Preliminary Study of Mini Hydropower Sites in Manna Region, Bengkulu, Indonesia

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October 2015

Abstract

In Indonesia, only 10% of its 75.000 MW hydropower potential is realized. This thesis report shows a preliminary study about a mini hydropower plant which is located in Sumatera Island, Indonesia. This thesis includes the literature review, the study of hydrology, basic power plant sizing, hydraulic studies, and also the financial analysis. The output of this report is a recommendation that the sites have potential for further developments. The study of hydrology includes potential evapotranspiration calculations based on the Turc and Thornwaithe methods, and the thornwaithe-mather water balance model. This models produces the design discharge value that is used for power plant sizing. In addition, the hydraulic model is created and simulated in order to find the hydraulic system reliability in transient conditions. Furthermore, based on the designed power plant size, the financial aspects of the mini hydropower are analyzed.

Keywords: minihydro, hydropower, energy design project, renewable energy, sustainable energy

1. Introduction

For today and the days to come, the challenges and issues in energy production and consumption will become more critical and important to be tackled. One of the most important issues of our generation is about energy sustainability. To ensure that the future generations will have at least the same comfort in using energy as we have now, it is imperative that we maintain an energy system with a green energy resource, an efficient energy conversion, and an optimum energy utilization. However, with current trends in energy production and consumptions, the world is moving toward an unsustainable future [1].

Between several alternative of sustainable energy sources, small hydropower plant has been proven as a reliable source of electricity in many regions. There are some advantages in developing a small hydropower [2]:

- Small hydropower provides a clean and green energy. Since a small hydropower utilizes a relatively small or medium river with not much water concentration, then it will have no impact in living organisms and ecology surrounding the hydropower plant. In addition, while in operation, the hydropower plant does not produce any greenhouse gases.
- Small hydropower technology is matured. Since the hydropower technology has been existed more than 100 years ago, the design, construction, and maintenance of the hydropower plant is already well known. Consequently the risk associated with engineering side of hydropower plant development is relatively tiny.
- Small hydropower can serve another added value. In some cases of small hydropower development, the small dam or water regulator can serve as flood control.
- In some remote areas, very often small hydropower is the best choice in producing electricity, since it produces the most economical choice.

1.1. Motivation

As a country with a high yearly economy growth rate of 5-7% for the last decade [3], coupled with rising demand for electrification in developing rural areas, Indonesia needs to provide a continuous electrical power, in a reliable and sufficient manner. Referring to article 28 and article 29 of No.30 Law year 2009 in Indonesia's Electrical Power Law, the permit to provide this electrical power service is granted to a state-owned company, Perusahaan Listrik Negara (PLN).

However, with vast development of the electricity demand in Indonesia, especially in Java region, quite often the supply of electricity cannot match the electricity demand. Yearly electricity demand growth of 8,5% is outpacing the electricity supply growth which only grows at 6,5% [4]. Consequently, this fact led to a series of rolling blackouts in some regions in order to keep the electrical power grid working at national level. A tireless effort and breakthrough ideas need to be implemented in order to avoid further crisis in the future, since it can hinder the development of economic growth and also plummet industrial competitiveness in Indonesia.

With current installed capacity around 50.000 MW, the projected demand growth of electricity is 35.000 MW in the next five years. In this program, PLN has already put a power plant development plan in place [3], by putting out coal fired power plant as a majority of its future power plan, at 56%. Along with another fossil fuel energy source, the plan shows a huge dependency on fossil fuel, with staggering 93% of energy source coming from fossil fuel based energy.

While this solution might be able to serve electricity demand in a short term, the author believes that in the long run the plan will be detrimental to the future of Indonesia's energy security. The first reason of that, is because of the polluting nature of fossil fuel energy. The rising energy production from fossil fuel will further push the emission level from Indonesia's power plant, and in the long run this will expose Indonesia's government to international pressure of reducing pollution level. The second reason is due to the fact that despite Indonesia is today the largest coal exporter in the world, at current (2015) rate, the coal will run out in 75 years, with no new field exploration [3].

The fact that Indonesia still rely on fossil fuel for the new power plant plan is very unfortunate, because actually there are alternative energy in form of renewable energy. The renewable energy resources in Indonesia is listed in Table 1.

Table 1: Renewable Energy in Indonesia [3]

No	Energy Resources	Energy Potential	Installed Capacity
1	Geothermal	16.502 MW	1.341 MW
2	Hydropower	75.000 MW	7.059 MW
3	Mini/Microhydro	769,7 MW	269 MW
4	Biomass	13.662 MWe	1.364 MWe
5	Solar Energy	4,8 kWh/m2/day	42,78 MW
6	Wind Energy	3-6 m/s	1,33 MW

*for wind energy, only wind speed data are available

From the Table 1, we notice that as of 2014, the installed hydropower capacity is only about 10% of its potential (7.059 MW out of 75.000 MW potential). On the other hand, there are still some area for improvement to realize mini-microhydro power energy potential. Currently, the potential for the mini/micro hydropower of around 770 MW exist, with 30% of it already developed [5].

1.2. Objectives

The main objective of the thesis is to work on field study, in the form of preliminary study of one of the potential mini hydropower site in Bengkulu (Manna River, Lahat) region in Indonesia. The preliminary study will consist of:

- Preliminary hydrology study was done based on region data and river discharge measurement. The mentioned region data includes topography map (1:50K) from indonesia geospatial portal (INA-SDI), and watershed data from Directorate of Planning and Evaluation of Watershed (Indonesia's Ministry of Forestry). This data was then analyzed in order to obtain the river discharge value.
- Preliminary topography map study. Study conducted in order to determine the location of the civil works and mechanical-electrical apparatus. Accurate coordinate location that was recorded by Global Positioning System (GPS) was used as basis to decide on preliminary layout of mini hydropower plant.
- System dynamics study. The scope of the study includes a transient condition analysis, specifically in water hammer calculations. These calculations will determine if the designed hydraulic system can withstand the effect of pressure spike in transient condition.
- Preliminary generated power calculation. Based on the preliminary hydrology study, the preliminary layout of mini hydropower plant, a size of electrical power generated is calculated. The calculation results will also determine the choice of the mechanical-electrical apparatus and structure sizing.
- Preliminary financial analysis. The scope of this analysis is including financial aspect of the preliminary study, such as investment cost, working capital, tariff structure, cost structure, and revenue projection.

2. Sites Location

There are two potential location of the sites, that is located on the Manna River, Ulu Manna Region, Bengkulu, Indonesia. Those sites are located around:

- Longitude: 103°1'55.22"E
- Latitude: 4°8'22.41"S

The location of the proposed weirs, forebays, and the powerhouses of the first and second sites of hydropower plants are shown in Figure 1, with their corresponding locations. The upper (upstream) site will be called the "Site 1", while the lower one will be called the "Site 2" in the following sections. The considerations of the preliminary locations are only based on the topographical map that is provided from Indonesian Geospatial Portal, with the resolution of 1:50.000. The site characteristics are listed

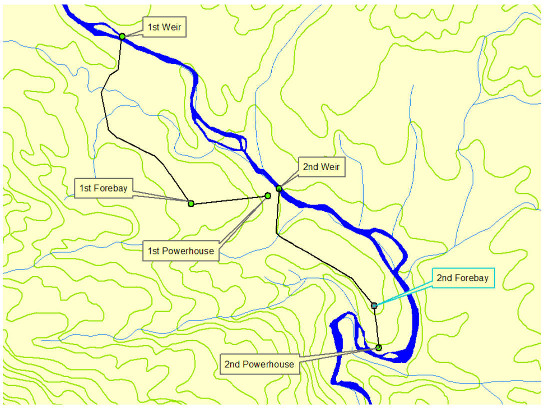

Figure 1: Site 1 and 2 Preliminary Location

in Table 2.

Table 2: Sites characteristics

Characteristics	Site 1	Site 2
Dam/Weir Elevation	599,8 m	534,2 m
Forebay Elevation	591,87 m	520,1 m
Powerhouse Elevation	542,68 m	501,9 m
Gross Head	49,175 m	18,2 m
Assumed Losses	5%	5%
Net Head	46,72 m	17,29 m
Penstock Length	625,181 m	173,096 m

3. Hydrology Study

3.1. Evapotranspiration

For some sites in Indonesia, where the data is not completely available or where the climate data in the site cannot be estimated accurately with available data, some empirical methods could be applied. The first method is by using temperature based evapotranspiration models, which are developed to estimate the evapotranspiration using data with low time resolution (for example, monthly temperature data) . One of the most used model is the Thornwaithe evapotranspiration model [6], which is described in equations (1) to (3).

$$PET = 16f\left(10\frac{T}{I}\right)^a \tag{1}$$

$$I = \sum_{1}^{12} i \tag{2}$$

$$i = \left(\frac{T}{5}\right)^{1{,}514} \tag{3}$$

PET is the monthly potential evapotranspiration (mm), T is the mean montly temperature (^oC), I is the yearly thermal index, i is the monthly thermal index, a is the empirical factor ($a = 675\text{x}10^{-9}I^3 - 77,1\text{x}10^{-6}I^2 + 17,92\text{x}10^{-3}I + 492,39\text{x}10^{-3}$), and f is the correction factor which is described as:

$$f = \frac{D_m \bar{N_m}}{360} \tag{4}$$

where D_m is the number of day in the month, and N_m is the mean daily sunshine duration (hours).

The second method, is based on radiation models. For tropical countries, Turc method has been known to perform well [7]. The required data for this method is the average daily temperature, daily relative humidity data, and solar radiation measurement. For the climate with relative humidity more than 50%, the potential evapotranspiration can be calculated in equation (5) [7].

$$PET = 0,313\frac{T}{T+15}(S_n + 2,1) \tag{5}$$

$$Sn = S_o(1-\alpha)\left(a_s + b_s\frac{n}{N}\right) \tag{6}$$

S_n is the amount of solar radiation (mm/day), S_o is the extraterrestial radiation (mm/day), $\alpha = 0,23$ is the albedo value [7], $a_s = 0,25$ and $b_s = 0,5$ is the Angstrom coefficient [8], and $\frac{n}{N}$ is the percentage of bright sunshine hours compared to the total day length.

The Thornwaithe method can provide quick and simple calculation, while Turc method has been proved as the method that worked well in humid climate [7]. For preliminary study purposes, it is assumed that the adjusted temperature data from the closest climatology station can represent the temperature data for the measured watershed [9]. The closest climate station that can provide reliable, long records of monthly values of climate data is the Pulau Baai (Bengkulu) climate station which is located about 100 km from the sites.

However, since the temperature data at the climate station are measured at the different elevation compared to the watershed areas, an adjustment has to be made. The approximate relationship between two locations with different elevation is described in equation 7 [9].

$$T = T_c - 0.006h \tag{7}$$

where T is the measured temperature at the climate station at the height h, and T_c is the temperature equivalent at the sea level.

On the other hand, to ensure that the data is reliable, it is important to know whether the data on the climate station's location can be correlated with the site's location. In order to validate the data, one way to do it is by using the correlation factor [9]. If the correlation factor is more than 0,7, then the site is reliable as a reference. Favourably, for the range of 100 km^2, the correlations factor for daily mean temperatures is 0,9 [9]. Thus, the data from the Baai climate station will be adjusted for calculations. Furthermore, for the daily sunshine duration data, the correlation value for climate space of 100 km is 0,75 [10]. A higher correlation factor is expected for a bigger time step [9].

The time step used for the calculation is the monthly average data, due to the data availability. Therefore, the data correlation is also expected to be higher for both cases.

Using the temperature data from Baai station [11] and also using equation (7), the adjusted monthly mean temperature data for watershed 1, 2, and 3 is shown in Figure 2. It is worth to note that the equation (7) only depends on the elevation difference. Thus, due to the similarity of the elevation between watershed 1 and 2, it is shown that for those watersheds, the values of the adjusted temperature are very similiar. The temperature data are then validated with the global area data from the last 30 years [12].

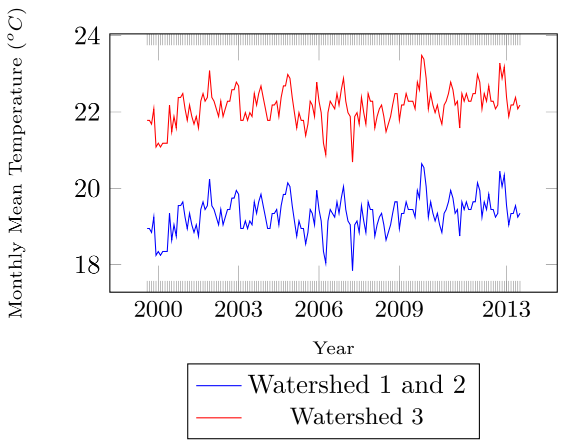

Figure 2: Monthly mean temperature in watersheds

For the radiation data, it is assumed that the monthly radiation data that are gathered from Baai station can represent the monthly radiation data in watershed 1, 2, and 3, with the correlation factor higher than 0,75.

Based on the monthly mean temperature and the monthly mean sunshine duration data, the monthly potential evapotranspiration are calculated using Thornwaithe and Turc models. The result of the calculations are shown in Figure 4.

In order to validate the data, the global yearly statistical data from NTSG, University of Montana are used [13]. The result of the validation is that both methods perform well in the area of interest by providing calculation results close to data from NTSG, which is 1200mm/year (data in Figure 4 is a monthly data).

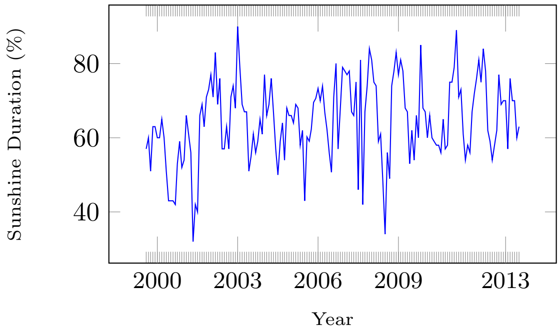

Figure 3: Monthly mean sunshine duration in watershed 1,2, and 3 (2000-2013)

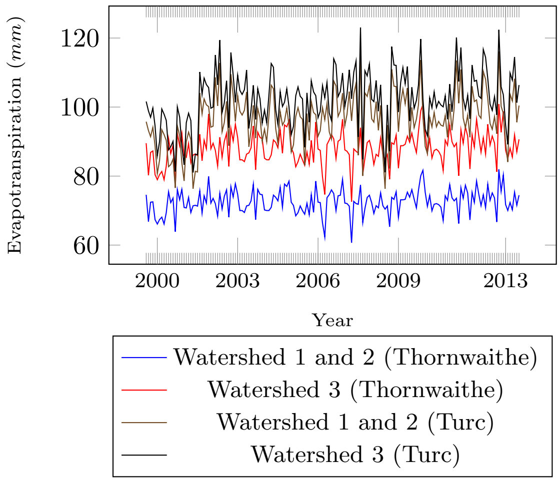

Figure 4: Monthly evapotranspiration calculation on watershed 1,2,3

3.1.1 Precipitation

Monthly precipitation data are calculated from the daily precipitation data that are obtained from the various rain gauges that are close to the sites, or are considered that they can represent the sites due to its similarity regarding vegetation types or elevation characteristics. The available rain gauge is marked on the map on Figure 5, with gauge from South Sumatera province marked by blue dot, and gauge from Bengkulu Province marked by teal dot. The data availability on the recent years is shown at Table 3.

Table 3: Data availability on the rain gauge near watershed

Rain Gauge	Data Available in Year
Pagar Alam	1985-1999, 2004-2008
Tanjung Sakti	1992-1996, 1998-2001
Lubuk Tapi	2000-2001, 2003-2006, 2008-2013
Batu Kuning	,2000-2001, 2003-2010, 2012-2013
Bungin Tambun	2000-2004, 2006-2013

Since three of the five references have data series mostly between year 2000-2013 (14 years), this range of time are used as the base time for the wa-

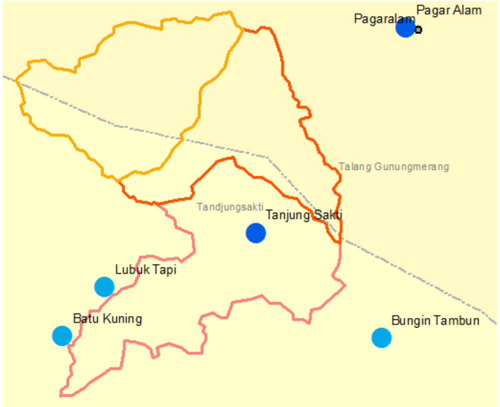

Figure 5: Climate Station Near the Watersheds

ter discharge calculation. The missing data between year 2000-2013 are obtained by averaging the precipitation values over time. For the Lubuk Tapi (elevation 152m), Batu Kuning (elevation 183m), and Bungin Tambun(elevation 205m) rain gauge, the minimum data period that are needed to obtain an accurate long time precipitation average are 6 years [9]. For the rain gauge with lack of data during these periods, Pagar Alam (elevation 725m) and Tanjung Sakti(730m), the missing data are forecasted. Therefore, for Pagar alam and Tanjung Sakti rain gauge, the average long-term precipitation data from the previous decade are used to forecast data for the year of 2000-2013.

Since the average long-term data are used, it is assumed that this data represents a stable climate, and any fluctuations in the precipitation data are represented in the average value of data.

To estimate the precipitation data in watershed 1 and 2, the data from Pagar alam, Bungin Tambun, and Tanjung sakti are used due to the similarity of land cover and elevation between these stations and both watersheds. The arithmetic average of these rain gauge stations are calculated to determine the estimate precipitation value. The calculation result is presented in Figure 6.

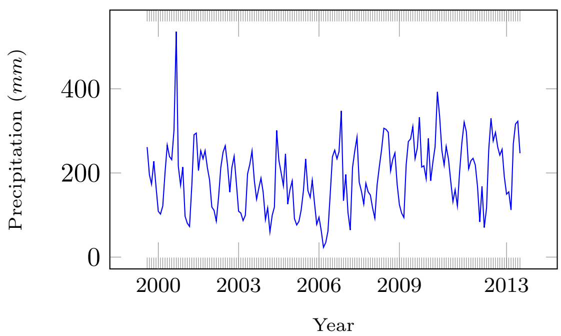

Figure 6: Estimated and logged monthly precipitation data in the watersheds 1&2

For the watershed 3 precipitation estimation, the data from Bungin Tambun, Batu Kuning, Lubuk Tapi, and Tanjung Sakti are used, by calculating the arithmetic average data from these stations. The averaged data are shown in Figure 7.

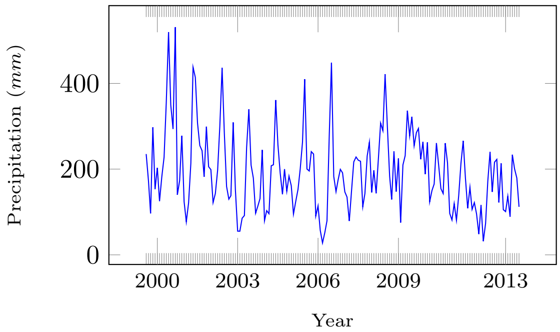

Figure 7: Monthly estimated and logged precipitation data in the watershed 3

The monthly precipitation data are then validated with the annual precipitation data that is gathered from the National Statistic Bureau [11], which state that the range of the annual precipitation is at the range of 1000-4000 mm/year.

3.1.2 Water Discharge Calculation

By means of Thornwaithe Mather water balance model, the water discharge value are calculated for the period 2000-2013.

Steps to calculate the water discharge using the Thornwaithe-Mather method is as follows:

- Determine the precipitation data and temperature data on the same time series and time step.
- Calculate the potential evapotranspiration based on the same time series.
- Calculate the amount of water available in the soil at each time step. At the time t, amount of water available in soil is:

$$H_t = \begin{cases} H^{max} & \text{if } P_t - ET_t \geq H^{max} - H_{t-1} \\ H_{t-1} + P_t - ET_t & \text{if } P_t - ET_t \leq H^{max} - H_{t-1} \end{cases} \tag{8}$$

  the value of H_max, water holding capacity, is determined by the product of soil depth and soil porosity.
- Calculate the water available in soil for evapotranspiration

$$H_t^{disp} = \begin{cases} min\Big((ETP_t - P_t)\frac{H_{t-1}}{H^{max}}; min(H_{t-1}, H^{max})\Big) & \text{if } P_t \leq PET_t \\ \text{No calculation is necessary} & \text{if } P_t \geq PET_t \end{cases} \tag{9}$$

- Calculate the real evapotranspiration during timestep t,

$$ET_t = \begin{cases} PET_t & \text{if } P_t - ET_t \leq P_t + H_t^{disp} \\ P_t + H_t^{disp} & \text{if } P_t - ET_t \geq P_t + H_t^{disp} \end{cases} \tag{10}$$

- calculate the water discharge Q_t, with α is the delay factor

$$\begin{aligned} Q_t &= \alpha X_t + (1-\alpha)X_{t-1}; \alpha = 0,5 \\ X_t &= P_t - ET_t - (H_t - H_{t-1}) \end{aligned} \tag{11}$$

The water discharge values for these periods are obtained as shown in Figure 8, for both Thornwaithe and Turc model.

Since both of sites have the same monthly precipitation data and potential evapotranspiration data (in mm), the trends of the water discharge for both sites are following the same pattens over the years. Moreover, the similiarity of mean elevation between those sites made the adjusted mean temperature value for both sites are also alike. For the site 1, the water discharge data is obtained from the watershed 1 water discharge calculation. On the other hand, for the site 2, the discharge data is obtained from the addition of watershed 1 and 2 calculations.

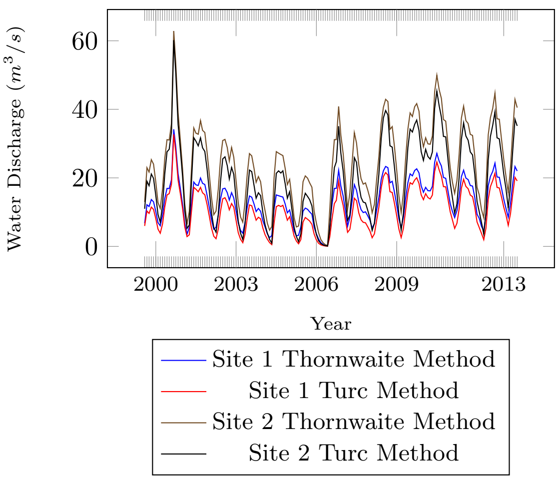

Figure 8: Monthly water discharge at preliminary site 1&2

In order to verify the water discharge calculations, these data is compared to one gaging station (Manna-Bandar Agung station) that is located in the southest end of watershed 3 (shown in pink line at Figure 5). Thus, the water discharge calculations from watershed 1, 2, and 3 are added and then compared to the gaging station, shown in Figure 9. However, since the measurement of the measuring station is only available for years 1984, 1992, 1996-1999, 2008-2009, the monthly mean value of these measurement period is used as the base of the comparison.

It is shown that the water discharge estimation at the point at the end of watershed 3 did not entirely match the data from the Manna-Bandar Agung station. The mean flow of the measurement station is 49,15 m^3/s, while the mean flow from Thornwaithe and Turc methods calculation results are 39,61 m^3/s and 32,91 m^3/s. The discrepancy between the calculation results and the measurement is quite often found in the study of mini hydropower in Indonesia, as there are lack of accurate and/or complete data. Nevertheless, in the case of this study, the calculated data will be used as the basis of power plant sizing, since the data for the calculation are already verified with the other source of data.

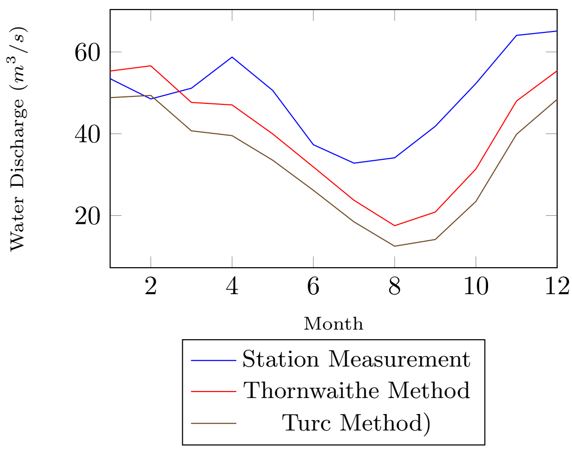

Figure 9: Comparison of the discharge calculation and Manna-Bandar Agung data

3.2. Turbine Design Discharge

Design discharge is calculated in order to find the optimum power plant design, that can provide the most beneficial outcome, both economically and design wise. In order to determine design discharge, a flow duration curve that uses water discharge data from previous section are arranged to understand the characteristics of the river flow. This curve shows the percent of time that a specified discharge exceeded in a given period. Since there are two calculation results for the design discharge, the one that resembles closer to the the Manna-Bandar Agung gaging station will be used. Therefore, the discharge value that uses Thornwaithe Potential Evapotransporation values are arranged as a flow duration curve.

3.2.1 Flow Duration Curve

The first step in creating the flow duration curve is to determine the data time step. Commonly, the daily time step is used when the data is available [14], that will give a steep curve. However, the available data at Manna site only provide mean monthly flow. Therefore, the monthly discharge data from years 2000-2013 are used, with total 168 data steps (see Figure 8). By increasing the time step, the resulting curve is expected to be flatter compared to the curve from lower time step, since the extreme low or high value are averaged out in these time periods. The resulting flow duration curve is shown in Figure 10.

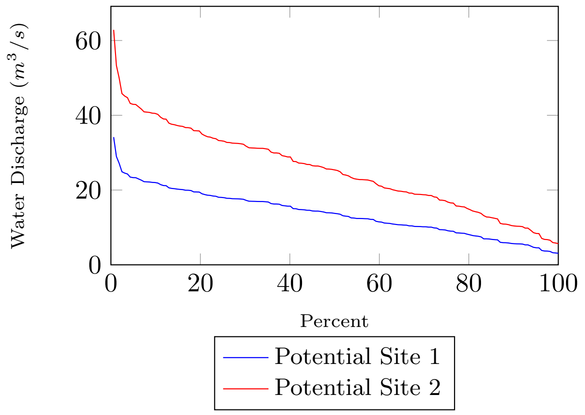

Figure 10: Flow Duration Curve in Potential Sites

3.2.2 Design Discharge

In order to determine the design discharge, an optimization process is conducted by comparing the economic parameters of few discharge values based on assumptions above. The optimization process done by taking into account the flow duration curve and the minimum technical turbine flow [15]. Since there are many possible alternatives of design discharge size, an optimum design flows need to be found through an optimization process. Based on the common practice, the design discharge flow is chosen at the maximum flow that happens at around 40% of time. This flow of water then divided into two flows, because of few reasons. First reason is that the water discharge varies heavily between dry season and wet season. In order to operate efficiently in dry season, one generator can be turned off. The other reason is in maintenance case, one generator can still be operating while the other is serviced. The summary of calculation results is listed in Table 4, based on the assumption as follows:

- The annual Operation and Maintenance costs are assumed to be 2,5% from the investment costs.
- There are additional contingency costs for about 10% of the total capital costs.
- The feasibility study and the planning cost is already included in the capital cost. The feasibility study cost is assumed to be the same for all of the site, while the planning costs is assumed to be 2% of the capital cost.
- The Investment Loan from bank is scheduled to be paid in 8 years, with the annual interest of 15%.
- The time that is required for the feasibility study and the development of the mini hydropower plant is three years (including time to arrange study and permit).
- Payment, expenses, and the income of the plant is calculated at the end of the year.
- Lifetime of the project is the same as the lifetime of the power purchase agreement contract, which is 20 years.
- The interest rate is assumed to be constant through the lifetime of the project, at 7,5%
- The long term USD to IDR exchange rate is set at 11.963,33 IDR per 1 USD, considering the exchange rate projections at the lifetime of the project [16].
- The income tax is calculated according to the Indonesian Government Law No. 46 Year 2013 about income tax.
- The tax discount is applied to the capital costs for the first six year of the project, with the amount of tax discount is equal to 5% of the total capital costs.
- The power purchase agreement energy price is as stated in Indonesia's Ministry Law No.19/2015, with the assumption that local investor made the majority investment.

For the first site, the design discharge of 13 m^3/s looks to be the most attractive option, since it provides the highest Net Present Value. It is worth to notice that the lower design discharge will provide higher Benefit/Cost Ratio and Internal Rate of Return, but also lower rated power. Therefore, the 13 m^3/s design discharge is still selected as the design discharge, since it can provide higher energy when only one of two generators working. For the second site, the design discharge of 20 m^3/s is selected due to similar reasons.

4. Hydraulic Circuit

The optimum design of the penstock is crucial. In addition to the penstock length, the penstock diameter have to be designed with regard to the system efficiency and economic benefits. The economic penstock diameter is calculated using following equations [17]:

$$D_{economic} = C_{EC}\, C_{MP}\, Q^{0.43}\, H^{-0.24} \tag{12}$$

where the $C_{EC} = 1,4$ is the coefficient of energy cost where the energy cost is moderate, $C_{MP} = 1,2$ is the steel pipe materials coefficient, H is the net

Table 4: Economic Parameters of Different Design Discharge

Parameters	Site 1 Alternative Flow			Site 2 Alternative Flow		
	9 m^3/s	13 m^3/s	16 m^3/s	15 m^3/s	20 m^3/s	25 m^3/s
Rated Power (kW)	2x1903	2x2754	2x2289	2x1178	2x1573	2x1968
Net Present Value (10^9 IDR)	187,014	218,356	210,343	131,913	150,813	149,656
Benefit/Cost Ratio	4,61	3,91	3,28	5,11	4,52	3,79
Internal Rate of Return	39,25%	33,65%	28,26%	41,66%	37,36%	32,58%
Payback Period	5 years	6 years	6 years	5 years	5 years	6 years

head, and Q is the design discharge. In order to improve the power plant reliability in the dry season, the flow divided into two channels, through their own penstock, and then discharged into each of the outlet channel's turbine. Using equation (12) along with some optimization to keep the water velocity limit between 3-4 m/s [17], returns the economic penstock diameter value of 1,65 m for site 1, and 1,9 m for site 2.

In addition, the parameter of penstock pipe thickness is also calculated. The empirical equation for penstock thickness is[17]:

$$t = 0.0084\,D\ + 0.001 \tag{13}$$

which will give the penstock pipe thickness of 0.0195 m for site 1, and 0.0224 m for site 2.

For the losses calculation in this study, the friction and singular losses from penstock are considered, assuming that the gross head is relatively constant.

In order to measure the range of losses that happen in the hydraulic circuit, the losses in the condition of minimum discharge, rated discharge, and maximum discharge are calculated. On the Table 5, series of calculation results are shown in order to obtain the Reynolds number (Re), friction factor (f), and hydraulic gradient values (J). The value of Reynolds number is calculated based on water temperature of $32^o\,C$ from preliminary visit data. The friction factor f is given by:

$$f = \frac{1}{(-2log[\frac{k}{3{,}7D} - \frac{5{,}16}{Re}log[\frac{k}{3{,}7D} + \frac{5{,}09}{Re^{0{,}87}}]])^2} \tag{14}$$

For steel pipe, $k = 0,00010$ [18].

Furthermore, the hydraulic gradient value is calculated by [17]:

$$J = f\frac{V^2}{2gD} \tag{15}$$

The calculated parameters and results are tabulated in Table 5.

On the top of the friction losses, the singularity losses are also calculated, based on the bends that recorded on the elevation model. However, instead of using friction factor f, the singularity factor ϵ_L is used.

$$\Delta H_{sing} = \epsilon_L \frac{V^2}{2gD}[17] \tag{16}$$

Since all of the bends angle have value less than 30^0, it is assumed that the singularity factor is 0,20 [19]. The result of these calculations are summarized in Tables 6 and 7.

Assuming that the most of the mini hydro power plant operation is happening in the optimum flow, the losses value from optimum flow is chosen as the losses for the penstock 1 and the penstock 2. Since the percentage value of losses is close to the assumptions, the losses percentage assumptions is kept at 5%. Thus, the net head for the potential site 1 is 46.72m, and the net head for potential site 2 is 17.29 m, respectively.

4.1. Turbine Design

Francis turbine is applied to the site 1, while Kaplan/propeller turbine is applied to the site 2 [20]. After the turbine type is selected, there are various parameters that need to be designed in order to ensure that turbine would work efficiently. Those parameters are specific speed, rotational speed, range of discharges through the turbine, and suction head.

For each of turbine types, the specific speed calculated using empirical equations. For the Kaplan turbine, the formula is as follow:

$$N_s = \frac{2700}{\sqrt{H}} \tag{17}$$

For the Francis turbine, the specific speed can be obtained as:

$$N_s = \frac{1}{2}(\frac{2330 + 1550}{\sqrt{H}}) \tag{18}$$

The number of pole in synchronous generator can be calculated as follows:

$$n_{pp} = \frac{3000rpm}{N} \tag{19}$$

where n_{pp} is the number of pair of poles, f is the system frequency (50 Hz in Indonesia), and N is the rotational speed (rpm).

In irregular conditions, the turbine runner can accelerate because of hydraulic conditions until it reach a maximum speed, which is called runaway speed (N_{rw}). These value depends on the specific speed (N_s) and the head values. The value of runaway speed estimated as follows:

$$n_{rw} = 0,63N_s^{0,2}N_o\frac{H^*}{H_o} \tag{20}$$

Table 5: Losses Parameters in Penstocks

Parameters	Penstock 1			Penstock 2		
	Optimum Flow	Maximum Flow	Minimum Flow	Optimum Flow	Maximum Flow	Minimum Flow
Q	6,5	7,15	2,6	10	11	4
v_{mean}	3,034	3,344	1,215	3,527	3,879	1,410
Re	6285454	3858976	2514182	6687885	7356673	2675154
f	0,0109	0,0109	0,108	0,0106	0,0106	0,0105
J	0,0031	0,0038	0,0005	0,0035	0,0043	0,0006

Table 6: Losses in Site 1 Penstocks

Head Loss Type	Head Losses (m, Optimum)	Losses %-age	Head Losses (m, Max)	Losses %-age	Head Losses (m, Min)	Losses %-age
Friction	1,934	3,934	2,342	4,762	0,306	0,622
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Singular	0,057	0,116	0,069	0,140	0,009	0,019
Total	2,277	4,630	2,756	5,605	0,361	0,733

Table 7: Losses in Site 2 Penstocks

Head Loss Type	Head Losses (m, Optimum)	Losses %-age	Head Losses (m, Max)	Losses %-age	Head Losses (m, Min)	Losses %-age
Friction	0,613	3,370	0,742	4,079	0,097	0,533
Singular	0,067	0,367	0,081	0,444	0,011	0,059
Singular	0,067	0,367	0,081	0,444	0,011	0,059
Singular	0,067	0,367	0,081	0,444	0,011	0,059
Total	0,814	4,470	0,985	5,411	0,129	0,709

where H^* is the turbine head, H_o is the net head, and N_o is the rotational speed.

For the Francis turbine, the runner diameter can be calculated with equation (21), while for the axial/propeller turbine, the runner diameter is calculated with equation (22) [21]:

$$D = 84,5(0,31 + 2,5\,10^{-}3N_s)\frac{\sqrt{H_o}}{60\,N} \tag{21}$$

$$D = 84,5(0,79 + 1,602\,10^{-}3N_s)\frac{\sqrt{H_o}}{60\,N} \tag{22}$$

The summary of those calculations is tabulated at Table 8.

Table 8: Turbine 1 & 2 Parameters

Parameters	Turbine 1	Turbine 2
D (Runner Diameter)	918 mm	1095 mm
N_o (Runner Speed)	600 rpm	600 rpm
N_s (Specific Speed) (100% Output)	257,78	674,95
N_{RW} (Max Runaway Speed)	1177,23 rpm	1427,198 rpm
P_R (Rated Power)	2754 kW	1573 kW

5. System Dynamics

The numerical equation for the penstock in hydraulic system is obtained and written as shown in equations (23) and (24).

$$C^+ : H_P = H_A - B(Q_P - Q_A) - RQ_P|Q_A| \tag{23}$$

$$C^- : H_P = H_B + B(Q_P - Q_A) + RQ_P|Q_B| \tag{24}$$

where P shows the value at current time step, A shows the value at $x - dx$ and B shows the value at $x + dx$. The pipeline characteristic impedance is also shown in B value, which can be calculated in equation (25)

$$B = \frac{c}{gA} \tag{25}$$

R value in equations (23) and (24) shows the resistance coefficient, and defined as

$$R = \frac{J\Delta x}{q_n^2} \tag{26}$$

In order to find the Q and H values, the equations (23) and (24) can be rearranged as shown in equations (27) and (28).

$$H(x,n) = \frac{1}{B}(C_A - BQ_R) + \frac{1}{B}C_B \tag{27}$$

$$Q(x,n) = \frac{1}{B}(H(x,n) - C_B) + Q(x+dx, n-1) \tag{28}$$

The values of C_A, C_B, and Q_R in equations (27) and (28) are known and written in equations (29), (30), and (31)

$$C_A = H(x-dx, n-1) - RQ(x-dx, n-1)|Q(x-dx, n-1)| \tag{29}$$

$$C_B = H(x+dx,n-1)+RQ(x+dx,n-1)|Q(x+dx,n-1)| \quad (30)$$

$$Q_R = Q(x+dx,n-1) - Q(x+dx,n-1) \quad (31)$$

5.1. Boundary Conditions Definition

In the addition of characteristic equations, the boundary conditions is needed. One of the boundary is in the reservoir, which will be assumed as a constant head, as shown in equation (32) and Figure 11.

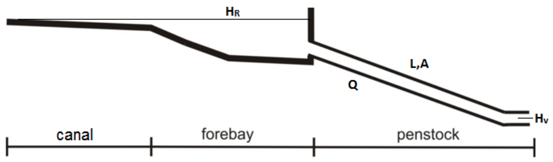

Figure 11: Intake assumptions for water hammer calculations

$$H(x_o,n) = Z_R \quad (32)$$

where Z_R is the hydraulic head in the reservoir.

The elevation of the turbine in the hydraulic system is also used as boundary conditions, since it affects the flow discharge that are going away from the turbine. Furthermore, the water flow that goes through the turbine is also defined in equation (33).

$$Q(L,n) = -0,5BC_V + 0,5\sqrt{(BC_V)^2 + 4C_VC_P} \quad (33)$$

where C_V is defined as

$$C_V = \frac{(kQ_n)^2}{H_n} \quad (34)$$

The pressure that occurs in (x,n), is calculated in equation (35)

$$H(x,n) = \left(\frac{Q(x,t)}{kQ_n}\right)^2 H_n \quad (35)$$

5.2. Water Hammer Calculations

In order to understand the condition that happen in the transient phase, different conditions in few scenarios are set for the valve closure times. The valve closure time scenarios is listed in Table 9, calculated with the penstock values from each site.

Table 9: Valve time closure scenarios

Scenario	1	2	3	4
Closure Time	L/c	3L/c	20L/c	50L/c
Site 1 Value (in sec)	0,60	1,79	11,92	29,81
Site 2 Value (in sec)	0,17	0,50	3,30	8,26

Those conditions includes the fast manoeuvre, with closure time less than 2L/c, and the slow closure with the closure time more than 2L/c.

5.2.1 Calculation Assumptions

The assumptions that are used in the calculations are based on the sites data obtained from previous sections. Those data are listed in Table 10, along with the constant that is used in the calculations.

Table 10: Data used in calculations

Parameters	Site 1	Site 2
Q (m^3/s)	6,5	10
L (m)	625	173
D (m)	1,65	1,9
c (m/s)	1048	1048
t	0,0195	0,0224
R	0.00045	0,00007
Z_R (m)	591,87	520,10
Δx (m)	5	5
t_s (s)	25	25

5.2.2 Calculation Method

The numerical problem is solved using Matlab programs, which is written based on (Brunes, 2009) model [22], with some modifications to adjust with the assumptions on the both sites. In each of the sites, since there are couple of turbines that have identical parameters, the water hammer simulation is assumed to be the same for both turbines. Thus, the simulation is only executed for each turbine in both sites.

5.3. Water Hammer Analysis

5.3.1 Site 1

For site 1, the comparison of pressure variation in the turbine between four scenarios are shown in Figure 12. It is shown that at the fast manoeuvre of the valve, rapid pressure changes occur. In the case of closure time of L/c and 3L/c, a very huge pressure spike up to 375 m and 247 m occurs, which is much higher than the design head of the turbine at 46,72 m.

On the other hand, the slow manoeuvre scenario (with valve closure time of 20L/c and 50L/c) provides the hydraulic circuit with a reduced pressure spike during transient condition.

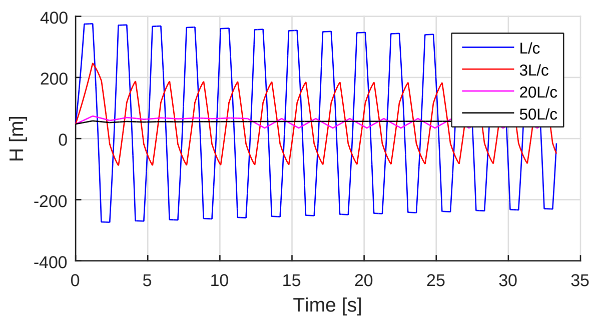

Figure 12: Comparison between different valve closure time at site 1

The maximum and minimum pressure are influenced by the valve closure time. The value of these pressures are listed in Table 11.

Table 11: Maximum and minimum head for different closure times at site 1

Parameter	L/c	3L/c	20L/c	50L/c
H_{max} (m)	375,87	247,46	73,81	57,95
H_{min} (m)	-273,94	-88,20	34,37	43,48

The typical value for allowable maximum transient head variations (Δ/H_o) is 0,5 for the head that is lower than 50 [18]. Abiding these criterion, the pressure can be maintained below the point that may break the pipe. In addition, by reducing the pressure spikes, the cavitation phenomena that caused by over pressure can also be avoided. Looking at the Table 11, based on the net head of 46,72 m, it is shown that the closure times of 20L/c and 50L/c are able to fulfil these criterion. Therefore, the design closure time has to be more than 12 seconds.

For closure time of 20L/c and 50L/c, the comparison between pressure envelops on the penstock and penstock profile are shown in Figures 13 and 14. For both cases it is shown that the pressure lines did not cut the penstock profile.

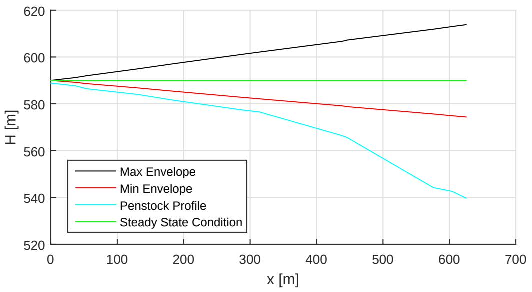

Figure 13: Head envelope and pipeline profile in site 1 (20L/c)

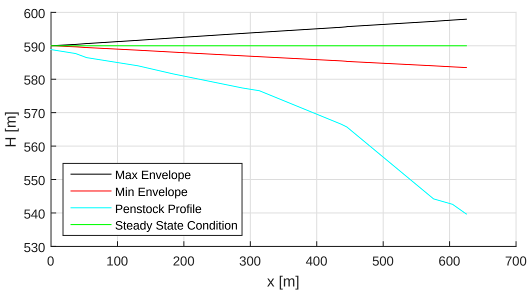

Figure 14: Head envelope and pipeline profile in site 1 (50L/c)

5.3.2 Site 2

For the site 2, the pressure spikes are also found in the simulation with the closure time of L/c and 3L/c, with the maximum values of 396,5 m and 241,29 m. Compared to the site 1, the maximum value of the pressure spikes in site 2 is quite similiar in value. Those fact can be explained with the relation between maximum pressure variation and flow velocity as shown in equation (36) [23].

$$\Delta p_j = \pm \rho c \Delta V \tag{36}$$

Since the velocity range of the water flow in both sites are within the same range ($3 - 4\,m/s$), then the maximum pressure variation are roughly similar. Furthermore, the same case is also found in site 2, where the maximum pressure values are also way above the turbine head design values of 17,29 m. On the other hand, it is also observable that by using longer valve closure time, the pressure spikes is reduced.

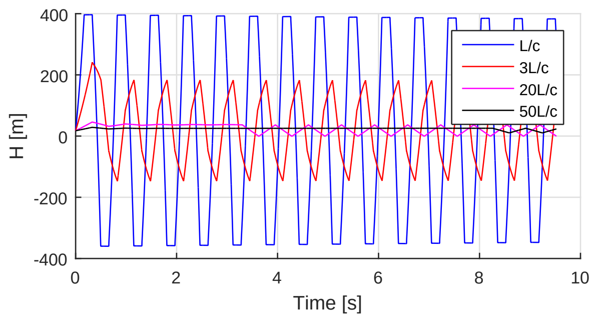

Figure 15: Comparison Between Different Valve Closure Time at Site 2

The value of maximum pressures during different valve closure scenarios are listed in Table 12.

Table 12: Maximum and minimum head for different closure times at site 2

Parameter	L/c	3L/c	20L/c	50L/c
H_{max} (m)	396,50	241,29	45,98	28,61
H_{min} (m)	-360,00	-147,31	-0,72	10,43

Since the net head of site 2 is also lower than 50 m, the same criterion is also used in order to determine the maximum allowable transient head variations. Based on Table 12, the valve closure time of 50L/c (8,26 sec) was able to fulfil the defined criteria, since the maximum variation of pressure is still below $0.5H_o$.

For closure time of 50L/c, the comparison between pressure envelops on the penstock and penstock profile for site 2 is described in Figure 16. It is shown in the figure that the pressure lines did not cut the penstock profile.

6. Conclusions

Hydropower plant study in Manna Region shows that the sites are showing promising economic re-

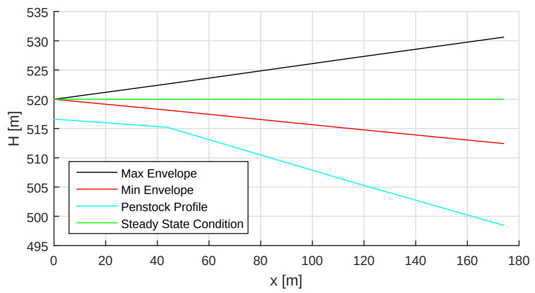

Figure 16: Head Envelope and Pipeline Profile in Site 2 (50L/c)

sults, due to the recent Feed-in-Tariff that enacted by Indonesian government. However, there are still several steps needed in order to fully ensure that the project is feasible both economically and technically.

The preliminary hydrology study is based on the data that is available in that region. The 1:50.000 topology map is used as a base for selecting the preliminary location for the weir, forebay, and the powerhouse. Based on these maps and also the map from Indonesia's Ministry of Forestry, the watersheds that relates to the sites are traced. Based on the Thornwaithe-Mather water balance method, the mean discharge value is obtained.

The water discharge data then arranged to create flow duration curve, which is used to determine the design discharge. The hydropower plant capacity is based on the design discharge value. Based on economical analysis, the optimum design discharge value is obtained at 13 m^3/s for site 1 and 20 m^3/s for site 2. Therefore, the rated capacity for site 1 is 2x2754 kW, while for site 2 it is 2x1573kW.

A Hydraulic study was done in order to test the hydraulic circuit in the case of transient condition. Using the Method of Characteristics (MOC), a water hammer model is created and then simulated for few scenarios, based on different valve closure times. Those study shows that on the both sites, the risk of water hammer can be mitigated by using the slow manoeuvre valve closure. In site 1, the closure time has to be more than 12 seconds, while on site 2 the closure time has to be more than 8,5 seconds.

The financial analysis was made in order to determine the economic parameters of the hydropower project. In this analysis, the revenue is heavily dependent on the annual energy generation. The site 1 is projected to be able to produce 40,34 GWh of energy in a year, while site 2 produces 24,73 GWh anually. With the Total Installed Cost estimation of IDR 15.000.000/kW, both of the projects are able to return the investments in less than 6 years.

Acknowledgements

This paper is based on the report from my internship in PT.LAPI ITB, as a part of my MSc Renewable Energy program from KIC Innoenergy. I would like to thank Dr. Ir. Abdul Aziz Jayaputra, MSCE (PT.LAPI ITB) for having me in the company, Dr. Mukmin Widyanto for his guidance on this topic, and also Prof. Helena Ramos (Instituto Superior Técnico) who adds valuable input to the report.

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