FEASIBILITY OF A SUPPLEMENTARY WATER STORAGE FOR BİRKAPILI HYDROELECTRIC POWER PLANT

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submitted by **MELİH BOZKURT** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Canan Özgen _______________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Güney Özcebe _____________________
Head of Department, **Civil Engineering**

Prof. Dr. Ali Melih Yanmaz _____________________
Supervisor, **Civil Engineering Dept., METU**

Nadi Bakır, M.Sc. _____________________
Co-Supervisor, **ERE Company**

Examining Committee Members:

Assoc. Prof. Dr. Nuri Merzi _____________________
Civil Engineering Dept., METU

Prof. Dr. Ali Melih Yanmaz _____________________
Civil Engineering Dept., METU

Assoc. Prof. Dr. Zafer Bozkuş _____________________
Civil Engineering Dept., METU

Assist. Prof. Dr. Şahnaz Tiğrek _____________________
Civil Engineering Dept., METU

Nadi Bakır, M.Sc. _____________________
ERE Company

Date: 09.09.2011

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name : MELİH BOZKURT

Signature :

ABSTRACT

FEASIBILITY OF A SUPPLEMENTARY WATER STORAGE FOR BİRKAPILI HYDROELECTRIC POWER PLANT

Bozkurt, Melih

M.Sc., Department of Civil Engineering

Supervisor: Prof. Dr. Ali Melih Yanmaz,

Co-Supervisor: Nadi Bakır, M.Sc.

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Climate change concerns, high oil prices and increasing government support are some of the driving reasons of increasing renewable energy legislation, incentives, and commercialization. Hydroelectricity is the most widely used form of renewable energy and refers to electricity generated by hydropower. In this study, a storage facility is proposed to store some additional water and increase the profitability of the existing Birkapılı Hydroelectric Power Plant. The storage facility is composed of a gravity dam and an uncontrolled spillway. With the help of the proposed storage facility, maximum utilization of the water is provided and shift of the electricity generation to peak demand periods becomes possible. Consequently, feasibility of the existing power plant is improved. A number of alternatives for a spillway are taken into account and the corresponding concrete gravity dam is designed. Stability

analyses and operation studies are conducted using spreadsheets to achieve an economical solution.

Keywords: Birkapılı HEPP, Reservoir, Additional storage, Optimization

ÖZ

BİRKAPILI HİDROELEKTRİK SANTRALİNE AİT İLAVE SU DEPOLAMA TESİSİNİN FİZİBİLİTESİ

Bozkurt, Melih

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Prof. Dr. Ali Melih Yanmaz

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Yenilenebilir enerji mevzuatının, teşviklerin ve ticarileşmenin artmasının arkasındaki itici nedenlerden bazıları iklim değişimi hakkındaki kaygılar, yüksek petrol fiyatları ve artan hükümet destekleridir. Hidroelektrik, yenilenebilir enerji türleri arasındaki en yaygın kullanılan tür olup, su gücü ile üretilen elektrik olarak adlandırılır. Bu çalışmada, mevcut Birkapılı Hidroelektrik Santralinin kârlılığını artırmak üzere bir depolama tesisi önerilmektedir. Depolama tesisi, bir ağırlık barajı ile kontrolsüz dolu savaktan oluşmaktadır. Önerilen depolama tesisinin yardımıyla, sudan azami faydalanma sağlanarak, elektrik üretiminin en yüksek talep dönemlerinde yapılabilmesi mümkün hale gelmektedir. Sonuç olarak mevcut santralin elverişliliği geliştirilir ve enerji üretimi azami talep dönemlerine kaydırılabilir. Santralin azami talep dönemlerinde çalıştırılmasıyla, yüksek elektrik talebinin karşılanmasına katkı sağlanır. Dolusavak için bir takım seçenekler hesaba katılmış ve ilgili betonağırlık baraj tasarlanmıştır. Ekonomik bir sonuç elde etmek için, hesap tabloları kullanılarak, yapı emniyet analizleri ve işletme çalışmaları yapılmıştır.

Anahtar Kelimeler: Birkapılı HES, Hazne, Ek depolama, Optimizasyon

To My Granddaddy and Family...

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

FIGURES

LIST OF ABBREVIATIONS

BOT	Build-own-transfer
CO_2	Carbon dioxide
DSİ	General Directorate of State Hydraulic Works
EİE	General Directorate of Electrical Power Resources Survey and Development Administration
EMRA	Energy Market Regulatory Authority
EU	European Union
ha	Hectare
HEPP	Hydroelectric Power Plant
GDP	Gross Domestic Product
GW	Gigawatt
km	Kilometer
krş	Kuruş
KRW	South Korean Won
m	Meter
mm	Milimeter
MW	Megawatt
OECD	Organization for Economic Co-operation and Development
TEİAŞ	Turkish Electricity Transmission Company
TL	Turkish Lira

TWh	Terawatt hour
USA	The United States of America
USBR	United States Bureau of Reclamation
USD	The United States Dollar

LIST OF SYMBOLS

A_{dam}	Profile area of dam body above ground level (m^2)
A_{foun}	Profile area of dam body foundation (m^2)
A_{lface}	Area of lateral faces of structures (m^2)
A_{totald}	Total profile area of dam body (m^2)
$A_{totalsp}$	Total profile area of spillway (m^2)
$A_{totalsw}$	Total profile area of side wall (m^2)
$A_{u-dface}$	Total area of upstream and downstream faces of structures (m^2)
B	Base width (m)
e	Eccentricity (m)
E	Energy (kWh)
E_{opeak}	Energy generated in the off-peak period (kWh)
$E_{opeakavg}$	Average annual off-peak energy generation (kWh)
E_{peak}	Energy generated in the peak period (kWh)
$E_{peakavg}$	Average annual peak energy generation (kWh)
E_{tot}	Total annual energy production (kWh)
f	Friction factor

F_{dh}	Earthquake force (horizontal component) (kN/m)
F_{dsilt}	Dynamic force induced by the accumulated silt during earthquake (kN/m)
F_{dv}	Earthquake force (vertical component) (kN/m)
F_f	Weight of fill which is found on the foundation (kN/m)
F_{fill}	Fill force (kN/m)
F_h	Horizontal hydrostatic force component (kN/m)
F_i	Ice force (kN/m)
Fr	Froude number
F_s	Weight of silt which is found on the foundation (kN/m)
F_{silt}	Silt force (kN/m)
FS_o	Overturning safety factor
FS_s	Sliding safety factor
FS_{ss}	Combined shear and sliding safety factor
F_u	Uplift force (kN/m)
F_v	Vertical hydrostatic force component (kN/m)
F_w	Dynamic force induced by water during earthquake (kN/m)
g	Gravitational acceleration (m/s^2)
h	Water depth (m)

H	Spillway height (m)
H_f	Downstream fill height (m)
H_{foun}	Height of the foundation (m)
H_{net}	Net head (m)
H_r	Depth of riprap protection (m)
H_s	Sediment accumulation height (m)
H_{sb}	Thickness of the stilling basin foundation (m)
H_{sl}	Length of the slopped part (downstream) (m)
H_{sp}	Height of spillway (m)
H_{sw1}	Height of side wall (upstream) (m)
H_{sw2}	Height of side wall (downstream) (m)
H^*	Upstream height of the rectangular part (m)
H'	Downstream height of the rectangular part (m)
I_{avg}	Average annual income (TL)
I_{opeak}	Off-peak period income (TL)
$I_{opeakavg}$	Average annual off-peak energy income (TL)
I_{peak}	Peak period income (TL)
$I_{peakavg}$	Average annual peak energy income (TL)
I_{tot}	Total income (TL)
k	Earthquake coefficient

L_c	Length of the curved part of spillway (m)
L_s	Length of the slopped part of the dentate sill (m)
m_1	Slope of the upstream face of dam body
M_O	Overturning moment (kN.m/m)
M_R	Resisting moment (kN.m/m)
n_1	Slope of the downstream face of dam body
P	Power (Watts)
P-Value	Parameter used in the Kolmogorov-Smirnov test
Q	Water discharge (m^3/s)
Q_d	Design discharge (m^3/s)
Q_{inc}	Incoming discharge (m^3/s)
S	Reservoir storage capacity (m^3)
T_c	Thickness of the dam crest (m)
T_{opeak}	Off-peak period (hours)
T_{peak}	Peak period (hours)
T_r	Return period (years)
u	Flow velocity (m/s)
V_{conc}	Volume of concrete (m^3)
V_f	Final reservoir volume (m^3)
V_i	Initial reservoir volume (m^3)

V_{inc}	Incoming water volume (m^3)
V_{max}	Maximum available water volume (m^3)
V_{opeak}	Water volume used to generate energy in the off-peak period (m^3)
V_{peak}	Water volume used to generate energy in the peak period (m^3)
V_r	Water volume released (m^3)
V_{rmin}	Minimum water volume released (m^3)
V_s	Water volume stored (m^3)
W	Weight of concrete (kN/m)
W_{dam}	Width of dam body (m)
W_{sp}	Width of spillway (m)
W_{sw}	Width of side wall (m)
x	Distance from center (m)
X	Horizontal distance of the spillway gravity center to the point where the stilling basin starts (m)
y	Flow depth (m)
y_c	Critical depth of flow (m)
y_2	Sequent depth of hydraulic jump (m)
Y	Vertical distance of the spillway gravity center to the bottom of the spillway foundation (m)

ΔM	Net moment (kN.m/m)
Δmax	Maximum P-value
γ_{conc}	Specific weight of concrete (kN/m^3)
γ_w	Specific weight of water (kN/m^3)
ε	Overall efficiency of the system
ρ	Water density (kg/m^3)
σ_{heel}	Stress at the heel (kN/m^2)
σ_{toe}	Stress at the toe (kN/m^2)
τ_s	Shear strength of concrete (kN/m^2)
$\sum F$	Total net force (kN)
$\sum H$	Sum of horizontal force components (kN)
$\sum V$	Sum of vertical force components (kN)

CHAPTER 1

INTRODUCTION

Water is being utilized to perform work for thousands of years. More than 2000 years ago, water was harnessed to grind wheat into flour by the Greeks. Besides grinding flour, water power was used to saw wood and power textile mills and manufacturing plants.

A French hydraulic and military engineer, Bernard Forest de Belidor initiated modern hydropower turbine evolution by writing the book, Architecture Hydraulique in which a vertical-axis versus a horizontal-axis machine was described [1].

In recent years, energy demand has soared up dramatically in the world. The primary energy demand has increased by more than 50% since 1980. This increase is expected to continue at an annual average rate of 1.6% in the future. Economies and populations of developing countries are growing much faster than those of OECD nations and over 70% of energy demand growth is forecast to come from developing countries [2].

Turkey, as a developing country, has scarce domestic energy resources. To satisfy its needs to meet its rapidly growing demand for electricity, Turkey pays a considerable amount of money to foreign countries for energy resources. That situation makes it a dependent country on imports [3]. The most important domestic energy resources that are commercially available for development are hydropower and lignite coal in Turkey. It should be noted that lignite coal used in energy generation is of low quality mostly.

In Turkey, the economic development is being followed by a steadily growing energy consumption rate in the past decades [4]. In the past, the Turkish electricity generation market was dominated by coal and hydroelectric power but nowadays the share of gas sources is highly increased. According to the data issued by Energy Market Regulatory Authority (EMRA) in September 2010, the total electricity generation capacity in Turkey is 46.5 GW. Hydropower and natural gas constitute 34.0% and 32.3% of installed generation capacity of Turkey, respectively. Coal and lignite-fired power stations represents 18.8% of the installed capacity. The other resources constituting the electricity generation capacity in Turkey are multi-fuelled plants (6.0%), oil (3.4%), imported coal (4.5%), and other renewable alternatives (1.0%) [5].

A curtailment in energy consumption in Turkey has been observed in late 2008 and 2009 as a result of the financial crisis. In contrary to this fact, Turkey’s state controlled electricity transmission company TEİAŞ estimated that the energy demand will increase by 4.5 – 7.5% on average annually until 2018. In this demand estimation, two different forecasts which are high and low forecasts are made. Energy demand and development in gross domestic product (GDP) per capita is closely linked in Turkey. An average Turkish citizen consumes one third as much electricity as the average European Union (EU) citizen. Demand in energy is expected to grow in Turkey as GDP per capita gradually increases [6].

Turkey’s economically viable hydroelectric capacity potential is estimated to be 35 GW by EMRA in September 2010. The installed capacity was 15.1 GW by that date meaning that nearly 43% of Turkey’s viable hydroelectric potential is utilized. Investment in hydropower generation is encouraged by the government recently and a significant hydropower expansion is underway in Turkey [5].

1.1 Upgrading Existing Hydropower Projects

To meet ever-growing energy demand in the world, new power plants of all types are continuously being constructed. Another way of meeting this demand is exploiting the existing facilities in the most optimum way. This can be done by upgrading the existing plants in many different ways. In the following paragraphs, some examples of upgrading the existing facilities are given.

1.1.1 Itaipu Hydropower Project (Parana River, Brazil and Paraguay)

The transmission capacity of the Itaipu Hydroelectric Power Plant is planned to be expanded within the context of Itaipu Hydropower Project. The company which has been awarded a USD 80 million contract is to construct a substation and expand the existing installation. Project construction began in September of 2010 and is planned to be completed in 2013. Especially, Paraguay is expected to gain benefits from the expansion project in terms of satisfying the growing demand for electricity in that country [7].

1.1.2 Jirau Hydropower Project (Madeira River, Brazil)

Jirau Hydropower Project which is being built on Madeira River by GFD Suez and International Power will be expanded with an additional 450 MW of installed capacity. Six new power generator units will be added to the existing project. 50 or more bulb turbines will be used in the project and installed capacity will grow from 3300 MW to 3750 MW. There will also be a development in the assured power output of the plant. It will be increased from 2184 MW to 2274 MW thanks to the announced expansion. The power plant is expected to commence commercial operation in 2014 [8].

1.1.3 Cambambe Hydropower Project (Kwanza River, Angola)

As a part of plan to repair, expand and build new hydroelectric facilities to reduce Angola's energy deficit, the Angolan government is planning to increase the capacity of the Cambambe Hydroelectric Plant in Kwanza-Norte province. There are four generator groups of each having 45 MW capacities operating at Cambambe Dam. Two of these generators have been out of service since 2009. With new increase, the capacity will be upgraded to 960 MW [9].

1.1.4 Holtwood Hydropower Project (Susquehanna River, USA)

PPL Holtwood, LLC is planning to increase the capacity of Holtwood Hydroelectric Plant which has been generating electricity since 1910, using the power of the water held back by a 16.75-meter-high dam across the Susquehanna River in central Pennsylvania. With the new facility, 108 MW of existing capacity will be increased by an amount of 125 MW. Also, improved passage for migratory fish along Susquehanna River and improved recreational opportunities will be supplied as additional benefits [10].

1.1.5 Rainbow Hydropower Project (Missouri River, USA)

PPL Montana LLC began USD 230 million upgradation project at the Rainbow Hydroelectric Power Plant on Missouri River in 2009. Several smaller units will be replaced by a 62 MW unit within the context of the project. When completed in 2012, according to estimated schedule, the plant's power generation capacity will increase by 70%. The existing Rainbow Plant having eight turbine generators will operate until the new power house is completed and then will be decommissioned [11].

1.1.6 Hanggang Hydropower Project (Hanggang Water System, South Korea)

Korea Hydro and Nuclear Power Co, Ltd. has been planning to add a 60-MW Unit 4 to the 79.6-MW Cheongpyeong Hydroelectric Project on Hanggang Water System in South Korea. Work on adding generators has started in March 2009 and is planned to be finished by June 2011. The new total project capacity will be 139.6 MW. Cutting a tunnel through a hill next to the dam, making a waterway, and installing a generator is involved in the project. The new unit will use water discharged from the project during the flood season. By implementing this project, it will be possible to decrease the amount of expenditures made on petroleum imports by KRW 4.1 billion, annually. In addition, it is expected that carbon dioxide (CO_2) emissions will be reduced by around 27000 tons based on 2007 data [12].

1.2 Scope of the Study

Birkapılı Hydroelectric Power Plant project was initiated on November 24, 2000 when ERE Hydroelectric Company and The Ministry of Energy and Natural Resources signed the related contract. After the energy market law with number 4628 had come into effect, the project started to be established under free electricity market conditions. Due to the urgency of work, the power plant was constructed without an additional storage facility. The site on which an additional facility is planned to be constructed has suitable geological formations for water storage. It is believed that the cost of a small storage facility is low and it can amortize itself in a short period.

In this thesis, an alternative approach to upgrade power generation for the existing Birkapılı Hydroelectric Power Plant (HEPP) is developed and the case of constructing the aforementioned storage facility at the upstream part of the plant is studied. The aim of this study is to design an economical storage facility which will store some additional water in order to maximize the utilization of water, shift the electricity generation to peak periods as much as possible, and consequently improve the feasibility of the existing power plant.

In the course of the studies, a flow analysis for the project site is carried out for the sake of determining the design discharge of the new storage facility. The basin area of the storage facility is calculated by utilizing the available maps. Relevant data of the stream gauging stations in the vicinity of the project site are used for correlation studies.

A flood frequency analysis is also performed to have idea about floods having high return periods. The results are used to finalize the designs of the spillway and dam body of the storage facility.

In order to determine the electricity sale prices in the market, the day-before prices, which are developed via the Market Financial Settlement Center, are examined and hourly generated the day-before prices are used to specify the peak hours when the energy is sold in the highest prices. These prices are utilized to determine the income of the project in economic analysis.

Different alternatives are taken into account for determining relevant dimensions of the dam body and spillway. For each alternative, a stability analysis is performed and safety of the contemplated structure is ensured. These analyses are performed for the spillway and dam body alternatives separately under different loading combinations.

In the last part of the studies, an economic analysis is performed to specify the cheapest alternative. Different cost items are included in the study and a bill of quantities is prepared. The calculated cost is increased by some amount considering some additional costs, unforeseen costs, and project costs.

Following this chapter, in Chapter 2, some general brief information including advantages, disadvantages, and types of hydropower is given. Besides, water cycles are mentioned and the situation of hydropower in the world and in Turkey is delineated. Then the studies performed in the determination of the best alternative for the intended storage facility are given in Chapter 3. These studies include the correlation studies carried out for flow analysis for the project site, stability analyses of various dam bodies and spillways, flood frequency analysis, etc. In the forth chapter, an economic analysis, which is performed in order to specify the optimum alternative for the storage facility, is depicted and the details of this analysis are given. Finally in the last chapter, the conclusions derived are presented.

CHAPTER 2

CHARACTERISTICS OF HYDROPOWER PLANTS

2.1 Hydroelectric Power

One of the key factors of the economic development is energy. It is obligatory to utilize energy resources which have no detrimental social effects and are continuously available for long durations for sustainable development. In other words, it is crucial to utilize sustainable energy resources. It is a fact that energy resources which are originated from fossils are both exhaustible and have detrimental effects to environment and society. Because of this fact it is inevitable to focus on alternative energy resources. Some important advantages which are possessed by these alternative energy resources including hydropower are being sustainable, renewable, environmentally friendly, and clean. Having these advantages, hydropower has an important role in contributing to the future world energy mix, especially in developing countries, such as Turkey [13].

Hydroelectric power is a renewable resource for energy. Other renewable resources include geothermal, wave power, tidal power, and solar power. In Turkey, hydroelectricity power is the most common renewable resource utilized for electricity production. Potential of energy that can be presented by hydroelectricity power is much higher than other renewable resources can.

A hydroelectric power plant is used for generation of electricity from water on large scale basis. A dam is built across a large river that has sufficient quantity of water throughout the river. In certain cases where the river is very large, more than one dam can be built across the river at different locations [14].

Constructing a big dam across a river has its own advantages but it is known that there are some economical, social, and environmental disadvantages related to big dams [15].

2.2 Hydropower Advantages

Hydropower is not a new-born technology and strengths and weaknesses of hydropower are equally well understood [16]. Some of the advantages belonging to hydropower can be given as follows:

<u>Cleanness:</u> One of the main benefits of hydropower is the ability to produce energy in a clean way. Waste gases and harmful particulate materials contributing to air pollution, global warming, acid rain, etc., are not emitted by hydropower plants during energy production. Thus it keeps the surrounding atmosphere clean and healthy for living. Also, noise pollution is not a result of generating energy via a hydropower plant. No pipelines, barges, trains or trucks are needed to bring fuel to the power plant site.

<u>Renewability:</u> Since hydropower energy follows the hydrologic cycle, it is renewable. As long as water exists in the world, there will be evaporation, condensation and precipitation; meaning that water flow to run the turbines.

<u>Reliability:</u> Water will stand as an important energy source as long as it exists. Energy generation from water may only be halted if all lakes and rivers feeding the hydropower plants dry up.

Efficiency: Hydropower is the most efficient energy generator. Ninety percent of the available energy can be converted into electricity via hydropower. If falling water running through a hydropower plant is to be compared to fossil fuels burned, the efficiency difference is huge. Even the most efficient fossil fuel plants have an efficiency of 60% [17]. The rest of the fossil fuel is wasted by means of heat and gases.

No Need for Fuel: One of the most salient advantages of hydropower is that no fuel is needed to operate the plant. Effects caused by the fuel cost are eliminated and hydroelectric power plants become immune to price increases for fossil fuels, such as oil, natural gas or coal. The plants do not require imported fuel to generate electricity.

Availability of Small Scale Hydro Systems: Small scale hydro systems are ideal for remote sites where water is available a long distance from power needs. Micro hydro systems can be thought as a miniature version of a hydroelectric system. Not much flow rate is needed to run the system and it is a cost-effective energy solution. The system could be built with a cost of USD 1000 to USD 20000 depending on site electricity requirements and location. Maintenance fees are also smaller than that of other technologies [18].

Constant Electricity Cost: The main reason of change in cost of electricity production is the changes in fuel prices. Hydroelectric power plants do not consume any fuel (such as coal, oil, natural gas, etc.) to generate electricity, thus the electricity cost is more or less constant for them. Also since the country does not have to import the fuel for running the hydroelectric power plant, local currency is saved in the country [19].

Long Life: The life of hydroelectric power plants is longer than the life of thermal power plants. There are some hydroelectric power plants that were built 50-100 years ago and are still running.

Small Electricity Generation Cost: Very few people are required in order to operate a hydroelectric power plant as most of the operations carried out to run the system are automated. That situation makes the labor costs of a hydroelectric power plant low. Furthermore, generation cost of electricity in a hydropower plant is abated as the power plant become older. The reason is that as the plant becomes older, the initial capital cost invested in the plant is recovered over the long operation periods.

Ability of Commencing and Stopping Energy Generation in a Short Period of Time: The daily power demand is changeable throughout the day and generally peak power demand occurs at night. To response the changes in power demand, flexible facilities like hydroelectric power plants are very handy. It is not possible to start and stop operation of thermal and nuclear power plants on daily basis, whereas hydroelectric power plants can be started or stopped in a very short time period. In facilities with a pondage or storage, water can be collected behind the dam in off-peak period and can be used to generate electricity during peak periods.

Wide Distribution of Hydro Resources: Hydro resources are widely distributed compared with fossil and nuclear fuels. This situation provides energy independence for countries which have no fossil fuel resources.

Beside the aforementioned benefits, plants with storage have advantages of irrigation of farmlands, recreational purposes, and preventing floods.

2.3 Hydropower Disadvantages

It is not possible to generate electricity by hydropower plants when there is no available water. Their electricity production is badly affected by drought.

Local environment is impacted by construction of new hydropower facilities. Humans, flora, and fauna may lose their natural habitat.

There may be more highly valued alternatives of land uses than electricity generation. In such cases hydropower plants cannot be constructed even if the project seems very beneficial.

Water quality and flow can be deteriorated by the construction of a hydropower facility. Dissolved oxygen levels in the water could be abated. This situation results in giving harm to riparian (riverbank) habitats. In order to be able to overcome such a problem, oxygenation of the water could be entailed.

A minimum amount of water should be maintained downstream to keep the survival of riparian habitats.

Migration of fish populations from upstream to downstream or vice versa should be maintained properly by providing fish ladders, elevators, lights, sounds, etc., which usually increase the cost of the project [20].

2.4 Working Principle of Hydroelectric Power Plants

Two types of energies, which are the potential energy due to the height of water and the kinetic energy due to flow of water are possessed by water flowing in a river. The potential energy of water is utilized to generate electricity in hydroelectric power plants. The potential energy is converted to kinetic energy in the penstock. This kinetic energy of the flowing water is converted to mechanical energy by making the turbine blades be turned by water. Finally, the turbine shaft rotates the generator and electrical energy is obtained [21].

Total power which can be generated from water in a hydroelectric power plant is given as:

$$P = \rho g \varepsilon Q H \tag{2.1}$$

where;

P: Power (Watts)

ρ: Water density (kg/m^3)

g: Gravitational acceleration (9.81 m/s^2)

Q: Water discharge (m^3/s)

H: Water gross head (m)

ε: Overall efficiency of the system

The conclusion which can be derived from the statement given above is that the total power that can be produced in a hydroelectric power plant is mainly a function of two major factors: the flow rate of water and head of water. As these two factors increase in magnitude, the electricity generated in the plant will also increase.

The difference between the levels of water in the forebay and power generation unit is tried to be maximized in order to obtain a high water head.

The maximum height of water in a reservoir is a function of natural factors like water amount, river bed height and other environmental factors. It could be preferred to locate the power generation unit at a level lower than the ground level so that maximum head of water can be maintained.

Depending on the requirements, flow rate of water in the penstocks can be adjusted during the operation of the facility. In a high power demand situation, more water could be allowed to flow through the penstocks and in a contrary situation it is vice versa [22].

2.5 Water Cycle

During the generation of hydropower, two types of water cycles are seen. These water cycles can be given as the water cycle in nature and water cycle in the hydroelectric power plant. In this text, only the water cycle in hydropower plants is discussed briefly.

The most widespread method used to generate electricity in a hydroelectric power plant is constructing dams across the rivers. Water is diverted from a river by pipelines and transmitted to the main plant where turbines are installed. Water is fallen to the turbine blades via penstocks and these blades start rotating. As blades rotate, the turbine shaft also rotates in electric generators and as a result electricity is produced. After rotating the turbine blades, water flows back to the river at lower levels. Hence, water is releashed to rivers through which they are collected in the oceans [23, 24]. In Figure 2.1, an illustrative sketch belonging to the water cycle in a hydraulic power plant is shown.

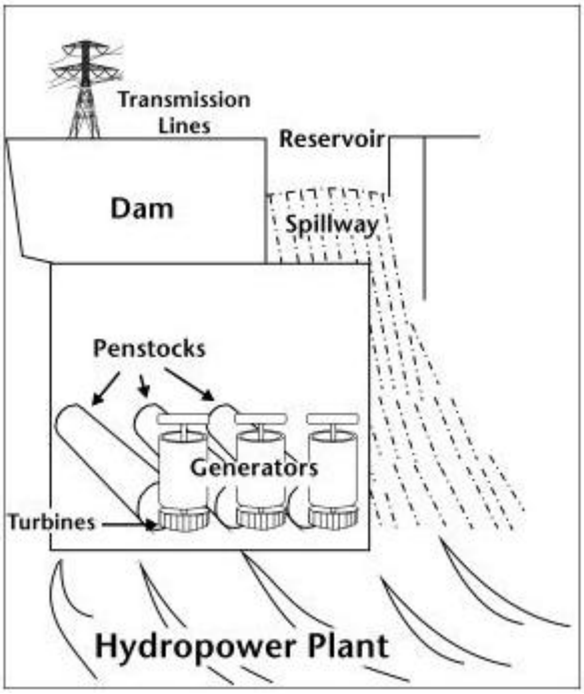


Figure 2.1 Water cycle in a hydropower plant [25]

2.6 Types of Hydroelectric Power Plants

2.6.1 Classification Based on Hydraulic Characteristics

The hydropower plants may be categorized into three types according to their hydraulic characteristics or flow regulation capacity as:

- run-of-river plants,
- storage plants
- pumped-storage plants

2.6.1.1 Run-of-River Plants

This type of plants has no storage and the energy production is completely depends on the flow coming to the facility throughout the year. That is why this kind of plants can be considered as base load plants [26]. The excessive amount of water is allowed to flow over the spillways in the rainy season, in other words, wasted and in the dry season, power generated via the facility is abated due to lack of flow.

A descriptive sketch belonging to a typical run-of-river hydroelectric power plant is given in Figure 2.2.

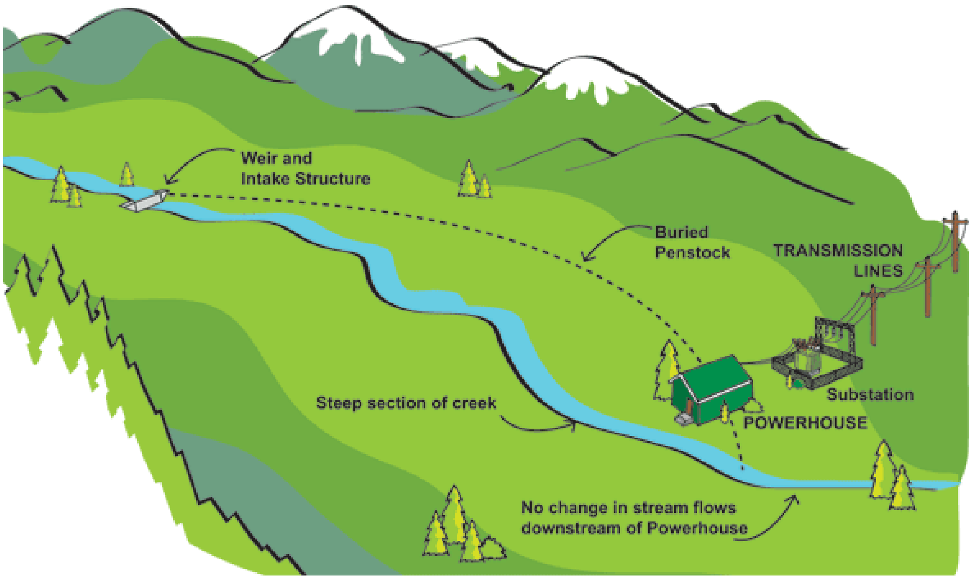


Figure 2.2 Run-of-river hydroelectric power plants [27]

This type of plants may further be sub-divided into two categories as:

- run-of-river plants without pondage
- run-of-river plants with pondage

Run-of-river plants without pondage are not able to store any water and they directly use the available running water in the stream. The dam constructed at the site most probably has an aim of other than hydropower development such as aiding irrigation or navigation. In these plants normal runoff flow in the stream is not affected due to construction of the facility since they have neither a large reservoir nor do they have diversion of water away from the main channel. The power house is located on the main course of the river. Non-uniformity and ambiguity of the water supply are the main weaknesses of this type of plants.

The stream capacity could be increased for short periods by facilitating a pondage behind the dam. The plants having pondages are enabled to store water during the off-peak period and use it in the peak hours of the same day or week. In other words, the plant gains the capability of meeting the hourly or daily fluctuations. The plant design discharge, thus, may be many times more than the minimum stream flow. Run-of-river plants with pondage could operate on the base of the load curve in the event that there is much water flow in the river. However, with abated stream flow, they might feasibly operate on the peak of the load curve.

A typical load curve is shown in Figure 2.3.

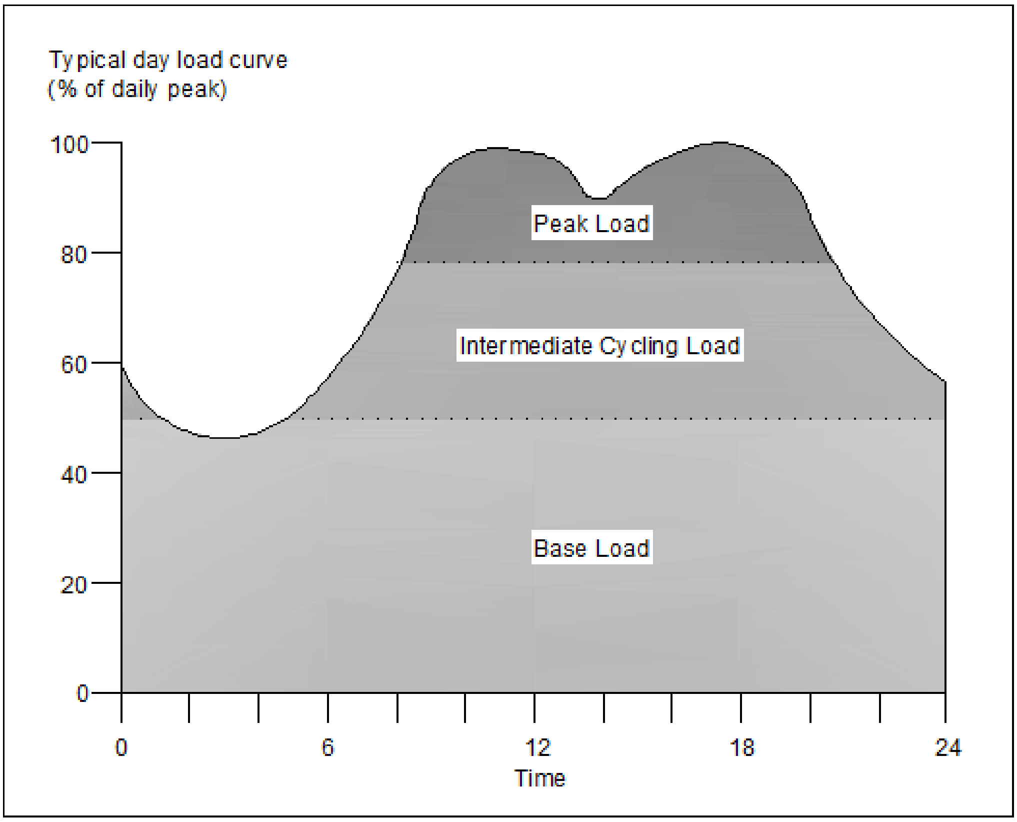


Figure 2.3 Daily load curve [28]

2.6.1.2 Storage-Plants

Storage-plants (most common type of hydroelectric power plants [29]) have fairly larger sized reservoirs which are able to provide sufficient storage to carry over from wet season to dry season and sometimes even from one year to other. The main advantage of this plant is the ability of not being affected by the fluctuations in the river supply during dry season. The energy production can be maintained in a uniform way.

In Figure 2.4, a storage-plant and its main components are shown.

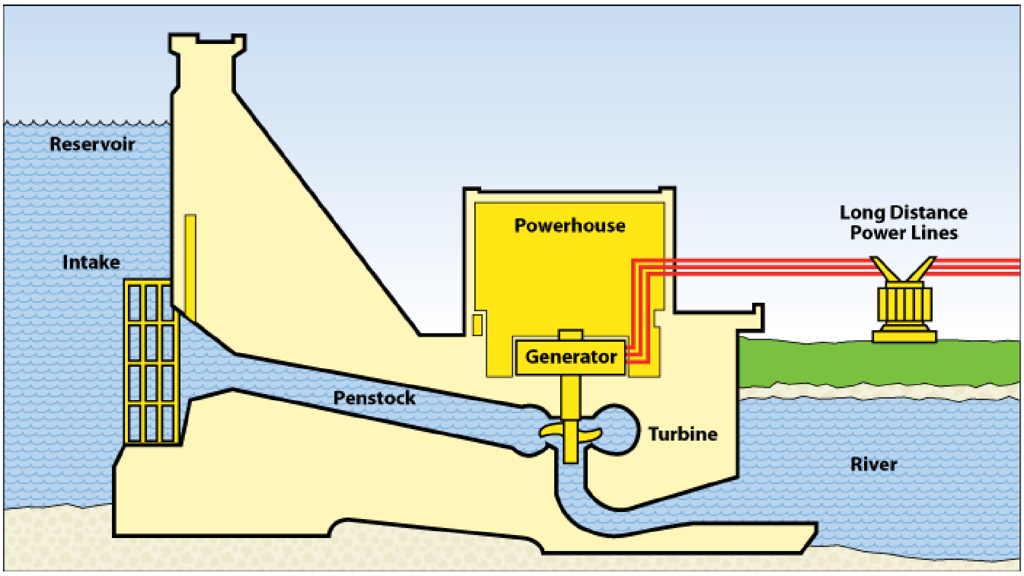


Figure 2.4 Storage-plants [30]

2.6.1.3 Pumped-Storage Plants

In pumped-storage plants, reversible-pump turbines transfer water from the tail race to the high level reservoir during off-peak hours. The energy used to pump water is derived from other available sources, such as nuclear, fossil, and renewable power plants, whose power output cannot be adjusted to meet load fluctuations. Power stations work in two phases. In the generating phase, the turbines and generators are producing electrical power and water flow is from the high level reservoir to the power house. In the pumping phase, the pumps and motors are active and the water flow is from the power house to the high level reservoir (See Figures 2.5 and 2.6). Therefore, pumped-storage plants can also be called as large batteries [31].

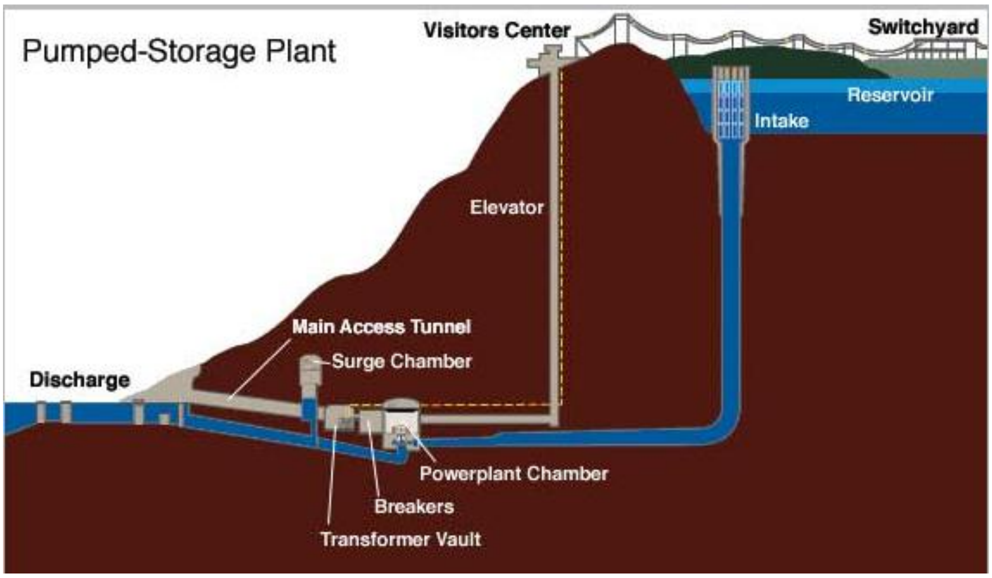


Figure 2.5 Pumped-storage plants [32]

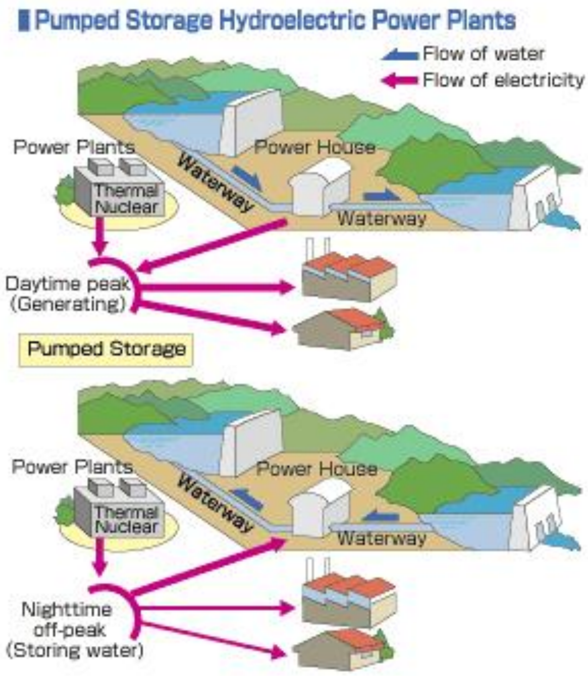


Figure 2.6 Working phases of a pumped-storage plant [33]

Some of the key benefits of this type of plants can be listed as follows [26, 34]:

- They increase profitability for plant owners in volatile electricity markets
- They allow optimization of power plant fleets and electrical network infrastructures
- High global cycle efficiency compared to other large storage solutions (approximately 80%)
- Positive environmental impact by enabling increased use of renewable energy sources

2.6.2 Classification Based on Head

Hydroelectric power plants can be classified into three based on their head as follows:

- low head plants
- medium head plants
- high head plants

2.6.2.1 Low Head Plants (Canal Power Plants)

A plant is called to be a low head plant when the operating head is less than 30 meters. A dam is constructed across the river and water is stored behind that dam. The power plant is installed near the base of the dam. Kaplan turbines or vertical shafts are preferred for this type of plants [35, 36].

2.6.2.2 Medium Head Plants

The operation of a medium head plant is similar to that of a low head plant. It works on heads between 30 to 100 meters. Francis or Kaplan type of turbines and propellers are preferred as prime movers in medium head plants. Water is transferred to the forebay from the main reservoir via a tunnel or a canal and then sent to turbines through a penstock. The forebay also serves as a surge tank in medium head plants [35, 36].

2.6.2.3 High Head Plants

In high head plants, water is stored at high elevations and dropped from a head of above 100 meters. The main parts of such a plant can be given as: the dam, the intake structure, the pressure tunnel, the surge tank, the penstock, the power house, and the tail race.

In order to prevent foreign particles from entering the tunnels, trash racks could be fitted at the inlet of the pressure tunnel. A surge tank is located before the valve house and after the tunnel from the head works. In valve house, generally, butterfly or sluice types of valves are utilized to regulate the flow driven into penstocks. Up to 300 meters of head, Francis turbines are installed into the facility and for heads above 300 meters, Pelton wheels are selected as the common prime movers [35, 36].

2.6.3 Classification According to the Nature of Load

Hydropower plants are classified into two according to basis of operation [37]:

- base load plants
- peak load plants

2.6.3.1 Base Load Plants

A plant is said to be a base load plant if it supplies base load (see load curve given in Figure 2.3) which is more or less constant. These plants run at all times through the year except in the case of repairs or scheduled maintenance. The efficiency of the generating units is usually high and they work on almost constant load. Base load plants fuelled by nonrenewable resources include nuclear and coal-fired thermal plants. As a hydroelectric power plant, run-of-river and storage types of power plants are examples of base load plants.

2.6.3.2 Peak Load Plants

Peak load plants are capable of meeting fluctuating peak loads. Run-of-river plants with pondage and pumped-storage plants are good examples of peak load plants. Storage type of plants may also provide peak load service. In general, it can be said that hydropower is quite suitable for peak load durations. Energy production can be initiated quickly and it is relatively easy to regulate energy production in peak load periods.

2.6.4 Classification Based on Plant Capacity

The classification based on the installed capacity of hydropower plants varies throughout the world and there is no concurrence of opinion. In Turkey, plants which have less than or equal to 50 MW of installed power are accepted as small hydroelectric power plants [6]. According to the European Commission and Small Hydro Association, the hydropower plants with an installed capacity of 10 MW or less are small hydropower plants and the plants having higher capacity are large [38]. The United States Department of Energy names the hydropower plants having an installed capacity of 100 kW to 30 MW as small hydropower [29]. In India plants with an installed capacity equal to or less than 25 MW are assumed to small hydropower [39] and in China the threshold value is 50 MW [40].

Small hydropower plants may further be divided into three categories as mini, micro, and pico hydropower since they have specific technical characteristics and deserve their own definitions. Anderson et al. (1999) defines hydropower plants as mini hydro if their installed capacity is between 100 kW and 1 MW, micro hydro if their installed capacity is between 5 kW and 100 kW and pico hydro if the installed capacity is less than 5 kW [41].

2.7 Hydropower in Turkey

Turkey is a developing country. Thus energy demand, especially for electricity, is growing rapidly in Turkey. Homebred energy resources by which energy can be generated by utilizing the resources found in Turkey itself are mainly hydro and lignite. Most of the renewable energy resources in Turkey could be used due to the country's advantageous geographic location. The country lies on a warm and humid climatic belt. Hydropower can majorly be utilized to obtain energy in the eastern part of the country. Since Turkey has no big oil and gas reserves and Turkish electricity supply is mainly dependent on gas from Russia and Iran, an important part of the resources that is used for energy generation is imported from foreign countries. This predicament exposes Turkey to price fluctuations and makes it increasingly dependent on imports [3]. Being a country that has no nuclear power yet, Turkey has to produce electricity by utilizing thermal power plants. In those thermal power plants, mainly, natural gas, coal, fuel oil, and lignite are consumed to generate energy [13]. The fuel mix development in Turkey from 1970 to 2010 is given in Figure 2.7. The expansion of gas-based generation is mainly driven by the private sector as gas-based generation is characterized by relatively low capital costs and relatively high fuel costs compared with other electricity generation techniques.

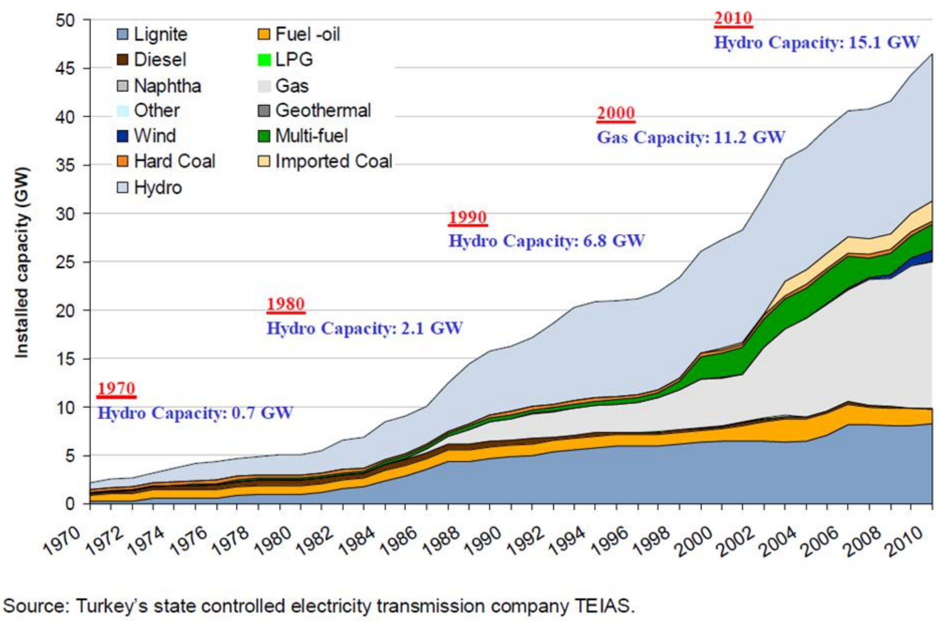
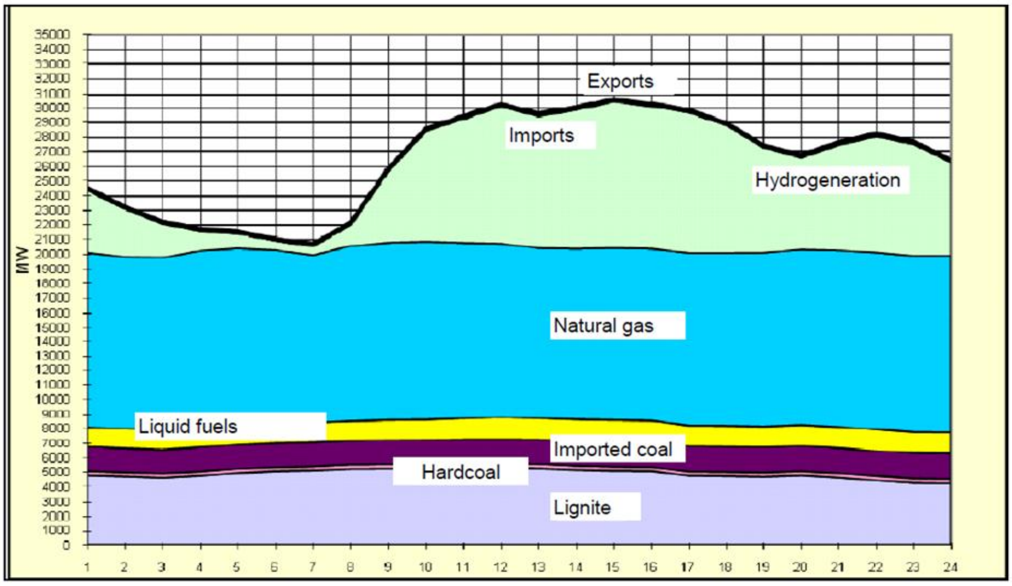


Figure 2.7 Fuel mix development in Turkey, 1970-2010 [42]

Figure 2.8 shows the different generations by fuel type on the day of peak demand in 2008. As can be seen when the demand is highest, hydro generation is utilized in order to keep the electricity prices down.



Source: Turkey's state controlled electricity transmission company TEIAS (the most recent available data)

Figure 2.8 Generation by fuel type on the day of peak demand (23 July) in 2008 [42]

According to long-term records, the annual average precipitation in Turkey is nearly 643 mm. This amount of precipitation corresponds to a volume of almost 500 km^3. The values of the average runoff coefficient and the annual runoff are 0.37 and 186 km^3 (2400 m^3/ha), respectively. The annual consumable water of Turkey can be calculated by subtracting the minimum stream flow requirement for pollution control, estimated water rights of neighboring countries, and topographic and geologic constraints from the aforementioned annual runoff value. The outcome emerges to be 107 km^3 [43].

Turkey is taking the issue seriously and has very concrete plans to develop its hydropower potential even further. To gain benefits from international cooperation in hydropower development, a number of bilateral agreements have been signed with various countries. Also schemes built on the build-own-transfer (BOT) concept have been encouraged recently [44].

It is aimed to utilize all technically and economically viable hydropower by the Turkish government by 2023, the 100th anniversary of the foundation of the Republic of Turkey.

With a varied topography and 26 river basins, Turkey has 1% and 16% of the world's and Europe's theoretical hydropower potential respectively [5].

It is possible in Turkey to attain the desired power with relatively smaller flows and develop relatively higher heads economically thanks to the abundance of the hilly regions throughout the country. Flows may be diverted at the top of a waterfall in order to obtain the highest drop via constructing a relatively simple diversion weir. Extensive investigations are being carried out for the sake of small and large hydropower development of Turkey and for putting this goal into practice; many small hydropower plants are still under construction [45].

Currently, 444 hydropower projects with a capacity over 3 MW are under development [5].

According to General Directorate of State Hydraulic Works (DSİ), Turkey has an annual total gross hydropower potential of 433 TWh, technically feasible hydropower potential of 216 TWh, and economically feasible hydropower potential of 140 TWh per year. It is also estimated by the General Directorate of State Hydraulic Works (DSİ) that the economically feasible potential will be increased to 150 TWh/year due to the projects developed by the private sector. According to the Turkey Water Report 2009, only 35% of Turkey's economically feasible potential is utilized [46].

On the other hand, a study carried out by Bakır (2005) reveals a new criterion which could be utilized in determining the economically feasible hydropower potential in Turkey. In the developed method not only some undervalued and even ignored benefits of hydropower plants but also some overvalued benefits of thermal power plants are taken into consideration. As a result of that study, the economically feasible hydropower potential of Turkey and the installed power are determined as 188 TWh/year and 55.1 GW, respectively [47].

2.7.1 Electricity Market in Turkey

The electricity market of Turkey is mainly based on balancing the energy demand and supply. The aim of this balance is to ensure the supply security and increasing energy quality. In the Turkish electricity market, the bulk of the electricity demand is expected to be covered by bilateral agreements (bilateral contracts). Bilateral contracts are the commercial contracts between license holder legal entities and real persons or legal entities or among license holder legal entities for the purchase and/or sale of active electricity under the provisions of civil law (Electricity Market Balancing and Settlement Regulation). In a bilateral contract, an agreement via which a supplier ensures that it will provide the necessary energy which is demanded by a specific consumer is on the nail. Turkish electricity market hybrid system combines those bilateral agreements with day-ahead and real-time balancing mechanisms, as well as settlement systems for imbalances [48].

The operation style of the Turkish electricity market changed in the first day of December of 2009 after the acceptance of a new regulation and its publication in the official gazette with the date of April 15, 2009 with the number of 27200. The name of the new regulation is "Electricity Market Balancing and Settlement Regulation". Before the amendment of the old regulation, the bids had been made only twice in a month but in the new regulation it is stated that participants of the system could make bids for every individual hour. With the new system, two main types of markets emerged, the day-ahead market and intra-day market (also known as hour-ahead market, adjustment market). In the very beginning of the process, electricity suppliers and consumers are wanted to declare their bids for each hour of the following day. After collecting the bids, the day-ahead market comes into picture. In this market, it is checked whether the supply – demand balance is maintained based on the bids. If the balance is maintained, the generated prices at the intersection of balance of demand and supply are accepted as the unit energy costs of the related hour. Otherwise, the balance is strived to be caught in the hour-ahead market within the relevant day. In other words, the balance of the system is tried to be maintained by pricing energy for each hour of the day, twice. These prices are called as day-ahead market and real time prices.

In this thesis, the income obtained by selling the electricity is to be determined in economic analysis. The day-ahead market prices and real-time prices have been recorded and published by the Market Financial Settlement Center since July 1st, 2009. For the sake of designation of the energy sale price, a 2-year period of day-ahead market prices is taken into account and average price generated in each hour of a day is calculated. By doing this, it is intended to determine the peak hours of a day in which the electricity sale prices are of the highest. Yet, the intended storage facility for the Birkapılı HEPP is designed, keeping in mind that the storage would satisfy the necessary water to the power plant for energy generation in the peak hours. Thus the profitability of the facility can be maximized. Trying to provide enough water to generate electricity during peak hours comprises the studies of determining the storage size of the facility. As a result, knowing the duration of peak hours is a major factor in deciding on the dimensions of the dam body and the location on which it is to be constructed.

2.7.2 Electricity Demand in Turkey

Currently, the total electricity generation capacity in Turkey is 46.5 GW (September 2010). As Turkey has vast hydropower resources, 34% of installed generation capacity is satisfied by hydropower today. The shares of resources to generate electricity in Turkey are given in Figure 2.9. In Turkey, there are no nuclear power stations yet but by the year 2020, it is being planned to possess up to 4.5 GW of nuclear capacity.

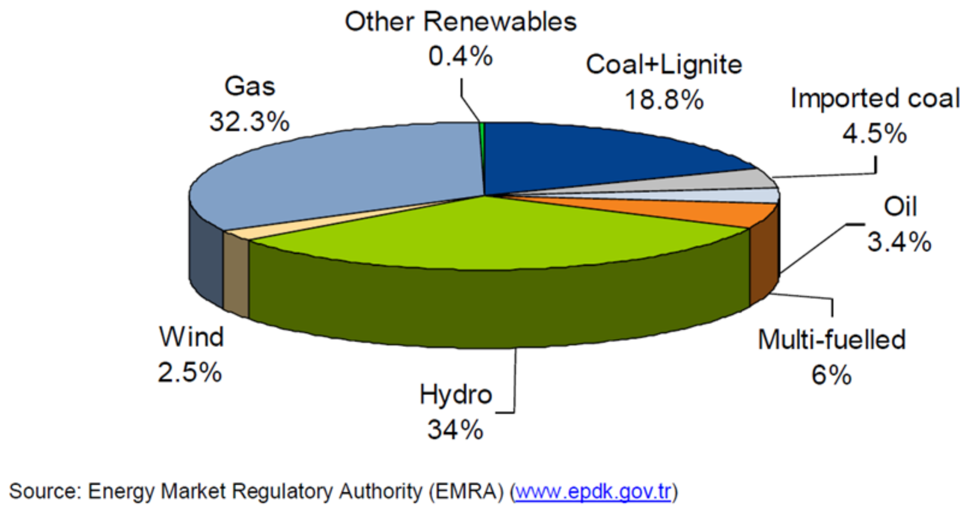


Figure 2.9 Turkish installed generation capacity by fuel (September 2010) [5]

It is expected that the demand for electricity will continuously and rapidly increase in the future. A high and low estimate has been made by the Turkey's state controlled electricity transmission company TEİAŞ and it is forecast that the demand will increase by 4.5 – 7.5% on average annually until 2018. The electricity demand projections for Turkey up to 2018 are given in Table 2.1. Even though an abate in electricity consumption observed in late 2008 and in 2009 due to the financial crisis felt by the vast of the world, demand for electricity is still expected to grow by 130 - 155 TWh up to 2018 [5].

Table 2.1 Electricity demand projections for Turkey to 2018

	Demand				Growth rate (%)			
	Peak (GW)		Volume (TWh)		Peak (GW)		Volume (TWh)	
	High case	Low case	High case	Low case	High case	Low case	High case	Low case
2005	25,2		160,8		7,2%		7,2%	
2006	27,6		174,6		9,6%		8,6%	
2007	29,2		190,0		6,0%		8,8%	
2008	30,5		198,1		4,3%		4,3%	
2009	29,9		194,0		-2,0%		-2,1%	
2010	31,2	31,2	202,7	202,7	4,5%	4,5%	4,5%	4,5%
2011	33,3	33,0	215,9	213,9	6,5%	5,5%	6,5%	5,5%
2012	35,8	35,2	232,1	228,2	7,5%	6,7%	7,5%	6,7%
2013	38,5	37,5	249,5	243,5	7,5%	6,7%	7,5%	6,7%
2014	41,3	40,0	268,2	259,8	7,5%	6,7%	7,5%	6,7%
2015	44,4	42,7	288,3	277,2	7,5%	6,7%	7,5%	6,7%
2016	47,7	45,5	309,7	295,5	7,4%	6,6%	7,4%	6,6%
2017	51,3	48,6	332,6	315,0	7,4%	6,6%	7,4%	6,6%
2018	55,1	51,8	357,2	335,8	7,4%	6,6%	7,4%	6,6%

Source: Turkey's state controlled electricity transmission company TEIAS (June 2009). www.teias.gov.tr

CHAPTER 3

DESCRIPTION OF THE EXISTING HYDROELECTRIC POWER PLANT

3.1 Initiation of the Project

The autoproducer contract for Birkapılı Hydroelectric Power Plant (HEPP) is signed by ERE Hydroelectric Co. and the Ministry of Energy and Natural Resources on November 24, 2000. Then the project is decided to be established under free electricity market conditions after the energy market law (No: 4628) comes into effect. A 40-year length of generation license is issued by the Energy Market Regulatory Authority (EMRA). After the completion of the construction, preliminary acceptance of the facility is done on March 11, 2004 and commercial operation is started. Due to urgency of the project, Birkapılı HEPP is constructed without a storage facility.

3.2 Location of Birkapılı HEPP

Birkapılı HEPP is located in İçel Province in the southern Turkey. The closest settlement center to the power plant is Dağpazarı Village of Mut County. Location of Dağpazarı is shown in Figures 3.1 and 3.2.

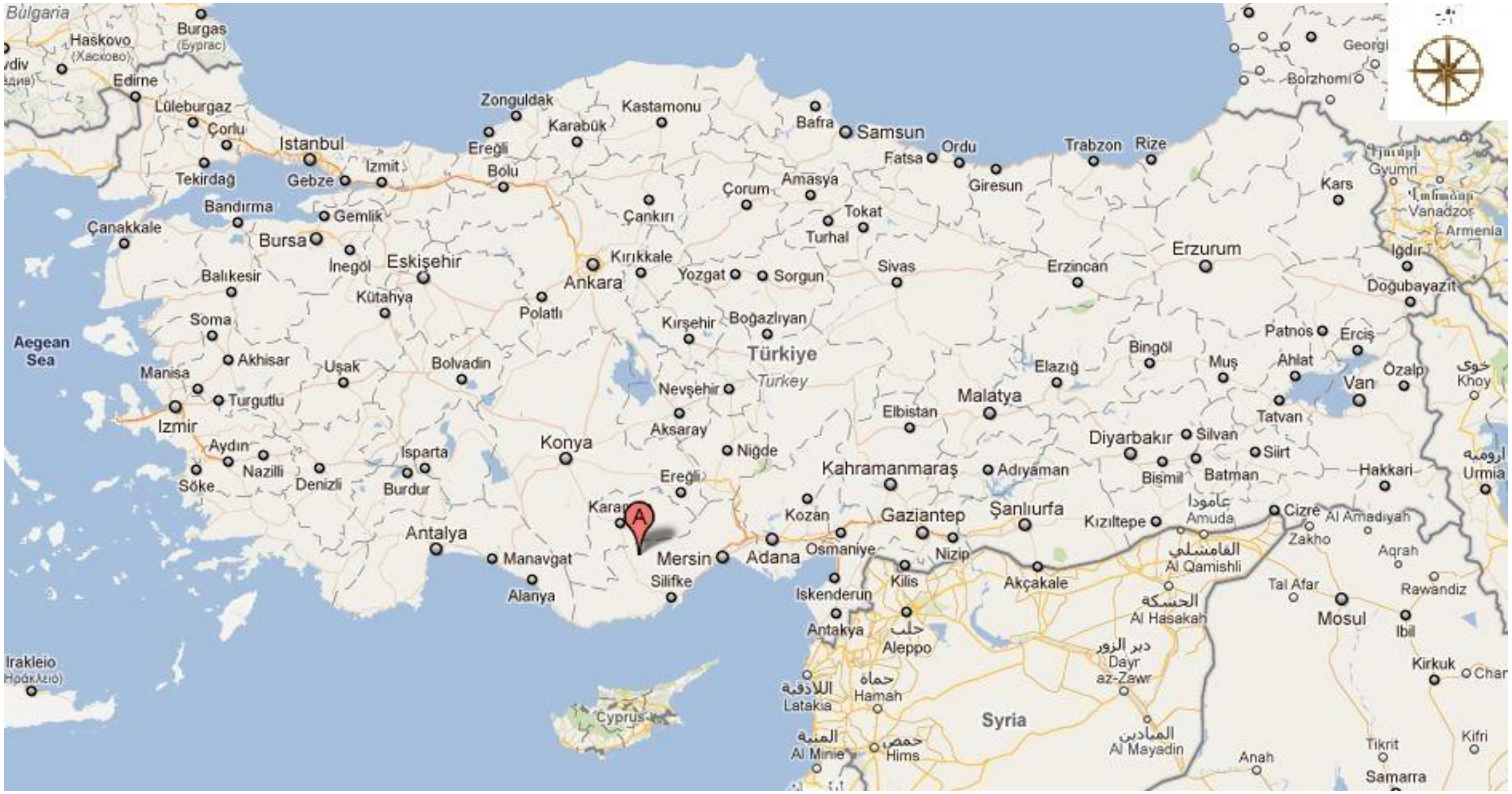


Figure 3.1 Location of the project in Turkey

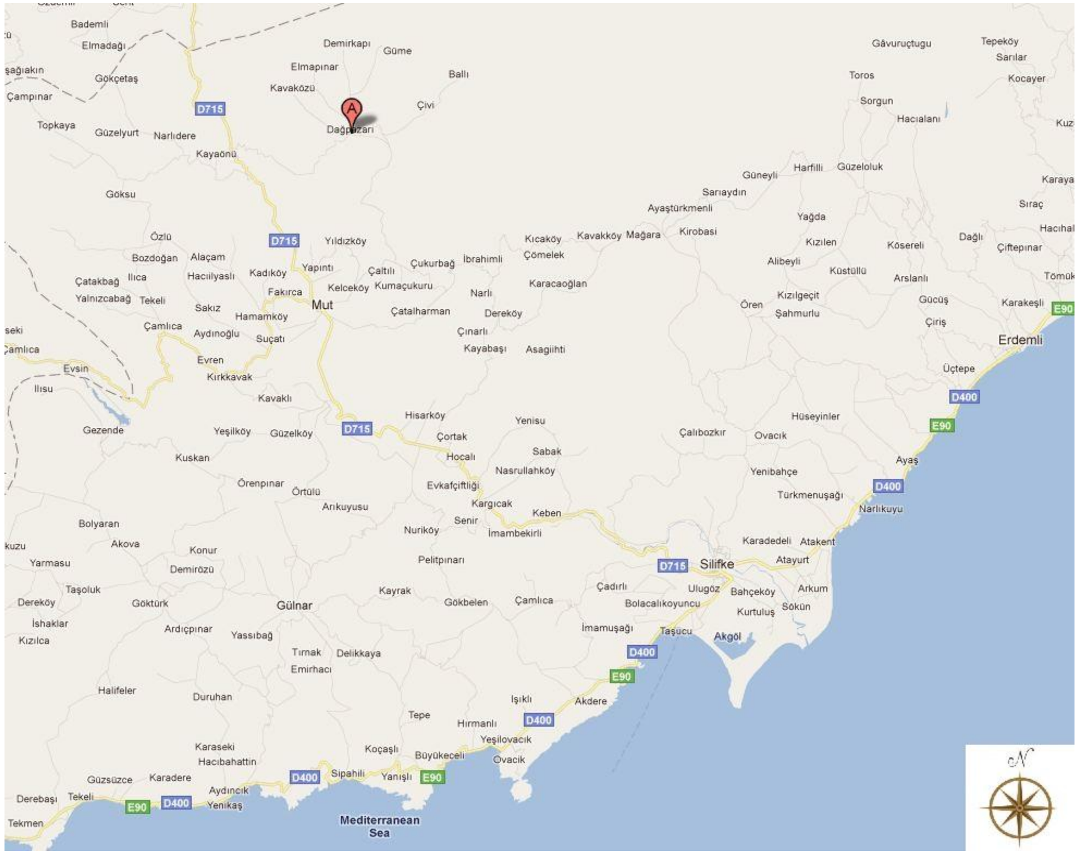


Figure 3.2 Regional map of the project

The topographic properties of the location where the power plant stands can be examined by utilizing O30-b3 map with a scale of 1/25000.

3.3 Existing Facilities

Water utilized by Birkapılı HEPP for energy generation is diverted to the power plant by the help of two diversion weirs. Names of these weirs are Dağpazarı and Söğütözü Weirs. Water diverted by Söğütözü Weir is carried to the drainage basin of Dağpazarı Weir via a canal having a length of 571.08 m. In other words, the drainage basin area of the power plant becomes the total drainage area of these two diversion weirs. Water coming to Dağpazarı Weir is conveyed to the forebay via tunnel and canal systems (each system is composed of two parts). Total lengths of the tunnel and canal systems are around 6300 and 2000 m, respectively. The slope of the tunnel system is 0.0006 and it is 0.0003 for the canal system. Large particles suspending in the flowing water is settled at a settling basin constructed at a suitable location through the conveyance system. After passing through the conveyance system, water reaches to a forebay having a storage volume of 8100 m^3. Then water is released to the steel penstock having a total length of 3483 m and diameter of 1.10 – 1.35 m. Electricity is generated in the power house containing a Pelton turbine having an installed capacity of 48.5 MW. The project discharge is specified as 6.18 m^3/s and the total gross head is 1013 m.

In this study, a storage facility is contemplated to be built just at the upstream of Dağpazarı Weir so that the water released from the storage facility can be regulated by Dağpazarı Weir and transmitted to the power plant. The proposed location of the facility is also suitable geologically and easily accessible for construction. The proposed project location and the existing establishments related to Birkapılı HEPP are shown in Figure 3.3.

Figure 3.3 The proposed location of the storage facility and existing facilities of Birkapılı HEPP

Schemes illustrating the plan and the profile of Birkapılı HEPP are given in Figures 3.4 and 3.5.

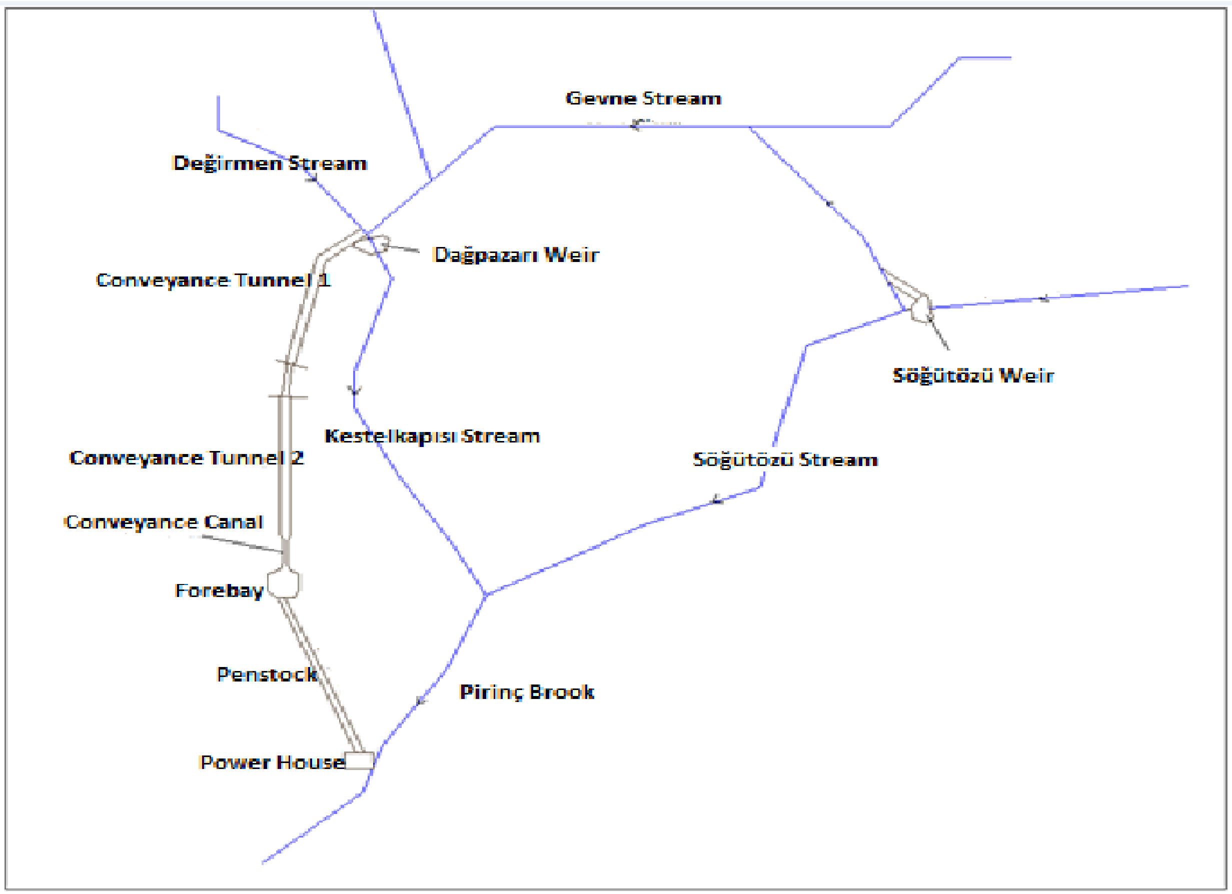


Figure 3.4 Schematic description of Birkapılı HEPP facilities and creeks in the vicinity of the project site (plan)

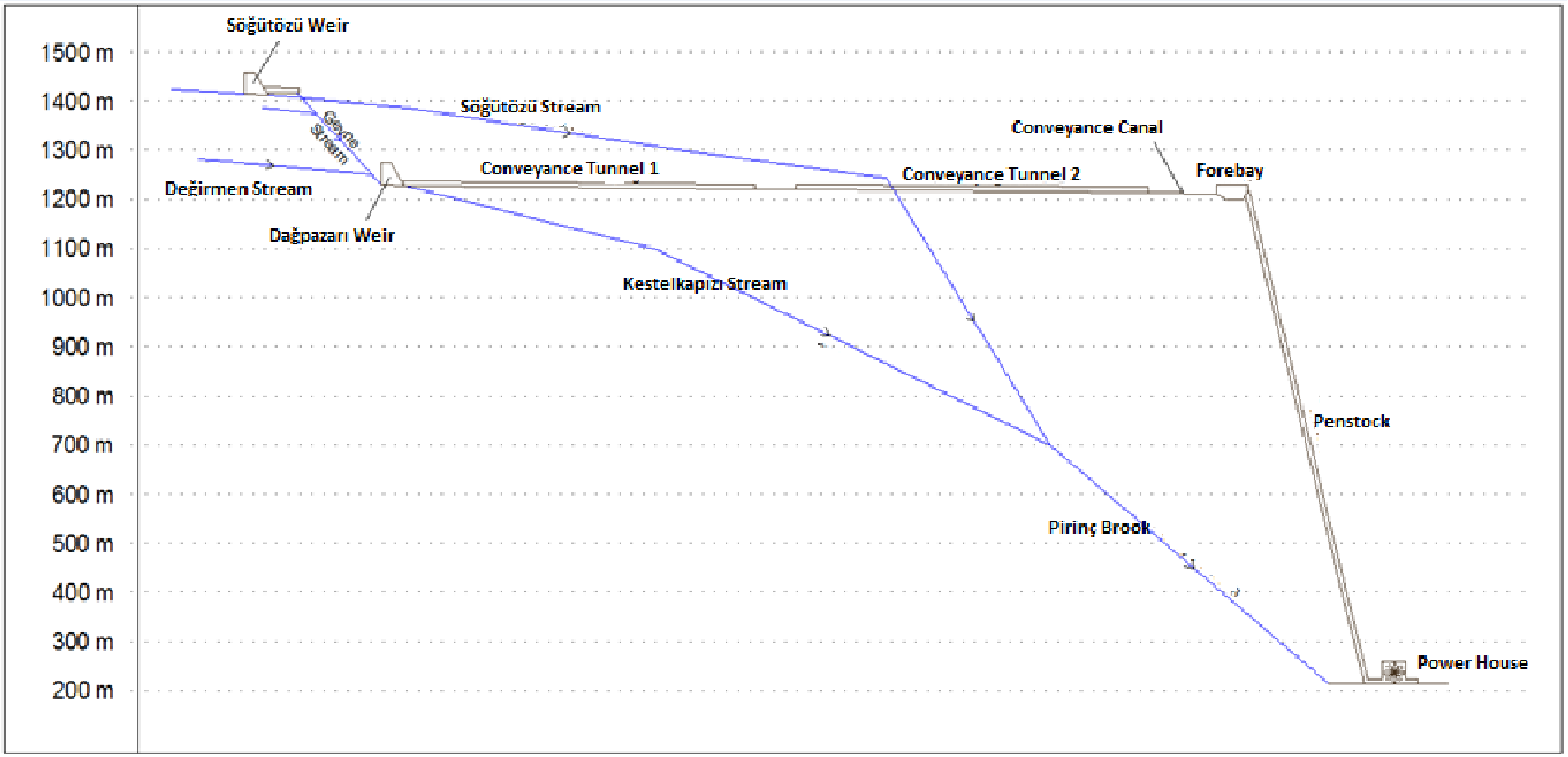


Figure 3.5 Schematic description of Birkapılı HEPP facilities and creeks in the vicinity of the project site (profile)

CHAPTER 4

CASE STUDY

4.1 Introduction

In this chapter, it is aimed to design an additional storage facility for the existing Birkapılı Hydroelectric Power Plant (HEPP) to increase its profitability. Among various dam types suitable for small heights, such as embankment, concrete gravity or roller compacted concrete dam, a concrete gravity dam is assumed to be a reasonable alternative. Because of limited construction space, extra construction equipment, such as static or vibrating rollers, will not be required for a concrete gravity dam. This may provide acceleration in construction period. Based on the oral communication with the official design authorities, the dam site location is also observed to be suitable geologically for the construction of a concrete gravity dam. A sluiceway is to be utilized for sediment evacuation to the downstream. It will also operate as a bottom outlet to discharge the required amount of water continuously to the power plant. The facility has an uncontrolled overflow spillway placed in the middle of the dam body. The width of the facility is chosen to be 50 m with reference to local conditions. Design steps which include flow analysis, flood frequency analysis, stability analysis, and the operation study are provided in the subsequent sections.

The data needed to generate the area-elevation-storage curves are given in Table 4.1. The generated area-elevation-storage curves of the proposed facility can be seen in Figure 4.1.

Table 4.1 Data of the area-elevation-storage curves

Elevation (m)	Storage (m^3)	Area (m^2)
1265.00	24286.91	16224.78
1265.50	34176.12	20191.89
1266.00	44065.32	24159.00
1266.50	59023.90	29707.52
1267.00	73982.47	35256.03
1267.50	94880.29	41648.56
1268.00	115778.11	48041.09
1268.50	143537.84	55235.76
1269.00	171297.56	62430.42
1269.50	210768.78	74205.79
1270.00	250240.00	85981.16
1270.50	297964.89	94966.23
1271.00	345689.78	103951.30

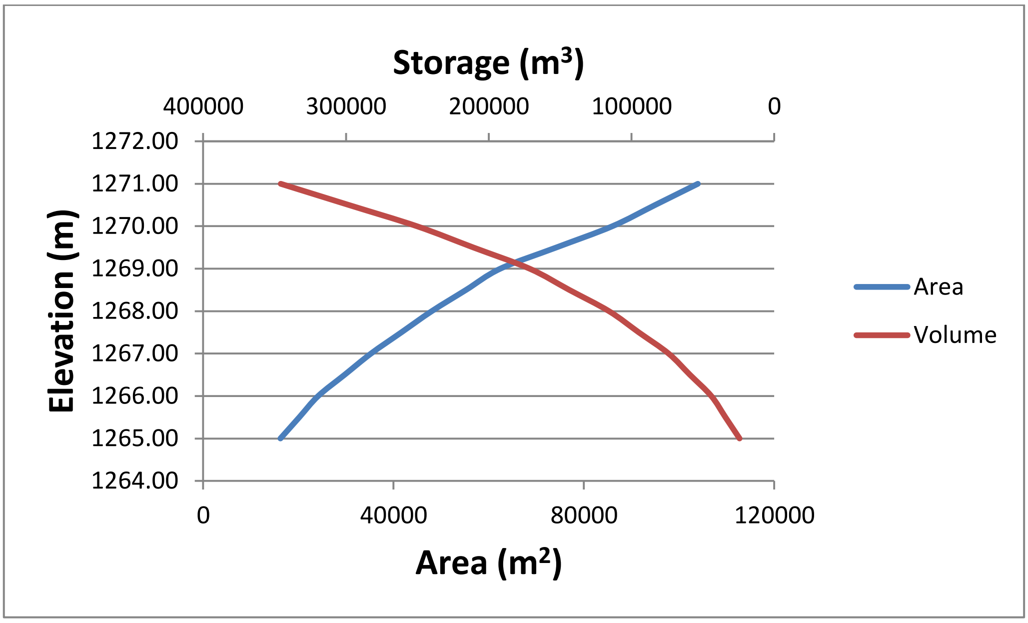


Figure 4.1 Area-elevation-storage curves

4.2 Flow Analysis

This study has been carried out to designate the average water discharge coming to the project site. The flow data obtained from two stream gauging stations, namely 1720 Göksu River - Hamam and 17-51 Pirinç Brook – Gençali Bridge, are utilized during the study. These stations have drainage areas of 4304.00 km^2 and 641.80 km^2, respectively. 17-51 Pirinç Brook – Gençali Bridge station is located in the vicinity of the project site and has a flow data of 4 years (2002, 2004, 2005, and 2006). The flow data recorded in these years are obtained from the State Hydraulic Works (DSİ). 1720 Göksu River – Hamam station is used for the correlation studies carried out to extend the data belonging to the nearby station (17-51 Pirinç Brook – Gençali Bridge) to the project site. 1720 Göksu River – Hamam station has flow data of 41 years, from 1966 to 2006. Since this station is in the same basin with the project site and has enough flow data recorded to carry out the correlation studies, it is preferred to be used for obtaining the flow data for the project site. The necessary data for the flow analysis is elicited from the General Directorate of Electrical Power Resources Survey and Development Administration (EİE) and used in the study.

A correlation study has been undertaken to extend the aforementioned data belonging to 17-51 Pirinç Brook – Gençali Bridge stream gauging station. In the original studies of Birkapılı HEPP, the flow data of 17-20 Pirinç Brook – Yapıntı stream gauging station was correlated to 17-11 Efrenk Brook – Hamzabeyli stream gauging station which is relatively far from the proposed project site and belongs to a neighboring basin (Bird's eye distance between the stations used in the correlation studies is measured as 500 km from Google Earth). In this study, a different approach is adopted and 1720 Göksu River – Hamam stream gauging station is chosen to be a candidate for the correlation study. This stream gauging station has a drainage basin area of 4304.00 km^2 and possesses a flow data of 41 years, between 1966 and 2006. The bird's eye distance between 17-51 and 1720 stations is found as 10 km. Since these two stations are in the same basin and very close to each other, there are no concerns about the hydrological accordance of stations' basins. Therefore, a correlation study is conducted with confidence.

The locations of the stations which are utilized in this study are illustrated in Figure 4.2.

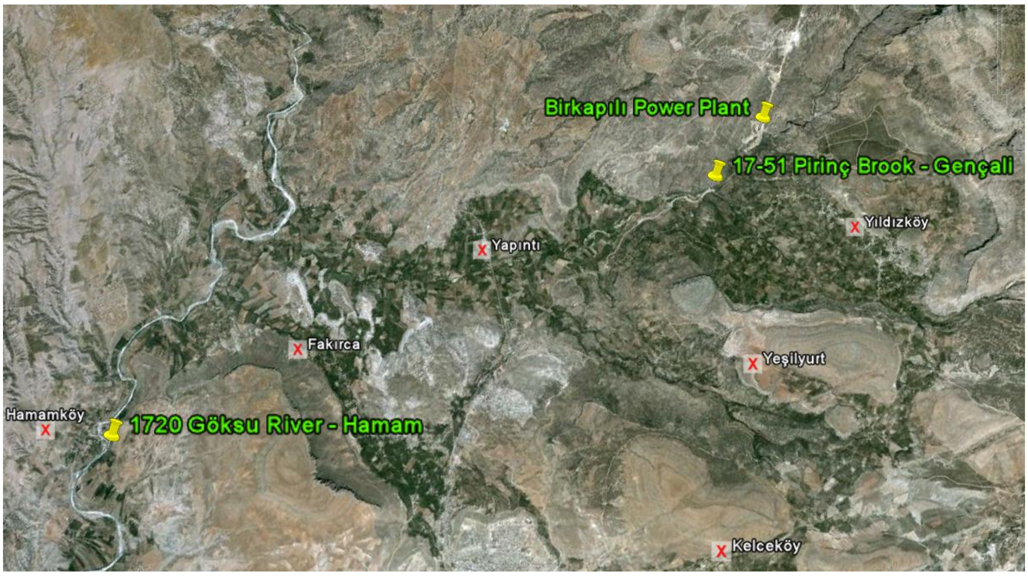


Figure 4.2 1720 and 17-51 stream gauging stations

4.2.1 Correlation Study

In order to determine the flow data belonging to the project site, a correlation study is conducted. The steps that are followed for this study are explained as follows:

- As the first step, the original flow data of the related stream gauging stations are gotten from the related organizations.
- A 4-year of common period of flow data belonging to both 1720 Göksu River – Hamam and 17-51 Pirinç Brook – Gençali Bridge stations referring to the years 2002, 2004, 2005, and 2006 are selected and assorted to get a correlation chart.
- The flow data belonging to 17-51 Pirinç Brook – Gençali Bridge and 1720 Göksu River – Hamam stations are located in the y and x axes of the graph, respectively (See Figure 4.3). A best-fit curve is drawn

regarding all the flow data and a regression equation is obtained. It can be seen that a relatively high correlation coefficient (R^2 = 0.751) is obtained (The correlation coefficient of the original study which was conducted by investigating the relationship between 17-20 Pirinç Brook – Yapıntı station and 17-11 Efrenk Brook – Hamzabeyli station was, R^2 = 0.706). The regression equation obtained in this study is given as follows:

$$Q_{17-51} = 0.000236Q_{1720}^2 + 0.0563Q_{1720} - 0.871 \quad (4.1)$$

where, Q_i is discharge in m^3/s.

- Using Equation (4.1), the missing data of the 17-51 Pirinç Brook – Gençali Bridge station are filled for 41 years by utilizing the measured flow data of 1720 Göksu River – Hamam station.

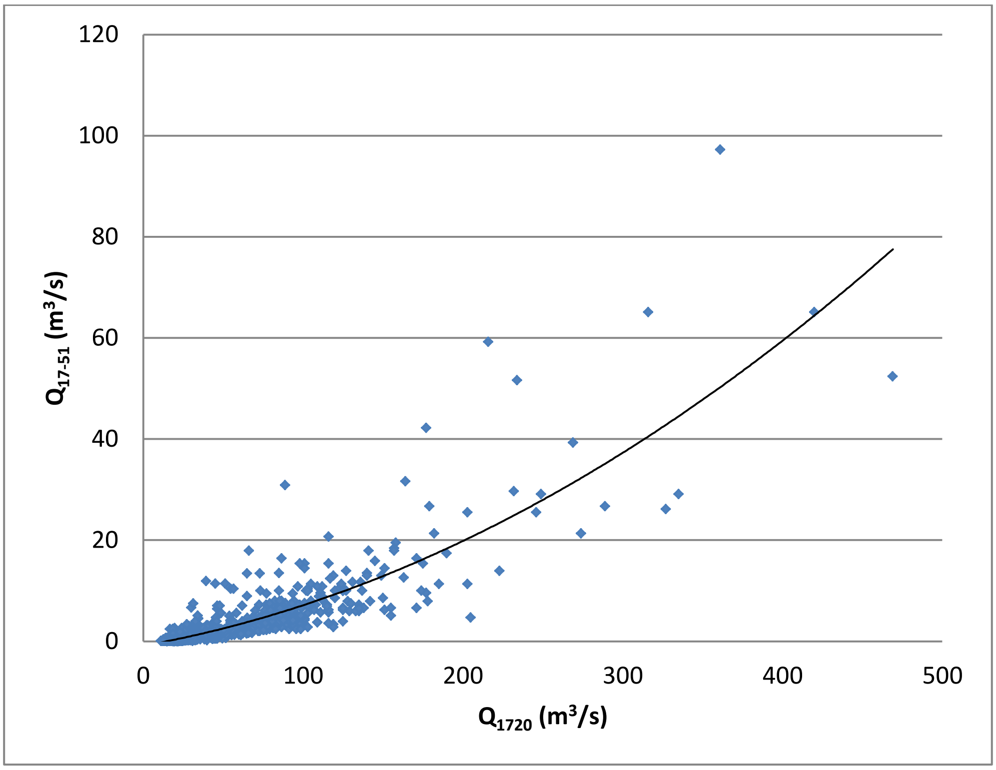


Figure 4.3 Correlation between 1720 and 17-51 stations

- As the next step, the drainage area of the project site is determined. In this study, utilizing the geographic maps representing the vicinity of the project region, the drainage basin of the project site is encircled by following the ridges of the heights. For this study, O32, O33 and O34 maps which all have a scale of 1/25000 have been provided and are utilized. The drainage basin area of the project site is eventually obtained as 416.23 km^2. The drainage basin area is illustrated in Figure 4.4.

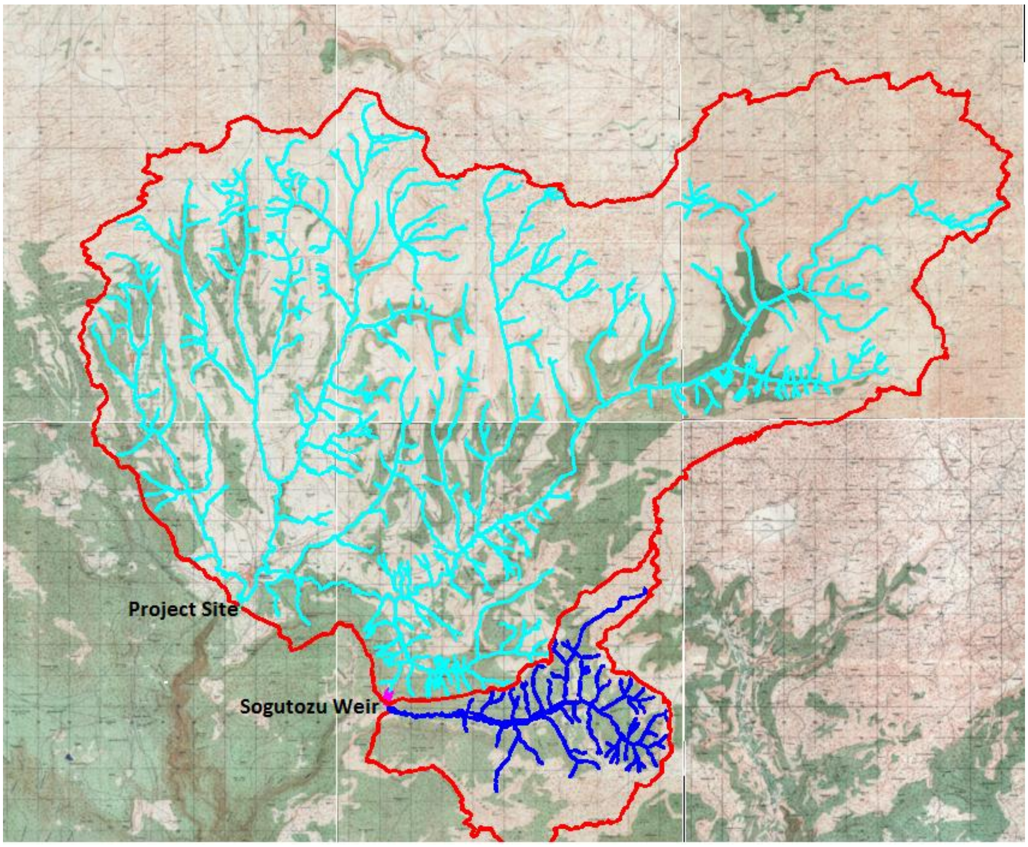


Figure 4.4 Drainage area of the storage facility

- After obtaining the discharge values for 17-51 Pirinç Brook – Gençali Bridge station for 41 years, these values are transferred to the project site by calculating the ratio of the drainage basin areas. As given before, the drainage basin areas of 17-51 Pirinç Brook – Gençali Bridge and the project site are 641.80 km^2 and 416.23 km^2, respectively. Thus the ratio is obtained as 0.649. By multiplying all the data obtained for 17-51 Pirinç Brook – Gençali Bridge stream gauging station by this ratio, daily discharge values of the project site for a 41-year period are obtained.

- The average discharge data of the project site is determined by calculating the average of all daily flow data transferred to the project site by the aforementioned procedure. The average discharge calculated for the project site is obtained as 1.64 m^3/s. Standard deviation of the discharge data is calculated as 4.47 m^3/s.

4.3 Flood Frequency Analyses

Flood frequency analyses are performed to relate the magnitude of flood discharges to their frequency of occurrence. Instantaneous annual peak flow discharge data are utilized in the analysis for calculating statistical information, such as mean values, standard deviations, and skewness. These statistical data are then used to construct frequency distributions. The frequency distributions are graphs and tables which illustrate the likelihood of various discharges as a function of recurrence interval and probability of exceedence. According to the United States Water Advisory Committee on Water Data (1982), the Log-Pearson type-III Distribution is the recommended technique for flood frequency analysis. It is also accepted as a suitable distribution for skewed annual flow data in this thesis.

In this thesis, the design flood is determined by using a spreadsheet widely used by the Turkish hydropower sector and State Hydraulic Works. This spreadsheet can perform flood frequency analysis using various types of flood frequency distributions. To determine the design flood in the vicinity of the project site, instantaneous annual peak flow discharge data obtained from 17-51 station which is in close downstream of the proposed storage facility is utilized. The data are used as inputs in the spreadsheet. The flow data used are given in Table 4.2 in which Q is the annual peak discharge.

Table 4.2 Annual peak discharge data of 17-51 stream gauging station

Year	2002	2003	2004	2005	2006	2007	2008	2009
Q (m^3/s)	129.0	106.5	137.0	59.2	32.3	57.5	68.6	56.0

Floods having return periods of 2, 5, 10, 25, 50, 100, and 500 years are calculated via the aforementioned spreadsheet by utilizing the Log-Pearson type-III distribution based on the available historical record. The discharge values obtained are given in Table 4.3 in which T_r is the return period.

Table 4.3 Calculated flood data

T_r (years)	2	5	10	25	50	100	500
Q (m^3/s)	73.70	110.69	136.03	168.74	193.43	218.26	271.82

The Kolmogorov - Smirnov test indicates that output values given by the program are reasonable. Prediction of discharges corresponding to various return periods is subject to error since the data used in the flood frequency analysis is small. The result obtained by conducting the Kolmogorov - Smirnov test for Log-Pearson type-III distribution is given in Table 4.4.

Table 4.4 Kolmogorov - Smirnov test for Log-Pearson type-III distribution

Distribution Type	P-Value (Theoretical)	P-Value (Ampirical)	Maximum P-Value Δmax	Observation Value (at P-Value)	Significance Percentage				
					0.80	0.85	0.90	0.95	0.99
Log-Pearson Type-III	0.449	0.556	0.106	68.6	OK	OK	OK	OK	OK
NOTE : Log - Pearson Type - III distribution is appropriate									

The flood with a return period of 100 years is used as the design flood in the design of the spillway of the storage facility. This return period is assumed to be reasonable for a small storage facility design.

To be on conservative side, a freeboard of 0.5 m is considered when determining the crest elevation of the dam body under the discharge having a frequency of 500 years. Flow analysis over the spillway crest and face is not carried out.

4.4 Stability Analyses

The design of a gravity dam is performed through an interactive process. This process includes a preliminary layout of the structure. A stability and stress analysis is carried on, considering this layout. In the case that the structure with the chosen dimensions in the preliminary study fails to meet the criteria (explained in detail in the following parts) then the layout is modified and the analysis is carried out again. This process is repeated until the criteria are met and an acceptable cross section is attained [49].

Birkapılı storage facility is designed to have a concrete gravity dam body and an overflow spillway structure. Stability analyses of these components are carried out considering various loads and loading conditions. Concrete gravity dams are proportioned so that their own weights show resistance to the forces that are exerted upon it. This type of dams is designed to take compressive stresses only.

For the overflow spillway of the Birkapılı Hydroelectric Power Plant (HEPP) storage facility, another analysis is carried out to determine the dimensions of the spillway, type of stilling basin and to designate the coordinates of the ogee-shaped profile of the spillway. The design is done in such a way that all safety requirements are satisfied. Stability analysis of the dam body and spillway are conducted using the developed spreadsheets with respect to different load combinations (usual, unusual, extreme, and empty cases).

The following failure modes given should be satisfied for a concrete gravity dam body and spillway under all loading conditions for the desired safety.

- Safety against overturning
- Safety against sliding
- Safety against shear and sliding
- Safety against the stresses in the body and at the foundation

4.4.1 Forces Acting on a Concrete Gravity Dam

The free body diagrams illustrating all possible forces that may act on a concrete gravity dam body and spillway are shown in Figures 4.5 and 4.6, respectively.

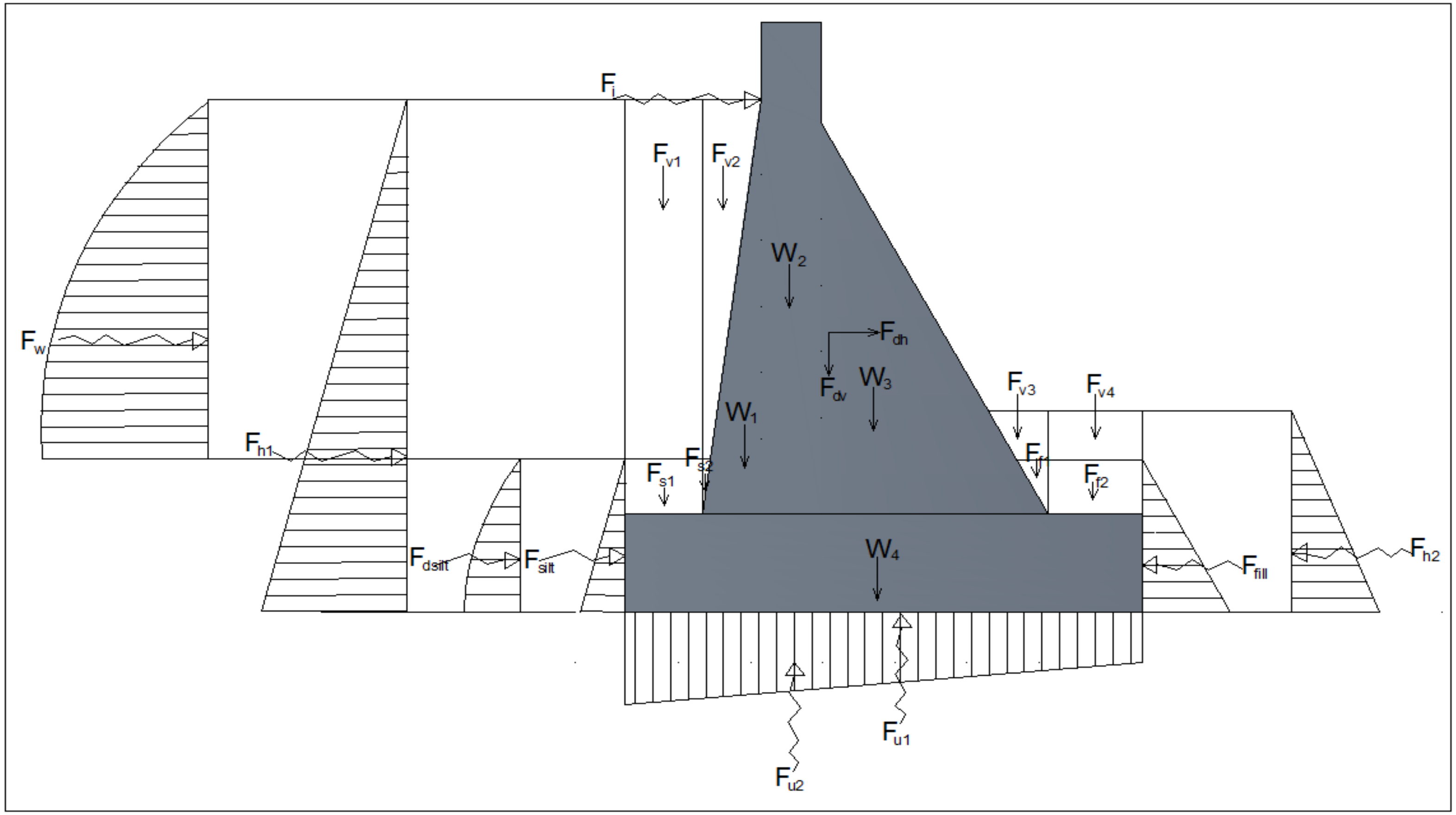


Figure 4.5 Forces developed on dam body

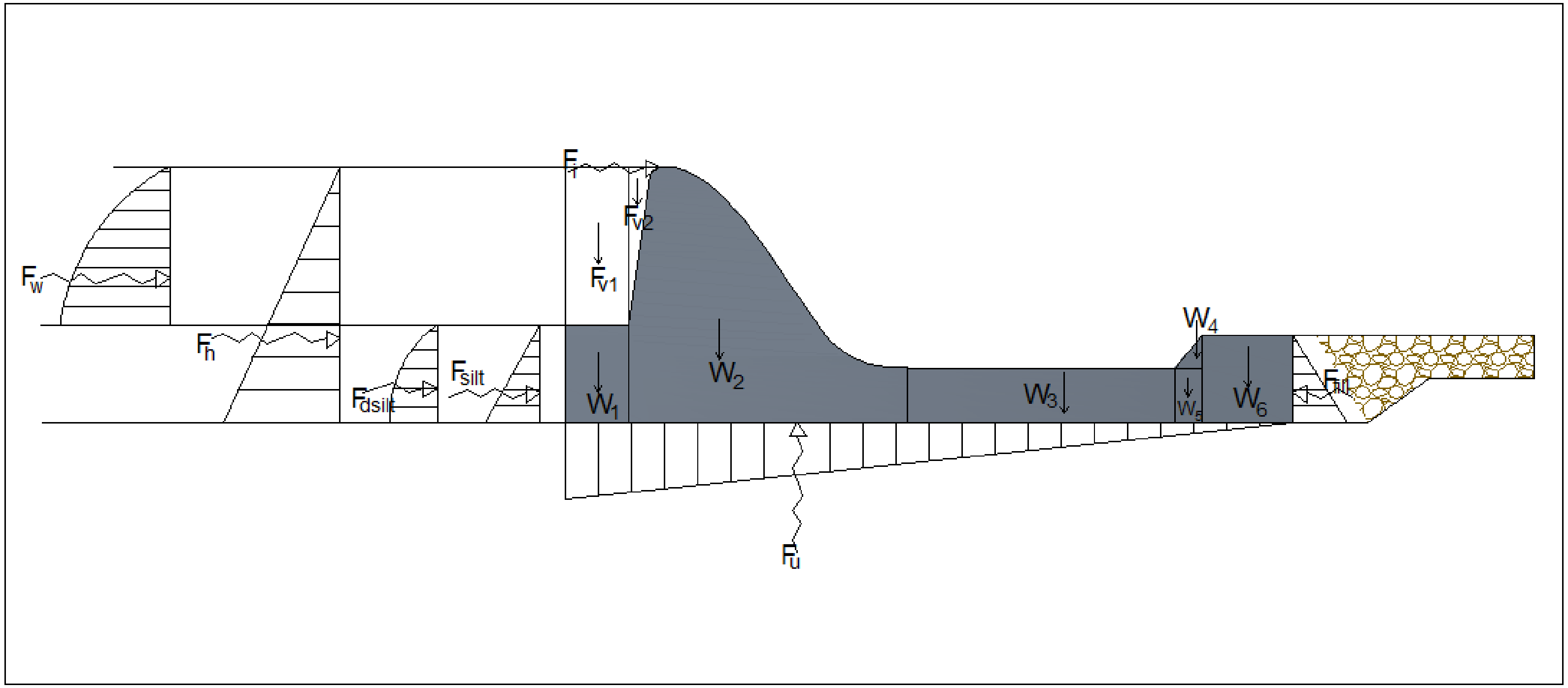


Figure 4.6 Forces developed on spillway

The forces shown in the figures are as follows:

W : Weight of concrete

F_h : Horizontal hydrostatic force component

F_v : Vertical hydrostatic force component

F_u : Uplift force

F_{silt} : Silt force

F_{fill} : Fill force

F_i : Ice force

F_{dh} : Earthquake force (horizontal component)

F_{dv} : Earthquake force (vertical component)

F_w : Dynamic force induced by water during earthquake

F_{dsilt} : Dynamic force induced by the accumulated silt during earthquake

F_f : Weight of fill which is found on the foundation

F_s : Weight of silt which is found on the foundation

In the stability analyses conducted by the developed spreadsheets, dam body and spillway cross-sections are divided into simpler geometrical shapes, such as triangles and rectangles for convenience. Areas and geometrical centers of these shapes can easily be determined. The points on which the forces are exerted (center of gravities) and the moment arms of forces can also be specified easily. The input data used in the analysis are provided in Table 4.5, in which γ_{conc} is the specific weight of concrete, F_i is ice force, and k is seismic coefficient.

Table 4.5 Input data used for stability analyses

Parameters	Value
γ_{conc} (kN/m^3)	24
F_i (kN/m)	100
Uplift force reduction (%)	45
k	0.1

4.5 Stability Criteria

The stability analyses performed in this study are based on the assumption that the gravity dam is composed of some vertical elements which are separated by contraction joints without keyways. In other words, each of the elements comprising the structure cannot transfer any load from or to adjacent vertical elements and only transfers its own load to the foundation [26].

The loading conditions for which the stability analysis is performed in this study are as follows. The forces mentioned in the upcoming parts are illustrated in Figures 4.5 and 4.6 [26].

Usual Loading:

- Hydrostatic force (consider the water depth of a reservoir at normal operating level) (F_h, F_v)
- Uplift force (F_u)
- Temperature stresses (for normal temperatures) (F_{tnor})
- Silt and fill loads (F_s, F_{fill})
- Ice load (F_i)
- Dead loads (W)

Unusual Loading:

- Hydrostatic force (at full upstream level) (F_h, F_v)
- Silt load (F_s)
- Uplift force (F_u)
- Temperature stresses (for minimum temperatures at full level) (F_{tmin})
- Dead loads (W)

Extreme or Severe Loading:

- All forces taken into account in usual loading
- Earthquake forces (F_{dh}, F_{dv})

The modes of failure are overturning, sliding over any horizontal plane, the shear-sliding in the dam and at foundation level, and stress distribution. The dam should have the ability of resisting the applied loads and should be safe against the aforementioned failure modes. Some safety factors are developed sufficiently beyond the static equilibrium condition in order to offset the uncertainties in the loads. In Table 4.6, minimum recommended safety factors are given for different failure modes and loading combinations. In this table, FS_o, FS_s, and FS_{ss} stand for overturning safety factor, sliding safety factor, and combined shear and sliding safety factor, respectively. Being on the safe side, the safety factor values which are used for usual case are assumed to be the same for empty case.

Stability calculations belonging to the dam body and the spillway of the best alternative are given in Appendices B and C.

Table 4.6 Safety factors for concrete gravity dams [50, 51, 52]

Loading Type	FS_o	FS_s	FS_{ss}
Usual	2.0	1.5	3.0
Unusual	1.5	1.2	2.0
Extreme	1.2	1.0	1.0
Empty	2.0	1.5	3.0

4.6 Design of the Spillway

Detailed information related to the design of an overflow spillway is depicted in the following paragraphs.

Abutments at the crest is considered to be rounded where the radius is greater than half of the total head over the spillway crest and head wall is placed less than or equal to 45^0 to the flow direction. The spreadsheets consider modifications for spillway discharge with reference to upstream face inclination, variable heads, apron level, and submergence.

Birkapılı HEPP storage facility has a spillway with an inclined upstream face to maintain the desired stability.

The profile of an ogee-crested spillway with a vertical upstream face can be determined by following the guidelines given by USBR (1987).

Downstream part of spillway is given by the standard USBR parabolic relation. The parabolic part of spillway is followed by a straight portion having an approximate inclination of 1V : 0.6H, where V and H stand for vertical and horizontal values of inclination. This slope and extension of the straight part depends on the stability criteria. The straight portion of the downstream face is joined to the apron by a curvature having a radius of nearly 25% of the spillway height [26]. By doing this, it is intended to offset the effect resulting from centrifugal pressures developed at the toe of the overflow spillway. These pressures may exert an important amount of stress on the spillway sidewalls. In order to avoid separation of water at the top of the spillway and cavitation damage, the upstream face of the crest is formed by smooth curves.

The spreadsheets developed in this study are also capable of designing standard USBR types of stilling basins.

After the calculation of the supercritical flow depth, the subcritical flow depth observed after the hydraulic jump can be computed by using the momentum equation. The corresponding stilling basin can then be designed.

4.7 Operation Study

In this study, an operation study is carried out to find the profitability of the new alternative that is presented for Birkapılı HEPP. From Silifke and Mut state meteorological stations, average evaporation and precipitation amounts are obtained as 111 and 35 mm/month, respectively. To be on the safe side, the water surface area which can be measured when the reservoir is full (85981.16 m^2) is taken into consideration. By multiplying water surface area by the data obtained from the related meteorological stations, monthly evaporation and precipitation volumes are obtained as 9544 m^3 and 3009 m^3, respectively. The monthly average incoming water volume to the reservoir is 4250880 m^3. Ratios of monthly evaporation and precipitation volumes to the monthly average inflow volume can be obtained as 0.00225 and 0.00071, respectively. Since the ratios are too small, effects of evaporation from and precipitation to the reservoir surface are neglected in the operation study.

The algorithm of the spreadsheet prepared for the calculations is explained in detail in the following sections. The used parameters are expressed and the operations done in each column are depicted one by one.

<u>Reservoir Storage Capacity (S)</u>

It is the water quantity that can be stored in the reservoir. Its unit is cubic meters (m^3).

Off-Peak Period (T_{opeak})

It is the time interval in which the power demand is not that so much and the electricity selling prices are relatively low. The unit of this parameter is used as hours (hr).

Peak Period (T_{peak})

It is the time interval in which the power demand is high and the electricity selling prices are higher than other times. This parameter’s unit is hours (hr).

The Design Discharge (Q_d)

It is the discharge with which the hydroelectric power plant is run optimally. The unit of the parameter is cubic meters per seconds (m^3/s). The design discharge of Birkapılı HEPP is 6.18 m^3/s.

Net Head of the Power Plant (H_{net})

Net head can be defined as the outcome which emerges from subtracting the head losses from the gross head value. The net head value used in the operation study is directly obtained from the existing power plant as 950.73 m.

Energy Generation (E)

In the operation study, the amount of electricity that is generated using the available water is calculated. The unit is kWh.

Date

It connotes the days for which the discharge values are obtained. This study is undertaken for a period of 41 years, starting with 01.10.1965 and ending in 30.09.2006.

Incoming Discharge (Q_{inc})

It is the discharge values that are calculated via the daily flow analysis for the site on which the project is intended to be implemented. The unit of the discharge values used in analysis is cubic meters per second (m^3/s).

Incoming Discharge Volume (V_{inc})

It is the total water volume that reaches to the project site in a complete day. In the calculation of the incoming discharge volume for each day, incoming discharge values are utilized.

Initial Reservoir Volume (V_i)

The water volume presents in the reservoir at the beginning of each day is defined as the initial reservoir volume. For the first day of the operation study, water amount existing in the reservoir is assumed to be the half of the reservoir capacity. For the subsequent days, it is the same value as the final reservoir volume of the previous day.

Minimum Water Volume Released (V_{rmin})

Some part of the water reaching to the project site is thought to be released to the creek without being utilized for energy production. This is the water amount which is needed to maintain the ecologic life of the creek on which the project is implemented. By using the State Hydraulic Work’s approach, the minimum amount of water to be released to continue the ecologic life is calculated as the one tenth of the average daily discharge values belonging to the last ten years of the discharge data. The unit of the life water is taken as cubic meters per second (m^3/s) and it is converted to volume unit, cubic meters (m^3), in the calculations.

In the case that the incoming discharge is greater than or equal to the minimum discharge to be released, the minimum discharge is released to the creek. Water volume released in a whole day is calculated using this discharge. However, if the water discharge coming to the project site is smaller than the minimum discharge to be released for maintaining the ecologic life in the creek, all coming discharge is released to the creek. In other words, no water is used to generate energy.

Water Volume Stored (V_S)

This quantity reflects the water amount available that can be stored in the reservoir throughout each day considered in the operation study. The logic behind the algorithm is explained in the subsequent paragraphs.

If the discharge volume coming to the reservoir in a whole day time is smaller than or equal to the minimum water volume to be released, then the water volume available to be stored in the reservoir is zero. In other words, it is impossible to store water in the reservoir unless the incoming discharge volume in a day is enough for the obligatory water amount to be released to the creek.

In the case that the incoming water volume in a day is greater than the minimum volume to be released to the creek, two different approaches are made to calculate the water volume stored in the reservoir in that day.

In the first case, it is thought that the difference between the incoming discharge volume to the reservoir and minimum water amount to be released is smaller than or equal to the difference of reservoir storage capacity and the initial reservoir water volume in a day. Herein, reservoir storage capacity is the maximum amount of water to be stored in the reservoir. In this situation, the water amount to be stored in any given day is calculated by deducting the minimum amount of water to be released to the creek from the volume of water coming to the project site.

In the second case, the difference between the reservoir storage capacity and water volume existing in the reservoir at the beginning of any day gives the water amount available to be stored in the reservoir in that day.

Water Volume Used to Generate Energy in the Off-Peak Period (V_{opeak})

This parameter is calculated to find out the water amount which is diverted to the turbines to generate energy in the off-peak period. The unit used for this parameter is cubic meters (m^3).

In the condition that sum of the water volume existing in the reservoir at the beginning of any day taken into account in the operation study and the volume stored in the reservoir throughout that day is smaller than the reservoir storage capacity, there will be no water available for energy generation in the off-peak period. Nonetheless, all available water is to be kept in order to be used in peak hours. For the reverse condition, namely the mentioned sum is greater than or equal to the reservoir storage capacity, there are two cases that is taken into consideration.

If the difference of the summation of the minimum water volume to be released to the creek and the water amount stored in the reservoir in any day and the incoming discharge volume is equal to or smaller than the water amount used for energy production at the design discharge then the calculated water volume is used for energy production in this period. Otherwise, the water volume that is used to generate energy in the off-peak period is the water volume needed to generate energy with the design discharge throughout the off-peak period.

Water Volume Used to Generate Energy in the Peak Period (V_{peak})

By calculating this parameter in the operation study, the water amount which is needed to generate electricity in peak period is obtained in cubic meters (m^3).

In order to determine peak period for energy production, unit electricity market prices of a 2-year period obtained from Market Financial Settlement Center are utilized. As a consequence of the analysis carried out using the mentioned data, number of hours in peak period and the average electricity selling unit price is obtained.

If the water amount that is needed to generate electricity in the determined peak period with the design discharge is smaller than or equal to the summation of the initial reservoir volume and the daily stored water volume in the reservoir, then water volume needed to generate energy in peak period will be the water amount needed to run the system with the design discharge throughout the peak period.

Otherwise, the water volume used to generate electricity in peak period will be the sum of the water amount stored in the reservoir in any day and initial reservoir volume of that day.

Maximum Available Water Volume (V_{max})

This parameter refers to the maximum amount of water which can be diverted from the creek and used in energy production. The maximum available water amount can be found by subtracting the water volume existing in the reservoir at any day from the summation of the minimum water volume to be released to the creek, the water amount needed to produce electricity throughout the day with the design discharge and the reservoir capacity. The unit used for this parameter is cubic meters (m^3).

Water Volume Released (V_r)

This parameter implies the excessive water volume which cannot be used in energy production and is released to the creek directly. The unit of the parameter is, therefore, cubic meters (m^3). In the case that the coming water to the project site is greater than the maximum usable water that can be diverted from the creek, unneeded part of the water is not taken to the system and released to the creek.

Final Reservoir Volume (V_f)

The final reservoir volume is found by calculating the water volume existing in the reservoir at the end of each day considered in the operation study. In determining this parameter, water volume which is released without using in electricity production (V_r) is taken into account and the unit of the parameter is selected as cubic meters (m^3). If any water is released then the final reservoir volume would be the same with the reservoir capacity. In the situation that, there is no excessive water to be released to the creek, it means that all water coming to the reservoir is used. Thus, the final reservoir volume is determined by subtracting the water volume used to generate electricity in peak period from the summation of the water volume existing in the reservoir at the beginning of each day and the water volume that is stored throughout the day.

Energy Generated in the Off-Peak Period (E_{opeak})

The amount of electricity generated in the off-peak period can be calculated by utilizing the water volume used to produce energy in the off-peak period (V_{opeak}). The unit of the energy is kilowatts per hour (kWh).

Energy Generated in the Peak Period (E_{peak})

The calculated water volume used to generate energy in peak period is converted to discharge and it is used in the equation to calculate the energy that can be produced in peak period. The unit of the parameter is kilowatts per hour (kWh).

Off-Peak Period Income (I_{opeak})

In calculation of the income from the electricity that is generated in the off-peak period, the unit price developed for the off-peak hours is taken into consideration (krş/kWh). By multiplying this price with the total energy produced in the off-peak period, the total income gained in the off-peak period is obtained.

Peak Period Income (I_{peak})

Peak period income is the result of the product of total energy produced in peak period and the unit price of electricity calculated for peak period.

Total Income (I_{tot})

It is the parameter which is calculated by summing the peak period and the off-peak period incomes day by day. By determining this parameter, total income that can be gained for each day considered in the operation study is found. Its unit is TL.

Average Annual Off-Peak Energy Production ($E_{opeakavg}$)

Average of the off-peak energy production amounts for each day of the whole data is calculated in kilowatts per hour (kWh) and the outcome is multiplied with the number of days in a year. Number of days in a year can be taken as 365.25 in order to include the effect of the leap years in the calculations.

Average Annual Off-Peak Energy Income ($I_{opeakavg}$)

It is obtained by multiplying the average annual off-peak energy production value with the off-peak energy unit price. The unit of the parameter is TL.

Average Annual Peak Energy Production ($E_{peakavg}$)

It is determined by calculating the product of the average of daily energy productions in the peak period and the number of days in a year (365.25). The result is in kilowatts per hour (kWh).

Average Annual Peak Energy Income ($I_{peakavg}$)

It is found by multiplying average annual peak energy production with the unit price of the energy that is produced in the peak period. The result is in TL.

Total Annual Energy Production (E_{tot})

It is the summation of average annual off-peak energy production and average annual peak energy production.

Average Annual Income (I_{avg})

It is the parameter which is obtained by summing average annual off-peak energy income and average annual peak energy income values. Average annual income is very important for the economic analysis through which the profitability of the project is obtained and it is decided whether the project is feasible or not.

Four typical months belonging to the operation study, each representing a specific case related to incoming discharge, is given in Appendix D. These cases are:

Case 1: Incoming discharge (Q_{inc}) is below the design discharge ($Q_{inc} << Q_d$)

Case 2: Incoming discharge is about half of the design discharge ($Q_{inc} \approx Q_d/2$)

Case 3: Incoming discharge is about the design discharge ($Q_{inc} \approx Q_d$)

Case 4: Incoming discharge is above the design discharge ($Q_{inc} >> Q_d$)

Variation of generated energy with respect to days of these cases is shown in Figure 4.7.

For Case 1, all water is left to the creek if the incoming discharge is smaller than the minimum discharge to be released for the maintenance of the ecological life. Water left after releasing the obligatory amount is stored in the facility to be used for energy generation in peak periods. If there is much water left from the previous day and the storage can be filled by the stored volume, energy is generated in off-peak periods (see Figure 4.7).

For Cases 2 and 3, the situation is similar with the previous case except all incoming water is not released to meet the minimum water amount to be released. Incoming water is again used for filling the storage for peak periods. If the storage is filled, the excess water is utilized to generate energy in off-peak periods. Electricity generation for Case 3 is almost double of that of Case 2, i.e. in the order of 6×10^5 kWh.

For case 4, after filling the storage for peak periods, water is utilized for energy generation in the off-peak period with the design discharge. The part of water which is greater than the design discharge is released to the creek without any utilization. This case leads to constant energy generation in the order of 1.3×10^6 kWh (Figure 4.7).

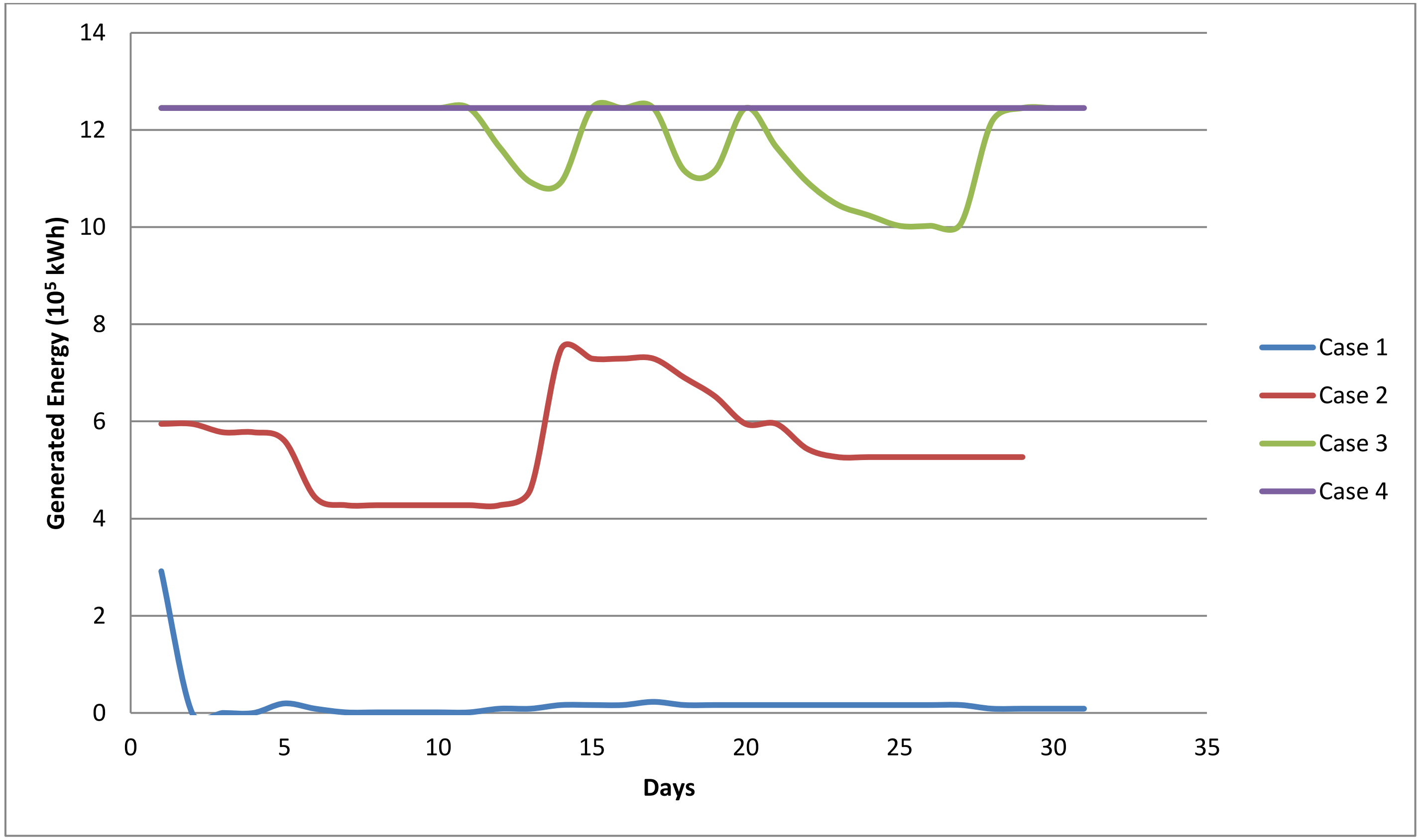


Figure 4.7 Variation of electricity generation for different cases

CHAPTER 5

OPTIMIZATION STUDY

5.1 Introduction

In order to determine the optimum dimensions of the related structures of the intended storage facility, an optimization study is carried out. In the course of this study, 20 different alternatives are investigated and the cost items of those alternatives are compared to the income obtained from each related alternative. All storage facility alternatives are located at a section having a width of 50 meters and the length of the dam body is calculated for each alternative by subtracting the spillway length from the project section width, i.e. 50 m. The height of the dam body is determined by the summation of the height of the spillway, a freeboard of 0.5 m and the head corresponding to the flood having a return period of 500 years (See Table 4.3). A return period of 500 years is used in the calculations since there is no residential area at the downstream part of the facility [26].

Dimensions of the spillway and dam body structures related to these alternatives are given in Table 5.1. Detailed profiles of these structures are given in Figures 4.5 and 4.6. Items corresponding to the cost and income are explained in the following subsections.

Table 5.1 Dimensions of spillway and dam body alternatives

Alternative	Spillway Height (m)	Spillway Length (m)	Dam Body Height (m)	Dam Body Length (m)
1	7.5	30	10.59	18
2	7.5	32	10.48	16
3	7.5	34	10.38	14
4	7.5	36	10.29	12
5	8.0	30	11.09	18
6	8.0	32	10.98	16
7	8.0	34	10.88	14
8	8.0	36	10.79	12
9	8.5	30	11.59	18
10	8.5	32	11.48	16
11	8.5	34	11.38	14
12	8.5	36	11.29	12
13	9.0	30	12.09	18
14	9.0	32	11.98	16
15	9.0	34	11.88	14
16	9.0	36	11.79	12
17	9.5	30	12.59	18
18	9.5	32	12.48	16
19	9.5	34	12.38	14
20	9.5	36	12.29	12

5.2 Costs

In cost determination study, the items that are taken into account are as follows:

- concrete volume
- formworks
- reinforcement

- cement
- transportation of materials
- other costs

For unforeseen costs, 20% of total cost calculated using the given items above is designated and is added to the calculated cost as it is done in common Turkish engineering practice.

For the cost calculations, unit prices of 2010 are utilized. Item numbers and the unit costs of the expenses used in the calculations are given in Table 5.2.

Table 5.2 Unit prices for cost items

Item Number	Work	Unit Prices
16.022/1	Reinforced concrete (C16, with sand and aggregate)	82.9 TL/m^3
21.011	Flat surfaced formworks for concrete and reinforced concrete	21.21 TL/m^2
21.022	Formworks for concrete with curved surfaces	40.38 TL/m^2
16.D/1-A	Cement cost	137.45 TL/m^3
23.002	Placement of steel bars with a diameter of 50 mm	1463.56 TL/tons
07.006/45	Transportation (200 km)	35.16 TL
07.006/48	Transportation (400 km)	76.18 TL

5.2.1 Concrete Volume

The cost of the concrete that is to be used in the construction of Birkapılı Hydroelectric Power Plant storage facility is an important cost item. In order to determine it, the volume of concrete is calculated in 3 parts:

- Concrete volume needed for the construction of the spillway
- Concrete volume needed for the construction of the dam body
- Concrete volume needed for the construction of the side walls of the spillway

To determine these costs, profile areas of the spillway and the dam body are calculated for each alternative and the calculated areas are multiplied with the width of these structures. For alternatives designated for the spillway, it is seen that all alternatives yields to the USBR type II stilling basins. Design of the stilling basins is made accordingly and additional structures to be constructed on the stilling basin for energy dissipation purposes are also taken into account while designating the cost of concrete.

The concrete amount needed for the construction of side walls standing at both sides of the stilling basin is also determined. Width of side walls are taken as 1.00 m as a constant value in the calculations. Height of the side walls belonging to each alternative is computed by using the following equation.

$$H_2 = y_2 + 0.8\, y_c \tag{5.1}$$

where,

H_2: Height of side wall

y_2: Sequent depth of hydraulic jump

y_c: Critical depth of flow

In equation (5.1), $0.8y_c$ stands for freeboard [53].

Parameters used in the calculations and concrete volume needed for construction of the spillway, dam body and the side walls of the stilling basin are given in Table 5.3 for each alternative.

Detailed computations of the best alternative are presented in Appendix A.

Table 5.3 Concrete volume calculated for each alternative

Alternative	Spillway (m³)	Dam Body (m³)	Side Walls (m³)	Total Volume (m³)	Total Cost (TL)
1	4,135.28	2,760.16	903.74	7,799.18	646,551.80
2	4,309.24	2,410.43	884.30	7,603.97	630,369.00
3	4,467.94	2,078.36	843.39	7,389.69	612,606.02
4	4,636.63	1,752.07	820.72	7,209.42	597,661.20
5	4,379.41	2,938.89	972.86	8,291.16	687,337.49
6	4,533.70	2,566.19	922.79	8,022.68	665,080.29
7	4,713.79	2,208.75	894.32	7,816.86	648,018.27
8	4,832.36	1,873.84	891.78	7,597.98	629,872.95
9	4,714.14	3,149.11	1,021.55	8,884.80	736,549.70
10	4,900.70	2,750.69	985.41	8,636.80	715,990.36
11	5,115.24	2,371.78	974.28	8,461.30	701,442.09
12	5,289.34	2,003.45	930.12	8,222.91	681,678.69
13	5,078.78	3,369.49	1,088.50	9,536.77	790,598.42
14	5,288.53	2,944.94	1,049.05	9,282.52	769,520.88
15	5,487.93	2,535.99	1,017.40	9,041.32	749,525.87
16	5,700.29	2,145.15	990.77	8,836.21	732,521.24
17	5,462.82	3,594.19	1,175.84	10,232.85	848,302.75
18	5,661.20	3,140.35	1,111.97	9,913.52	821,831.67
19	5,892.49	2,708.01	1,080.13	9,680.63	802,524.33
20	5,974.25	2,291.65	1,004.01	9,269.91	768,475.79

5.2.2 Formworks

In order to determine the cost of formworks to be used in the construction of the Birkapılı HEPP storage facility; the spillway, dam body, and the side walls standing at both sides of the stilling basin are taken into account. Formworks used for the spillway construction are investigated in two different approaches. For the curved parts of the spillway, a higher unit price is included in the cost determination calculations. For other parts, the unit price determined for flat type of is used.

Lateral surface areas are taken into consideration in the calculation of the formwork amount needed. The formwork amount is found, in square meters for each alternative investigated. Determination of the amount of formwork needed for the best alternative is explained in detail, in Appendix A.

Total amount of formwork and the related expenses are given in Table 5.4 for each alternative.

Table 5.4 Formwork amount and costs

Alternative	Flat Formwork for Spillway (m^2)	Curved Formwork for Spillway (m^2)	Flat Formwork for Dam Body (m^2)	Flat Formwork for Site walls (m^2)	Total Cost for Flat Formwork (TL)	Total Cost for Curved Formwork (TL)
1	882.51	490.50	930.79	1,853.12	77,764.61	19,806.39
2	932.32	500.16	850.21	1,813.53	76,272.40	20,196.46
3	933.95	541.96	772.98	1,731.12	72,921.11	21,884.34
4	959.20	569.88	696.03	1,685.23	70,851.20	23,011.75
5	946.46	570.60	974.53	1,992.60	83,007.38	23,040.83
6	942.64	538.24	890.69	1,891.72	79,008.41	21,734.13
7	969.77	566.44	809.35	1,834.13	76,637.16	22,872.85
8	1,023.89	648.00	732.97	1,828.65	76,048.61	26,166.24
9	968.89	532.50	1,025.02	2,091.60	86,653.60	21,502.35
10	994.59	561.60	937.77	2,018.59	83,799.61	22,677.41
11	1,049.92	658.92	854.02	1,995.69	82,711.12	26,607.19
12	1,050.56	620.28	771.69	1,906.84	79,094.15	25,046.91
13	1,020.82	554.70	1,076.99	2,227.19	91,733.32	22,398.79
14	1,049.48	583.36	986.49	2,147.48	88,730.91	23,556.08
15	1,075.41	615.74	898.30	2,083.51	86,053.63	24,863.58
16	1,104.96	644.76	813.40	2,029.74	83,739.04	26,035.41
17	1,099.20	641.40	1,129.09	2,403.42	98,238.44	25,899.73
18	1,098.37	605.76	1,034.71	2,274.92	93,493.51	24,460.59
19	1,129.05	639.20	943.97	2,210.57	90,855.08	25,810.90
20	1,152.40	672.48	855.90	2,057.82	86,242.42	27,154.74

5.2.3 Reinforcements

In this study, costs corresponding to the reinforcements needed in the construction of the facility are determined considering that weight of reinforcement in tones are 5% of the concrete volume in cubic meters. After making some negotiations with authorities from hydropower sector, it is decided that this approach is appropriate for feasibility studies and is adopted in this study. To give an example, it is decided that 5 tones of reinforcement is used in a concrete bulk having a volume of 100 m^3.

Keeping in mind the mentioned approach above, the weight of the reinforcement and the corresponding cost is calculated accordingly for each alternative and the details are given in Table 5.5.

Table 5.5 Cost and quantity calculations of reinforcements

Alternative	**Total Concrete Volume (m^3)**	**Total Reinforcement Weight (ton)**	**Reinforcement Cost (TL)**
1	7,799.18	389.96	570,728.19
2	7,603.97	380.20	556,443.21
3	7,389.70	369.48	540,763.37
4	7,209.42	360.47	527,571.18
5	8,291.16	414.56	606,730.80
6	8,022.68	401.13	587,083.78
7	7,816.87	390.84	572,022.69
8	7,597.98	379.90	556,005.34
9	8,884.80	444.24	650,171.70
10	8,636.80	431.84	632,023.43
11	8,461.30	423.07	619,181.30
12	8,222.90	411.15	601,735.62
13	9,536.77	476.84	697,881.92
14	9,282.52	464.13	679,276.23
15	9,041.33	452.07	661,626.10
16	8,836.20	441.81	646,615.67
17	10,232.84	511.64	748,819.04
18	9,913.53	495.68	725,452.32
19	9,680.63	484.03	708,409.23
20	9,269.91	463.50	678,353.69

5.2.4 Cement

Concrete structures belonging to the Birkapılı HEPP storage facility are assumed to have a weight of 30% of their concrete volume. This approach is a common Turkish engineering practice. Thus, cement weights in tones are calculated by multiplying the concrete volumes by 0.30 and total cost of cement is computed by considering the unit price of cement.

Details of cement cost calculations for each alternative is given in Table 5.6.

Table 5.6 Cement costs for each alternative

Alternative	Total Concrete Volume (m^3)	Total Cement Weight (ton)	Cement Cost (TL)
1	7,799.18	2,339.75	321,599.08
2	7,603.97	2,281.19	313,549.65
3	7,389.70	2,216.91	304,714.23
4	7,209.42	2,162.83	297,280.57
5	8,291.16	2,487.35	341,886.15
6	8,022.68	2,406.80	330,815.27
7	7,816.87	2,345.06	322,328.51
8	7,597.98	2,279.40	313,302.91
9	8,884.80	2,665.44	366,364.62
10	8,636.80	2,591.04	356,138.27
11	8,461.30	2,538.39	348,901.87
12	8,222.90	2,466.87	339,071.42
13	9,536.77	2,861.03	393,248.81
14	9,282.52	2,784.76	382,764.70
15	9,041.33	2,712.40	372,819.05
16	8,836.20	2,650.86	364,360.83
17	10,232.84	3,069.85	421,951.31
18	9,913.53	2,974.06	408,784.42
19	9,680.63	2,904.19	399,180.83
20	9,269.91	2,780.97	382,244.86

5.2.5 Transportation of Materials

In the determination of material transportation cost; transportation of cement, steel, and excavation is considered. After examining the vicinity of the project site, it is decided that cement is provided from Mersin (200 km) and steel from İskenderun (450 km).

The transportation cost is computed by utilizing Table 5.2. In Table 5.7, the corresponding transportation costs are given.

Table 5.7 Transportation costs of the project

Alternative	Total Cement Weight (tones)	Total Steel Weight (tones)	Cement Transportation Cost (TL)	Steel Transportation Cost (TL)
1	2,339.75	389.96	82,265.72	29,707.07
2	2,281.19	380.20	80,206.66	28,963.52
3	2,216.91	369.48	77,946.54	28,147.36
4	2,162.83	360.47	76,045.00	27,460.69
5	2,487.35	414.56	87,455.20	31,581.04
6	2,406.80	401.13	84,623.24	30,558.39
7	2,345.06	390.84	82,452.31	29,774.45
8	2,279.40	379.90	80,143.54	28,940.72
9	2,665.44	444.24	93,716.84	33,842.19
10	2,591.04	431.84	91,100.92	32,897.55
11	2,538.39	423.07	89,249.83	32,229.11
12	2,466.87	411.15	86,735.18	31,321.04
13	2,861.03	476.84	100,593.87	36,325.57
14	2,784.76	464.13	97,912.02	35,357.12
15	2,712.40	452.07	95,367.90	34,438.41
16	2,650.86	441.81	93,204.27	33,657.10
17	3,069.85	511.64	107,936.04	38,976.90
18	2,974.06	495.68	104,567.92	37,760.64
19	2,904.19	484.03	102,111.30	36,873.52
20	2,780.97	463.50	97,779.04	35,309.10

5.2.6 Other Costs

Cost items which cannot be specified in this feasibility study, such as sluiceway, are assumed to add up approximately 20% of the total calculated cost, which is widely accepted by the Turkish engineering practice, especially for feasibility studies. Additional costs of 120,000 TL and 250,000 TL are also taken into account for construction of a pathway to the project site and engineering expenses, respectively. These costs are included in the cost determination studies. Overall costs calculated for each alternative is represented in Table 5.8.

Table 5.8 Total cost of each alternative

Alternative	Total Project Cost (TL)	Alternative	Total Project Cost (TL)
1	2,468,107.43	11	2,650,387.02
2	2,417,201.08	12	2,583,619.62
3	2,360,779.58	13	2,929,336.84
4	2,313,857.91	14	2,862,541.52
5	2,603,246.67	15	2,799,633.46
6	2,528,684.22	16	2,746,160.27
7	2,474,927.47	17	3,118,149.04
8	2,422,576.38	18	3,029,621.28
9	2,756,561.21	19	2,968,918.23
10	2,691,553.07	20	2,860,671.57

5.3 Income Calculations

The only income item of the intended storage facility is the income obtained from electricity generation and sale. By constructing such a storage facility at the upstream side of the existing plant, it is possible to store water when the electricity sale prices are low and use the stored water to generate energy when electricity sale prices are high.

5.3.1 Determination of Electricity Prices and Peak Hours

By utilizing the data belonging to electricity prices being recorded and published by the Market Financial Settlement Center, electricity sale prices are determined by calculating the average of each hour in a day. Since a complete 2-year period is considered in the analysis, being affected from seasons is avoided for the electricity prices and homogeneous data are maintained for the study. In this thesis, number of peak hours is determined according to the water volume stored in the reservoir. Average hourly electricity prices obtained by taking averages of data of a 2-year period are given in Table 5.9. A part of the whole data covering two months in summer and winter times in 2011 are provided in Appendix E.

Table 5.9 Average hourly electricity prices

Hour	Average Price (TL/MWh)	Hour	Average Price (TL/MWh)
0 - 1	130.92	12 - 13	144.70
1 - 2	109.77	13 - 14	147.11
2 - 3	94.05	14 - 15	152.57
3 - 4	78.75	15 - 16	146.82
4 - 5	71.31	16 - 17	144.75
5 - 6	69.24	17 - 18	137.10
6 - 7	68.05	18 - 19	132.71
7 - 8	89.66	19 - 20	130.23
8 - 9	125.96	20 - 21	135.37
9 - 10	140.23	21 - 22	131.09
10 - 11	152.72	22 - 23	148.37
11 - 12	161.36	23 - 24	138.07

5.3.2 Income Calculations for Different Alternatives

In the income calculations, five different alternatives can be taken into account. These five alternatives are due to height differences only as the width of the facility does not affect the income. To specify the income for the aforementioned five alternatives, the water amount that can be stored behind the overflow spillway with heights of 7.5, 8.0, 8.5, 9.0, and 9.5 meters are determined by utilizing a digitized map of the project area and drawing software. After determining the water volume values for each height, durations of energy generation for each alternative are designated using the design discharge of the project. In the calculation of income values, the operation study spreadsheet is utilized. Since this program accepts the peak-hour period and reservoir storage capacity as input parameters, these values are introduced to the program for each alternative before running it. The table demonstrating the annual income obtained from electricity generation is given in Table 5.10. The income values are assumed to be constant throughout the project's economic life of 50 years.

Table 5.10 Annual income values for each alternative

Spillway Height (m)	Annual Energy Income (TL)
7.5	959,402.84
8.0	1,037,442.34
8.5	1,106,544.11
9.0	1,131,839.36
9.5	1,118,997.58

5.4 Future Projection of Costs

The investment expenses of a typical hydropower project are mainly incurred before the commencement of the project or during construction period. But, income obtained from the project is gained year by year throughout the economical life span of the project. In order to measure the profitability of any project, the cost and income items of the project should be investigated for a common time medium. To do that, the investment cost made at the beginning of the project is converted to annual payments throughout economic life span of the project. This is done by using the capital recovery factor. By using the capital recovery factor, a present value can be converted into a stream of equal annual payments over a specified time. A specified discount rate (interest) is also included in the study [54].

Annual cost of investment is determined by multiplying the total investment by the capital recovery factor. In the case of this study, the interest rate and the economic life of the structure is assumed to be 9.5% and 50 years, respectively. These values are the most commonly used values in the hydropower sector. Annual cost of the project throughout its economic life span is estimated by multiplying the total computed costs by the capital recovery factor as shown in Table 5.11. This cost, which is distributed to every single year of the economic project life of the intended storage facility, can be compared with the income obtained for each year. Net annual profit obtained in each year is given in Table 5.11.

Table 5.11 Annual investment costs and net benefits

Alternative	Spillway Height - Spillway Length	Total Cost (TL)	Annual Investment Costs (TL)	Annual Energy Income (TL)	Annual Net Benefit (TL)
1	7.5 x 30	2,468,107.43	236,938.31	959,402.84	722,464.53
2	7.5 x 32	2,417,201.08	232,051.30	959,402.84	727,351.54
3	7.5 x 34	2,360,779.58	226,634.84	959,402.84	732,768.00
4	7.5 x 36	2,313,857.91	222,130.36	959,402.84	737,272.48
5	8.0 x 30	2,603,246.67	249,911.68	1,037,442.34	787,530.66
6	8.0 x 32	2,528,684.22	242,753.69	1,037,442.34	794,688.66
7	8.0 x 34	2,474,927.47	237,593.04	1,037,442.34	799,849.30
8	8.0 x 36	2,422,576.38	232,567.33	1,037,442.34	804,875.01
9	8.5 x 30	2,756,561.21	264,629.88	1,106,544.11	841,914.23
10	8.5 x 32	2,691,553.07	258,389.09	1,106,544.11	848,155.01
11	8.5 x 34	2,650,387.02	254,437.15	1,106,544.11	852,106.95
12	8.5 x 36	2,583,619.62	248,027.48	1,106,544.11	858,516.62
13	9.0 x 30	2,929,336.84	281,216.34	1,131,839.36	850,623.02
14	9.0 x 32	2,862,541.52	274,803.99	1,131,839.36	857,035.37
15	9.0 x 34	2,799,633.46	268,764.81	1,131,839.36	863,074.55
16	**9.0 x 36**	**2,746,160.27**	**263,631.39**	**1,131,839.36**	**868,207.97**
17	9.5 x 30	3,118,149.04	299,342.31	1,118,997.58	819,655.27
18	9.5 x 32	3,029,621.28	290,843.64	1,118,997.58	828,153.94
19	9.5 x 34	2,968,918.23	285,016.15	1,118,997.58	833,981.43
20	9.5 x 36	2,860,671.57	274,624.47	1,118,997.58	844,373.11

As can be observed from Table 5.11, the 16th alternative gives the maximum annual net benefit. The resultant designs of spillway and dam body are shown in the Appendix Section (Figures A.1 and A.2).

CHAPTER 6

CONCLUSION

In this thesis, it is aimed to design a storage facility for the existing Birkapılı Hydroelectric Power Plant to improve its electricity generation capability. By providing such a storage facility, more of the coming water to the project site can be utilized for energy generation and the profitability of the facility can be changed in a positive manner by bringing flexibility to its electricity generation period. By making the electricity generation in peak hours possible, income which is obtained from electricity sale can be increased in an appreciable amount.

A flow analysis is carried out to determine the discharge coming to the project site. The catchment area of the project site is determined by consulting the related maps with a scale of 1/25000 and the flow values obtained from the stream gauging stations by regression analysis are transferred to the project site by using area ratios of the related stream gauging station and the project site.

A flood frequency analysis is also an ingredient of this study. Floods having return periods of 100 and 500 years are calculated and used for the finalization of designs of the spillway and dam body.

In order to ensure the safety of the spillway and dam body belonging to the storage facility, stability analyses are implemented by considering various loads under several loading conditions. In the analyses, the related safety factor values are determined and the analyses are finalized accordingly. All stability analyses are carried out for different alternatives of spillway and dam body, all having different dimensions.

For the completion of the economic analysis, income which is obtained by electricity sale is determined and an investigation of the Turkish electricity market is held. Hourly average prices for electricity are found by evaluation of the data (day-ahead electricity prices) recorded and published by the Market Financial Settlement Center having a period of 2 years. Calculating average sale prices for the designated peak and base hours, an operation study is undertaken and annual income values are obtained for several alternatives.

The expenses done for the implementation of the project are specified by considering main cost items and necessary computations are done for each investigated alternative. After obtaining investment costs for each alternative, the values obtained are converted to annual payments by multiplying them by the capital recovery factor.

Finally, it is seen that the most profitable design comprises a spillway of 9.0 m of height and 36.0 m of width. The height and the width of the dam body are 11.8 m and 12.0 m, respectively. By constructing the proposed storage facility, it is seen that an additional annual net benefit of 868,208 TL is obtained.

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APPENDIX A

DETERMINATION OF CONCRETE AND FORMWORK AMOUNT

In this part, calculations carried out to determine the material costs of dam body and spillway are shown in detail.

A.1 Dam Body Concrete:

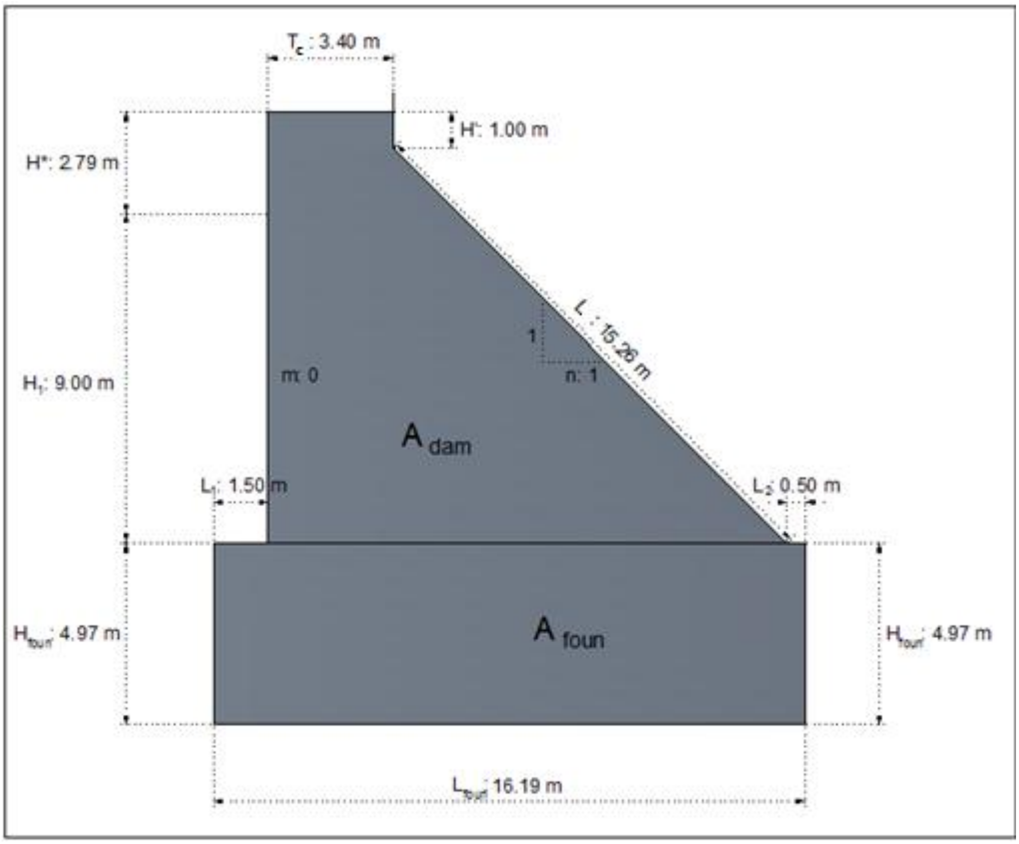

Figure A.1 Dam body cross-section

Table A.1 Dimensions of dam body cross-section

Dimensions		Value
T_c	Thickness of the dam crest (m)	3.40
H_1	Dam height at reservoir level (m)	9.00
H*	Upstream height of the rectangular part (m)	2.79
H’	Downstream height of the rectangular part (m)	1.00
m	Upstream face slope	0
n	Downstream face slope	1
L_1	Length of the upstream foundation extension (m)	1.50
L_2	Length of the downstream foundation extension (m)	0.50
H_s	Sediment accumulation height (m)	4.79
H	Downstream fill height (m)	4.79
H_{foun}	Height of the foundation (m)	4.79
L	Length of the slopped part (downstream)(m)	15.26
L_{foun}	Length of the foundation (m)	16.19

$$A_{foun} = H_{foun} \times L_{sl}$$

$$A_{foun} = 4.97 \times 16.19 = 80.46\, m^2$$

$$A_{dam} = \left[T_c \times (H_1 + H^*) + 0.5 \times {H_1}^2 \times m_1 + 0.5 \times (H_1 + H^* - H')^2 \times n_1)\right]$$

$$A_{dam} = [3.4 \times (9.0 + 2.79) + 0.5 \times 9.0^2 \times 0.0 + 0.5 \times (9.0 + 2.79 - 1.0)^2 \times 1.0)] = 98.30\, m^2$$

$$A_{totald} = A_{dam} + A_{foun} = 80.46 + 98.30 = 178.76\, m^2$$

The concrete volume is estimated by multiplying the area of the dam body cross-section and the length of the dam body:

$$V_{conc} = A_{totald} \times L_{dam} = 178.76 \times 12.0 = 2145.15\, m^3$$

A.2 Spillway Concrete:

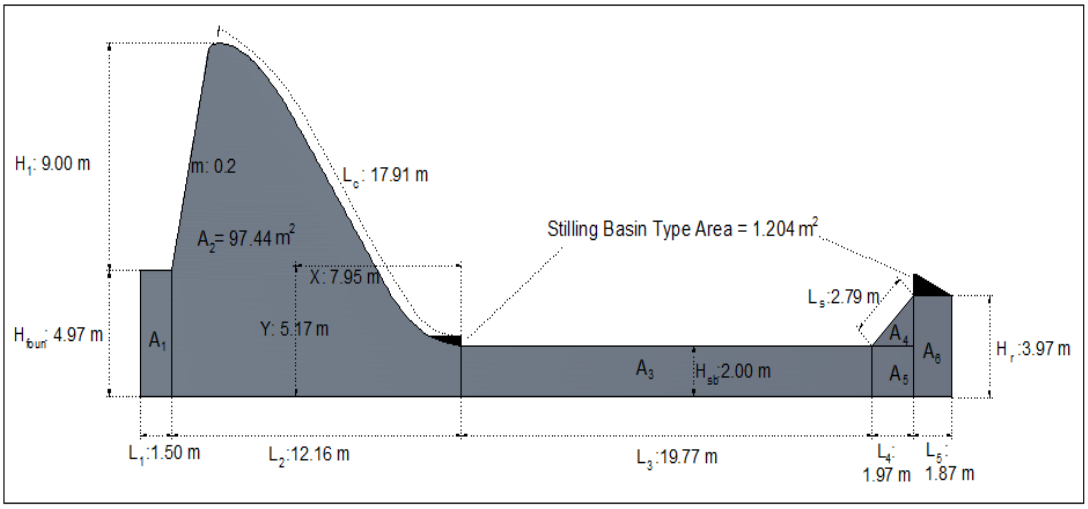

Figure A.2 Spillway cross-section

Table A.2 Dimensions of spillway cross-section

Dimensions		Value
H_s	Height of the spillway (m)	9.00
H_{foun}	Height of the foundation (m)	4.97
m	Upstream slope of the spillway	0.20
L_1	Length of the upstream foundation extension (m)	1.50
L_2	Length of the spillway (m)	12.16
L_3	Length of the stilling basin (m)	19.77
L_4	Length of the slopped part of the stilling basin (m)	1.97
L_5	Length of the last part of the stilling basin before riprap (m)	1.87
H_r	Height of the riprap(m)	3.97
H_{sb}	Thickness of the stilling basin foundation (m)	2.00
X	Horizontal distance of the center of gravity to the end of the spillway(m)	7.95
Y	Vertical distance of the center of gravity to the end of the spillway (m)	5.17
A_2	Area of the spillway body (m^2)	97.44
L_c	Length of curved part of the spillway (m)	17.91
L_s	Length of the slopped part of the dentate sill (m)	2.79

$A_1 = H_{foun} \times L_1 = 4.97 \times 1.5 = 7.46\, m^2$

$A_2 = Spillway\ area\ \ is\ estimated\ by\ using\ AutoCAD = 97.44\, m^2$

$A_3 = H_{sb} + L_3 = 2.0 \times 19.77 = 39.54\, m^2$

$A_4 = 0.5 \times L_4 + (H_r - H_{sb}) = 0.5 \times 1.97 \times (3.97 - 2.0) = 1.94\, m^2$

$A_5 = H_{sb} \times L_4 = 2.0 \times 1.97 = 3.94\ m^2$

$A_6 = H_r \times L_5 = 3.97 \times 1.87 = 7.42\ m^2$

$$
\begin{aligned}
A_{totalsp} &= A_1 + A_2 + A_3 + A_4 + A_5 \\
&= 7.46 + 97.44 + 39.54 + 1.94 + 3.94 + 7.42 \\
&= 157.74\ m^2
\end{aligned}
$$

The concrete volume is estimated by multiplying the area of the spillway cross-section and the length of the dam body, In addition, the concrete needed for chute blocks and dentate sill are also calculated and found as 21.67 m^3.

$V_{sp} = A_{totalsp} \times L_{sp} = 157.74 \times 36.0 + 21.67 = 5700.29\, m^3$

A.3 Side Walls Concrete:

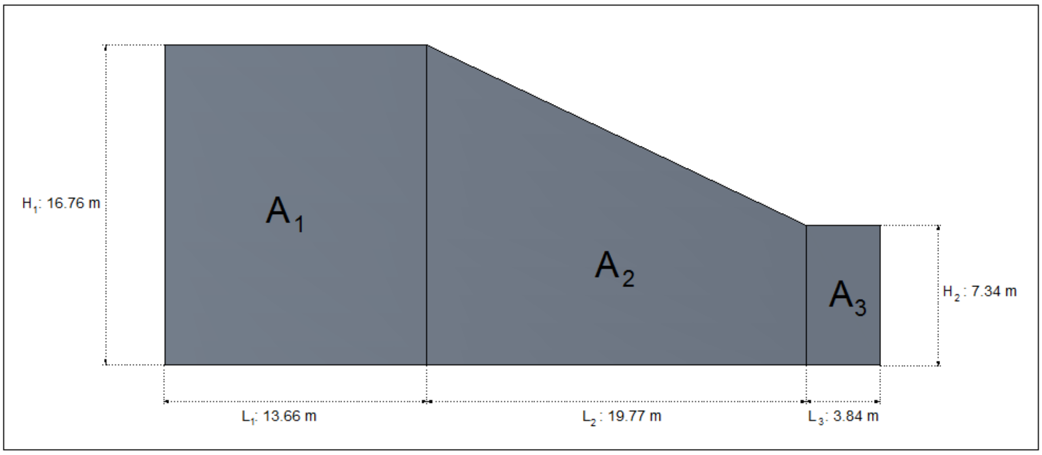

Figure A.3 Side wall cross-section

Table A.3 Dimension of side wall cross-section

Dimensions		Value
H_1	Upstream height of the side wall (m)	16.76
H_2	Downstream height of the side wall (m)	7.34
L_1	Length of the upstream rectangular part of the wall (m)	13.66
L_2	Length of the trapezoidal part of the side wall (m)	19.77
L_3	Length of the downstream rectangular part of the wall (m)	3.84

$$A_1 = H_1 + L_1 = 16.76 \times 13.66 = 228.94\, m^2$$

$$A_2 = 0.5 \times (H_1 + H_2) \times L_2 = 0.5 \times (16.76 + 7.34) \times 19.77 = 238.25\, m^2$$

$$A_3 = H_2 \times L_3 = 7.34 \times 3.84 = 28.19\, m^2$$

$$A_{totalsw} = A_1 + A_2 + A_3 = 228.94 + 238.25 + 28.19 = 495.39\, m^2$$

The concrete volume is estimated by multiplying the area of the side wall cross-section and the length of the dam body:

$$V_{sw} = A_{totalsw} \times L_{sw} = 495.39 \times 2.0 = 990.77\ m^3$$

A.4 Dam Body Formwork:

Flat formwork for the upstream and downstream faces of the dam body can be estimated as:

$$A_{u-dface} = (H_{foun} + H_1 + H^* + H' + L_{sl} + H_{foun}) \times L_{dam}$$

$$= (4.97 + 9.0 + 2.79 + 1.0 + 15.26 + 4.97) \times 12.0 = 455.88\ m^2$$

Flat formwork for the two lateral faces of the dam body can be estimated by multiplying the cross section area in Figure A-1 by two.

$$A_{lface} = A_{total} \times 2 = 178.76 \times 2 = 357.52\ m^2$$

The total formwork for dam body:

$$A_{ftotal} = A_{u-dface} + A_{lface} = 455.88 + 357.52 = 813.40\ m^2$$

A.5 Spillway Formwork

Two types of formwork are considered in the calculation of formwork area for spillway alternatives:

A.5.1 Flat Formwork

Flat formwork area needed for the upstream and downstream faces of the spillway can be estimated as:

$$A_{u-dface} = \left(H_{foun} + H_s + L_s + H_t\right) \times L_{spill}$$
$$= (4.97 + 9.00 + 2.79 + 3.97) \times 36.0 = 746.28\ m^2$$

Flat formwork area for the two lateral faces of the spillway can be estimated by multiplying the cross section area in Figure A-2 by two. In addition, the formwork needed for chute blocks and dentate sill are also calculated and found as 43.34 m^2.

$$A_{lface} = A_{total} \times 2 + 43.34 = 157.74 \times 2 + 43.34 = 358.82\ m^2$$

Total flat formwork for spillway is calculated as:

$$A_{ftotal} = A_{u-dface} + A_{lface} = 746.28 + 358.82 = 1105.10\ m^2$$

A.5.2 Curved Formwork

Curved formwork area needed for the spillway can be estimated by multiplying the length of the curved shape (via AutoCAD) by the length of the spillway:

$$A_{sp-cur} = L_c \times L_{sp} = 17.91 \times 36.0 = 644.76\ m^2$$

A.6 Side Walls Formwork

Flat formwork area needed for the upstream and downstream faces of the side walls can be estimated as:

$$A_{u-dface} = (H_1 + H_2) \times L_{sw} = 16.76 \times 2.0 + 7.34 \times 2.0 = 48.20\, m^2$$

Flat formwork for the two lateral faces of the side walls can be estimated by multiplying the cross section area in Figure A-3 by four.

$$A_{lface} = A_{total} \times 4 = 495.39 \times 4 = 1981.56\ m^2$$

Total flat formwork for side walls is calculated as:

$$A_{ftotal} = A_{u-dface} + A_{lface} = 48.20 + 1981.56 = 2029.76\ m^2$$

APPENDIX B

STABILITY ANALYSIS OF DAM BODY

Table B.1 Stability analysis of dam body for usual case

Usual Case					
Forces	**Value (kN/m)**	**Moment wrt Toe (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_i	100.00	13.97	Horizontal	Overturning	1397.00
F_{v1}	135.00	15.44	Vertical	Resisting	2084.40
F_{v2}	0.00	0.00	Vertical	Resisting	0.00
F_{v3}	0.31	0.58	Vertical	Resisting	0.18
F_{v4}	1.25	0.25	Vertical	Resisting	0.31
F_{h1}	975.80	4.66	Horizontal	Overturning	4544.00
F_{s1}	0	15.44	Vertical	Resisting	0.00
F_{s2}	0	14.69	Vertical	Resisting	0.00
F_{f1}	0	0.50	Vertical	Resisting	0.00
F_{f2}	0	0.25	Vertical	Resisting	0.00
F_{silt}	177.85	1.66	Horizontal	Overturning	294.63
F_{fill}	277.89	1.66	Horizontal	Resisting	460.36
W_1	0.00	14.69	Vertical	Resisting	0.00
W_2	962.06	12.99	Vertical	Resisting	12497.21
W_3	1397.09	7.69	Vertical	Resisting	10748.27
W_4	1931.14	8.10	Vertical	Resisting	15632.60
F_{u1}	845.12	8.10	Vertical	Overturning	6841.23
F_{u2}	708.31	10.79	Vertical	Overturning	7645.05
F_{h2}	136.24	1.74	Horizontal	Resisting	237.06

Table B.2 Determination of factor of safety values and stresses belonging to dam body for usual case

			Situation
Net moment (ΔM)	M_R-M_o	20938.50	
Overturning Safety	$FS_o = (M_R/M_o)$	2.01	**Ok**
Sliding Safety	$FS_s = (f \sum V / \sum H)$	2.57	**Ok**
Shear-Sliding Safety	$FS_{ss} = (f \sum V + rA\tau_s) / (\sum H)$	40.75	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	124.33	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	230.63	**Ok**
Base Width	B (m)	16.19	
Total net force	$\sum F$ (kN/m)	2873.43	
Eccentricity	e	7.29	
The distance from center	x	0.81	

Table B.3 Stability analysis of dam body for unusual case

Unusual Case					
Forces	**Value (kN/m)**	**Moment wrt Toe (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_{v1}	176.85	15.44	Vertical	Resisting	2730.56
F_{v2}	0.00	0.00	Vertical	Resisting	0.00
F_{v3}	0.31	0.58	Vertical	Resisting	0.18
F_{v4}	1.25	0.25	Vertical	Resisting	0.31
F_{h1}	1404.49	5.59	Horizontal	Overturning	7846.41
F_{s1}	0	15.44	Vertical	Resisting	0.00
F_{s2}	0	14.69	Vertical	Resisting	0.00
F_{f1}	0	0.50	Vertical	Resisting	0.00
F_{f2}	0	0.25	Vertical	Resisting	0.00
F_{silt}	177.85	1.66	Horizontal	Overturning	294.63
F_{fill}	277.89	1.66	Horizontal	Resisting	460.36
W_1	0.00	14.69	Vertical	Resisting	0.00
W_2	962.06	12.99	Vertical	Resisting	12497.21
W_3	1397.09	7.69	Vertical	Resisting	10748.27
W_4	1931.14	8.10	Vertical	Resisting	15632.60
F_{u1}	845.12	8.10	Vertical	Overturning	6841.23
F_{u2}	934.16	10.79	Vertical	Overturning	10082.73
F_{h2}	136.24	1.74	Horizontal	Resisting	237.06

Table B.4 Determination of factor of safety values and stresses belonging to dam body for unusual case

			Situation
Net moment (ΔM)	M_R-M_o	17241.57	
Overturning Safety	$FS_o = (M_R/M_o)$	1.69	**Ok**
Sliding Safety	$FS_s = (f \sum V / \sum H)$	1.73	**Ok**
Shear-Sliding Safety	$FS_{ss} = (f \sum V + rA\tau_s) / (\sum H)$	29.17	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	62.44	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	269.80	**Ok**
Base Width	B (m)	16.19	
Total net force	$\sum F$ (kN/m)	2689.43	
Eccentricity	e	6.41	
The distance from center	x	1.68	

Table B.5 Stability analysis of dam body for extreme case

Extreme Case					
Forces	**Value (kN/m)**	**Moment wrt Toe (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_i	100.00	13.97	Horizontal	Overturning	1397.00
F_{v1}	135.00	15.44	Vertical	Resisting	2084.40
F_{v2}	0.00	0.00	Vertical	Resisting	0.00
F_{v3}	0.31	0.58	Vertical	Resisting	0.18
F_{v4}	1.25	0.25	Vertical	Resisting	0.31
F_{h1}	975.80	4.66	Horizontal	Overturning	4544.00
F_{s1}	0	15.44	Vertical	Resisting	0.00
F_{s2}	0	14.69	Vertical	Resisting	0.00
F_{f1}	0	0.50	Vertical	Resisting	0.00
F_{f2}	0	0.25	Vertical	Resisting	0.00
F_{silt}	177.85	1.66	Horizontal	Overturning	294.63
F_{dsilt}	22.60	2.05	Horizontal	Overturning	46.27
F_w	41.16	8.68	Horizontal	Overturning	357.22
F_{fill}	277.89	1.66	Horizontal	Resisting	460.36
W_1	0.00	14.69	Vertical	Resisting	0.00
W_2	962.06	12.99	Vertical	Resisting	12497.21
W_3	1397.09	7.69	Vertical	Resisting	10748.27
W_4	1931.14	8.10	Vertical	Resisting	15632.60
F_{u1}	845.12	8.10	Vertical	Overturning	6841.23
F_{u2}	934.16	10.79	Vertical	Overturning	10082.73
F_{h2}	136.24	1.74	Horizontal	Resisting	237.06
F_{dv1}	0	14.69	Vertical	Overturning	0.00
F_{dh1}	0	3.25	Horizontal	Overturning	0.00
F_{dv2}	96.21	12.99	Vertical	Overturning	1249.72
F_{dh2}	96.21	10.87	Horizontal	Overturning	1045.28
F_{dv3}	139.71	7.69	Vertical	Overturning	1074.83
F_{dh3}	139.71	8.57	Horizontal	Overturning	1196.84
F_{dv4}	193.11	8.10	Vertical	Overturning	1563.26
F_{dh4}	193.11	2.49	Horizontal	Overturning	479.89

Table B.6 Determination of factor of safety values and stresses belonging to dam body for extreme case

			Situation
Net moment (ΔM)	M_R-M_o	11487.51	
Overturning Safety	$FS_o = (M_R/M_o)$	1.38	**Ok**
Sliding Safety	$FS_s = (f \sum V / \sum H)$	1.25	**Ok**
Shear-Sliding Safety	$FS_{ss} = (f \sum V + rA\tau_s) / (\sum H)$	25.31	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	0.00	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	285.64	**Ok**
Base Width	B (m)	16.19	
Total net force	$\sum F$ (kN/m)	2218.55	
Eccentricity	e	5.18	
The distance from center	x	2.92	

Table B.7 Stability analysis of dam body for empty case

Empty Case					
Forces	**Value (kN/m)**	**Moment wrt Heel (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_{v3}	0.31	16.61	Vertical	Resisting	5.19
F_{v4}	1.25	15.94	Vertical	Resisting	19.93
F_{s1}	0.00	0.75	Vertical	Resisting	0.00
F_{s2}	0.00	1.50	Vertical	Resisting	0.00
F_{f1}	0.00	15.69	Vertical	Resisting	0.00
F_{f2}	0.00	15.94	Vertical	Resisting	0.00
F_{fill}	177.85	1.66	Horizontal	Overturning	294.63
W_1	0.00	1.50	Vertical	Resisting	0.00
W_2	962.06	3.20	Vertical	Resisting	3078.60
W_3	1397.09	8.50	Vertical	Resisting	11870.60
W_4	1931.14	8.10	Vertical	Resisting	15632.60
F_{u1}	422.56	10.79	Vertical	Overturning	4560.82
F_{h2}	136.24	1.74	Horizontal	Overturning	237.06
F_{dv1}	17.78	1.66	Vertical	Overturning	29.46
F_{dh1}	17.78	3.25	Horizontal	Overturning	57.80
F_{dv2}	0.00	1.50	Vertical	Overturning	0.00
F_{dh2}	0.00	10.87	Horizontal	Overturning	0.00
F_{dv3}	96.21	3.20	Vertical	Overturning	307.86
F_{dh3}	96.21	8.57	Horizontal	Overturning	824.17
F_{dv4}	139.71	8.10	Vertical	Overturning	1130.94
F_{dh4}	139.71	2.49	Horizontal	Overturning	347.18

Table B.8 Determination of factor of safety values and stresses belonging to dam body for empty case

			Situation
Net moment (ΔM)	M_R-M_o	19734.22	
Overturning Safety	$FS_o = (M_R/M_o)$	2.82	**Ok**
Sliding Safety	$FS_s = (f \sum V / \sum H)$	3.47	**Ok**
Shear-Sliding Safety	$FS_{ss} = (f \sum V + rA\tau_s) / (\sum H)$	46.61	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	624.15	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	0.00	**Ok**
Base Width	B (m)	16.19	
Total net force	$\sum F$ (kN/m)	4298.33	
Eccentricity	e	4.59	
The distance from center	x	3.50	

APPENDIX C

STABILITY ANALYSIS OF SPILLWAY

Table C.1 Stability analysis of spillway for usual case

Usual Case					
Forces	**Value (kN/m)**	**Moment wrt Toe (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_i	100.00	13.97	Horizontal	Overturning	1397.00
F_{v1}	135.00	36.52	Vertical	Resisting	4930.20
F_{v2}	81.00	35.17	Vertical	Resisting	2848.77
F_{h1}	975.80	4.66	Horizontal	Overturning	4544.00
F_{silt}	71.57	1.66	Horizontal	Overturning	118.56
F_{fill}	197.01	1.32	Horizontal	Resisting	260.71
W_1	178.92	36.52	Vertical	Resisting	6534.16
W_2	2338.56	31.56	Vertical	Resisting	73804.95
W_3	948.96	13.73	Vertical	Resisting	13024.48
W_4	46.57	2.53	Vertical	Resisting	117.67
W_5	94.56	2.86	Vertical	Resisting	269.97
W_6	178.17	0.94	Vertical	Resisting	166.59
F_u	1431.82	24.85	Vertical	Overturning	35575.96

Table C.2 Determination of factor of safety values and stresses belonging to spillway for usual case

			Situation
Net moment (ΔM)	M_R-M_o	60321.98	
Overturning Safety	$FS_o = (M_R/M_o)$	2.45	**Ok**
Sliding Safety	$FS_s = (f\ \sum V\ /\ \sum H)$	2.03	**Ok**
Uplift Force Safety	$FS_{ss} = (f\ \sum V + rA\tau_s)\ /\ (\sum H)$	2.64	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	122.65	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	15.26	**Ok**
Base Width	B (m)	37.27	
Total net force	$\sum F$ (kN/m)	2569.92	
Eccentricity	e	23.47	
The distance from center	x	-4.84	

Table C.3 Stability analysis of spillway for extreme case

Extreme Case					
Forces	**Value (kN/m)**	**Moment wrt Toe (m)**	**Direction of force**	**Type of force**	**Moment (kN/m.m)**
F_i	100.00	13.97	Horizontal	Overturning	1397.00
F_{v1}	135.00	36.52	Vertical	Resisting	4930.20
F_{v2}	81.00	35.17	Vertical	Resisting	2848.77
F_{h1}	975.80	4.66	Horizontal	Overturning	4544.00
F_{silt}	28.80	1.66	Horizontal	Overturning	47.71
$F_{d\ silt}$	3.66	2.05	Horizontal	Overturning	7.49
F_w	35.99	8.68	Horizontal	Overturning	312.33
F_{fill}	197.01	1.32	Horizontal	Resisting	260.71
W_1	178.92	36.52	Vertical	Resisting	6534.16
W_2	2338.56	31.56	Vertical	Resisting	73804.95
W_3	948.96	13.73	Vertical	Resisting	13024.48
W_4	46.57	2.53	Vertical	Resisting	117.67
W_5	94.56	2.86	Vertical	Resisting	269.97
W_6	178.17	0.94	Vertical	Resisting	166.59
F_u	1431.82	24.85	Vertical	Overturning	35575.96
F_{dv1}	17.89	36.52	Vertical	Overturning	653.42
F_{dh1}	17.89	2.49	Horizontal	Overturning	44.46
F_{dv2}	233.86	31.56	Vertical	Overturning	7380.50
F_{dh2}	233.86	5.17	Horizontal	Overturning	1209.04
F_{dv3}	94.90	13.73	Vertical	Overturning	1302.45
F_{dh3}	94.90	1.00	Horizontal	Overturning	94.90
F_{dv4}	4.66	2.53	Vertical	Overturning	11.77
F_{dh4}	4.66	2.66	Horizontal	Overturning	12.37
F_{dv5}	9.46	2.86	Vertical	Overturning	27.00
F_{dh5}	9.46	1.00	Horizontal	Overturning	9.46
F_{dv6}	17.82	0.94	Vertical	Overturning	16.66
F_{dh6}	17.82	1.99	Horizontal	Overturning	35.37

Table C.4 Determination of factor of safety values and stresses belonging to spillway for extreme case

			Situation
Net moment (ΔM)	M_R-M_o	49275.64	
Overturning Safety	$FS_o = (M_R/M_o)$	1.94	**Ok**
Sliding Safety	$FS_s = (f \sum V / \sum H)$	1.24	**Ok**
Uplift Force Safety	$FS_{ss} = (f \sum V + rA\tau_s) / (\sum H)$	2.38	**Ok**
Stress at heel	σ_{heel} (kN/m^2)	95.25	**Ok**
Stress at toe	σ_{toe} (kN/m^2)	22.34	**Ok**
Base Width	B (m)	37.27	
Total net force	$\sum F$ (kN/m)	2191.35	
Eccentricity	e	22.49	
The distance from center	x	-3.85	

APPENDIX D

OPERATION STUDY

Table D.1 Operation study for Case 1

Date [1]	Incoming Discharge (m^3/s) [2]	Incoming Discharge Volume (m^3) [3]	Initial Reservoir Volume (m^3) [4]	Minimum Water Volume Released (m^3) [5]	Water Volume Stored (m^3) [6]
1-Oct-65	0.19	16813.68	125120.00	16813.68	0.00
2-Oct-65	0.19	16813.68	0.00	16813.68	0.00
3-Oct-65	0.19	16813.68	0.00	16813.68	0.00
4-Oct-65	0.19	16813.68	0.00	16813.68	0.00
5-Oct-65	0.31	26992.04	0.00	18576.00	8416.04
6-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
7-Oct-65	0.22	19112.76	0.00	18576.00	536.76
8-Oct-65	0.22	19112.76	0.00	18576.00	536.76
9-Oct-65	0.22	19112.76	0.00	18576.00	536.76
10-Oct-65	0.22	19112.76	0.00	18576.00	536.76
11-Oct-65	0.22	19112.76	0.00	18576.00	536.76
12-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
13-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
14-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
15-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
16-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
17-Oct-65	0.33	28392.52	0.00	18576.00	9816.52
18-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
19-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
20-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
21-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
22-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
23-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
24-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
25-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
26-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
27-Oct-65	0.30	25594.57	0.00	18576.00	7018.57
28-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
29-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
30-Oct-65	0.26	22345.48	0.00	18576.00	3769.48
31-Oct-65	0.26	22345.48	0.00	18576.00	3769.48

Table D.1 (Continued)

Date	Turbined Water Volume in Off-peak Period (m^3) [7]	Turbined Water Volume in Peak Period (m^3) [8]	Max. Available Water Volume (m^3) [9]	Water Volume Released (m^3) [10]	Final Reservoir Volume (m^3) [11]
1-Oct-65	0.00	125120.00	675,886	0.00	0.00
2-Oct-65	0.00	0.00	801,006	0.00	0.00
3-Oct-65	0.00	0.00	801,006	0.00	0.00
4-Oct-65	0.00	0.00	801,006	0.00	0.00
5-Oct-65	0.00	8416.04	802,768	0.00	0.00
6-Oct-65	0.00	3769.48	802,768	0.00	0.00
7-Oct-65	0.00	536.76	802,768	0.00	0.00
8-Oct-65	0.00	536.76	802,768	0.00	0.00
9-Oct-65	0.00	536.76	802,768	0.00	0.00
10-Oct-65	0.00	536.76	802,768	0.00	0.00
11-Oct-65	0.00	536.76	802,768	0.00	0.00
12-Oct-65	0.00	3769.48	802,768	0.00	0.00
13-Oct-65	0.00	3769.48	802,768	0.00	0.00
14-Oct-65	0.00	7018.57	802,768	0.00	0.00
15-Oct-65	0.00	7018.57	802,768	0.00	0.00
16-Oct-65	0.00	7018.57	802,768	0.00	0.00
17-Oct-65	0.00	9816.52	802,768	0.00	0.00
18-Oct-65	0.00	7018.57	802,768	0.00	0.00
19-Oct-65	0.00	7018.57	802,768	0.00	0.00
20-Oct-65	0.00	7018.57	802,768	0.00	0.00
21-Oct-65	0.00	7018.57	802,768	0.00	0.00
22-Oct-65	0.00	7018.57	802,768	0.00	0.00
23-Oct-65	0.00	7018.57	802,768	0.00	0.00
24-Oct-65	0.00	7018.57	802,768	0.00	0.00
25-Oct-65	0.00	7018.57	802,768	0.00	0.00
26-Oct-65	0.00	7018.57	802,768	0.00	0.00
27-Oct-65	0.00	7018.57	802,768	0.00	0.00
28-Oct-65	0.00	3769.48	802,768	0.00	0.00
29-Oct-65	0.00	3769.48	802,768	0.00	0.00
30-Oct-65	0.00	3769.48	802,768	0.00	0.00
31-Oct-65	0.00	3769.48	802,768	0.00	0.00

Table D.1 (Continued)

Date	Energy Generated in Off-Peak Period (kWh) [12]	Energy Generated in Peak Period (kWh) [13]	Income in Off-Peak Period (TL) [14]	Income in Peak Period (TL) [15]	Total Income (TL) [16]
1-Oct-65	0.00	291,737	0.00	42716.08	42716.08
2-Oct-65	0.00	0	0.00	0.00	0.00
3-Oct-65	0.00	0	0.00	0.00	0.00
4-Oct-65	0.00	0	0.00	0.00	0.00
5-Oct-65	0.00	19,623	0.00	2873.24	2873.24
6-Oct-65	0.00	8,789	0.00	1286.90	1286.90
7-Oct-65	0.00	1,252	0.00	183.25	183.25
8-Oct-65	0.00	1,252	0.00	183.25	183.25
9-Oct-65	0.00	1,252	0.00	183.25	183.25
10-Oct-65	0.00	1,252	0.00	183.25	183.25
11-Oct-65	0.00	1,252	0.00	183.25	183.25
12-Oct-65	0.00	8,789	0.00	1286.90	1286.90
13-Oct-65	0.00	8,789	0.00	1286.90	1286.90
14-Oct-65	0.00	16,365	0.00	2396.15	2396.15
15-Oct-65	0.00	16,365	0.00	2396.15	2396.15
16-Oct-65	0.00	16,365	0.00	2396.15	2396.15
17-Oct-65	0.00	22,889	0.00	3351.37	3351.37
18-Oct-65	0.00	16,365	0.00	2396.15	2396.15
19-Oct-65	0.00	16,365	0.00	2396.15	2396.15
20-Oct-65	0.00	16,365	0.00	2396.15	2396.15
21-Oct-65	0.00	16,365	0.00	2396.15	2396.15
22-Oct-65	0.00	16,365	0.00	2396.15	2396.15
23-Oct-65	0.00	16,365	0.00	2396.15	2396.15
24-Oct-65	0.00	16,365	0.00	2396.15	2396.15
25-Oct-65	0.00	16,365	0.00	2396.15	2396.15
26-Oct-65	0.00	16,365	0.00	2396.15	2396.15
27-Oct-65	0.00	16,365	0.00	2396.15	2396.15
28-Oct-65	0.00	8,789	0.00	1286.90	1286.90
29-Oct-65	0.00	8,789	0.00	1286.90	1286.90
30-Oct-65	0.00	8,789	0.00	1286.90	1286.90
31-Oct-65	0.00	8,789	0.00	1286.90	1286.90

Table D.2 Operation study for Case 2

Date [1]	Incoming Discharge (m^3/s) [2]	Incoming Discharge Volume (m^3) [3]	Initial Reservoir Volume (m^3) [4]	Minimum Water Volume Released (m^3) [5]	Water Volume Stored (m^3) [6]
1-Feb-84	3.17	273728.09	0.00	18576.00	250240.00
2-Feb-84	3.17	273728.09	0.00	18576.00	250240.00
3-Feb-84	3.08	266333.38	0.00	18576.00	247757.38
4-Feb-84	3.08	266333.38	0.00	18576.00	247757.38
5-Feb-84	3.00	258986.75	0.00	18576.00	240410.75
6-Feb-84	2.42	208906.41	0.00	18576.00	190330.41
7-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
8-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
9-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
10-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
11-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
12-Feb-84	2.34	201944.38	0.00	18576.00	183368.38
13-Feb-84	2.50	215916.52	0.00	18576.00	197340.52
14-Feb-84	3.93	339829.37	0.00	18576.00	250240.00
15-Feb-84	3.84	331369.23	0.00	18576.00	250240.00
16-Feb-84	3.84	331369.23	0.00	18576.00	250240.00
17-Feb-84	3.84	331369.23	0.00	18576.00	250240.00
18-Feb-84	3.64	314618.23	0.00	18576.00	250240.00
19-Feb-84	3.45	298092.91	0.00	18576.00	250240.00
20-Feb-84	3.17	273728.09	0.00	18576.00	250240.00
21-Feb-84	3.17	273728.09	0.00	18576.00	250240.00
22-Feb-84	2.91	251688.19	0.00	18576.00	233112.19
23-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
24-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
25-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
26-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
27-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
28-Feb-84	2.83	244437.70	0.00	18576.00	225861.70
29-Feb-84	2.83	244437.70	0.00	18576.00	225861.70

Table D-2 (Continued)

Date [1]	Turbined Water Volume in Base Period (m^3) [7]	Turbined Water Volume in Peak Period (m^3) [8]	Max. Available Water Volume (m^3) [9]	Water Volume Released (m^3) [10]	Final Reservoir Volume (m^3) [11]
1-Feb-84	4912.09	250240.00	802768.00	0.00	0.00
2-Feb-84	4912.09	250240.00	802768.00	0.00	0.00
3-Feb-84	0.00	247757.38	802768.00	0.00	0.00
4-Feb-84	0.00	247757.38	802768.00	0.00	0.00
5-Feb-84	0.00	240410.75	802768.00	0.00	0.00
6-Feb-84	0.00	190330.41	802768.00	0.00	0.00
7-Feb-84	0.00	183368.38	802768.00	0.00	0.00
8-Feb-84	0.00	183368.38	802768.00	0.00	0.00
9-Feb-84	0.00	183368.38	802768.00	0.00	0.00
10-Feb-84	0.00	183368.38	802768.00	0.00	0.00
11-Feb-84	0.00	183368.38	802768.00	0.00	0.00
12-Feb-84	0.00	183368.38	802768.00	0.00	0.00
13-Feb-84	0.00	197340.52	802768.00	0.00	0.00
14-Feb-84	71013.37	250240.00	802768.00	0.00	0.00
15-Feb-84	62553.23	250240.00	802768.00	0.00	0.00
16-Feb-84	62553.23	250240.00	802768.00	0.00	0.00
17-Feb-84	62553.23	250240.00	802768.00	0.00	0.00
18-Feb-84	45802.23	250240.00	802768.00	0.00	0.00
19-Feb-84	29276.91	250240.00	802768.00	0.00	0.00
20-Feb-84	4912.09	250240.00	802768.00	0.00	0.00
21-Feb-84	4912.09	250240.00	802768.00	0.00	0.00
22-Feb-84	0.00	233112.19	802768.00	0.00	0.00
23-Feb-84	0.00	225861.70	802768.00	0.00	0.00
24-Feb-84	0.00	225861.70	802768.00	0.00	0.00
25-Feb-84	0.00	225861.70	802768.00	0.00	0.00
26-Feb-84	0.00	225861.70	802768.00	0.00	0.00
27-Feb-84	0.00	225861.70	802768.00	0.00	0.00
28-Feb-84	0.00	225861.70	802768.00	0.00	0.00
29-Feb-84	0.00	225861.70	802768.00	0.00	0.00

Table D.2 (Continued)

Date [1]	Energy Generated in Off-Peak Period (kWh) [12]	Energy Generated in Peak Period (kWh) [13]	Income in Off-Peak Period (TL) [14]	Income in Peak Period (TL) [15]	Total Income (TL) [16]
1-Feb-84	11453.30	583473.52	1202.87	85432.15	86635.03
2-Feb-84	11453.30	583473.52	1202.87	85432.15	86635.03
3-Feb-84	0.00	577684.90	0.00	84584.59	84584.59
4-Feb-84	0.00	577684.90	0.00	84584.59	84584.59
5-Feb-84	0.00	560555.08	0.00	82076.44	82076.44
6-Feb-84	0.00	443784.99	0.00	64978.97	64978.97
7-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
8-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
9-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
10-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
11-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
12-Feb-84	0.00	427551.93	0.00	62602.13	62602.13
13-Feb-84	0.00	460130.15	0.00	67372.23	67372.23
14-Feb-84	165578.72	583473.52	17389.75	85432.15	102821.90
15-Feb-84	145852.60	583473.52	15318.03	85432.15	100750.19
16-Feb-84	145852.60	583473.52	15318.03	85432.15	100750.19
17-Feb-84	145852.60	583473.52	15318.03	85432.15	100750.19
18-Feb-84	106795.02	583473.52	11216.05	85432.15	96648.20
19-Feb-84	68263.67	583473.52	7169.33	85432.15	92601.48
20-Feb-84	11453.30	583473.52	1202.87	85432.15	86635.03
21-Feb-84	11453.30	583473.52	1202.87	85432.15	86635.03
22-Feb-84	0.00	543537.36	0.00	79584.70	79584.70
23-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
24-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
25-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
26-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
27-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
28-Feb-84	0.00	526631.73	0.00	77109.38	77109.38
29-Feb-84	0.00	526631.73	0.00	77109.38	77109.38

Table D.3 Operation study for Case 3

Date [1]	Incoming Discharge (m^3/s) [2]	Incoming Discharge Volume (m^3) [3]	Initial Reservoir Volume (m^3) [4]	Minimum Water Volume Released (m^3) [5]	Water Volume Stored (m^3) [6]
1-Apr-84	7.69	664249.34	250240.00	18576.00	0.00
2-Apr-84	7.41	640344.12	250240.00	18576.00	0.00
3-Apr-84	7.41	640344.12	250240.00	18576.00	0.00
4-Apr-84	7.41	640344.12	250240.00	18576.00	0.00
5-Apr-84	7.41	640344.12	250240.00	18576.00	0.00
6-Apr-84	7.32	632442.49	250240.00	18576.00	0.00
7-Apr-84	7.60	656247.55	250240.00	18576.00	0.00
8-Apr-84	8.16	704759.09	250240.00	18576.00	0.00
9-Apr-84	8.16	704759.09	250240.00	18576.00	0.00
10-Apr-84	7.14	616739.37	250240.00	18576.00	0.00
11-Apr-84	6.34	547727.93	250240.00	18576.00	0.00
12-Apr-84	5.99	517924.19	0.00	18576.00	250240.00
13-Apr-84	5.64	487205.17	0.00	18576.00	250240.00
14-Apr-84	5.64	487205.17	0.00	18576.00	250240.00
15-Apr-84	7.69	664249.34	0.00	18576.00	250240.00
16-Apr-84	7.41	640344.12	0.00	18576.00	250240.00
17-Apr-84	6.87	593435.09	0.00	18576.00	250240.00
18-Apr-84	5.76	497379.41	0.00	18576.00	250240.00
19-Apr-84	5.76	497379.41	0.00	18576.00	250240.00
20-Apr-84	6.60	570431.27	0.00	18576.00	250240.00
21-Apr-84	5.99	517924.19	0.00	18576.00	250240.00
22-Apr-84	5.64	487205.17	0.00	18576.00	250240.00
23-Apr-84	5.41	467053.00	0.00	18576.00	250240.00
24-Apr-84	5.30	457785.61	0.00	18576.00	250240.00
25-Apr-84	5.19	448574.64	0.00	18576.00	250240.00
26-Apr-84	5.19	448574.64	0.00	18576.00	250240.00
27-Apr-84	5.22	450695.24	0.00	18576.00	250240.00
28-Apr-84	6.25	540226.92	0.00	18576.00	250240.00
29-Apr-84	7.60	656247.55	0.00	18576.00	250240.00
30-Apr-84	6.51	562830.11	0.00	18576.00	250240.00

Table D.3 (Continued)

Date [1]	Turbined Water Volume in Base Period (m^3) [7]	Turbined Water Volume in Peak Period (m^3) [8]	Max. Available Water Volume (m^3) [9]	Water Volume Released (m^3) [10]	Final Reservoir Volume (m^3) [11]
1-Apr-84	283712.00	250240.00	552528.00	111721.34	250240.00
2-Apr-84	283712.00	250240.00	552528.00	87816.12	250240.00
3-Apr-84	283712.00	250240.00	552528.00	87816.12	250240.00
4-Apr-84	283712.00	250240.00	552528.00	87816.12	250240.00
5-Apr-84	283712.00	250240.00	552528.00	87816.12	250240.00
6-Apr-84	283712.00	250240.00	552528.00	79914.49	250240.00
7-Apr-84	283712.00	250240.00	552528.00	103719.55	250240.00
8-Apr-84	283712.00	250240.00	552528.00	152231.09	250240.00
9-Apr-84	283712.00	250240.00	552528.00	152231.09	250240.00
10-Apr-84	283712.00	250240.00	552528.00	64211.37	250240.00
11-Apr-84	283712.00	250240.00	552528.00	0.00	0.00
12-Apr-84	249108.19	250240.00	802768.00	0.00	0.00
13-Apr-84	218389.17	250240.00	802768.00	0.00	0.00
14-Apr-84	218389.17	250240.00	802768.00	0.00	0.00
15-Apr-84	283712.00	250240.00	802768.00	0.00	0.00
16-Apr-84	283712.00	250240.00	802768.00	0.00	0.00
17-Apr-84	283712.00	250240.00	802768.00	0.00	0.00
18-Apr-84	228563.41	250240.00	802768.00	0.00	0.00
19-Apr-84	228563.41	250240.00	802768.00	0.00	0.00
20-Apr-84	283712.00	250240.00	802768.00	0.00	0.00
21-Apr-84	249108.19	250240.00	802768.00	0.00	0.00
22-Apr-84	218389.17	250240.00	802768.00	0.00	0.00
23-Apr-84	198237.00	250240.00	802768.00	0.00	0.00
24-Apr-84	188969.61	250240.00	802768.00	0.00	0.00
25-Apr-84	179758.64	250240.00	802768.00	0.00	0.00
26-Apr-84	179758.64	250240.00	802768.00	0.00	0.00
27-Apr-84	181879.24	250240.00	802768.00	0.00	0.00
28-Apr-84	271410.92	250240.00	802768.00	0.00	0.00
29-Apr-84	283712.00	250240.00	802768.00	0.00	0.00
30-Apr-84	283712.00	250240.00	802768.00	0.00	0.00

Table D.3 (Continued)

Date [1]	Energy Generated in Off-Peak Period (kWh) [12]	Energy Generated in Peak Period (kWh) [13]	Income in Off-Peak Period (TL) [14]	Income in Peak Period (TL) [15]	Total Income (TL) [16]
1-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
2-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
3-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
4-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
5-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
6-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
7-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
8-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
9-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
10-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
11-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
12-Apr-84	580834.54	583473.52	61001.60	85432.15	146433.76
13-Apr-84	509208.35	583473.52	53479.13	85432.15	138911.28
14-Apr-84	509208.35	583473.52	53479.13	85432.15	138911.28
15-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
16-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
17-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
18-Apr-84	532931.18	583473.52	55970.60	85432.15	141402.75
19-Apr-84	532931.18	583473.52	55970.60	85432.15	141402.75
20-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
21-Apr-84	580834.54	583473.52	61001.60	85432.15	146433.76
22-Apr-84	509208.35	583473.52	53479.13	85432.15	138911.28
23-Apr-84	462220.43	583473.52	48544.27	85432.15	133976.42
24-Apr-84	440612.06	583473.52	46274.87	85432.15	131707.02
25-Apr-84	419135.24	583473.52	44019.29	85432.15	129451.44
26-Apr-84	419135.24	583473.52	44019.29	85432.15	129451.44
27-Apr-84	424079.76	583473.52	44538.58	85432.15	129970.73
28-Apr-84	632836.81	583473.52	66463.09	85432.15	151895.25
29-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54
30-Apr-84	661518.70	583473.52	69475.38	85432.15	154907.54

Table D.4 Operation study for Case 4

Date [1]	Incoming Discharge (m^3/s) [2]	Incoming Discharge Volume (m^3) [3]	Initial Reservoir Volume (m^3) [4]	Minimum Water Volume Released (m^3) [5]	Water Volume Stored (m^3) [6]
1-Mar-69	17.22	1488204.58	250240.00	18576.00	0.00
2-Mar-69	14.30	1235667.22	250240.00	18576.00	0.00
3-Mar-69	13.49	1165632.81	250240.00	18576.00	0.00
4-Mar-69	13.72	1185475.71	250240.00	18576.00	0.00
5-Mar-69	13.72	1185475.71	250240.00	18576.00	0.00
6-Mar-69	16.10	1391249.55	250240.00	18576.00	0.00
7-Mar-69	17.86	1543236.97	250240.00	18576.00	0.00
8-Mar-69	17.61	1521123.86	250240.00	18576.00	0.00
9-Mar-69	16.10	1391249.55	250240.00	18576.00	0.00
10-Mar-69	12.48	1077992.31	250240.00	18576.00	0.00
11-Mar-69	11.49	993056.03	250240.00	18576.00	0.00
12-Mar-69	10.13	875144.41	250240.00	18576.00	0.00
13-Mar-69	9.22	796818.43	250240.00	18576.00	0.00
14-Mar-69	9.72	839999.01	250240.00	18576.00	0.00
15-Mar-69	8.73	754472.50	250240.00	18576.00	0.00
16-Mar-69	8.54	737767.82	250240.00	18576.00	0.00
17-Mar-69	11.17	965344.87	250240.00	18576.00	0.00
18-Mar-69	28.57	2468879.02	250240.00	18576.00	0.00
19-Mar-69	31.31	2704798.61	250240.00	18576.00	0.00
20-Mar-69	28.57	2468879.02	250240.00	18576.00	0.00
21-Mar-69	29.37	2537265.57	250240.00	18576.00	0.00
22-Mar-69	28.57	2468879.02	250240.00	18576.00	0.00
23-Mar-69	21.75	1878883.84	250240.00	18576.00	0.00
24-Mar-69	17.61	1521123.86	250240.00	18576.00	0.00
25-Mar-69	14.30	1235667.22	250240.00	18576.00	0.00
26-Mar-69	13.38	1155761.43	250240.00	18576.00	0.00
27-Mar-69	14.07	1215490.46	250240.00	18576.00	0.00
28-Mar-69	16.97	1466425.32	250240.00	18576.00	0.00
29-Mar-69	18.90	1633024.83	250240.00	18576.00	0.00
30-Mar-69	17.86	1543236.97	250240.00	18576.00	0.00
31-Mar-69	16.47	1423267.43	250240.00	18576.00	0.00

Table D.4 (Continued)

Date [1]	Turbined Water Volume in Base Period (m^3) [7]	Turbined Water Volume in Peak Period (m^3) [8]	Max. Available Water Volume (m^3) [9]	Water Volume Released (m^3) [10]	Final Reservoir Volume (m^3) [11]
1-Mar-69	283712.00	250240.00	552528.00	935676.58	250240.00
2-Mar-69	283712.00	250240.00	552528.00	683139.22	250240.00
3-Mar-69	283712.00	250240.00	552528.00	613104.81	250240.00
4-Mar-69	283712.00	250240.00	552528.00	632947.71	250240.00
5-Mar-69	283712.00	250240.00	552528.00	632947.71	250240.00
6-Mar-69	283712.00	250240.00	552528.00	838721.55	250240.00
7-Mar-69	283712.00	250240.00	552528.00	990708.97	250240.00
8-Mar-69	283712.00	250240.00	552528.00	968595.86	250240.00
9-Mar-69	283712.00	250240.00	552528.00	838721.55	250240.00
10-Mar-69	283712.00	250240.00	552528.00	525464.31	250240.00
11-Mar-69	283712.00	250240.00	552528.00	440528.03	250240.00
12-Mar-69	283712.00	250240.00	552528.00	322616.41	250240.00
13-Mar-69	283712.00	250240.00	552528.00	244290.43	250240.00
14-Mar-69	283712.00	250240.00	552528.00	287471.01	250240.00
15-Mar-69	283712.00	250240.00	552528.00	201944.50	250240.00
16-Mar-69	283712.00	250240.00	552528.00	185239.82	250240.00
17-Mar-69	283712.00	250240.00	552528.00	412816.87	250240.00
18-Mar-69	283712.00	250240.00	552528.00	1916351.02	250240.00
19-Mar-69	283712.00	250240.00	552528.00	2152270.61	250240.00
20-Mar-69	283712.00	250240.00	552528.00	1916351.02	250240.00
21-Mar-69	283712.00	250240.00	552528.00	1984737.57	250240.00
22-Mar-69	283712.00	250240.00	552528.00	1916351.02	250240.00
23-Mar-69	283712.00	250240.00	552528.00	1326355.84	250240.00
24-Mar-69	283712.00	250240.00	552528.00	968595.86	250240.00
25-Mar-69	283712.00	250240.00	552528.00	683139.22	250240.00
26-Mar-69	283712.00	250240.00	552528.00	603233.43	250240.00
27-Mar-69	283712.00	250240.00	552528.00	662962.46	250240.00
28-Mar-69	283712.00	250240.00	552528.00	913897.32	250240.00
29-Mar-69	283712.00	250240.00	552528.00	1080496.83	250240.00
30-Mar-69	283712.00	250240.00	552528.00	990708.97	250240.00
31-Mar-69	283712.00	250240.00	552528.00	870739.43	250240.00

Table D.4 (Continued)

Date [1]	Energy Generated in Off-Peak Period (kWh) [12]	Energy Generated in Peak Period (kWh) [13]	Income in Off-Peak Period (TL) [14]	Income in Peak Period (TL) [15]	Total Income (TL) [16]
1-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
2-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
3-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
4-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
5-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
6-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
7-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
8-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
9-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
10-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
11-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
12-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
13-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
14-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
15-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
16-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
17-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
18-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
19-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
20-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
21-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
22-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
23-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
24-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
25-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
26-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
27-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
28-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
29-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
30-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54
31-Mar-69	661518.70	583473.52	69475.38	85432.15	154907.54

APPENDIX E

ELECTRICITY PRICES

Table E.1 Hourly electricity prices for winter time (January, 2011)

January, 2011								
Hours/Days	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
0-1	110.00	120.00	110.00	132.00	115.00	116.00	110.00	130.00
1-2	120.00	120.00	48.00	86.51	104.51	104.51	98.51	115.00
2-3	81.00	85.00	26.56	47.00	105.00	71.51	78.51	105.00
3-4	11.70	49.00	26.26	47.00	71.51	71.51	120.00	89.51
4-5	11.03	49.00	26.26	47.00	71.51	71.51	89.00	89.51
5-6	10.98	49.00	26.46	47.00	71.51	71.51	122.00	89.51
6-7	11.70	49.00	47.00	89.00	115.00	110.00	78.51	89.51
7-8	11.90	47.00	125.00	125.00	100.00	115.00	100.00	80.00
8-9	76.00	100.00	140.00	142.00	150.00	164.90	165.00	141.00
9-10	77.00	100.00	120.00	160.00	166.40	166.90	166.40	150.00
10-11	94.51	120.00	140.00	165.00	171.00	169.00	169.00	164.50
11-12	110.00	139.50	141.00	165.00	173.00	170.00	167.40	165.50
12-13	81.00	120.00	99.51	141.00	165.00	165.40	164.40	140.00
13-14	94.51	120.00	120.00	147.00	165.40	165.90	164.50	164.40
14-15	110.00	120.00	130.00	164.50	165.50	166.90	165.90	164.50
15-16	125.00	120.00	140.00	150.00	165.50	166.90	165.90	164.00
16-17	130.00	125.00	141.00	164.90	166.90	169.00	166.90	164.90
17-18	141.00	141.00	150.00	164.90	168.00	173.50	170.50	165.50
18-19	140.00	140.00	140.00	160.00	166.90	165.90	165.50	164.90
19-20	125.00	120.00	130.00	140.00	159.00	164.50	155.00	147.00
20-21	105.00	120.00	120.00	130.00	143.00	145.00	142.00	130.00
21-22	89.51	100.00	99.51	104.51	140.00	140.00	140.00	115.00
22-23	125.00	125.00	140.00	164.00	164.90	150.00	147.00	142.00
23-24	89.51	120.00	120.00	142.00	143.00	141.00	140.00	120.00

Table E.1 (Continued)

January, 2011								
Hours/Days	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
0-1	142.00	127.00	130.00	140.00	141.00	141.00	104.51	125.00
1-2	110.00	115.00	116.00	120.00	104.51	100.00	79.00	100.00
2-3	115.00	85.51	99.51	103.51	102.51	99.51	79.00	90.00
3-4	79.00	18.37	93.51	102.51	21.00	99.51	47.00	48.00
4-5	47.00	18.37	93.51	102.51	16.00	120.00	16.00	47.00
5-6	47.00	34.00	93.51	110.00	21.00	99.51	16.00	47.00
6-7	47.00	115.00	105.00	115.00	100.00	110.00	79.00	47.00
7-8	47.00	125.00	115.00	125.00	98.51	119.00	95.00	16.00
8-9	89.51	165.00	165.00	166.90	165.00	166.90	140.00	47.00
9-10	105.00	170.00	174.00	174.00	169.00	171.50	164.40	120.00
10-11	142.00	175.00	175.00	174.50	169.00	174.00	165.40	140.00
11-12	142.00	175.00	174.50	174.50	168.80	173.00	165.90	140.00
12-13	143.00	166.40	166.00	166.40	165.00	165.00	164.00	130.00
13-14	142.00	167.40	167.90	169.00	165.40	165.90	164.00	140.00
14-15	142.00	167.90	168.40	169.00	165.90	166.90	164.00	120.00
15-16	140.00	166.90	168.40	166.40	165.40	166.40	142.00	120.00
16-17	145.00	173.00	174.50	173.50	166.90	167.90	164.00	140.00
17-18	147.00	175.50	175.00	176.00	172.50	174.00	164.90	143.00
18-19	147.00	168.40	167.90	172.50	168.40	169.80	164.40	142.00
19-20	143.00	165.90	165.90	166.40	165.50	166.50	147.00	140.00
20-21	142.00	164.90	164.90	165.50	164.90	164.90	141.00	120.00
21-22	140.00	164.50	164.40	164.90	165.00	164.90	140.00	105.00
22-23	145.00	164.90	165.00	166.40	166.90	166.90	160.00	124.00
23-24	124.00	143.00	154.00	159.00	165.40	160.00	140.00	110.00

Table E.1 (Continued)

January, 2011								
Hours/Days	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
0-1	132.00	120.00	118.00	125.00	130.00	125.00	142.00	124.00
1-2	100.00	99.51	94.51	99.51	99.51	110.00	120.00	90.00
2-3	47.00	100.00	89.00	94.51	99.51	99.51	115.00	90.00
3-4	16.00	94.51	47.00	89.00	95.00	90.00	90.00	41.00
4-5	16.00	90.00	47.00	48.00	95.00	99.51	49.00	41.00
5-6	54.33	100.00	89.00	99.51	99.51	105.00	90.00	90.00
6-7	90.00	100.00	100.00	100.00	99.51	110.00	48.00	115.00
7-8	105.00	120.00	130.00	105.00	99.51	99.51	48.00	125.00
8-9	164.90	166.90	167.00	165.90	164.50	141.00	102.51	165.50
9-10	165.50	171.00	172.00	167.00	165.90	164.50	120.00	172.00
10-11	166.50	173.50	173.00	166.90	166.40	165.00	142.00	173.00
11-12	166.90	174.00	172.00	167.40	166.90	165.50	145.00	173.50
12-13	164.50	169.30	166.90	165.00	154.00	164.40	142.00	167.40
13-14	164.50	168.50	167.90	165.40	164.00	164.40	142.00	170.00
14-15	164.90	169.00	168.40	165.00	164.40	164.50	142.00	171.00
15-16	164.00	166.00	167.40	164.50	160.00	164.00	120.00	167.40
16-17	164.90	167.00	168.40	164.90	164.40	164.50	144.00	171.00
17-18	165.90	173.00	171.00	166.90	166.50	165.90	164.50	173.50
18-19	164.50	167.40	167.90	164.90	164.50	164.90	155.00	172.50
19-20	150.00	165.00	165.00	164.00	160.00	164.50	147.00	167.40
20-21	147.00	164.50	164.40	147.00	147.00	155.00	145.00	165.90
21-22	147.00	164.50	147.00	150.00	147.00	160.00	143.00	165.90
22-23	143.00	165.00	165.90	144.00	143.00	154.00	145.00	165.00
23-24	141.00	160.00	150.00	140.00	130.00	141.00	130.00	164.00

Table E.1 (Continued)

January, 2011							
Hours/Days	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1	130.00	120.00	125.00	120.00	100.00	110.00	98.51
1-2	110.00	100.00	99.51	85.00	90.00	90.00	78.00
2-3	115.00	100.00	100.00	120.00	16.00	190.00	41.00
3-4	99.51	49.00	85.00	90.00	11.80	90.00	16.00
4-5	85.00	62.21	120.00	47.00	11.90	48.00	12.00
5-6	99.51	90.00	122.00	90.00	11.90	48.00	24.00
6-7	115.00	104.51	116.00	120.00	11.90	48.00	90.00
7-8	139.50	130.00	120.00	105.00	24.00	47.00	90.00
8-9	165.90	165.40	155.00	140.00	115.00	94.99	145.00
9-10	171.00	173.50	168.80	164.50	140.00	110.00	164.50
10-11	173.50	174.80	173.00	165.00	145.00	130.00	165.00
11-12	174.00	174.80	173.00	165.90	150.00	130.00	165.50
12-13	167.40	170.00	165.90	164.00	140.00	140.00	164.40
13-14	169.00	171.50	166.90	164.90	140.00	130.00	164.90
14-15	171.50	173.00	168.00	165.00	140.00	125.00	165.00
15-16	166.90	165.90	165.00	164.90	140.00	130.00	165.00
16-17	171.00	169.00	165.90	164.90	140.00	130.00	164.90
17-18	173.00	172.50	166.00	165.40	150.00	141.00	165.40
18-19	167.40	165.00	164.40	164.50	140.00	130.00	164.40
19-20	165.90	164.40	141.00	141.00	135.00	130.00	140.00
20-21	164.90	140.00	139.50	120.00	120.00	120.00	125.00
21-22	164.50	119.51	119.51	110.00	109.51	120.00	110.00
22-23	164.00	142.00	130.00	130.00	130.00	122.00	122.00
23-24	155.00	130.00	116.00	110.00	109.51	120.00	110.00

Table E.2 Hourly electricity prices for summer time (June, 2011)

June, 2011								
Hours/Days	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
0-1	89.00	88.00	90.00	47.00	125.00	88.00	79.51	79.51
1-2	88.00	35.00	79.00	30.00	88.00	20.00	35.00	43.00
2-3	89.00	9.23	20.00	20.00	88.00	8.82	20.00	79.51
3-4	88.00	12.40	12.17	5.00	88.00	11.30	10.51	20.00
4-5	88.00	11.41	10.66	4.17	35.00	9.89	9.18	20.00
5-6	20.00	12.27	11.41	2.70	5.00	10.62	9.83	9.21
6-7	20.00	9.89	9.21	3.40	4.19	8.58	7.94	7.60
7-8	93.00	81.00	52.00	3.79	5.00	20.00	7.10	40.00
8-9	100.00	140.00	90.00	20.00	75.00	124.00	121.41	110.00
9-10	89.51	100.00	125.00	52.00	75.00	120.00	90.00	130.00
10-11	105.00	122.00	85.00	88.00	115.00	120.00	172.00	148.00
11-12	125.00	122.00	120.00	115.00	110.00	145.00	121.00	152.00
12-13	100.00	99.00	88.00	88.00	88.00	110.00	89.00	90.00
13-14	89.51	121.00	88.00	89.00	125.00	120.00	121.00	120.00
14-15	105.00	95.00	90.00	115.00	88.00	120.00	80.00	151.00
15-16	100.00	110.00	95.00	88.00	42.00	130.00	80.00	140.00
16-17	89.51	95.00	95.00	88.00	115.00	120.00	80.00	120.00
17-18	100.00	122.00	89.00	40.00	40.00	122.00	88.00	120.00
18-19	89.00	99.00	48.00	40.00	40.00	89.00	48.00	110.00
19-20	89.00	99.00	48.00	48.00	39.00	88.00	48.00	110.00
20-21	110.00	110.00	89.00	79.00	88.00	122.00	85.00	120.00
21-22	130.00	110.00	89.00	88.00	115.00	80.00	88.00	120.00
22-23	130.00	110.00	89.00	70.00	125.00	90.00	125.00	110.00
23-24	89.51	95.00	88.00	53.00	42.00	93.00	80.00	85.00

Table E.2 (Continued)

June, 2011								
Hours/Days	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
0-1	88.00	120.00	130.00	122.00	122.00	122.00	47.00	94.00
1-2	75.00	120.00	121.00	20.00	54.00	75.00	47.00	75.00
2-3	69.00	120.00	126.00	5.00	5.55	115.00	5.00	42.59
3-4	69.00	120.00	120.00	4.88	7.79	7.32	6.81	20.00
4-5	20.00	110.00	111.00	3.79	6.80	6.40	5.95	20.00
5-6	8.56	99.00	110.00	1.79	7.30	6.82	6.34	5.95
6-7	6.90	104.00	107.00	1.79	5.87	5.49	5.10	5.00
7-8	72.00	121.00	121.00	3.40	5.32	115.00	93.00	93.00
8-9	123.00	125.00	130.00	119.00	117.00	115.00	120.00	115.00
9-10	140.00	158.00	145.00	39.00	120.00	89.51	120.00	110.00
10-11	153.00	172.00	140.00	60.00	120.00	121.00	120.00	130.00
11-12	171.50	181.57	158.00	125.00	145.00	153.00	120.00	140.00
12-13	150.00	158.00	150.00	39.00	108.00	110.00	121.00	110.00
13-14	151.00	154.00	150.00	60.00	100.00	110.00	121.00	110.00
14-15	153.00	171.00	150.00	60.00	114.00	110.00	124.00	140.00
15-16	152.00	167.50	130.00	119.00	120.00	110.00	124.00	126.00
16-17	143.00	158.00	120.00	39.00	108.00	110.00	110.00	90.00
17-18	130.00	125.00	110.00	11.44	110.00	115.00	110.00	90.00
18-19	130.00	125.00	120.00	11.35	86.00	90.00	110.00	75.00
19-20	120.00	120.00	126.00	11.44	93.00	89.51	105.00	75.00
20-21	120.00	120.00	130.00	110.00	89.51	110.00	89.51	93.00
21-22	120.00	122.00	126.00	110.00	95.00	110.00	93.00	98.00
22-23	120.00	120.00	135.00	110.00	108.00	110.00	110.00	93.00
23-24	120.00	110.00	126.00	125.00	117.00	89.51	93.00	93.00

Table E.2 (Continued)

June, 2011								
Hours/Days	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**
0-1	98.00	98.00	130.00	120.00	115.00	120.00	121.41	120.00
1-2	93.00	48.00	40.00	120.00	110.00	82.51	121.41	115.00
2-3	70.00	39.00	20.00	120.00	130.00	85.00	110.00	85.00
3-4	49.26	20.00	20.00	130.00	110.00	11.34	110.00	85.00
4-5	5.15	20.00	4.85	4.78	130.00	4.85	100.00	65.00
5-6	5.51	3.79	2.69	5.10	20.00	4.42	120.00	48.00
6-7	4.46	3.79	2.67	4.13	20.00	3.79	82.51	48.00
7-8	20.00	48.00	4.90	93.00	110.00	82.51	120.00	110.00
8-9	107.00	130.00	39.00	120.00	132.00	126.50	123.00	122.00
9-10	110.00	90.00	82.51	126.00	150.00	145.00	126.00	126.00
10-11	140.00	90.00	120.00	145.00	150.00	155.00	140.00	140.00
11-12	140.00	140.00	120.00	158.00	171.00	165.00	145.00	137.00
12-13	93.00	135.00	120.00	130.00	133.00	148.00	140.00	100.00
13-14	90.00	110.00	110.00	140.00	150.00	150.00	129.00	103.00
14-15	140.00	110.00	110.00	153.00	160.00	165.00	140.00	130.00
15-16	122.00	110.00	130.00	122.00	167.00	150.00	130.00	130.00
16-17	110.00	110.00	100.00	120.00	171.00	149.00	110.00	133.00
17-18	115.00	100.00	65.00	125.00	150.00	150.00	121.00	122.00
18-19	78.00	110.00	50.00	121.00	133.00	140.00	120.00	120.00
19-20	40.00	110.00	50.00	110.00	110.00	120.00	82.51	120.00
20-21	40.00	110.00	130.00	130.00	115.00	140.00	121.00	110.00
21-22	94.00	115.00	120.00	135.00	122.00	130.00	126.00	110.00
22-23	94.00	110.00	115.00	130.00	130.00	139.50	125.00	121.00
23-24	93.00	110.00	110.00	121.00	121.00	121.00	121.00	120.00

Table E.2 (Continued)

June, 2011						
Hours/Days	**25**	**26**	**27**	**28**	**29**	**30**
0-1	110.00	110.00	125.00	110.00	115.00	127.00
1-2	110.00	110.00	110.00	110.00	115.00	120.00
2-3	110.00	130.00	130.00	110.00	110.00	110.00
3-4	100.00	39.00	6.99	100.00	100.00	82.51
4-5	99.00	3.40	4.99	82.51	82.51	82.51
5-6	85.00	1.79	3.59	85.00	48.00	95.00
6-7	85.00	1.70	3.12	85.00	48.00	95.00
7-8	100.00	1.89	80.00	82.51	82.51	105.00
8-9	120.00	50.00	95.00	110.00	125.00	127.00
9-10	130.00	81.51	120.00	120.00	130.00	127.00
10-11	130.00	110.00	120.00	121.00	140.00	130.00
11-12	130.00	110.00	130.00	121.00	154.00	154.00
12-13	120.00	95.00	120.00	120.00	140.00	135.00
13-14	130.00	120.00	126.00	120.00	145.00	130.00
14-15	121.00	82.51	130.00	140.00	154.00	154.00
15-16	120.00	120.00	130.00	135.00	154.00	154.00
16-17	110.00	121.00	130.00	125.00	150.00	145.00
17-18	120.00	95.00	121.00	120.00	145.00	140.00
18-19	82.51	95.00	120.00	120.00	135.00	127.00
19-20	11.00	115.00	110.00	110.00	120.00	120.00
20-21	120.00	95.00	110.00	110.00	120.00	120.00
21-22	120.00	110.00	126.00	120.00	130.00	127.00
22-23	120.00	110.00	120.00	120.00	127.00	127.00
23-24	110.00	121.00	110.00	110.00	125.00	125.00