BALKUSAN DAM AND HEPP: INVESTIGATION OF BETTER ALTERNATIVES TO EIEI FORMULATION

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ABSTRACT

BALKUSAN DAM AND HEPP: INVESTIGATION OF BETTER ALTERNATIVES TO EIEI-FORMULATION

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Due to the economic and social growth of Turkey, there is a rapid increase in energy demand. Turkey does not have sufficient natural gas and petroleum reserves; however, it has large hydropower potential. Hydropower is the most widely used form of renewable energy. To generate electricity between elevations 1500 m and 450 m on Balkusan Creek, General Directorate of Electrical Power Resources Survey and Development Administration (EIEI) conducted a feasibility study for a hydroelectric power plant (HEPP) composed of a rock-fill dam in 1999. In 2009, this formulation was not only revised but also constructed by Hidromark Company. The Hidromark-formulation consists of a roller compacted concrete dam and two diversion weirs. The aim of this study is to find a more beneficial alternative to the previous formulations. In order to avoid dam body and expropriation costs the Alternative-formulation composed of a run-of-river HEPP is developed. In the economic analysis, HEPPs are assumed to have two equal sized turbines and net benefits of different formulations are compared. Additionally, energy generation calculations are carried out for two turbines with different installed capacities only for the Alternative-formulation in order to investigate impact of turbine size on energy generation.

Keywords: Balkusan HEPP, Economic Analysis, Turbine Size

ÖZ

BALKUSAN BARAJI VE HİDROELEKTRİK SANTRALİ: EİEİ ÖNERİSİNE KIYASLA DAHA İYİ ALTERNATİFLERİN ARAŞTIRILMASI

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Türkiye'de ekonomik ve sosyal alanlardaki gelişime bağlı olarak enerjiye olan talep hızla artmaktadır. Türkiye, yeterli miktarda doğalgaz ve petrol rezervlerine sahip olmamakla birlikte büyük bir hidroelektrik potansiyeline sahiptir. Hidroelektrik enerji, en çok kullanılan yenilenebilir enerji kaynağıdır. Balkusan deresi üzerinde 1500 ve 450 kotları arasında enerji üretimi maksadıyla Elektrik İşleri Etüt İdaresi (EİE) 1999 yılında, kaya dolgu barajdan oluşan bir hidroelektrik santrali (HES) için fizibilte çalışması gerçekleştirmiştir. Bu formülasyon 2009 senesinde Hidromark Firması tarafından hem revize hem de inşa edilmiştir. Hidromark Firmasının önerdiği HES formülasyonu silindirle sıkıştırılmış beton (SSB) baraj ve iki çevirme yapısından oluşan iki ayrı sistemden oluşmaktadır. Bu çalışmanın amacı, önceki formülasyonlara kıyasla daha uygulanabilir bir alternatifin araştırılmasıdır. Çalışma için geliştirilen alternatifte baraj gövdesi ve kamulaştırma masraflarından kaçınmak için nehir tipi bir HES tercih edilmiştir. Ekonomik analizde, karşılaştırılan hidroelektrik santrallerde iki eşit kapasiteli türbin kombinasyonu kullanıldığı varsayılmış ve santrallerin net gelirleri karşılaştırılmıştır. Buna ek olarak, tez için geliştirilen formülasyonda farklı kapasitelerdeki iki türbin kombinasyonlarının enerji üretimine etkisi araştırılmıştır.

Anahtar Kelimeler: Balkusan Hidroelektrik Santrali, Ekonomik Analiz, Türbin Kapasitesi

To My Family...

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H_g : Gross Head
H_n : Net Head
H_l : Head Loss
η_h : Hydraulic Efficiency
η_t : Turbine Efficiency
η_g : Generator Efficiency
η : Total Efficiency
P : Power
ρ_w : Specific Weight of Water
P_{ins} : Installed Capacity
E : Energy
Qmin : Minimum Turbine Discharge
H_{for} : Water Level in the Forebay
H_{TW} : Tailwater Elevation
$\Delta H_{penloss}$: Energy Losses in Penstock
hf : Friction Loss
f : Dimensionless Friction Coefficient
L : Length of the Penstock
D : Penstock Diameter
V : Average Velocity in the Penstock
g : Acceleration due to gravity
ε : Roughness Height
Re : Reynolds Number
ϑ : Viscosity of Water
h_m : Minor Head Loss
K : Dimensionless Loss Coefficient
n : Number of Years
i : Interest Rate
ΔP : Loss Power
H_{sta} : Static Pressure
H_{ap} : Analysed Point in the Penstock
H_{dyn} : Dynamic Pressure
T_c : Turbine Closure Time
F : Circumferential Tensile Force
wt : Wall Thickness
σ_s : Lateral Unit Stress of Steel
A_{crs} : Cross-Sectional Area of the Penstock
A_o : Outer Cross-Sectional Area
A_i : Inner Cross-Sectional Area
$\forall_p$: Volume of Analysed Penstock Part
W_p : Weight of Analysed Penstock Part

LIST OF ABBREVIATIONS

IEA : International Energy Agency
TEIAS : Turkish Electricity Transmission Company
UCS : Union of Concerned Scientists
EERE : U.S. Department of Energy Efficiency and Renewable Energy
IRENA : International Renewable Energy Agency
EUAS : Electricity Generation Company
WEC : World Energy Council
HEPP : Hydroelectric Power Plant
EIEI : General Directorate of Electrical Power Resources Survey and Development Administration
DSI : State Hydraulic Works
KAREN : Kahramanmaras Energy Production Distribution Business Company Corporation
MOS : Metrological Observation Station
RCC : Roller Compacted Concrete
GRP : Glass Fibre Reinforced Plastic
UNESCO : United Nations Educational, Scientific and Cultural Organization
USBR : U.S. Department of the Interior Bureau Reclamation
DMI : Turkish State Meteorological Services
AFAD : Disaster and Emergency Management Presidency
CRF : Capital Recovery Factor
USACE : U.S. Army Corps of Engineers
USCS : United States Geological Survey
SSR : Sequential Streamflow Routing
FDC : Flow-Duration Curve
ESHA : European Small Hydropower Association
ASCE : American Society of Civil Engineers
JICA : Japan International Cooperation Agency
EMRA : Electricity Market Regulation Authority

CHAPTER 1

INTRODUCTION

1.1 Introductory Remarks

Socio-economic and industrial growth have resulted in higher living standards. This caused an increase in demand for energy. World's total energy consumption has risen 79% since 1973 (IEA, 2011). To meet the energy need, new power plants are being planned or are under construction at the developing countries all around the world.

High quality coal and imported natural gas has been the biggest sources of energy for electricity generation in Turkey for the past decade (TEIAS, 2011). Due to limited coal and natural gas reserves, Turkey became more and more dependent on foreign countries and this holds back the economic growth. Besides, fossil fuel use leads to various environmental problems such as global warming, air quality deterioration, acid rain, etc. (UCS, 2002). Governments have been forced by international agreements to search for alternative sources such as renewable energy to satisfy the sustainable development standards.

Hydropower is the most common form of renewable energy in the world (IEA, 2010). Hydropower does not require any fuel, produces relatively less pollution and waste and it is a high reliable energy source with relatively low operating cost (EERE, 2003). It has the longest lifetime among all other types of power plants (IRENA, 2012). Worldwide, hydropower forms 16.3% of global electricity production (IEA, 2010). Hydropower plants generate 22.8% of Turkey's total electricity (EUAS, 2011). Only 40% of Turkey's hydropower potential has been developed. In other words, 85 billion kW/year of electricity generation is still to be developed (WEC, 2012). Turkey has 156 hydropower projects in operation, 250 projects under construction and 1073 projects in planning stage (Ozkaldi, 2011). One such project that has been in operation since the first quarter of 2012 is Balkusan HEPP. Balkusan HEPP was constructed in Karaman, a province in the south central region of Turkey. This study focuses on investigation of a better alternative to the existing and previously developed formulations of Balkusan HEPP. Since Balkusan HEPP has already been constructed, this study does not have any practical value for Balkusan HEPP, but it may provide some valuable guidelines for future hydropower projects.

1.2 Objective of the Thesis

Three different formulations for Balkusan HEPP are compared in this study. The first formulation is suggested in the reconnaissance study conducted by EIEI in 1999. This formulation consists of a rock-fill dam, a diversion weir and a long transmission line with an installed capacity of 38 MW to harvest water energy between the elevations of 1500 m and 450 m on Balkusan Creek. This formulation will be referred to as the EIEI-formulation in this study. Karen Enerji, the company responsible for the construct of Balkusan HEPP worked on a different formulation together with Hidromark Engineering and Consultancy Company. This formulation will be referred to as the Hidromark-formulation in the rest of this study. The Hidromark-formulation has the same installed capacity and utilizes water energy between the elevations of 1485 m and 400 m on Balkusan Creek. It consists of two separate systems, a roller compacted concrete dam and a diversion weir for the first system and a run-of-river plant for the second system.

In this study, only one run-of-river plant is suggested to avoid high investment and expropriation costs of the dam and to save fertile lands which have already been destroyed by the dam. This formulation will be referred to as the Alternative-formulation in this study. Conventionally, HEPPs constructed in Turkey contain two turbines with equal capacities. Thus, in this study, an economic analysis is conducted for the Alternative-formulation with two equal sized turbines and net benefits of three different formulations are compared. In addition, energy generation calculations are carried out for two-turbine combination with different installed capacities for only the Alternative-formulation in order to investigate impact of turbine size on energy generation.

First, daily streamflow data (2000-2009) of stream gauging station 1736 operated by EIEI and located on Balkusan Creek are obtained. In addition, daily streamflow data from stream gauging station 1723 (i.e Çavuşköy – Ermenek station) which is located in the vicinity of the project site are taken from EIEI as well. The observation period for 1723 is from 1986 to 2004. A regression analysis is performed by using the common period (2000-2004) to evaluate the correlation between these two stations. The degree of correlation between these gauging stations is found to be sufficient (R=0.90) so the daily streamflow values of 1736 gauging station are extended by using the regression equation. Then, extended data are used to carry out operation studies for all three formulations and these three alternatives are compared in terms of incomes and costs.

Three different energy income calculation methods - EIEI, State Hydraulic Works (DSI) and single price method - are used for comparing three alternative formulations. DSI and EIEI methods assign different values for firm energy and secondary energy generations while the single price method assigns a fixed unit price per MWh of electricity generated. This study aims to present a comparison of three alternative formulations only in terms of economical aspects. Environmental, ecological, and social impacts of different formulations are not investigated. It has to be kept in mind that the final decision among different formulations must consider economical aspects together with environmental, ecological, and social impacts.

1.3 Description of the Thesis

Thesis study is composed of five chapters. Location of the project, hydrological, meteorological and seismic conditions at the project site and characteristics of the EIEI-formulation, the Hidromark-formulation and the Alternative-formulation are presented in Chapter 2. Economic analysis of the Alternative-formulation is provided in Chapter 3. Chapter 4 contains comparison of the EIEI-formulation, the Hidromark-formulation and the Alternative-formulation using benefit-cost ratio method and discussions of the results. Finally, conclusions derived from this study are presented in Chapter 5.

CHAPTER 2

PROJECT DESCRIPTION

2.1 Description of The Project Site

EIEI and Hidromark developed two different HEPP formulations on Balkusan Creek, a branch of Ermenek River in Karaman, to generate electricity from water energy having nearly 1080 m of gross head. Project site on a map of Turkey is given in Figure 2.1 . A detailed map is given in Figure 2.2.

Figure 2.1 Location of the Project Site on a Map of Turkey

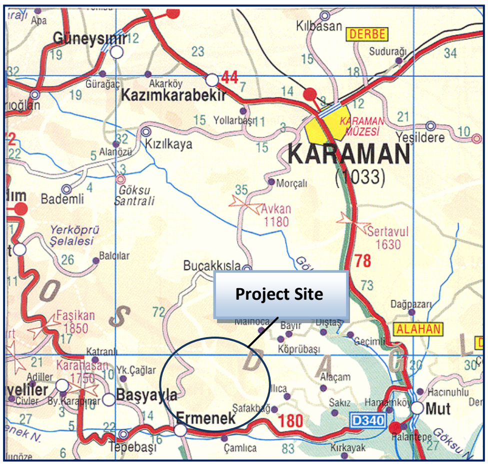


Figure 2.2 Project Site

2.2 The EIEI-formulation

The EIEI-formulation, a rock-fill dam on Balkusan Creek, is proposed by EIEI in 1999. The water of Kumdan Creek, a tributary of Balkusan, is diverted by a diversion weir and transmitted to the reservoir by a diversion tunnel. The characteristics of the EIEI-formulation are given in Table 2.1. The plan and profile showing the EIEI-formulation are given in Figure 2.3 and Figure 2.4, respectively.

Table 2.1 Characteristics of the EIEI-formulation (EIEI, 1999).

Type	**Rock-Fill Dam**
Thalweg Elevation (m)	1450
Dam Height (m)	55
Maximum Water Elevation (m)	1500
Tail Water Elevation (m)	450
Gross Head (m)	1050
Drainage Basin Area (km^2)	259
Design Discharge (m^3/s)	4.4
Transmission Line Length (km)	11.75
Penstock Length (km)	3.5
Penstock Diameter (m)	3
Installed Capacity (MW)	38.4
Construction Duration (year)	5
Firm Energy Generation (GWh)	77.3
Secondary Energy Generation (GWh)	40.1

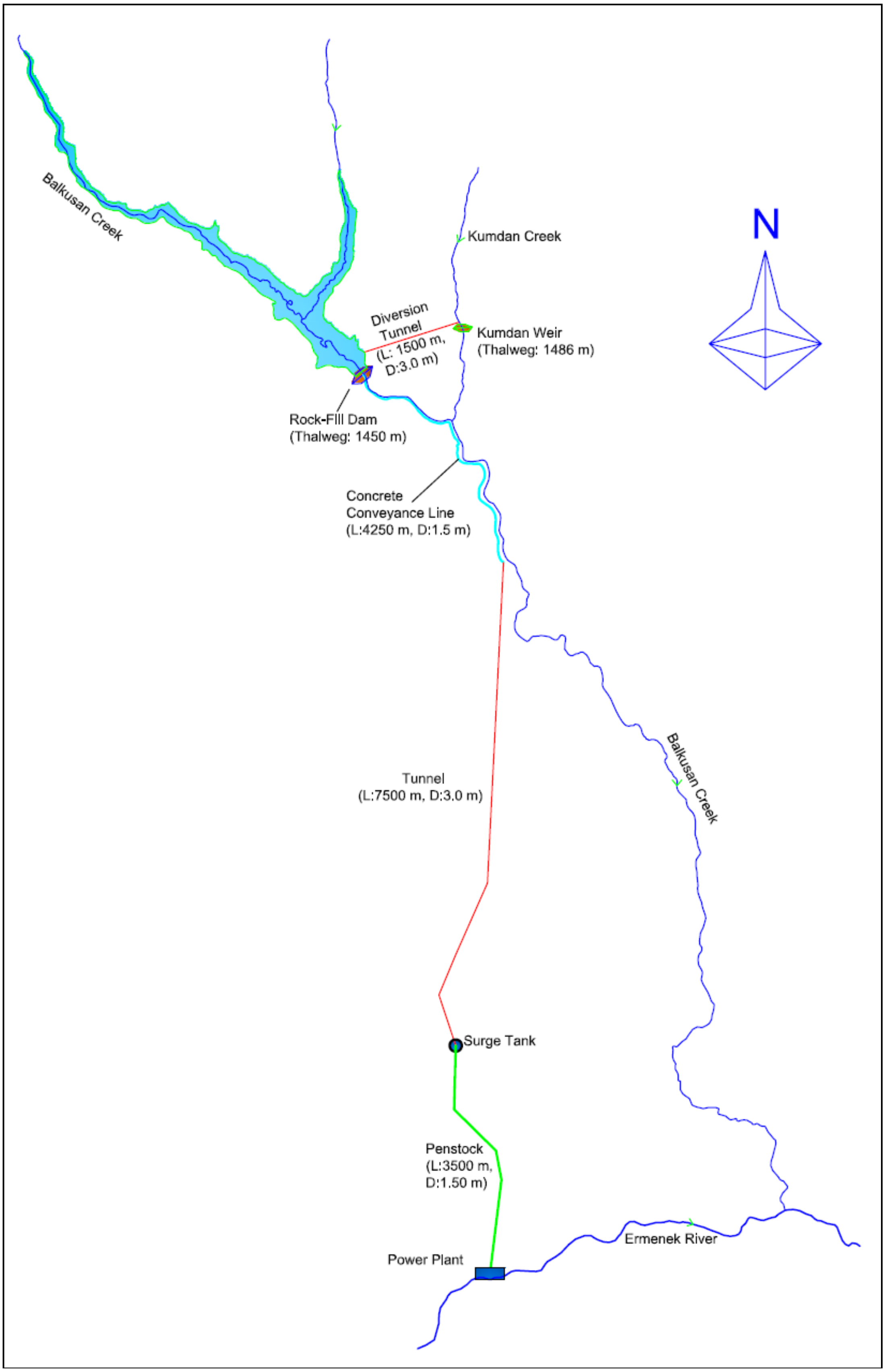
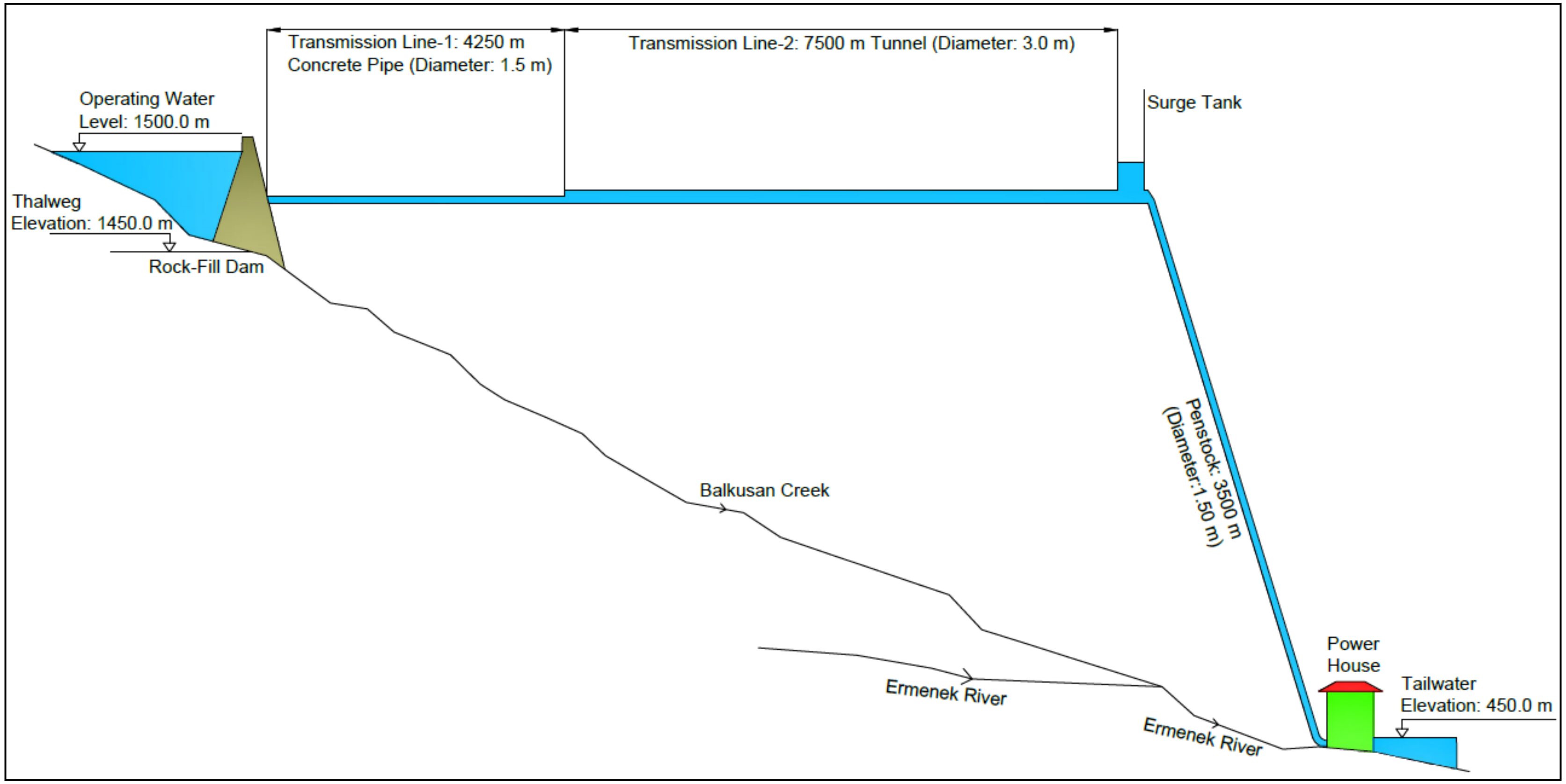


Figure 2.3 Plan of the EIEI-formulation (Not to Scale)

Figure 2.4 Profile of the EIEI-formulation (Not to Scale)



2.3 The Hidromark-formulation

This formulation is proposed by Hidromark Project Company. Hidromark prepared an economic report and a technical feasibility report for Kahramanmaras Energy Production Distribution Business Company Corporation (KAREN) who bought the energy generation licence of Balkusan region.

The Hidromark-formulation is composed of two hydropower plants, namely Balkusan HEPP 1 and Balkusan HEPP 2. Balkusan HEPP 1 is composed of a roller compacted concrete (RCC) dam and a weir transmitting water of Kumdan Creek to the dam reservoir by a box-shaped channel. Balkusan HEPP 2 consists of a diversion weir located just downstream of the power house of HEPP 1 to use the tailwater of HEPP 1 and water collected below the dam axis of HEPP 1 from Balkusan basin. The characteristics of the Balkusan HEPP 1, Balkusan HEPP 2 and the overall Hidromark-formulation are given in Tables 2.2, 2.3, and 2.4, respectively. The plan and profile showing the Hidromark-formulation are given in Figure 2.5 and Figure 2.6, respectively.

Table 2.2 Characteristics of Balkusan HEPP 1 (Hidromark, 2009).

Type	**RCC**
Thalweg Elevation (m)	1450
Maximum Water Elevation (m)	1485
Tail Water Elevation (m)	1090
Gross Head (m)	410
Drainage Basin Area (km^2)	259
Design Discharge (m^3/s)	4.09
Transmission Line Length (km)	8.31
Penstock Length (km)	1.14
Penstock Diameter (m)	1
Installed Capacity (MW)	13
Construction Duration (year)	2
Firm Energy Generation (GWh)	27.35
Secondary Energy Generation (GWh)	13.61
Total Energy Generation (GWh)	40.96

Table 2.3 Characteristics of Balkusan HEPP 2 (Hidromark, 2009)

Type	**Run-of-River**
Thalweg Elevation (m)	1087
Maximum Water Elevation (m)	1091.15
Tail Water Elevation (m)	400
Gross Head (m)	691.15
Drainage Basin Area (km^2)	290
Design Discharge (m^3/s)	4.59
Transmission Line Length (km)	6.28
Penstock Length (km)	1.84
Penstock Diameter (m)	1.1
Installed Capacity (MW)	13
Construction Duration (year)	2
Firm Energy Generation (GWh)	49.32
Secondary Energy Generation (GWh)	30.22
Total Energy Generation (GWh)	79.54

Table 2.4 Characteristics of the Hidromark-formulation Overall Capacity (Hidromark, 2009)

Overall Installed Capacity (MW)	38
Overall Firm Energy (GWh)	76.67
Overall Secondary Energy (GWh)	43.83
Total Energy Generated by The Whole System (GWh)	120.5

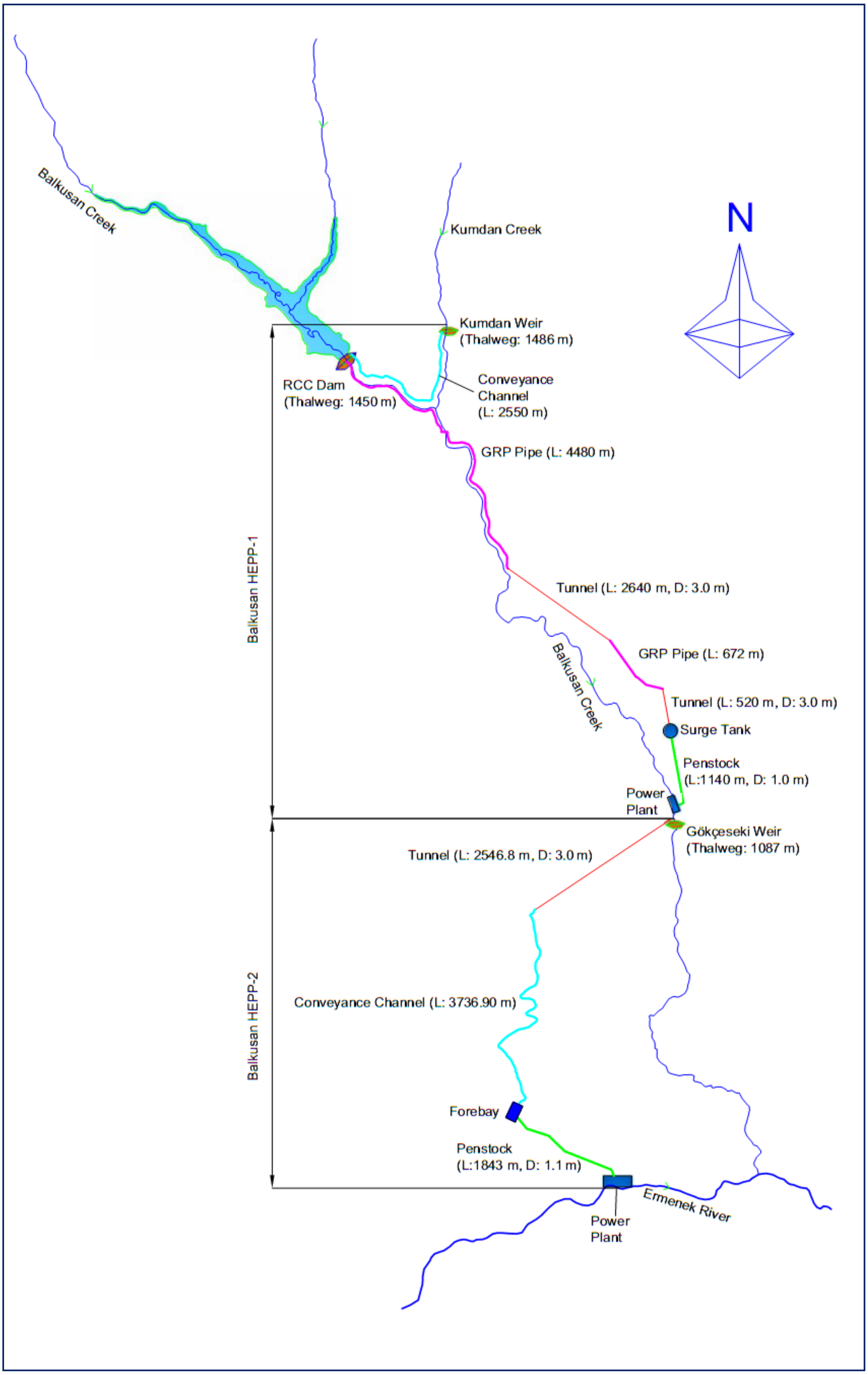
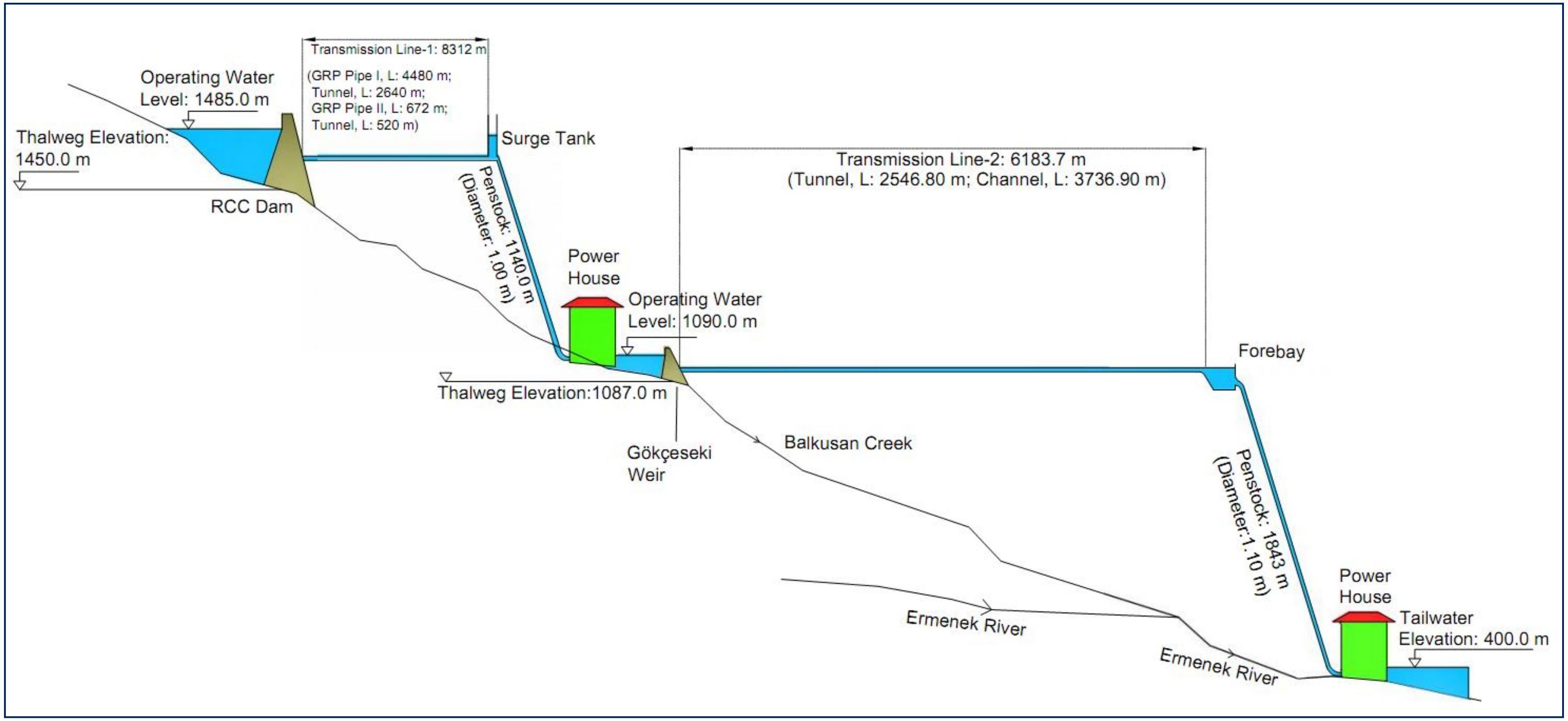


Figure 2.5 Plan of the EIEI-formulation (Not to Scale)

Figure 2.6 Profile of the Hidromark-formulation (Not to Scale)



2.4 The Alternative-formulation

As the Alternative-formulation, a single run-of-river hydropower plant is suggested. The fundamental goal of this alternative is to save the fertile land which is destroyed by the Hidromark-formulation and to avoid high initial investment cost of the dam body. The diversion weir is located nearly 650 m downstream of the confluence of Balkusan and Kumdan Creeks in order to avoid costs of Kumdan Creek Diversion weir, the cost of the tunnel transmitting water to the reservoir and avoid excessive sedimentation problem. In addition, there is no residential and plantation area around the location of the diversion weir of the Alternative-formulation. Therefore, there is little or nearly no expropriation costs related with it's diversion weir. To transmit the diverted water to the forebay, a three-part transmission line (GRP pipe, tunnel and conveyance channel from upstream to downstream) is suggested.

In a conversation happened on 25th March 2011, people from Hidromark Company project office confirmed that left bank of the Balkusan Creek is identified as a landslide zone and most of this zone is composed of talus formation (See Figure 2.7). As a result, it makes impossible to lower the tailwater elevation of the Alternative-formulation on Ermenek river and increase the energy generation. In the selection of the location of power plant and the route of the penstock, geotechnical and topographical investigations conducted by Hidromark company are taken into account and the same location and route are implemented for the Alternative-formulation.

Figure 2.7 An Example of Talus Slide at Balkusan Project Site (Hidromark, 2009)

The most important factors effecting the selection of the route of transmission line are safety and economy (Cofcof, 2008). The glass-fibre reinforced plastic pipe (GRP), conveyance channel and tunnel are proposed for the transmission line. The ascending order of costs from the cheapest to the most expensive among these parts are the conveyance channel, the GRP pipe and the tunnel. In a visit to Balkusan HEPP 2 construction site on 19th of May 2011, people working for Hidromark Company Balkusan HEPP construction site stated that Balkusan valley has sharp hillsides at the downstream of conveyance of Balkusan and Kumdan creeks (see Figure 2.8). This makes impossible to construct conveyance channel on the valley. The GRP pipe is preffered for the first part of the transmission line. Due to topograpy of the region, tunnel is preferred in the second part of the transmission line (see Figure 2.8). At the outlet of the tunnel, topographic conditions allow construction of a conveyance channel as the third part to connect the tunnel to the entrance of the forebay.

Figure 2.8 Balkusan Valley (2011)

The forebay is the hydraulic structure between the transmission line and the penstock converting free flow into the pressurized flow. The hydraulic slopes of the GRP pipe, the tunnel and the conveyance channel are prescribed as 0.001 (Cofcof, 2008). Total length of the transmission line is 8.23 km. The hydraulic loss is about 9 m from the operating water level at the weir if minor losses at the entrances and transition structures throughout the transmission line are taken into account. The most suitable location for the forebay is selected considering the intersection point of the water level at the end of the transmission line and the route of the penstock. The prescribed length and static pressure of the penstock are 4900 m and 1030 m, respectively.

The characteristics of the Alternative-formulation are given in Table 2.5. The plan and profile showing the Alternative-formulation are given in Figures 2.9 and 2.10, respectively.

Table 2.5 Characteristics of the Alternative-formulation

Type	Run-of-River
Thalweg Elevation (m)	1431
Operating Water Elevation (m)	1436
Tail Water Elevation (m)	400
Gross Head (m)	1036
Drainage Basin Area (km^2)	266
Design Discharge (m^3/s)	2.2
Transmission Line Length (km)	8.23
Penstock Length (km)	4.9
Installed Capacity (MW)	18.38
Construction Duration (year)	2
Firm Energy Generation (GWh)	10.46
Secondary Energy Generation (GWh)	43.84
Total Energy Generation (GWh)	54.30

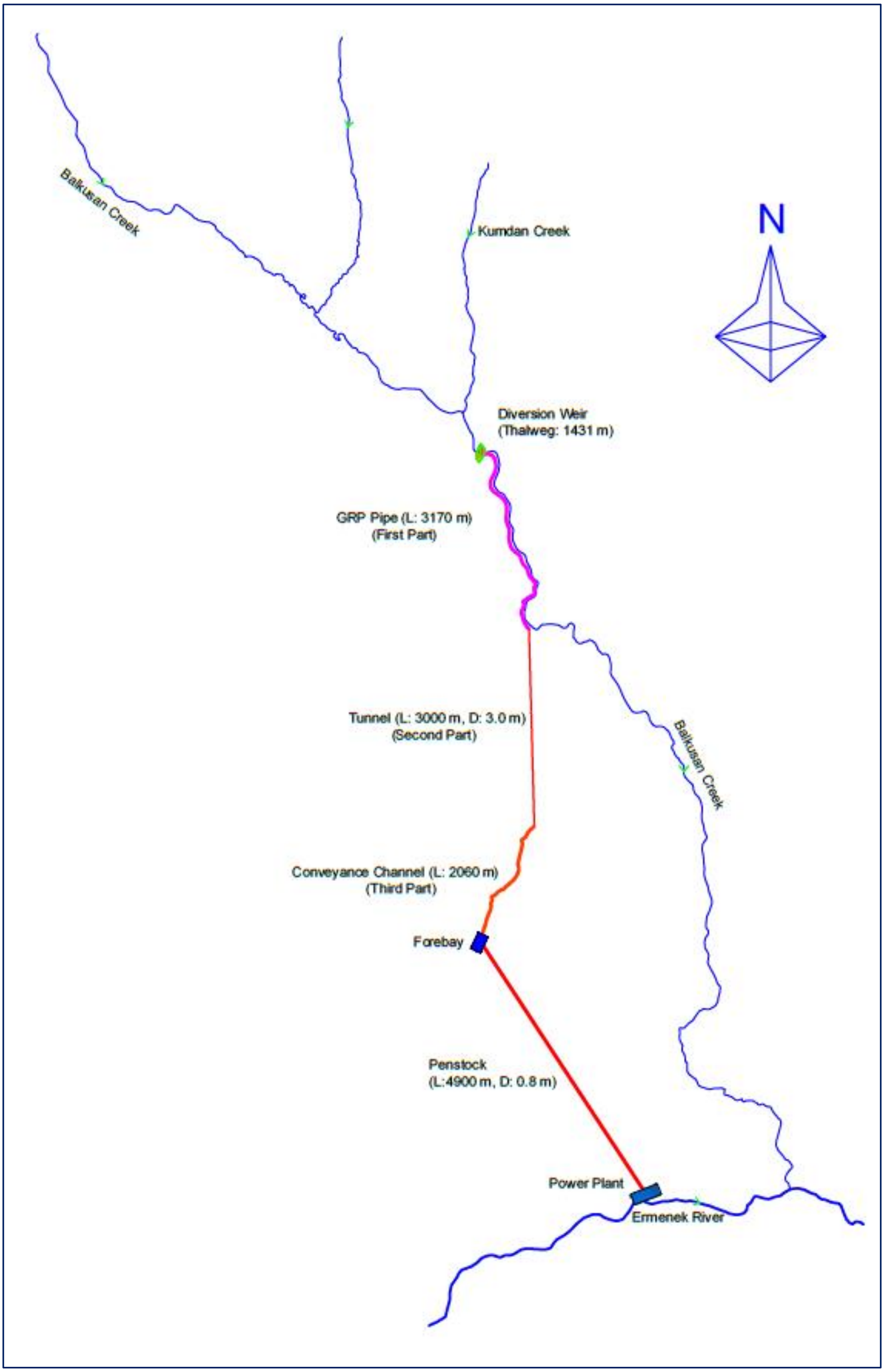
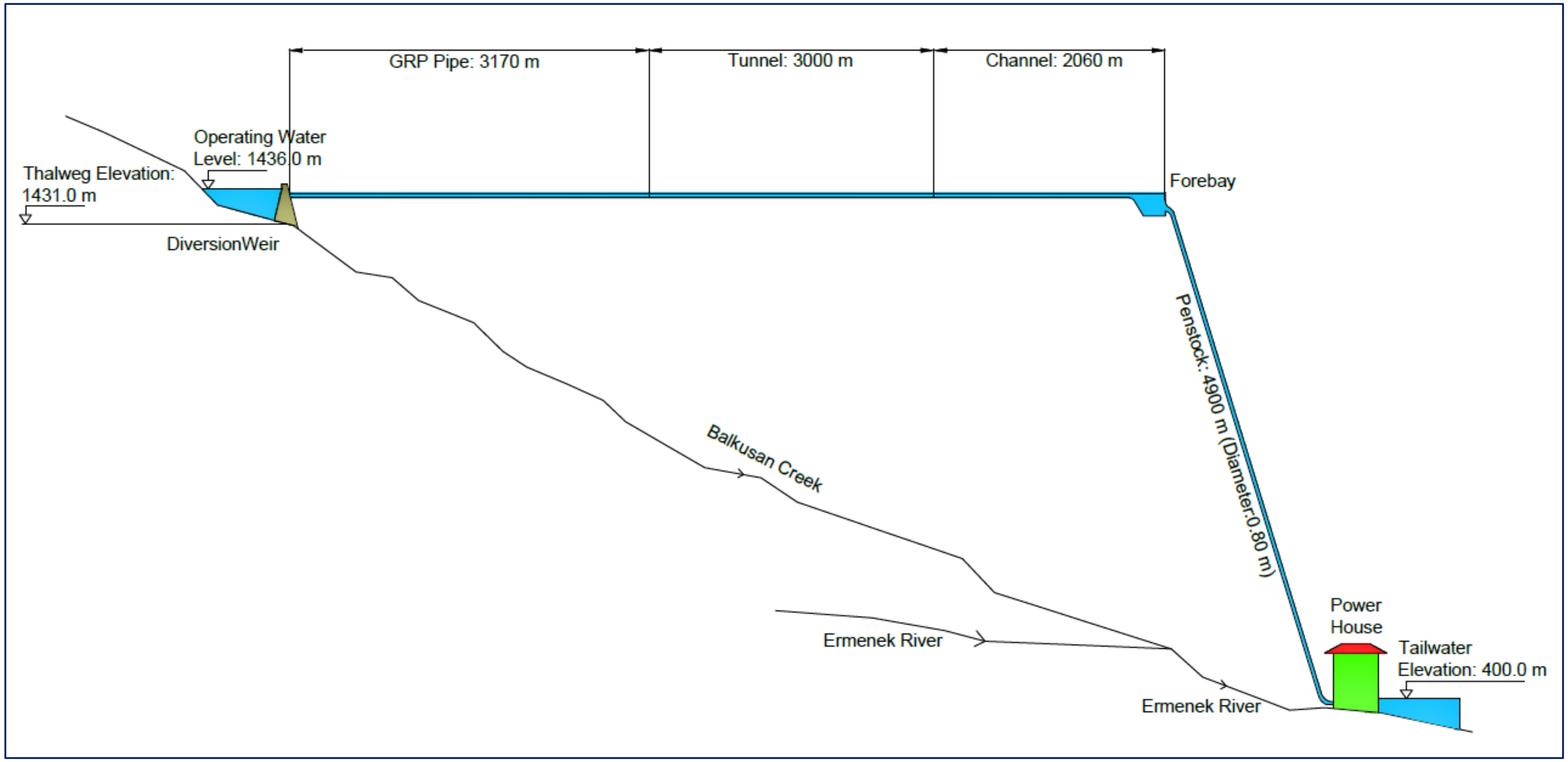


Figure 2.9 Plan of the Alternative-formulation (Not to Scale)

Figure 2.10 Profile of the Alternative-formulation (Not to Scale)



2.5 Advantages and Disadvantages of the EIEI-formulation

The main advantages of the EIEI-formulation are high firm energy production and energy generation during the peak power demand periods. Disadvantages of the EIEI-formulation are high investment costs due to dam body and expropriation. In addition, the transmission line (tunnel) is too long with respect to the other alternatives resulting in a high cost. The surge tank is located on cliffs. Usually large amounts of excavation is required to provide ground stability on this kind of rocky zone. These further increase the cost.

2.6 Advantages and Disadvantages of the Hidromark-formulation

Replacing Kumdan Creek transmission tunnel with a box-shaped channel results in a lower cost for the Hidromark-formulation compared to the EIEI-formulation. Furthermore, the separation of the system into two HEPPs not only shortens the length of the tunnel lowering the total cost but also transmission line and penstocks are located on a topographically more suitable route (Hidromark, 2009). The total pressure in penstocks is not as high as that of the EIEI-formulation and the Alternative-formulation. As a result of this, thinner walls can be used for the penstocks resulting in lower penstock cost. The Balkusan HEPP 2 benefits from Balkusan HEPP 1 tailwater and water collected downstream of HEPP 1 dam axis on Balkusan basin. Additional water to Balkusan HEPP 1 tailwater provides higher installed capacity and therefore more energy production capability for Balkusan HEPP 2. On the other hand, the Hidromark-formulation has two power plants. The costs related to the power houses become relatively higher with respect to the other alternatives.

2.7 Advantages and Disadvantages of the Alternative-formulation

The main advantage of the Alternative-formulation is avoiding high investment costs associated with storage type HEPPs such as expropriation and dam body costs. In addition, fertile lands are saved. The diversion weir is located in the downstream of the confluence of Balkusan and Kumdan Creeks and this not only eliminates the costs of Kumdan Creek diversion weir and the transmission line but also enables utilization of all the water collected from Balkusan and Kumdan basins. Other alternatives utilize waters from smaller sub-basins.

In river bends, sediment is deposited in the inner edge. Water intake should be located in the second half of the outer bend (Cecen, 1996). Thus, in the Alternative-formulation, in order to overcome possible sedimentation problems, the water intake is located at the outer edge of the second half of the first bend below the confluence of Balkusan and Kumdan Creeks. Additional measures can as well be taken for the sedimentation problem. For example, regular monitoring and optimum dimensioning of sluiceway prevents the deposition of sediment in front of the intake (UNESCO, 1985). Furthermore, in the event of deposition limiting the ability to divert water at low flows, excavation may be carried out (USBR, 2005). However, the costs associated with monitoring or excavations are not included in this study.

The disadvantages include variability of energy generation due to lack of a storage unit and a loss of head because of the difference in water intake levels of the Hidromark-formulation and the Alternative-formulation. Moreover, this formulation has the longest penstock which leads to higher penstock cost compared to other the alternatives.

2.8 Information about the Project Area

2.8.1 Hydrological Conditions

Drainage areas of the EIEI-formulation and the Hidromark-formulation are the same (i.e., 259 km^2). The EIEI and Hidromark-formulations both utilize water collected from the sub-basin located upstream of the Balkusan dam axis and Kumdan Creek weir. In the Alternative-formulation, the weir is located approximately 650 m downstream of the confluence point. The drainage area of the Alternative-formulation is 266 km^2. In 1999, EIEI installed a stream gauging station (1736) approximately 50 m downstream of the dam axis and this gauge has been collecting daily stream flow data since 2000. The daily streamflow data of EIEI 1736 between 2000 and 2009 are obtained for the water supply study. The vicinity of the project site is investigated to find a highly correlated stream gauge with EIEI 1736 which has longer data series. As a result, 1723 Çavuşköy Ermenek stream gauging station is identified and daily flow data (1986-2004) of this station are used to extend EIEI 1736's data for the period between 1986 and 1999. The locations of the dam axis, EIEI 1736 and EIEI 1723 stream gauging stations are given in Figure 2.11.

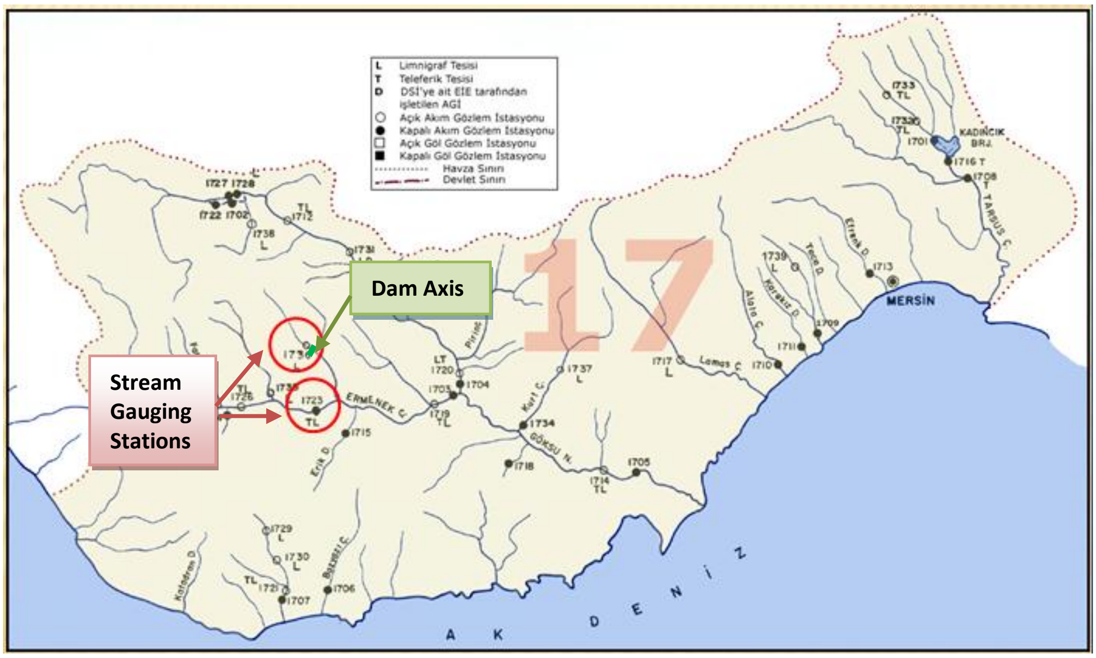


Figure 2.11 Locations of Dam Axis, EIEI 1736 and EIEI 1723 Stream Gauging Stations

2.8.2 Meteorological Conditions

Monthly evaporation and precipitation data are required to estimate the change in the amount of water stored in the reservoirs which is directly used in the reservoir operation studies of HEPPs. The closest meteorological stations to the project area are Ermenek and Hadim Meteorological Observation Stations (MOS). Ermenek MOS which is located at an elevation of 1250 m has an observation period of 55 years (1954-2004). Precipitation data of Ermenek MOS are used in energy generation calculations of the EIEI and Hidromark-formulations. The average annual precipitation observed in this station is 544.1 mm which is less than the average of Turkey (640.9 mm) (DMI, 2011). Hadim MOS which is located at an elevation of 1552 m is the only station at which evaporation measurements are conducted in the vicinity of project site. The average annual temperature observed at Hadim MOS is 9.7°C (Hidromark, 2009). Evaporation data of Hadim MOS are used in energy generation calculations of the EIEI and Hidromark-formulations in Chapter 4.

2.8.3 Earthquake Conditions

Balkusan Creek is a branch of Ermenek River and is located in the Ermenek District. Project site is located at the 5th degree earthquake zone (AFAD, 1996) which is the safiest among all zones (Figure 2.12).

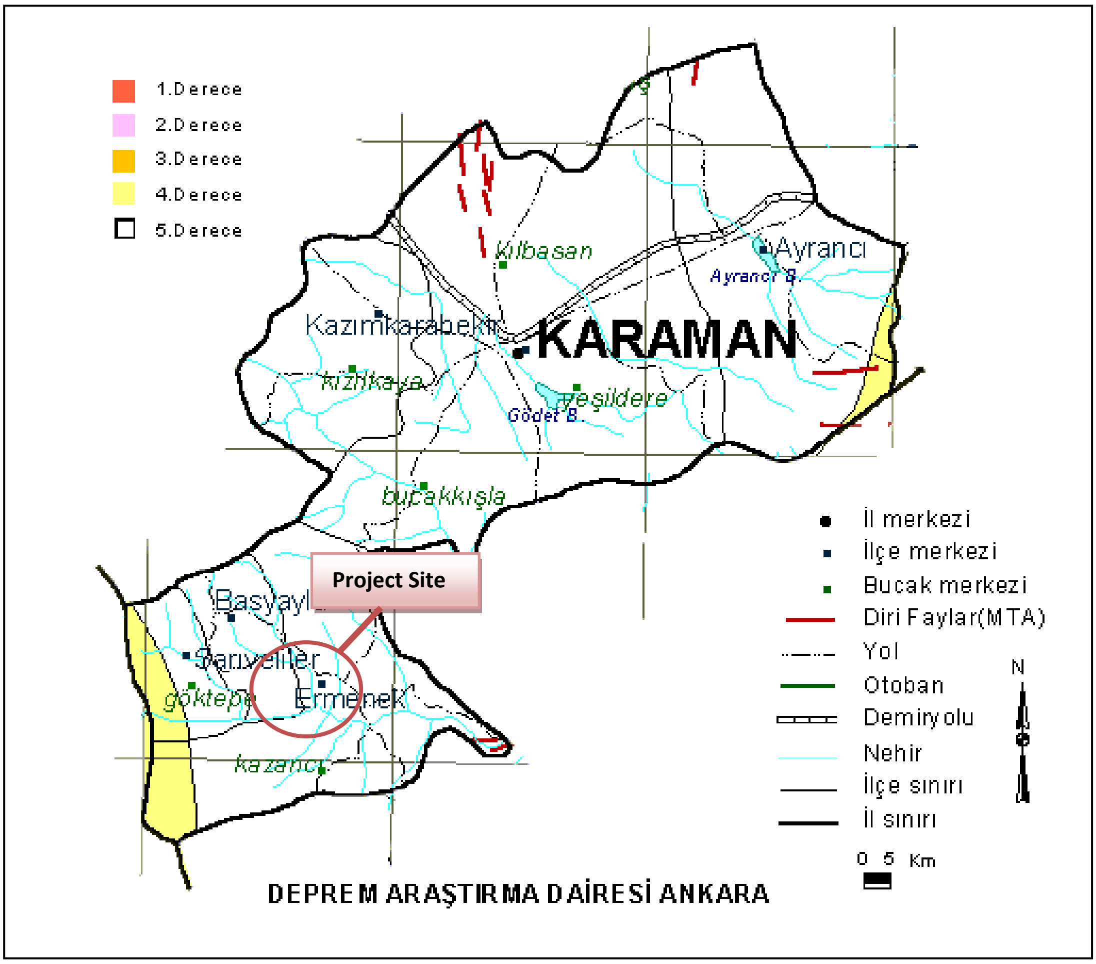


Figure 2.12 Earthquake Map of Karaman Province (Ozmen, 1997)

CHAPTER 3

ECONOMIC ANALYSIS FOR THE ALTERNATIVE-FORMULATION

3.1 Methodology

One of the most important steps of the feasibility study of a hydropower plant is the determination of the best installed capacity. The selection of the best installed capacity is based on an economic analysis. The basic elements of the economic analysis are the costs associated with the implementation of the project and the benefits obtained from energy generation. Costs and benefits of a hydropower plant change with respect to its installed capacity. Since the energy generation of a plant is evaluated annually, the initial investment costs need to be converted into annual investment costs by using an appropriate capital recovery factor (CRF). The net benefit can be calculated by subtracting total annual cost from annual energy income. In Figure 3.1, the relation between total benefits and total costs of a water resources investment (i.e. hydropower plant, irrigation network) is shown.

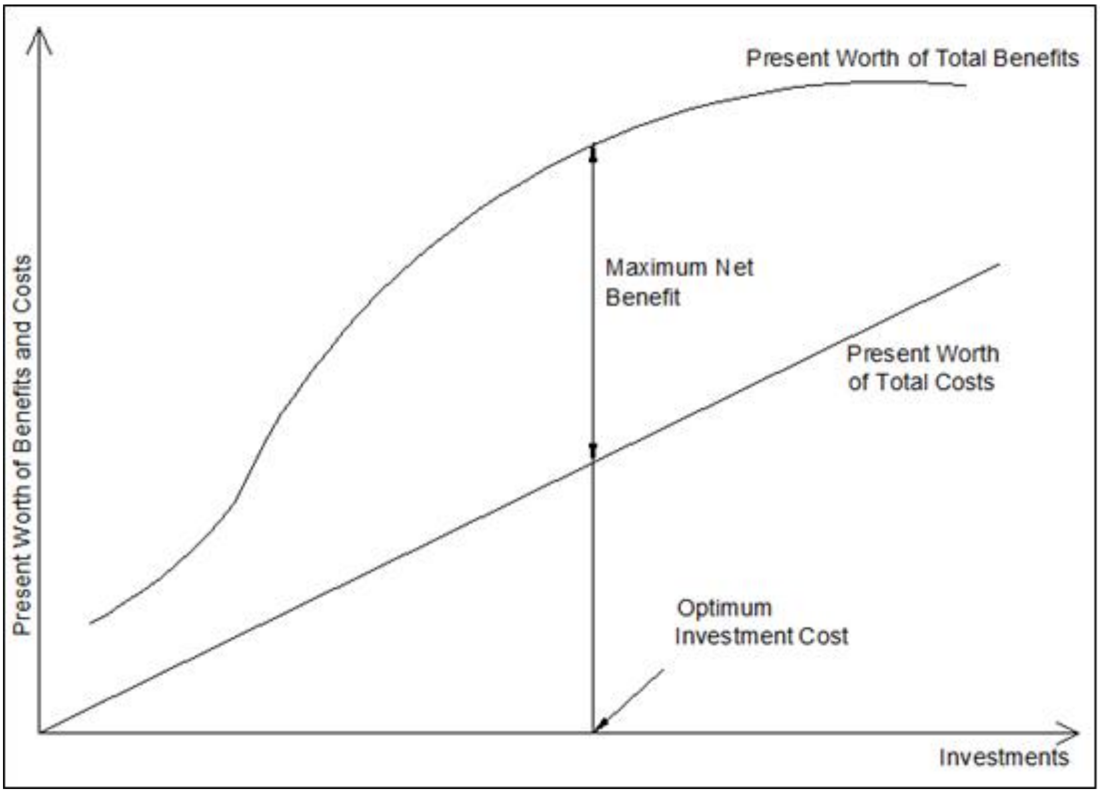


Figure 3.1. Cost and Income versus Installed Capacity Chart for Water Resources Investments (Karataban, 1976)

In the case of a hydropower plant, the design discharge and the corresponding installed capacity resulting in the highest net benefit is selected as the best installed capacity. To find the best installed capacity of the Alternative-formulation, a set of alternative discharges and the related installed capacities are evaluated through an economic analysis. An outline of the economic analysis is given in Figure 3.2.

Figure 3.2 Economic Analysis Procedure

To estimate the annual energy income of a hydropower plant which is a function of the annual energy generation, an operation study has to be conducted for each alternative installed capacity. The flow-duration method is the preferred method to calculate annual energy generation for run-of-river plants (USACE, 1985). Therefore, this method is utilized in the economic analysis of the Alternative-formulation. In the determination of energy income, methods developed by DSI and EIEI and single price method are used.

For each alternative installed capacity, the related costs need be determined as well. In the economic analysis, the costs of the channel, the glass-fibre reinforced plastic pipe, the penstock, the turbine, the transformer and the generator costs which vary considerably with the installed capacity are taken into account. On the other hand, the costs, which are not significantly affected by the installed capacity, such as the weir, the settling basin and the tunnel costs are not included in the economic analysis. Finally, the net benefits for each installed capacity are estimated, and the installed capacity corresponding to the maximum net benefit is selected as the best installed capacity. The details of the economic analysis are explained in the following sections.

3.2 Energy Income Estimation

3.2.1 Preparation of Required Hydrological Data

Estimation of the amount of incoming water is important for energy generation calculations of hydropower projects. To carry out an operation study for a hydropower plant, sufficient and dependable streamflow data is required.

3.2.1.1 Drainage Basins

The same location along Balkusan Creek is selected as the dam axis (thalweg elevation: 1450 m) in EIEI and Hidromark-formulations. The drainage area of this location is determined as 209.50 km^2 for both formulations. In addition to Balkusan Creek, the water coming from Kumdan Creek is also diverted to the Balkusan Creek by a weir for both formulations. For transmission of Kumdan Creek water, EIEI and Hidromark-formulations utilized a tunnel and a conveying channel, respectively. The drainage basin for Kumdan Creek at the weir location is estimated as around 49.50 km^2 for both formulations. Therefore the total drainage area is determined as 259 km^2. In the Alternative-formulation, differently from previous studies, the thalweg elevation is lowered from 1450 m to 1431 m which is nearly 650 m downstream of where the two creeks meet. This provides an additional 7 km^2 drainage area to the formerly determined drainage area. Thus, the total drainage area for the Alternative-formulation becomes 266 km^2 (See Figure 3.3).

EIEI 1736 stream gauging station is located approximately 50 m downstream of the dam axis. The drainage area of EIEI 1736 is determined as 203.5 km^2. The data measured at EIEI 1736 are transferred to water intake structure axes of EIEI, Hidromark and Alternative-formulations by using drainage area ratio method. The ratio of ungauged basin area to the gauged basin area must be within the range of 0.3 to 1.5 (USGS, 2005).

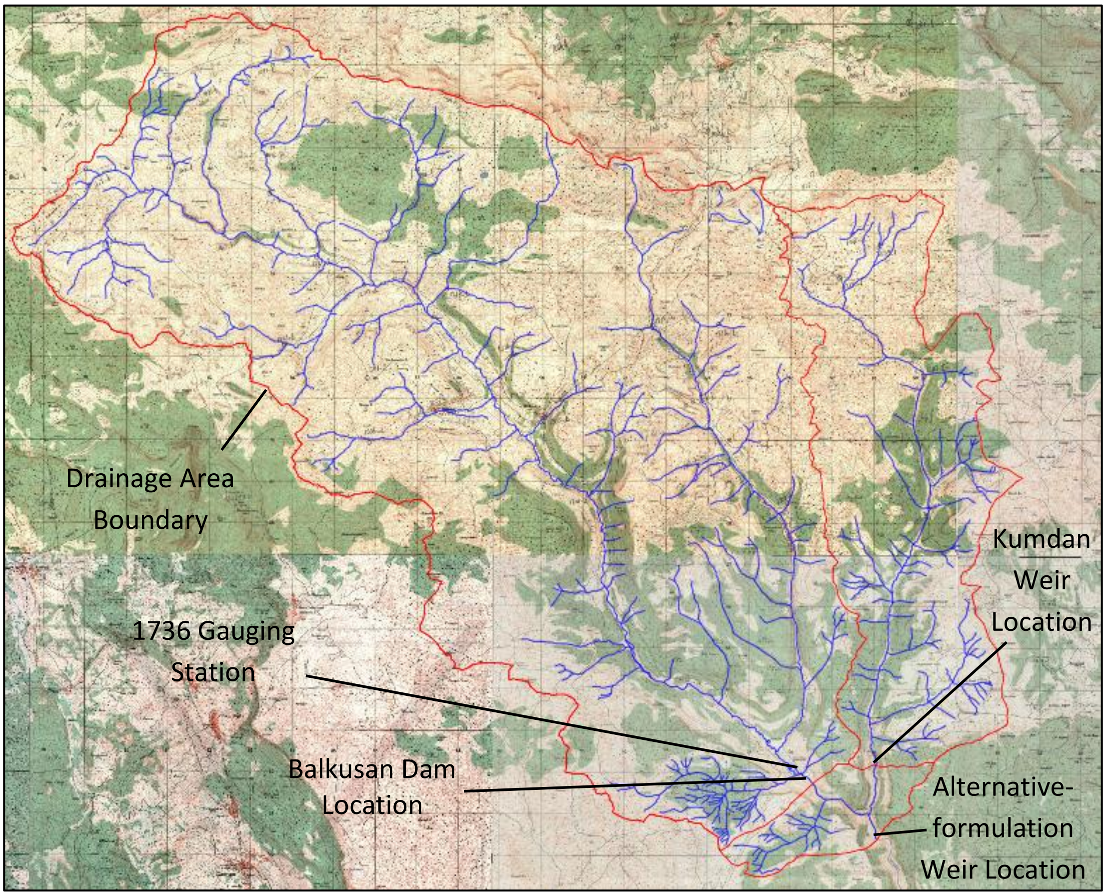


Figure 3.3 The Drainage Basin Areas for Three Formulations (Not To Scale)

3.2.1.2 Water Supply Study

The daily streamflow data are required for run-of-river plants where a portion of the stream flow is diverted depending on the amount available in the river (Prakash, 2004). Therefore, daily streamflow values of stream gauging stations EIEI 1736 and EIEI 1723 are used in the hydrological studies of the Alternative-formulation.

The daily observation period for EIEI 1736 (See Figure 2.11) stream gauging station is 10 years (2000–2009). The vicinity of the project site is investigated to identify other stream gaging stations close to EIEI 1736 which have longer daily data series. EIEI 1723 stream gauging station which is on Ermenek Stream (See Figure 2.11) has 19 years of daily observations between 1986 and 2004. The common observation period for EIEI 1736 and EIEI 1723 is 5 years. The elevations of EIEI 1736 and EIEI 1723 are 1452 m and 515 m. Although the elevations are considerably different a regression analysis is performed to evaluate the correlation between these two stream gauging stations. The equation of the fitted line is $Q_{1736}\ (m^3/s)\ = 0.0413937 \times (Q_{1723}\ (m^3/s))^{\ 0.85044}$, where $Q_{1736}\ (m^3/s)$ and $Q_{1723}\ (m^3/s)$ are the daily streamflow values for EIEI 1736 and EIEI 1723, respectively (see Figure 3.4). The correlation coefficient (R) is 0.9 which indicates that the correlation between two stations' streamflow values is acceptable. The correlation equation is used to extend EIEI 1736 daily streamflow data between the period 1986 and 2000 using EIEI 1723 streamflow data. Lastly, the extended daily streamflow data (1986–2009) of EIEI 1736 are transferred to related axis for EIEI, Hidromark and Alternative-formulations by using the drainage area ratio method.

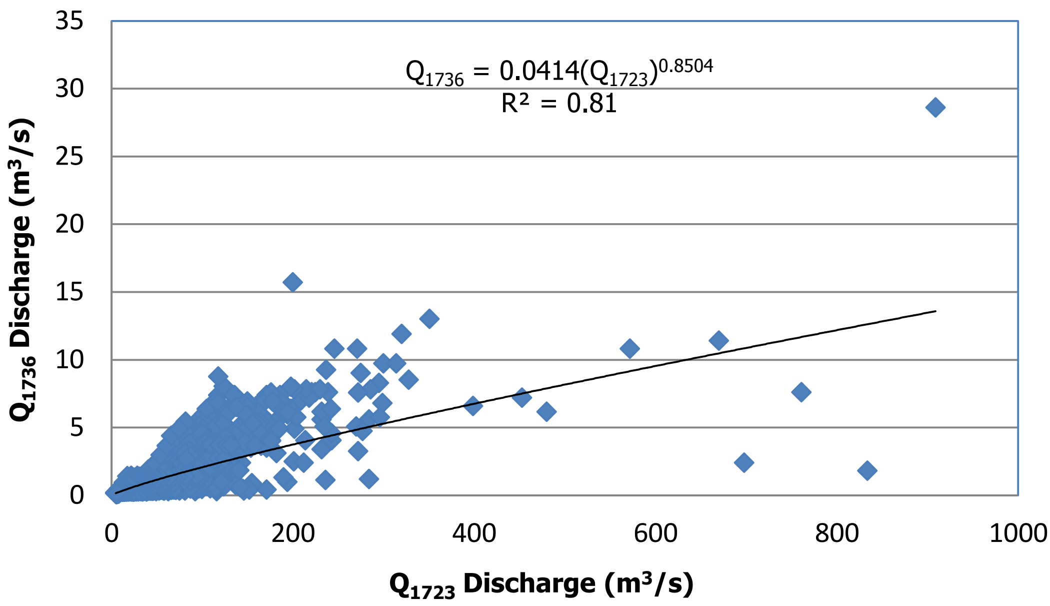


Figure 3.4 The correlation Between EIEI 1736 and EIEI 1723 Stream Gauging Stations

Although, a flow duration curve having a minimum record length of 30 years is preferable in hydropower projects (Yanmaz, 2006), due to unavailability of data 24-year-long extended daily streamflow data are used in energy generation calculations. The annual average streamflow for the Alternative-formulation is calculated as 1.11 m^3/s using the extended streamflow data that is transferred to the Alternative-formulations axis.

3.2.2 Energy Generation Calculations

3.2.2.1 Basic Definitions about Hydropower and Energy

In this section, some basic definitions related with hydropower and energy are provided.

Gross Head (H_g) (m) is the difference between the upstream water level at the intake and the downstream water level at the tailrace canal after the power house.

Net Head (H_n) (m) is the head available for energy production.

$$H_n = H_g - H_l \tag{3.1}$$

where H_l (m) is head loss.

Hydraulic Efficiency (η_h) is the ratio of the net head to the gross head.

Turbine Efficiency (η_t) is the ratio of the net potential energy running the turbines to the converted mechanical energy. It depends on the turbine type and its value changes within the related discharge and head range.

Generator Efficiency (η_g) is the ratio of the energy generation converted by the generator to the mechanical energy.

The Total Efficiency (η) is the product of the turbine, generator and transformer efficiencies.

$$\eta = \eta_t \eta_g \eta_{tr} \tag{3.2}$$

where η_{tr} is the transformer's efficiency.

Power (P) is the rate of energy production which can be estimated as follows:

$$P = \rho_w g\, Q H_n \eta \tag{3.3}$$

where P is power (kW), Q is the discharge (m^3/s), g is the gravitational acceleration which is 9.81 (m/s^2), ρ_w is the density of water which is 1 (g/cm^3), η is the total efficiency (%), and H_n is the net head (m).

Installed Capacity (P_{ins}) (kW) is the sum of power generating capacities in a power plant or power system.

Energy (E) (kWh) is the energy of doing work.The energy from water can be either potential energy by virtue of position, pressure energy due to the water pressure, or kinetic energy by virtue of water's moving force or action (Warnick, 1984) The energy production of a hydropower plant can be estimated using the following equation:

$$E = \int P dt = \int \rho_W g Q H_n \eta\, dt \tag{3.4}$$

Firm Energy (kWh) is defined as the power that can be delivered by a specific plant during a certain period of the day with at least 90 –95% certainty. For the Alternative-formulation, certainty level is selected as 90%.

Secondary Energy (kWh) is the energy available in excess of firm energy.

Annual Energy Generation (kWh) is the the summation of annual firm energy generation and annual secondary energy generation.

3.2.2.2 Operation Study of the Alternative-formulation

Operation Study of a Run-of-River Hydropower Plant

Two methods are commonly used for estimating the energy potential in hydropower plants: non-sequential or flow-duration curve method and sequential streamflow routing (SSR) method. Flow-duration curve method is the most appropriate method for high head run-of-river projects where the change in the head is limited and depends on the amount of inflow (Karamouz, 2003). To utilize this method, a flow-duration curve is developed based on the observed data.

The Alternative-formulation consists of a run-of-river hdyropower plant. The term "run-of-river" refers not only to the type of the hydropower plant but also to the operation mode. This kind of hydropower plant has no storage capacity. Hence, energy production at any time is directly related with the inflow.

The Flow-Duration Method Procedure

1. A flow-duration curve (FDC) is constructed using available streamflow data.
2. Flow losses reducing power generation (residual flow, evaporation, etc.) are determined.
3. A head versus discharge curve is developed that reflects the variation of tailwater elevation with inflow to identify head used in power equation. The change in tailwater elevation is very small compared to high gross head and can be taken as a fixed elevation (400 m) in high head run-of-river hydropower plants (Karamouz, 2003). Therefore, in this study head computation is included directly in the power equation by computing net heads for usable range of flows.

4. The design discharge is selected. The design discharge has to be identified through a decision making study. Once the set of alternative discharges are selected based on the constructed FDC and the net head is estimated, the type of turbine is identified (ESHA, 2004).
5. The usable flow range is defined. The portion of streamflow which can be used for power generation is limited by the characteristics of the selected turbine type and flow losses such as residual water.
6. The available discharges existing in the usable range of flow and corresponding net heads are used in power equation to calculate daily power generation by using Equation 3.3.
7. Daily energy generation is estimated by multiplying power generations with the period of 24 hours (see Equation 3.4). Each daily generation is summed up and divided the number of years of developed water supply study to calculate average annual energy generation. The procedure is applied to all identified alternative discharges.

In this study, an Excel spreadsheet is prepared to carry out flow-duration curve procedure to conduct the operation study for the Alternative-formulation. It should be remembered that daily streamflow data is used to generate the flow-duration curve. The necessary explanations and calculations of flow-duration method steps are given in the following sections.

Development of the Flow-Duration Curve and Identification of the Set of Alternative Discharges

A flow-duration curve illustrates the percentage of the time for which a given discharge equalled or exceeded. Historical streamflow records can be well represented by a FDC. In the Alternative-formulation, the FDC is used to identify the alternative discharges; then corresponding installed capacity are calculated using equations provided in Section 3.2.2.1. The use of daily streamflow data is recommended for development of the flow-duration curve (Gulliver and Arndt, 1991). The FDC of the Alternative-formulation is derived and given in Figure 3.5.

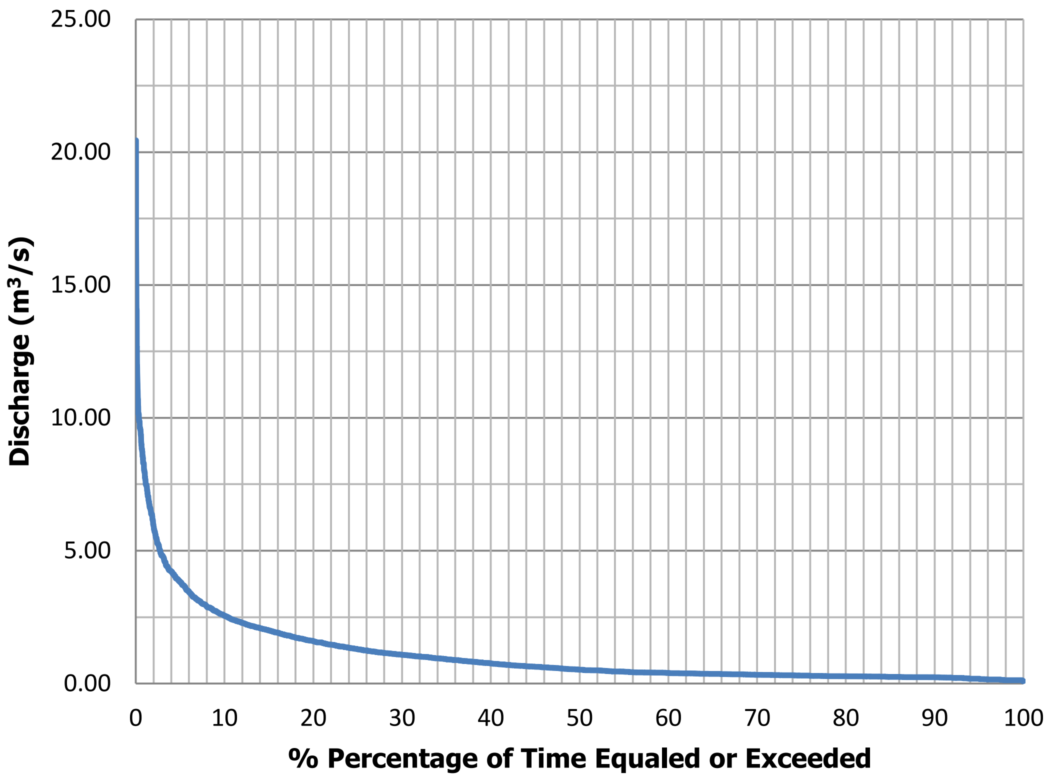


Figure 3.5 Flow-duration Curve of the Alternative-formulation

The sizing of a run-of-river type hydropower plant is a hard task in terms of cost effectiveness. The lack of water storage reservoir and non-uniformity and seasonal variation of natural flow rate make it difficult to determine the best installed capacity (Anagnostopoulos, 2007). The selection of the best installed capacity of a run-of-river plant mainly depends on the flow availability and the analysis of flow-duration curve which shows the percentage of time that the project site flow equals or exceeds a certain value (Santolin, 2011).

The flow-duration curve of the Alternative-formulation is utilized to identify the set of alternative discharges used in the selection of the best installed capacity. The discharges equaled or exceeded between 10% and 40% of the time are usually considered in the determination of the most advantageous design discharge (Pekcagliyan, 2003). The installed capacity having the most profitable design discharge is then identified as the best installed capacity.

The discharges equaled or exceeded between 10% and 40% of the time for the Alternative-formulation are shown in Table 3.1. The discharges starting from 0.8 m^3/s and reaching 2.5 m^3/s with an increment of 0.1 m^3/s are used in this study. This gives a total number of 18 alternative discharge values.

Table 3.1 Discharges (m^3/s) equaled or exceeded 10% and 40% of the time for the Alternative-formulation

% Time Equaled or Exceeded	Discharges (m^3/s)
40	0.76
10	2.56

Residual Water Flow

The necessary amount of water has to be released from the watercourse so that serious impacts of drought on aquatic life could be avoided. This amount is called the residual water flow. According to water use right agreements, the residual flow can be taken as at least 10% of the average of the last ten years' streamflow at the water intake location in feasibility studies (DSI, 2006). However, amount of residual water depends on the specific characteristics of the river section and aquatic life that exists in that section. Therefore, the characteristics of the project site has to be evaluated and the amount of water released from water intake structure should be increased if necessary.

Initially, 10% of the average of the last ten years' streamflow measurements are used in the operation study. The average of last ten years streamflow value is 1.11 m^3/s for the Alternative-formulation. The average of last ten years streamflow value is 1.08 m^3/s for EIEI and Hidromark-formulations. Additionally a detailed analysis is conducted with varying residual water flows. The results of this analysis is provided in Section 3.5.

Determination of Type, Size and Number of Turbines

Turbines are the machines transforming water potential energy into mechanical energy. Widely used turbine types throughout the world are francis, pelton and kaplan. The type of a hydraulic turbine of a hydropower plant whose net head and design discharge are already determined is selected from charts prepared by turbine manufacturing companies (Yuksel, 2010). One such chart is given in Figure 3.6. Since the gross head of the Alternative-formulation is about 1000 m, pelton turbine is found appropriate for this study.

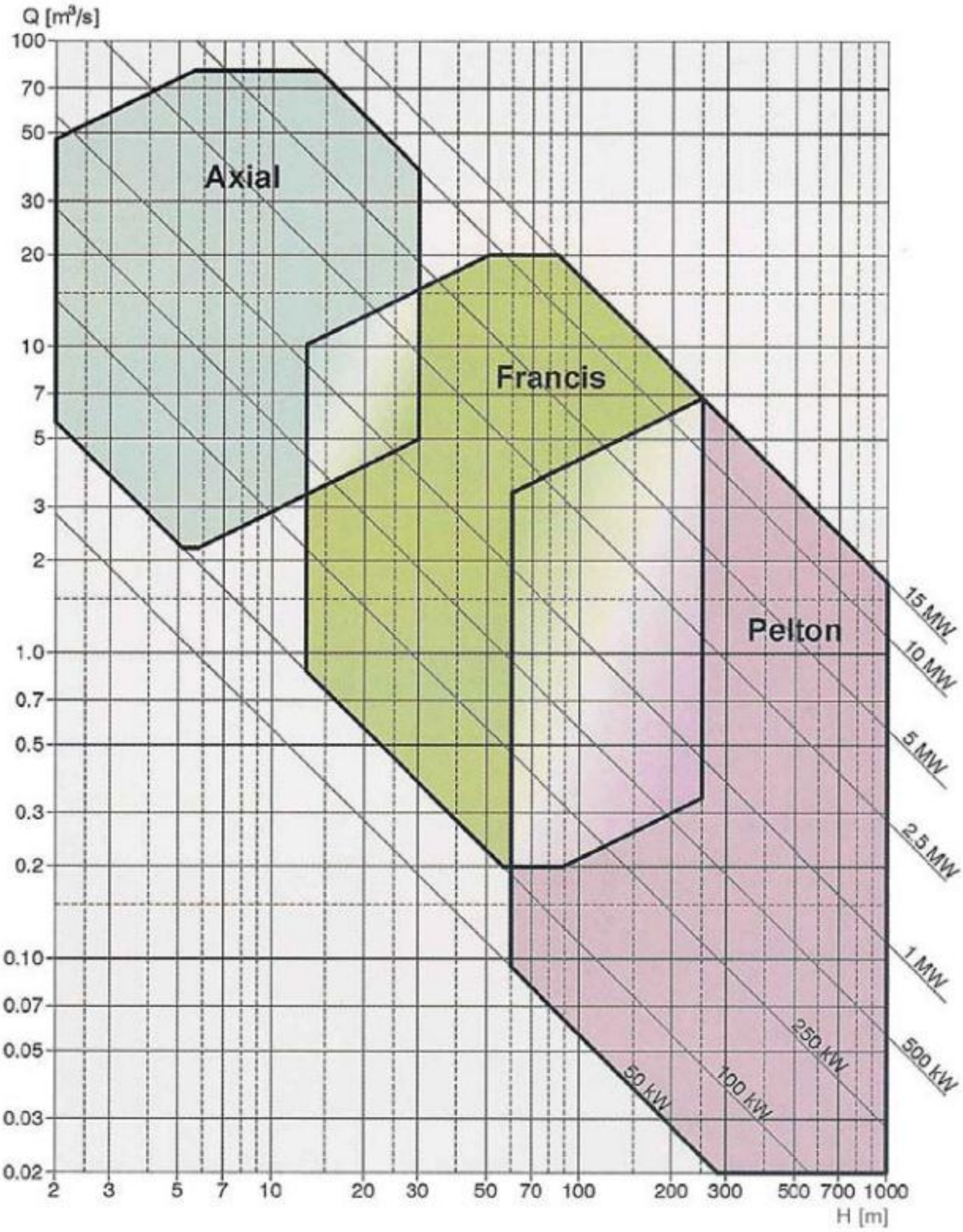


Figure 3.6 Turbine Type Selection Chart (ESHA, 2004)

Diverse views exist in the determination of the number and size of turbines and related generating units for hydropower plants. As larger size of turbine and corresponding generating units are selected, the capital and maintenance cost per kilowatt decreases. Therefore, it is more economical to use fewer numbers of turbines with larger capacities than a larger number of small size turbines for the same total capacity (Deshpande, 2010). However, Turbine efficiency characteristics also affect the determination of the number of turbines in a plant. The rate of energy production with discharges less than the design discharge can be increased by increasing the number of turbines with smaller capacities since small turbines can be operated with these discharges more efficiently than larger turbines (see Figure 3.7). The final choice requires an economic analysis (Creager, 1958).

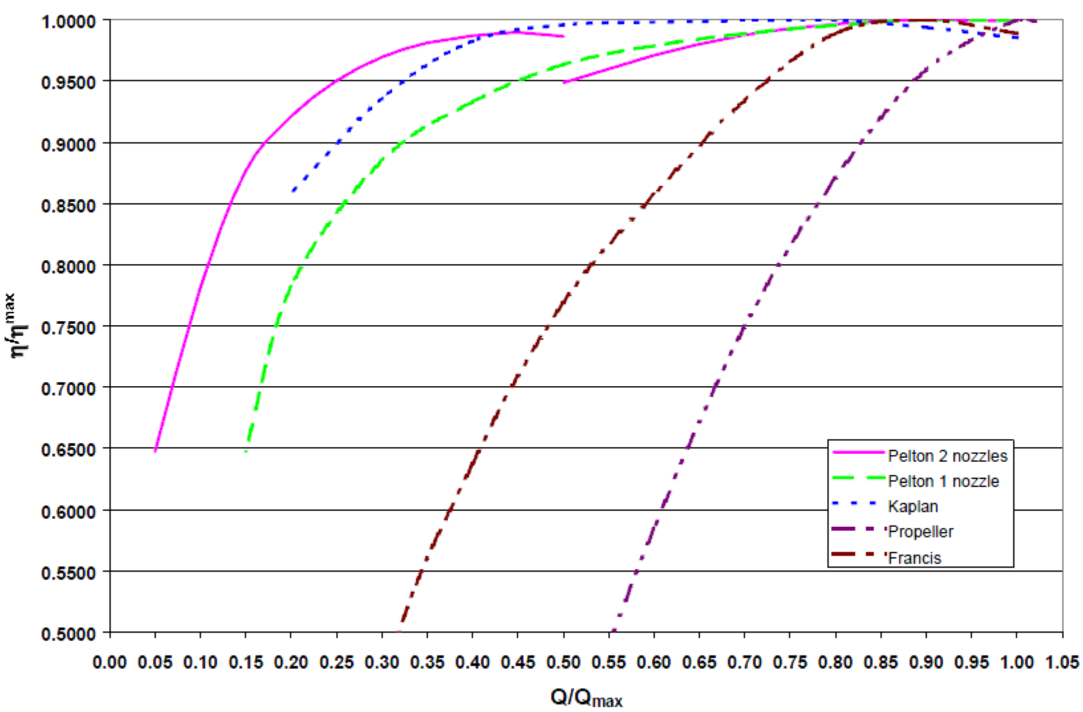


Figure 3.7 Efficiency Curves of Various Hydro Turbines (ESHA, 2004)

Determination of Usable Range of Flow

Usable range of flow is the range of flow for which hydraulic turbines can operate. The range is restricted by the residual water amount and turbine characteristics such as minimum turbine discharge.

Minimum turbine discharge (Q_{min}) is the minimum permissible discharge through a turbine (ASCE, 1989). For pelton turbine which is used in the Alternative-formulation, Q_{min} is taken as 10% of design discharge of each turbine (ESHA, 2004).

The following rules are used for the determination of usable range of flow during operation (Santolin, 2011) :

1. If the inflow is smaller than the required residual water amount, there will be no energy generation. All the coming water is released to the creek.
2. If the inflow is between required residual water amount and the design discharge, firstly downstream water requirement is met and then the excess water is operated unless it is less than the minimum turbine discharge.
3. If the inflow is greater than the design discharge of available turbines, the excess water is spilled from the spillway or bottom outlet.

The foregoing rules are used to determine the amount of water that will be send to the turbines.

In hydropower projects, more than one, especially two or three and sometimes even more turbines are used in order to maintain energy generation in the break-down situations and increase the efficiency of energy generation. Conventionally, two turbines with equal capacities are preferred in Turkey (Guner, 2008). In this study, the full economical analysis of the Alternative-formulation is conducted for two turbines with equal installed capacities. However, in order to investigate impact of two turbines with different sizes a detailed energy generation analysis is performed and presented in Section 3.6. A logic chart for flow allocation among two equal sized turbines is developed and used in the operation study of the Alternative-formulation. The logic chart is given in Figure 3.8. An Excel Spreadsheet is developed to conduct the operation study for the Alternative-formulation.

For inflows coming to the turbines which are equal or larger than the design discharge, all the turbines are run at full capacity. On the other hand, when the inflow is less than the design discharge, it should be allocated to the turbines rationally to obtain maximum energy generation. Therefore, for incoming flow less than design discharge, the main goal is to utilize all the incoming water as much as possible to generate maximum energy. After the allocation of inflow to available turbines, turbine efficiency is calculated using the ratio of allocated inflow to the turbine discharge using Figure 3.7. The generator and transformator efficiencies are taken as 0.96 and 0.98, respectively (Yildiz, 1992).

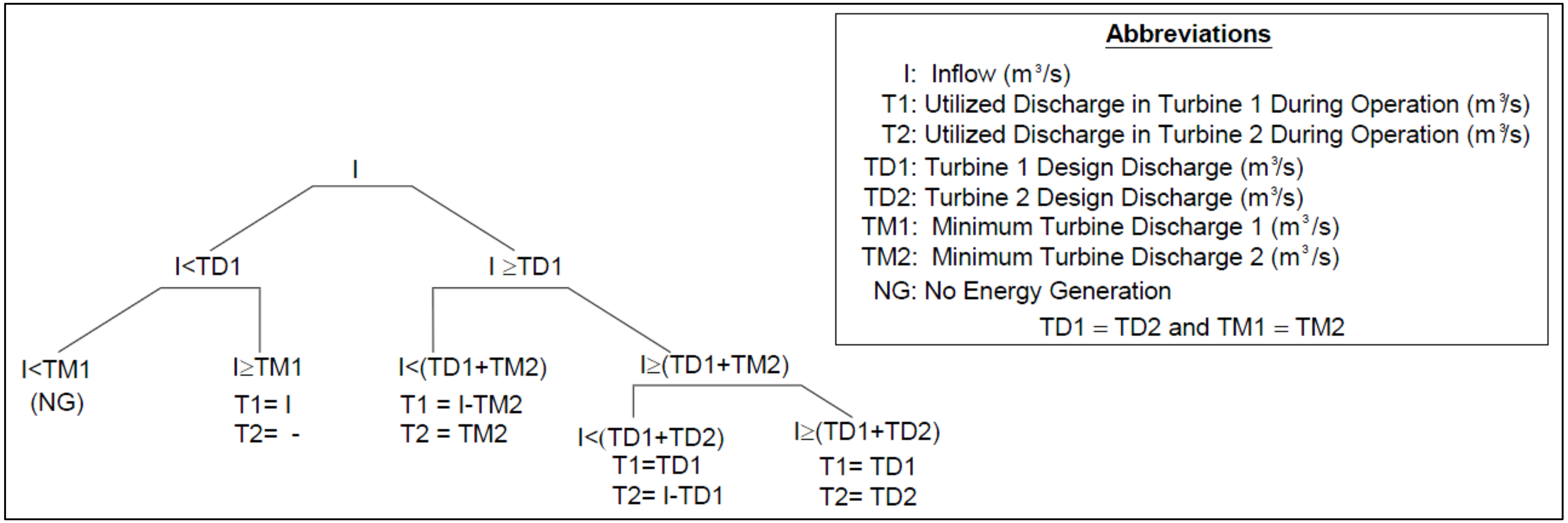


Figure 3.8 Logic Chart for Flow Allocation among Two Equal Sized Turbines

The Calculation of The Net Head

In order to calculate energy production, the net head has to be determined. The equation for net head is provided below:

$$\mathrm{H_{net}} = \mathrm{H_{for}} - \mathrm{H_{TW}} - \Delta\mathrm{H_{penloss}} \tag{3.5}$$

where $\mathrm{H_{net}}$ (m) is the net head for run-of-river plant, $\mathrm{H_{for}}$ (m) is the water level in the forebay, $\mathrm{H_{TW}}$ (m) is the tailwater elevation and $\Delta\mathrm{H_{penloss}}$ (m) is the total energy loss associated with the penstock due to friction and minor losses.

The friction loss occurred in the penstock can be estimated by using the Darcy-Weisbach Equation. This equation is given as below (Munson, Young, and Okiishi, 1998) :

$$hf = f\,\frac{L}{D}\,\frac{V^2}{2g} \tag{3.6}$$

where hf (m) is the friction loss, f is the dimensionless friction coefficient, L (m) is the length of the penstock, D (m) is the penstock diameter, V (m/s) is the average velocity in the penstock and g is the acceleration due to gravity (9.81 m/s^2). Here, L, D and g are known. The average velocity V can be estimated by dividing the discharge, Q (m^3/s) entering the penstock with the cross-sectional area ($\pi\,D^2/4$).

$$V = \frac{Q}{\pi\,D^2/4} \tag{3.7}$$

The Swamee-Jain equation is used to determine the friction coefficient in the net head calculations. The equation is given by the following expression (Munson, Young, and Okiishi, 1998) :

$$f = \frac{1.325}{\left\{ln\left[\frac{\varepsilon}{3.7D} + \frac{5.74}{Re^{0.9}}\right]\right\}^2} \tag{3.8}$$

where, ε (m) is the roughness height, D (m) is the diameter of penstock and Re is the Reynolds Number calculated using the formula given below.

$$Re = \frac{VD}{\vartheta} \tag{3.9}$$

where, ϑ (m^2/s) is the kinematic viscosity of water.

The minor losses observed in the penstock mainly consist of entrance contractions, expansions, bends, gates and valve constrictions. For all alternative penstock diameters, these minor losses should be estimated. Usually the minor losses can be expressed as a fraction of the velocity head as shown in the formula (Munson, Young, and Okiishi, 1998) :

$$h_m = K\,(\frac{V^2}{2g}) \tag{3.10}$$

where h_m (m) is the minor head loss and K is the dimensionless loss coefficient.

3.2.2.3 Energy Income Estimation Methods

Three different methods are used to estimate energy incomes of EIEI, Hidromark and Alternative-formulations:EIEI method, DSI method and Single Price method.

EIEI and DSI methods both classify the energy generation into two categories as firm and secondary energy according to supply priority to the system. The prices assigned to firm energy is greater than the prices assigned to secondary energy in both methods. EIEI and DSI methods uses "peak power benefit" concept in estimating the energy income. Calculation of peak powers by EIEI and DSI methods are given below:

Peak Power estimation by EIEI Method: This approach is suggested by experts of Japan International Cooperation Agency (JICA) (Pekcagliyan, 2003). The approach is based on a comparison between hydropower plant and combined natural gas thermal plant. Firm energy is estimated as the fuel, operation and maintenance costs of a combined thermal power plant. Secondary energy is determined as the savings from variable fuel, operation and maintenance costs of a coal thermal plant. Peak power is calculated by the following formula (Pekcagliyan, 2003) :

$$Peak\ Power\ (kW) = \frac{Annual\ Firm\ Energy\ (kWh)}{0.33\ x\ 8760\ (hours)} \tag{3.11}$$

Peak Power estimation by DSI Method: In this approach, firm energy benefit is calculated as the fixed and variable operation and fuel costs spent per kilowatt of energy generated in combined natural gas thermal power plant. Secondary energy is taken as the summation of variable (operation, maintenance and fuel) costs of a imported coal firing thermal power plant. Peak power can be calculated by the following formula (Pekcagliyan, 2003) :

$$Peak\ Power\ (kW) = Ins.Cap.(kW) - \frac{Annual\ Firm\ Energy\ (kWh)}{0.72\ x\ 8760\ (hours)} \tag{3.12}$$

Peak power term is more applicable to storage type plants, thus EIEI and DSI energy income methods are only used in the energy income estimations of EIEI and Hidromark-formulations. In other words, the energy income estimation of Alternative-formulation is conducted using only single price method.

The prices for firm and secondary energy used by EIEI and DSI are given in Table 3.2.

Table 3.2 Income Prices for EIEI and DSI Methods (Pekcagliyan, 2003)

Type of Energy	Prices	
	EIEI	DSI
Firm Energy	4.5 cent/kWh	6.0 cent/kWh
Secondary Energy	3.5 cent/kWh	3.3 cent/kWh
Peak Power	240.0 $/kW	85.0 $/kW

Third method is the single price method which is the simplest one among three. It assigns a fixed price (i.e. 7.30 cent/kWh) for the generated energy (EMRA, 2011). The exchange rate of USD to TL is taken as 1.8 in this study.

Energy incomes are calculated by the following formulas:

$$Energy\ Income\ (EIEI)(\$) = EIEI\_Peak\ Power\ (kW) \times Peak\ Power\ Price\ (\$/kW) + Firm\ Energy\ (kWh) \times Firm\ Energy\ Price\ \left(\$/_{kWh}\right) + Secondary\ Energy\ (kWh) \times Secondary\ Energy\ Price\ \left(\$/_{kWh}\right) \tag{3.13}$$

$$Energy\ Income\ (DSI)(\$) = DSI\_Peak\ Power\ (kW) \times Peak\ Power\ Price\ (\$/kW) + Firm\ Energy\ (kWh) \times Firm\ Energy\ Price\ \left(\$/_{kWh}\right) + Secondary\ Energy\ (kWh) \times Secondary\ Energy\ Price\ \left(\$/_{kWh}\right) \tag{3.14}$$

$$Energy\ Income\ (Single\ Price)(\$) = Firm\ Energy\ (kWh) \times Fixed\ Price\ \left(\$/_{kWh}\right) + Secondary\ Energy\ (kWh) \times Fixed\ Price\ \left(\$/_{kWh}\right) \tag{3.15}$$

3.3 Determination of Costs

The diversion weir, the settling basin, the GRP pipe, the tunnel, the conveyance channel, the forebay, the penstock, the power house, the turbine, the transformer, and the generator costs are considered as the components of the Alternative-formulation. Costs of the GRP pipe, the conveyance channel, the penstock, the powerhouse, the turbine, the transformer and the generator are considerably affected by the installed capacity. Hence, these costs need to be evaluated while selecting the best installed capacity.

The costs of the Alternative-formulation components are obtained using bill-of-quantities method and given in the following sections. These costs are summed with unforeseen costs and named as facility costs. Investment costs are obtained by adding project, surveying and control costs to facility costs. Investment costs are annualized using CRF and named as annual investment costs. Annual investment costs are summed with operation and maintenance costs to obtain the total annual cost. In the economic analysis of the Alternative-formulation, only annualized costs of hydraulic component values are taken into consideration. The facility cost, investment cost, operation and maintenance cost and total annual cost terms are calculated in comparison of alternatives in Chapter 4.

As energy incomes for each alternative installed capacity are calculated on yearly basis, the related estimated costs should be brought to the same time base. These costs are converted using the capital recovery factor (CRF).

3.3.1 Capital Recovery Factor (CRF)

The investment costs of a hydropower plant project are generally incurred before the commencement of the project or during construction period. However, energy generation incomes are estimated on an annual basis throughout the economic life span of the project. In order to determine whether the project is economically feasible, the cost and income items of the project should be investigated for a common time base (Bozkurt, 2011). CRF is used to convert present value into equal annual payments over a specified time.

The CRF is given by the following formula (Mussatti, 2002) :

$$CRF = \frac{i(1+i)^n}{(1+i)^n - 1} \tag{3.16}$$

where, n is number of years and i is the interest rate.

The investment costs of the hydropower plant project are multiplied by CRF to have their annualized values. In the Alternative-formulation, as in most of the hydropower practices in Turkey, economic life of a hydropower project and interest rate are taken as 50 and 9.5%, respectively. The CRF is calculated as 0.096.

3.3.2 Penstock Cost

Penstock cost forms the major part of a high head hydropower project cost. Therefore, selection of the penstock diameter and wall thickness is vital (Yildiz, 1992). An economic analysis is carried out for the Alternative-formulation to select economically best penstock diameter for each alternative discharge. The diameters examined in the economic analysis are chosen with an increasing increment of 0.1 m starting from 0.6 m. Flow velocity in penstock should be around 3 m/s to 6 m/s (Cofcof, 1996).

The analysis is composed of two kinds of cost: loss energy income and investment cost of the penstock. Both costs are calculated in annual base. The diameter which minimizes the summation of loss energy income and material cost is selected as the economically best penstock diameter for corresponding alternative discharge (see Figure 3.9).

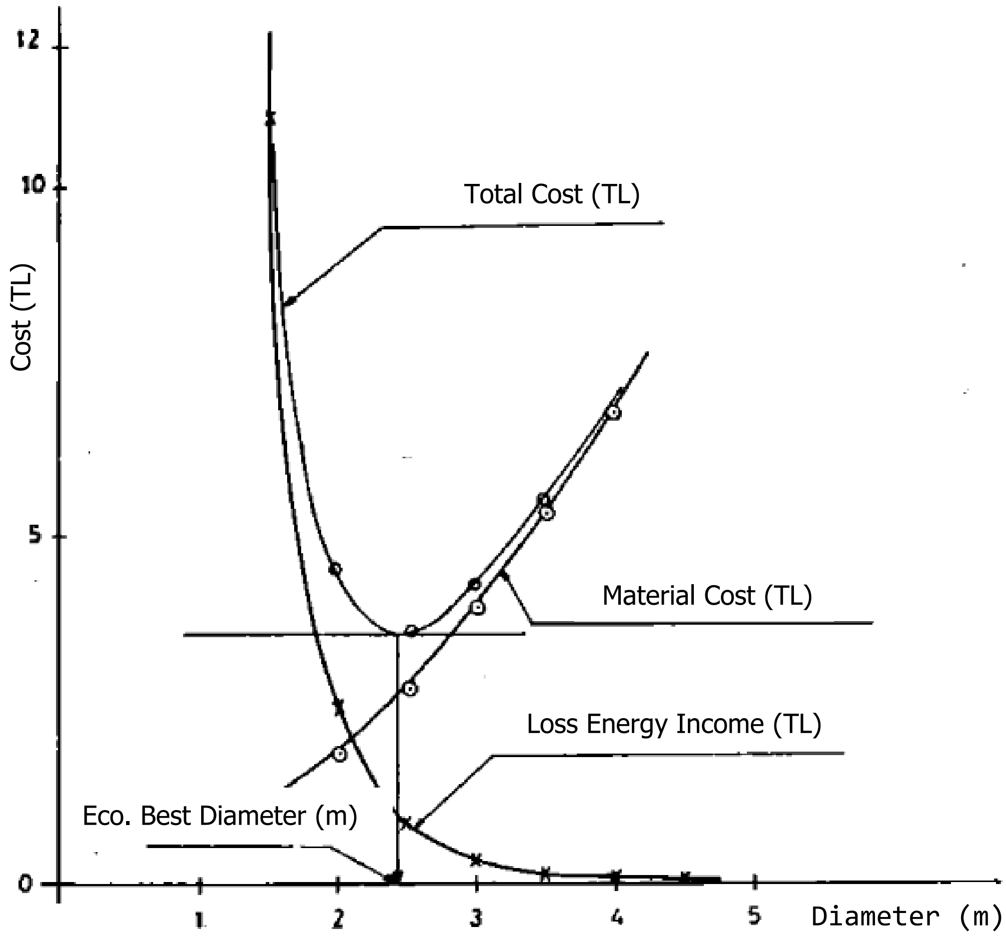


Figure 3.9 Selection of Economically Best Penstock Diameter (Keloglu, 1995)

Loss Energy Income

Loss energy income is the income which cannot be generated due to losses occurred in penstock.

In the calculation of loss energy generation, the operation study identified in Section 3.2.2.2 is directly used with only one difference. Head loss occurred in the penstock is used instead of net head. The power equation becomes:

$$\Delta P = \rho_w g \eta Q \Delta \mathrm{H}_{\mathrm{penloss}} \tag{3.17}$$

where ΔP is the loss power (kW), Q is the discharge (m^3/s), g is the gravitational acceleration (9.81 m/s^2), ρ_w is the density of water (1 g/cm^3), η is the total efficiency (%), $\Delta\mathrm{H}_{\mathrm{penloss}}$ is the head loss in penstock (m).

Loss energy is calculated by taking the integral of loss power over a year period. Single price method is applied (7.30 cent/kWh) and loss energy income is estimated for each penstock diameter determined for alternative discharges.

Material Costs of the Penstock

Steel is preferred as penstock pipe material since it has very high bearing capacity for continuous and variable pressure exposures (Erdem, 2006). The costs of penstocks are mostly based on steel cost. The weight of the penstock is found multiplying the volume of steel used for construction by cost of steel per kilogram. The unit cost of steel is taken as 3 $/kg (Toprak-Su, 2011). The volume of the penstock depends on the length and cross-sectional area of the penstock. Cross-sectional area varies with required wall thickness.

The penstock wall thickness depends on material, ultimate tensile strength, diameter and operating pressure. Operating pressure is composed of static pressure and dynamic pressure. Static pressure is formed by the water elevation in the forebay. A sudden change of flow due to valve closure or turbine load rejection causes a great mass of water wave moving inside penstock known as water hammer. It can cause dangerously high pressures, in other words dynamic pressure, in the penstocks (ESHA, 2004). The calculation methods of static pressure and dynamic pressure throughout the penstock are given in the following paragraphs.

Static Pressure

Static water pressure at any point is the elevation difference between the maximum water level in the forebay and analysed point in the penstock (Ak, 2011). The related formula is given below:

$$H_{sta} = \mathrm{H}_{\mathrm{for}} - H_{ap} \tag{3.18}$$

where H_{sta} (m) is the static pressure, $\mathrm{H}_{\mathrm{for}}$ (m) is the water level in the forebay, and H_{ap} is the elevation of analysed point in the penstock.

Dynamic Pressure

Dynamic pressure formula is given below (Yildiz, 1992) :

$$H_{dyn} = \frac{2vL}{gT_c} \tag{3.19}$$

where H_{dyn} (m) is the dynamic pressure, v (m/s) is the maximum velocity in the penstock, L (m) is the length of penstock and T_c is the turbine closure time. T_c is taken as 6 seconds as conventionally done in Turkey. In Figure 3.10, the penstock cross-section, forces and pressures on the cross-section are shown.

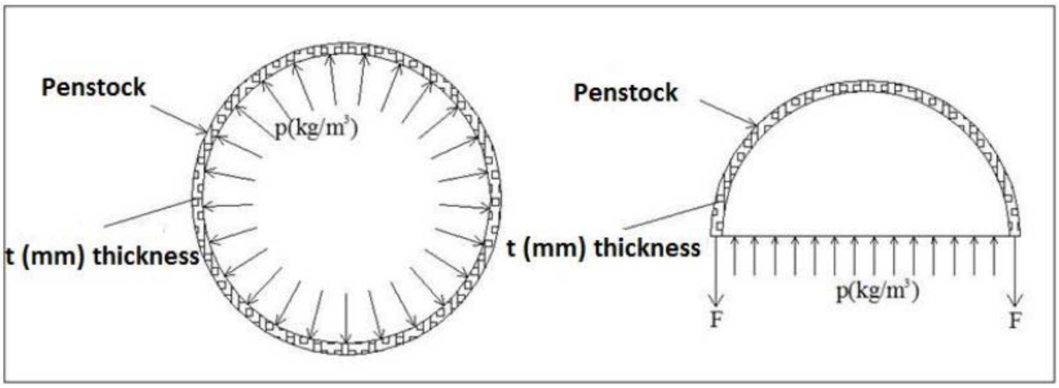


Figure 3.10 Penstock Cross-Section and the Forces and Pressures on it (Calamak, 2010)

The circumferential tensile force should be at least equal to or greater than total pressure occurred in the penstock (See equation 3.16) :

$$D(H_{sta} + H_{dyn})\rho_w \leq 2\,F \tag{3.20}$$

where D (m) is penstock diameter, H_{sta} (m) is static pressure, H_{dyn} (m) is the dynamic pressure, ρ (N/m^3) is the specific density of water and F (N/m) is the circumferential tensile force per unit length of penstock (ESHA, 2004).

The wall thickness wt (mm) is calculated by the following formula:

$$wt = \frac{F}{1000\sigma_s} \tag{3.21}$$

where σ_s (N/m^2) is lateral unit stress of steel and F (N/m) is the circumferencial tensile force per unit length of penstock. 2.0 mm of additional cover is added to the wall thickness for corrosion and safety.

To be on the safe side, the calculated wall thickness on analyzed point is rounded up to the closest commercially available thickness. Total pressure at the bottom parts of penstock is higher than the pressure on upper parts. Therefore, designing the penstock with variable wall thickness is more economic than penstock with single wall thickness. A tabular wall thickness calculation is prepared for the Alternative-formulation. The penstock is divided into a number of parts and wall thicknesses of each part is estimated taking static and dynamic pressures occurred for each section into consideration. An illustrative calculation of penstock wall thickness and weight is given in Table 3.3.

Table 3.3 The Penstock Wall Thickness and Weight Calculations for the Alternative-formulation

Part Number	(i) – Static Pressure in the Beginning of Pentstock (m)	Vertical Projection of the Penstock (m)	Length of the Parts (m)	Total Length of the Penstock (m)	(ii) – Dynamic Pressure Head (m) (Equation (3.19))	Total Hydraulic Pressure [(i)+(ii)] (m)	The Calculated Thickness (mm) (Equation (3.21))	Commercially Available Thickness (mm)	Weight of the Penstock Parts (kg) (Equation (3.24))	Total Weight of the Penstock (kg)
1	3	2.46	11.66	11.66	0.96	6.42	0.16	6	1910	2200
2	3	45.46	203.68	215.34	17.71	66.17	1.65	6	33330	38330
3	3	88.46	203.68	419.02	34.46	125.92	3.15	6	33330	38330
4	3	131.46	203.68	622.71	51.21	185.67	4.64	8	44520	51200
5	3	174.46	203.68	826.39	67.96	245.42	6.13	10	55760	64120
6	3	217.46	203.68	1030.07	84.71	305.17	7.62	10	55760	64120
7	3	260.46	203.68	1233.75	101.46	364.92	9.12	12	67030	77080
8	3	303.46	203.68	1437.43	118.21	424.67	10.61	14	78340	90090
9	3	346.46	203.68	1641.11	134.96	484.42	12.1	16	89690	103150
10	3	389.46	203.68	1844.79	151.71	544.17	13.59	16	89690	103150
11	3	432.46	203.68	2048.47	168.46	603.92	15.09	18	101080	116250

Table 3.3 The Penstock Wall Thickness and Weight Calculations for the Alternative-formulation (continued)

Part Number	(i) – Static Pressure in the Beginning of Pentstock (m)	Vertical Projection of the Penstock (m)	Length of the Parts (m)	Total Length of the Penstock (m)	(ii) – Dynamic Pressure Head (m) (Equation (3.19))	Total Hydraulic Pressure [(i)+(ii)] (m)	The Calculated Thickness (mm) (Equation (3.21))	Commercially Available Thickness (mm)	Weight of the Penstock Parts (kg) (Equation (3.24))	Total Weight of the Penstock (kg)
12	3	475.46	203.68	2252.15	185.21	663.67	16.58	20	112520	129390
13	3	518.46	203.68	2455.83	201.96	723.42	18.07	22	123990	142590
14	3	561.46	203.68	2659.51	218.71	783.17	19.56	22	123990	142590
15	3	604.46	203.68	2863.19	235.46	842.92	21.06	24	135500	155830
16	3	647.46	203.68	3066.87	252.21	902.67	22.55	26	1470600	169110
17	3	690.46	203.68	3270.55	268.96	962.42	24.04	28	1586500	182450
18	3	733.46	203.68	3474.24	285.71	1022.17	25.53	28	158650	182450
19	3	776.46	203.68	3677.92	302.46	1081.92	27.03	30	170280	195820
20	3	819.46	203.68	3881.6	319.21	1141.67	28.52	32	181960	209250
21	3	862.46	203.68	4085.28	335.96	1201.42	30.01	34	193670	222720
22	3	905.46	203.68	4288.96	352.71	1261.17	31.5	34	193670	222720
23	3	948.46	203.68	4492.64	369.46	1320.92	33	36	205420	236240
24	3	991.46	203.68	4696.32	386.21	1380.67	34.49	38	217220	249800
25	3	1034.46	203.68	4900	402.96	1440.42	35.98	38	217220	249800

Cross-section area is calculated after the wall thickness is estimated throughout the penstock with the following equation:

$$A_{crs} = A_o - A_i = \pi \frac{(D + 2wt)^{\,2}}{4} - \pi \frac{D^2}{4} \tag{3.22}$$

where A_{crs} (m^2) is the cross-sectional area of the penstock, D (m) is penstock diameter, wt (m) is the wall thickness, A_o (m^2) and A_i (m^2) are the outer and inner cross-sectional areas, respectively. The volume of the analysed penstock part can be calculated by multiplying the cross-section with the length, L of the analysed penstock part.

$$Volume\ of\ part = \ \forall_p = A_{crs}L \tag{3.23}$$

$$Weight\ of\ part = W_p = \gamma_{\text{ st}}\, \forall_p \tag{3.24}$$

where $\gamma_{\text{ st}}$ is the specific weight of steel (7.85 t/m^3). The overall weight of the penstock can be determined by the summation of calculated weights of all parts. Additional costs of penstock such as expansion joints, anchor and support block costs are taken into account by increasing the cost of overall weight of the penstock by%15 as suggested by Suis Company (2010).

Selection of the Penstock Diameter

Economic analysis is carried out for discharges identified for the Alternative-formulation. Best penstock diameter selection study with 2.2 m^3/s is given in Table 3.4. The change in loss energy income, material costs and total cost of the penstock with respect to diameter for the discharge 2.2 m^3/s are shown in Figure 3.11.

Table 3.4 Best Penstock Diameter Selection Study for the Alternative-formulation

Discharge (m^3/s)	Penstock Diameter (m)	Loss Energy Income (TL)	Material Costs (TL)	Total Cost (TL)	Flow Velocity in Penstock (m/s)
2.2	0.4	9,002,549	780,666	9,783,215	17.51
	0.5	2,899,064	895,179	3,794,243	11.20
	0.6	1,155,352	1,025,180	2,180,532	7.78
	0.7	533,196	1,184,507	1,717,703	5.72
	0.8	273,866	1,388,946	1,662,812	4.38
	0.9	152,599	1,583,825	1,736,424	3.46
	1	90,643	1,805,798	1,896,441	2.80
	1.1	56,687	2,053,403	2,110,090	2.31

As seen in Table 3.4, 0.8 m is selected as the best penstock alternative giving the minimum total cost among all alternatives for the discharge 2.2 m^3/s.

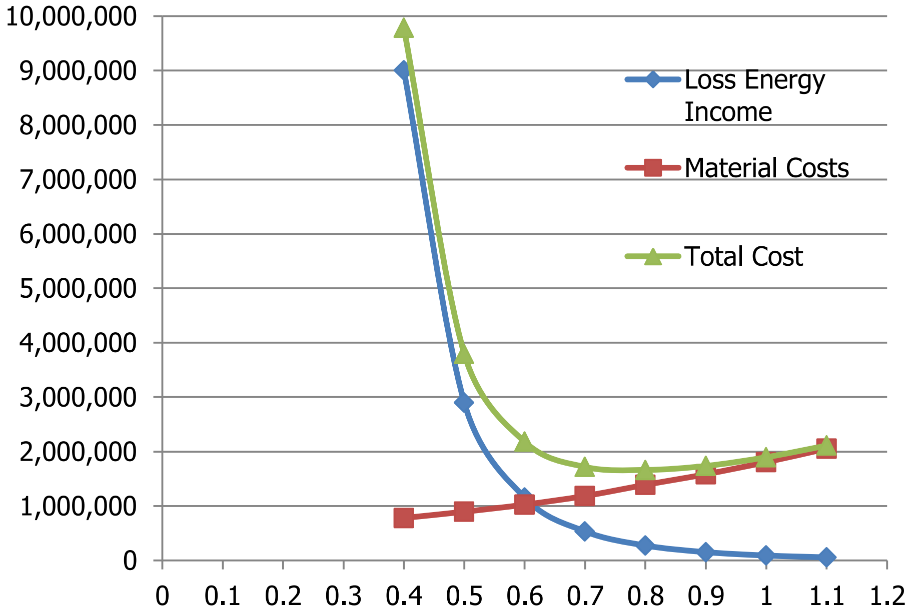


Figure 3.11 The Change in Loss Energy Income, Material Costs and Total Cost of the Penstock with respect to Diameter

3.3.3 Diversion Weir Cost

The spillway crest height is prescribed as 6 m for the Alternative-formulation. The length of the crest is estimated as 25 m using regional flood frequency analysis developed by Hidromark (2009). In diversion weir cost of the Alternative-formulation, the cost items of a diversion weir with a spillway length of 25 m and spillway crest height of 5 m estimated by Eser Company are used and they are given in Table 3.5. Some items related to drainage system, gates and steel appurtenances are not included.

Table 3.5 Diversion Weir Cost (Eser, 2009)

Item Number	Definition of Work	Amount	Unit	Unit Price (TL)	Price (TL)
14.006	Excavation of Hard Rocks by Explosives, Backfilling or Storing excavation material	792	m^3	20.83	16,497
14.014/1	Industrial Manufacturing Excavation by Using Explosives on every kind of Rocky Ground	3,168	m^3	36.55	115,790
15.024/2	Cost Raise Due To Difficulties Occurred Underwater Excavations on Every Kind of Rocky Soil By Machinery	3,960	m^3	1.24	4,910
16.012/1	Concrete with suggested amount of Aggregate with no steel	504	m^3	83.01	41,837
B-16.505	Concrete	2,016	m^3	95.71	192,951
B-16.501/A	Cement Supply	756,000	kg	0.1387	104,857
21.011	Smooth Faced Formwork	2,200	m^2	21.21	46,662
21.021	Inclined Faced Formwork	250	m^2	36.94	9,235
B-23.002	Reinforcement Concrete (ø 14 or bigger)	101,000	kg	1.435	144,629
D.18.503/1	PVC Sealing Gasket (A Type First Class)	416	m	42.51	17,684
07.006/32	Transportation of Cement	756,000	kg	0.0089	6,736
07.006/ekst.	Transportation of Reinforcement	101,000	kg	0.122	12,311
07.006/12	Transportation of Aggregate for Concrete	5,443,000	kg	0.00126	6,858
				Total Price (TL)	**720,959**

3.3.4 Settling Basin Cost

To estimate the cost of a settling basin, the grain size to be settled should be determined. Table 3.6 shows the grain size needs to be settled with respect to gross head of a hydropower plant. Since, the gross head of the Alternative-formulation is about 1000 m, the grain size is selected as 0.1 mm.

Table 3.6 Appropriate Grain Size to Settle with respect to Hydropower Gross Head (Ada Engineering Consultancy, 2010)

The Gross Head (m)	Grain Size (mm)
80-100	No settling required
100-200	0.6
200-300	0.5
300-500	0.3
500-1000	0.1

Geometry (width, height and length), concrete volume (m^3), weight of steel bars (kg), and excavation volume (m^3) of settling basin are determined from design charts by using design discharge and grain size (Bollaert, 2004). The charts are given in Appendix A. Diversion weir costs associated with the Alternative-formulation are given in Table 3.7.

Table 3.7 Settling Basin Cost for a Hydropower System with 1000 m Gross Head

Item Number	Definition of Work	Amount	Unit	Unit Price (TL)	Price (TL)
14.006	Excavation of Hard Rocks by Explosives, Backfilling or Storing excavation material	600	m^3	20.83	12,498
14.014/1	Industrial Manufacturing Excavation by Using Explosives on every kind of Rocky Ground	2,400	m^3	36.55	87,720
15.024/2	Cost Raise Due To Difficulties Occurred Underwater Excavations on Every Kind of Rocky Soil By Machinery	3,000	m^3	1.24	3,720
16.012/1	Concrete with suggested amount of Aggregate with no steel	440	m^3	83.01	36,524
B-16.505	Concrete	1,760	m^3	95.71	168,450
B-16.501/A	Cement Supply	660,000	kg	0.1387	91,542
21.011	Smooth Faced Formwork	2,200	m^2	21.21	46,662
21.021	Inclined Faced Formwork	500	m^2	36.94	18,470
B-23.002	Reinforcement Concrete (ø 14 or bigger)	88,000	kg	1.4348	126,263
D.18.503/1	PVC Sealing Gasget (A Type First Class)	300	m	0.0425	12,753
07.006/32	Transportation of Cement	660,000	kg	0.00891	5,881
07.006/ekst.	Transportation of Reinforcement	88,000	kg	0.1221	10,747
07.006/12	Transportation of Aggregate for Concrete	4,752,000	kg	0.00126	5,988
				Total Price (TL)	**627,218**

3.3.5 Glass-Fibre Reinforced Plastic (GRP) Pipe Cost

In the estimation of GRP pipe cost, the design table prepared by Suis Company (see Table 3.8) is used. It gives the unit price of GRP pipes including concrete, excavation, filling, formwork and transportation items for various compressive strengths. Since hydraulic loss in the transmission line is estimated to be around 10 m, pipes having compressive strength of 4 Atm (40 m) are preferred. The diameter giving a speed of 1–1.5 m/s is selected for each alternative discharge as suggested by Suis Company.

Table 3.8 Unit Price of GRP Pipe for Various Compressive Strengths (Suis, 2012)

Diameter (mm)	Prices per unit length (TL)			
	Compressive Strengths			
	4 ATM	6 ATM	10 ATM	16 ATM
400	145.2988	154.04	163.30	178.91
450	154.5432	168.15	182.96	204.21
500	175.7232	187.16	199.33	223.06
600	224.2801	241.51	260.06	292.20
700	278.3385	295.77	314.28	357.45
800	341.3758	359.10	377.75	429.45
900	393.9097	410.83	428.49	491.14
1000	455.1404	473.34	492.27	573.46
1200	615.1064	648.08	682.82	805.98
1300	656.0586	688.86	723.30	855.65
1400	826.7613	832.12	837.51	992.41
1500	947.6551	954.50	961.40	1,142.29
1600	1108.416	1,119.04	1,129.77	1,356.42
1700	1190.571	1,204.69	1,218.98	1,480.07
1800	1301.253	1,318.53	1,336.04	1,625.82
1900	1428.66	1,451.39	1,474.49	-
2000	1326.743	1,463.30	1,613.91	1,820.91

The GRP cost is calculated as 3,441,740 TL for the discharge 2.2 m^3/s.

3.3.6 Tunnel Cost

In a conversation on 15th May 2012, people from Suis Company stated that the most important factors affecting tunnel costs are the diameter and length of tunnel. For the ventilation of tunnel and the transportation of tunnel excavation material, minimum three-meter diameter is required. The tunnel with three-meter diameter transmits all the discharges identified for the determination of the best installed capacity in free flow condition. Hence, three meters is selected as the tunnel diameter of the Alternative-formulation. The cross-section of three-meter diameter tunnel used commonly in transmission lines of hydropower projects is given in Figure 3.12.

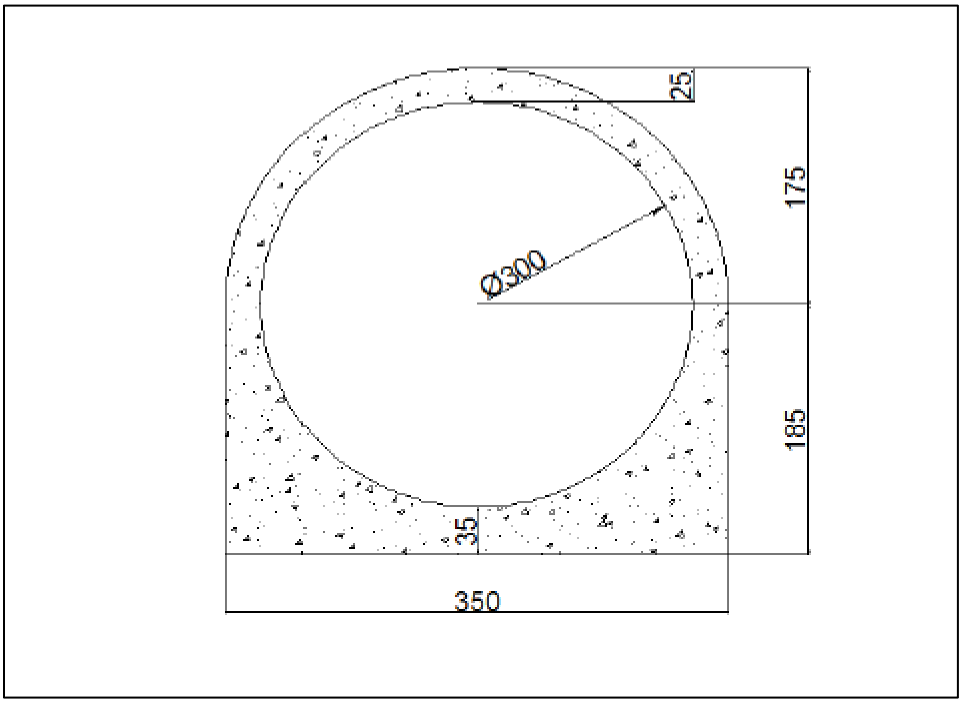


Figure 3.12 Tunnel Cross-Section (All dimensions are in cm) (Hidromark, 2009)

The costs associated with the excavation, concrete works and formwork are calculated using Hidromark tunnel cross-section dimensions. Rest of the tunnel costs are taken as 10% of calculated tunnel costs.

Volume of Excavation and Concrete

The area of excavation per unit length is calculated in the following equation in conformity with the cross section dimensions given in Figure 3.12.

$$3.50 \times 1.85 + \frac{\pi 3.25^2}{8} = 10.62\ m^2$$

Unforeseen exacvation situations are taken into consideration and the unit excavation area is increased by 10%.

The concrete area per unit length (m) can be calculated as:

$$10.62\ m^2 - \frac{\pi 3^2}{4} = 3.55\ m^2$$

Volume of excavation and concrete are calculated by multiplying corresponding areas with the length of channel. Cost of reinforcement is calculated by reinforcement to cement ratio (80 kg reinforcement to 300 kg cement for a cubic meter of concrete) (Hidromark, 2009)

Formwork Area

Formwork estimations of the tunnel only includes the inner region of the structure. This region is the circular area the flow occurs. The surface where formwork per unit length (m) is applied can be calculated as:

$$D_{tunnel}\pi = 3.00\pi = 9.42\ m^2/m$$

Total amount of formwork is calculated by multiplying this value with the length of the channel. The tunnel cost for the Alternative-formulation is given in Table 3.9.

Table 3.9 Tunnel Cost for the Alternative-formulation

Item Number	Definition of Work	Unit	Unit Price (TL)	Amount	Price (TL)
32.001	Tunnel Excavation	m^3	115.89	31860	3,692,255
B-16.538	Concrete	m^3	129.36	10650	1,377,684
B-16.501/A	Cement	kg	0.1339	3,195,000	427,842
B-23.002/1	Reinforcement	kg	2.2072	2,022,000	4,463,898
B-21.D/1	Formwork	m^2	0.0467	28260	1,320,025
32.006	Transportation of Excavation	kg	0.00844	31,860,000	268,898
B-07.D/1	Transportation of Cement	kg	0.02693	3,195,000	86,041
B-07.D/2	Transportation of Reinforcement	kg	0.04834	2,022,000	97,765
-	Other Costs (10%)	-	-	-	1,173,441
			Total Price		**12,907,850**

3.3.7 Conveyance Channel Cost

The geometry of conveyance channel is selected as rectangular due to project site topography. Manning equation is applied in the calculations and roughness coefficient is taken as 0.012. Channel designs for identified discharges include a freeboard of 15 cm (ESHA, 2004). The concrete volume (m^3), reinforcement (kg) and formwork (m^2) used for the conveyance channel and excavation (m^3) required for the channel are estimated by using design charts (Androodi, 2006). These charts are given in Appendix A. Conveyance channel costs for a discharge of 2.2 m^3/s are given in Table 3.10.

Table 3.10 Conveyance Channel Cost for a Discharge of 2.2 m^3/s

Item Number	Definition of Work	Amount	Unit	Unit Price (TL)	Price (TL)
15.001/1	Soil Excavation by Machinery	2020	m^3	1.66	3,353
15.006/1	Excavation of Loose Rock	16950	m^3	2.64	44,748
15.014/1	Excavation of Hard Rock by Explosives	1960	m^3	11.19	21,932
15.002	Opening Canal on Every Type of Soil	2560	m^3	1.76	4,506
21.011	Smooth Formwork	10820	m^2	24.06	260,329
16.022/1	Reinforced Concrete (Using Executively Suggested Aggregate)	2200	m^3	102.40	225,280
16.002/1	Concrete (Executively Suggeststed Aggregate and Gravier)	200	m^3	95.78	19,156
16.D/1-A	Cement Pouring	653,000	kg	0.1366	89,174
23.002	Reinforcement (equal or bigger than f14)	243,000	kg	2.2551	547,997
D.18.503/1	PVC Sealing Gasget (A type first class)	200	m	47.76	9,552
07.006/32	Transportation of Cement	653,000	kg	0.01045	6,824
07.006/ekst.	Transportation of Reinforcement	243,000	kg	0.1372	33,347
07.006/12	Transportation of Aggregate	25,307,000	kg	0.0015	37,454
07.006/12	Transportation of Excavation Material	20,000,000	kg	0.0015	29,600
		Total Price (TL)			**1,333,251**

3.3.8 Forebay Cost

Length/width criterion and velocity criterion are satisfied in the dimensioning of forebay. The ratio of length to the width of the forebay is taken from 2.5 to 3.0 (Erdem, 2006). The flow velocity in forebay is between 0.6 and 0.8 m/s (Yildiz, 1992). Forebay cost as a function of design discharge is given in Figure 3.13. The Forebay Cost is estimated as 185,146 TL for the discharge of 2.2 m^3/s using Figure 3.13.

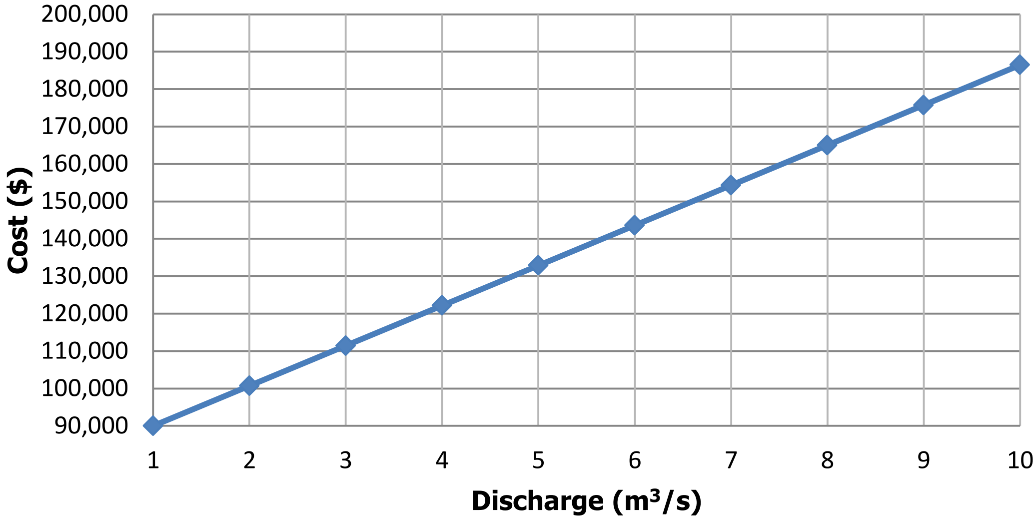


Figure 3.13 Forebay Cost as a Function of Discharge (EIEI, 2006)

3.3.9 Turbine, Transformer and Generator Costs

A unit cost of 320 $/kW is used (Hidroen, 2012) for the costs of turbine, transformer and the generator.

3.3.10 Power House Cost

In the determination of power house cost for the Alternative-formulation, the cost items of a power house with an installed capacity of 18 MW estimated by Suis Company are used and they are given in Table 3.11.

Table 3.11 Power House Cost (Suis, 2011)

Item Number	Definition of Work	Unit	Unit Price (TL)	Amount	Price (TL)
B-15.310	Excavation of Rock	m^3	11.46	3400	38,964
B-15.301	Soil Excavation	m^3	1.98	2270	4,495
B15.307	Backfilling with Excavation Material	m^3	1.16	5660	6,566
B-15.308	Excavation of Soft Rock	m^3	7.53	2270	17,093
B-16.576/1	Concrete	m^3	125.23	3020	378,195
B-16.576/1	2. Phase Concrete	m^3	125.23	450	56,354
B-16.501	Cement	kg	0.1402	1,950,000	273,312
B-15.344/1	Flushing of Aggregate	m^3	0.88	8120	7,146
B-23.002	Reinforcement	kg	2.2569	230,000	519,089
B-21.015/1	Smooth Formwork (F1)	m^2	22.71	2270	51,552
B-21.015	Smooth Formwork (F2)	m^2	32.44	2270	73,639
B-21.015/2	Smooth Formwork (F3)	m^2	42.18	1130	47,663
B-21.024/3	Inclined Formwork (F3)	m^2	55.15	380	20,957
B-23.255	Operation Gates and Their Embedded Parts	kg	14.88	4530	67,406
B-23.302	Service Gate Lifting Apparatus	kg	23.88	1210	28,895
B-18.501	PVC Waterstop (A Type)	kg	13.03	3020	39,351
Invoiced	Architectural, Medical and Lighting Equipments		25000.00	1	25,000
Invoiced	Globular Vane	number	20000.00	2	40,000
Invoiced	Pressure Dissipator Vane	number	120000.00	1	120,000
Invoiced	Mobile Crane	number	50000.00	1	50,000
Invoiced	Monoray Crane	number	20000.00	1	20,000
B-07.D/1	Transportation of Cement	kg	0.09284	1,950,000	181,038
B-07.D/2	Transportation of Reinforcement	kg	0.18759	240,000	45,022
B-07.D/4	Transportation of Crushed Stone	m^3	7.92	8680	68,746
B-07.D/3	Transportation of Soft Rock	m^3	2.78	7930	22,045
B-07.D/4	Transportation of Soil Excavation	m^3	2.78	5660	15,735
B-07.D/5	Transportation of Rock Excavation	m^3	2.78	3400	9,452
				Total Price (TL)	**2,227,712**

3.3.11 Unforeseen Costs

Unforeseen costs are assumed to be 10% of the costs of all Alternative-formulation component costs (Hidromark, 2010).

3.3.12 Project, Surveying and Control Costs

The project, surveying, and control costs are taken as 10% of the facility costs (Suis, 2011).

3.3.13 Operation and Maintenance Costs

Operation and maintenance costs are taken as 1% of the total annual investment cost (Suis, 2011).

3.4 Determination of Net Benefits

After the best penstock diameter is selected for each alternative discharge, daily streamflow operation study is carried out for each alternative. Firm and secondary energy generations are calculated and energy incomes are estimated for identified set of discharges by using Equations 3.13 to 3.15. The Alternative-formulation costs are converted into annualized costs by multiplying CRF identified for the Alternative-formulation.

Costs of some hydropower plant components such as costs of GRP pipe, conveyance channel, forebay, penstock, turbines, generator and transformer change considerably with the change in installed capacity. The costs of these items are taken into account in best installed capacity selection procedure. The costs of other HEPP components become a part in the calculation of total cost of the system for comparison of alternatives in Chapter 4.

3.4.1 Identification of the Best Installed Capacity

The installed capacities and net heads for identified set of discharges are given in Table 3.12. For each installed capacity, the difference between annual income and annual cost is calculated to obtain corresponding net benefits. Net benefits associated with identified set of installed capacities using single price method is given in Table 3.13. Economic analysis of EIEI and Hidromark-formulations are conducted by using EIEI, DSI and single price methods. The results of these analysis and comparison of three different formulations with respect to the single price method is presented in Chapter 4.

Table 3.12 Alternative Installed Capacities for the Alternative-formulation

Discharge (m^3/s)	Net Head (m)	Installed Capacity (MW)	Discharge (m^3/s)	Net Head (m)	Installed Capacity (MW)
0.80	985.37	6.88	1.70	944.50	14.03
0.90	974.85	7.66	1.80	934.87	14.69
1.00	963.17	8.41	1.90	924.72	15.34
1.10	950.34	9.13	2.00	914.05	15.96
1.20	984.78	10.32	2.10	963.02	17.66
1.30	977.77	11.10	2.20	957.01	18.38
1.40	970.24	11.86	2.30	950.73	19.09
1.50	962.19	12.60	2.40	944.19	19.78
1.60	953.60	13.32	2.50	937.39	20.46

Table 3.13 Net Benefits for the Alternative-formulation Using the Single Price Method

Discharge (m^3/s)	Annual Energy Generation (GWh)	Installed Capacity (MW)	Annual Energy Generation Income (TL)	Annual Conveyance Channel Cost (TL)	Annual GRP Pipe Cost (TL)	Annual Forebay Cost (TL)	Annual Penstock Cost (TL)	Annual Turbine, Transformer, Generator Costs (TL)	Net Benefit (TL)
0.8	33.97	6.88	4,463,658	81,362	139,693	15,182	822,278	211,354	3,193,789
0.9	36.12	7.66	4,746,168	82,290	153,110	15,367	870,379	235,315	3,389,707
1	38.02	8.41	4,995,828	83,219	166,527	15,552	918,480	258,355	3,553,695
1.1	39.69	9.13	5,215,266	84,150	179,943	15,737	966,581	280,474	3,688,380
1.2	42.53	10.32	5,588,442	85,082	193,360	15,922	1,014,683	317,030	3,962,364
1.3	44.06	11.10	5,789,484	86,016	206,777	16,107	1,062,784	340,992	4,076,808
1.4	45.45	11.86	5,972,130	86,951	220,194	16,293	1,110,885	364,339	4,173,469
1.5	46.63	12.60	6,127,182	87,888	233,610	16,478	1,158,986	387,072	4,243,148
1.6	47.75	13.32	6,274,350	88,826	247,027	16,663	1,207,087	409,190	4,305,556
1.7	48.75	14.02	6,405,750	89,765	260,444	16,848	1,255,188	430,694	4,352,810
1.8	49.62	14.69	6,520,068	90,707	273,861	17,033	1,303,290	451,277	4,383,901
1.9	50.42	15.34	6,625,188	91,649	287,277	17,218	1,351,391	471,245	4,406,408
2	51.15	15.96	6,721,110	92,593	300,694	17,404	1,399,492	490,291	4,420,636
2.1	53.59	17.66	7,041,726	93,539	314,111	17,589	1,447,593	542,515	4,626,379
2.2	**54.30**	**18.38**	**7,135,020**	**94,486**	**327,527**	**17,774**	**1,495,694**	**564,634**	**4,634,905**
2.3	54.94	19.09	7,219,116	95,434	340,944	17,959	1,543,796	586,445	4,634,538
2.4	55.46	19.78	7,287,444	96,384	354,361	18,144	1,591,897	607,642	4,619,016
2.5	55.95	20.46	7,351,830	97,336	367,778	18,329	1,639,998	628,531	4,599,858

The installed capacity giving the highest the net benefit is selected as best installed capacity. The best install capacity and corresponding net benefit for the Alternative-formulation are calculated as 18.38 MW correponding to a design discharge of 2.2 m^3/s and 4,634,905 TL, respectively. Annual income (TL), annual cost (TL) and net benefit (TL) calculated by the single price method versus alternative installed capacities is given in Figure 3.14.

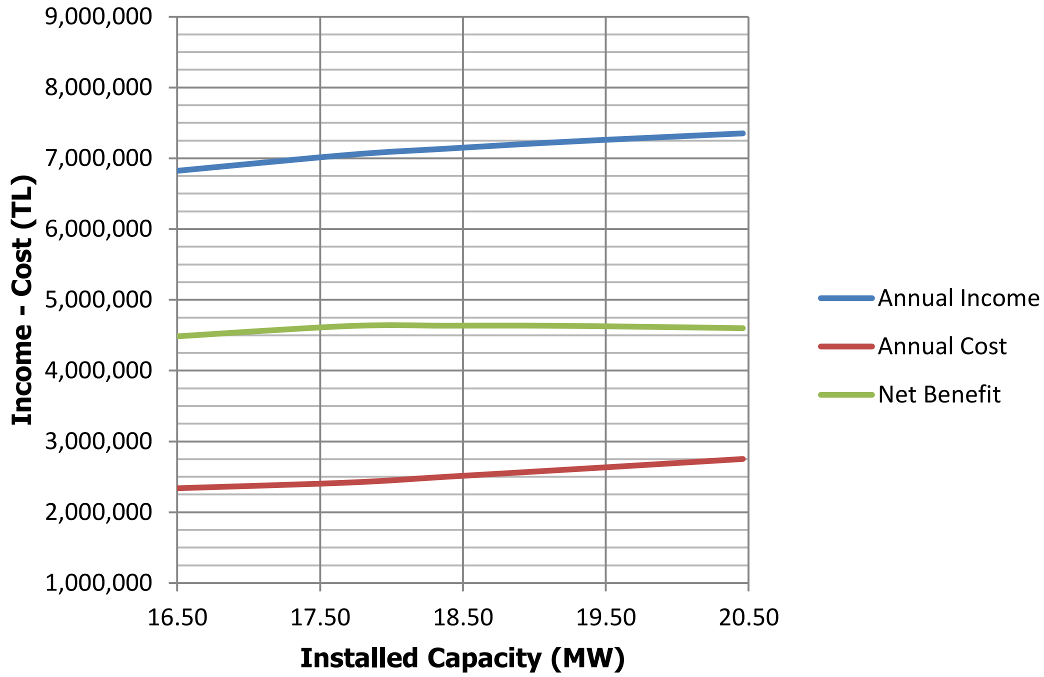


Figure 3.14 Annual Income, Annual Cost and Net Benefit in the Economic Analysis of the Alternative-Formulation (Single Price Method)

3.5 Impact of the Amount of Residual Water on the Annual Energy Generation

In the economic analysis of the Alternative-formulation, 10% of annual average streamflow is selected as the amount of residual water. In this section, an analysis is carried out to observe the decrease in average annual energy generation of the Alternative-formulation with respect to different amounts of residual water. Change in the average annual energy generation for different residual water amounts is given in Table 3.14. The graphical representation of the impact of residual water on energy generation is provided in Figure 3.15.

Table 3.14 The Change in Average Annual Energy Generation as a Function of Residual Water

Residual Water		Average Annual Energy Generation	
Percentage (%)	**Amount (m^3/s)**	**Amount (Gwh)**	**Change with respect to Generation with 10.00% Residual Water (%)**
2.50	0.03	58.80	8.30
5.00	0.06	57.90	6.60
10.00	0.11	54.30	-
12.50	0.14	52.21	-3.90
15.00	0.17	50.72	-6.60
20.00	0.22	46.95	-13.50

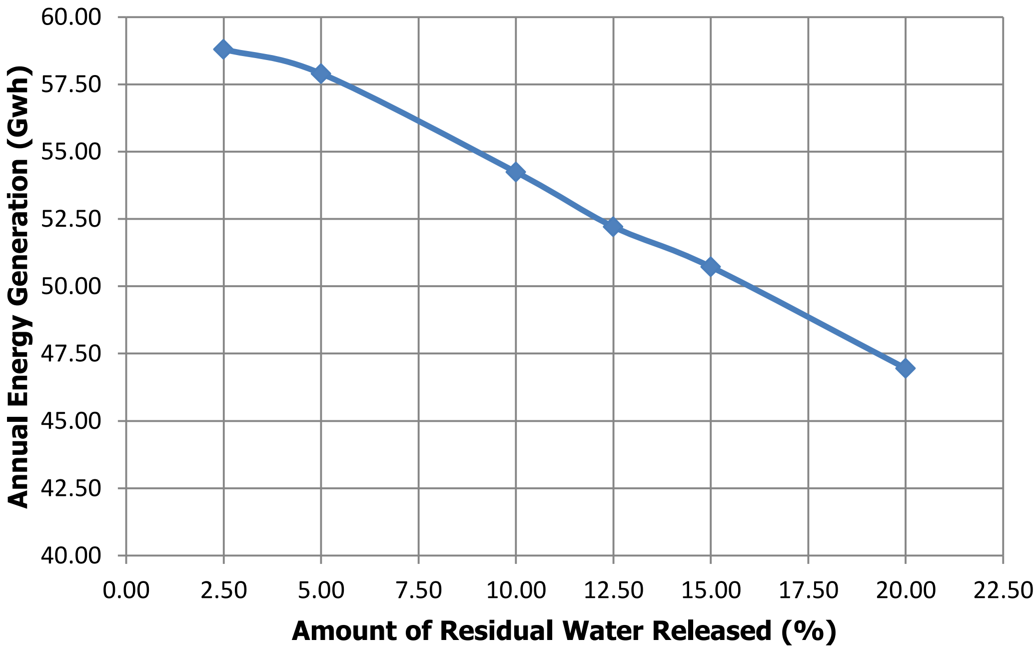


Figure 3.15 The Change in Annual Energy Generation as a Function of Residual Water

As can be seen from Table 3.15, if residual water is taken as 10% of the annual average streamflow the annual energy generation is 54.3 GWh. If the residual water is decreased to 5%, the annual energy generation increases about 6.6% and reaches to 57.9 GWh. On the other hand, if the residual water is increased to 15% of the annual average streamflow, the annual energy generation decreases again about 6.6% and becomes 50.7 GWh. This linear trend can be observed from Figure 3.15. A detailed environmental and ecological study needs to be carried out to select the best amount of residual water. The amount of residual water that is required to maintain a healthy aquatic ecosystem in the downstream of the hydropower plant requires a detailed study of the stream and all the aquatic organisms. Figure 3.15 provides impact of the residual water on average annual energy generation in terms of monetary values. These results need to be combined with environmental and ecological findings to make a healthy decision about the amount of necessary residual water.

3.6 Impact of Different Turbine Sizes on the Energy Generation

Energy generation calculations of the Alternative-formulation are conducted using two equal sized pelton turbines in the economic analysis. In this section, the impact of different two-turbine combinations on energy generation for the Alternative-formulation is investigated. Similar to equal sized turbine case, the following procedure is applied:

1) The set of identified alternative discharges are selected from the flow-duration curve constructed for the Alternative-formulation. The discharges from 0.8 m^3/s to 2.5 m^3/s with an increment of 0.1 m^3/s are used.
2) For each alternative discharge, run-of-river operation study for the Alternative-formulation with single turbine and varying size two turbine combinations (for a total of eight combinations) is run and average annual energy generations are estimated.

As for turbine combinations, first a single turbine is considered and design discharge (DD) is assumed to be exploited in this single turbine. For the two-turbine combinations, the ratio between Turbine 1 Design Discharge (TD1) and Turbine 2 Design Discharge (TD2) is taken as 1.0, 0.9, 0.8, 0.7, 0.6, 0.50, 0.25 and 0.1. For example, if the turbine design discharge ratio between TD1 and TD2 is selected as 0.8, a DD of 2.2 m^3/s is distributed depending on this ratio and TD1 and TD2 become 0.98 m^3/s and 1.22 m^3/s, respectively. The corresponding installed capacities of turbine 1 and turbine 2 for this case are going to be 8.17 MW and 10.21 MW, respectively.

The following operation modes are taken into account for each two turbine combination in the energy generation calculations (assuming that TD1 is equal or less than TD2) (Anagnostopoulos, 2007):

i. I (Inflow) < TM1 : Both turbines shut down. No energy production.
ii. TM1 < I < TM2 : Only T1 is in operation.
iii. TM2 < I < TD1 : Only T1 is in operation.
iv. TD1 < I < TD2 : Only T2 is in operation.
v. TD2 < I < TM1 + TD2 : Both turbines are in operation.
vi. TM1 + TD2 < I < TD1 + TM2 : Both turbines are in operation
vii. TD1 + TD2 < I : Both turbines are in operation at maximum flow rate.

The modes aforementioned above are programmed in Excel spreadsheet to carry out energy generation calcutions. The turbine installed capacities and average annual energy generations of identified alternative discharges with identified turbine combinations are given in Table 3.15.

Table 3.15 Turbine Installed Capacities and Average Annual Energy Generations of Atlernative Discharges with Identified Turbine Combinations

Alternative Discharges (m^3/s)	TD 1/TD 2	Turbine 1 Installed Capacity (MW)	Turbine 2 Installed Capacity (MW)	Average Annual Energy Generation (Gwh)
0.8	Single	6.88	-	33.43
	1	3.44	3.44	33.66
	0.9	3.26	3.62	33.64
	0.8	3.06	3.82	33.68
	0.7	2.83	4.05	33.70
	0.6	2.58	4.3	33.71
	0.5	2.29	4.59	33.74
	0.25	1.38	5.5	33.74
	0.10	0.63	6.25	33.64
0.9	Single	7.66	-	35.59
	1	3.83	3.83	35.84
	0.9	3.63	4.03	35.89
	0.8	3.41	4.25	35.92
	0.7	3.15	4.51	35.94
	0.6	2.87	4.79	35.96
	0.5	2.55	5.11	35.99
	0.25	1.53	6.13	35.99
	0.10	0.7	6.96	35.86
1	Single	8.41	-	37.47
	1	4.205	4.205	37.81
	0.9	3.98	4.43	37.84
	0.8	3.74	4.67	37.91
	0.7	3.46	4.95	37.92
	0.6	3.15	5.26	37.62
	0.5	2.8	5.61	37.95
	0.25	-6.29	14.7	37.97
	0.10	-8.3	16.71	37.80
1.1	Single	9.13	-	38.87
	1	4.565	4.565	39.51
	0.9	4.32	4.81	39.53
	0.8	4.06	5.07	39.57
	0.7	3.76	5.37	39.59
	0.6	3.42	5.71	39.65
	0.5	3.04	6.09	39.66
	0.25	1.83	7.3	39.71
	0.10	0.83	8.3	39.48

Table 3.15 Turbine Installed Capacities and Average Annual Energy Generations of Alternative Discharges with Identified Turbine Combinations (continued)

Alternative Discharges (m^3/s)	TD 1/TD 2	Turbine 1 Installed Capacity (MW)	Turbine 2 Installed Capacity (MW)	Average Annual Energy Generation (Gwh)
1.2	Single	10.32	-	41.68
	1	5.16	5.16	42.43
	0.9	4.89	5.43	42.46
	0.8	4.59	5.73	42.50
	0.7	4.25	6.07	42.53
	0.6	3.87	6.45	42.56
	0.5	3.44	6.88	42.63
	0.25	2.06	8.26	42.68
	0.10	0.94	9.38	42.46
1.3	Single	11.1	-	43.09
	1	5.55	5.55	44.02
	0.9	5.26	5.84	44.01
	0.8	4.93	6.17	44.08
	0.7	4.57	6.53	44.11
	0.6	4.16	6.94	44.16
	0.5	3.7	7.4	44.23
	0.25	2.22	8.88	44.30
	0.10	1.01	10.09	44.03
1.4	Single	11.86	-	44.08
	1	5.93	5.93	45.41
	0.9	5.62	6.24	45.45
	0.8	5.27	6.59	45.51
	0.7	4.88	6.98	45.56
	0.6	4.45	7.41	45.60
	0.5	3.95	7.91	45.64
	0.25	2.37	9.49	45.76
	0.10	1.08	10.78	45.46
1.5	Single	12.6	-	45.26
	1	6.3	6.3	46.70
	0.9	5.97	6.63	46.75
	0.8	5.6	7	46.78
	0.7	5.19	7.41	46.83
	0.6	4.71	7.89	46.91
	0.5	4.2	8.4	46.95
	0.25	2.52	10.08	47.08
	0.10	1.15	11.45	46.80

Table 3.15 Turbine Installed Capacities and Average Annual Energy Generations of Alternative Discharges with Identified Turbine Combinations (continued)

Alternative Discharges (m^3/s)	TD 1/TD 2	Turbine 1 Installed Capacity (MW)	Turbine 2 Installed Capacity (MW)	Average Annual Energy Generation (Gwh)
1.6	Single	13.32	-	46.31
	1	6.66	6.66	47.79
	0.9	6.31	7.01	47.84
	0.8	5.92	7.4	47.96
	0.7	5.48	7.84	48.00
	0.6	4.99	8.33	48.09
	0.5	4.44	8.88	48.02
	0.25	2.66	10.66	48.26
	0.10	1.21	12.11	47.96
1.7	Single	14.02	-	47.16
	1	7.01	7.01	48.82
	0.9	6.64	7.38	48.86
	0.8	6.23	7.79	48.94
	0.7	5.77	8.25	49.08
	0.6	5.26	8.76	49.13
	0.5	4.67	9.35	49.21
	0.25	2.8	11.22	49.33
	0.10	1.27	12.75	49.00
1.8	Single	14.69	-	47.88
	1	7.345	7.345	49.71
	0.9	6.96	7.73	49.82
	0.8	6.53	8.16	49.90
	0.7	6.05	8.64	50.03
	0.6	5.51	9.18	50.09
	0.5	4.9	9.79	50.15
	0.25	2.94	11.75	50.30
	0.10	1.34	13.35	49.93
1.9	Single	15.34	-	48.48
	1	7.67	7.67	50.53
	0.9	7.27	8.07	50.60
	0.8	6.82	8.52	50.77
	0.7	6.32	9.02	50.83
	0.6	5.75	9.59	50.94
	0.5	5.11	10.23	50.99
	0.25	2.62	12.72	51.18
	0.10	1.39	13.95	50.77

Table 3.15 Turbine Installed Capacities and Average Annual Energy Generations of Alternative Discharges with Identified Turbine Combinations (continued)

Alternative Discharges (m^3/s)	TD 1/TD 2	Turbine 1 Installed Capacity (MW)	Turbine 2 Installed Capacity (MW)	Average Annual Energy Generation (Gwh)
2	Single	15.96	-	49.02
	1	7.98	7.98	51.26
	0.9	7.56	8.4	51.35
	0.8	7.29	8.67	51.47
	0.7	6.57	9.39	51.57
	0.6	5.98	9.98	51.65
	0.5	5.32	10.64	55.10
	0.25	3.19	12.77	55.32
	0.10	1.45	14.51	51.56
2.1	Single	17.66	-	51.50
	1	8.83	8.83	53.60
	0.9	8.36	9.3	53.89
	0.8	7.85	9.81	53.99
	0.7	7.27	10.39	54.14
	0.6	6.62	11.04	54.21
	0.5	5.89	11.77	54.34
	0.25	3.53	14.13	54.55
	0.10	1.61	16.05	54.15
2.2	Single	18.38	-	49.69
	1	9.19	9.19	54.30
	0.9	8.71	9.67	54.39
	0.8	8.17	10.21	54.73
	0.7	7.57	10.81	54.87
	0.6	6.89	11.49	54.96
	0.5	6.13	12.25	55.10
	0.25	3.68	14.7	55.32
	0.10	1.67	16.71	54.90
2.3	Single	19.09	-	52.34
	1	9.545	9.545	54.87
	0.9	9.04	10.05	55.06
	0.8	8.48	10.61	55.35
	0.7	7.86	11.23	55.48
	0.6	7.16	11.93	55.64
	0.5	6.36	12.73	55.72
	0.25	3.82	15.27	55.97
	0.10	1.74	17.35	55.58

Table 3.15 Turbine Installed Capacities and Average Annual Energy Generations of Alternative Discharges with Identified Turbine Combinations (continued)

Alternative Discharges (m^3/s)	TD 1/TD 2	Turbine 1 Installed Capacity (MW)	Turbine 2 Installed Capacity (MW)	Average Annual Energy Generation (Gwh)
2.4	Single	19.78	-	52.72
	1	9.89	9.89	55.47
	0.9	9.37	10.41	55.63
	0.8	8.79	10.99	55.76
	0.7	8.14	11.64	56.08
	0.6	7.42	12.36	56.21
	0.5	6.59	13.19	56.35
	0.25	3.96	15.82	56.61
	0.10	1.8	17.98	56.25
2.5	Single	20.46	-	52.96
	1	10.23	10.23	55.97
	0.9	9.69	10.77	56.01
	0.8	9.09	11.37	56.30
	0.7	8.42	12.04	56.61
	0.6	7.67	12.79	56.77
	0.5	6.82	13.64	56.93
	0.25	4.09	16.37	57.20
	0.10	1.86	18.6	56.83

The results show that the minimum energy generations are obtained for the single turbine option for each alternative discharge. As the TD1/TD2 ratio decreases from 1.00 to 0.25, the average annual energy generation increases but it starts to decrease when the ratio becomes smaller than 0.25. Among the alternative discharges, the design discharge 2.5 m^3/s with turbine ratio of 0.25 has the highest annual average energy generation as expected. The change in average annual energy generation for the discharge of 2.5 m^3/s with respect to turbine combinations is shown in Figure 3.16. The turbine combination with the ratio of 0.25 has the maximum average annual energy generation for all alternative discharges. Instead of using equal sized two turbine combination, the combination TD 1/TD 2 equals to 0.25 can be used in the economic analysis. However, it should be remembered that as smaller size of turbine is selected, the capital and maintenance cost per kilowatt increases (see Section 3.2.1.1). Therefore, a compherensive economic analysis including varying size turbine related costs could be carried out to identify best installed capacity in terms of economic feasibility.

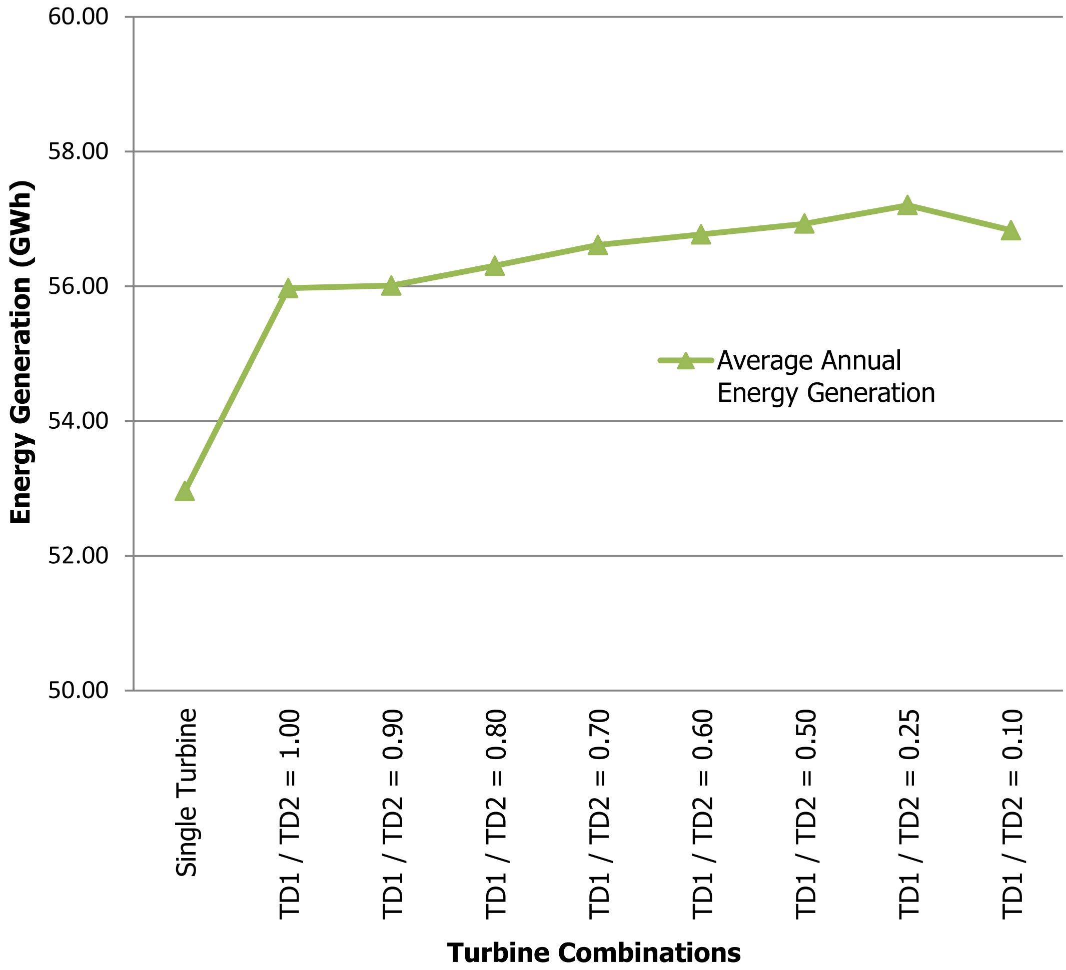


Figure 3.16 The Change in Average Annual Energy Generation of the Discharge 2.5 m^3/s with respect to Different Turbine Combinations

CHAPTER 4

COMPARISON OF DIFFERENT FORMULATIONS

In this chapter, the comparison of different formulations developed for Balkusan basin is presented. In this study, EIEI and Hidromark–formulations are operated using the water supply study developed for the Alternative-formulation. In addition, the Alternative-formulation is operated using the water supply study developed for EIEI and Hidromark-formulations. Water supply studies developed for the Alternative-formulation and for EIEI and Hidromark-formulations are explained in Section 4.3. Economic analysis of EIEI and Hidromark-formulations are conducted by using EIEI, DSI and single price methods while only the single price method is used for the Alternative-formulation. In the estimation of firm and secondary energy generations, the reservoir operation study (ROS) developed by Mümtaz Ak (2011) is used.

Hidromark Company calculated energy income of their formulation by using EIEI, DSI and single price (7.5 cent/kWh) methods (Hidromark, 2009). On the other side, EIEI calculated energy income of their formulation using only the single price (8 cent/kWh) method (EIEI, 1999).

The energy income methods are explained in Chapter 3. Among these methods, DSI and EIEI method are not applicable for the Alternative-formulation. The main reason for this is that "Peak Power" benefits considered in EIEI and DSI methods is only valid for storage-type hydroelectric power plants. "Peak Power" term stands for dependable capacity in international hydropower glossary (Pekcagliyan, 2003). That is, it represents the load carrying capacity of a hydropower station under specified conditions. Additionally, the unit prices assigned for firm and secondary energy generations prevent performance of a realistic economic analysis for a run-of-river hydropower plant. Higher unit prices are assigned for the firm energy generation compared to those assigned to the secondary energy generation (see Table 3.2). This situation favors firm energy generation. Although storage type hydropower plants have the ability of maximizing firm energy production by using their reservoirs, run-of-river plants can be operated only with the available streamflow. Due to these reasons it is not reasonable to compare run-of-river projects with storage type projects using EIEI and DSI methods. Thus, economical comparison of the Alternative formulation with the other two formulations is conducted with respect to the results obtained by the single price method.

4.1 Economic Analysis of the Alternative-Formulation

The summary of benefit and costs obtained in Chapter 3 for the Alternative-formulation are given in Table 4.1. In the determination of the net benefit of the project, the items which do not vary considerably with the installed capacity are also included.

Table 4.1 Economic Analysis Results of the Alternative-formulation

	Item	Cost and Benefit (TL)
	Diversion Weir	720,960.00
	Settling Basin	627,220.00
	Transmission Line	17,303,819.00
	Forebay	185,146.00
	Penstock	15,580,146.00
	Power House	2,449,315.00
	Turbine, Transformer, Generator (320 $/kW) (18.42 MW)	10,609,920.00
	Unforeseen Costs	4,747,652.60
	Facility Cost	52,224,178.60
	Project, Surveying and Controlling	5,222,417.86
	Operation and Maintenance	522,241.79
	Investment Cost	57,446,596.46
	Annual Investment Cost	5,514,873.26
SP Method	**Total Annual Cost**	**6,037,115.05**
	Annual Energy Income	**7,135,020.00**
	Annual Net Benefit of the Project	**1,097,904.95**
	Benefit/Cost Ratio	**1.18**

As can be seen from Table 4.1, annual energy income is higher than the annual cost and the annual net benefit of the Alternative-formulation is calculated as 1,097,905 TL. This indicates that the Alternative-formulation proposed in this study is a feasible alternative.

One other economic analysis method which has been widely used in the evaluation of water resources projects is the benefit-cost ratio method. It is defined as the ratio of equivalent worth (annual worth, present worth or future worth) of benefits to costs (Karamouz, 2003). When the ratio is greater than 1.0, the project is considered to be economically feasible (ASCE, 1989). The benefit-cost ratio of the Alternative-formulation is calculated as 1.18 which again indicates that it is economically feasible.

4.2 Benefit-Cost Analyses of the EIEI and Hidromark-Formulations

Annual energy incomes of EIEI and Hidromark-formulations are calculated in this section using water supply study developed for the Alternative-formulation in Chapter 3. The drainage area for these two formulations is 259 km^2. Annual average streamflow is calculated as 1.08 m^3/s. The residual water is selected as %10 of the average value of last ten years streamflow data which is 0.13 m^3/s. Table 4.2 and Table 4.3 show the economic analyses results of EIEI and Hidromark-formulations, respectively.

Table 4.2 Economic Analysis Results of the EIEI-formulation

	Item	Cost and Benefit (TL)
	Dam Body	16,527,776.00
	Spillway	6,291,862.00
	Derivation Tunnel	1,783,245.00
	Water Intake Structure	317,995.00
	Kumdan Creek Diversion Weir	520,013.44
	Surge Tank	1,546,261.00
	Transmission Line (D = 1.50 m)	3,117,323.00
	Energy Tunnel	46,860,384.00
	Penstock	35,876,794.00
	Power House	4,613,460.00
	Turbine, Transformer, Generator (325 $/kW) (38.4 MW)	12,480,000.00
	Road Relocation	1,158,000.00
	Expropriation	4,320,000.00
	Unforeseen Costs	12,993,511.34
	Facility Cost	142,928,624.78
	Project, Surveying and Controlling	14,292,862.48
	Operation and Maintenance	1,429,286.25
	Investment Cost	162,699,487.26
	Annual Investment Cost	15,619,150.78
EIEI Method	**Total Annual Cost**	**17,048,437.03**
	Annual Energy Income	**11,506,346.40**
	Annual Net Benefit of the Project	**-5,542,090.63**
DSI Method	**Total Annual Cost**	**17,048,437.03**
	Annual Energy Income	**10,078,993.00**
	Annual Net Benefit of the Project	**-6,969,444.03**
SP Method	**Total Annual Cost**	**17,048,437.03**
	Annual Energy Income	**9,986,400.00**
	Annual Net Benefit of the Project	**-7,062,037.03**
EIEI Method	**Benefit/Cost Ratio**	**0.67**
DSI Method	**Benefit/Cost Ratio**	**0.59**
SP Method	**Benefit/Cost Ratio**	**0.59**

Table 4.3 Economic Analysis Results of the Hidromark-formulation

Item	Cost and Benefit (TL)
HEPP 1	
Derivation Conduit and Bottom Outlet	691,984.66
Dam Body and Cofferdams	9,221,664.97
Spillway	746,875.41
Water Intake Structure	147,819.80
Injection Works	657,732.82
Service Road	968,737.18
Kumdan Creek Weir and Settling Basin	520,013.44
Kumdan Creek Derivation	2,302,130.50
GRP Pipe	10,824,745.80
Energy Tunnel	18,265,036.45
Surge Tank and Vane Room	1,019,204.48
Penstock	2,707,345.66
Power House	4,122,719.12
Turbine, Transformer, Generator (13 MW)	4,225,000.00
Road Relocation	965,000.00
Expropriation	2,707,200.00
Unforeseen Costs	5,642,101.03
Facility Cost	62,063,111.32
Project, Surveying and Controlling	6,206,311.13
Operation and Maintenance Costs	620,631.11
Investment Cost	71,941,622.45
Annual Investment Cost	6,906,395.76
Total Annual Cost (I)	**7,527,026.87**
HEPP 2	
Gökçeseki Weir and Settling Basin	908,297.28
Transmission Tunnel	8,639,217.96
Conveyance Channel	4,234,561.59
Forebay	521,850.55
Penstock	6,665,353.95
Power House	6,361,937.82
Turbine, Transformer, Generator (25 MW)	8,125,000.00
Unforeseen Costs	3,545,621.92
Facility Cost	39,001,841.07
Project, Surveying and Controlling	3,900,184.11
Operation and Maintenance	390,018.41
Investment Cost	42,902,025.17
Annual Investment Cost	4,118,594.42
Total Annual Cost (II)	**4,508,612.83**

Table 4.3 Economic Analysis Results of the Hidromark-formulation (Continued)

EIEI Method	Total Annual Cost (I+II)	12,036,539.70
	Annual Energy Income	11,012,670.00
	Annual Net Benefit of the Project	-1,023,869.70
DSI Method	Total Annual Cost (I+II)	12,036,539.70
	Annual Energy Income	10,061,442.00
	Annual Net Benefit of the Project	-1,975,097.70
SP Method	Total Annual Cost (I+II)	12,036,539.70
	Annual Energy Income	10,402,938.00
	Annual Net Benefit of the Project	-1,633,601.70
EIEI Method	Benefit/Cost Ratio	0.91
DSI Method	Benefit/Cost Ratio	0.84
SP Method	Benefit/Cost Ratio	0.86

As can be seen from Tables 4.2 and 4.3, the annual net benefits calculated by using EIEI, DSI and single price methods all produce negative net benefits for both EIEI and Hidromark-formulations. In terms of the benefit-cost ratios again both formulations are identified as infeasible with all three energy income estimation methods. However, the reader should remember that the economic analysis provided in Tables 4.2 and 4.3 are conducted by using the water supply study conducted for the Alternative-formulation. According to the water supply study conducted for the Alternative-formulation the annual average streamflow was identified as 1.11 m^3/s as explained in Chapter 3 while in the feasibility reports prepared by EIEI and Hidromark the annual average streamflow is estimated as 1.65 m^3/s. This issue is further explained in the following section.

4.3 Comparison of Water Supply Studies

The annual average streamflow (i.e., 1.65 m^3/s) estimated in water supply studies conducted by Hidromark and EIEI are quite different from that is estimated in this study (i.e., 1.11 m^3/s). The main reason for this is the water supply study used by Hidromark and EIEI are based on streamflow measurements for years 1966 to 2006, while this study utilizes years 1986 to 2009. The years from 1966 to 1985 are very wet years compared to the years 1986 to 2009 (EIEI, 2007). Besides, the average of observed streamflow data of EIEI from 2007 to 2009 which are used in this study are less than the average flow data observed at the same gauging station between 2000 and 2009. It is believed that recent streamflow data used in this study (i.e., 1986-2009) is more representative of the current situation at the project site since it better includes the climate change impacts.

Another important point is the formerly estimated amounts for residual water flow. EIEI and Hidromark estimate the residual water as 0.03–0.04 m^3/s which is far less than the 10% of average of last ten years streamflow (0.13 m^3/s). This results in an artificially higher amount of water availability for electricity generation for EIEI and Hidromark-formulations. It is unlikely that such low residual water flow values will be approved by the Ministry of Forest and Environment. Table 4.4 gives a summary of economic analysis results for all three formulations.

Table 4.4 Summary of Economic Feasibility of Alternatives with Benefit-Cost Ratio Method

Energy Income Method	EIEI Formulation (38.4 MW)		Hidromark Formulation (38 MW)		Alternative-Formulation (18.38 MW)	
	Alternative-formulation Water Supply Study	Formerly Developed Water Supply Study	Alternative-formulation Water Supply Study	Formerly Developed Water Supply Study	Alternative-formulation Water Supply Study	Formerly Developed Water Supply Study
DSI	0.59	-	0.84	1.09	-	-
EIEI	0.67	-	0.91	1.23	-	-
Single Price	0.59	1.26	0.86	1.28	1.18	1.62

As can be seen from Table 4.4, although with respect to the formerly conducted water supply study by EIEI and Hidromark, EIEI and Hidromark-formulations are evaluated as economically feasible alternatives, when more recent streamflow data is used these two formulations are identified as economically infeasible. The reason for this infeasibility is that structural components of EIEI and Hidromark-formulations were designed using formerly conducted water supply study. However, they become unnecessarily large and accordingly expensive when operated with the water supply study conducted for the Alternative-formulation. On the other hand, the Alternative-formulation has a benefit cost ratio of 1.18 and is economically feasible. Thus, it can be concluded that the Alternative-formulation presents a more beneficial solution for utilization of the hydropower potential at Balkusan Creek when streamflow measurements of 1986 to 2009 is used. In order to make a more conclusive decision a detailed analysis on the impact of climate change in water resources of the region has to be conducted.

CHAPTER 5

CONCLUSION

Hydropower is the most commonly used form of renewable energy in the world. Hydropower creates relatively low pollution and greenhouse gas emissions, and provides reliable energy production with low maintenance and operating costs. Turkey has high hydropower potential and only about half of this potential is in operation and under construction. One such hydropower plant which has recently been put in operation is Balkusan HEPP. Formerly, two formulations were developed for this project: the EIEI-formulation and the Hidromark-formulation. The EIEI-formulation consists of a rock-fill dam, a diversion weir and a long transmission line and a long penstock. The Hidromark-formulation consists of two separate systems: a roller compacted concrete dam and a diversion weir for the first system and a run-of-river plant for the second system.

In this study, economically a more beneficial alternative to these formulations is investigated and named as the Alternative-formulation. A single run-of-river plant is suggested as the Alternative-formulation to avoid high investment and expropriation costs of a dam and to save fertile lands which have been submerged by the reservoir of the existing dam. Energy generation is strictly a function of inflow and the term "peak power benefit" is inapplicable for run-of-river plants since they have no storage. An economic analysis is conducted for the Alternative-formulation with two equal sized turbines to compare it with EIEI and Hidromark-formulations. A flow-duration curve is developed based on daily streamflow data and the flow-duration curve method is used for the estimation of Alternative-formulation energy generation. It must be noted that this method uses a statistical approach and provides amount of energy generation corresponding to different percent of times specified discharges are equaled or exceeded. Real-time energy generations cannot be predicted using the flow duration curve method.

One of the main drawbacks of the Alternative-formulation is the sedimentation problem. Various operational problems may occur due to sedimentation in run-of-river plants. Regular monitoring the amount of sediment carried by the inflow and proper operation of sluiceway and flushing gates are necessary in order to control sedimentation problem in run-of-river plants. The possible repair and maintenance costs that may be associated with sedimentation related problems are not included into the economic analysis carried out in this study.

Further analysis is conducted to investigate the effect of varying size two-turbine combinations on energy generation for the Alternative-formulation. The analysis shows that as the difference between the sizes of turbines increases, the amount of available water exploited by turbines and correspondingly energy generation increases. On the other hand, when a smaller sized turbine is selected, the capital and maintenance costs per kilowatt increase. Therefore, an additional economic analysis should be conducted by considering capital and maintenance costs of various sized turbines while identifying the optimum turbine combination for run-of-river plants.

Another analysis is carried out by releasing different amounts of residual water to show its impact on annual energy generation. The results show that the amount of residual water directly affects the amount of annual energy generation. This study only evaluated impact of residual water in terms of energy generation however the necessary amount of residual water should be decided by taking environmental and ecological downstream requirements into consideration for each hydropower project.

In the former feasibility analysis both EIEI and Hidromark-formulations were identified as feasible. In this study, EIEI and Hidromark-formulations are found to be infeasible. The main reason for this was that in the former analysis streamflow measurements of 1966 to 2007 were utilized while the current water supply study is based on streamflow data of 1986 to 2009. These two year ranges result in two different annual average streamflow estimates: (i) when 1966-2006 data is used annual average streamflow is calculated as 1.65 m^3/s, (ii) when 1986-2009 data is used annual average streamflow is calculated as 1.11 m^3/s. The difference between these two estimates may be due to climate change and considerably effects economic feasibility of hydropower plant formulations. However, it should be mentioned that associated structures for EIEI and Hidromark-formulations are not re-dimensioned with respect to 1986-2009 streamflow data; original designs are used with the current water supply study (i.e. streamflow data between 1986 and 2009) results. On the other hand current water supply study results are used for the Alternative-formulation; however, this time all the associated structures of the Alternative-structure are designed with respect to the current water supply study. The Alternative-formulation has a benefit cost ratio higher than 1.0 and is found to be economically feasible.

The feasibility of three different formulations is only investigated in terms of monetary terms in this study. A comprehensive evaluation should include environmental and social impacts associated with the proposed hydropower plant. This is a challenging task because it is very hard to quantify associated social and environmental impacts of various activities associated with dam constructions such as relocation of people. This study only intends to compare three different alternative formulations for Balkusan HEPP in terms of monetary values.

As final words, each hydropower project is unique when hydrological, geological, and topographical conditions of the project area and temporal water availability are considered. Since Turkey has limited coal, petroleum and natural gas reserves, utilization of available hydropower in the best manner is vital for sustainable development of the country and requires case specific evaluations.

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APPENDIX A

DESIGN CHARTS

A1. Design Charts for Settling Basin

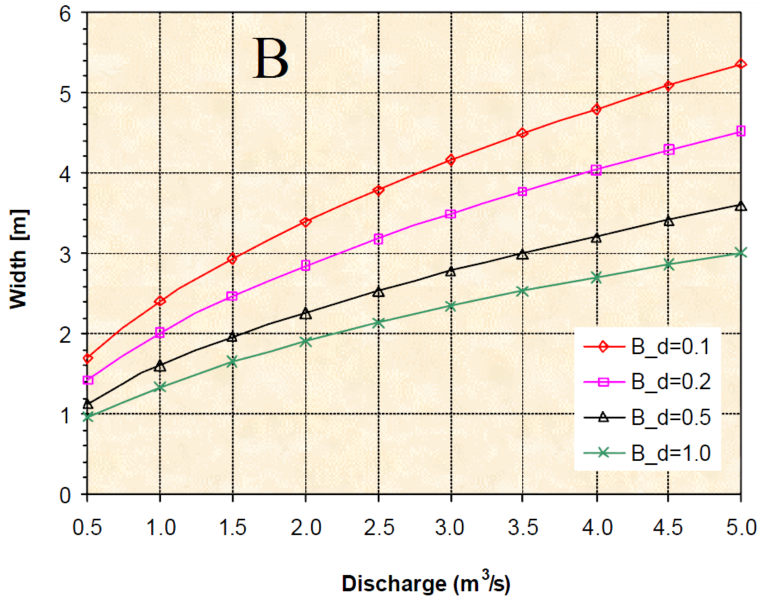

Figure A1-1 Width of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

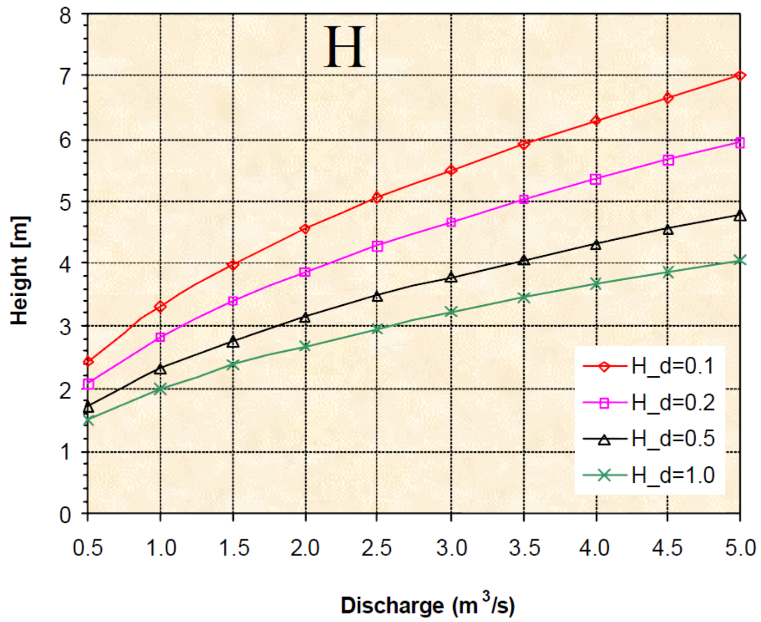

Figure A1-2 Height of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

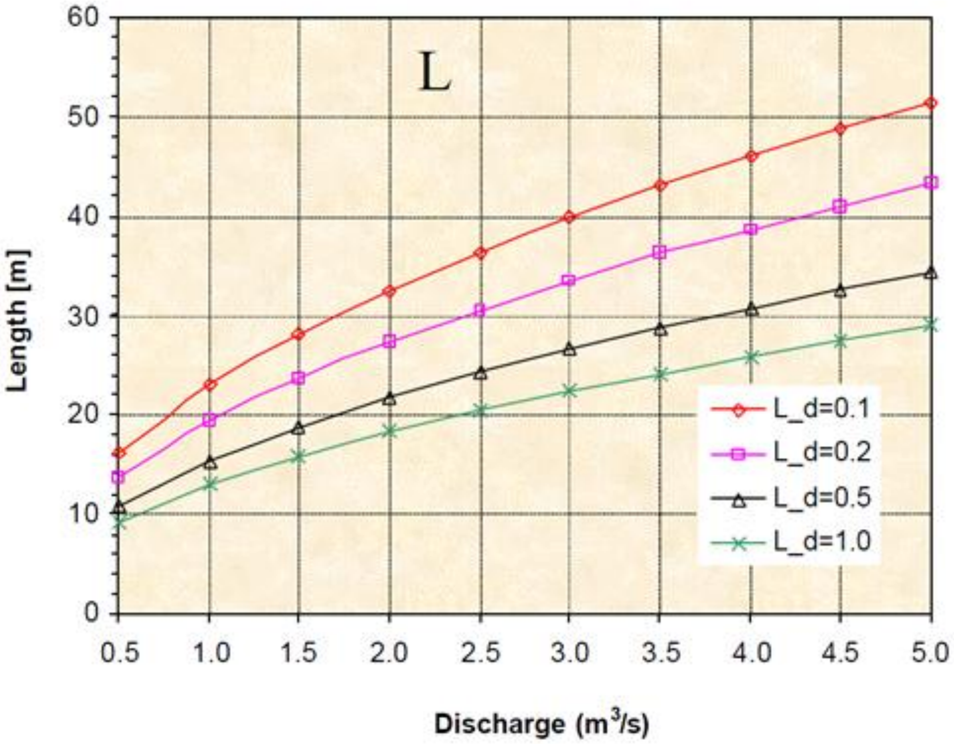

Figure A1-3 Length of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

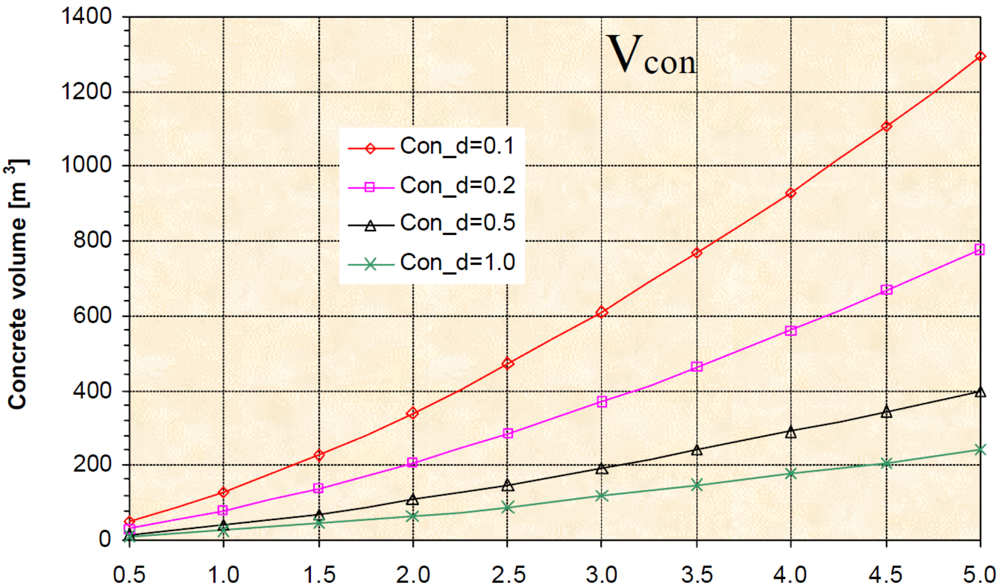

Figure A1-4 Concrete Volume of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

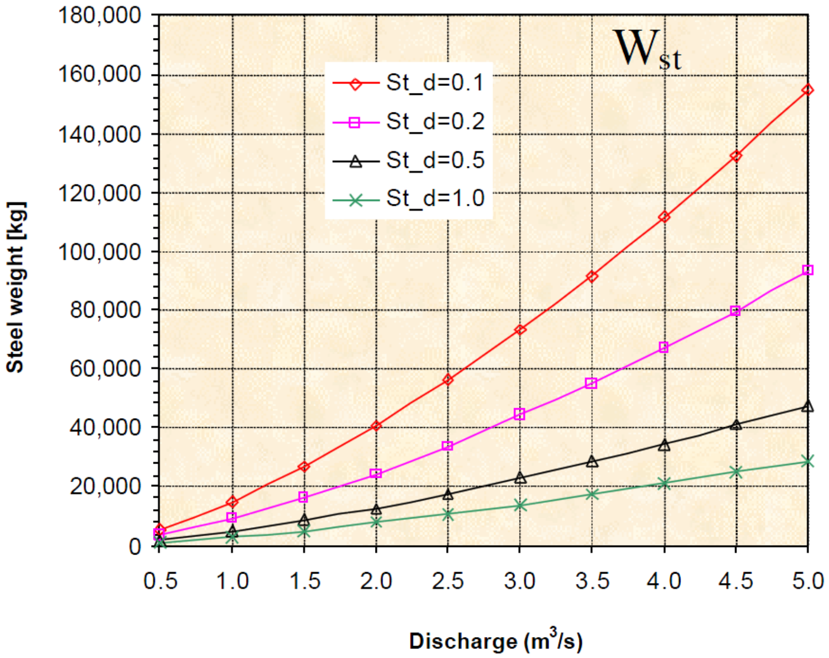

Figure A1-5 Steel Weight of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

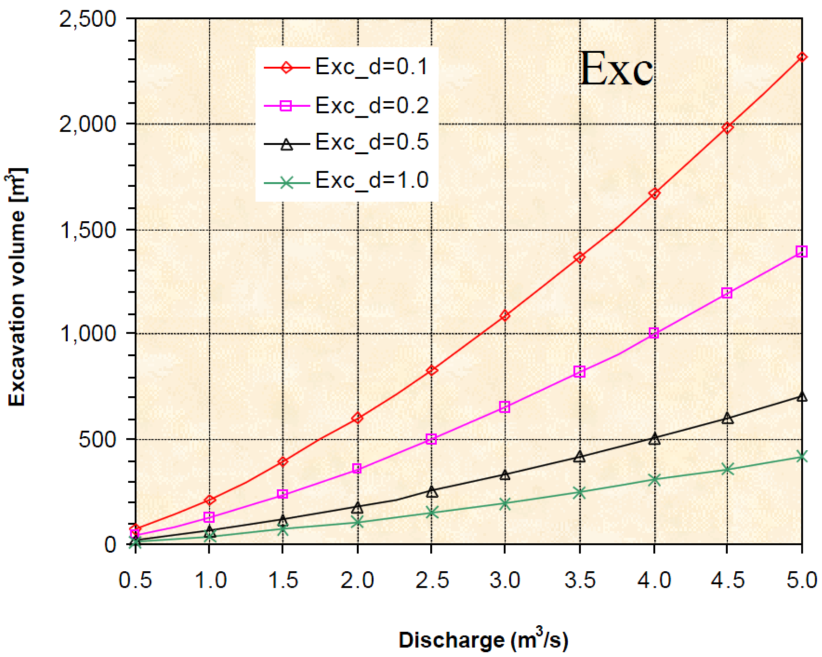

Figure A1-6 Excavation Volume of Settling Basin as a Function of Discharge and Design Grain Size as 0.1, 0.2, 0.5 and 1.0 mm

A2. Design Charts for Conveyance Channel

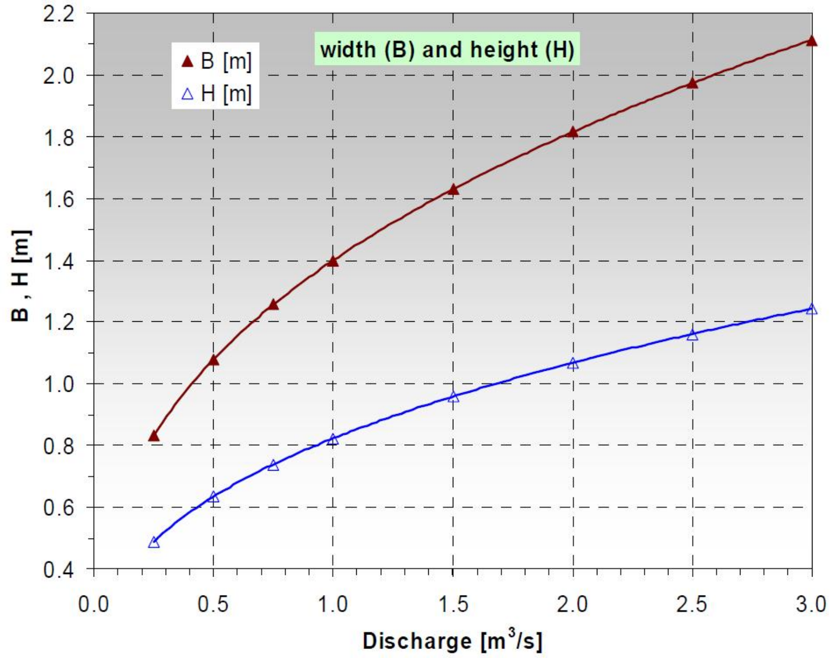

Figure A2-1 Width and Height of Conveyance Channel as a Function of Discharge

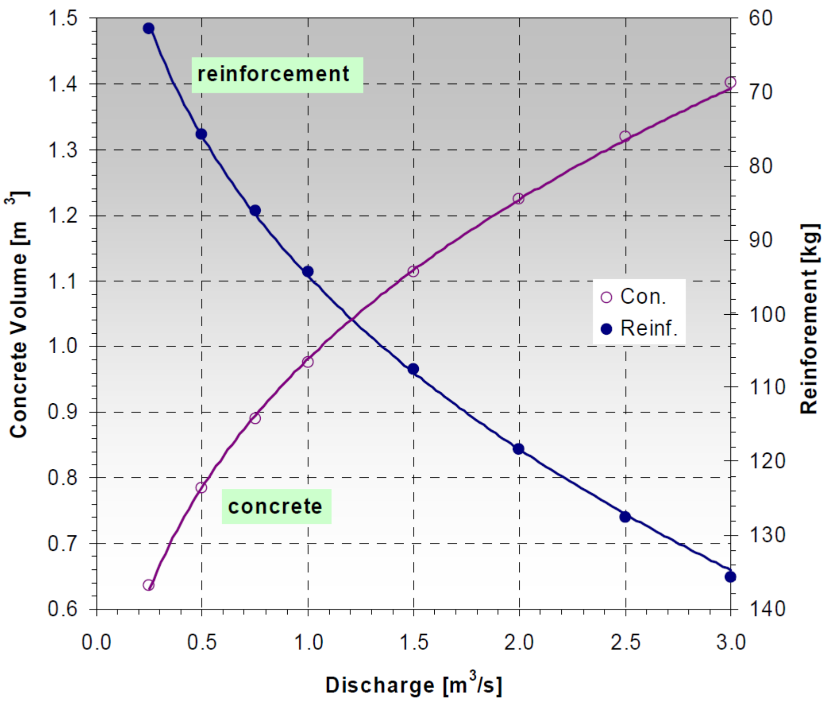

Figure A2-2 Concrete Volume and Reinforcement of Conveyance Channel as a Function of Discharge

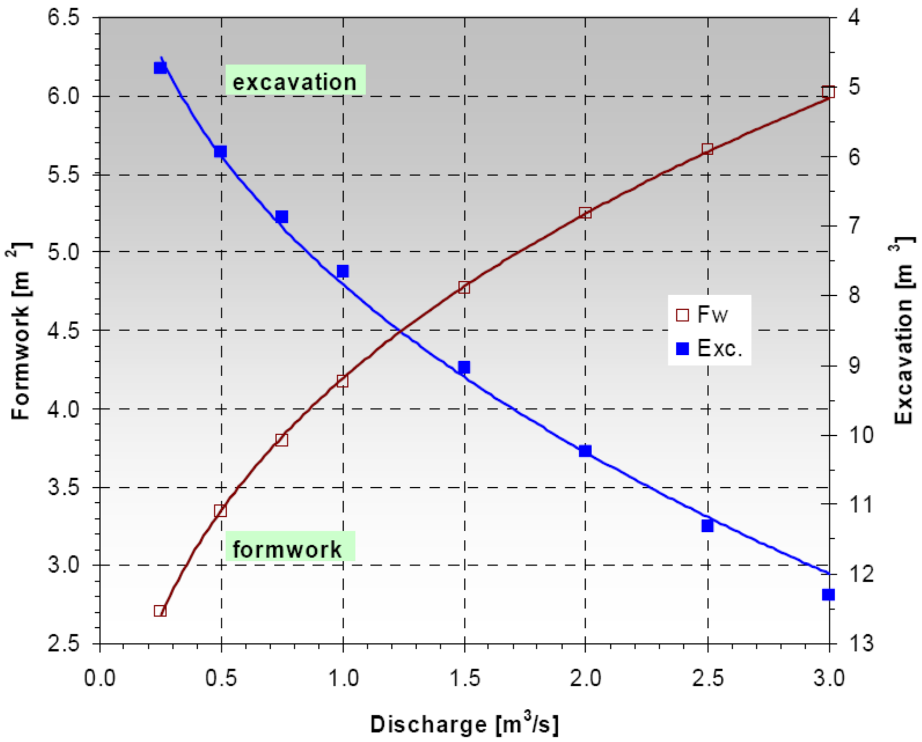

Figure A2-3 Excavation and Formwork of Conveyance Channel as a Function of Discharge

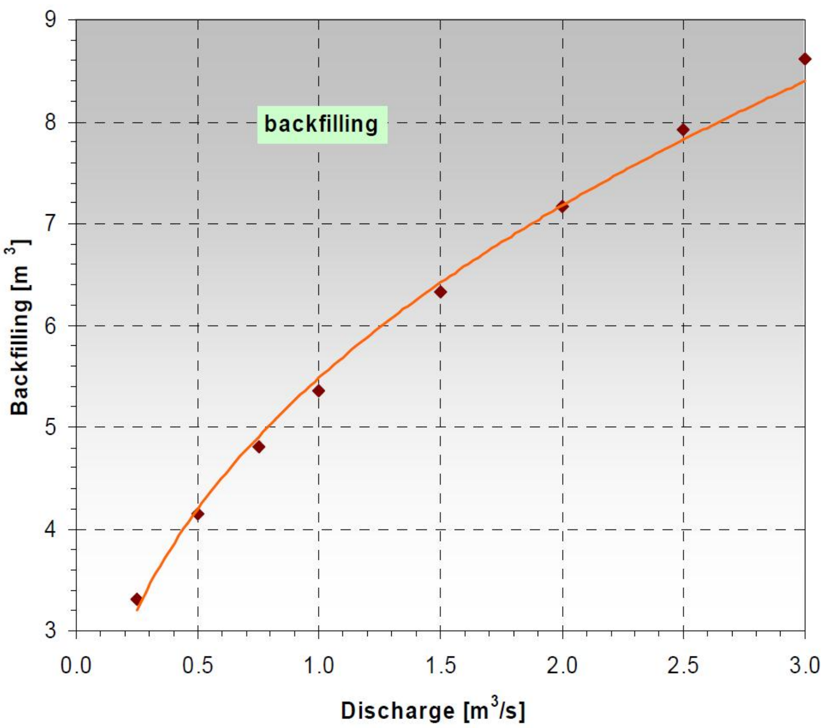

Figure A2-4 Backfilling of Conveyance Channel as a Function of Discharge