# Design calculations for the civil works and the practical implementations for an 8 kW - to 22 kW - upgrade of a micro hydropower run-off river scheme in Nepal

## This paper comprises the implementation of: "Two Ideas to Regain the Power Otherwise Lost in Run-off River Schemes with Elevated Powerhouses" (A paper available from Practical Action Publishing 2025)

Provided by:
Steen Carlsen
11 Regenburgsgade
8000 Aarhus C, Denmark.
Tel. ++45 23-63 69 68
E-mail: carlsen@power-electronics.dk

# Calculation and Design of the Micro Hydropower Plant of Katunje village district of Kavbre, Nepal 2002

## Partly Sponsored by Danida

Provided by:
Carlsen Power Electronics
The Science Park, Gustav Wiedsvej 10
8000 Aarhus C, Denmark.
Tel. ++45 86-19 03 19
E-mail: carlsen@power-electronics.dk

&

Hyonjan Electrical Engineering
Fabricator
Swoyambhu Ring Road,
Kathmandu, Nepal
Tel: 00977-1-270 / 270 991;
E-mail: hyonjan@wlink.com.np
2002 May 21.

# Contents



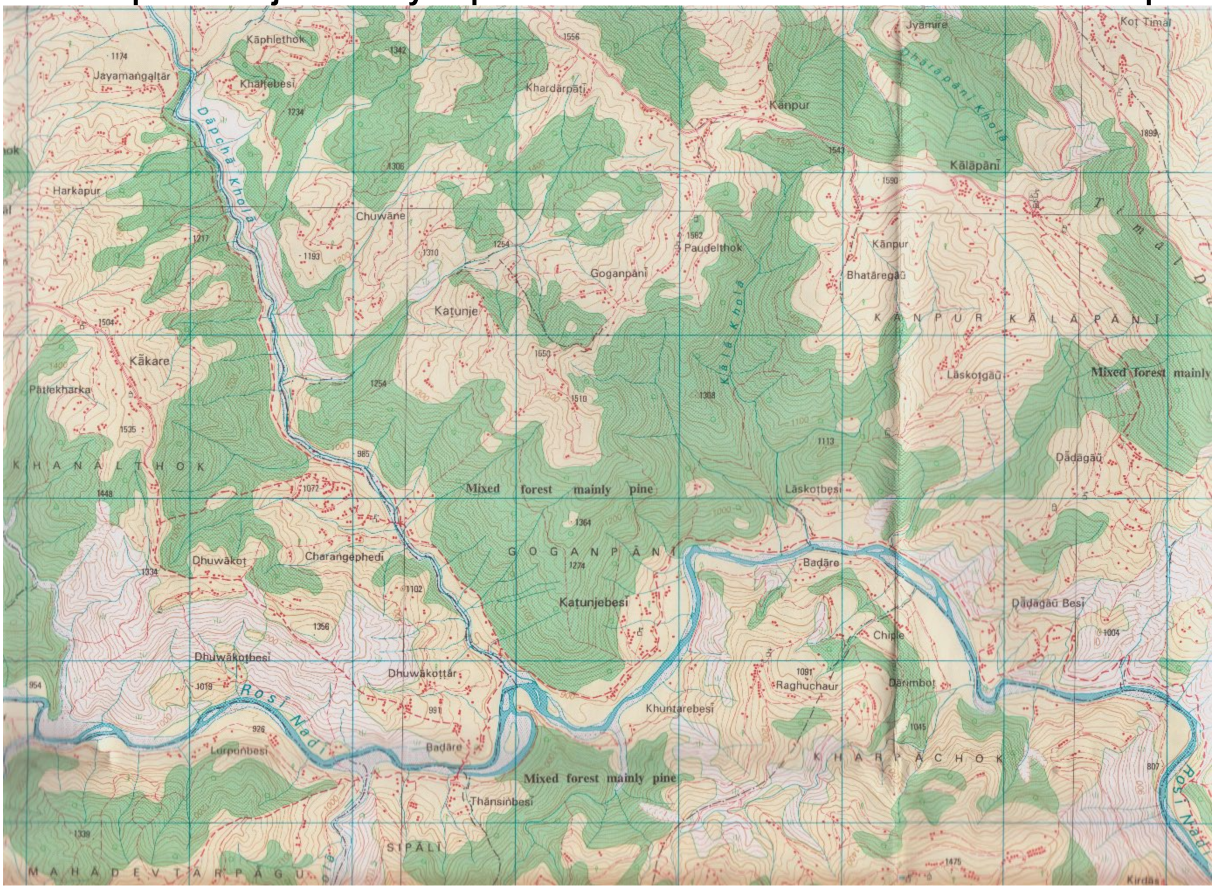
Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

## 0. Abstract

This paper contains the interrelations, design calculations and the trade-offs for an upgrade of an 8 kW - to a 22 kW - micro hydropower run-off river schemes in the village of Katunje in Nepal. The design took place in 2001 and 2002 and was based on the paper: “Two Ideas to Regain the Power Otherwise Lost in Run-off River Schemes with Elevated Powerhouses” (available from Practical Action). The present paper addresses the interactions and the practical implementation of the various civil works and components of a micro hydropower scheme, from the water intake, through the penstock, etc., the powerhouse with the draft tube, the tail race and back to the river.

In 1998 - prior to this project - I completed a 22 kW hydropower project in Denmark. In 2002 - due to the civil war in Nepal and other political issues, Danida (the Danish Governmental Aid Organization) canceled the implementation of the present project.

## 1. Map of Katunje Micro Hydropower Plant - and Data from REDP for the Former plant

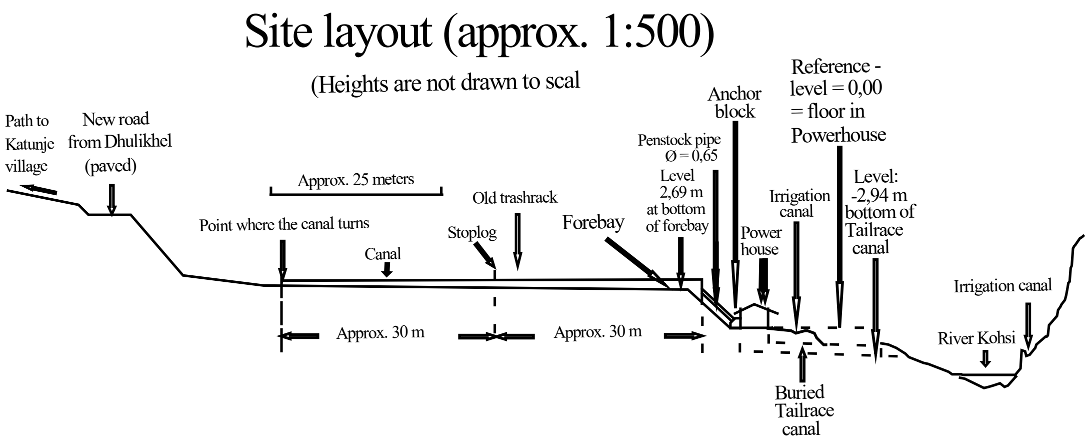



Map of the village of Katunje; Nepal.    Powerhouse = red dot = 27º31”05’ N & 85º41”00’ E.
60 metre North, Northwest of River Rosi Nadi.  (Approx. 70 km East of Kathmandu.)

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

In "Information on Roshi Khola Micro Hydro Scheme at Katunjebesi, Katunjebesi VDC, Kavbre Palanchok District" - on page 3 - the data for the old micro hydropower plant of Katunje are presented:

| | |
|---|---|
| Gross head: | 5 m |
| Design flow: | 400 l/s |
| Power Output: | 8 kW |
| Total power utilized: | 7 kW (6 pm. to 6 am.) |
| Beneficiary households: | 46 |
| Weir: | Temporary. |
| Headrace canal: | 650 m (length of power canal (measured by Steen Carlsen)) |
| Other civil structures: | Gravel, trap settling basin forebay. |
| Penstock: Specifications: | Mild steel, 380 mm Outer Diameter, thickness = 4,5 mm. |
| Length: | 13,24 m. |
| Tailrace canal: | 10 m. |
| Turbine: | Static head = 4,5 m Design flow 400 l/s; Propeller. |
| Generator: | 13 KVA, single phase 220 V Asynchronous. |
| Controller: | IGC, 14 kW with Ballast tank. |
| Distribution lines: | Wooden poles. |
| Total length: | 1410 m. |
| End uses: | Lighting, Agro-processing. |
| Tariff: | Rs 1/ Watt/Month (Rate 2002: 1 DKr = 8,5 NRs (Steen Carlsen)). |
| Test run: | Baishakh 20, 2055 (Converted from the Nepali calendar = 4. May 1998) |
| Inauguration Date: | July 16, 1998 |




**2. Hydrologic Data Recorded April/May 2002**

During April and May 2002 subsequent measurements of the discharge were made: (Using the methods stated in the table below):

| Place of Measuring | Rod (kinetic) | Ossberger (bottle) | Canal Slope |
|---|---|---|---|
| Intake – velocity | 1.53 m/s | 1.34 m/s | - |
| Intake – discharge | 851 l/s | 745 l/s | - |
| Forebay – velocity | 1.25 m/s | 1.33 m/s | 1.72 m/s |
| Forebay – discharge | 469 l/s | 441 l/s | 702 l/s |

### 2.1. The Discharge just after the Intake

*Discharge measured according to Ossberger's method.*

On 28/4 2002 the velocity of the flow of the power canal was measured just after the intake. At a fairly straight part of the power canal, a distance of 10 meters was measured, and the start and the end points were marked. A plastic bottle 2/3 filled with water was placed at the middle of the flow a few meters upstream of the first mark. The time for the bottle to cover the distance (10 m) from the first mark to the second mark was measured to be

7.42 s.

The velocity of the bottle was calculated to be: V = 10 m / 7.42 s

= 1.34 m/s.

A translation (from German and French) is provided in Ossberger Turbinen Fabrik's guidelines on how to find the Flow Q in canals of different materials, as follows:

$$Q = v * A * C$$

The input parameters being:

v = The velocity of a floating object placed in the middle of the stream.

A = Area (cross section) of water in the canal, (i.e. the wetted area of the canal).

U = Length (peripheral) of the part of the cross-section of the canal, which is filled / in touch with the water (i.e. the perimeter of the wetted area of the canal).

C = is a constant. The table provides the coefficient C as function of the ratio between A and U.

(The original text uses F as designator for Area, because the German word for Area is "Fläche").

| A/U [m] | Even Wood or Concrete [m] | Unplaned wood or brick wall [m] | Wall of boulders [m] | Soil [m] |
|---|---|---|---|---|
| 0.1 | 0.860 | 0.840 | 0.748 | 0.565 |
| 0.2 | 0.865 | 0.858 | 0.792 | 0.645 |
| 0.3 | 0.870 | 0.865 | 0.812 | 0.685 |
| 0.4 | 0.875 | 0.868 | 0.822 | 0.712 |
| 0.5 | 0.880 | 0.870 | 0.830 | 0.730 |
| 0.6 | 0.885 | 0.871 | 0.835 | 0.745 |
| 0.7 | 0.890 | 0.872 | 0.837 | 0.755 |

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| | | | | |
|---|---|---|---|---|
| 0.8 | 0.892 | 0.873 | 0.839 | 0.763 |
| 0.9 | 0.895 | 0.874 | 0.842 | 0.771 |
| 1.0 | 0.895 | 0.875 | 0.844 | 0.778 |
| 1.2 | 0.895 | 0.876 | 0.847 | 0.786 |
| 1.4 | 0.895 | 0.877 | 0.850 | 0.794 |

The cross-section area of the canal is almost identical to the template used for the concreting of the canal i.e. Height (h)

66 cm

Width top ($w_t$) 130 cm

Width bottom ($w_b$) 65 cm

Yielding an area of flowing water = $A = 1/2\ h * (w_t + w_b)$

$= 1/2 * 0.66\ m * (1.3\ m + 0.65\ m)$ $= 0.64\ m^2$

To find the length of / the perimeter of the part of the wetted cross-section of the power canal, one must calculate the height of the sides of the canal ($s_1$).

$s_1^2 = h^2 + (1/2(w_t - w_b))^2 \Rightarrow s_1 = ((0.66\ m)^2 * (1/2\ (1.3\ m - 0.65\ m))^2)^{\square}$ $= 0.926\ m$

U = Width of, heights of, bottom of, and sides of power canal covered by water:

$U = w_t + w_b + 2\ s_1 = 1.30\ m + 0.65\ m + 2 \times 0.926\ m$ $= 3.80\ m$

The ratio of $A/U = 0.64\ m^2 / 3.80\ m$ $= 0.169\ m$

The above table (provided by Ossberger) states that, for a canal with brick-wall surface and an

1. A/U ratio of 0,1 the coefficient c is 0.860
2. A/U ratio of 0,2 the coefficient c is 0.865

By interpolation the coefficient c for A/U = 0.17 is found to 0.864

And the flow is calculated as:

$Q = v * A * C = 1.34\ m/s * 0.64\ m^2 * 0.864$ = 745 l/s $= 0.745\ m^3/s$

*Discharge Measured based on Kinetic to Potential Energy Conversion.*

At the same point in the power canal, the velocity of the water was measured by measuring the rise of the water flowing toward the end side of a stick perpendicular to the flow.

Water hitting the side of the stick rose $\Delta h =$ 0.12 m

The velocity v is then found by $v = (2 * G * \Delta h)^{\square} = (2 * 9.81\ m/s^2 * 0.12\ m)^{\square}$ $= 1.53\ m/s$

Using the same (Ossberger) coefficient C = 0.862 yields

$Q = 1.53\ m/s * 0.64\ m^2 * 0.862$ $= 851\ l/s = 0.851\ m^3/s$

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Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

## 2.2. The Discharge Measured at the 90-degree Corner, just Before the Forebay

The flow was also measured at the 90° corner (the corner with the concrete slab near to the road - just before the stop log into the spillway).

At a fairly straight part of the power canal just before the corner a length of 5 meters was measured and the start- and the end points of the stretch were marked. A twig was placed at the middle of the flow a few meters upstream of the first mark. The time for the twig to cover the distance (5 m) from the first mark to the second mark was measured as 3 s.

The velocity of the twig was calculated to be v = 5 m / 3 s
= 1.33 m/s.
The shape of the power canal at this site is identical to the template:

| | |
|---|---|
| Height (h) | 66 cm |
| Width top ($w_t$) | 130 cm |
| Width bottom ($w_b$) | 65 cm |

The depth ($h_2$) of the water (above the bottom of the power canal) was 0.50 m
A = Area of flowing water = ½ h * ($w_t$ + $w_b$) = ½ * 0.50 m * (1.3 m (0.50 m / 0.66 m) + 0.65 m) = 0.41 $m^2$

To find the circumference of the part of the cross-section of the canal, which is in touch with the water, the height of the sides of the power canal ($s_2$) are calculated to:
$s_2 = s_1 * h_2/h_1$ = 0.926 m * (0.50 m / 0.66 m) = 0.702 m

U = The width, the heights of bottom and sides of canal, which are covered by water =
U = $h_2/h_1$ ($w_t$ + 2 $s_1$) + $w_b$ = (0.50 m / 0.65 m) (1.30 + 2 x 0.926 m) + 0.65 m = 3.07 m
The ratio of A/U = 0.41 $m^2$ / 3.07 m = 0.133 m
Interpolating the table provided by Ossberger yields that for a canal with concrete surface the coefficient $C_2$ (for A/U = 0.133) is 0.862 m

From this, the flow is calculated as: Q = v *A * $C_2$; Q = 1.33 m/s * 0.409 $m^2$ * 0.862 = 0.469 $m^3$/s

At the same point in the power canal, the velocity of the water was measured by measuring the rise of the water flowing toward the end-side of a stick perpendicular to the flow. Water striking the side of the stick rose to a height of delta h = 8 cm.

The velocity v is found by v = $(2 * G * \Delta h)^{\square}$ = $(2 * 9.81\ m/s^2 * 0.08)^{\square}$ = 1.25 m/s
Using the same coefficient $C_2$ = 0.862 m yields: Q = 1.25 m/s * 0.409 $m^2$ * 0.865 = 0.441 $m^3$/s

## 2.3. The Discharge Found According to the Slope and the Cross Section of the Power Canal

In section “4.1. The Design of the Power Canal” the discharge is calculated based on the slope and the water level. The equivalent discharge Q is found to be 702 l/s, and the velocity v is found to be 1.8 m/s. The calculations are based on a water level in the power canal of 0.50 m.

The calculations in section 4.1 comprise two figures (with estimated tolerances):

1. The roughness coefficient of the cement plaster is here estimated to be 0.015 (+/- 0.005)
2. The slope of the power canal (as it was found by the surveying done by TD from REDP combined with Steen Carlsen's estimation). Here found to be 0.0047. (+/-70%)

This calculation (702 l/s) deviates significantly from the discharge (441 l/s - 469 l/s) found in section 2.2, and should, hence, be taken with reservation.

## 2.4. The Seepage from the Power Canal

The discharge measured just after the intake (0.745 $m^3$/s – 0.851 $m^3$/s) - and just before the forebay (0.469 $m^3$/s – 0.441 $m^3$/s) - shows that the difference turns is between 0.276 $m^3$/s and 0.410 $m^3$/s.

Even with the actual wide tolerances taken into account, the difference indicates a significant seepage along the power canal. A seepage of such magnitude can / will cause soil erosion, which over time can undermine the power canal and/or wash out / spoil the fields between the power canal and the river.

Also, significant power can be gained by reducing this seepage. This potential power has influenced the choice of the power rating of the turbine and the generator. Both turbine and generator will be able to utilize the potential discharge, which will be available if / when the power canal will be repaired. One can hope that the villagers will realize, how much power can be gained by repairing the power canal.

### 3. The State of the Civil Works, as Recorded in April/May 2002

#### 3.1. The Intake

The present intake from the Roshi River is in a very bad state, and renovation of the intake is severely
needed. Yet, the funding of this project is based on a very limited funding from Danida and Dhulikhel District Committee, combined with private funding from Hyonjan Electrical Engineering Fabricators, and Carlsen Power Electronics. Hence, the funding available does not allow for the needed rectification of the intake (and power canal).

The improvements required for the intake are:

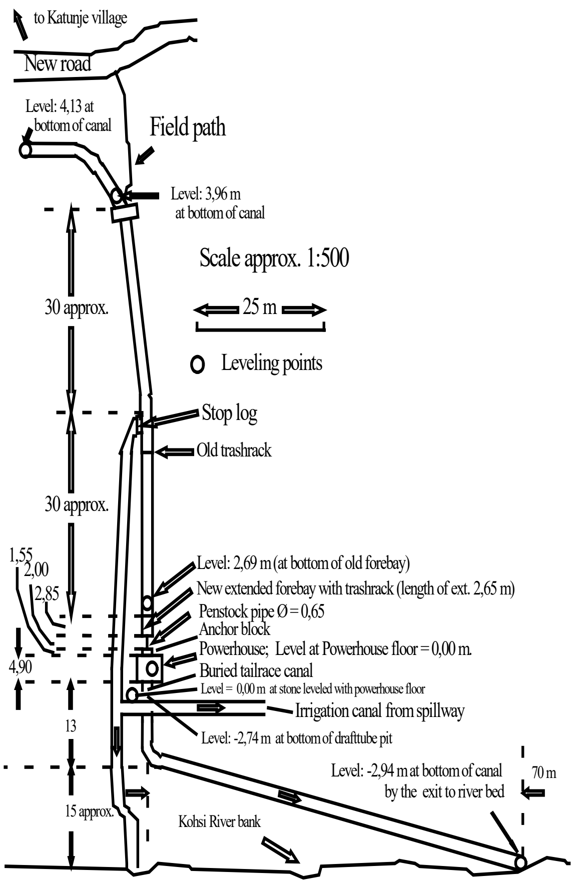
1. Structures - either in form of concrete or boulders - to divert (a part of) the flow towards the intake at the left (Northern) bank of the river.

2. An overhanging obstruction over the intake, to prevent floods (during the monsoon) to cause excessive flow into the power canal.

3. A coarse trash-rack, and a stop log.

4. Establishing of a settling basin just after the intake.

#### 3.2. The Power Canal and the Spillways

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Several parts of the power canal - and the spillways - are (as mentioned in point 2.4.) in a very poor condition. The funding of the present project does, however, not permit payment for repair of the power canal. It is our hope, however, that the villagers will (as a minimum) - patch the faulty parts of the power canal as soon as possible.

The idea of this project is to upgrade the installations in- and around- the powerhouse, such that it - after a repair and / or upgrading of the intake and the power canal - will be suitable for flows >600 l/s.

**3.3. The Conditions of the Old Forebay, Spillway, and the Associated Installations**
The concrete of the old forebay appears to be in a fairly good state. Yet, the construction seems to be unstable (as shown in 6.2.1.). When adding the vector of the hydrostatic forces to the vector presenting the weight of the sidewall (of the forebay), it appears that the resulting vector (acting on the sidewall(s)) points- / falls outside the base of the sidewalls.

This indicates that the sidewalls of the forebay may collapse, as they are only held together by the very limited and unpredictable tensile strength of the concrete (- and for the part of the forebay near to the penstock – by the end-wall). This is a dangerous situation; as concrete constructions must be designed such, that the tensile strength of concrete is always below zero – i.e. a positive compressive strength.

Power line and penstock

## 4. The New Hydrological Design

This chapter addresses data and hydrological limitations for the structures of the present site, which will not be changed in this project.

In the choice of the nominal discharge (= flow), for which the upgraded plant should be designed, subsequent aspects have been taken into consideration:

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1. The difference in the discharge measurements made just after the intake (745 l/s - 851 l/s) and just before the forebay (469 l/s - 441 l/s) - indicates a potential significant increase in the discharge, if / when the power canal is repaired. The repair of the power canal can be done along the way by the villagers themselves. A discharge of 702 l/s - as estimated in section 4.1 – must be regarded as unrealistic.

2. In "Information on Roshi Khola Micro Hydro Scheme at Katunjebesi, Katunjebesi VDC, Kavbre Palanchok District" the discharge is given as 400 l/s.

3. The efficiency of most crossflow turbines attains their maximum at approx. 70-80% of rated flow and remain fairly constant down to 50% of rated flow. A comparable similar efficiency curve applies to asynchronous generators.

Point 1 reflects the uncertainty of the measurements of the discharge. In this situation, one must face, that the choice of nominal flow will be subjective. Here, the choice of nominal flow has been chosen to be 600 l/s. As shown in section 5, this (with a net head 6,16 m) yields just below 22 kW output power (with the assumed efficiencies).

If the actual flow turns out to be as low as 400 l/s, the efficiency of both turbine and generator will still remain very near to the same as that for 600 l/s (but both components will operate under de-rated conditions), and the output power will be approx. 14 kW.

**4.1. The Design of the Power Canal**

The size of the template (found in the powerhouse and used for the construction) of the power canal is:

| | |
|---|---|
| Height | 0.66 m |
| Width top | 1.30 m |
| Width bottom | 0.65 m |

The angle of the slope of sides θ = arc tan (2 * 66 / (130-65)) = 63.8°

The discharge, which can be obtained by the present canal, is calculated, using subsequent parameters:

v The design velocity of the power canal
S The slope of the power canal (here 0.0047; described in section 4.2)
r The hydraulic radius of the water in the power canal
n The roughness coefficient of the power canal (for plastered surfaces as here 0.01)
A The cross-section area of the power canal covered with water
Q The discharge.

$$S = \left(\frac{nV}{r^{2/3}}\right)^2 \quad => \quad V = \frac{\sqrt{S}}{n} r^{\frac{2}{3}}$$

The discharge Q = v * A
The hydraulic radius r is found as function of:
1. The (wetted) area A of the power canal
2. The angle between the sides of the power canal and horizontal.

The hydraulic radius r is given by:

$$r = \frac{1}{2}\sqrt{\frac{\sin\theta}{2 - \cos\theta}}\ \sqrt{A}$$

Using these equations and the estimated slope of the power canal subsequent discharges have been calculated as a function of the height h of the water in the power canal:

| h [m] | 0.40 | 0.45 | 0.50 | 0.55 | 0.60 | 0.65 |
|---|---|---|---|---|---|---|
| A [$m^2$] | 0.316 | 0.373 | 0.409 | 0.477 | 0.550 | 0.627 |
| r [m] | 0.202 | 0.232 | 0.230 | 0.249 | 0.267 | 0.285 |
| V [m/s] | 1.575 | 1.622 | 1.715 | 1.809 | 1.894 | 1.980 |
| Q [l/s] | 498 | 605 | 702 | 863 | 1042 | 1241 |

The constants used for the calculations are trapezoidal shaped power canal with:

| | |
|---|---|
| Top width | 1.30 m |
| Bottom width | 0.65 m |
| Height from bottom to top of canal sides | 0.66 m |
| Slope of sides of power canal - | 63.9$^o$ |
| Roughness coefficient of power canal (plastered cement) | 0.015 |
| Length of power canal (estimated) | 60 m |
| Slope s (of actual section of power canal 0.28 m / 60 m) | 0.0047 |
| Required velocity of water to avoid silt deposit is | 0.3 m/s |

### 4.2 The Forebay and the Settling Basin

During the surveying of the site, the level of the subsequent points was measured (with reference level 0 equal to the floor in the powerhouse):

1. The bottom of the power canal approx. 20 m before the sharp corner with the concrete slab was measured to be 4.13 m (level of top of the sides: 4.13 + 0.66 m = 4.79 m).

2. Just before the concrete slab, the level at the bottom of the power canal was 3.96 m

   (level at top of the sides = 3.96 m + 0.66 m = 4.62 m).

3. The level at the bottom of the forebay was measured to be 2.69 m (relative to the floor in the powerhouse). The exact height of the forebay was (by a mistake) not recorded. Judged from a photo of Steen Carlsen in the forebay and knowing his height (1.80 m) the depth of the forebay is 1.65 m and the estimated level of the top of sides of the old forebay walls is approx. 4.34 m.

4. The length of the old forebay was also not measured, but is assumed to be approx. 6 m.

Assuming same free board in the power canal and in the forebay yields a drop in head of 4.62 m – 4.34 m = 0.28 m over a distance of approx. 60 m (from the sharp corner to the forebay).

The slope S = 0.28 m / 60 m = $4.7\ 10^{-3}$.

The tolerance of the slope is estimated to +/- 70%

**The Settling Basin**
The forebay also serves as a settling basin. Any particle heavier than water will move towards the bottom (i.e. settle) with a certain velocity $v_o$. According to the Micro Hydropower Sourcebook page 166
$v_o = 60\ Q/A_s$ - where Q is the discharge and $A_s$ is the surface area of the settling basin. Settling velocities for suspended particles are:

1. For 0.1 mm particles 1 m/min
2. For 0.8 mm particles 5 m/min

The estimated cross-sectional area of the forebay is 1.3 m * 1.65 m = 2.15 m$^2$
The length of the forebay is set to $L_{fore}$ 6.0 m

The top of the baffle wall supporting the trash rack at the end of the forebay is estimated to be 4.00 m, i.e. submerged 0.63 m. The minimum settling velocity ($v_O$) of a particle, which should sink from the surface $h_1$ (at the entrance to the forebay) - down to the top of the baffle wall $h_2$ (supporting the trash rack [at the end of the forebay]) must be:

$$v_o = v(Q) * (h_1 - h_2) / L_{fore} = v(Q)\ (4.63 - 4.00) / 6.0 = v(Q) * 0.1$$

where $v_o$ is the velocity of the particle
v(Q) is the velocity of the water as a function of the discharge.
$L_{fore}$ is the length of the forebay.

| Q [l/s] | 400 | 500 | 600 | 700 |
|---|---|---|---|---|
| V [m/s] | 0.19 | 0.23 | 0.28 | 0.33 |
| $v_o$ m/s | 0.019 | 0.023 | 0.028 | 0.033 |
| $v_o$ [m/min] | 1.14 | 1.38 | 1.68 | 1.98 |

The figures indicate that particles with diameters of approx. > 0.2 mm will settle in the forebay.

### 4.3 The Trash Rack
The trash rack is chosen to have a clearance of 10 mm between bars (equal to Danish standards). The thickness of the bars should be approx. 2,5 mm. To minimize head loss in the trash rack, the bars shall be sharpened in one end (downstream) and rounded in the other end (upstream), as shown for the second bar from the right on fig. 5.193 on page 164 in the Micro Hydropower Sourcebook.

#### 4.3.1. The Head Loss of the Trash Rack

$$h = K_t \left(\frac{t}{b}\right)^{4/3} \frac{v^2}{2g} \sin\phi$$

where:
- h = The head loss across trash rack [m].
- $K_t$ = Trash rack loss coefficient (1.0 for bars pointed in one end and rounded in the other).
- t/b = Ratio of max. bar thickness to space between bars.
- v = Approach velocity (i.e. design flow [Q]) / (cross section area of water upstream of rack)
- g = Gravity constant (9.8 m/s$^2$)
- $\phi$ = Angle of bars with the horizontal

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

Subsequent values are used for calculating the head loss in the trash rack:

| | |
|---|---|
| Width of the trash rack W = | 1.0 m |
| Length of the part of the trash rack passed by water h = | 1.25 m |
| Area A = 1.0 m * 1,25 m = | 1.25 $m^2$ |
| Coefficient of resistance (for the specified shape of bars) $K_t$ = | 1.0 |
| Thickness of bars t = | 2.5 mm |
| Space between bars b = | 10.0 mm |
| Velocity of approaching water v = 0.6 $m^3/s$ / 1.25 $m^2$ = | 0.48 m/s |
| Angle of approaching water phi = | $30^0$ |
| => Head loss in trash rack | = 1.9 mm |

### 4.4 The Intake Stop Valve

The intake stop valve will be in the form of a gate (here an iron sheet) hinged to the upper end of the trash rack (shown at the figure next page). The intake stop valve will - when released - drop down and cover the trash rack. The gate is - during normal operation - held in the upper position by an electromagnet supplied from the grid power. In case the voltage supplying the electromagnet is interrupted, this will shut off the water supply to the turbine

The gate will be suspended by a steel wire extending from a tackle over the forebay down to the powerhouse, so that the gate can be raised / controlled from within the powerhouse. A rule of thumb states, that for the steel wire to last/endure, the pulley diameter should be more than 28 times the diameter of the steel wire.

As shown in the subsequent section, the force required to raise the gate from submerged position is approx. 275 kg. Hence a 5-way tackle (giving a reduction of 10 times) is used. The jet-guide vane at the turbine provides another means to shut of the supply of water.

#### 4.4.1. Deducing the Torque Required to Lift the Closing Gate over the Trash Rack

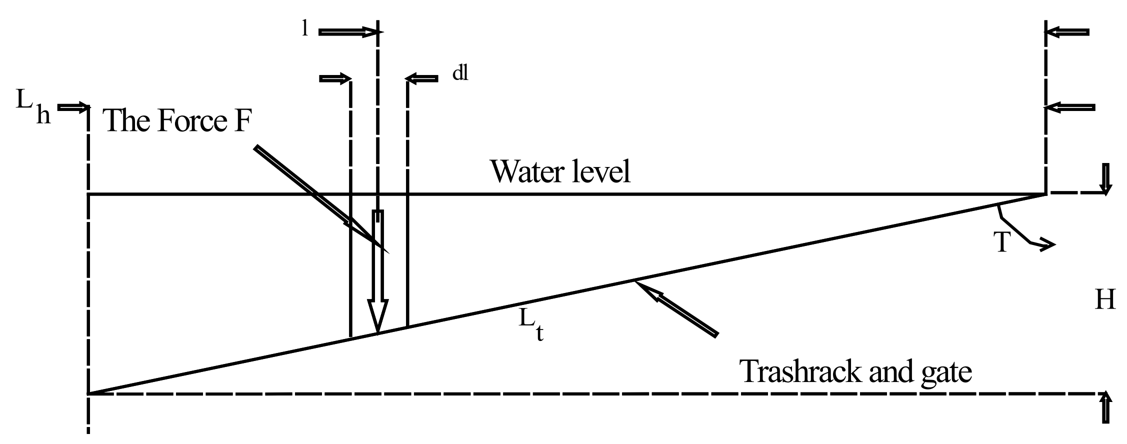



**Figure 1. Forces acting on Stop Log and Trash Rack**

The torque required to lift the 1,9 X 1,0-meter iron sheet covering the trash rack is calculated as follows: At the upstream end of the trash rack, the sheet/gate will be submerged H = 0,5 m below the surface of the water. At the downstream end it will be pivoted with a hinge just above nominal water level.

| | |
|---|---|
| Length of (sloping) trash rack $L_t$ | 1.90 m |
| Width W (of forebay from inner side to inner side) | 1.00 m |
| Depth at extreme end of sheet (H) | 0.50 m |
| Length of "arm" = horizontal projection of trash rack $L_h$ | 1.65 m |

The torque T is:

$$T = D W G \int_0^{L_h} l \frac{H}{L} l \, dl = \frac{1}{3} D W G H L_h^2$$

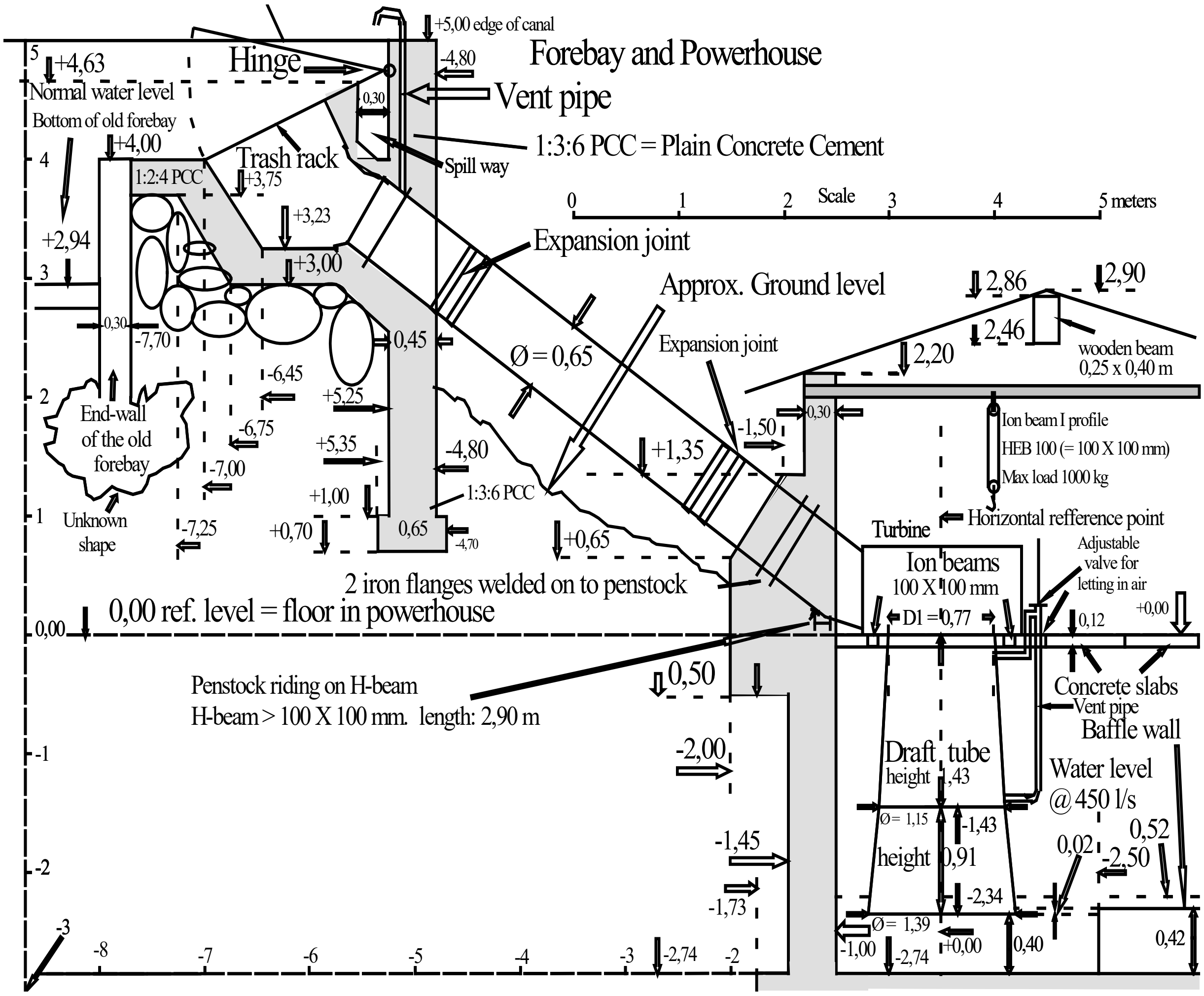
Where: D is the density of water (1000 kg/m$^3$)
G is the constant of gravity (9.81 m/s$^2$)

T = 1/3 * 1000 kg/m$^3$ * 1.5 m * 9.81 m/s$^2$ * 0.5 m * (2.2 m)$^2$ = 4441 Nm

Thus, the required lifting force $F_{lift}$ (in kg) at the upstream end of the gate is:
$F_{lift}$ = T / (G $L_h$) = 4441 Nm / (9.81 m/s$^2$ * 1.65 m) = 275 kg.

In this calculation, only the weight of the water is included.
The weight of the steel sheet has been neglected.

## 4.5. The Penstock




The head loss in penstocks can be calculated from the equation given in the Micro Hydropower Sourcebook page 126 and page 129:

Head loss in penstock: $h_f = 10\ L\ n^2\ Q^2 / D^{5,3}$ - and

Head loss at entrance of penstock: $h_t = K_t\ v^2 / (2\ G)$

where:

- L is the length of the penstock.
- n is the roughness coefficient between water and inside of penstock.
  (n depends on the inner surface of the penstock.)
  - n for welded steel is 0.012
  - n for corrugated metal 0.022-0.030
- Q is the discharge [$m^3/s$]
- D is the internal diameter of penstock.
- $K_t$ is the coefficient of the turbulence at the intake of the penstock.

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

### 4.5.1. The Losses in Old Penstock

The specifications for the old penstock are:

| | |
|---|---|
| Penstock material | mild steel |
| Penstock outer diameter $Ø_{outer}$ | 0.38 m |
| Thickness | 4.5 mm |
| Penstock inner diameter $Ø_{inner}$ | 0.37 m |
| Penstock roughness n (estimated) | 0.012 |
| Penstock length is L = | 13.24 m |
| Penstock entrance turbulence factor $K_{h-t}$ = | 0.2 |
| | |
| The head loss in old penstock @ 400 l/s = 3,72 m/s | 0.59 m |
| The head loss in old penstock @ 600 l/s = 5,58 m/s | 1.33 m |
| | |
| The turbulence loss at entrance @ 400 l/s = 3,72 m/s | 0.15 m |
| The turbulence loss at entrance @ 600 l/s = 5,58 m/s | 0.31 m |
| | |
| Total head loss in old penstock | |
| @ 400 l/s: | 0.74 m |
| @ 600 l/s: | 1.64 m |

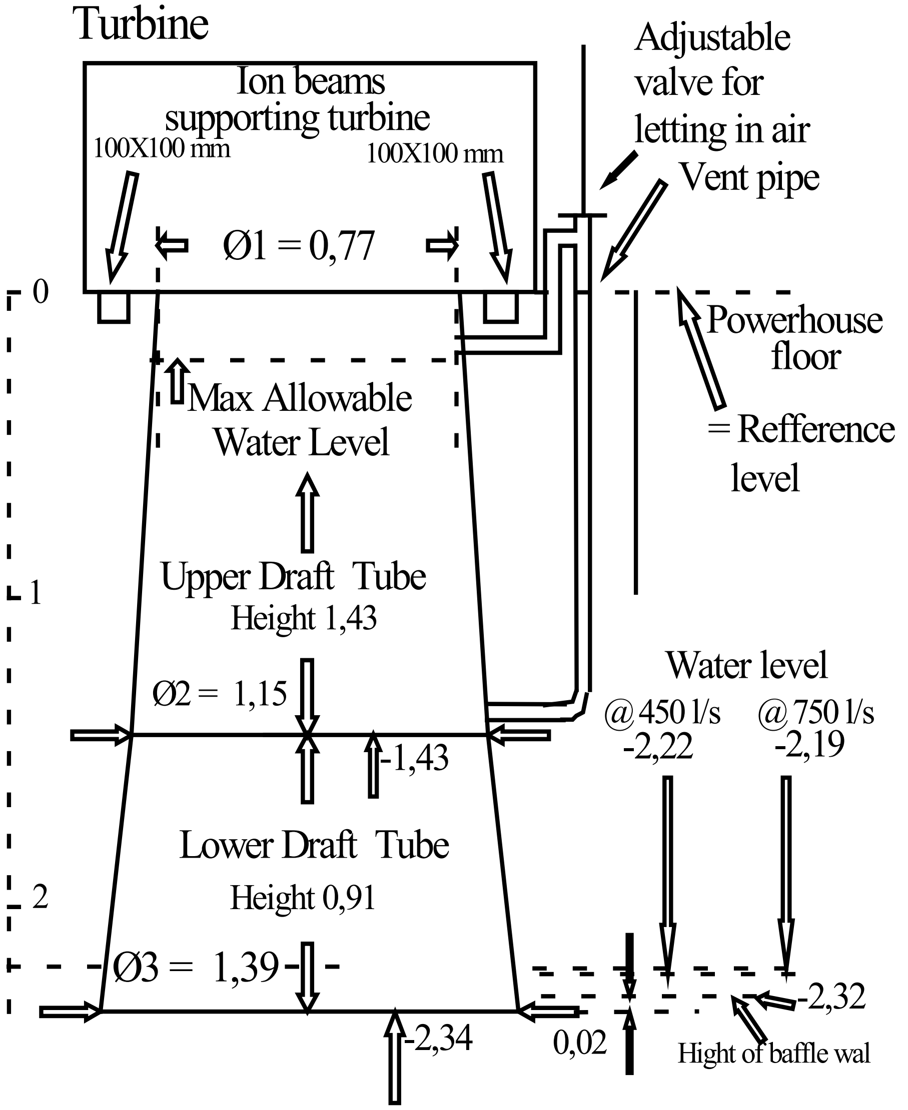
This indicates that the penstock is a bottleneck for the old plant, and that an increased diameter of the penstock is required if a discharge of 600 l/s shall be achieved.

### 4.5.2. The Losses in New Penstock

| | |
|---|---|
| The outer diameter of the new penstock has been chosen to be | 0.65 m |
| The thickness of the steel plate of the new penstock has been chosen to be | 4.0 mm |
| The inner diameter of the new penstock is | 0.64 m |
| The length of the new penstock will be | 6.0 m |
| | |
| n The roughness coefficient for welded steel is | 0.012 |
| n The roughness coefficient for corrugated metal | 0.022 - 0.030 |
| Safeguarding by assuming corrugated metal n is set to | 0.03 |
| | |
| The new penstock: | |
| The head loss in new penstock @ 400 l/s = 0.24 m/s | 0.09 m |
| The head loss in new penstock @ 600 l/s = 2.80 m/s | 0.21 m |
| | |
| Turbulence loss at entrance @ 400 l/s = 1.24 m/s | 0.016 m |
| Turbulence loss at entrance @ 600 l/s = 2.80 m/s | 0.035 m |
| | |
| Total head loss in new penstock | |
| @ 400 l/s: | 0.11 m |
| @ 600 l/s: | 0.25 m |

(For head loss as function of discharge please refer to section 5. New Design and Potential Power.)

### 4.6 The Turbine




The type of turbine chosen for the site is a crossflow turbine. The reasons for the choice are:

1. It is suitable for the head and discharge at the site.
2. It is manufactured locally.
3. It is self-cleaning.

Cross flow turbines are impulse turbines, and the runner must operate in air. When draft tubes are used (as in this project) the pressure below the runner (inside the turbine) will be lower than the ambient pressure. Hence, the turbine bearings must airtight. By impulse turbines the pressure difference H across the nozzle is converted into kinetic energy according to the equation:

$$\frac{1}{2} m v^2 = m * H * G \quad \Rightarrow v = (2 * H * G)^{\frac{1}{2}}$$

where:

m is the mass of the water
v is the velocity of the water, as it leaves the nozzle
H is the pressure difference (in meter water column = net head)
G is the constant of gravity (9.81 $m/s^2$)

With a net head of 6.1 m the speed of the water jet will be:
$V = (2 * H * G)^{½} = (2 * 6.1 \text{ m} * 9.81 \text{ m/s}^2)^{½}$ 10.9 m/s

At the optimum point of operation, the peripheral speed of the runner should be near to 0.50 – 0.55 times the speed of the water jet.

The diameter of the runner is chosen as a trade-off between:
1. Cost. - The cost of the turbine increases with the diameter of the runner.
2. The specific speed of the turbine. - The specific speed decreases by increasing runner diameter.
3. Gear ratio. - The cost of the gearing often increases with increasing gear ratio.

A tentative choice of runner diameter $Ø_{run}$ is chosen to be 400 mm
Yielding a nominal RPM of N. $N * Ø_{run} * \pi = 0.53 * v \Rightarrow N = v * 0.53 / (Ø_{run} * \pi)$
$= 10.9 \text{ m/s} * 0.53 / (0.4 \text{ m} * \pi) =$ 4.61 RPS
N in RPM (rotations per minute) = RPS (rotations per second) * 60 s = 4.61 RPS * 60 s= 276 RPM

An estimate of the cross-sectional area A of the jet is:
$A = 0.23 Q / (H^{½}) = (0.23 * 0.6 \text{ m}^3/\text{s}) / ((6.2 \text{ m})^{½})$ 0.055 $m^2$
With a jet thickness of 0.05 m the length of the turbine comes to approx. 1.1 m
Horizontal area below runner (0.4 m+ 0.1 m) * 1.1 m 0.55 $m^2$
Mounting flange below turbine for interface to draft tube $(0.55 \text{ m}^2)^{½} =$ 0.77 m X 0.77 m

## 4.7 The Gearing between the Turbine and the Generator

High gear ratios between turbine and the generator will usually cause poor efficiency.
The speeds of Francis- and crossflow turbines will often be in the range of 100 - 300 R.P.M.
Thus, to limit gear ratio, high-pole (6 - 10 poles) motors with nominal speeds in the range of 550 - 950 R.P.M. are usually to be preferred. The price of standard asynchronous motors (/generators) rise with the no of poles.

A standard 6 pole 22 kW asynchronous generator has a slip of approx. 2.5%
This yields a nominal RPM of a 6-pole motor used as generator of 1025 RPM
With a turbine speed of 276 RPM, this gives a gear ratio of 0.270 (= 1 / 3.70).

When using V-belts (or flat belts) the contact surface between belt and pulley must be sufficient to ensure that the belt can transfer the required torque. Small pulley diameters will require higher belt tensions to transfer a given torque leading to reduced efficiency and early wear out of belts and bearings.

On the other side, too low belt tensions can cause slip. Also, low belt tensions can in certain cases cause an insufficient radial force on the turbines main bearing (particularly if this is spherical). This in turn can cause increased bearing temperature and reduced lifetime for the bearing.

A rough rule of thumb states, that max. power transmission per belt should be in the order of 5 kW for V-belts of C-type. Use of multiple belts increases reliability and reduces the required axial force. The disadvantage of multiple belts is that multiple belts should be of same brand, lot and age to secure the same tension for all belts.

For the crossflow turbine in Katunje, the turbine shaft will be horizontal. It will be most practical, for the turbine pulley not to extend below floor level. Hence, the diameter of the bigger pulley is chosen equal to the diameter of the runner Ø = 400 mm
The diameter of the generator pulley is 400 mm / 3.7 108 mm
Angular speed $\omega_{tur}$ of turbine (@ 227 RPM) 28.9 rad/s
Torque of turbine (@ 276 RPM) = P / ω = 22 kW / 28.9 rad/s 761 Nm
Angular speed of generator $\omega_{gen}$ @ 227 RPM 106.9 rad/s
Torque of generator (@ 276 RPM) = P / $\omega_{gen}$ = 22 kW / 106.9 rad/s 206 Nm
Peripheral force of V-belt = T / R = 761 Nm / 0.20 m 3805 N
Radial force on turbine & generator bearings $F_{rad}$ = approx. 4 X 3805 N 15220 N

Thus, it would be desirable to increase the diameters of both pulleys.

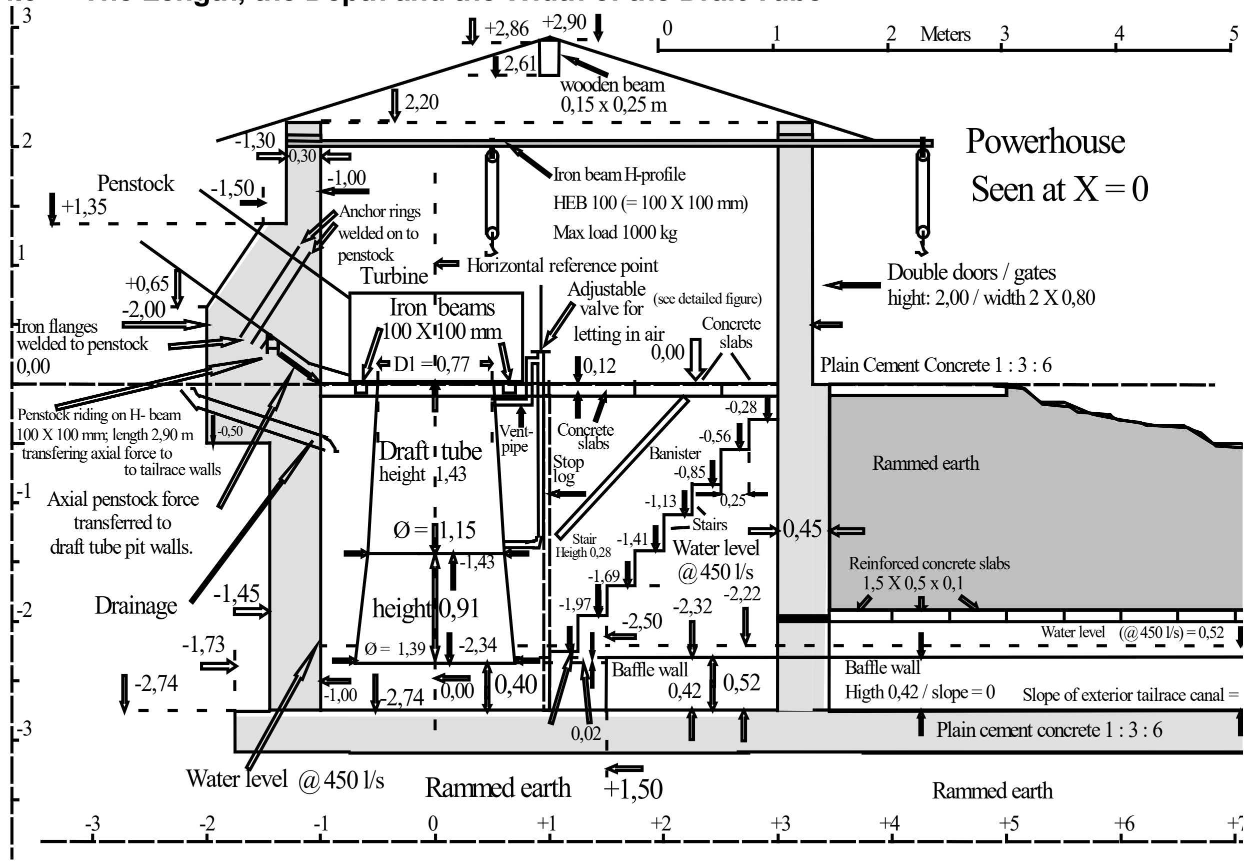
### 4.8 The Generator and the Controller

The power rating for a standard motor is defined as the output power at the shaft. Any motor will have an efficiency of less than 100%; hence the power delivered to the motor will always be higher than the output. The bottleneck in any motor design is the temperature of the windings, and these will be designed to handle a current corresponding to the input power, which is higher than the nominal (shaft) power of the motor. This implies that any asynchronous motor used as generator will automatically be de-rated by $1/n^2$ (where n is the efficiency of the motor) when used as generator. (Magnetic saturation must however be taken into account / i.e. be avoided.)

For a standard 22 kW 6 pole WEG motor
Full load efficiency is 91.5%
At 0.75 load efficiency is 90.9%
At 0.50 load efficiency is 89.9%

To generate a 3-phase system with the capability to withstand asymmetric loads, the generator windings must be rated for approx. 230 V and connected in star.
A standard 22 kW 6 pole WEG motor has a rated windings current of 42.8 A

## 4.9 The Length, the Depth and the Width of the Draft Tube




To maximize output power, the draft tube should be as long as possible. Yet, water must be able to flow freely out of the draft tube. That means that water - after leaving the draft tube - should flow through a cross section, with an area no less than the area of the cross section at the bottom of the draft tube.

Looking at: 1. An imaginary vertical cylinder below the bottom end of the draft tube - and
2. The bottom of the tailrace.

the area of:

1. $A_{side}$ = the side of the imaginary vertical cylinder - should be at least as big as the area of
2. $A_{bottom}$ = the top of the cylinder - which is the same as the bottom of the draft tube.

$$A_{side} = l_{clearance} * \pi D_2; \; \& \; A_{bottom} = * \pi (D_2/2)^2;$$

$$A_{side} > A_{bottom} \Rightarrow l_{clearance} > D_2/4$$

| | |
|---|---|
| By iterating the length and the associated clearance a suitable $l_{clearance}$ (i.e. distance between tailrace floor and lower rim of draft tube) was found to be | 0.40 m |
| The drop from the powerhouse floor to the riverbed is | 2.94 m |
| - The drop from the tailrace floor (below the draft tube) to the riverbed is | 0.20 m |
| - The clearance $l_{clearance}$ is chosen to be | 0.40 m |
| Yielding a length of the draft tube $L_{d.}$ = (2.94 – 0.20 – 0.40) m | 2.34 m |

| | |
|---|---|
| Diameter of draft tube orifice $D_2 = D_1 + 2 * L_{d.} \sin \alpha = 0.77$ m. + 2 * 2.34 m sin 7,5° | 1.38 m |
| Velocity at entrance to draft tube v = Q / A = 600 l/s / 0.55 m² = | 1.09 m/s |
| Velocity at bottom of draft tube v = Q / A | |
| = 600 l/s / ($\pi$ (1.38 m / 2)² = 600 l/s / 1.496 m² = | 0.40 m/s |
| Area of imaginary cylinder below draft tube $A_{imag} = l_{clear.}\ \pi\ D_2$ = 0.4 m * $\pi$ 1.38 m = | 1.73 m² |

| | |
|---|---|
| Head loss in terms of velocity of water leaving the draft tube = $v^2 / (2 G)$ | |
| = (0.40 m/s)² / (2 * 9.81 m/s²) | 0.008 m |
| Power lost at bottom of draft tube P = ½ * Q * v² | |
| = ½ * 600 kg/s * (0.40 m/s)² = | 48 W. |

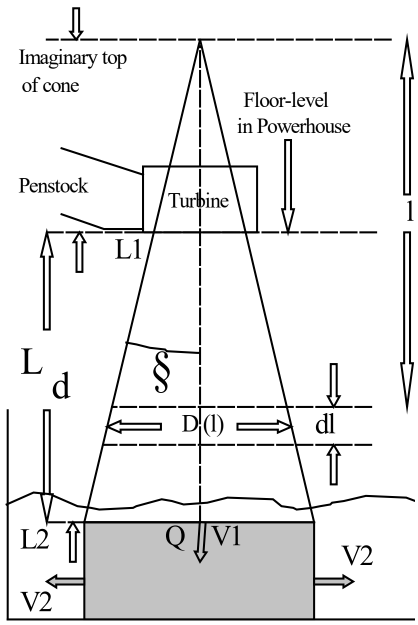
### 4.10 The Deduction of the Head losses in a Conical Draft Tube

A general equation for head loss in conical draft tubes has not been found in the literature. Hence, it has been deduced as follows:

The head loss for a cylindrical penstock pipe is given as:

$$h_f = 10\ L\ n^2\ Q^2 / D^{5,3}.$$

According to the Micro Hydropower Sourcebook the max. angle α of the sides of the draft tube is 8°. For α > 8° the water will detach from the inner side of the draft tube, and the draft will be lost.




**Draft Tube Losses**

The conical tube can be regarded as a number of short lengths (dl) of tubes with a diameter D, where D is a function the distance l from the top of the cone.

Letting the number of tubes go toward infinity, causes dl to go towards zero; and the calculation turns into subsequent integral:

$$h_f = \int_{L1}^{L2} \frac{10\, n^2\, Q^2}{D(l)^{5.3}}\, dl$$

$$= 10\, n^2\, Q^2 \int_{L1}^{L2} \frac{1}{(l\ 2 \sin \alpha)^{5.3}}\, dl \ \wedge\ D(l) = l\ 2 \sin \alpha$$

$$= \frac{10\, n^2\, Q^2}{(2 \sin \alpha)^{5.3}} \int_{L1}^{L2} l^{-5.3}\, dl$$

$$= \frac{10\, n^2\, Q^2}{(2 \sin \alpha)^{5.3}} \left[ \frac{1}{-4.3\ l^{-4.3}} \right]_{L1}^{L2}$$

Any velocity of the water leaving the draft tube represents lost energy. To minimize loss, the diameter at the lower end of the draft tube must be maximized, without exceeding the maximum angle of 8°. Thus, to allow for a suitable tolerance, the draft tube is designed for α = 7,5°. (I.e. the slope of each side is 3.75 °.)

The smallest size of the aperture / opening at the bottom of the turbine is estimated as $D_1$ = 0.77 m.
$D_1 = L_1\ 2 \sin \alpha$. Hence, $L_1$ = 2.96 m.

The length of the draft tube is determined by:

1. The level difference between the powerhouse floor and the riverbed (2.74 m)
2. The clearance below the draft tube (0.40 m).

Length / depth of draft tube $L_2$ = 2.34 m

The parameters used to calculate the head $h_f$ loss in a draft tube as a function of discharge are as follows:

Roughness coefficient n inside draft tube (for corrugated iron) n = 0.03
Cone angle α 7.5°

$$= \frac{10\, n^2 Q^2}{(2 \sin \alpha)^{5.3} \bullet -4.3} = -2.58\, Q^2$$

$$[l^{-4.3}]_{2.96\, m}^{5.31\, m} = (5.31\, m)^{-4.3} - (2.96\, m)^{-4.3}$$

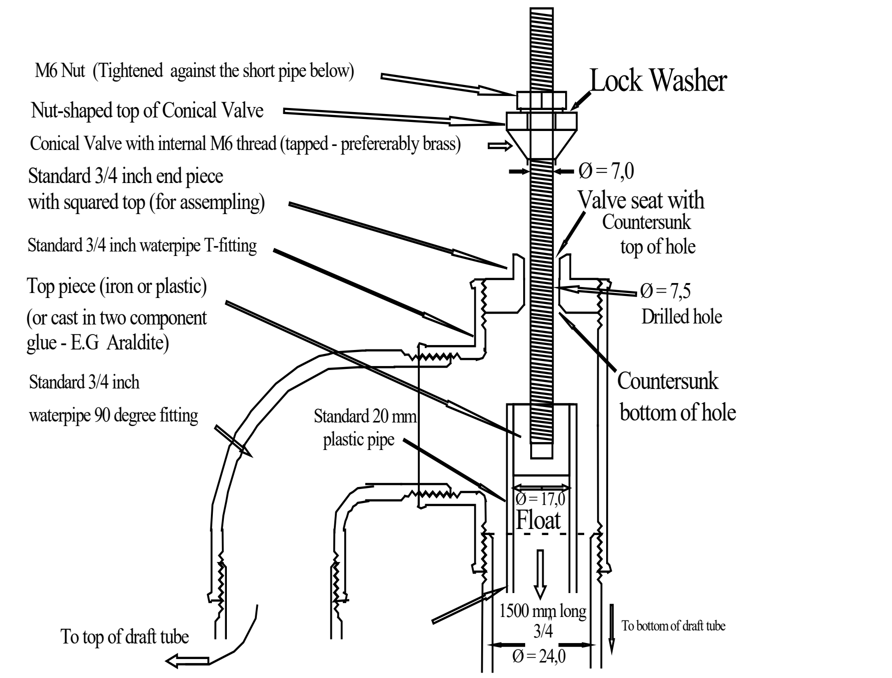
$$= 7.62\ 10^{-6} - 9.407\ 10^{-3})\, m^{-4.3} = -9.40\ 10^{-3}\ m^{-4.3}$$

$$=> h_f = -9.40\ 10^{-3} \bullet -2.58\, Q^2 = 0.024\, Q^2$$

| Q [l/s] | 400 | 500 | 600 | 700 |
|---|---|---|---|---|
| $h_f$ [mm] | 4 | 6 | 9 | 12 |

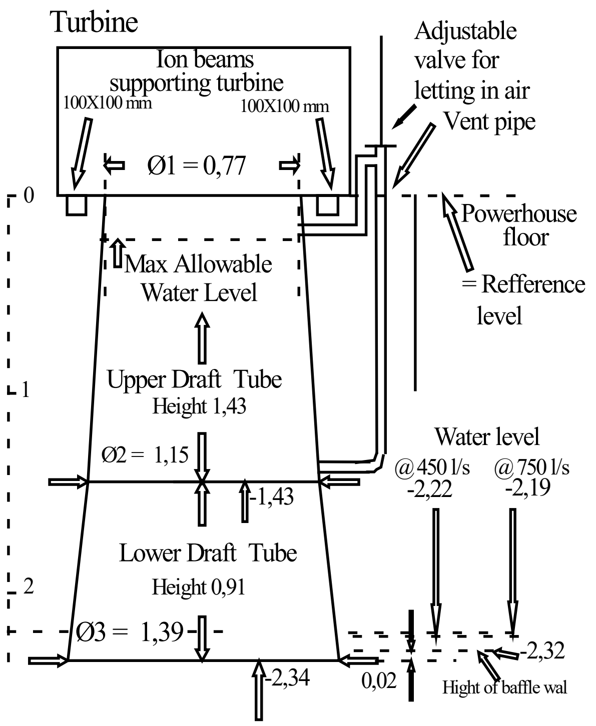
### 4.11 Adjustable Air Valve for Draft Tube

If the water level in the draft tube rises to a level, where the water falling from the runner will splash back onto the runner, the efficiency of the turbine will drop significantly. On the other hand, the water level in the draft tube should be as high as possible in order to maximize the head.




**Air Valve for Maintaining Water Level in Draft Tube**

Hence, In order to obtain maximum efficiency, it is desirable to be able to adjust the water level in the draft tube. Ossberger turbine factory in Germany controls the level in their draft tube by a valve with a pre-stressed spring. This concept, however, does not allow for taking up variations due to varying tail-water levels e.g. due to varying flows.




Thus, a different (and to my knowledge - new) concept is chosen, where a float detects the water level and lets in air to the draft tube if / when the water level raises above a certain set point. Such a control mechanism can be made cheap and simple.

Two pipe stubs - one near to the bottom and one near to the top and placed above each other - are welded on to the upper part of the draft- tube (see the figure above).

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

A tube system as shown on the figure above is made out of standard steel 3/4" water pipes and associated fittings. Inside the longest of the vertical pipes is inserted a float - made of a standard 20 mm Ø plastic pipe with airtight terminations in both ends. A standard M6 steel rod (0.4 m long) is fastened to the top end of the plastic pipe / float. On the M6 rod is mounted a piece of conical brass with inside thread, so that its position on the rod can be adjusted. The brass piece rests on the 7.0 mm hole through the top fitting and forms the air valve.

The air pressure within the turbine is reduced due to the water column in the draft tube,
i.e. is (1.0 – 2.34 m / 10) Bar = (1 – 0.23) Bar

With a 7 mm hole in the top 3/4" fitting the downward force from the ambient air is
$0.23 \text{ Bar} * 2.3 \text{ m} * (7.0 \text{ mm} / 2)^2 * \pi \;\; 1000 \text{ kg/m}^3 * 9.81 \text{ m/s}^2 =$ 0.868 N

The weight of the 0,4 m M6 rod is $L_{rod} * (Ø/2)^2 * \pi * G$
$0.4 \text{ m} * (5.8 \text{ mm} / 2)^2 * \pi * 7860 \text{ kg/m}^3 * 9.81 \text{ m/s}^2 =$ 0.814 N

The weight of the 0.8 m plastic pipe is $L_{pipe\ +} * (Ø_{pipe} / 2)^2 * \pi * D$
$0.8 \text{ m} * (20.00\text{-}1.37) \text{ mm} * 1.37 \text{ mm} * \pi * 9.81 \text{ m/s}^2 * 1490 \text{ kg/m}^3$
= 0.298 N

The required length of the 20 mm plastic pipe to give a buoyancy (= up-lift) of

$F = 0.868 \text{ N} + 0.814 \text{ N} + 0.298 \text{ N} =$ 1.977 N

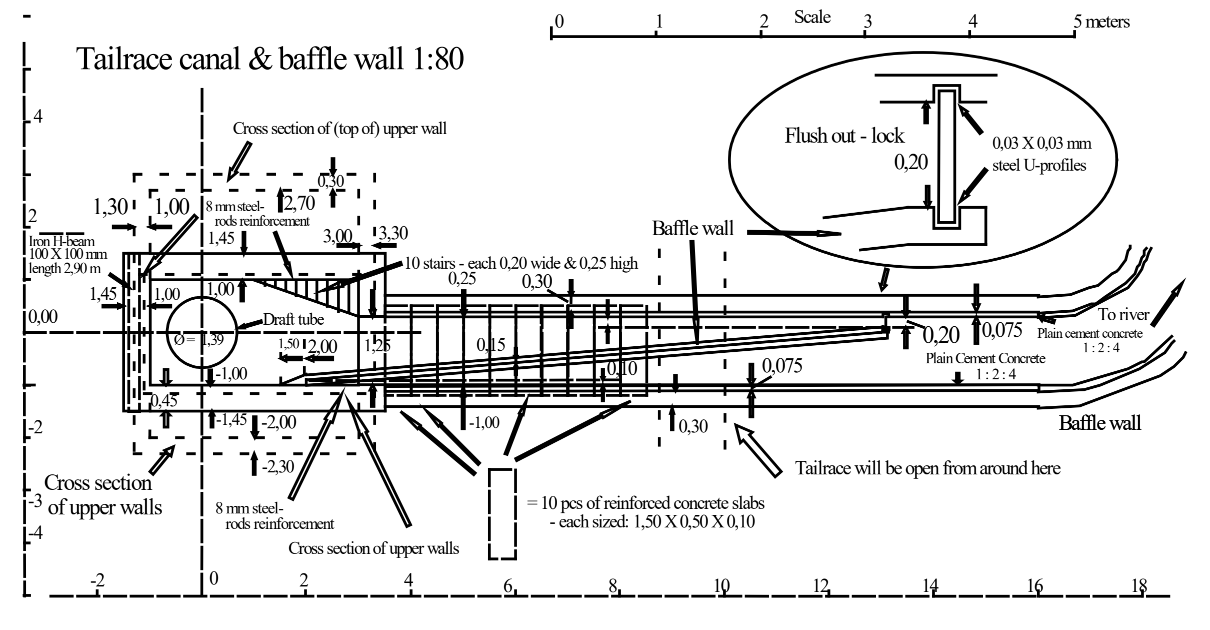
is: $= L_{pipe} * (Ø_{pipe}/2)^2 * \pi * G * D$

$= L_{pipe} * (0.01/2)^2 * \pi * 9.81 \text{ m/s}^2 * 1000 \text{ kg/m}^3$

$\Rightarrow L_{pipe} = 1.679 \text{ N} / ((0.01 / 2)^2 * \pi * 9.81 \text{ m/s}^2 * 1000 \text{ kg/m}^3)$ = 0.641 m

To ensure the top of the float is well above the surface, the length of the plastic pipe is chosen to 0.8 m.

## 4.12. The Tailrace Canal




**The tailrace with the baffle wall**

The level difference from the top of the baffle wall to the tailrace floor below the draft-tube is 0.40 m.

The level difference from the tailrace floor (below the draft-tube) to the riverbed is 0,20 m.

The length of tailrace canal $l_{trc}$ is approx. $l_{trc} = (70^2 + 15^2)^{1/2} + 13 + 0.45 + 4$ m = 89 m.

Corresponding to a slope in tailrace canal $s_{trc}$ 0,0022.

This slope is assumed sufficient to drain the tailrace.

To facilitate maintenance of the tailrace, two vertical grooves must be made in the sides of the tailrace (at the downstream end), so that a stop log can be established.

## 5. The Design and the Potential Power Calculations for the New Plant

Calculations of the potential power for the site are presented below.
The power is calculated as function of head and flow.
(Other parameters can be set / varied within the spread sheet.
The Parameter (Par.) defines the coefficient / parameter for the actual row.)
Spreadsheets showing worst and best cases are presented on following pages:

### The Power and Efficiency as Function of the Discharge; With Draft Tube - Worst Case

| Water level in forebay [m] | 1 | Par. | 4.30 | | | | 4.63 | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Flow [l/s] | 2 | | 400 | 500 | 600 | 700 | 400 | 500 | 600 | 700 |
| Power - theoretical [kW] | 3 | | 25.68 | 32.05 | 38.40 | 44.73 | 26.81 | 33.66 | 40.34 | 46.99 |
| Turbine head loss [m] = Par | 4 | 1.0 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Penstock head loss per m [m] for Ø = 0,65 m | 5 | 1.0 | 0.014 | 0.022 | 0.032 | 0.043 | 0.014 | 0.022 | 0.032 | 0.043 |
| Penstock head loss per m [m] for other $D_x$. Par = $(0.65 \text{ m}/D_x)^{5.3}$ | 6 | 1.0 | 0.01 | 0.02 | 0.03 | 0.04 | 0.01 | 0.02 | 0.03 | 0.04 |
| Total Penstock loss [m] for the length given in Par. | 7 | 6.0 | 0.08 | 0.13 | 0.19 | 0.26 | 0.08 | 0.13 | 0.19 | 0.26 |
| Effective Draft Tube Length [m] | 8 | 2.25 | 2.24 | 2.23 | 2.22 | 2.21 | 2.24 | 2.23 | 2.22 | 2.21 |
| Net head [m] | 9 | | 5.46 | 5.40 | 5.33 | 5.25 | 5.79 | 5.73 | 5.66 | 5.58 |
| Net theoretical power [kW] | 10 | | 21.40 | 26.46 | 31.34 | 36.02 | 22.70 | 28.08 | 33.28 | 38.28 |
| Turbine output [kW] Par = turb. eff. | 11 | 0.65 | 13.91 | 17.20 | 20.37 | 23.41 | 14.76 | 18.25 | 21.63 | 24.88 |
| Output (Gear efficiency) [%] | 12 | 0.90 | 12.52 | 15.48 | 18.33 | 21.07 | 13.28 | 16.43 | 19.47 | 22.39 |
| Output power (Gen. eff.) [%] | 13 | 0.80 | 10.02 | 12.38 | 14.66 | 16.86 | 10.62 | 13.14 | 15.58 | 17.91 |
| Total eff. | 14 | | 0.39 | 0.39 | 0.38 | 0.38 | 0.40 | 0.39 | 0.39 | 0.38 |

## Power and Efficiency as Function of Discharge; With Draft Tube - Best Case

| Water level in forebay [m] | 1 | Par. | 4.50 | | | | 4.75 | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Flow [l/s] | 2 | | 400 | 500 | 600 | 700 | 400 | 500 | 600 | 700 |
| Power theoretical [kW] | 3 | | 26.46 | 33.03 | 39.57 | 46.10 | 27.28 | 34.25 | 41.04 | 47.81 |
| Turbine head loss [m] = Par | 4 | 0.4 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 |
| Penstock head loss per m [m] for Ø = 0.65 m | 5 | 1.0 | 0.014 | 0.022 | 0.032 | 0.043 | 0.014 | 0.022 | 0.032 | 0.043 |
| Penstock head loss per m [m] for other $D_x$. Par = $(0.65\ m/D_x)^{5.3}$ | 6 | 1.0 | 0.01 | 0.02 | 0.03 | 0.04 | 0.01 | 0.02 | 0.03 | 0.04 |
| Total Penstock loss [m] for the length given in Par. | 7 | 6.0 | 0.08 | 0.13 | 0.19 | 0.26 | 0.08 | 0.13 | 0.19 | 0.26 |
| Effective Draft Tube Length [m] | 8 | 2.25 | 2.24 | 2.23 | 2.22 | 2.21 | 2.24 | 2.23 | 2.22 | 2.21 |
| Net head [m] | 9 | | 6.26 | 6.20 | 6.13 | 6.05 | 6.51 | 6.45 | 6.38 | 6.30 |
| Net theoretical power [kW] | 10 | | 24.54 | 30.38 | 36.04 | 41.50 | 25.52 | 31.61 | 37.51 | 43.22 |
| Turbine output [kW] Par = turb. eff. | 11 | 0.85 | 20.86 | 25.82 | 30.63 | 35.28 | 21.69 | 26.87 | 31.88 | 36.74 |
| Output (Gear efficiency) [%] | 12 | 0.95 | 19.82 | 24.53 | 29.10 | 33.52 | 20.61 | 25.53 | 30.29 | 34.90 |
| Output power (Gen. Eff.) [%] | 13 | 0.90 | 17.84 | 22.08 | 26.19 | 30.17 | 18.55 | 22.98 | 2726 | 31.41 |
| Total eff. | 14 | | 0.67 | 0.67 | 0.66 | 0.65 | 0.68 | 0.67 | 0.66 | 0.66 |

For a typical situation with:

| | |
|---|---|
| A draft tube length | 2.25 m |
| Net Head | 4.50 m |
| Discharge | 600 l/s |
| Turbine efficiency | 70% |
| Gearing efficiency | 93% |
| Generator efficiency | 89% |
| The output power comes to | 20.3 kW |

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

## Power and Efficiency as Function of Discharge. Without draft Tube

| Water level in forebay [m] | 1 | Par. | 4,50 | | | | 4,63 | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Flow [l/s] | 2 | | 400 | 500 | 600 | 700 | 400 | 500 | 600 | 700 |
| Power theoretical [kW] | 3 | | 17.64 | 22.05 | 26.46 | 30.87 | 18.15 | 22.69 | 27.22 | 31.76 |
| Turbine head loss [m] = Par | 4 | 0.5 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Penstock head loss per m [m] for Ø = 0.65 m | 5 | 1.0 | 0.014 | 0.022 | 0.032 | 0.043 | 0.014 | 0.022 | 0.032 | 0.043 |
| Penstock head loss per m [m] for other $D_x$. Par = $(0.65\ m/D_x)^{5.3}$ | 6 | 1.0 | 0.01 | 0.02 | 0.03 | 0.04 | 0.01 | 0.02 | 0.03 | 0.04 |
| Total Penstock loss [m] for the length given in Par. | 7 | 6.0 | 0.08 | 0.13 | 0.19 | 0.26 | 0.08 | 0.13 | 0.19 | 0.26 |
| Net head [m] | 8 | | 3.92 | 3.87 | 3.81 | 3.74 | 4.05 | 4.00 | 3.94 | 3.87 |
| Net theoretical power [kW] | 9 | | 15.37 | 18.96 | 22.40 | 25.66 | 15.88 | 19.60 | 23.17 | 26.55 |
| Turbine output power [kW] Par = turb. eff. | 10 | 0.70 | 11.53 | 14.22 | 16.80 | 19.25 | 11.91 | 14.70 | 17.38 | 19.91 |
| Output (Gear efficiency) [%] | 11 | 0.95 | 10.95 | 13.51 | 15.96 | 18.29 | 11.31 | 13.97 | 16.51 | 18.91 |
| Output power (Gen. Eff.) [%] | 12 | 0.9 | 9.86 | 12.16 | 14.36 | 16.46 | 10.18 | 12.57 | 14.86 | 17.02 |
| Total eff. | 13 | | 0.56 | 0.55 | 0.54 | 0.53 | 0.56 | 0.55 | 0.55 | 0.54 |

## 6. Civil works

### 6.1.1. The Forebay and the Intake

Based on (an uncertain) memory the walls of the old forebay have a height of approx. 1.65 m

The part of the sidewalls above ground are remembered to have a width of approx.

0.40 m

### 6.1.2. Stability of Walls of the Old Forebay

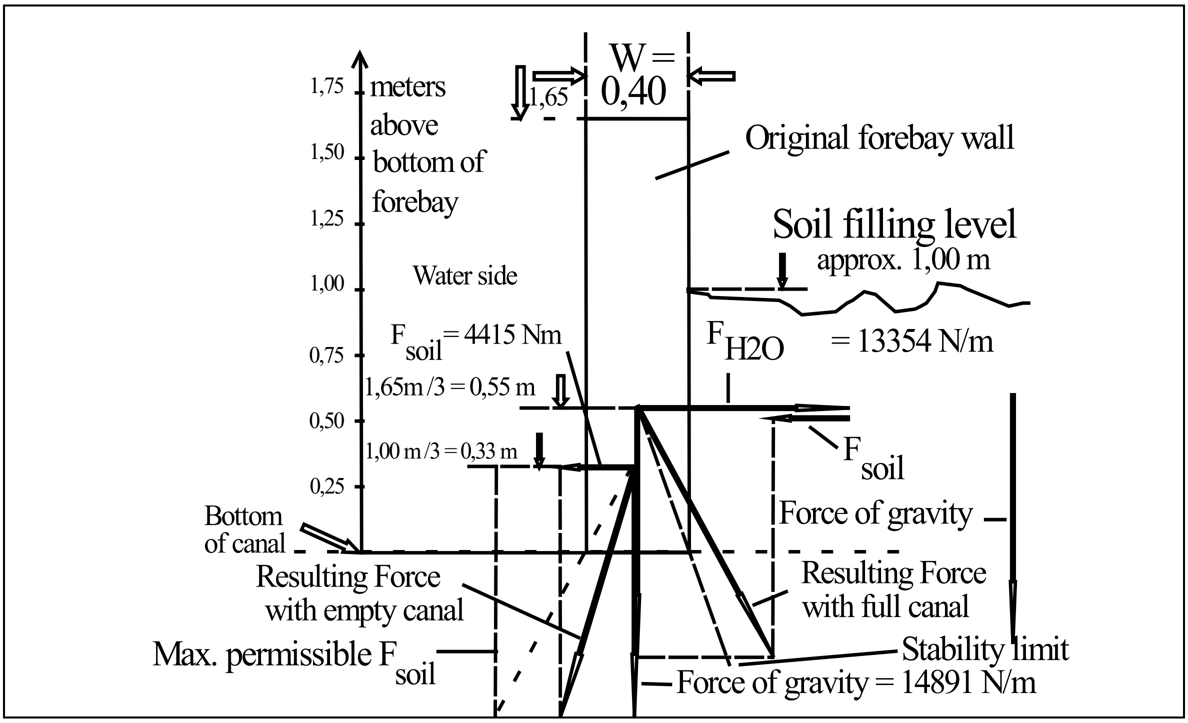



**Figure 5** Gravity, water, and soil forces acting on the old (existing) forebay walls

**Vertical Force.** The walls of the forebay produce a vertical (gravity) force $F_{Grav}$ (per unit length). The underground shape of the present walls is unknown. Yet, assuming they are rectangular with

| | |
|---|---|
| Height | 1.65 m |
| Width | 0.40 m |
| Density D of un-vibrated concrete | 2200 kg/m$^3$ |

The vertical (gravity) force is approx.

$F_{Grav\text{-}old}$ per meter = W * H * G * $D_{concrete}$ per meter

= 0.4 m * 1.65 m * 9.81 m/s$^2$ * 2200 kg/m$^3$ / m = 14243 N/m

Extending the wall to a height of 2.15 m will increase the vertical (gravity) force to:

$F_{Grav\text{-}new}$ per meter = W * H * G * $D_{concrete}$ per meter

= 0.4 m * 2.15 m * 9.81 m/s$^2$ * 2200 kg/m$^3$ / m = 18560 N/m

**Horizontal Forces.** The walls of the forebay and power canal are exposed to two horizontal forces:

1. From the water in the power canal / forebay
2. From the soil / stone-filling outside the power canal / forebay.

The walls must be designed to ensure stability in the two worst-case situations:

1. When the power canal is full, where water might cause the walls to fall outward.
2. When the canal is empty, in which case the soil might cause the walls fall into the canal.

The calculations are based on subsequent data for the old / existing walls:

| | |
|---|---|
| Height (H) | 1.65 m |
| Earth fill outside the power canal is approx. | 1.00 m |
| Part of walls above ground. | 0.65 m |
| Width (W) of walls approx. | 0.40 m |

The resulting horizontal force $F_{hoz}$ (per unit length [N/m]) on a wall, loaded up to a height H on one side by a material with a density D, is:

$$F_{hoz} = \int_0^H p(h)\,dh = \int_0^H ((H - h) * G * D)\,dh$$

$$G\,D \int_0^H (H - h)\,dh = G\,D\,[H\,h - \frac{1}{2} h^2\,]_0^H$$

$$G\,D\,(\,H^2 - \frac{1}{2} H^2\,) = \frac{1}{2} G\,D\,H^2$$

and the resulting torque T referred to the bottom of the wall is:

$$T = \int_0^H h\;p(h)\,dh = \int_0^H h\,((H - h) * G * D)\,dh$$

$$G\,D \int_0^H (Hh - h^2)\,dh = G\,D\,[\frac{1}{2} H\,h^2 - \frac{1}{3} h^3\,]_0^H$$

$$G\,D\,(\frac{1}{2} H^3 - \frac{1}{3} H^3) = \frac{1}{6} G\,D\,H^3$$

(The unit of torque per unit length is N = Nm/m, which equals Nm per meter being torque per meter).

The height $R_{resultant}$, at which, the resulting force will act is found from:

$T = F_{hoz} * R \Rightarrow R = T / F_{hoz} = (1/6\ G\ D\ H^3) / (1/2\ G\ D\ H^2)$

=> $R = 1/3\ H$

Using data for the old forebay wall:

| | | |
|---|---|---|
| H | The height of the load (on one side of the wall) | 1.65 meter |
| G | The constant of gravity | 9.81 $m/s^2$ |
| $D_{H2O}$ | The density of the water | 1000 $kg/m^3$ |

yields a horizontal force of

$F_{Hoz\ H2O}$ = 1/2 * 9.81 $m/s^2$ * 1000 $kg/m^3$ * $(1.65\ m)^2$ = 13354 N/m

Acting at a level of R = 1/3 * H = 1/3 * 1.65 m = 0.55 m

Giving a Torque $T_{H2O}$ = R * $F_{Hoz\ H2O}$ = 0.55 m * 13354 N/m = 7345 Nm/m

The horizontal force of the water will partly be counteracted by the horizontal force of the soil filling outside the power canal. This force $F_{soil}$ can be adjusted by changing the filling level i.e. how high, the soil will rest towards the outside walls.

Only a fraction of the pressure of the soil will act horizontally as a horizontal force $F_{soil}$ on the wall. The magnitude of the fraction, C, of the pressure equals sine to the angle between horizontal and a side of a heap of the same kind of soil and the ground.

In this case, the angle of a heap of local soil has been assumed to be approx. 30°

Yielding a factor of C = 0.5

The soil density $D_{soil}$ is assumed to be 1800 $kg/m^3$

and the filling height of 1.00 m

$F_{hoz\ soil}$ = C * ½ * G * $D_{soil}$ * $H^2$ = 0.5 * ½ * 9.81 $m/s^2$ * 1800 $kg/m^3$ * $(1.00\ m)^2$ = 4415 N/m

Acting at a level of R = 1/3 * H = 1/3 * 1.00 m = 0.33 m

Giving a Torque $T_{soil}$ = R * $F_{soil}$ = 0.33 m * 4415 N/m = 1457 Nm/m

The vectors representing the forces (see the figure) shows, that the resulting force intersects the base outside the base of the wall. Hence, unless the lower part of the old wall (the shape of which is unknown) is different from the assumed rectangular shape, the structure is unstable. If the walls of the forebay walls are as assumed, there is a risk that the forebay will collapse if filled to the rim.

Increasing the soil filling level can reduce the danger of collapse. At the figure, the lower left dotted lines show, that the sum of the horizontal components of $F_{soil}$ and $F_G$ can be increased to approx. the double of the present, without risking the wall to collapse, when the power canal is empty.

Even if the walls would not be extended, it is still recommendable to raise the soil filling level.
The magnitude of the horizontal force of the soil increases by the soil filling level to the power of 2.
The height above the base, at which the force acts, increases proportionally to the soil filling level.
The horizontal components of $F_{soil}$ and $F_G$ increases by the soil filling level to the power of 3.

As the sum of the horizontal components of $F_{soil}$ and $F_G$ can be increased by a factor of 2, the level to which the soil should be filled at the outside of the walls, should be raised from approx. 1 m
to a level above the bottom of the power canal = $2^{1/3}$ * 1 m
= 1.23 m

Raising the soil filling from 1.00 m to 1.23 m will increase $F_{soil}$ from approx. 4415 N/m to 6653 N/m, thus reducing the resulting horizontal force (when the power canal is full of water) to approx. 6700 N/m. This will bring the resulting force (with water) to intersect just within the wall base, very near to the bottom right corner of the wall - thus significantly increasing the static stability of the wall.

### 6.1.3. Increasing the Height of the Power Canal- and the Forebay-Walls

The drop of the section of the power canal from the sharp corner near to the paved road to the forebay is approx. 0.35 m. In order to increase the net head, the plan is to raise the walls of this section of the power canal and of the forebay. The height will be increased by approx. 0.5 m yielding approx. 0.15 m of free board.

(If the height of the walls of the power canal on the upper section (approx. 600 m) from the sharp corner to the intake would be increased, the discharge and head could also be increased. This will, however, be left for a possible later upgrading of the plant.)

**The Spillway.** The extended walls of the forebay must provide sufficient freeboard in all situations - including if the gate over the trash rack is suddenly closed. This freeboard is ensured by placing the spillway at the upstream side of the forebay (i.e. at the side towards the existing spillway). The width of the spillway must be sufficient to allow for the max. discharge to pass over the spillway without raising the water level to the rim of the forebay.

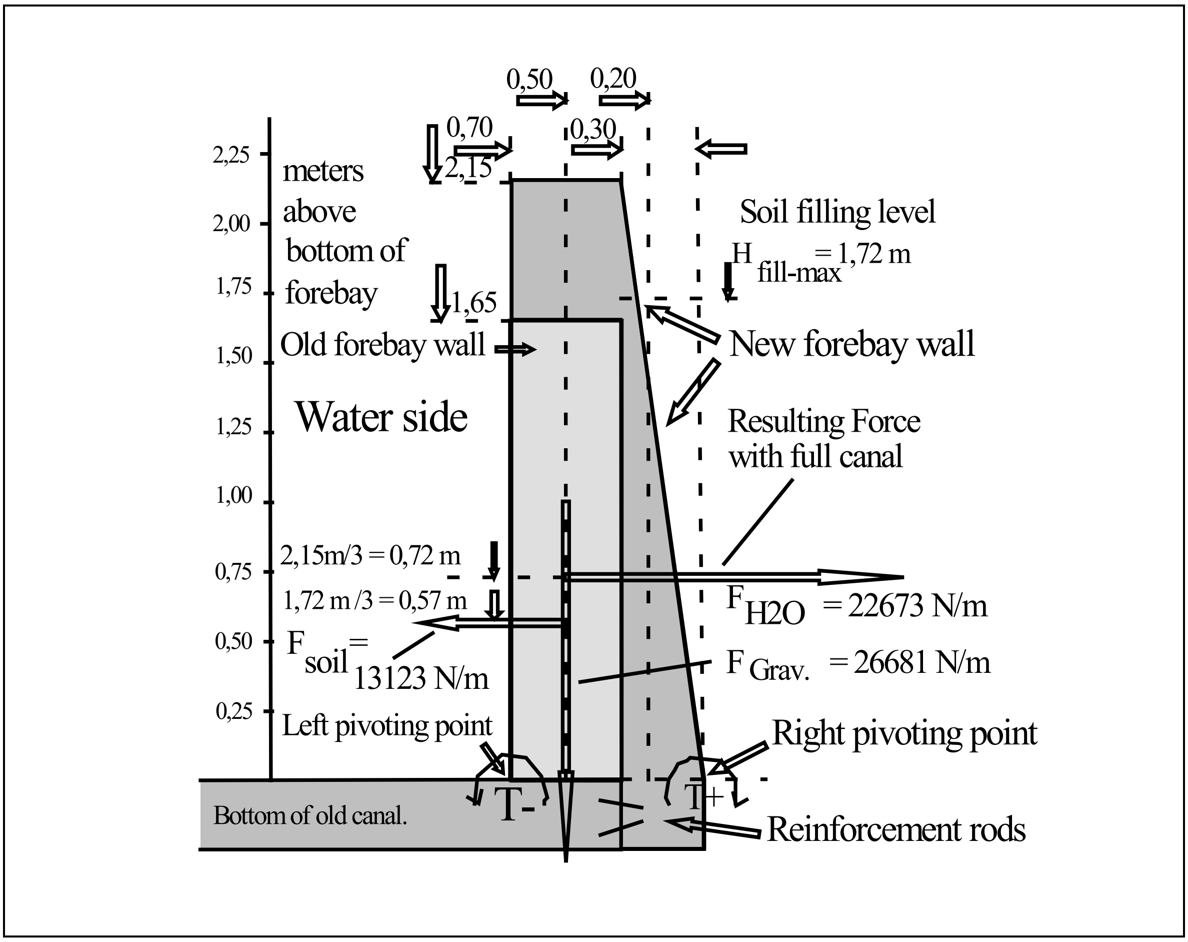
As shown in section 6.6 (The Baffle Wall) a flow of 721 l/s through a weir with a width of 11 m will produce a raise in the water level of 0.11 m. Thus, the spillway will be implemented as a suspended part of the wall of the forebay, where the top of the wall is lowered to the nominal water level.

The width (/ length) of the spillway is chosen to be 11 m

The maximum discharge $Q_{max}$ through the spillway is calculated for h = 0,15 m:

$Q_{max} = 1.8\ (11\ m - 0.2 * 0.15\ m)\ (0.15\ m)^{3/2}$ 1147 l/s

### 6.1.4. Stability of Walls of the New Forebay




**Gravity, water, and soil forces acting on the New forebay walls**

For the new forebay wall to be stable, its width must be increased towards the bottom as shown on the figure above. The required increase of the width of the bottom of the walls is found to be 0.7 m.

The deduction of the point of gravity in the extended structure is provided on the following page. (Extending the height of the walls without increasing the base of the existing wall has been calculated and proved not to be safe.)

Calculating the torque, the wall is regarded as a solid unit standing loose at the bottom of the forebay.

When pos. (clockwise) torques are applied, the wall is regarded as being hinged at the right corner.

When neg. (counterclockwise) torques are applied, the wall is regarded as hinged at left corner.

**The torque of the water.** In this case the (extended) wall can be regarded as being hinged (to the base of the forebay) at the outer (right) bottom side.

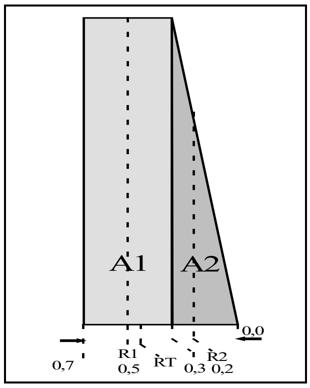
The torque $T_{H2O}$ applied to the wall - when water stands to the rim of the power canal / forebay - is:

$F_{hoz\text{-}H2O} = ½ * 9.81\ m/s^2 * 1000\ kg/m^3 * (2.15\ m)^2$ = 22673 N/m

$T_{H2O+} = F_{hoz\text{-}H2O} * R_{R2O} = ½ * G * D_{H2O} * H^2 * 1/3 * 2.15\ m$ = 16249 Nm/m

This torque ($T_{H2O}$) must be counteracted by:

1. The torque $T_G$ formed by the gravity / weight of the wall - and
2. The torque $T_{soil}$ applied by the soil filling outside the wall/canal.




Point of gravity of a composed structure.

**Point of gravity of the combined structure** to find the torque $T_G$ (produced by the gravity). First the point of gravity of the combined structure consisting of the left rectangle and the right triangle must be found using subsequent equation:

$$A_T * R_T = A_1 * R_1 + A_2 * R_2$$

where:

$A_T$ is the area of the combined structure consisting of the left rectangle and the right triangle.

$R_T$ is the distance from the origin to the resulting point of gravity.

$A_1$ is the area of the left rectangle

$R_1$ is the distance from the origin to point of gravity of the left rectangle.

$A_2$ is the area of the right triangle.

$R_2$ is the distance from the origin to point of gravity of the right triangle.

As the Point of reference / origin the lower right (outer) corner of the triangle is chosen.

$R_1$ The point of gravity of the rectangle lies in the middle of the rectangle, which has a width of 0.4 m. Thus, from the middle to the side of the triangle is 0.2 m - plus the width of the triangle 0.3 m makes $R_1$ = 0.5 m

$R_2$ The point of gravity of the triangle lies at 2/3 of the width of the triangle, (which has a width of 0.3 m). $R_2$ = 2/3 * 0.3 m = 0.2 m

$A_1$ is area of the left rectangle is: 0.4 m * 2.15 m 0.86 $m^2$

$A_2$ is area of the right triangle is: 1/2 * 0.3 m * 2.15 m 0.32 $m^2$

$A_T$ = 0.86 $m^2$ + 0.32 $m^2$ 1.18 $m^2$

$R_T$ = $A_1 * R_1 + A_2 * R_2 / A_T$
= (0.86 $m^2$ * 0.5 m + 0.32 $m^2$ * 0.2 m) / 1.18 $m^2$ = 0.418 m

**The gravity** produces a force and a torque of:

$F_G = A_T * G * D_{concrete}$ = 1.18 $m^2$ * 9.81 $m/s^2$ * 2300 $kg/m^2$ 26681 N/m

$T_G = R_T * F_G$ = 0.418 m * 26681 N/m 11157 Nm/m

The force and torque of the soil filling outside the canal must provide at least the difference

between $T_{H2O}$ = 16249 Nm/m

and $T_G$ = 11157 Nm/m

=> $T_{soil}$ >5092 Nm/m

**The Soil Filling.** The max. soil filling level, (towards the sides of the canal), which will be safe (to avoid the risk of the wall falling into the canal, when the canal is empty), is calculated. Because the force of the soil $F_{soil}$ would cause the wall to fall into the canal, the wall is now regarded as being hinged (to the base of the forebay) at the bottom inner (left) side.

With the wall hinged at the inner left side, the force of gravity is the same, but the horizontal distance from the new point of reference (left pivoting point) to the point of gravity $R_{T'}$ now changes to:

$R_{T'} = W_{Bot.} - R_T = 0.7$ m – 0.418 m = 0.282 m

$T_{G'} = 0{,}282$ m * 26681 N/m 7524 Nm/m

The torque of the soil is $T_{soil} = R * F_{soil}$. – and $F_{hoz\ soil} = C * 1/2 * G * D_{soil} * H^2$ (C = 0,5)

R = 1/3 * H (where H is the height of the load - in this case the filling level of the soil)

=> $T_{soil} = R * F_{soil} = (1/3\ H) * (C * 1/2 * G * D_{soil} * H^2) = 1/6 * C * G * D_{soil} * H^3$

$$T_{soil\text{-}max} = 1/6 * C * G * D_{soil} * H_{max}^3$$

=> $H_{max} = \{(6\ T_{soil\text{-}max} / (C * G * D_{soil})\}^{1/3}$ (for the coefficient C = 0.5; see point 6.1.2).

$H_{max} = \{(6 * 7524\ \text{Nm/m} / (0.5 * 9.81\ \text{m/s}^2 * 1800\ \text{kg/m}^3)\}^{1/3}$ 1.72 m

$F_{soil} = T_{soil} / (1/3\ H_{soil}) = 3 * 7524$ Nm/m / (1.72 m) = 13123 N/m

Acting at R = 1/3 * 1.72 m = 0.57 m

As the sum of torques - counteracting the torque of the water - is higher that the latter, the structure is stable.

$$T_{grav} + T_{soil} > T_{H2O}$$

11157 Nm/m + 7524 Nm/m = 18681 Nm/m > 16249 Nm/m

The safety margin is 15 %, which is a bit low. Other factors do, however, add to the safety margin:

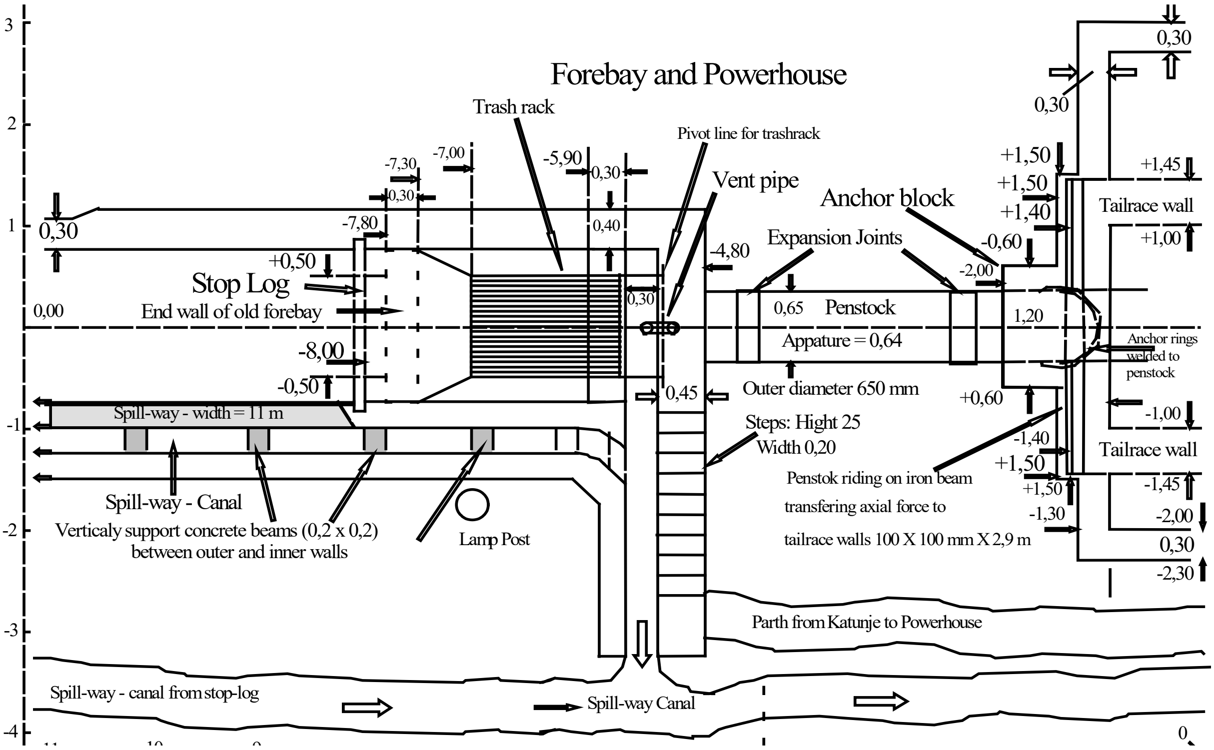
1. $T_{soil}$ will be somewhat higher, as the vertical component of the force on the sloping outside of the wall is here neglected.

2. Reinforcement rods will be poured into the bottom of the existing canal securing good connection between the new outside walls and the base of the old canal.

**Ensuring Watertight Joints Between Old and New Concrete.** To secure watertight joints between existing concrete (poured in 1998) and new concrete, the top layer (approx. 5 cm) of the existing walls must be removed. Reinforcement steel rods with a diameter of 6 - 8 mm must be anchored in the existing wall with a spacing of 20-30 cm. and with a distance from the inner side of the wall of approx. 5 cm. Every second rod should be mounted so that it tilts up-stream and all other rods (in between those tilting up-stream) should tilt down-stream.

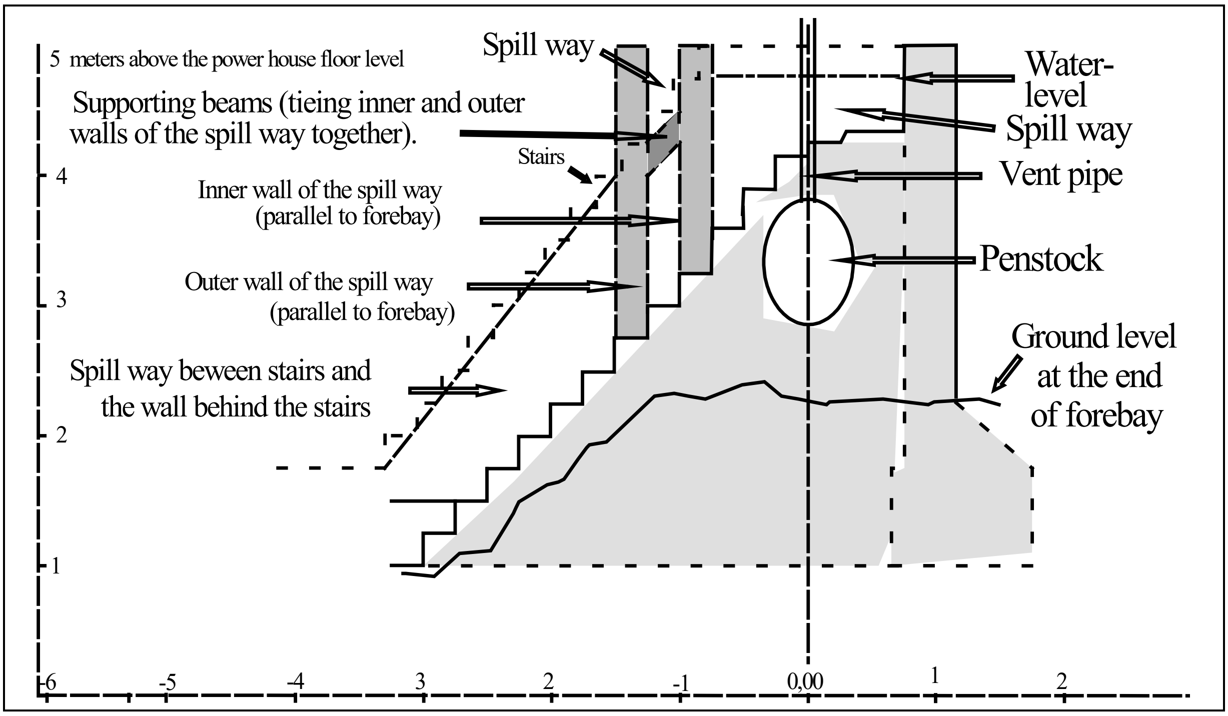
Just before the new concrete is poured, the old concrete is covered with a thin layer of cement-milk -
one-part cement to 3-part water – measured by volume.

## 6.2. The Anchoring of the Penstock




**The Forebay and Trash Rack Layout**

**Post Trash Rack Spillway and Stairs to the Forebay**




**Cross Section of Spill Way; Just Upstream of Vent-pipe**


Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

The vertical and horizontal forces of the penstock and the water in the penstock must be taken up-(/supported) at the entrance to the powerhouse (otherwise the northern wall of the draft tube pit will collapse.) The lay out of the powerhouse, invites to use the sidewalls of the draft tube pit to transfer the axial forces from the penstock into the foundation. This also reduces the requirements in terms of the size of the anchor block.

The size of the penstock is:

| | |
|---|---|
| Outer diameter $Ø_{pen}$ | 650 mm |
| Thickness $t_{pen}$ | 4.5 mm |
| Length $l_{pen}$ | 6.0 m |
| Slope α, tan α = 3.23 m / 6.0 m => α | = 28.4° |
| Material | mild steel |
| Weight of penstock ($Ø_{pen}$ * π * $t_{pen}$ * $l_{pen}$ $D_{iron}$ | |
| = 650 mm * π * 4.5 mm * 6.0 m * 7860 kg/m$^3$ | 433 kg |
| Weight of water in penstock ($Ø_{pen}$/2)$^2$ * π * $l_{pen}$ $D_{H2O}$ | |
| = (650 mm/2)$^2$ * π * 6.0 m * 1000 kg/m$^3$ | 1990 kg |
| Total weight $M_{penst+H2O}$ of water and penstock | 2424 kg |

During normal operation there will be no horizontal force $F_{hoz-pen}$ acting on the penstock. If, however, the intake stop log is opened very quickly, water will flow down the penstock and hit the turbine at the end of the penstock as a hammer. Within the hydropower sector, this phenomenon is called water hammer, and it is the source of both fear and discussion, as the way the air in the penstock will be either pushed out or trapped and hence the forces on the penstock are more or less unpredictable.

The following calculation is done for one specific situation to quantify the magnitude of the horizontal force, by which the penstock will act on the H-beam placed (and concreted) inside the wall of the powerhouse. For the calculation it is assumed, that the stop log will be raised instantly, and that the water will be moving as a solid cylinder inside the penstock. Also, it is assumed, that the guide vane in the turbine is closed, that the air trapped in the penstock will be compressed, and in this way will break the flow of the water.

The energy $E_{potential}$ gained by the water when flowing from the forebay to the turbine level is
$E_{potential}$ = M * G * H,
Where:
M is the mass of water,
G is the constant of gravity
H is the vertical distance from the water level of the forebay to the turbine.

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

Compressing the trapped air will require an energy $E_{comp}$

$$E_{com} = -\int_{l_o}^{l} F(l)\,dl = -\int_{l_o}^{l} A\ P(l)\,dl$$

$$also\quad P_o\ V_o = P(l)\ V(l)\ \ =>\ \ P(l) = P_o \frac{V_o}{V(l)} = P_o \frac{l_o}{l}$$

$$=> -\int_{l_o}^{l} A\ P_o \frac{l_o}{l}\,dl = -A\,P_o\,l_o\,[\ \ln(l)\ ]_{l_o}^{l}$$

where

F(l) is the force as function of the length of the air cylinder.
A is the cross section of the penstock
P(l) is the pressure as function of the length of the air cylinder.
V(l) is the volume of the air cylinder as function of the length.
$l_o$ is the length of penstock, i.e. the length of the air cylinder before compression
$l_{H2O}$ is the height of a water column yielding a pressure of 1 bar: (i.e. approx. 10 m).

When the water is brought to a stop by the compressed air, the potential energy $E_{potential}$ will equal the energy used to compress the air in the penstock. Multiplying the pressure by the area of the penstock yields the resulting horizontal force, with which the penstock will act on the civil structure. As only the front part of the water has gained a potential energy equal to M G H, and whereas the water just entering the penstock has not yet gained any potential energy, the total $E_{potential}$ of the entire water column is
$E_{potential}$ is = 1/2 M G H.

$$E_{potential} = E_{comp} =>$$

$$\frac{1}{2} M\ G\ H = -A\,P_o\,l_o\,[\ \ln(l)\ ]_{l_o}^{l}$$

$$\frac{1}{2}\ A\ l_o\ D_{H2O}\ G\ H = A\ P_o\ l_o \ln\left(\frac{l_o}{l}\right)$$

$$=> \ln\frac{l_o}{l} = \frac{D_{H2O}\ G\ H}{2\,P_o} = \frac{D_{H2O}\ G\ H}{2\,l_{H2O}\,D_{H2O}\,G}$$

$$=> l = l_o\ e^{-\frac{H}{2\,l_{H2O}}}$$

Inserting $l_{H2O}$ = 10 m and H = 4.68 m yields:

$l = 0.791\ l_o \ \ =>\ \ P(l) = 0.791^{-1} * P_o$

The resulting force acting on the structure is
(P(l) - P) A = (1.263-1.00) * 10 m * 1000 kg/m$^3$ * 9.81 m/s$^2$ * (0.65 m/2)$^2$ * π = 8561 N

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Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

**Deflection of H-beam.** The task of the H-beam is to minimize the horizontal deflection. Hence, the beam shall be placed such, that it has its maximum section modulus (i.e. its resistance to deflection) in the horizontal plane, i.e. the two parallel sections of the beam must be vertical.

The impact of the water will create a force $F_{imp,}$ which will cause a deflection d of the H beam. The H beam is regarded as suspended between the two tailrace walls 2000 mm apart.

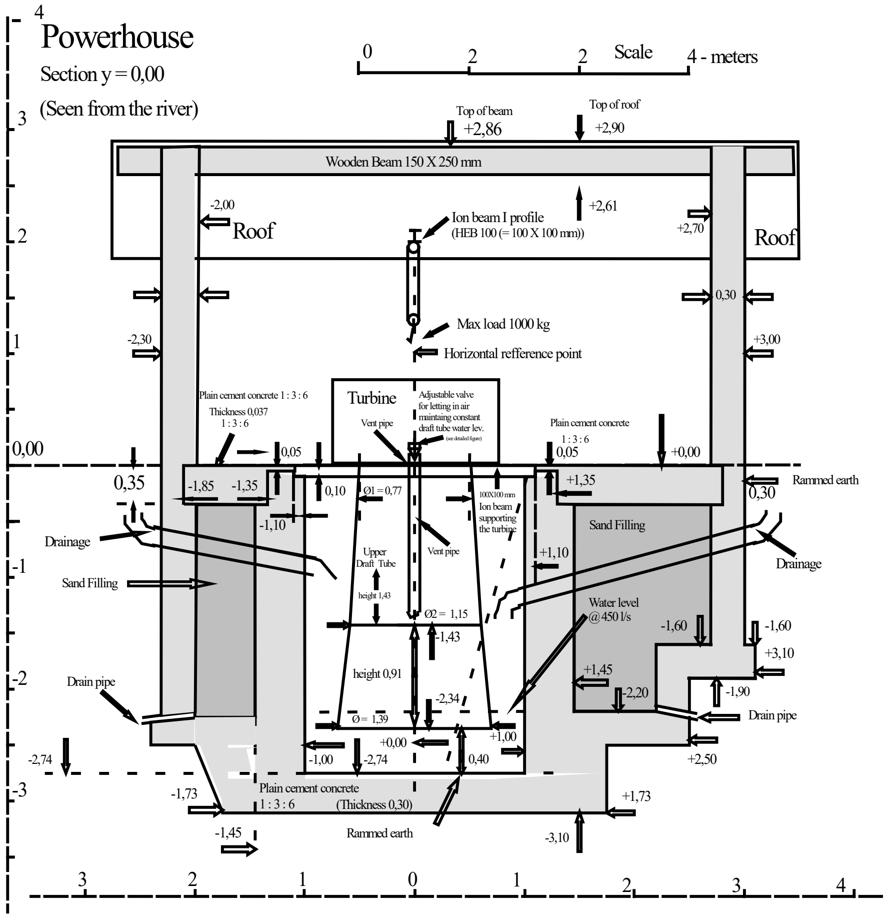
The force $F_{imp}$ will act at the middle of the beam creating a torque
$T_{beam}$ (in the beam) = 1/4 $F_{imp}$ $l_{beam}$ = 1/4 * 8561 N * 2000 mm 4.28 $*10^6$ Nmm

The H beam is chosen as a standard HEB 100 x 100 (Euro standard 531962)
with a section modulus $I_y$ = 4.5 * $10^6$ $mm^4$
Young's module $e_{steel}$ for steel = 2 * $10^5$ $N/mm^2$
The deflection d = 1/12 ($T_{beam}$ * $l_{beam}^2$) / ($e_{steel}$ $I_y$)
1/12 (4,28*$10^6$ Nmm * $(2000\ mm)^2$) / (2 * $10^5$ $N/mm^2$ * 4.5 $10^6$ $mm^4$) = 1.58 mm

A rule of thumb states, that a concrete wall will not crack as long as it's deflection per unit length remains below 0.002. Thus, the length between the supports of the beam is 2000 mm, allows for a max. deflection of 4 mm. Hence, with these assumptions the deflection is well within limit. In addition, the force to raise the stop log will require a significant time to open the stop log, causing the penstock to fill up slowly.

This is by no mean a depleting analysis of the situations, which can occur when water is suddenly applied to the penstock. It is merely an indicator of order of magnitude. In general, a shock breaking function can be obtained by placing a closed vertical pipe just before the inlet to the turbine. The air trapped in the pipe will lengthen the de-acceleration of the water, thus reducing the force required.

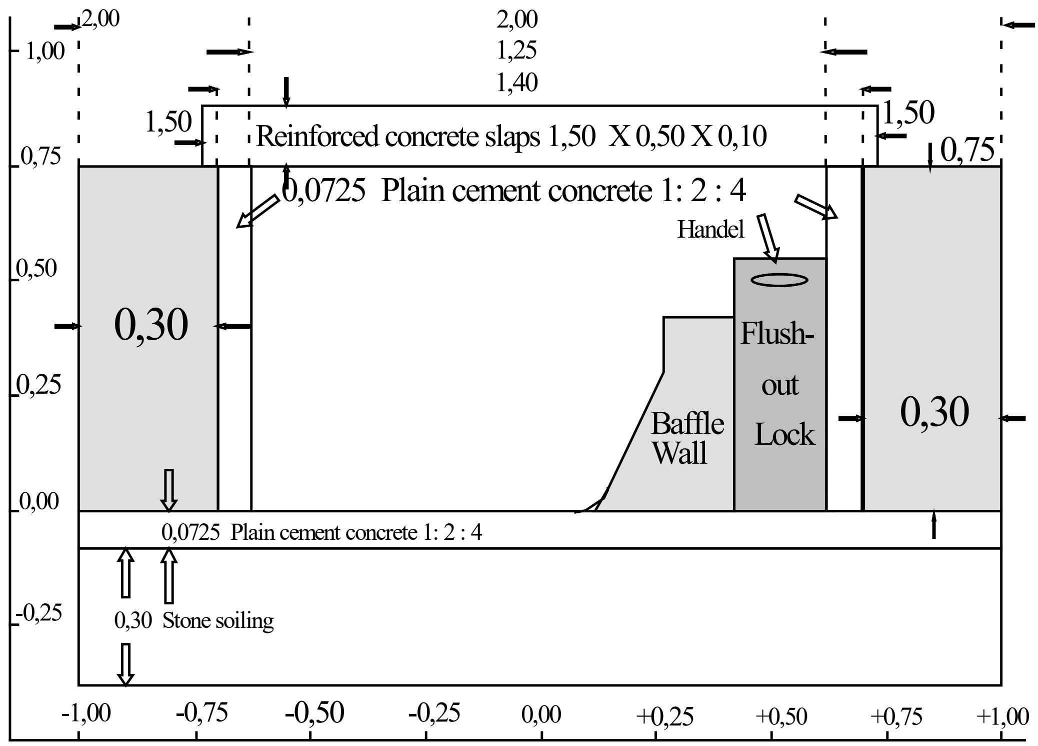
## 6.3 The Powerhouse




**The Powerhouse and the Tailrace**

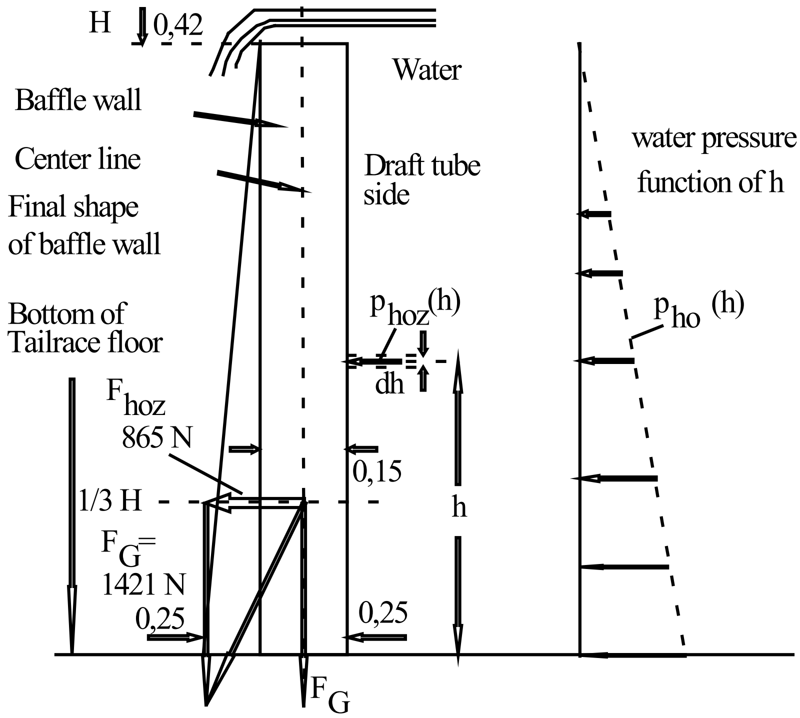
The first / old / existing powerhouse is placed just above the water level of the flood, which occurred 35 years ago, as it is remembered by the old people in the village. With the new lay out, the new powerhouse will be situated approx. 5 meters further back. This will require digging into the existing hillside just below the forebay. The new placement of the house adds a further protection against floods.

## 6.4. The Tailrace




**The Tailrace and its Baffle wall**

### 6.5 The Baffle Wall Stability




**The baffle wall within the tailrace**

One of the sides of the baffle wall in the tailrace will be exposed to the horizontal force from the water in the side of the draft tube. This is because on the draft tube side, water will rise to - and depending on the discharge - above the top of the baffle wall.

The maximum horizontal pressure of the water is $H_{baffle}$ * G * $D_{H2O}$, where:

| | | |
|---|---|---|
| $H_{baffle}$ | is the height of the baffle wall | 0.42 meter |
| G | is the constant of gravity | 9.81 m/s$^2$ |
| $D_{H2O}$ | is the density of water | 1000 kg/m$^3$ |
| h | is the height of above the bottom of the tailrace | 0.42 meter |

This gives a maximum horizontal pressure at the bottom of the baffle wall of

$P_{Hor.} = H_{baffle} * G * D_{H2O} = 0.42$ m * 9.81 m/s$^2$ * 1000 kg/m$^3$ = 4120 N/m$^2$

At the riverside of the baffle wall, the water level will depend on the discharge, which, for the worst case, will be zero. Thus, in this case there is no counteracting force to be accounted for from the riverside.

For a stable design, the concrete must never take up any tensile strength. Thus, the sum of:

1. The vector representing the force of the gravity of the wall.
2. The vector representing the force of the water acting on the wall.

must fall on a line, which must remain within the cross section of the structure, for the wall to be stable.

The total (i.e. mean) horizontal force $F_{hoz}$ (per unit meter) imposed by the water on the baffle wall is:

$F_{hoz.}$ = 1/2 * $H_{baffle}$ * $H_{baffle}$ * G * $D_{H2O}$
= 1/2 * $(0.42\ m)^2$ * 9.81 $m/s^2$ * 1000 $kg/m^3$ 865.24 N per meter

To find the height $H_T$, at which the horizontal force $F_{hoz}$ acts on the wall the torque (which the water imposes on the baffle wall) - $H_T$ is found.

To find the torque, the baffle wall is regarded as if it was hinged to the bottom of the tailrace. The horizontal force per square $p_{ho}$ [$N/m^2$] of the pressure decreases proportionally with height above the bottom of the tailrace, according to:

$$F_{ho} = (H - h) * G * D_{H2O}.$$

The torque of the water refereed to the bottom of the baffle wall is:

$$T = \int_0^H h\ p(h)\ dh$$

$$= \int_0^H h\ ((H - h) * G * D_H\ 2O)\ dh$$

$$G\ D_{H2O} \int_0^H (Hh - h^2)\ dh$$

$$= G\ D_{H2O} [\frac{1}{2} H\ h^2 - \frac{1}{3} h^3\ ]_0^H$$

$$G\ D_{H2O} (\frac{1}{2} H^3 - \frac{1}{3} H^3)$$

$$= \frac{1}{6} G\ D_{H2O}\ H^3$$

The unit is N (which equals Nm per meter i.e. torque per meter). The torque T equals $H_T$ * $F_{hoz}$, thus

$$H_T = \frac{T}{F_{hoz}} = \frac{\frac{1}{6} G\ D_{H2O}\ H^3}{\frac{1}{2} G\ D_{H2O}\ H_2} = \frac{1}{3} H$$

Hence the horizontal force $F_{hoz.}$ will act at a height (above bottom of the tailrace) of
1/3 H = (0.42 m) / 3 0.13 m

For a rectangular concrete wall, the vector representing the gravity force will act on the centre line. The density $D_{con}$ of concrete is approx. 2300 $kg/m^3$
The weight of the baffle wall (per unit length) is W * H * G * $D_{con}$ =
0.15 m * 0.42 * 9.81 $m/s^2$ * 2300 $kg/m^3$ 1421 N/m

Looking at the figure of the baffle wall the intersection of:
1 The centre line of the original rectangular baffle wall
2 The horizontal line, at which the vector representing the horizontal force attacks

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.

is found. From this point the sum of the vectors is composed. The line corresponding to the sum of the sum of the vectors intercept with the bottom of the tailrace approx. 0.13 m from the centreline of the baffle wall.

Thus, based on this to ensure stability (and to be on the safe side), a slope of 0.10 m / 0.42 m will be added to the side of the baffle wall (towards the river). The side towards the draft tube remains horizontal. This gives a top width of 0.15 m and a bottom width of 25 m.

**8. Acknowledgement**
In this paper, numerous references are given to “MICRO-HYDROPOWER SOURCEBOOK” – a Practical guide to Design and Implementation in Developing Countries” by Allen R. Inversin.

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Steen Carlsen

Carlsen Power Electronics. The Science Park, Gustav Wiedsvej 10, 8000 Aarhus C. Denmark;
Tel. +45-86-19 03 19; E-mail: carlsen@power-electronics.dk.