Small hydropower plants standardization, between myth and reality

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Abstract. Many providers for small hydropower plants equipment have tried to standardize the components and even the entire equipment. So called "compact turbines" were launched on the market, ensuring the pre-designed solution of the modular components, but usually with lower efficiency than turbines specially designed for a certain site. For civil works it is possible to standardize some components, such as the powerhouse, the surge tank or the headrace, but not the intake and the weir.

Part of the hydropower plants can be standardized, but not the entire project, because there are a lot of variables that influence the design. Among these, the dimension, materials and design of the canal and the penstock are given by the hydrology, topography and the geology of the project's area.

This paper presents an attempt at standardization by using different heads and different installed flows. The case study is made on the Lukosi River from Tanzania, because there is a good hydrological database on power and energy calculation. For the powerhouse, pressure tower and intake dimensioning, the assumptions and materials considered cover all challenges that could appear in the geological and topographical structure of the project's area (worst case, most expensive). The study has highlighted African climatologic and hydrological conditions and the adapting of current technology to these conditions.

1. Introduction

An important question that remains viable in time is: “Can the small hydropower plants be standardized?”

Many equipment providers have tried to standardize the equipment or the components; many producers have launched on the market so called “compact turbines” that ensure the pre-design solution of the modular components. But will they reach the same efficiency as a site design turbine?

It is obvious that the water quality and the materials used imply certain conditions in order to use compact turbines. For civil works it is possible to standardize some of their components, such as the powerhouse, surge tank or headrace, but not the intake and the weir.

This article presents an attempt at standardization by using different heads and different installed flows. The case study is made on the Lukosi River from Tanzania, because the authors have a good...
hydrological database on power and energy calculation, as reference. The research study has been drawn up at the end of 2015.

1.1. Project location
For this research study, the location of the project was the Lukosi River, Tanzania, due to the particularities that the African river has, compared to the European rivers. Referring to standardization, this shall be applied everywhere in any topographical, geological and hydrological conditions.

Tanzania is a republic in East Africa, on the Indian Ocean (Figure 1). A diverse country in which almost 100 different languages are spoken, Tanzania was formed by the federation of the nations of Tanganyika and Zanzibar in 1964. The country’s name is a combination of the first syllables of the component territories’ names. Tanzania is bounded in the north by Kenya and Uganda; in the east by the Indian Ocean; in the south by Mozambique, Malawi, and Zambia; and in the west by the Democratic Republic of Congo (DRC), Burundi, and Rwanda. The country includes the islands of Zanzibar and Pemba, and other offshore islands in the Indian Ocean. Dar es Salaam is the executive capital and the largest city; the smaller city Dodoma is now the legislative centre of Tanzania and has been designated as the capital.

Geography. The landscape of mainland Tanzania is generally flat and low along the coast, but a plateau that has an average altitude of about 1,200 m (about 4,000 ft) constitutes the greater part of the country. Isolated mountain groups rise in the northeast and southwest. The volcanic Kilimanjaro (5,895 m/19,341 ft), the highest mountain in Africa, is located near the north-eastern border.

Three of the great lakes of Africa lie on the borders of the country and partially within it. Lake Tanganyika is located on the western border, Lake Victoria on the northwest, and Lake Malawi on the southwest. Lakes Malawi and Tanganyika lie in the Great Rift Valley, a tremendous geological fault system extending from the Middle East to Mozambique.

Iringa Region. Iringa region lies in the Southern Highlands of Mainland Tanzania. It stretches from the semi-arid central Tanzania in the north to the shores of Lake Nyasa in the South. The region is located between 7005’ - 36032’ South and 33047’ – 36032’ East. In the North, the Iringa region borders the Dodoma region, the Mbuya region to the West and the Morogoro region to the East. To the South, the Iringa region partly borders the Ruvuma region and partly Lake Nyasa. The region lies between latitudes 70 and 90 South of Equator, and between longitudes 32º and 35º East of the Greenwich Meridian. The Iringa region lies at an altitude of 475 masl (meters above sea level) with high peaks of 2,981 masl.

Climate. Tanzania is a big African country, overlooking the Indian Ocean, lying just south of the Equator. Most of the country is covered by a plateau, home to a large amount of world-famous touristic attractions, with several parks and nature reserves in the environment of the savannah. This plateau has a tropical climate, warm but tempered by the altitude. The thin flat coastal stretch is rather hot and humid throughout the year, especially from November to April. As regards the rain pattern, the country can be divided into four zones (Figure 2). In the north and east, excluding the region of Lake Victoria, there are two rainy seasons: one less intense, known as short rains season between October and December, and the other more intense, known as long rains season from March to May, with the peak in April.

There were two site visits in the study location: first between 16th September and 21st September 2015 and the second in October 2015. The rainy season was about to start, but everything was very wild and dry on these mountains. Lukosi River is a large river of Tanzania, with at least 10 m³/s in the dry season.
2. Hydrological Data

2.1. Great Rwaha Basin
The Great Ruaha River is a river in south-central Tanzania that flows through the Usangu wetlands and the Ruaha National Park east into the Rufiji River. Its basin catchment area is 83,970 km². The population of the basin is mainly sustained by irrigation and water-related livelihoods such as fishing and livestock keeping. Great Ruaha is about 475 km long, its tributary basin has a catchment area of 68,000 km² and the mean flow is 140 m³/s.

The major rivers contributing to the Great Ruaha River are Lukosi (Figure 3), Yovi, Kitete, Sanje, Little Ruaha, Kisigo, Mbarali, Kimani and Chimala whereas the small ones include Umrobo, Mkoji, Lunwa, Mlomboji, Ipatagwa, Mambi and Mswiswi rivers.
2.2. Historical Hydrological Data
The project area was visited twice, in September 2015 together with RPA representatives and in October 2015 together with CAL’s experts. Two spot measurements were taken in the time of the site visits and presented in the Site Visit Report.

**Available data and studies.** The following information / studies were collected from the Developer:
- Preliminary Estimate of Water Resource Availability: Flow Duration Analysis of Stream Discharge Records recorded at Mtandika Gage Station from 1957 to 1987 On the Lukosi River, Tanzania, J.L. McAdoo, Hydrologist, February 2015;
- Mean annual rainfall map;
- Lower Lukosi River Elevation Profile from 90m DEM From Confluence with the Ruaha to Confluence with the Mlowa;
- Upper Lukosi River Elevation Profile from 90m DEM Confluence of the Lukosi with the Mlowa To Its Headwaters;
- Matindika - 1957 - 2010 data;
- Monitoring Stations_Metadata.

2.3. Measurements on River [2]
The finding of the mean flow was 12 m$^3$/s from visual estimation and measurement with the propeller (Figure 4). Lukosi River has a lot of tributaries, but in the dry season the tributaries are completely dry (Figure 5). Lukosi River’s banks are almost non-accessible. The area is mostly rocky with few inhabitants and to build access roads will be a real challenge (involving blasting works). But this aspect will be analyzed in the geological survey that will be the next step of the hydropower development study.

![Figure 4. Measurements of river using propeller](image1)

![Figure 5. Lukosi River Tributaries in dry season](image2)

This measurement is somehow similar with the first two findings, presented above. The river flow, in the dry season, is between 10 m$^3$/s and 12 m$^3$/s which is also the visual estimation.

**Measurement with propeller** (Figure 4).
- Measurement with propeller was done one shot only because the river is too large – about 20 m;
- Depth: 1.24 m;
- Average speed: 0.5 m/s;
- River width: 20 m (approximation, impossible to measure);
- Average area [m$^2$] = 24.8 m$^2$;
- Return of flows: 12.4 m$^3$/s.

**Measurement with bottle**
Downstream of the first place measurements with an empty bottle were made; 
Length of river sector: 15 m; 
Width: 15 m; 
Depth: 5 m; 
Time for \( \frac{3}{4} \) empty bottle: 61 s; 
Area: 60 m\(^2\); Speed: \( \frac{150}{61} = 0.205 \) m/s; 
Flow: 12.3 m\(^3\)/s.

2.4. Mtandika Gauging Station. Previous hydrological study used in the project
The precise location of the gage station is a necessary piece of information for applying the flow duration analysis to potential hydropower sites in the Lukosi Basin other than at the gage. The location of the gage determines the total area of the rainfall catchment, based on elevation delineation.

Catchment area and catchment statistics of the watershed drained by the Lukosi River upstream from, and contributing flow to, the Mtandika Gauging Station were re-determined using the methodology and resources described below. The Mtandika location is originally described in the GRDC dataset as being located at: -7.56667 Decimal degrees (-South) Latitude (07° 34' 00.01" S); 36.433333 Decimal degrees East Longitude (36° 25' 59.93" E); 1,210 masl; 3,363 km\(^2\) catchment area.

In February 2015 a study was performed: “Preliminary Estimate of Water Resource Availability: Flow Duration Analysis of Stream Discharge Records Recorded at Mtandika Gage Station from 1957 to 1987 on the Lukosi River, Tanzania”, by J.L. McAdoo, hydrologist (Figure 6). According to this study, Lukosi River, at Mtandika Gauging Station, is expected to have at least 21 m\(^3\)/s for 90% at its peak in April. In the rest of the time, 9 m\(^3\)/s is the assured average of flow produced by the river at least 90% of time.

![Figure 6. Lukosi River – Multiannual mean monthly flows for the period 1957 – 1987 (30 years)](Image)

**Lukosi River Flow Duration Analysis and Exceedance Probabilities**
Neither the average nor median flow rates represent the minimum amount of water flowing through the system 50% of the time. This is due to the fact that stream flow curves (hydrographs) do not follow a normal distribution. To find the actual duration of flows, it is therefore necessary to calculate the frequency statistics of each particular stream flow. To compute flow duration curves for each month, the raw daily discharge measurements at Mtandika Gauging Station were re-organized on a monthly basis, ranked, and assigned a frequency distribution.

The flow duration curve and the exceedance probability for each particular observation \( o \) are calculated according to the relation (1):
\[ P_0 = 100 \times \left[ M_0 \div (n + 1) \right] , \]  

where:
\( P \) = probability that the flow will be greater or at least equal than a certain value (% of time),
\( M \) = the ranked position of the flow data point “\( o \)” (dimensionless),
\( n \) = the number of flow data points in the period of record.

The frequency of a particular flow rate occurring in the period of record is its expected probability of recurrence in the future, under the assumptions that (1) the existing 30 years of data capture the representative climate variability for the region and (2) that there are no statistically significant long-term trends affecting hydrologic controls, such as upstream water extraction, retention by dams, or any significant changes to watershed vegetation cover, land use, or climate. For viewing convenience, the stream flow values that are equal to or exceed 65% and 90% of the time, respectively, are summarized in Table 1 which is used for representing Figure 7.

**Table 1.** Monthly stream flows that correspond to 65% and 90% exceedance probabilities, Mtrandika station, in m³/s

| Month | 65%   | 90%   |
|-------|-------|-------|
| Jan   | 18.54 | 12.81 |
| Feb   | 19.43 | 15.02 |
| Mar   | 21.22 | 15.15 |
| Apr   | 33.19 | 20.94 |
| May   | 28.54 | 21.07 |
| June  | 19.72 | 15.74 |
| July  | 17.54 | 14.23 |
| Aug   | 15.40 | 12.41 |
| Sep   | 13.78 | 10.64 |
| Oct   | 12.50 | 10.27 |
| Nov   | 12.68 | 8.72  |
| Dec   | 16.34 | 11.34 |

**Figure 7.** Lukosi River – Flow duration Curve for the period 1957 – 1987 (30 years)
3. Conceptual design criteria for standardization

Design criteria

1) fixed hydrostatic head scenarios of 50, 75, 100, 150 m,
2) annually variable water flow rate ranging between 3 to 10 m³/s, and
3) installation of dual hydro turbines of identical model and capacity at each powerhouse.

Based on the design of each of the four powerhouses, the designer shall identify and specify appropriate types and configurations of hydro-turbine generating equipment and related electro-mechanical equipment.

Powerhouse Layout

There shall be provided four standardized powerhouse layout options. Layouts and drawings shall be provided for each one of the four standardized powerhouses. Each powerhouse layout shall include a control room, a room for auxiliary mechanical and electrical equipment, and an erection bay for the two hydro turbines. Each powerhouse design shall also include an appropriately sized, single transformer substation adjacent to each powerhouse to step up or down voltage as required for export to the grid or a mini-grid.

Steelworks and Penstocks

It has been developed a hydraulic grade-line spreadsheet between data along selected reaches of the Lukosi River basin. Utilizing the grade-line, conceptual designs and fabrication considerations shall be provided for buried and above ground steel penstocks in three separate design scenarios to accommodate water flow rates of 3, 7, and 10 m³/s.

Unit-of-length construction cost estimates shall be drawn up for the penstocks for each water flow rate scenario.

Diversion Weir and Intake Structure

Conceptual designs and construction details shall be provided for one or more alternatives appropriate to the Lukosi River basin for a concrete weir to divert river flow into a concrete RoR HEPP (run-of-river hydroelectric power plant) intake structure incorporating flow control gates, trashracks, and sediment traps. These features shall be incorporated into digital drawings for use in the feasibility study. The vertical height of each diversion weir in these conceptual designs shall not exceed 5 m. Unit-of-area construction cost estimates shall be drawn up for the weir-face area together with an appropriate intake structure based on water flow rate scenarios of 3, 7, and 10 cubic meters per second.

Open Channel, Headraces or Canals

A conceptual design shall be developed for the construction of open channel headraces, or canals, which extend from the concrete diversion weir and intake structure to a head-works at the top of the penstocks. The conceptual design will be made with sufficient unit-of-length cost and performance estimations to compare the cost-benefit of open channel canals versus closed tubular conduits. The headrace canal designs shall include three separate design scenarios to accommodate water flow rates of 3, 7, and 10 cubic meters per second.

It was analyzed in a total of 12 variants, as in Table 2: 3 installed flow scenarios and for each of them, 4 head alternatives. It was chosen to have 3 scenarios due to similarities of civil works. For example, for the 3 m³/s scenario, the intake, penstock or canal is the same. The difference lies in the thickness of penstock wall and the powerhouse equipment’s area.
Table 2. Scenarios used in standardization project

| Head [m] | Flow [m$^3$/s] | Scenario 1 3 m$^3$/s | Scenario 2 7 m$^3$/s | Scenario 3 10 m$^3$/s |
|----------|----------------|----------------------|---------------------|-----------------------|
| 50 m     | Var 01         | Var 02               | Var 03              |
| 75 m     | Var 04         | Var 05               | Var 06              |
| 100 m    | Var 07         | Var 08               | Var 09              |
| 150 m    | Var 10         | Var 11               | Var 12              |

As it can be observed in the figure above, the scheme used for all scenarios is RoR (run-of-river) with weir/intake, desalting chambers, channel, fore bay, penstock and powerhouse [1].

4. Technical calculations [3], [4]

4.1. Weir/Intake and sand trap

Intake elements: Right bank recessed wall; Dam (spillway); Eco flow opening; Washing gate; Intake opening; Access channel; Sand trap; Overflow channel; Washing channel; Connection channel; Left bank recessed wall; Upstream concrete – stone slab; Energy dissipation basin; Gangway.

The calculation was done using the MathCAD software. An example is given in Figure 9.

The environmental flow was considered 10% of the mean flow. Since the space does not allow the presentation of the entire project’s calculation, it being too large and complex, only a few examples shall be presented for intake. The results are shown in Table 3.
1. Intake opening dimensions

\[ Q_\text{in} = \frac{3 \text{ m}^3}{8} \]

\[ h_\text{in} = 1.8 \cdot \text{m} \quad b_\text{in} = 5.0 \cdot \text{m} \]

\[ A = b_\text{in} \cdot h_\text{in} = 9 \text{ m}^2 \]

\[ d_\text{bars} = 2.5 \cdot \text{cm} \]

\[ l_\text{bars} = h_\text{in} = 1.8 \text{ m} \]

\[ o_\text{bars} = 5.5 \cdot \text{cm} \]

\[ n_\text{bars} = \frac{b_\text{in} \cdot h_\text{in}}{o_\text{bars}} = 90.909 \]

\[ A_{\text{act}} = A - n_\text{bars} \cdot d_\text{bars} \cdot l_\text{bars} = 4.909 \text{ m}^2 \]

\[ V_{\text{in}} = \frac{Q_\text{in}}{A_{\text{act}}} = 0.611 \frac{\text{m}}{\text{s}} \]

\[ V_{\text{max}} = 0.7 \cdot \frac{\text{m}}{\text{s}} \]

\[ A_{\text{rec}} = \frac{Q_\text{in}}{V_{\text{max}}} = 4.286 \text{ m}^2 \]

**Figure 9.** Example of calculation for intake opening, scenario 1 [3], [4]

**Table 3.** Results of intakes calculations

|                  | SCENARIO 1 | SCENARIO 2 | SCENARIO 3 |
|------------------|------------|------------|------------|
| \( Q_{\text{med}} \) [m³/s] | 22.26      | 22.26      | 22.26      |
| \( Q_{\text{eco}} \) [m³/s] | 2.22       | 2.22       | 2.22       |
| \( Q_{\text{max}} \) [m³/s] | 223.63     | 223.63     | 223.63     |
| Rated (installed) flow [m³/s] | 3.00       | 7.00       | 10.00      |
| **Dam (spillway)** |            |            |            |
| Width [m]        | 14.25      | 14.25      | 14.25      |
| Thickness [m]    | 0.40 – 4.70| 0.40 – 4.70| 0.40 – 4.70|
| Eco flow opening |            |            |            |
| Width [m]        | 1.00       | 1.00       | 1.00       |
| Height [m]       | 0.72       | 0.72       | 0.72       |
| Washing gate     |            |            |            |
| Width [m]        | 1.80       | 1.80       | 1.80       |
| Height [m]       | 2.80       | 3.30       | 3.30       |
| Intake opening   |            |            |            |
| Width [m]        | 5.00       | 8.00       | 11.50      |
| Height [m]       | 1.80       | 2.30       | 2.30       |
| **Water access channel into sand trap:** |          |            |            |
| Arc shape        |            |            |            |
| Length (interior) [m] | 6.90       | 6.15       | 6.15       |
| Parameter                                    | Value 1   | Value 2   | Value 3   |
|----------------------------------------------|-----------|-----------|-----------|
| Length (exterior) [m]                        | 12.55     | 14.40     | 18.8      |
| Middle wall length [m]                       | 5.5       |           |           |
| Depth [m]                                    | 3.20      | 4.20      | 4.2       |
| Depth (water) [m]                            | 2.40      | 2.90      | 2.9       |
| Width [m]                                    | 5.00      | 8.00      | 11.5      |
| **Flushing gate**                            |           |           |           |
| Width [m]                                    | 0.30      | 0.30      | 0.30      |
| Length [m]                                   | 0.60      | 0.60      | 0.60      |
| **Flushing concrete step**                   |           |           |           |
| Height [m]                                   | 0.20      | 0.20      | 0.20      |
| Width [m]                                    | 0.35 – 1.60 | 0.35 – 1.60 | 0.35 – 1.60 | 0.35 – 1.60 |
| **Sand trap basin:**                         |           |           |           |
| Number of rooms                              | 2.00      | 3.00      | 4.00      |
| Max retained particle [m]                    | 0.50      | 0.50      | 0.50      |
| Length [m]                                   | 5.00      | 7.00      | 7.00      |
| Height (total) [m]                           | 3.60      | 4.35      | 4.35      |
| Height (wet) [m]                             | 2.80      | 3.35      | 3.35      |
| Shape (top view) [m]                         | rectangular | Rectangular | rectangular |
| Room cross section [m]                       | trapezoidal | Trapezoidal | trapezoidal |
| Top width (one room) [m]                     | 2.40      | 2.50      | 2.70      |
| Bottom width (one room) [m]                  | 0.80      | 1.10      | 1.30      |
| Slope [%]                                    | 3%        | 3%        | 3%        |
| **Connection channel:**                      |           |           |           |
| Shape (top view)                             | trapezoidal | Trapezoidal | trapezoidal |
| Length [m]                                   | 2.75      | 5.50      | 9.80      |
| Large base [m]                               | 6.00      | 8.30      | 12.00     |
| Small base [m]                               | 4.00      | 4.40      | 5.00      |
| Shape (cross section)                        | rectangular | Rectangular | rectangular |
| Height upstream (total) [m]                  | 2.10      | 3.20      | 3.20      |
| Height upstream (wet) [m]                    | 1.60      | 2.20      | 2.20      |
| Height downstream (total) [m]                |           |           |           |
| Height downstream (wet) [m]                  |           |           |           |
| Inclination [%]                              |           |           |           |
| **Sand trap washing channel:**               |           |           |           |
| Shape (top view)                             | Arc       | Arc       | Arc       |
| Cross section                                | rectangular | Rectangular | rectangular |
| Width                                        | 2.50      | 2.50      | 2.50      |
| Length                                       | 5.60      | 7.85      | 11.45     |
| Height                                       | 1.00      | 0.95      | 0.95      |
| **Energy dissipation basin:**                |           |           |           |
| Width                                        | 18.45     | 26.90     | 28.40     |
| Length                                       | 20.50     | 27.00     | 27.00     |
### Gangway:

| Width | 1.10 | 1.10 | 1.10 |
|-------|------|------|------|
| Length| 20.70| 20.70| 20.70|

3. Washing gate

\[
P_{\text{gate}} = 0.05 \, \text{m} \quad \text{washing gate upstream step height}
\]

\[
h_{\text{gate}} = 1.8 \, \text{m} \quad \text{washing gate width}
\]

\[
c_{\text{gate}} = 4.5 \, \text{m} \quad \text{washing gate sidewalls length}
\]

\[H_{\text{spa}} = h_{\text{a}} + 1 \, \text{m} = 2.8 \, \text{m} \quad \text{water height in gate}\]

\[
g = 9.807 \, \frac{\text{m}}{\text{s}^2}
\]

\[m_i = \frac{0.3 + 0.55}{2} = 0.425 \quad \text{usually flow coefficient flow coefficient (0.3 - 0.55)}
\]

\[\sigma = 1 \quad \text{drowning spillway coefficient}
\]

\[\varepsilon = 1 \quad \text{coefficient of contraction}
\]

\[k = 1 \quad \text{coefficient of skew}
\]

\[
m_i = \left(0.405 + \frac{0.0027}{H_{\text{spa}} \cdot \frac{1}{m}} \cdot \left(1 + 0.55 \left(\frac{H_{\text{spa}}}{H_{\text{spa}} + P_{\text{gate}}}\right)^2\right)\right) = 0.621
\]

\[m_{\text{gate}} = m_i \cdot \sigma \cdot \varepsilon \cdot k = 0.621 \quad \text{flow coefficient calculated}
\]

\[
Q_{\text{gate}} = m_{\text{gate}} \cdot h_{\text{gate}} \cdot \sqrt{2 \cdot g \cdot H_{\text{spa}}^3} = 23.212 \, \frac{\text{m}^3}{\text{s}} \quad \text{maximum flow for given H}
\]

**Figure 10.** Example of calculation for washing gate, scenario 1; [3], [4]

Since the river considered as reference is Lukosi, the intake dimensions are quite constant, but the openings and other items that are proportional with the rated flow are different.

### 4.2. Channel

The channel section that was considered is a rectangular section (to be easy calculated). An example for the calculation model is given in Figure 11 and the results are presented in Table 4.
13. Channel dimensions

\[ R = \frac{0.8}{n} \]
proposed hydraulic radius

\[ n = 0.014 \]
Manning's roughness coefficient

\[ y = 2.5 \sqrt{n} - 0.13 - 0.75 \sqrt{R} \cdot \sqrt{n - 0.10} \]
Chezy coefficient

\[ C = \frac{1}{n} \cdot R^\frac{1}{2} \approx 0.6023 \]

\[ i = \frac{0.1}{1000} \]
slope of the channel

\[ V_d = \frac{0.223}{\theta} \]
velocity in sandtrap

\[ v_1 = C \cdot \sqrt{R \cdot i} \approx 0.617 \]
medium velocity in channel - Chezy

\[ v_2 = \frac{R \cdot i}{n} \approx 0.616 \]
medium velocity in channel - Bernoulli

\[ v_{\text{ref}} = \frac{v_1}{ \frac{m}{a} + v_2 \frac{m}{a} } \approx 0.016 \frac{m}{s} \]

\[ Q = 3 \frac{m^3}{s} \]
installed flow

\[ A_{\text{th, min}} = \frac{Q}{v_{\text{ref}}} = 4.867 \text{ m}^2 \]
minimum channel area

\[ b_a = 3.2 \text{ m} \]

\[ h_{b\theta} = \frac{b_a}{2} = 1.6 \text{ m} \]
width of channel

\[ h_a = 1.6 \text{ m} \]
height of channel

\[ A_a = b_a \cdot h_a = 5.12 \text{ m}^2 \]
channel area

\[ \text{check}_{\text{channel area}} = \text{if } (A_{\text{th, min}} < A_a, \text{"Channel OK"}, \text{"Channel not OK")} \]

\[ R_h = \frac{A_a}{b_a \cdot h_a} = 0.8 \text{ m} \]
hydraulic radius

Figure 11. Channel calculation example [3], [4]

### Table 4. Results of channels calculations

| Channel                  | Option 1 | Option 2 | Option 3 |
|--------------------------|----------|----------|----------|
| Section                  | M/U      | rectangular | Rectangular | rectangular |
| Width (outside)          | m        | 4.00     | 5.20     | 5.80     |
| Width (inside)           | m        | 3.20     | 4.40     | 5.00     |
| Depth (total)            | m        | 2.10     | 2.70     | 3.00     |
| Depth (wet)              | m        | 1.60     | 2.20     | 2.50     |
| Height                   | m        | 2.50     | 3.10     | 3.40     |
| Inclination              | %        | 0.10     | 0.10     | 0.10     |
| Walls thickness          | cm       | 40.00    | 40.00    | 40.00    |
| Base plate thickness     | cm       | 40.00    | 40.00    | 40.00    |

4.3. Forebay

An example for the forebay calculation model is given in Figure 11 and the results are presented in Table 5.
14. Fore Bay calculation

\[ d = 0.5 \times \text{mm} \]  
minimum particle retained

\[ g = 9.80665 \times \frac{m}{s^2} \]  
gravitational acceleration

\[ \varphi_{\text{max}} = 2000 \times \frac{kg}{m^3} \]  
density of particle (sand)

\[ \varphi_{\text{water}} = 998.2 \times \frac{kg}{m^3} \]  
density of water

\[ \mu_{\text{water}} = 0.001002 \times \frac{kg}{m \cdot s} \]  
viscosity of water

\[ V_t = g \cdot \varphi^2 \cdot \frac{\varphi_{\text{max}} - \varphi_{\text{water}}}{18 \cdot \mu_{\text{water}}} = 0.130 \frac{m}{s} \]  
settling speed

\[ h_{\text{pt}} = 3.5 \times \text{m} \]  
medium height of sandtrap

\[ h_{\text{pt,min}} = \frac{h_{\text{pt}}}{1.25} = 2.8 \text{ m} \]  
width of sandtrap

\[ \text{check}_{\text{pt}} = \text{if} \left( h_{\text{pt}} > h_{\text{pt,min}}, \text{“OK”}, \text{“not OK”} \right) \]

\[ A_{\text{pt}} = h_{\text{pt}} \times b_{\text{pt}} = 14.7 \text{ m}^2 \]  
area of pt

\[ V_{\text{pt}} = \frac{Q}{A_{\text{pt}}} = 0.204 \frac{m}{s} \]  
velocity in sandtrap of pt

\[ V_{\text{max,pt}} = 0.5 \frac{m}{s} \]  
max velocity in sandtrap

\[ \text{check}_{\text{pt}} = \text{if} \left( V_{\text{pt}} < V_{\text{max,pt}}, \text{“OK”}, \text{“not OK”} \right) \]

\[ h_{\text{pt}} = 2.1 \text{ m} \]  
head of sand particle

\[ T_{\text{pt}} = \frac{h_{\text{pt}}}{V_{\text{pt}}} = 15.422 \text{ s} \]  
settling time

**Figure 11.** Forebay calculation example [3], [4]

**Table 5.** Results of forebays calculations

| Fore Bay                  | M/U | Option 1                  | Option 2                  | Option 3                  |
|---------------------------|-----|---------------------------|---------------------------|---------------------------|
| Concrete step Shape (top view) |     | arc with 4.7 radius       | arc with 5.4 radius       | arc with 7.50 radius      |
| Height m                  |     | 1.40                      | 1.40                      | 1.40                      |
| Fine rack                 |     |                           |                           |                           |
| Inclination deg           |     | 65                        | 70                        | 70                        |
| With m                    |     | 4.20                      | 5.40                      | 6.20                      |
| Height m                  |     | 2.00                      | 2.50                      | 2.50                      |
| Loading chamber Height m  |     | 5.00                      | 6.40                      | 6.40                      |
| Height (wet) m            |     | 4.50                      | 5.90                      | 5.90                      |
| Cross section             |     | Trapezoidal               | Trapezoidal               | Trapezoidal               |
| Width (top) m             |     | 4.20                      | 5.40                      | 6.20                      |
| Width (base) m            |     | 2.40                      | 3.60                      | 4.40                      |
| Inlet pipe DN (upstream) m|     | 2.00                      | 2.80                      | 2.80                      |
| DN (downstream) m         |     | 1.60                      | 2.50                      | 2.50                      |
| Overpressure pipe DN 300  |     |                           |                           |                           |
4.4. Penstock

An example for the calculation model is given in Figures 12 - 13 and the results are presented in Table 6.

**Figure 12.** Penstock calculation example (1/2)

**Figure 13.** Penstock calculation example (2/2)

**Table 6.** Results of penstocks calculations

| Penstock | Option 1 | Option 2 | Option 3 |
|----------|----------|----------|----------|
| General: |          |          |          |
| DN [mm]  | DN 1600 mm | DN 2500 mm | DN 2500 mm |
| Distance between 2 saddles [m] | 6.00 m | 6.00 m | 6.00 m |
| Wall thickness and material [mm] | 9.5 mm steel S235JR – max pressure PN 19 | 14.3 mm steel S235JR – max pressure PN 19 | 19.1 mm steel S235JR – max pressure PN 25 |
| 50 m | 11.9 mm steel S235JR – max pressure PN 24 | 15.9 mm steel S235JR – max pressure PN 21 | 22.2 mm steel S235JR – max pressure PN 25 |
| 75 m | 12.7 mm steel S235JR – max pressure PN 26 | 19.1 mm steel S235JR – max pressure PN 25 | 23.8 mm steel S235JR – max pressure PN 29 |
| 100 m | S235JR – max pressure PN 26 | 15.9 mm steel S235JR – max pressure PN 21 | 23.8 mm steel S235JR – max pressure PN 31 |
| 150 m | S235JR – max pressure PN 26 | S235JR – max pressure PN 21 | 23.8 mm steel S275JR – max pressure PN 36 |
4.5. Powerhouse

The powerhouse results are presented in Table 7.

| Option 1 | Option 2 | Option 3 |
|----------|----------|----------|
| Transformer room: | Transformer room: | Transformer room: |
| Width: 2.65 m | Width: 2.65 m | Width: 2.65 m |
| Length: 4.00 m | Length: 4.00 m | Length: 4.00 m |
| Height: 4.85 m | Height: 4.85 m | Height: 4.85 m |
| Channel (for oil spills) | Channel (for oil spills) | Channel (for oil spills) |
| Width: 0.50 m | Width: 0.50 m | Width: 0.50 m |
| Depth: 1.10 m | Depth: 1.10 m | Depth: 1.10 m |
| Length: 1.50 m | Length: 1.50 m | Length: 1.50 m |
| MV room: | MV room: | MV room: |
| Width: 2.00 m | Width: 2.00 m | Width: 2.00 m |
| Length: 4.00 m | Length: 4.00 m | Length: 4.00 m |
| Height: 3.75 m | Height: 3.75 m | Height: 3.75 m |
| Water exhaust channels: | Water exhaust channels: | Water exhaust channels: |
| Number: 2 | Number: 2 | Number: 2 |
| Width: 3.40 m | Width: 4.55 m | Width: 4.55 m |
| Height (under base plate): 1.30 m | Height (under base plate): 3.10 m | Height (under base plate): 3.10 m |
| Inclination: 12‰ | Inclination: 12‰ | Inclination: 12‰ |
| Concrete block: | Concrete block: | Concrete block: |
| Width: 3.00 m | Width: 4.00 m | Width: 4.00 m |
| Length: 11.95 m | Length: 3.00 m | Length: 3.00 m |
| Height: 4.00 m | Height: 5.50 m | Height: 5.50 m |
| Material: reinforced concrete | Material: reinforced concrete | Material: reinforced concrete |

5. Conclusions [5]

It is difficult to give a response to the question: “Can the small hydropower plants be standardized?”

As it can be seen after this study, the hydropower plants are hard to be standardized, because the structures are variables of many external non-standardized factors; because the "fuel" used is variable and different from one case to the other and the locations’ conditions are variable. It is not the fact that it can be said “here I will choose to install a hydropower plant”. The hydropower plant needs to be installed in the best suitable position that will give the best returns for the investment made. Of course, here is not the case of hydropower plants with the installed capacity lower than 250 kW, for which there are some standardized options offered by the electromechanical providers (so called “package hydropower plants”). But even in this case the efficiencies and also the lifetime of these structures are not very clearly presented by the manufacturers.
Any investment in plants based on renewable energy sources, especially investments in hydropower plants which are much higher than in other sources, needs to lead to a power output that will assure the right returns. For this reason the best efficiency, both financial and technical, must be reached and this is not possible with full small hydropower development standardization.

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