overturning
2. Slippage	$\frac{f}{Et} \geq 2$ $f = \mu N$ f = Friction between foundation soil and the concrete base of the wall μ = Coefficient of friction N = Total reaction of soil at the base of the wall $Et = \frac{1}{2} \times Ka \times \gamma \times H^2$ Et = Active thrust of soil leading to slippage Ka = Thrust coefficient of soil $\frac{Cos\theta - \sqrt{Cos^2\theta - Cos^2\phi}}{Cos\theta + \sqrt{Cos^2\theta - Cos^2\phi}}$ θ = Angle of the soil to the horizontal (slope); in some cases this value is zero ϕ = Internal friction angle of the soil (laboratory tests) γ = Specific mass of soil; may be T/m³ O in kg/m³ H = Total height of wall (in m)
3. Slumping	Consists of verifying: σ soil admissible > σmax > σmin $\sigma mín = \frac{N}{B}\left[1 - \frac{6 \times e}{B}\right]$ $\sigma máx = \frac{N}{B}\left[1 + \frac{6 \times e}{B}\right]$ σmin = minimum force transmitted by the wall and part of the soil over the footing to the foundation soil in kg/cm² σmax = maximum force transmitted by both the wall and soil to the foundation soil in kg/cm² B = Base of the wall in metres e = Eccentricity in metres $e = \frac{\Sigma Mo}{N}$ ΣMo = Sum of moments with respect to the centre of the base, of all the forces involved: stabilising and overturning N = Total reaction of soil on base of wall

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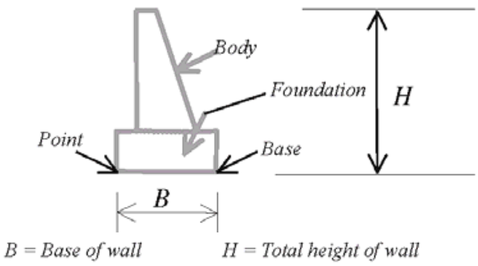
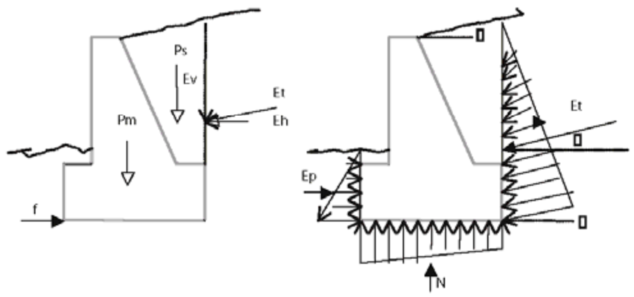
FIGURE 26 DIAGRAM OF FORCES AND PRESSURES ACTING ON THE WALL



Example of cross section

Pm = weight of wall; Ps = weight of soil over back of wall; Et = active thrust of soil on rear of wall; Ep = passive thrust; Eh = horizontal component of soil thrust; f = friction between soil and foundation soil and base; N = upward reaction of soil; θ = angle of soil to horizontal

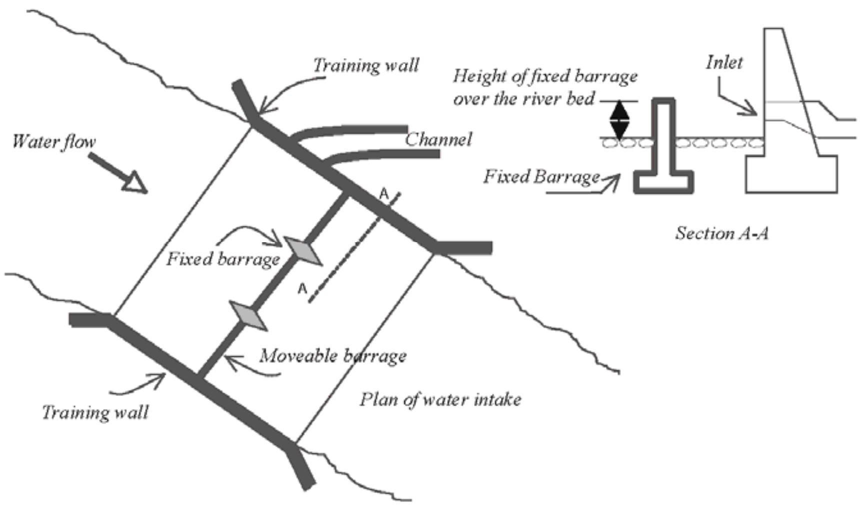


(2) Mixed barrage

The mixed barrage includes a fixed component and a movable component. The fixed component generally consists of reinforced concrete columns or slabs embedded in the river bottom. The moveable part consists of timbers placed in the channels of the fixed barrage. They are moveable to facilitate removal of the material carried down by the river.

Before embarking on the design, we need to know the depth of the river in the dry and rainy seasons. The depth of those rivers that do not dry up varies between 0.20 and 0.70 m. Therefore the height of the barrage may vary from 0.40 to 0.60 m. The barrage structure as a whole lies perpendicular to the river current.

FIGURE 27 BARRAGE PERPENDICULAR TO THE RIVER CURRENT



(3) Water intake

The water intake is located in one of the training walls. The lower sill should be at least 15 cm above the river bed. This measurement depends on the size of stones carried by the river in the rainy season. It should lead to a security grill to prevent stones larger than 3" from entering the channel. The intake is generally rectangular. In the dry season it is partially submerged and in the rainy season it is completely submerged.

The following formula is used to calculate the dimensions of the intake:

$$Q = C \times L \times h^{1.5} \quad (5.a)$$

Where:

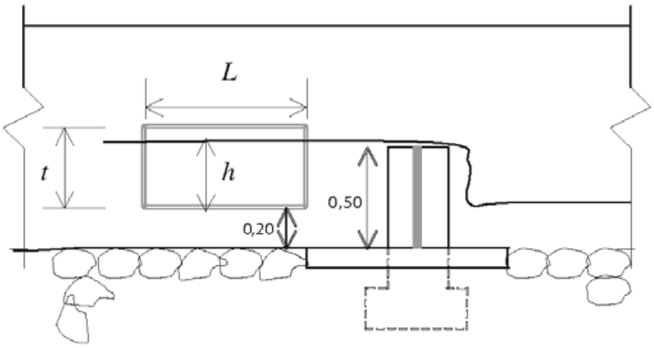
Q = design flow rate

C = discharge coefficient, equal to 1.6

L = length of the intake and

h = height of water entering the intake under normal conditions. See Figure 28.

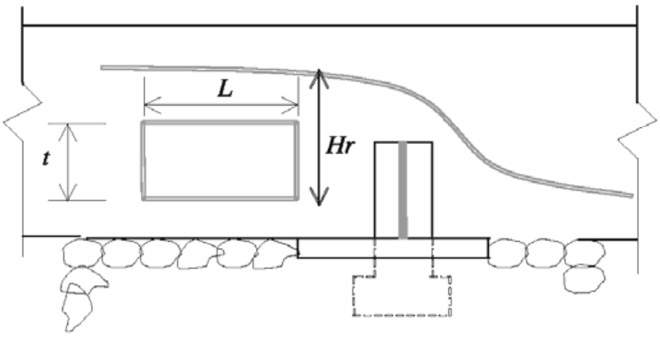
FIGURE 28 WATER INTAKE



h = height of water in inlet during dry season

If, Q = 0.300 m^3/s, h = 0.30 m, C = 1.6 from formula 5.a, L = 1.15 m

So that: t = h + 0.05 m = 0.35 m



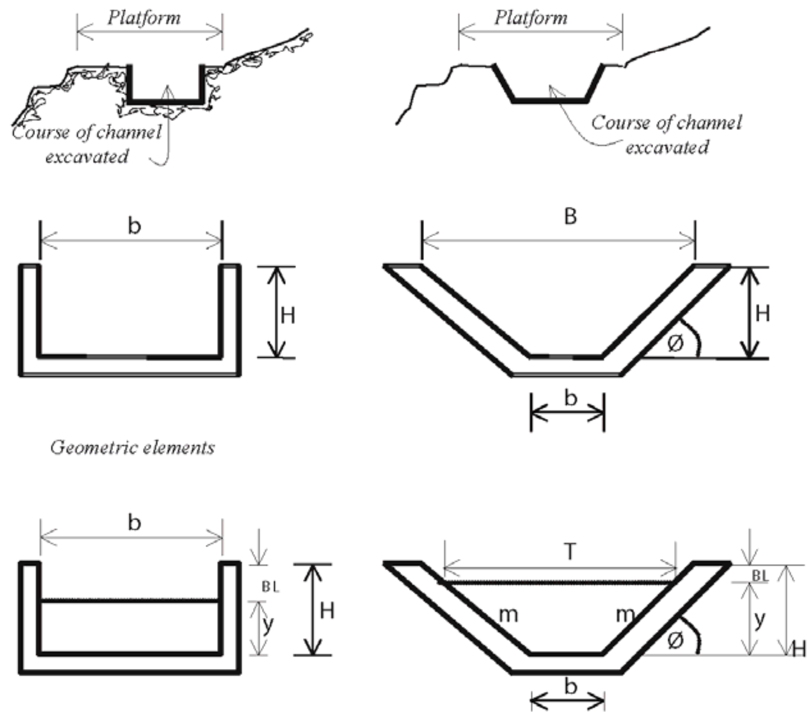
Under flood conditions the inlet is submerged. The dimensions of the inlet do not change, but the dimensions of the inlet channel should depend on the increase in flow rate, which may be 30% to 60% more, and in addition a lateral spillway envisaged in the channel.

the river bed near the intake is protected with a concrete slab, which also ensures the stability of the barrage. Upstream and downstream of this slab is a stone channel resting on a pavement of 1:6 grouted with a 1:3 cement sand mortar.

5.1.2 Inlet Channel

The purpose of the inlet channel is to take water from the intake to the head tank. It may be open or closed, and is generally rectangular, trapezoidal or circular in cross section. The characteristics of the channel are shown in figure 29.

FIGURE 29 INLET CHANNEL



Hydraulic elements

Wetted perimeter: $P = b + 2y$ $P = b + 2m$

Area: $A = b.y$ $A = \frac{(T+b)}{2}.y$

Hydraulic radius: $R = \frac{A}{P}$ $R = \frac{A}{P}$

Slope: S = Tangent of the angle formed by the vertical projection h and the horizontal q of the length of the channel.

Roughness: n depends on the lining material of the channel. See roughness table.

The flow rate in the channel is equal to the product of the wetted area and water velocity.

The general formula is: $Q = A \times V$

Where:

- A = wetted area in m^2
- V = velocity in m/s
- Q = flow rate in m^3/s

The velocity of the water is calculated using Manning's formula:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where:

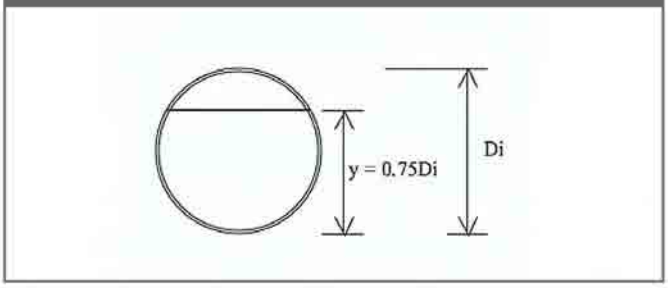
- n = Roughness coefficient of the lining material
- R = Hydraulic radius
- S = Channel slope expressed as the equivalent decimal number (so much percent or so much per thousand)

Circular cross section channels are generally plastic pipe in which the water depth is 0.75D.

The flow rate is calculated using the following formula:

$$Q = 28,4 \, x Di^{8/3} x S^{1/2}$$

FIGURE 30 CIRCULAR CROSS SECTION CHANNEL



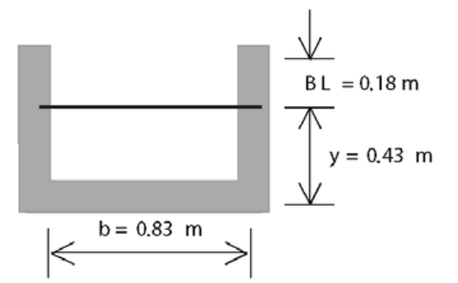
Where:

- Di = Inside diameter (m)
- y = water depth (m)
- n = 0.01
- S = slope

TABLE 10. ROUGHNESS COEFFICIENTS: N	
Surface Lining	**n**
Clay	0.013
Sand and gravel	0.021
Irregular rock	0.045
Stone and cement masonry	0.020
Concrete - fine finish	0.010
Concrete rough finish	0.014
Irregular concrete	0.020
Planed timber	0.011
Rough timber	0.0125
PVC pipe	0.010

Example: Calculate the cross section of a rectangular channel to carry 0.400 m^3/s, with a slope of 2 per thousand, lined with fine-finished concrete.

FIGURE 31. RECTANGULAR CROSS SECTION CHANNEL



Attempt	B (m)	y (m)	A(m²)	P (m)	R (m)	$R^{2/3}$	n	1/n	S	S½	V(m/s)	Q (m³/s)
1	0.80	0.40	0.320	1.60	0.20	0.342	0.014	71.42	0.002	0.044	1.07	0.343
2	0.86	0.43	0.369	1.72	0.214	0.362	0.014	71.42	0.002	0.044	1.13	0.419

The data are obtained from each attempt until the design flow rate is reached using the respective formulae.

Assume a value of b = 0.80, which is the depth of the water; it is recommended that this should be one half of b (rectangular channel with maximum hydraulic efficiency); then P, A, R, V and Q are calculated.

The known values are S, and the value of n is taken from the table roughness coefficients.

The first attempt gives us a value of 0.343 m^3/s. Thereafter, as recommended, we make a second attempt assuming a value of b = 0.86, then 0.43 m and so on until the value of Q = 0.419 m^3/s is obtained, very close to the design flow rate of 0.400 m^3/s. Finally, it is recommended that the freeboard should be greater than 0.15 m and the thickness of the channel not less than 0.10 m.

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In example 2, if we want to calculate a channel with a trapezoidal cross section, we follow the same steps using the data shown below. It should be remembered that the maximum efficiency of a trapezoidal cross section channel occurs when the walls of the channel forms an angle of 60º to the horizontal. For reasons of geometry, when Ø = 60º, the depth y = 0,866 b; T = 2b; b = m.

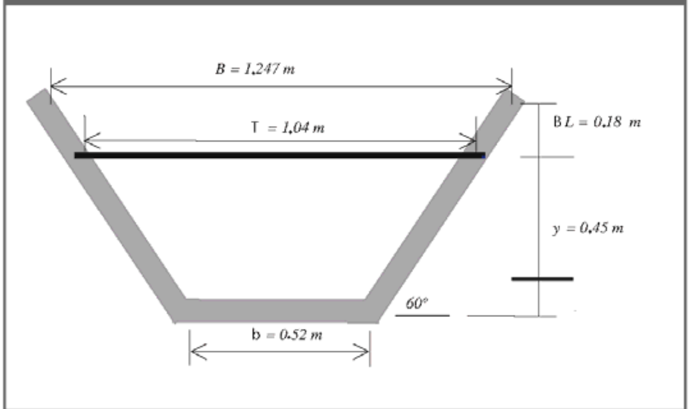
Details:

Q = 0.400 m^3/s

S = 0.002

n = 0.014 roughness coefficient for fine-finish concrete

FIGURE 32. TRAPEZOIDAL CROSS SECTION CHANNEL



Calculation:

Attempt	B (m)	y (m)	T(m)	A (m²)	P (m)	R (m)	R 2/3	n	1/n	S	S½	V(m/s)	Q (m³/s)
1	0.48	0.415	0.96	0.298	1.44	0.206	0.353	0.014	71.42	0.002	0.044	1.10	0.330
2	0.52	0.450	1.04	0.351	1.56	0.225	0.373	0.014	71.42	0.002	0.044	1.178	0.413

In this second attempt we obtain a flow rate of 0.413 m^3/s. The freeboard is assumed to be equal to the previous case, and the value B = 1.247 m is obtained in accordance with the assumed value of the freeboard and from the ratios with respect to the angle 60º.

Example 3. Calculate the internal diameter of a pipe to carry 0.400 m^3/s with a slope of 2 per thousand.

x x

Use the formula $Q = 28,4xDi^{8/3}xS^{1/2}$

$$Di = \left[\frac{0x400}{28,40x0,002^{1/2}}\right]^{3/8} = 0,65m = 26''$$

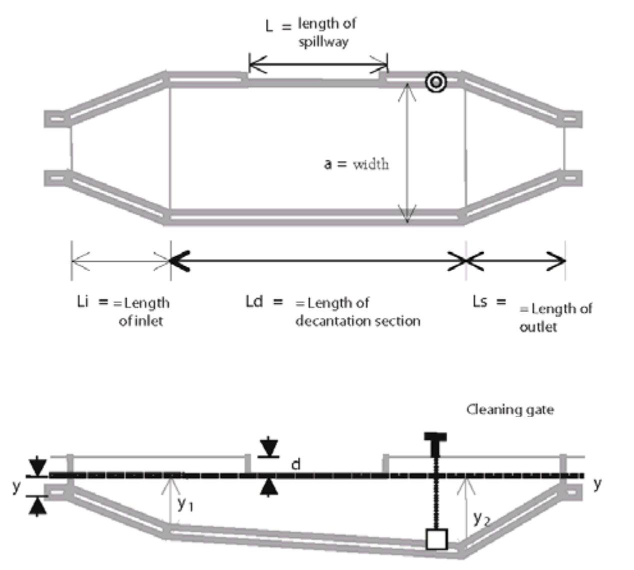
Of these three options the one with the lowest construction cost will be chosen.

5.1.3 Settling Basin

The settling basin removes particles carried by the water, which are sedimented out along the length of the inlet channel. It is usually placed after the intake, alongside the head tank.

Elements of a settling basin:

FIGURE 33. DIAGRAM OF A SETTLING BASIN



y = water depth in channel
y_1 = depth at end of inlet
y_2 = depth at end of decantation section, including depth of decantation and sediment collection
d = height of lateral spillway in sand trap.

The length of the decantation section includes the sediment collection zone and a drainage system to clean out the sediment through a gate.
The spillway is connected to a channel which automatically controls any excess flow that may occur.

The particles of sand, mud and clays deposited in the settling basin have a decantation velocity (Vd) and a horizontal velocity (Vh). Researchers in this field have provided us with the following values:

Particle diameter in mm	Vd in m/s
0.1	0.02
0.3	0.03
0.5	0.1
1.0	0.4

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In order for the particles to sediment out, it is recommended that the horizontal velocity of the water in the settling basin should not be less than 0,2 m/s or greater than 0,4 m/s. This is achieved by changing the cross section of the channel before the settling basin, through the inlet. On exiting the settling basin the water achieves channel velocity because of the law of continuity.

The formulae that enable us to calculate the dimensions of the settling basin:

$$a = \frac{Q}{Vh \times y}$$

Where:

Q = Design flow rate (equal to that of the inlet channel), in m^3/s.

Vh = Horizontal velocity of particles in m/s.

y = Depth of water in the channel, in m.

a = Width of the settling basin over the length where sedimentation occurs.

$$Ld = \frac{Vh}{Vd} \times y \times F$$

Where:

Ld = Decantation length in m

Vh = Horizontal velocity of water in the settling basin, in m/s.

Vd = Decantation velocity of particles in m/s.

y = Decantation depth; generally the same as the depth of water in the channel, or slightly greater.

F = Safety factor; varies between 2 and 3

For practical reasons, the length of the inlet and outlet are the same as the width of the settling basin.

Depth $y_1 = y +$ (from 40 to 60% of y).

Depth $y_2 = y_1 +$ (from 4% to 5% of the decantation length).

In special cases the quantity of material carried by the river in the rainy season has to be determined, then the quantity of fine material entering the channel, the cleaning period for the settling basin and the minimum dimensions of the sediment basin.

These depths are depend on the level of the water controlled by the lateral spillway. The total depths of the settling basin will be as calculated, plus the height of the lateral spillway, including its freeboard.

Example: Calculate the dimensions of the settling basin in accordance with the design flow rate of the inlet channel, $Q = 0{,}400\ m^3/s$, when the largest sedimentable particles are 0.2 mm in diameter and the channel is rectangular in cross section and $y = 0.43$ m .

Details:

$Q = 0.400\ m^3/s$

$Vh = 0.3\ m/s$ (as recommended)

$y = 0.43\ m$

Formula:

$$a = \frac{Q}{Vh \times y}$$

Calculate a,

$$a = \frac{0.400}{0.3 \times 0.43} = 3.10m$$

Calculate the decantation length and depths:

Details:

$Vh = 0.3\ m/s$

$Vd = 0.03\ m/s$ (from the decantation velocity table, for particle diameter 0.3 mm)

$y = 0.43\ m$

$F = 2$ (in accordance with recommended range)

Formula:

$$Ld = \frac{Vh}{Vd} \times y \times F$$

Calculate Ld:

$$Ld = \frac{0.3h}{0.03d} \times 0.43 \times 2 = 8.60m$$

$Li = a = 3.10\ m = Ls$

$y_1 = 1.5y = 1.5 \times 0.43 = 0.645\ m$

$y_2 = 0.645 + 0.04 \times 8.60 = 0.989\ m$

Calculate the height of the settling basin lateral spillway, including free-board.

In this case the lateral spillway is envisaged for use in an emergency in which all water flowing into the settling basin has to be evacuated automatically without opening the cleaning sluice. So, we apply the formula for wide-wall free spillways:

$$Q = C \times L \times h^{1.5}$$

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Details:

$Q = 0.400\ m^3/s$

$h = 0.14$ m (assumed)

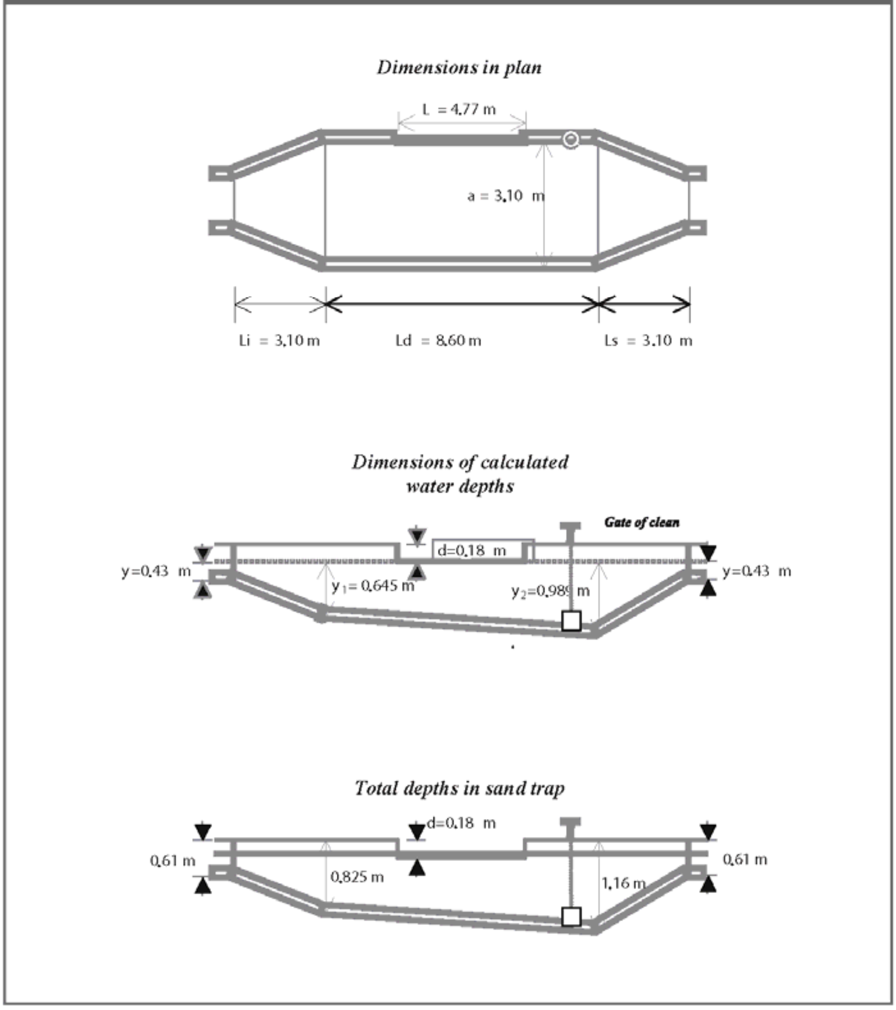
$C = 1.6$ (discharge coefficient)

Calculate L =

$$L = \frac{Q}{C \times h^{1.5}} = \frac{0.400}{1.6 \times 0.14^{1.5}} = 4.77m$$

The following gives details of the final dimensions of the settling basin. The depths include the total height of the spillway and freeboard d = h + BL = 0.14 + 0,04 = 0,18 m.

FIGURE 34. FINAL DIMENSIONS OF THE SETTLING BASIN



For the cleaning system and emergency evacuation, the settling basin should be fitted with a shutoff sluice before the cleaning sluice and a suitably prepared spillway.

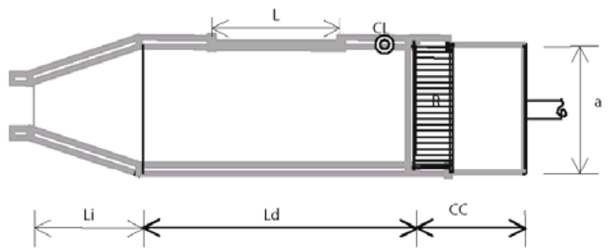
5.1.4 Head Tank

The head tank is located at the end of the inlet channel. It also has a settling basin, so that the water enters the penstock without sand particles that could reduce the life of the penstock and turbine.

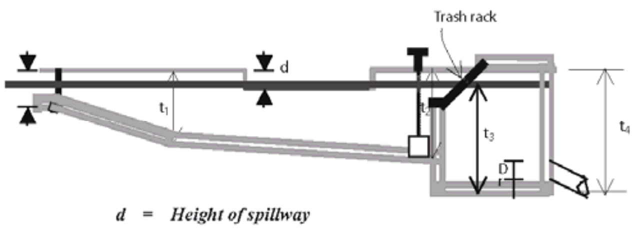
Components of the head tank

The head tank is prism-shaped. The penstock is located on one wall and the water enters through a safety grill to retain leaves pieces of timber, plastic bottles, etc. For safety reasons it is permanently covered.

FIGURE 35. DIAGRAM OF THE HEAD TANK



Li = Length of inlet
Ld = Length of decantation section
CC = Surge chamber
L = Length of spillway
CL = Cleaning gate
R = Trash rack



d = Height of spillway
t_1 = Depth at end of Li, with respect to water level
t_2 = Depth at end of Ld, with respect to water level
t_3 = depth of surge chamber with respect to water level
t_4 = Total depth of surge chamber
D = Diameter of penstock
r = Distance from start of penstock to bottom of surge chamber

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Dimensions of the head tank:

The dimensions of the head tank are determined by the following practical criteria:

$t_3 = r + 4,5D$

$t_4 = t_3 + d$

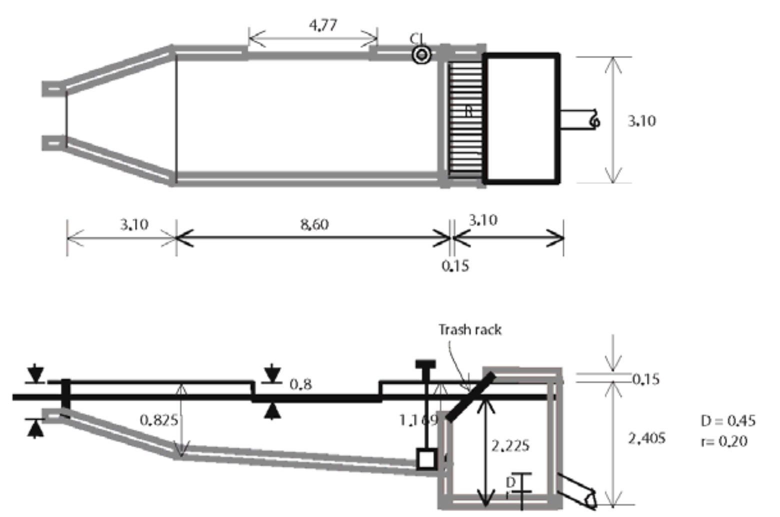
D = diameter of the penstock; determined by the design of the penstock.

r = between 0.15 m and 0.30 m, in depending on whether the pipe is smaller or larger.

The width and length of the head tank are the same as those of the settling basin.

In the above example, the dimensions of the head tank are given below, where the minimum pipe diameter is 0.45 m.

FIGURE 36. DIMENSIONS OF THE HEAD TANK



Trash rack characteristics

The trash rack is fitted to the wall dividing the settling basin and the head tank, and is mounted on the roof. The mounting is hinged and fitted with a safety device, which should also allow access to the head tank for maintenance and cleaning.

In general, grills have a stainless steel frame and steel bars, the distance between which is less in centimetres than the diameter of the turbine nozzle, if it is a Pelton turbine, or that recommended by the turbine manufacturer.

5.1.5 Penstock

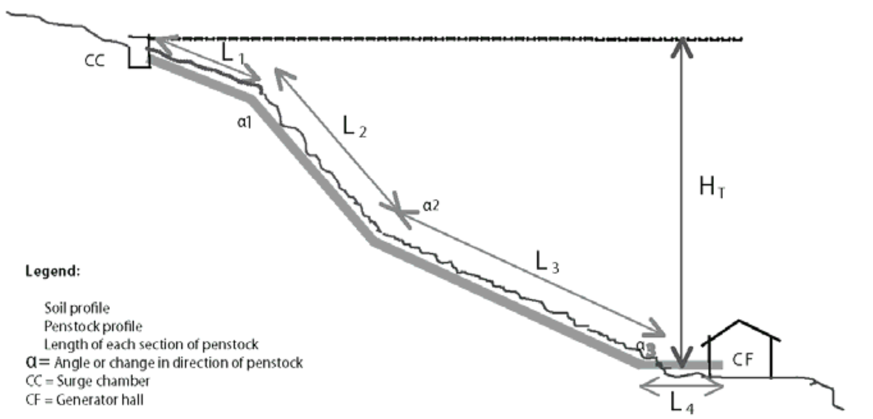
The penstock is also called the pressure pipe, being a closed conduit carrying water under pressure from the head tank to the turbine in the turbine hall.

Its cross-section is circular, and modern mini/micro hydropower plants use high pressure PVC, which is lighter, easy to install and cheaper than steel pipe. When the head is greater than 150 m, steel pipe must be used for the lowest section.

It is recommended that the penstock should run in a straight line. Changes in direction in the vertical plane will depend on the topography of the terrain.

PVC pressure pipe is buried and where it changes direction PVC plates made according to the pipe profile are fixed to it.

FIGURE 37. INSTALLATION OF THE PENSTOCK



(1) Design of the penstock

The design of the penstock consists in determining:

a) The profile of the pipe depending on where the head tank and generator hall are to be built and the profile of the terrain.

b) The diameter and thickness of the pipe, depending on the pressure and overpressure it has to bear.

To obtain the profile of the pressure pipe, the profile of the terrain is first drawn, then the route of the penstock depending on the material chosen. If the pipe is to be PVC, it should be buried to a depth of 0.80 m in accordance with the manufacturer's recommendations. If it is to be steel, it runs overground resting on concrete supports, and in both cases it is secured to the terrain by anchors (concrete blocks).

The profile of the pressure pipe provides important data, such as the following:

- Changes of direction expressed in degrees, to determine the curves necessary.
- Length of each section of the pipe
- Location of the anchors and supports.
- Location of the head tank and generator hall.
- The vertical distance between the contour at which the generator hall is to be built and the river level in the rainy season.
- The gross height (vertical distance between the water level in the head tank and the centre-line of the turbine).
- The partial head and accumulated head for each section of the penstock.
- The upstream and downstream angles of the penstock with the horizontal at each change of direction (anchors).

The penstock calculation follows these steps:

a) Interior diameter of the pipe

$$d_i = \sqrt{\frac{4 \times Q}{\pi \times V}}$$

A value between 2 and 3 (m/s) is calculated for the water velocity in the penstock.

With di taken from the pipe manufacturers' catalogue (guaranteed by the quality certificate) the external diameter and thickness are noted to see whether the previously calculated diameter will be the final one for each section, then the following calculations are undertaken.

b) Calculate the loss of head in each section of the penstock: hp

$$hp = hf + \mathrm{ht}$$

1) hf = loss of head through friction against the internal walls of the penstock, in m.

2) ht = loss of head through turbulence as water enters the penstock, at each change of section (curves, valves, etc.)

$$hf = 0{,}08 \times \frac{f \times L_i \times Q^2}{d_i^5}$$

f =Friction factor
L_i=Length at section of the penstock (m)
Q=Design flow rate (m³/s)
d_i=Internal diameter of penstock (tentative)

$$hf = 0,08 \times \frac{f \times L_i \times Q^2}{5}$$

Calculation of friction factor (f)

Moody diagrams are used for friction losses in steel or PVC pipes.

The Moody diagram uses the following values:

$$1\ 27 \times \frac{Q}{d_i} \text{ , and } K/d_i$$

K = Absolute roughness of the internal walls of the penstock, in mm (see table)

Internal diameter d.i = internal diameter, in m

To obtain this ratio, both should use the same units of measurement.

FIGURE 38. MOODY DIAGRAM

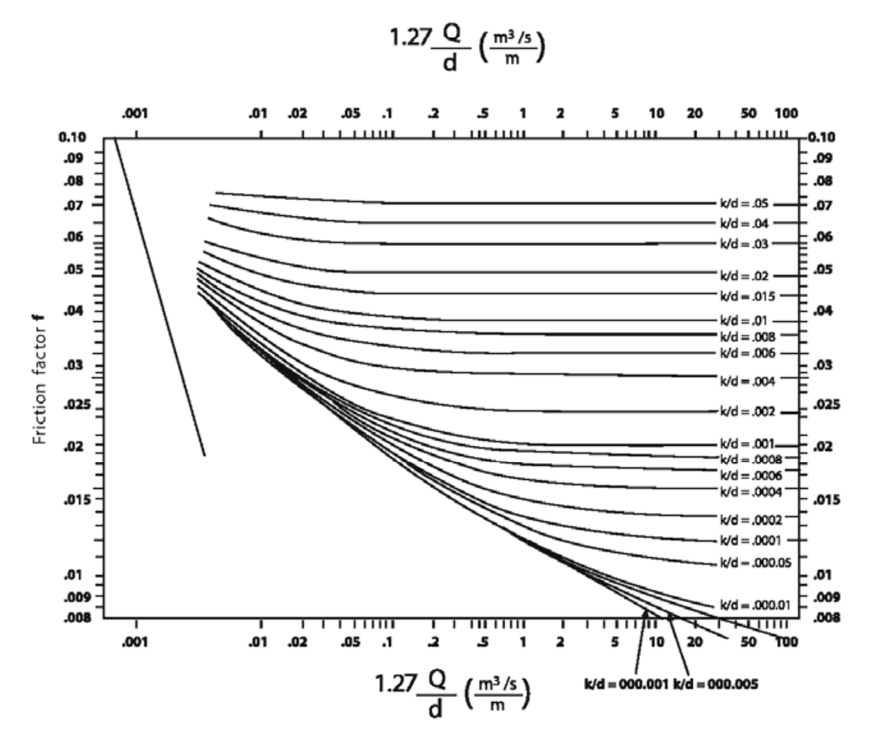


TABLE 11. VALUES OF ABSOLUTE ROUGHNESS K IN MM			
Penstock material	State and age of the penstock		
	Good < 5 years	Normal 5 - 15 years	Bad < 15 years
PVC, polyethylene, polyester resin and glass fibre	0.003	0.01	0.05
Concrete	0.06	0.15	1.5
Mild steel			
a) Unpainted	0.01	0.1	0.5
b) Galvanised	0.06	0.15	0.3
Cast Iron			
a) New:	0.15	0.3	0.6
b) Old: b.1 Slight corrosion	0.6	1.5	3
b.2 Moderate corrosion	1.5	3	6
b.3 Severe corrosion	6	10	20

Source: Adam Harvey. *Micro-hydro design manual.* 1993.

The Moody diagram can be replaced using the following formula:

$$f = \frac{0.25}{\left[Log\left(\frac{\frac{K}{d_i}}{3,7} + \frac{5.74}{\left(\frac{V \times d_i}{\mu}\right)^{0,9}} \right) \right]^2}$$

K = Absolute roughness, depending on material, in mm.

d_i = Internal diameter of penstock, using the same units as K.

V = Water velocity in the section (m/s).

μ = Kinematical viscosity of the water in m^2/s.

Calculating the loss of head through turbulence:

$$h_t = \frac{V^2}{2g}\left(K_1 + K_2 + ...Kn\right)$$

V = Water velocity in the section in question.

K_i = K_1, K_2, ……. Kn is a factor associated with elbows, curves, changes in direction, etc., (see table of head loss coefficients through turbulence).

g is the acceleration due to gravity = 9.81 m/s^2

This step may be omitted of the calculated values are less than the values for loss through friction.

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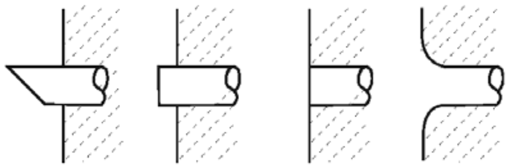
FIGURE 39. HEAD LOSS COEFFICIENTS THROUGH TURBULENCE

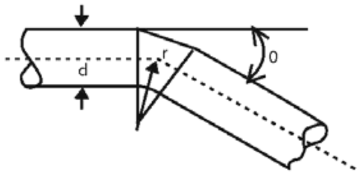
Coefficients of loss through turbulence

Entry profiles



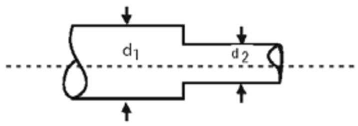
K_{entry}	1.0	0.8	0.5	0.2

Bend profiles



r/d	0°	1	2	3	5
K_{bend}	20	0.36	0.25	0.20	0.15
K_{bend}	45	0.45	0.38	0.30	0.23
K_{bend}	90	0.60	0.50	0.40	0.30

Reducer profiles



di/d2	1.0	1.5	2.0	2.5	5
$K_{reducer}$	0	0.25	0.35	0.40	0.50

Valve coefficients

Valve type	Ball	Gate	Butterfly
K_{valve}	0	0.1	0.3

Source: Adam Harvey. *Micro-hydro design manual.* 1993.

Calculating these values we find $hp = hf + ht$

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c) Calculation of percentage head loss through friction

$$Loss = \frac{h_p}{H_B} \times 100$$

This result, depending on the quality of the materials, length of the penstock, pressures, etc., is usually less than 10%. If it is not, then the calculation should be repeated using different thicknesses and diameters.

d) Calculation of the wall thickness of each section of the penstock

The wall thickness also depends on:

1) The highest pressure Δh..

These are transitory or temporary pressures caused by valves being closed or by water hammer, that sometimes occur over fractions of a second when the turbine has only one nozzle, as is the case with Pelton turbines.

2) Reduction in the thickness of the penstock walls during service, caused by corrosion, wear or manufacturing defects.

Transitory pressure in m is calculated using the following formula:

$$\Delta h = \frac{a \times V}{g}$$

Where:

V = Water velocity in the section in question.

a = Pressure wave propagation velocity, in m/s

g = Acceleration due to gravity = 9.81 m/s^2.

The propagation velocity of the pressure wave(a) depends on the diameter, thickness and material of the penstock. For a general approximation the following values can be used:

Mild steel pipe a = 900 m/s

Cast iron pipe a = 1.250 m/s

PVC pipe a = 350 m/s

The following formula is used for more accurate calculations:

$$a = \frac{1420}{\sqrt{1 + \left(\frac{E_{ag} \times d_i}{E_{pvc} \times t}\right)}}$$

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Where:

E_{ag} = Modulus of elasticity of water, in kg/cm^2.

E_{pvc} = Modulus of elasticity of PVC, in kg/cm^2.

d_i = Internal diameter, in m.

t = Thickness of the pipe, in m.

If the pipe is made of steel or other materials, the modulus of elasticity to be used should be taken from the manufacturer's tables appearing in its catalogues.

3) The theoretical thickness T or expected thickness is calculated using the following formula:

$$T = \frac{5 \times f_S \times h_T \times d_i \times K_j}{\sigma} + K_c$$

A = Theoretical or expected thickness in mm

f_s= Safety factor according to the material of the pipe

If the material is PVC, an fs greater than 2 is recommended

$$h_T = H_i + \Delta h$$

h_T = is the maximum pressure in m, in the section in question.

H_i = The gross head or pressure in the section, in m.

Δh = The highest transitory pressure, in m, in the section in question.

d_i = Internal diameter of the pipe, in m.

σ = Maximum tensional stress of the material, in N/m^2.

K_j = Thickness correction factor depending on the type of pipe material.

For steel pipes:

K_j = 1.1 mm, for pipe with welded joints.

K_j = 1.2 mm, for rolled and welded plate.

K_c =1.0 mm, corrosion factor (1 mm in 10 years).

For PVC pipes:

K_j = 1.0 mm.

K_c = 1 mm every 10 years is the wear factor.

The results obtained should be compared determine the diameter and thickness required; otherwise the calculations should be done again.

This formula can be simplified taking into account the following logical deductions:

$$T = \frac{5 \times f_s \times h_T \times d_i \times K_j}{\sigma} + Kc$$

For PVC pipe: $K_i = 1$

Therefore: $T - K_c = t$

And therefore, the equivalent formula for PVC is:

$$t = \frac{5 \times fs \times h_T \times d_i}{\sigma}$$

This formula is used to determine the safety factor

$$fs = \frac{t \times \sigma}{5 \times h_T \times d_i}$$

with the condition that fs ≥ 2 is used for each section, the units used are:
t (mm), σ in N/m², h_T in m, and d_i in m

N.B.: In both cases if the units de σ are expressed in kg-f/cm2, the equivalent formula is:

For PVC: $$fs = \frac{t \times \sigma}{5 \times h_T \times d_i \times 10}$$

For steel: $$fs = \frac{t \times \sigma}{5 \times h_T \times d_i \times K_i \times 10}$$

4) Data taken from tables and catalogues of steel and PVC pipe manufacturers

E_{AG} = Modulus of elasticity or expansion for water = 21.000 kg-f/cm²

μ = Kinetic viscosity of water (15 ºC) = 1,14 x 10^{-6} kg-f/cm²

E_{AC} = Modulus of elasticity of steel = 2,000,000 kg-f/cm²

E_{PVC} = Modulus of elasticity of PVC = 28,000 kg-f/cm²

σ_{AC} = Maximum tensional stress of steel (breaking point) = 3.500 kg-f/cm²

σ_{PVC} = Maximum tensional stress of PVC (breaking point) = 560 kg-f/cm²

K_{AC} = Absolute roughness or roughness of steel = 0.1 mm

K_{PVC} = Absolute roughness or roughness of PVC = 0.01 mm

d_e = Nominal outside diameter of the pipe (m)

d_i = Inside diameter of the pipe (m)

t = Pipe thickness

Catalogue tables are given below for reference.

TABLE. 12. PENSTOCK SYSTEMS, RIEBER JOINTS. NPT ISO 4422 STANDARD

Rieber flexible joint				Series 6.6 (Class 15) Working pressure at 20 °C: 15 bar			Series 10 (Class 10) Working pressure at 20 °C: 10 bar			Series 13.3 (Class 7.5) Working pressure at 20 °C: 7,5 bar			Series 20 (Class 5) Working pressure at 20 °C: 5 bar		
Nominal diameter		Length (m)		Int. dia. mm	Wall thickness mm	Approx. weight kg/U	Int. dia. mm	Wall thickness mm	Approx. weight kg/U	Int. dia. mm	Wall thickness mm	Approx. weight kg/U	Int. dia. mm	Wall thickness mm	Approx. weight kg/U
mm	inch	Useful	Total												
63	2	5.90	6.00	54.20	4.40	7.32	57,00	3.00	5.13	58.40	2.30	3.99	59.80	1.60	2.83
75	2.5	5.89	6.00	64.40	5.30	10.48	67.80	3.60	7.32	69.40	2.80	5.78	71.20	1.90	4.00
90	3	5.89	6.00	77.40	6.30	14.96	81.40	4.30	10.50	83.40	3.30	8.18	85.60	2.20	5.56
110	4	5.88	6.00	94.60	7.70	22.35	99.40	5.30	15.81	102.00	4.00	12.13	104.60	2.70	8.34
140	5.5	5.87	6.00	120.40	9.80	36.06	126.60	6.70	25.21	129.80	5.10	19.42	133.00	3.50	13.48
160	6	5.85	6.00	137.60	11.20	47.28	144.60	7.70	33.42	148.40	5.80	25.58	152.00	4.00	17.96
200	8	8.84	6.00	172.00	14.00	73.88	180.80	9.60	52.09	185.40	7.30	40.24	190.20	4.90	27.53
250	10	5.81	6.00	215.00	17.50	115.44	226.20	11.90	80.75	231.80	9.10	62.71	237.60	6.20	43.52
315	12	5.77	6.00	271.00	22.00	182.89	285.00	15.00	128.25	292.20	11.40	99.01	299.60	7.70	68.15
355	14	5.75	6.00	305.40	24.80	231.07	321.20	16.90	161.23	329.20	12.90	124.52	337.60	8.70	85.01
400	16	5.74	6.00	344.00	28.00	293.92	361.80	19.10	205.28	371.00	14.50	157.72	380.40	9.80	107.90

Source: Amanco del Perú S.A. *Catalogue.*

TABLE 13. TECHNICAL CHARACTERISTICS OF PVC

Físicas		Mecánicas	
Specific mass	1.41 g/cm3 at 25 °C	**Design stress**	100 kg/cm²
Water absorption	<40 g/m2	**Tensile strength**	560 kg/cm²
Dimensional stability	at 150 °C < 5%	**Bending strength**	750 – 780 kg/cm²
Thermic dilatation coefficient	0,06 mm/m/°C	**Compression strength**	610 – 650 kg/cm²
Dielectric constant	A 103-106 Hz:3-3.8	**Modulus of elasticity**	≈ 30.000 kg/cm²
Flammability	Self-extinguishing		
Coefficient of friction	n = 0,009 Manning C =150 Hazen-Williams		
Vicat point	≥80 °C for water conduit		

Source: Amanco del Perú S. A. *Catalogue.*

TABLE 14. RIGID PVC PIPE WORK

Nominal diameter (ASTM)	Outside diameter mm	Class 15 (215) RDE-14.3 Thickness mm	Class E 10 (145) RDE-21 Thickness mm	Class 7.5 (108) RDE-27-7 Thickness mm	Class 5 (72) RDE-41 Thickness mm	Length Icluding EC m
2"	60	4.2	2,9	2.2	1.8	5
2 ½"	73	5.1	3.5	2.6	1.8	5
3"	88.5	6.2	4.2	3.2	2.2	5
4"	114	8.0	5.4	4.1	2.8	5
6"	168	11.7	8.0	6.1	4.1	5
8"	219	15.3	10.4	7.9	5.3	5
10"	273	---	13.0	9.9	6.7	5
12"	323	---	15.4	11.7	7.9	5

Source: Interquímica S. A. (Peru) *Catalogue.*

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TABLE 15. STEEL PIPE SYSTEMS

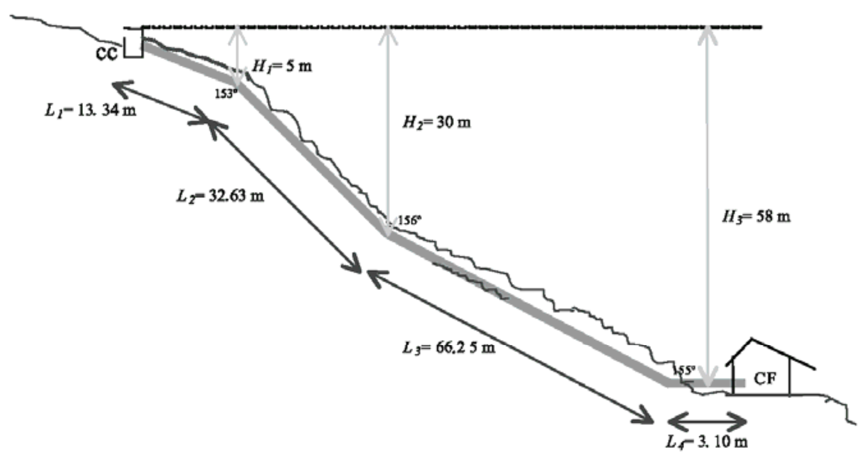
Medida nominal pulg.	Schedule 40				Schedule 80				Schedule 160			
	Diám. exterior pulg.	Espesor pared pulg.	Diám. interior pulg.	Peso lib/pie	Diám. exterior pulg.	Espesor pared pulg	Diám. interior pulg.	Peso lib/pie	Diám. exterior pulg.	Espesor pared pulg	Diám. interior pulg	Peso Lib/pie
2	2,375	0.154	2,067	3,66	2,375	0.218	1,939	5.03	2,375	0.343	1,689	7,45
2 1/2	2,875	0.203	2,469	5,80	2,875	0.276	2,323	7.67	2,875	0.375	2,125	10,00
3	3,500	0.216	3,068	7,58	3,500	0.300	2,900	10,30	3,500	0.437	2,626	14,30
3 1/2	4,000	0.226	3,548	9,11	4,000	0.318	3,364	12,50	4,000	---	---	---
4	4,500	0.237	4,026	10,80	4,500	0.337	3,826	15,00	4,500	0.531	3,438	22,60
5	5,563	0.258	5,047	14,70	5,563	0.375	4,813	20,80	5,563	0.625	4,313	33,00
6	6,625	0.280	6,065	19,00	6,625	0.432	5,761	28,60	6,625	0.718	5,189	45,30
8	8,625	0.322	7,981	28,60	8,625	0.500	7,625	43,40	8,625	0.906	6,813	74,70
10	10,750	0,365	10,02	40,50	10,750	0.593	9,564	64,40	10,750	1,125	8,500	116,00
12	12,750	0,406	11,938	53,60	12,750	0,687	11,376	88,60	12,750	1,312	10,126	161,00
14 OD	14,000	0,437	13,126	63,30	14,000	0.750	12,500	107,00	14,000	1,406	11,188	190,00
16 OD	16,000	0,500	15,000	82,80	16,000	0.843	14,314	137,00	16,000	1,562	12,876	241,00
18 OD	18,000	0,562	16,876	105,00	18,000	0.937	16,126	171,00	18,000	1,750	14,500	304,00

Source: Metales Andinos S.A., Peru *Catalogue*.

These calculations can be shortened by using a spreadsheet, as shown on the following page. The data to be entered are coloured green; those coloured blue are data for pre-established formulae.

Example: Using the penstock profile data from the previous example and a design flow rate of 0.400 m^3/s, calculate the diameter of the penstock using high-pressure PVC and flexible joints.

FIGURE 40. CALCULATING THE DIAMETER OF A HIGH PRESSURE PVC PIPE WITH FLEXIBLE JOINTS



Data:

a) Lengths of the pipe sections

$L_1 = 13.34$ m
$L_2 = 32.63$ m
$L_3 = 66.25$ m
$L_4 = .10$ m

b) Head

$H_1 = 5{,}00$ m
$H_2 = 30{,}00$ mm
$H_3 = 58{,}00$ m $= HT$

c) Deign flow rate

$Q = 0.400\ m^3/s$

a) Calculating the (tentative) inside diameter, assuming V = 3 m/s

$$d_i = \sqrt{\frac{4 \times Q}{\pi \times V}} = \sqrt{\frac{4 \times 0{,}400}{3{,}14 \times 3}} = 0.41m$$

b) Depending on the tentative diameter, we enter the figures in section A of the spreadsheet (coloured green).

In section B we enter the length of the pipe and head for each section, the outside diameter and wall thickness in mm for each class or strength of pipe in metres of head (classes 5, 7.5 and 10).

Classes are chosen bearing in mind that class 5 copes with a head of up to 50 m, class 7.5 with heads of up to 75 metres and class 100 with heads of up to 100 metres, such that the formulae immediately evaluate the strength and thickness of the pipe chosen when values given are introduced. In the example the safety factor for the diameter and thickness chosen is greater than 2; therefore, the chosen diameter and thickness are valid for the project.

TABLE 16. PENSTOCK SPREADSHEET

a) Data

Design flow rate		m^3/s	0,4
Gross head		M	58
Total length total of penstock		M	,32
Number of changes in direction			3
Number of changes in section			3
Module of expansion of the fluid		kgf/cm^2	21.000
Kinematic viscosity of water (15 C)		m^2/s	1,14E-06
Module of elasticity of steel		kgf/cm^2	2.500.000
Module of elasticity of PVC		kgf/cm^2	24.700
Maximum tensional stress of steel		kgf/cm^2	3.500
Maximum tensional stress of PVC		kgf/cm^2	400
Roughness coefficient of steel		Mm	0,1
Roughness coefficient of PVC		Mm	0,009

b) Evaluating the strength of the pipe

			PVC UF Class 5	PVC UF Class 7.5	PVC UF Class 10
Section length		M	13,34	32,63	69.35
Head in section		M	5	30	58
Nominal diameter		inch	15,74	15,74	15.74
Outside diameter		M	0,4	0,4	0.4
Inside diameter		M	0,3804	0,371	0.3618
Thickness (take corrosion, etc. into account)		Mm	9.8	14,5	19,1
Water velocity		m/s	3,52	3,70	3.89
Wave velocity		m/s	243,52	297,69	343.34
Critical closure time		S	0,11	0,22	0.40
Maximum transitory pressure		M	87,37	112,28	136.17
Maximum pressure in penstock		M	92.37	142,28	194,17
Safety factor for thickness			2.23	2,20	2,18

c) Calculating head loss

Relative roughness			2,37E-05	2,43E-05	2,49E-05
Reynolds number			1.174.420,80	1.204.177,02	1.234.797,33
Friction factor			0,01	0,01	0.01
Friction head loss		M	0.26	0,73	1,75
Secondary loss coefficient			1.20	1,20	1,20
Secondary head loss		M	0.76	0,84	0,93
Head loss in the section		M	1.02	1,57	2,68
Total head loss		M	4,25		
Percentage loss		%	7,32		
Effective head		M	53.75		
Mechanical power at shaft		kW	158,04		
Electrical potential at alternator terminals		kW	135,12		

In the calculations for steel pipe, the steps are the same and a spreadsheet has also been prepared for this case.

It is recommended that the curve at each change in direction should be one class higher than the pipe itself for safety reasons. These curved sections should be made by the same supplier.

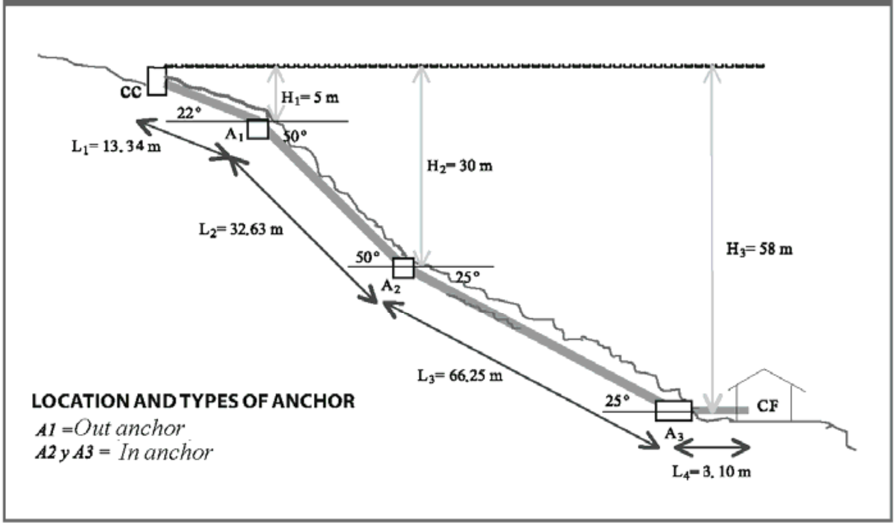
The weight of the PVC UF pipe is indicated in the catalogue for each diameter and thickness. If steel pipes are being compared with PVC, the installation of PVC UF is easy for the beneficiaries of the hydropower plant; on the other hand steel requires skilled personnel, the cost is higher and additional costs have to be incurred such as X-ray testing to ensure that there will be no leaks from the welded joints. Nevertheless, for heads greater than 150 m steel pipe is essential for the lower sections, where the pressure exceeds 150 m.

(2) Anchors

The anchors are concrete blocks that secure the penstock to the ground. They are located at each change in direction, inside and outside.

Looking at the penstock profile, the inside anchors are those on the concave curve of the pipe and outside anchors are those on the convex curve. The outside anchors are more dangerous as the force of the water can lift up the pipe.

FIGURE 41. LOCATION AND TYPES OF PENSTOCK ANCHORS.



(3) Design of the anchors

Designing the anchors consists of calculating the dimensions of the type of anchor required to secure the penstock to the ground.

Some of the data used in designing the anchors is obtained from the penstock profile such as:

- The angles α and β formed by the pipe and the horizontal (α = upstream, β = downstream) in the anchor under study.
- The head.
- The length of the penstock for each intervening force.

Thereafter, each intervening force is calculated

- We will address the three conditions of anchor stability: overturning, slippage and slumping, taking into account that the pipe is subject to dilatation and contraction.

Forces acting on the anchors (steel pipe)

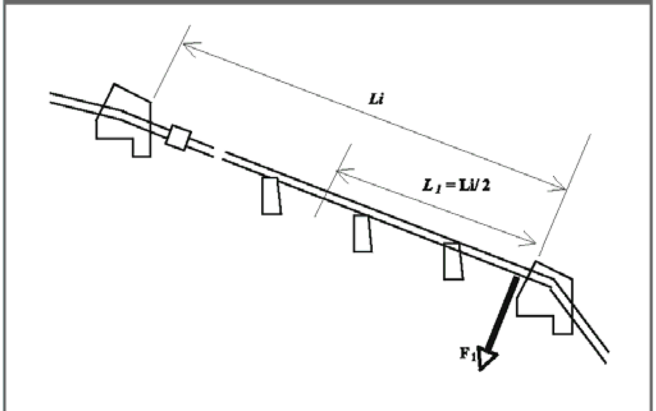
F_1 = Weight of the pipe + el weight of water

$$F_1 = W \times L_1 \times Cos\alpha$$

W = in kg-f/m = (weight of pipe + weight of water) over a length of 1 m

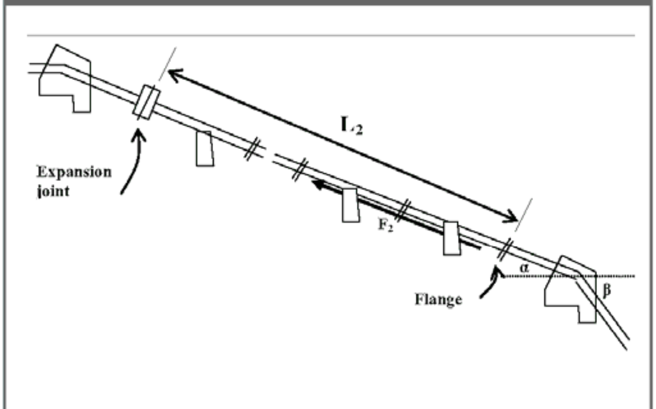
L_1 = Li/2, in m

FIGURE 42. FORCES ACTING ON THE ANCHORS



F_2 = Friction between pipe and supports. For this force to exist the must be at least one upwards anchor.

FIGURE 43. FRICTION BETWEEN THE PIPE AND SUPPORTS

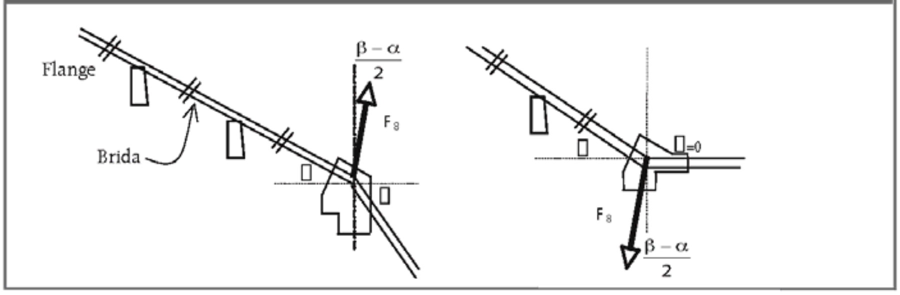


μ = Coefficient of friction between concrete and steel = 0.5

W = in kg-f/m = (weight of pipe + weight of water) over a length of 1 m

L_2 = Length between first flange near to upper anchor and expansion joint.

FIGURE 44. HYDROSTATIC PRESSURE IN ELBOW OR BEND



F_3 = For the hydrostatic pressure in the elbow or bend, at the change of direction.

$$F_3 = 1.6 \times 10^3 \times H_i \times d_i^2 \times Sin\frac{\beta - \alpha}{2}$$

H_i = Head pressure at anchor, in m

d_i = Inside diameter of pipe

F_3, in kg-f

F_4 = Component of weight of pipe parallel to the pipe

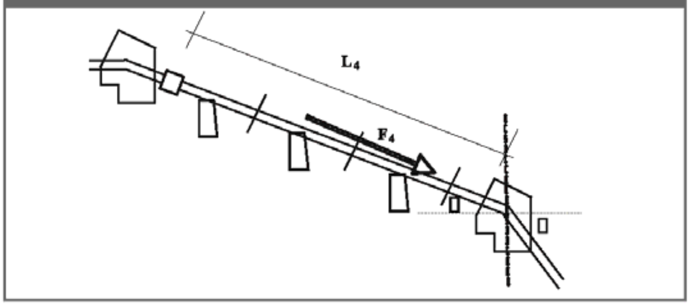
$$F_4 = W_t L_4 \times Sin\alpha$$

F_4, in kg-f

W_t = weight of the pipe in kg-f/m

L_4, in m (in practice, the length of the full section minus half the length of one pipe)

FIGURE 45. COMPONENT OF THE WEIGHT OF THE PIPE PARALLEL TO THE PIPE



F_5 = Stress caused by temperature changes in the pipe.

This stress is generated when the pipe is on the surface and is not fitted with an expansion joint.

$$F_5 = 31 \times d_i \times t \times E \times a \times \Delta T$$

F_5, in kg-f;

d_i, in m

t, in mm (pipe thickness)

E, module of elasticity of the pipe material in kg-f/cm^2

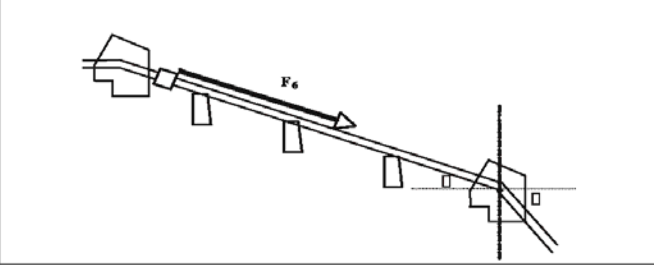
a' = coefficient of linear dilatation of the pipe, in ºC-1

This stress is = 0 when the steel pipe section contains an expansion joint.

PVC pipe is buried and is therefore not exposed to changes in temperature.

Therefore, $F_5 = 0$.

FIGURE 46. FRICTION IN EXPANSION JOINT



F_6 = Friction in expansion joint

Originates between the packing and the parts of the expansion joint. It acts in two directions, when the pipe expands and when it contracts.

$$F_6 = 3{,}1 \times d_i \times C$$

d_i = Inside diameter Interior of pipe, in m

C = Friction in expansion joint expressed in kg-f/unit of length of circumference

Approximately:

$$F_6 = 10 \times d_i$$

d_i, in mm

F_6, in Kg-f

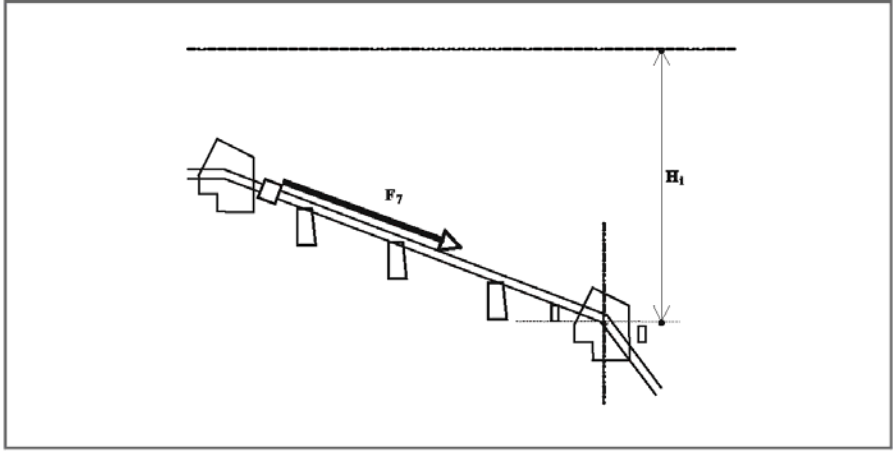
In PVC pipe, F6 = 0, no expansion joint is fitted.

F7 = Force due to hydrostatic pressure within the expansion joint. Tries to separate the two sections of the pipe joined by the expansion joint.

$$F_7 = 3{,}1 \times H_i \times d_i \times t$$

H_i = Head in the pipe in way of the anchor, in m
d_i = Inside diameter of the pipe, in m
t = Thickness of the pipe in mm.
F_7 = 0 in PVC pipe; no expansion joint is fitted.

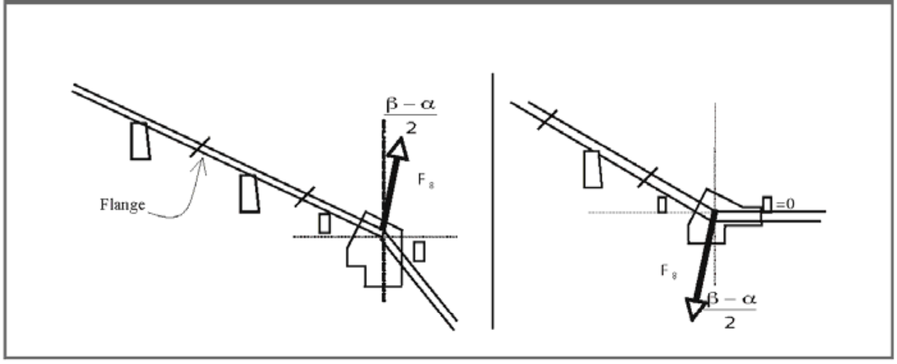
FIGURE 47. FORCE CAUSED BY HYDROSTATIC PRESSURE WITHIN THE EXPANSION JOINT



F8 = Force caused by change in the direction of the water flow in the elbow or bend. The direction is the same as F3.

$$F_8 = 250 \frac{Q}{d_i}^2 \times Sin\left(\frac{\beta-\alpha}{2}\right)$$

FIGURE 48. FORCE CAUSED BY CHANGE IN DIRECTION OF THE WATER FLOW IN THE ELBOW OR BEND.



This force is small compared to the others; it can therefore be ignored.

F_9 = Force caused by the reduction in inside diameter. It acts in the direction of the reduction, parallel to the pipe axis.

$$F_9 = 1\times10^3 \times H_i \times \left(d_i^2{}_{max} - d_i^2{}_{min}\right)$$

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H_i = Head at the reduction, in m

$d_{i_{max}}$ = largest diameter of the pipe, in m

$d_{i_{min}}$ = minimum diameter of the pipe, in m

Example:

Calculate anchor A1 (outside type) using the penstock profile data from the previous example. Use a design flow rate of 0.400 m^3/s and PVC for the penstock material, the diameter and thickness of which were calculated earlier.

FIGURE 49. F_9 TOWARDS THE REDUCTION

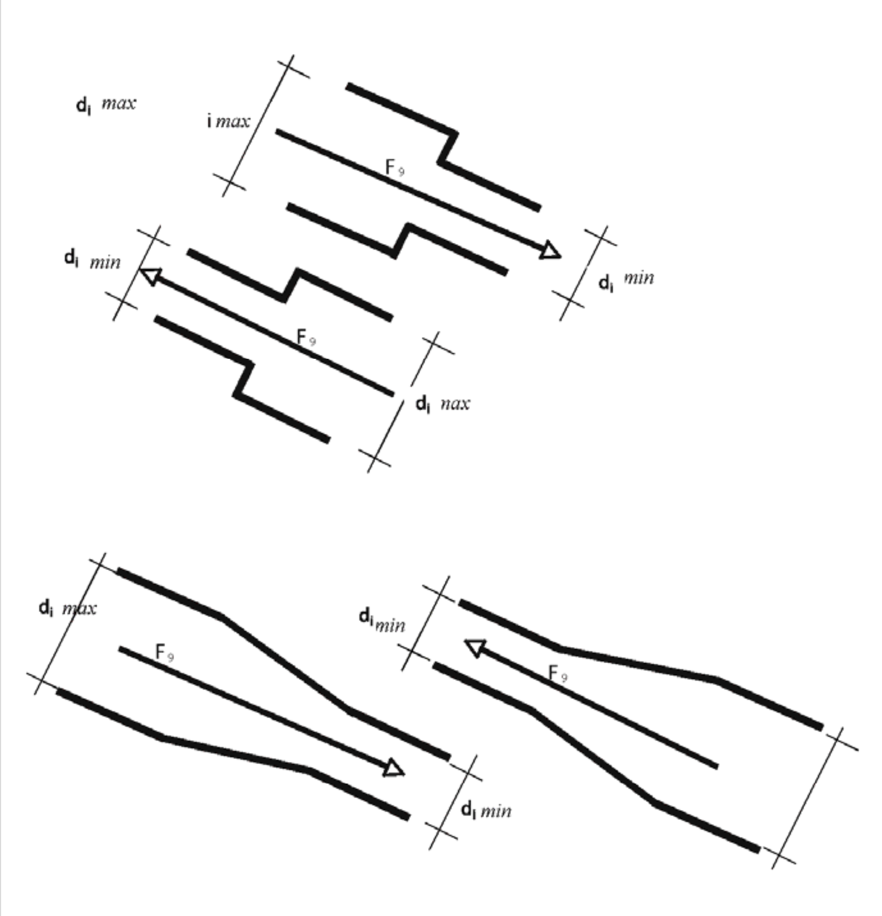
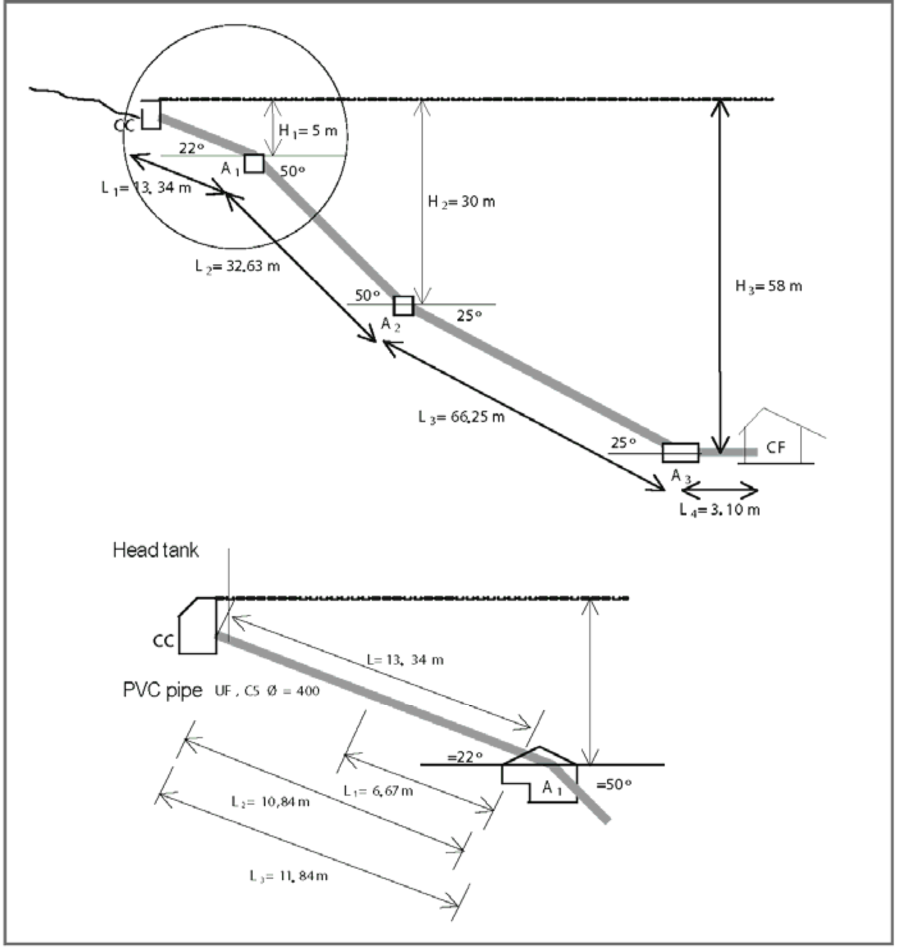


FIGURE 50. DETAILS FOR CALCULATING ANCHOR A1



Data:

$Q = 0.400\ m^3/s$

From the penstock profile:

$H = 5{,}00\ m$

$L = 13.34\ m$

$L_1 = 0.5\,L = 6.67\ m$

$L_2 = L - 2.50\ m = 10.84\ m$;

(2.50 m = 1 from the joint with the bend and 1,50 m before entering the head tank there is a Tee to which the vent pipe is connected)

$L_3 = L - 1.50 = 11.84\ m$

$\alpha = 22º$;

$\beta = 50º$

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Calculated characteristics of the pipe

$d_i = 0{,}38$ m

$D_e = 0{,}40$ m

$t = 9{,}8$ mm

$\gamma_T = 1{,}450$ kg-f/m^3 specific height of pipe

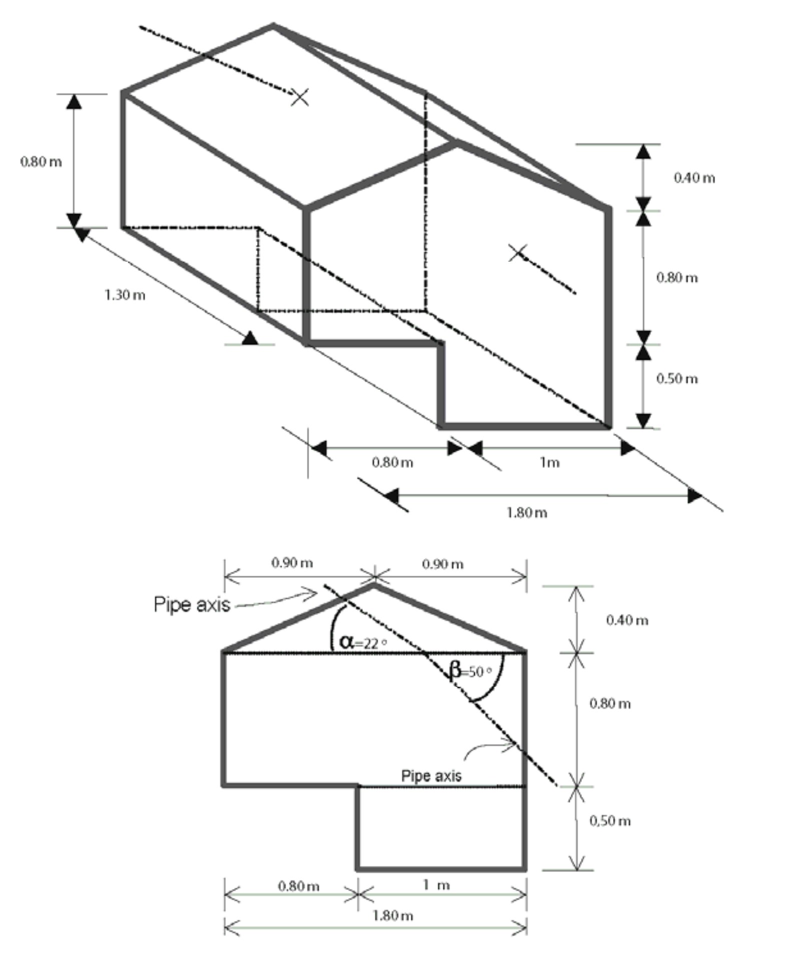
$\mu = 0{,}4$ coefficient of friction between PVC and the soil

Soil characteristics:

$\sigma_T = 1$ kg-f/cm^2:

a) Choosing the shape and dimensions of the anchor. See Figure 51.

FIGURE 51. DIMENSIONS OF THE ANCHOR



b) We must verify that the dimensions chosen meet the three conditions of equilibrium.

Calculating the weight of the penstock and water per linear metre:

$W = Wt + Wa$

1) $$Wt = \gamma_t \times \frac{\pi}{4}\left[de^2 - di^2\right] = 1.450 \times \frac{3.1416}{4}\left[0.40^2 - 0.38^2\right] = 17.75 kg - f/m$$

2) $$Wa = \gamma_a \times \frac{\pi}{4} \times d_i^2 = 1.000 \times \frac{3,1416}{4} \times 0.38^2 = 113.35 kg - f/m$$

$W = 131.10$ kg-f/m

Calculating the forces acting on the anchor (in kg-f)

$F_1 = W.L_1.\text{Cos } 22° = 131,10 \times 6.67 \times \text{Cos } 22° = 810,76$

$F_2 = \mu.W.L_2.\text{Cos } 22° = 0.4 \times 131,10 \times 1.084 \times \text{Cos } 22° = 527,05$

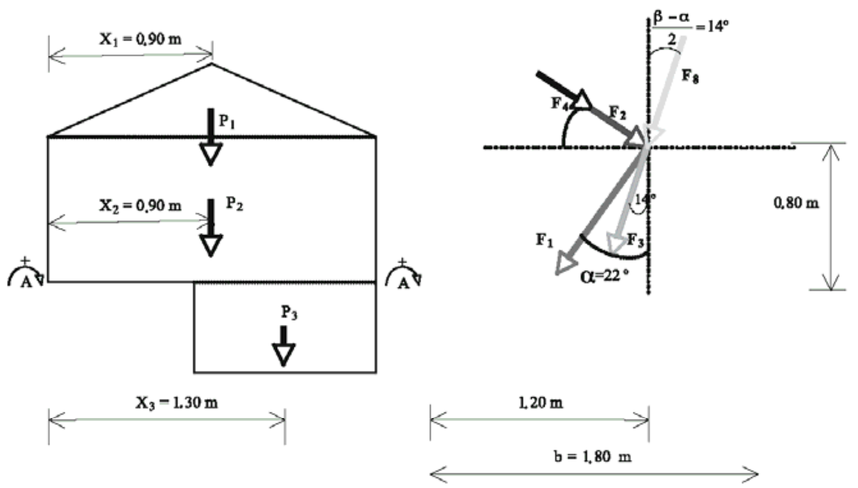
$F_3 = 1.6 \times 10^3 \times Hi \times d_i^2 \times \text{Sin}(50° - 22°)/2 = 1.600 \times 5 \times 0.38^2 \times \text{Sin } 14° = 279,46$

$F_4 = Wt \times L_4 \times \text{Sin } 22° = 17,75 \times 11,84 \times \text{Sin } 22° = 78,72$

$F_8 = 250(Q/d_i)^2 \times \text{Sin }(50° - 22°)/2 = 250(0.40/0.38)^2 \times \text{Sin } 14° = 67$

As the pipe is made of PVC, the forces F_5, F_6, F_7 and $F_9 = 0$

FIGURE 52. EQUILIBRIUM CONDITIONS OF THE ANCHOR



P_1, P_2 y P_3 = = Partial weights of concrete anchor and diagram of forces when the pipe dilates

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1) The elbow or point where the direction changes and where the forces act, is located horizontally at (2/3)b and vertically at distance d from point A.

2) The auxiliary calculations for the concrete block, in order to verify stability, are summarised in the following chart:

TABLE 17. AUXILIARY CALCULATIONS TO VERIFY THE STABILITY OF THE CONCRETE BLOCK

N.°	Area m^2	Volume m^3	Weights: Pi Kg-f	XI. m	Ai.Xi m^3.m	Pi.Xi kg-f.m
1	0.36	0.468	1029.6	0.90	0.324	926.64
2	1.44	1.872	4118.4	0.90	1.296	3,706.56
3	0.50	0.65	1430.0	1.30	0.65	1,859.0
Total	2.30	2.99	6578.0		2.27	6,492.2

Example one when the pipe dilates

Verification of stability:

1) Against overturning: $\Sigma M_{stabiliser} / \Sigma M_{over\ turn} > 2$

$\Sigma M_{stabiliser}$ in $A = P_1.X_1 + P2.X_2 + P_3 \times X_3 + F_1.Cos\ 22° \times 1.20 +$

$(F_3 + F_8)\ Cos 14° \times 1.20 + (F_2 + F_4)\ Sin\ 22° \times 1.20 = 8{,}069.74$ kg-f.m

$\Sigma M_{over\ turn} = -F_1 \times Sin\ 22° \times 0.80 - (F_3 + F_8)\ Sin\ 14° \times 0.80 + (F_2 + F_4)\ Cos\ 22° \times$ 0,80 = 138.43 kg-f/m 8,069.74/138.43 > 258 > 2 ….. Ok

2) Against slippage: $\Sigma Fx < \mu.\Sigma Fy$

$\Sigma Fx = -F_1 \times Sin\ 22° - (F_3 + F_8)\ Sin\ 14° + (F_2 + F_4)\ Cos\ 22° = 173.67$ kg-f

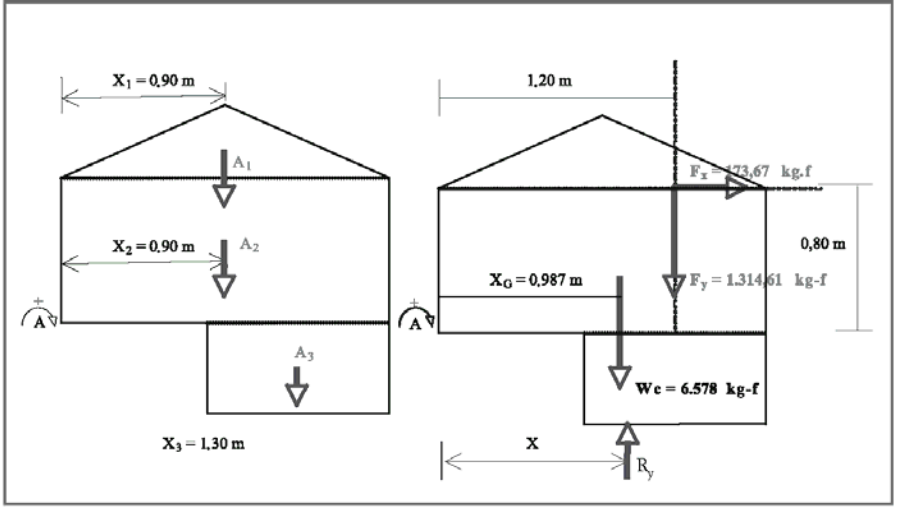
$\Sigma Fy = -P_1 - P_2 - P_3 - F_1.Cos\ 22° - (F_3 + F_8)\ Cos\ 14° - (F_2 + F_4)\ Sin\ 22° = -7{,}892.61$ kg-f

Verifying:

173.67 < 0.5 × 7,892.61 kg.f

173.67 < 3,946.30 .... Ok

FIGURE 53. PARTIAL AREAS AND RESULTING FORCES WHEN THE PIPE DILATES



3) Soil stability

Calculating Xg for the concrete block, using the method of areas

$$Xg = \frac{A_1 \cdot X_1 + A_2 \cdot X_2 + A_3 \cdot X_3}{\sum Ai} = \frac{2,27}{2,30} = 0.987m$$

Calculating X and R_y

$\sum M_A$: $-Ry.X + 6,578 \times 0.987 + 173.67 \times 0.80 + 1,314.61 \times 1.20 = 0$

$Ry.\underline{X} = 8,208,9$ kg-f.m ….. (1)

Ry = Wc + Fy = 6,578 + 1,314.61 = 7,892.61 kg.f

X = 8,208,9/7,892.61 = 1.04 m

Calculating eccentricity e = X – b/2

e = 1,04 – 1,80/2 = 0,14 m

Calculating the maximum and minimum load capacity of the soil:

$$\sigma = \frac{Ry}{A}\left[1 \pm \frac{6xe}{b}\right] = \frac{7.892,61}{180 \times 130}\left[1 \pm \frac{6 \times 0,14}{1,80}\right] = 0,3372\left[1 \pm 0,46\right]$$

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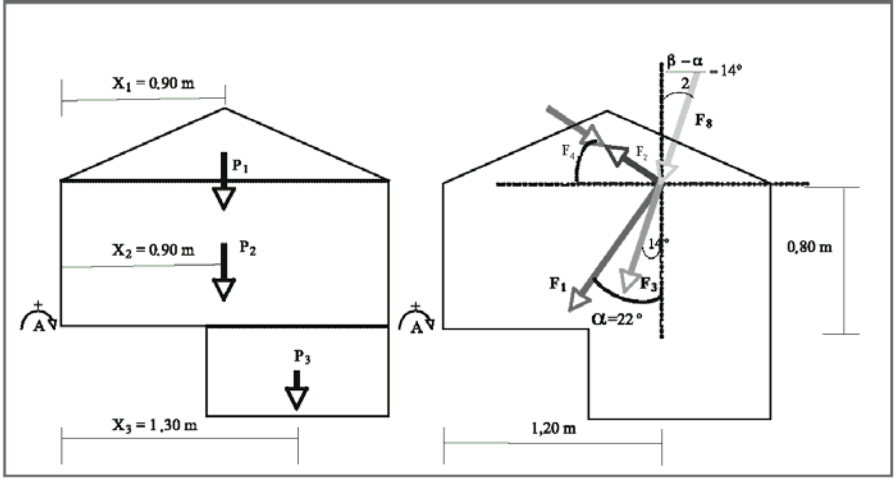
$\sigma_{max} = 0.3372(1.46) = 0.49$ kg-f/cm^2

$\sigma_{min} = 0.3372(0.54) = 0.18$ kg-f/cm^2

We have proved that: 1 kg-f/cm^2 > 0.49 kg-f/cm^2 > 0.18 kg-f/cm^2 ... Ok

These calculations show that the anchor is stable. We now have to prove that it is also stable for the second case; in other words, when the pipe contracts.

FIGURE 54. PARTIAL WEIGHTS OF THE CONCRETE ANCHOR AND FORCES ACTING WHEN THE PIPE CONTRACTS



Example two: when the pipe contracts

The forces acting on it are the same. Only force F_2 changes direction.

Verification of stability

The same steps are taken as in the previous case. The results are:

1) Against overturning $\frac{\sum M \ stabilizer}{\sum M \ overturnning} \rangle 2$

$$11.83 > 2$$

2) Against slippage: $\sum F_x < \mu . \sum F_y$

$$802.92 < 0.5 \times 7{,}496.18$$

$$802.92 \text{ kg-f} < 3{,}748.09 \text{ kg-f}$$

3) Soil stability: $R_y = 7{,}496.15$

$X = 0.93$

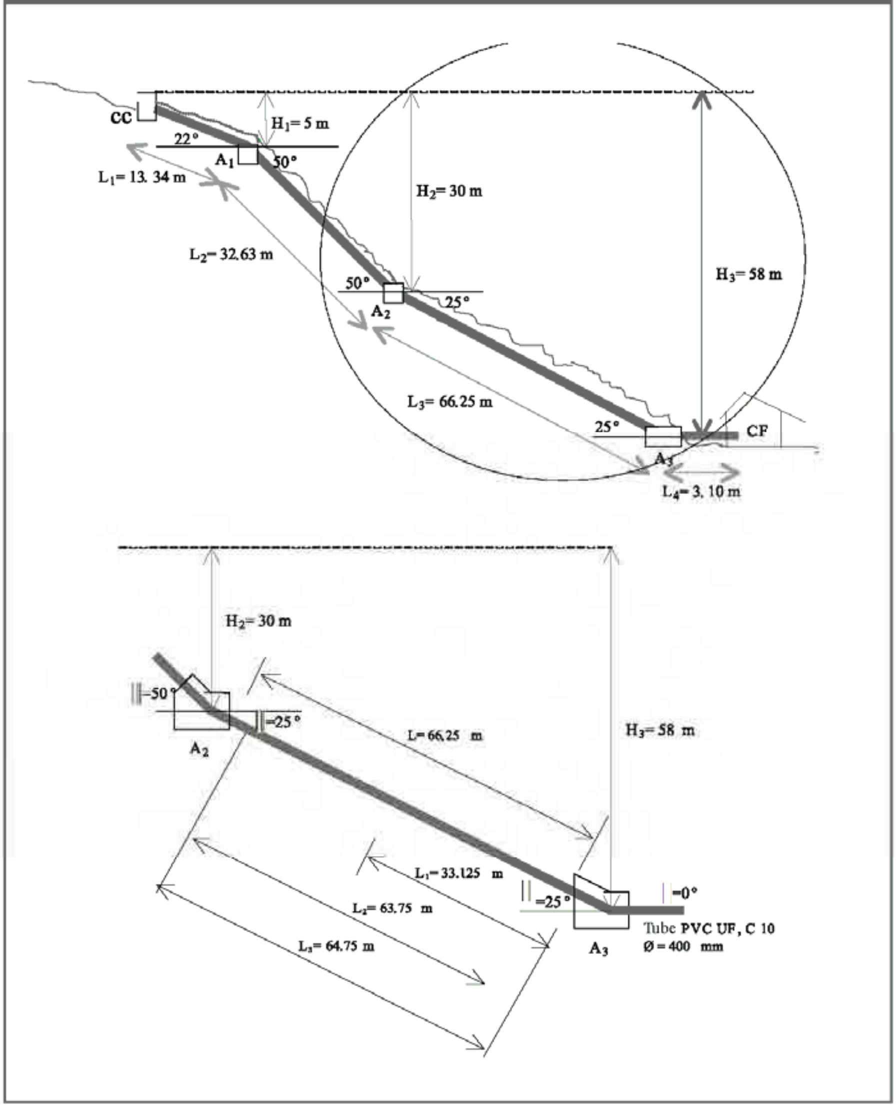
$e = X - b/2 = 0.927 - 0.90 = 0.027$ m

$\sigma_{máx} = 0.35$ kg-f/cm²

$\sigma_{mín} = 0.29$ kg-f/cm²

We have proved that 1 kg-f/cm² > 0.35 kg-f/cm² > 0.29 kg-f/cm² …Ok

FIGURE 55. DETAILS FOR CALCULATING ANCHOR A3



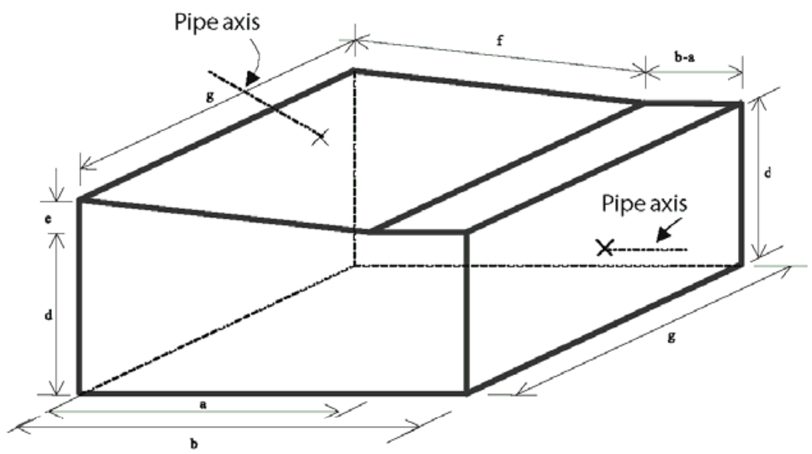
Inside anchor

Anchors 2 and 3 of the penstock profile are inside type.

The stability calculation is performed as with the outside anchor and the same formula are used when the pipe dilates and contracts.

A) PRE-DIMENSIONING THE PROFILE IN ORDER TO PROVE THE STABILITY CONDITIONS.

FIGURE 56. DIMENSIONS OF THE ANCHOR IN ORDER TO PROVE ITS STABILITY



B) THE WEIGHT OF THE ANCHOR AND THE FORCES ACTING ON IT ARE CALCULATED.

FIGURE 57. WEIGHTS OF THE CONCRETE AND FORCES ACTING ON IT

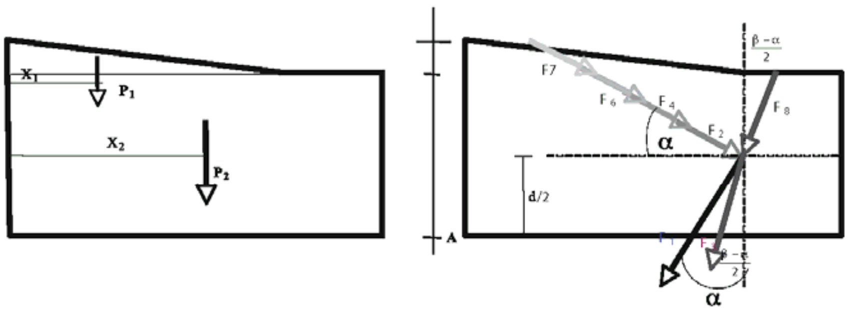
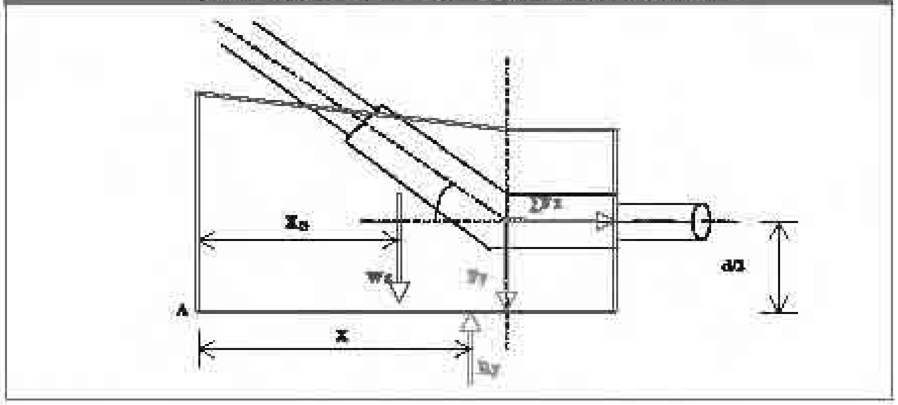


FIGURE 58. FORCES RESULTING, FOR SOIL STABILITY CALCULATION



5.1.6 Head Tank Spillway

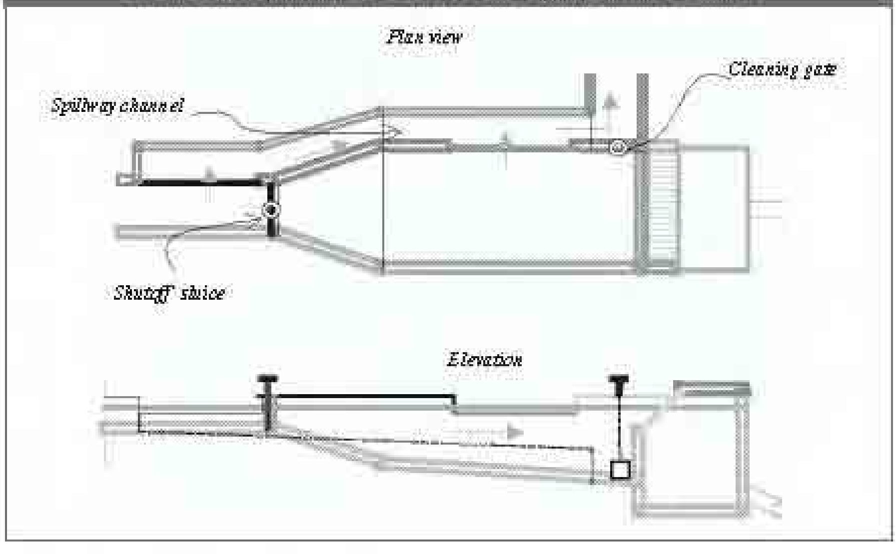
The head tank spillway is designed to evacuate excess water from the chamber under normal conditions, or all the water when the turbine is stopped in an emergency.

The spillway consists of a channel which receives water from the head tank under the above conditions, through overflow from the head tank or from the inlet channel. This channel discharges in a safe area to prevent flooding and soil erosion adjacent to the head tank. Usually, the water is returned to the river.

The spillway calculations follow the procedure developed for the channels, the difference being that the slope varies with the terrain.

The following scheme indicates the location of the spillway channel as complementary and necessary infrastructure for the head tank.

FIGURE 59. SPILLWAY AND HEAD TANK OVERFLOW, IN PLAN AND ELEVATION



5.1.7 Tailrace

The tailrace takes the water from the base of the turbine to the river. If spent water is not returned to the river it may be discharged where it will not erode the surrounding soil. This may be a creek or existing channel designed for irrigation or other purposes, such as fish farming, drinking water, etc.

To ensure normal operation of the tailrace we must choose adequate levels for the bottom of the race at the start and finish, so that the river level in the rainy season is always lower than the bottom at the end of the channel.

The design of the tailrace can be the same as that of the inlet channel, as it carries the same flow rate. In most cases the cross section can be smaller than that of the inlet channel and can take advantage of the terrain to increase the slope; the water velocity should be less than 3 m/s.

It is recommended that concrete be used as it will withstand higher velocities without suffering from erosion. Under other circumstances piping should be used for the tailrace, with a water depth equal to 0.75Di, as recommended by the manufacturers.

Example:

Flow rate of discharge: 0.400 m^3/s

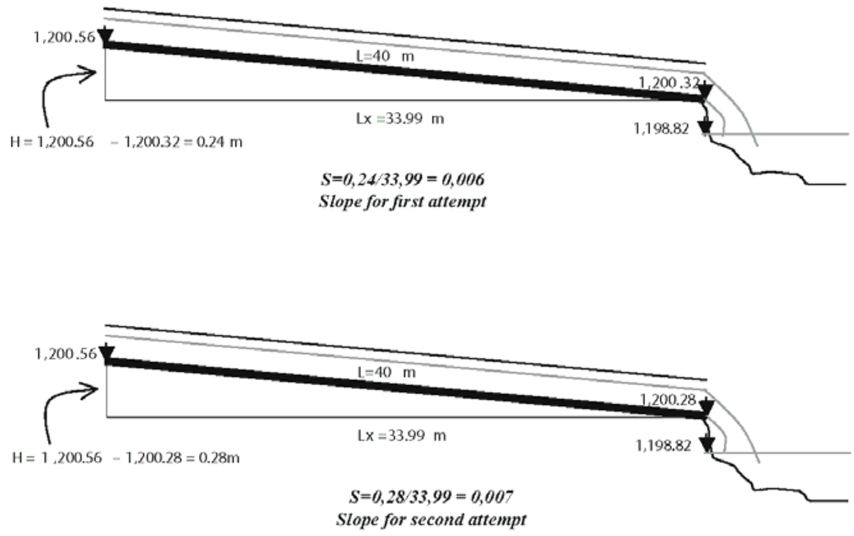
Contour of bottom of tailrace: 1,200.56 m

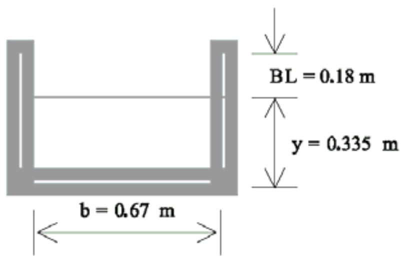
Contour of bottom at end of tailrace: 1,200.32 m

Level of water in the river during the rainy season: 1,198.82 m

Length of bottom of tailrace = 40 m

FIGURE 60. DESIGN OF THE TAILRACE





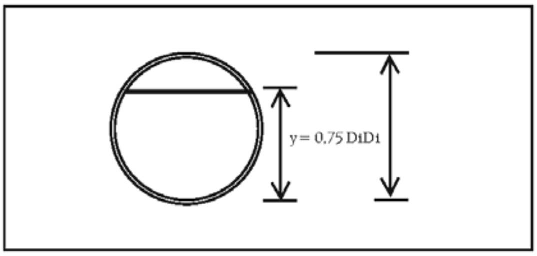
Solution:

The first attempt considers a workable slope of 6 :1000. The second attempt considers a slope of 7:1000 to obtain a minimum of 0.400 m^3/s and because the height of water in the river allows this. We choose the second calculation as valid for a rectangular channel as shown in the figure, to carry a minimum of 407 m^3/s.

Attempt	b (m)	y (m)	A (m^2)	P (m)	R (m)	$R^{2/3}$	n	1/n	S	$S^{1/2}$	V (m/s)	Q (m^3/s)
1	0.67	0.325	0.224	1.34	0.168	0.304	0.014	71,42	0,006	0.077	1.68	0.377
2	0.67	0.335	0.224	1.34	0.168	0.304	0.014	71,42	0,007	0.0836	1.81	0.407

If PVC pipe is to be used for the tailrace, the diameter would be:

$$Q = 28.4 x Di^{8/3} x S^{1/2}$$



Di = (Q/28.4 x S1/2)3/8

Di = (0.400/28.4 x 0.007 ½)3/8

Di = 51.12 m

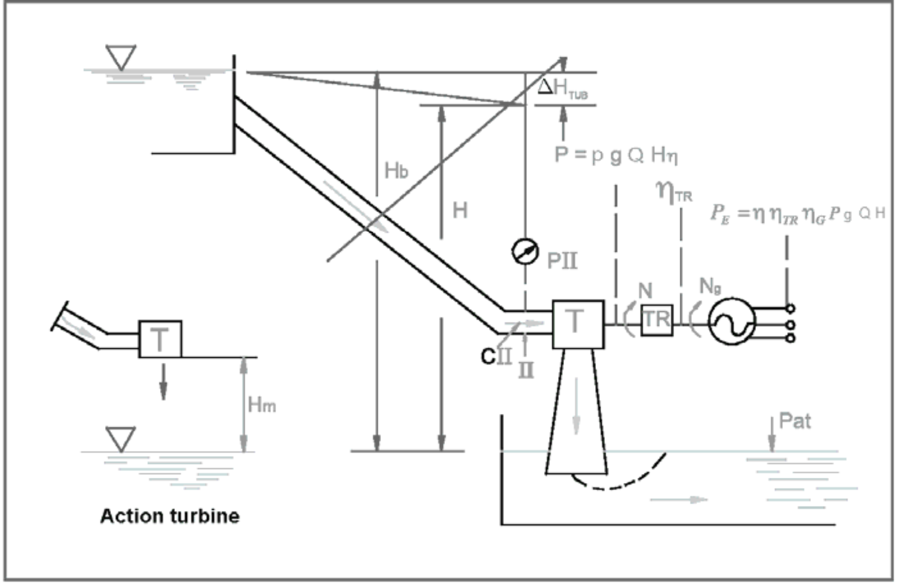
5.2 Electromechanical Design

5.2.1 Preliminary Design of the Turbines

(1) Turbine selection

In this second stage of the selection procedure, data from the location are compared with the preliminary data, when we have an idea of the type of turbines that are to be used. Afterwards, the dimensions of the turbine are calculated taking into account the following considerations.

FIGURE 61. ARRANGEMENT OF A HYDROPOWER GENERATING SET



(2) Turbine power output

The following equations are used:

$$P_E = P.\eta_{TR}.\eta_G$$

$$P = \frac{\gamma Q H \eta}{K} = \frac{P_E}{\eta_{TR}.\eta_G} = \frac{QH\eta}{102}$$

$$\eta_{GR} = \eta.\eta_{TR}.\eta_G$$

where

η_{GR} = Efficiency of the generating set (adimensional)

K = Constant K = 1.000 W/kW

H = Height in metres

Q = flow rate in L/s

Regarding the determination of the effective head, the following procedure may be used:

Reaction turbines H = $H_T - H_m$

Action turbines H = $H_b -$ $H_T - H_m$

Where:

H_b = Gross head, m

ΔH_T = Head loss in the penstock, m

H_m =Height of turbine installation

If no direct information on the efficiency of the turbine or generator is available, the values from the following table may be used for the efficiency of the generating set .

TABLE 18. EFFICIENCY OF DIFFERENT TYPES OF TURBINE				
Power (kW)	**TURBINE**			
	Pelton	**Michell-Banki**	**Francis**	**Axial**
<50	58-65%	54-62 %	59-65%	58-66 %
51-500	65 - 69	62 - 65	66 - 70	66 - 70
501-5000	69 - 73	65*	70 - 74	70 - 74

(3) Specific number of revolutions

The design and construction of hydraulic turbines involves solutions to a series of problems that cannot always be addressed mathematically and which have to be resolved experimentally using models. Models enable the cost of experimental trials to be reduced and give better cost control and accuracy. Water or air can be used as the experimental fluid. The model enables us to verify the theoretical calculations before building the prototype (full size turbine) and reveals the improvements needed to achieve the desired performance. The relationship between model and prototype requires the establishment of laws of similarity (geometrical, kinematic and dynamic) from the analysis of which we obtain characteristic figures. Among these are the specific speeds, which best express the similarity between model and prototype.

These numbers are as follows:

Specific speed for flow rate, or Brauer number (N_q)

$$N_q = \frac{N\sqrt{Q}}{H^{3/4}}$$

Specific speed for power output, or Camerer number (N_s)

$$N_s = \frac{N\sqrt{P}}{H^{5/4}}$$

Where:

N_q and N_s = Specific speed, RPM.

N, Turbine rotation speed, RPM.

Q, Turbine flow rate, m^3/s

H, Effective head, m.

P, Power at turbine shaft, HP or kW.z.

Both numbers can be used indistinctly, but for hydraulic turbines we prefer to continue using the number N_s.

$$N_s = N.\frac{\sqrt{\frac{\rho g Q H \eta}{K}}}{H^{5/4}}$$

The relation with N_q is as follows:

$$N_s = N.\frac{\sqrt{\frac{\rho g Q H \eta}{K}}}{H^{5/4}} \times N_q$$

For large turbines:

$$N_s = 3{,}040\ N_q$$

For small turbines:

$$N_s = 3{,}03\ N_q$$

The specific speed can be defined as the speed of rotation of a prototype turbine having a model which operates under unitary parameters.

Thus the number N_q rrepresents the speed of rotation in RPM of the model when:

$$Q = 1\ m^3/s \ \text{y}\ H = 1.0\ \text{m}$$

Similarly, N_s epresents the speed of rotation RPM of the model when:

$$P = 1\ \text{HP ó 1 kW. y}\ H = 1\ \text{m}$$

It is important to point out that the valuer N_q or N_s will depend on the system of units used, given that in the imperial system, the SI system or any other, the values are different.

Given that the specific speed represents the speed of rotation, it is customary to classify turbines as "slow", "normal" or "fast", however it should be clearly understood that these terms do not refer to the real speed of rotation of the turbine, but to the head and flow rate conditions.

(4) Pelton turbine

The principal dimensions of the turbine are shown below:

- Velocity of the jet issuing from the nozzle. Being an action turbine where all the effective head is converted into kinetic energy, the velocity will be:

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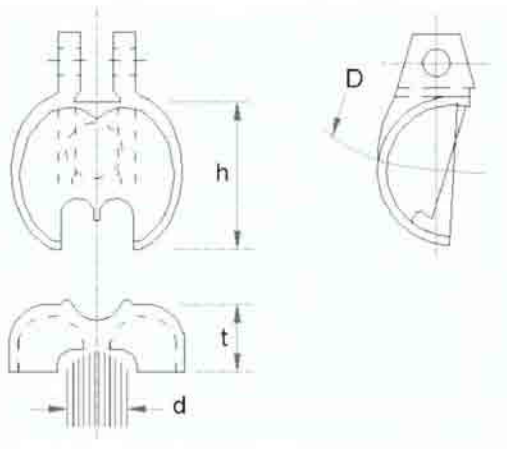
FIGURE 62. DIMENSIONS OF PELTON TURBINE



b= 2.8 a 3.2 d
h= 2.8 a 3.2 d
t= 0.8 d

$$C = \phi\sqrt{2gH}$$

Where:

$$\phi = \sqrt{1 - \frac{\Delta H_i}{H}}$$

In such equations, C is in m/s, Ø is the velocity coefficient, which depends on nozzle losses ΔHi en m; its value varies between 0,95 and 0,99.

- Diameter of jet. This value is measured in the window:

$$d = 0.55\left(\frac{Q}{\sqrt{H}}\right)^{1/2}$$

The diameter d is given in m, and Q in m^3/s. This equation is valid for an average velocity coefficient of Ø = 0.97

- Diameter of Pelton wheel. This diameter is the mean circumference of the buckets, tangential to the mean line of the jet:

$$D = (37\ a\ 39)\frac{\sqrt{H}}{N}$$

In this expression, D is in m, N in RPM and it is valid for Ø = 0,97 and a total average efficiency of η = 0,88. Low values of the coefficient are assumed for turbines with a high N_s and high values for those with a low N_s.

- D/d ratio for a jet:

$$D/d = 7.0 \; para \;\; N_s = 30$$

$$D/d = 15.0 \; para \;\; N_s = 15$$

EIntermediate values can be interpolated assuming an approximately linear variation. It is worth mentioning that in Turgo turbines the ratio D/d is 4.0, which enables them to work with higher flow rates and this constitutes an advantage for the type.

- Specific velocity. This equation is valid for Ø = 0.97 and η = 0.88

$$N_s = 240(\frac{d}{D})$$

- Number of buckets:

$$Z = \frac{1}{2}(\frac{D}{d}) + 14 \; a \; 16$$

- Minimum height:

$$Hm = 10d + \frac{H}{200}$$

Where d and H are in m.

- Basic bucket dimensions. Low values are employed when the maximum efficiency is achieved at partial load and high values when maximum efficiency is required to be achieved under full load (see Figure 62).

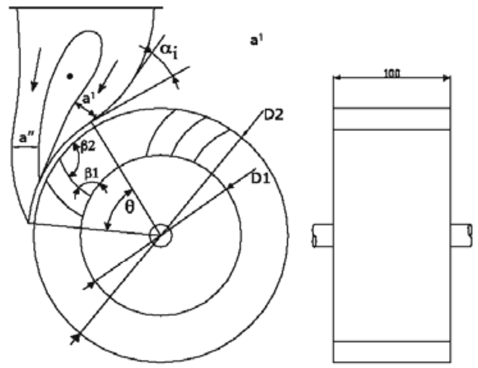
(5) Michell Banki turbine

The principal dimensions of the turbine are shown below:

- Jet velocity. Given the proximity between the injector and the wheel, a slight, insignificant, overpressure in the space under the in let arc, given by angle θ. Then:

$$C = \phi\sqrt{2gH}$$

FIGURE 63. DIMENSIONS OF A MICHELL BANKI TURBINE



$$a = a' + a''$$

The coefficient Ø has the same meaning as for Pelton turbines and may be considered as about 0.95

- Jet thickness: $a = K_a D_2$

Where a is in m, K_a it is a coefficient that depends on the angle of the nozzle and the angle of admission θ. For $\alpha_i = 16°$, the following values can be used:

θ°	60°	90°	120°
Ka	0.1443	0.2164	0.2886

If a central guide vane is used:

$$a = a' + a''$$

- Outside and inside diameter:

$$D_2 = (37\ a\ 39)\frac{\sqrt{H}}{N}$$

$$D_1 = 0.66 D_2$$

Low values of the coefficient correspond to fast turbines (with wide runners) and high values to slow turbines (with narrow runners). Runner diameters of 200, 300 and 400 mm are recommended.

- Width of the runner ($\alpha_i = 16°$):

$$B = 98\ 8\frac{Q}{D_2\sqrt{H}}\frac{1}{\theta}$$

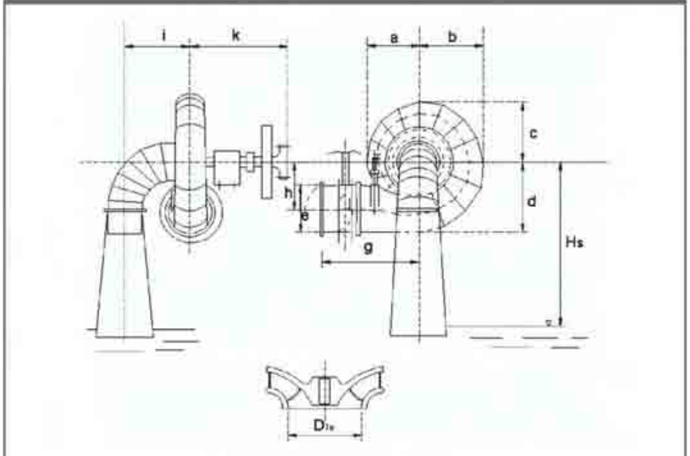
Nozzle angle: $\alpha_1 = 15°\ a\ 20°$

- Number of blades. Varies between 24 and 30 blades depending on the size of the runner.

(6) Francis turbine

Calculating the dimensions is more laborious for the Francis turbine. Nevertheless, we can estimate the general dimensions in accordance with the following figure:

FIGURE 64. DIMENSIONS OF A FRANCIS TURBINE



Basic dimensions

a	b	c	d	e	f	g	h	i	k
1.5	1.9	1.7	2.0	1.3	1.05	2.9	1.4	2.0	3.8

Note: To obtain the actual dimensions multiply the values of the table times the diameter of the runner D1e

5.2.2 Selecting the Generator

As we have mentioned, electrical generators recommended for micro hydropower units are of 1.800, 1.200 and 900 RPM. The lower the speed the higher the cost. The following specifications must be given to the manufacturer on ordering:

- The shaft power of the turbine
- That it will operate with a hydraulic turbine. It requires 2 bearings
- Working altitude in m.a.s.l.
- Number of poles
- Frequency of 60 Hz in Peru
- Number of phases
- Operating voltage
- Type of connection

For mini hydropower units, selection of the generator depends on the power and an evaluation of costs, and also takes into account the standard power information provided by the manufacturers.

For units of less than 100 kW, which cover micro hydropower units, if the turbine speed is not the same as the generator speed, then 1.800 RPM generators can be used. These cost less and are geared to increase speed, using belts, gearing, etc. For power ranges greater than 200 kW, the type of transmission needs to be analysed, and the respective resistance calculations worked out; if not, the turbine must be designed in accordance with the synchronism of the generator.

In order to generate an adequate voltage, modern generators are fitted with terminals so that changes can be made in accordance with the agreed voltage, using either star or delta connection.

5.2.3 Choosing the Speed Regulator

The speed regulator should be chosen taking into account that it has to be suitable for the generation characteristics. In mini hydropower installations with power outputs of less than 200 kW, dummy load electronic regulators are recommended; the energy that is not consumed by the primary load is diverted to the secondary load where it is dissipated through resistances that are generally water cooled. If picoturbines are used these resistances can be overhead-type.

As can be seen in the previous chapter, electronic load regulation has advantages for micro hydropower units. For units greater than 200 kW power output, mixed-type velocity regulation is recommended; in other words, coarse, synchronised electronic regulation of the flow rate. According to the main load evaluation (energy demand), if the variation in demand is not significant over 24 hours, electronic regulation capacity will be less than for higher variations in demand. For example, in the case of 300 kW installed capacity and energy demand that varies between 200 and 300 kW, fine regulation (electronic regulation) will be 100 kW or slightly more and flow rate regulation will take effect when demand falls below 200 kW, in extreme cases reducing the flow rate to 100 kW, which would be diverted to secondary load through the electronic load regulator.

Selection of an electronic load regulator should take into account the generator voltage, type of connection and whether single phase or three-phase electricity is being generated.

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