TAMPEREEN TEKNILLINEN YLIOPISTO
TAMPERE UNIVERSITY OF TECHNOLOGY

RISTOMATTI LUMMIKKO

DEVELOPMENT OF WATER SYSTEM MODELS WITH STEP-RESPONSE TESTS

Master of Science Thesis

Examiner: Postdoctoral Researcher Henrik Tolvanen
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ABSTRACT

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More than 50 percent of the electricity in the Nordics is produced with hydropower. Hydropower production is flexible and it is capable of responding to the fluctuating electricity demand. Hydropower production is dependent on current hydrological situation and the supply of hydropower is a significant price driver in deregulated Nordic electricity wholesale markets. The variation in electricity price requires the producer to utilize price-dependent production planning, which is important for the producer to succeed, but also to respond to the variations in electricity demand. In addition, hydropower is useful in managing water levels in reservoirs which helps to mitigate flooding.

Optimal hydropower planning requires precise price forecasts and knowledge of the available amount water, but also detailed knowledge of the hydro system limitations is essential. Hydropower production is planned with optimization models which utilize mathematical methods to form the optimal power production combination. All the water systems are unique and the specific characteristics of the water system must be included to the models.

This thesis is focused on a single river system, and especially to a specific part of it. The river system is located in Finland. Experimental knowledge shows that the river section is hard to model with existing data. Thus, step-response tests are planned and implemented in the river system. More precise stream flow routing and reservoir storing capacity are modelled with both historical data and data acquired from step-response tests. The end result of this thesis is a forecasting tool, which strives to model the stream flow routing and water level behavior.

The function of the forecasting model created in this study is to simplify the operation and short-term planning of the river system. The forecasting tool based on step-response tests results is compared to other alternative or prior forecasting tool results. The forecasting tool predicts the water level behavior more precisely than antecedent models. The errors in water levels in production planning are decreased and the modelled water levels are closer to realized when created forecasting tool is used.

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Yli 50 prosenttia Pohjoismaiden sähköntuotannosta tuotetaan vesivoimalla. Vesivoima on hyvin joustava tuotantomuoto, joka pystyy vastaamaan nopeastikin muuttuvaan sähkön kysyntään. Vesivoiman tuotanto on kuitenkin hyvin riippuvainen hydrologisesta tilanteesta ja sen saatavuus on suurin yksittäinen hintaan vaikuttava tekijä sähkön avoimilla tukkumarkkinoilla. Sähkön markkinahinnan vaihtelun vuoksi vesivoiman tuotannonsuunnittelu on hintalähtöistä, jotta tuottajat pystyvät menestymään sähkömarkkinoilla, mutta myös vastaamaan muuttuvaan sähkön kysyntään. Vesivoiman avulla pystytään myös hallitsemaan vesivarastojen käyttäytymistä, joka on yksi avaintekijä esimerkiksi tulvasuojelussa.

Vesivoiman optimaalinen tuotannonsuunnittelu vaatii tarkkaa hintaennustetta ja tietoa käytettävissä olevan veden määrästä, mutta myös vesistön tarkkaa tuntemusta. Tuotannonnuunnittelussa käytetään optimointimalleja, jotka hyödyntävät erilaisia matemaattisia metodeja löytääkseen optimaalisen tuotantosuunnitelman. Käytännössä kaikki vesistöt ovat eriaisia, joten tuotannonsuunnittelussa käytetyt mallit ovat räätälöity vastaamaan vesistön ominaisuuksia.

Tässä diplomityössä on keskitytty mallintamaan erään Suomessa sijaitsevan joen ominaisuuksia, ja erityisesti yhtä sen osaa. Tämä osa joesta on koettu kokemusperäisesti vaikeaksi mallintaa olemassa olevalla datalla. Sen vuoksi vesistöön on suunniteltu ja suoritettu askelvastekokeita, joiden avulla veden kulkeutumista ja varastoaltaan kokoa on mallinnettu tarkemmin. Askelvastekokeista saadun datan avulla on luotu malli, joka pyrkii ennustamaan veden kulkeutumista ja vesipintojen käyttäytymistä edellisiä malleja paremmin.

Tässä työssä tehdyn mallin on tarkoitus helpottaa ja parantaa vesivoiman lyhyen aikavälin suunnittelua ja operointia. Askelvastekokeiden pohjalta luodun ennustemallin tuloksia on verrattu muiden olemassaolevien ennustemallien tuloksiin. Malli ennustaa vesipintojen käyttäytymisen aikaisempia käytössä olleita malleja tarkemmin. Tässä työssä luodun ennustemallin avulla suunnittelussa syntyvät virheet pienenevät ja mallinnetut vesipinnat ovat lähempänä toteutuneita.

PREFACE

This thesis has been written to UPM Energy and for the department of the Chemistry and Bioengineering of Tampere University of Technology.

I would like to thank UPM Energy for giving me this opportunity to work with an interesting subject which develops my understanding of hydropower and also improves my capabilities in my present duties. I would also like to thank M.Sc. (Tech) Matti Vuorinen, who acted as an instructor on UPM Energy's behalf, for giving guidance, advice and instructions throughout the project. Furthermore, I would like to thank Postdoctoral Researcher Henrik Tolvanen for giving me advices and finishing stroke for completing this thesis.

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Tampere, 21 March 2017

Ristomatti Lummikko

CONTENTS

ABBREVIATIONS

GDP	Gross domestic product
Hour Unit (HU)	Hydropower discharge hour unit, 1 HU = 3600 m^3
MAE	Mean absolute error
NASDAQ OMX	Trading house for financial contracts
Nord Pool Spot	Market place for physical electricity trading
ROR	Run-of-the-river type of hydropower
Spot-market	Day-ahead market for physical electricity trading
SSE	Sum of square error
SYKE	Finnish Environment Institute

1. INTRODUCTION

The purpose of this thesis is to study and calibrate hydro reservoir and water flow dynamics in a specific hydropower system with a set of planned test runs. The thesis includes description of hydropower production and how hydropower is planned. The focus is not on deregulated cross-boarding Nordic electricity markets and its mechanics, although most of the hydropower produced in the Nordic countries is sold there. Rather, the focus is on modelling hydropower production and water movements. However, hydropower production and Nordic electricity markets are highly linked to each other and thus the Nordic electricity markets could not be bypassed in this thesis.

1.1 Background

During a year with normal rain and snowfall, hydropower accounts for half of the Nordic countries electricity demand. Hydropower is an efficient and a renewable source of electricity combined with good ability to balance the power system. On a Nordic level hydropower production has a significant effect to the electricity price because it accounts for more than half of the joint Nordic electricity production (Nordic Energy Regulators 2014).

In the Nordic wholesale electricity markets, called Nord Pool Spot, the buyer needs to assess how much energy it will need to meet its load the following day and how much it is willing to pay for this energy, hour by hour. The seller, for example the owner of a hydroelectric power plant, needs to decide how much he can deliver and at what price hour by hour. The price is set where the supply (production) and demand (consumption) meet (NordPool 2016).

As previously mentioned, hydropower producers need to plan their production hour-by-hour for the following day. In a typical fashion for deregulated markets, also hydropower producers strive to maximize their profit with production optimization. An optimal power production requires refined knowledge of water system combined with advanced and excellent inflow and price forecasts (Scharff et al. 2014).

In deregulated Nordic electricity markets, producers need to:

1. Forecast both near and further future price level
2. Forecast inflow levels in order to estimate how much energy is arriving to reservoir in the future
3. Determine at what price they are willing to produce at which hour.

Price uncertainty can be mitigated with price dependent bidding. However in a complex river system that does not solve everything, due to limited flexibility (Scharff et al. 2014).

Producers need to have accurate models to determine production for following day. Deviations between forecasted and realized inflows may cause a situation where the production will differ from what was sold to the day-ahead markets earlier, thus resulting in an imbalance of electricity. In Nordic electricity market members can sell their production also to intraday-markets to minimize their balancing costs.

Both, financial and physical markets are available in Nordic energy markets, but this thesis concentrates only on physical market and results of this thesis are only use in physical trading. Although financial market is important part of energy markets, it is excluded from this thesis.

1.2 Research Objectives

This thesis is a research from the unique river system main channel located in Nordic area. The object is to study the water dynamics in the river system, which has technical and environmental constraints. The aim of this study is to increase the knowledge of the water dynamics and create a forecasting model based on the results. The improved understanding of the water dynamics can be utilized in hydropower production planning and the operation of hydropower plants. The river production is sold to the Nordic electricity markets.

After the river production is sold to the Nordic Day-ahead markets, hydropower producer faces the following questions:

1. How should the sold energy be produced?
2. How the river, and especially the water levels will behave when the sold energy is produced?
    a. Are the environmental limits followed?
    b. What are the costs from deviating from production plan?
3. What opportunities do other marketplaces offer (Figure 5)?
    a. This is also taken account during production plans are made.

With respect to part 2, inflows influence directly hydropower plant’s water level. Typically part of the inflow is run-offs which are uncertain, but change relatively slowly.

1.3 Structure

This thesis contains five chapters, including an introductory chapter and finishing with a conclusion chapter. The thesis starts with a theoretical foundation. Empirical part starts from chapter three and the results are presented in chapter four. This thesis structure is presented as following.

Chapter 2 focuses hydropower specifics and gives a detailed outlook to hydropower modelling. The chapter starts with an introduction of the hydrologic environment and the main drivers of Nordic hydropower production. It continues with the features of hydropower stations and electricity generation including a short description of the most common hydropower plant and turbine types. At the end of chapter 2, fundamental explanation of intrinsic hydropower physical fundaments are presented. In addition, chapter 2 includes an explanation of the many stages of operational hydropower planning.

Chapter 3 starts with a description of the studied water system with its hydropower units and reservoirs. In addition, this chapter contains a presentation of the river systems different hydropower plants co-owning pattern. Chapter 3 continues with a presentation of the planning procedure of this co-owned water system. The chapter also discusses the challenges of planning and operating this particular river system and it continues by presenting the step-response test cases, methods and constraints used to analyze the acquired data. The chapter ends by presenting forecasting tool targets.

The analyzed step-response tests results are presented in Chapter 4. The chapter includes an exhaustive presentation from the parameters used in the forecasting tool and how the results of tests are utilized. In addition, the objectives and benefits of a workable forecasting tool are presented in Chapter 4. The chapter continues with forecasting tool results and comparison to other alternative or prior forecasting tools. The measure goodness of each forecasting tool is presented in this chapter. At the end of this chapter, an example of forecasting tool applicability is shown.

The final chapter of this thesis, Chapter 5, provides conclusions and a review how well the forecasting tool improves the water system handling. A proposal of improvements and prospects of future research topics are also presented in this chapter.

2. HYDROPOWER PRODUCTION MODELLING

2.1 Hydrologic environment

Hydrologic cycle, also known as the water cycle, describes water movement on Earth. In long time perspective, the amount of the water on the Earth remains constant but water movement and phase change is a continuous process. The movement of water through the cycle is erratic, both in time and over the area (Linsley 1982).

In the hydrologic cycle, water is stored in different reservoirs like oceans or lakes in liquid form. This cycle, shown in Figure 1, is visualized to begin with evaporation from the reservoirs to the atmosphere caused by the sun and wind. The resulting vapor condensates and eventually forms clouds which are transported by moving air masses. The transported clouds release water through precipitation and water falling upon land is dispersed in several ways. Some water is temporarily retained in the soil near where it falls and is returned to the atmosphere by evaporation and transpiration by plants. A portion of the water might be collected back to the reservoir and other portion is absorbed through the soil surface. This absorbed water in the soil surface moves step by step towards lower elevations by the influence of gravity, until it finally merges into a reservoir. Furthermore, water can also take alternative routes. For example, the water reservoir can exist as snow or ice. When the snow or ice melts, it runoffs to an underside river and the river flow carries the water to the lake or ocean (Linsley 1982).

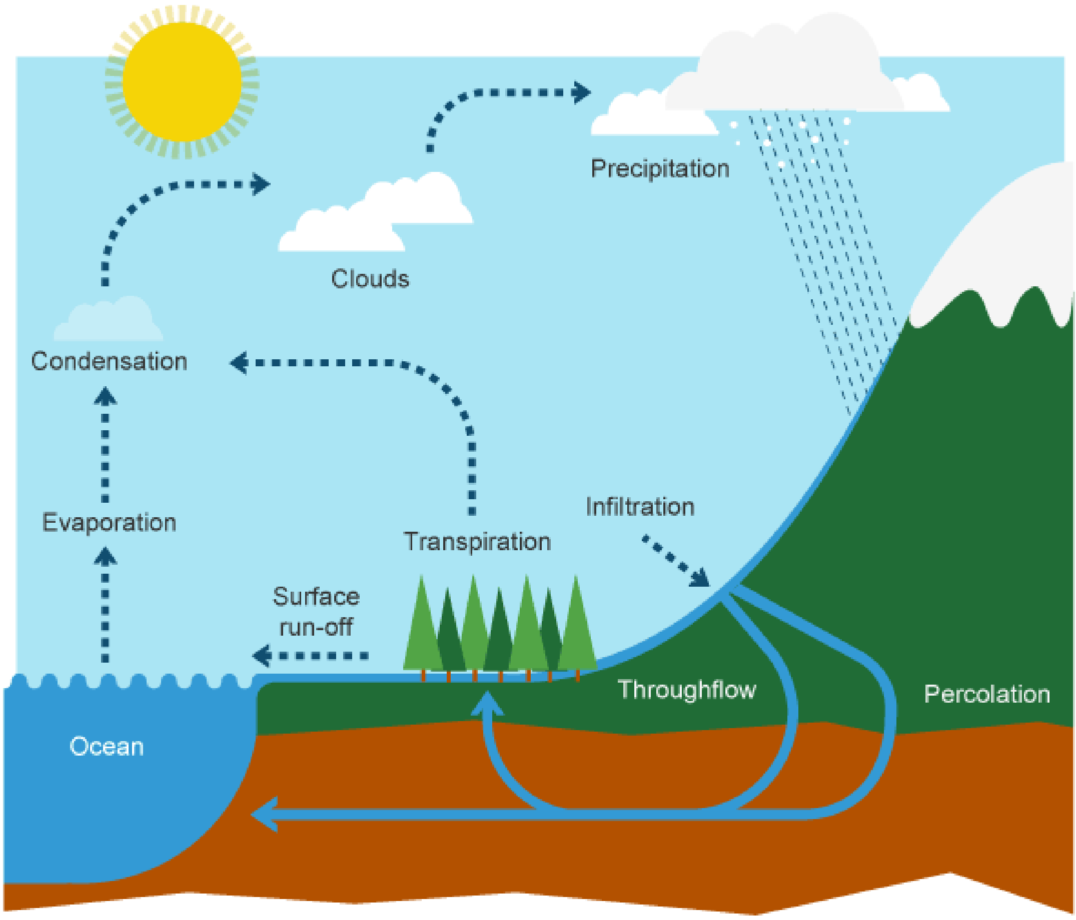

***Figure 1:** Hydrological cycle (BBC 2016).*

Hydropower is based on water cycle and the water cycle is an endless cycle, thus hydropower can be considered as a renewable energy source.

2.1.1 Water inflow

Hydrology forecasts, including inflow forecasts, are used for different purposes, for example irrigation, city water supply, flood warnings, hydroelectric power, and converging water resources. Several models with different methods are used for forecasting the behavior of a hydrologic system. In Finland, watershed simulation and forecasting is operated by Finnish Environment Institute (SYKE). SYKE utilizes meteorology, hydrology and aerial data in their watershed simulation and forecasting system (Linsley 1982; Finnish Environmental Institute 2016).

In hydropower planning, the inflow can be considered as the most important input. The amount of energy and possibility to allocate it depend on available inflow. The inflows can vary remarkably during the year and between different years and predicting the inflow is difficult, not only yearly basis but also for few weeks or days ahead. When water inflow is twice the normal basis, it is called “wet” year and when it is half of normal, it is called “dry year”. The water inflow strongly depends on weather, thus the inflow can be predicted relatively accurately as long as weather forecasts have been accurate. In some case, during a long dry and warm season, the inflow can be temporarily negative, which means that amount of water absorbed to soil and evaporation are greater than inflow to the reservoir (Antila 1997; Sorooshian et al. 2008)

Water can exist in different forms, which are dependable on the season. In autumn, most of the inflow comes as rain in the Nordics. During winter, majority of the inflow comes down as snow, but this type of inflow is at least partially solid and therefore not suitable directly for hydropower production until the snow melts. In spring, when temperature rises and the snow starts to melt, the inflow rises heavily and it causes spring flooding. The spring flood is a high peak in inflows and it can’t be avoided.

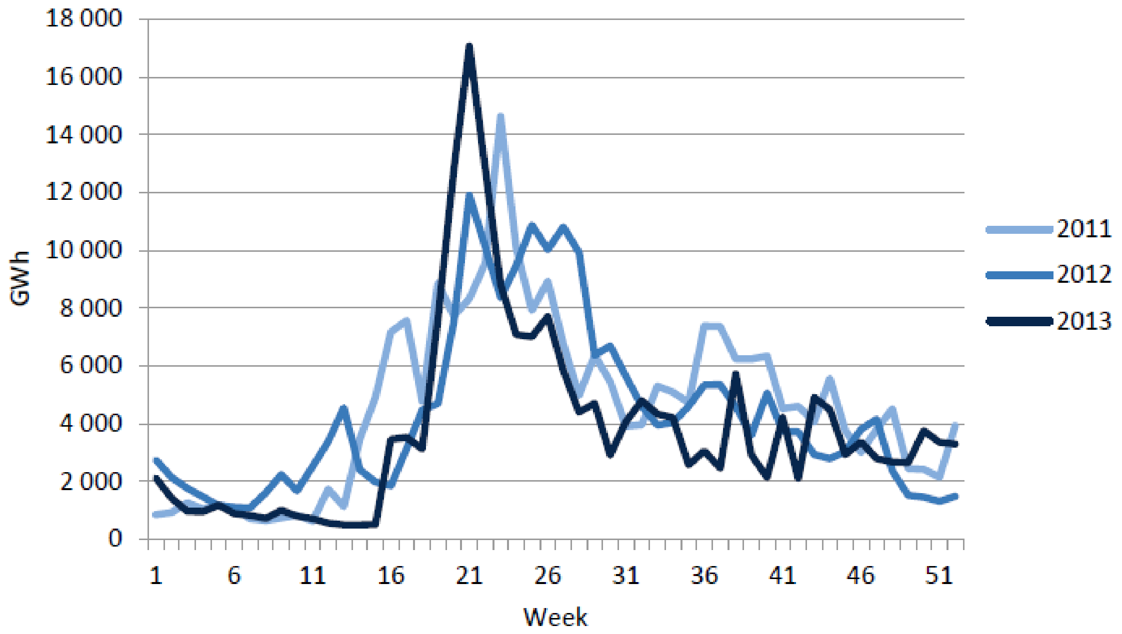

Figure 2: *Effective inflows to the Nordic water reservoirs 2011 - 2013 (Nordic Energy Regulators 2014).*

As seen from Figure 2, the amount of inflow is nonidentical between different years and during the year. The inflow peak takes a place during the spring and the timing of peak varies between different years. The timing and the size of the inflow peak is hard to predict and the predicting mostly depends on weather forecasts, as previously mentioned. The inflow is usually lowest during the winter.

Nordic water reservoirs are usually drained during winter because of low inflow. Low water levels after winter are a result from natural behavior of water systems and hydropower regulating: hydropower producers are preparing to spring floods and making room for strong inflow. The combined Nordic reservoir levels are presented in Figure 3.

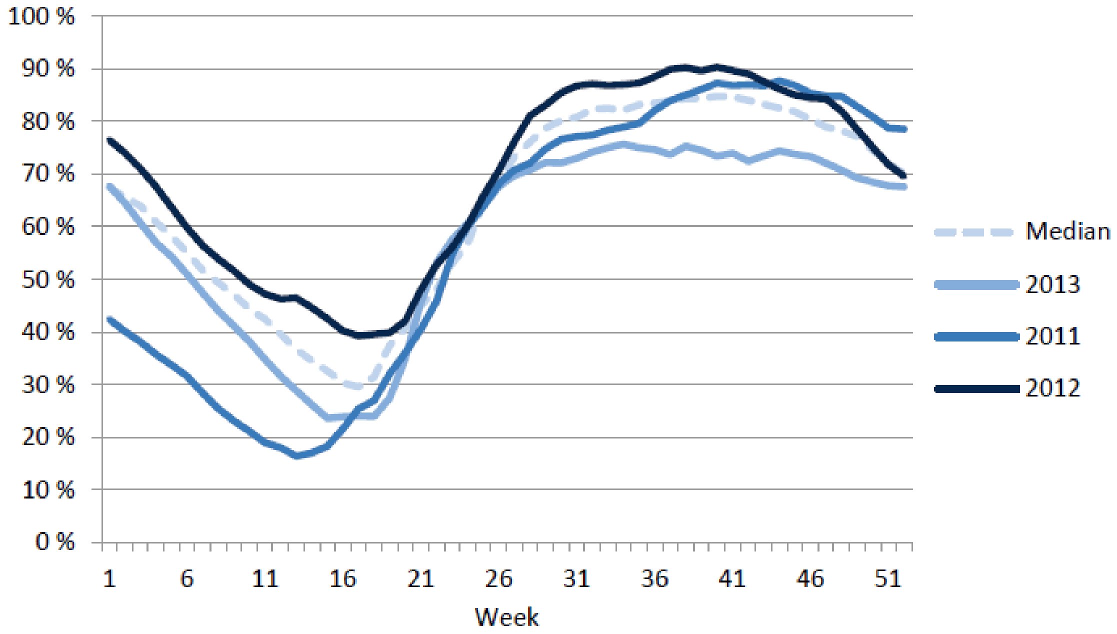

Figure 3: *Reservoir levels in the Nordic region 2010 - 2013 (Nordic Energy Regulators 2014).*

As seen from Figure 3, reservoirs are usually drained during late autumn and winter until the spring flood fill reservoirs again. The spilling during the heavy inflow peak is minimized in this way (Bye 2008).

During high inflow, there is a significant risk of damaging flood. The flood dangerousness can be minimized with water management which can be carried out with hydropower plants. Hydropower can be used for purposeful regulation of reservoir levels, which aims to provide space to reservoirs before high inflows. Hence, the water result from high inflow can be stored to reservoir. In addition of flood mitigation, it is also sensible to operate in this way because the spillage past the turbines is avoided or minimized. Proper water management of reservoirs handling is one of the main responsibilities of hydropower companies (Mill et al. 2010).

2.2 Hydropower stations

Hydropower is a unique way to generate energy compared to other forms. It is carbon dioxide free and renewable energy production form, but it also provides others services, such as flood control, irrigation opportunities, and water storage. At the same time, hydropower can provide lowest-cost electricity compared to any other source. Hydropower stations often have significant flexibility: it can be designed to meet base load demands or large share of peak electricity demand. Hydropower is also a good fit for power system balancing because production can be rapidly ramped with low costs. Electricity consumption in Nordics is highly dependent of average temperatures of the year and GDP. Because hydropower is suitable for power system regulation and water can be stored, it can respond to fluctuating demand. Therefore, there is less volatility in Nordic electricity price structure, than for example in Germany (IRENA 2015; Nordic Energy Regulators 2014).

An important property of hydropower is the ratio of nominal discharge and the average inflow to the plant. The ratio and the size of the water reservoir above the plant describes the ability to produce regulating power. If the nominal discharge of the hydropower plant is as same as average inflow to reservoir, hydropower plant has no controlling capacity and then the unit is operating like a base load plant. If the nominal discharge is substantially greater than average inflow, the freedom of controlling production increases. The amount of inflow is usually dependent on the season and it can vary a lot between the years and months. As a summary, the flexibility of hydropower unit depend on the water flow in the river system, the volume of reservoir, seasonal variations of inflow, and both technical and environmental constraints. Environmental constraints are defined in a power plant’s permit and can also have seasonal variations (Antila 1997).

Although hydropower is a low-cost renewable energy source, it affects negatively to local environment. The environmental impacts mostly come from dam construction and reservoir regulation. For example, the construction of a dam has an impact on the movement of fish which affects fish stocks and hinders fishing. The negative impact to fish has been reduced by restocking the fish and constructing alternative route to fish’s movement such as fish ladders. Hydropower projects can encounter social resistance because of the negative impacts to water available, ecosystem and the environment. In some cases, the project may lead to relocation of population. Large hydropower projects requirement larger dams and electricity transmission grid etc., and thus those are raising more environmental and social opposition than small scale hydropower (IEA-ETSAP 2016; IRENA 2015; Energiateollisuus ry 2016).

2.2.1 Hydropower plant types

Hydropower can be classified by their characteristic such as ability to store water or size. There are two basic configurations in hydropower plants: dams with reservoir and run-of-river plants, with no reservoir. World’s major hydroelectric plant types are either run-of-the-river (ROR) or hydropower plants with a dam, also referred to as conventional hydropower. The dam scheme can be subdivided into small dams with day-night regulation, large dams with seasonal storage, and pumped storage reversible plants for energy storage and day-and-night regulating according to variety of electricity demand. The hydropower units over 10 MW are commonly defined as large hydropower, units in the 1-10 MW range are called small-scale hydropower, and under 1 MW units are called micro-scale hydropower. Small scale hydropower is normally designed to use in ROR units (IEA-ETSATP 2016; IRENA 2015)

In a ROR-type of hydropower the storage capacity, in the practical sense, does not exist or it is very restricted. ROR units are environment-friendly option because they do not significantly interfere to the natural flow of the river. Thus, the impact of ROR units on the local surrounding is more limited compared to conventional hydropower. The electricity generation of ROR units is strongly connected to water flow which can vary remarkably during the year and between years. ROR hydropower unit without reservoir is unsuitable for regulating purposes and it is mostly producing only base power. In ROR hydropower, different units located in same riverbed are strongly hydraulic coupled to each other and the inflow to successive plant is the delayed discharge of the previous plant combined with possible run-offs and therefore the operation of the single plant cannot be planned separately. The strength of hydraulic coupling is relatively to size of the unit reservoir and the delay of discharge depends on the distance from a previous plant (Vilkko 1999; IEA-ETSATP 2016; IRENA 2015).

Conventional hydropower works similarly to run-of-river hydropower plants. However, conventional hydropower utilizes reservoir which can be natural or a man-made artificial reservoir, where water can be stored. The storing capacity can be small or large, depending on the type and location of reservoir, environmental issues, characteristics of the site, and the economics of the dam construction. Unlikely ROR units, large hydropower plants can encounter social opposition because of their impact on water availability, ecosystems, and the environment. Conventional hydropower plants are specifically used to adjust the production to meet the demand. Production is flexible and the turbines can be ramped up and down rapidly, therefore it is suitable for keeping up the stability of the electricity system. In countries, where hydropower is available in large scale, the balancing requirement of electricity system is mainly satisfied by hydropower (IEA-ETSAP 2016; IRENA 2013; Kemijoki Oy 2016).

Pumped-storage hydropower plants operate in a similar way as conventional hydropower, with the notable difference that pumped-storage unit can move the water from lower elevation reservoir back to higher elevation reservoir. Modern pumped-storage units do not need a separate pumping unit because the turbine can operate opposite direction as well. In other words, pumped-storage hydropower can operate as a load unit as well as a production unit. At times of low electricity demand, such as during night-times and weekends, water is pumped to upper reservoir and electricity is produced during periods with higher demand. Pumped-storage hydropower plant suits extremely well to electricity grid balance management: excess energy from electricity grid is used to water pumping, while shortfall of energy in grid can be fulfilled with releasing water from upper reservoir. The energy conversion efficiency in modern pumped-storage hydropower units with state-of-the-art technology is over 80 %. Pumped-storage hydropower plants are nowadays enjoying special attention as they are at present most competitive option for large-scale storage to be used in combination with variable renewables (IEA-ETSAP 2016; National Hydropower Association Hydro Technology 2016).

2.2.2 Electricity generation

All hydroelectric power units produce electricity in the same way. Hydroelectric power station utilizes the height difference between two water surfaces to produce power. The potential energy of a higher water elevation is converted to kinetic energy when water is discharged through intake, into the penstock, and past the turbine to a lower elevation. The energy is absorbed by the turbine which in turn transfers its rotation energy to a generator. The mechanical energy is converted into useable electric energy by the generator. Generator then feeds the produced electric energy into the electric grid. The amount of power that is generated can be presented as a function of discharge, height of the head of the plant, and the combined efficiency of turbine and the generator (Crona 2012; Olsson 2005).

A plant reservoir exists above a hydropower plant. A dam blocks the water that hydropower utilizes, storing it in to the reservoir and prevents the water running past the turbines. The inflow into plant can be temporarily stored in the reservoir. Also, the stored water can be discharged from the reservoir. The size of the reservoir demarks how much water and how long it can be stored in a reservoir (Crona 2012; Vilkko 1999).

Some of the water cannot be discharged through the turbines in certain situations. For example if the turbines are not in operation or the inflow is high and there is no regulating volume available in the reservoir. For these situations there must exist an alternative way to discharge the water. The fundamental part of a hydropower station is a spillway which is used to let water run past the station in certain circumstances. The process is called spilling and with spilling producers can control water level in the same way as with discharge through the turbines. Spillage is typically needed during the spring thaw when the inflow is strong, or if for one reason or another, the power plant is in maintenance. Flood particularly occurs in spring or in autumn in Nordic area. Spilling the water wastes the potential energy of the water and therefore hydropower companies strive to avoid spillage unless it is absolutely necessary. Usually, maintenances can be planned well beforehand and are scheduled to a period when the inflow is low enough to avoid spillage (Crona 2012).

As previously mentioned, the amount of generated power is a function of discharge, head, and combined efficiency of the turbine and generator. In reality, the relation between discharge and power production is non-linear. The river usually resists the flow caused by discharge or spillage, and if the flow through the dam is high, tail water tends to increase and head level tends to decrease. This effectively decreases the production rate of hydropower plant (Vilkko 1999).

The potential energy of the water can be presented as following function:

$$U = mgh \tag{1}$$

Where U (J) is an amount of potential energy to stored water, m (kg) is a mass of water, g ($^{m}/_{s^2}$) is a local gravitation constant of Earth, and h (m) is a head of a plant. Mass can be shown as multiplication between density ρ ($^{kg}/_{m^3}$) and volume V (m^3):

$$m = \rho V \tag{2}$$

The power P ($^{J}/_{s}$) that is theoretical converted from the water is a division between energy (J) and time (s):

$$P_{theoretical} = \frac{U}{t} \tag{3}$$

By linking equation (1), (2) and (3) to each other, the theoretical amount of power that can be converted from water is:

$$P_{theoretical} = \frac{\rho V g h}{t} \tag{4}$$

The discharge Q ($^{m^3}/_{s}$) can be shown as division between volume V (m^3) and time t (s). The head of the hydropower plant is subtraction between intake level and tail water level. By combining these relations, the theoretical power that hydropower plant can convert from the water is:

$$P_{theoretical} = \rho Q g (h_{intake} - h_{tail\ water}) \tag{5}$$

In reality, all the potential energy water contains cannot be directly converted to electric energy. This means that power P must be scaled with efficiency factor η:

$$P_{actual} = \eta \rho Q g (h_{intake} - h_{tail\ water}) \tag{6}$$

The losses come from friction in waterways, the efficiency of turbines, amounting to 12-14 % loss from the potential energy (Finn R. Førsund 2007).

2.2.3 Hydropower turbine types

There are three commonly used types of hydro turbine: Pelton, Francis, and Kaplan. Pelton turbine is particularly suited for high head and low discharge applications. It is the only impulse type hydraulic turbine in common use today. In Pelton turbines the rotor consists of a circular disk with a number of bucket-blades. Bucket-blades are fed by water jet from nozzles in such a way that each nozzle directs its jet along a tangent to the circle through the centers of the bucket-blades. The regulating method of Pelton turbine is arranged by needle valve and deflector plate. The optimum efficiency of Pelton turbine is approximately 90 % (Dixon et al. 2014).

A benefit of a Francis turbine is its wide operation conditions: it is suitable for high and low head applications. Francis turbine consists of three main parts: spiral casing, guide vanes, and runner blades. Water enters via spiral casing called a volute or scroll that surrounds the runner. From the volute, the water flow enters a ring of stationary guide vanes, which direct it to the runner blades at the most appropriate angle. The Francis turbine is controlled by adjusting the angle of guide vanes. The maximum efficiency of a Francis turbine is approximately 95 % (Dixon et al. 2014).

The Kaplan turbine is suitable for low head and relatively high water flow applications. The benefit of Kaplan turbine is good efficiency on partial loads. In Kaplan turbine, the flow enters from a volute into inlet guide vanes, which impart a degree of swirl to the flow determined by the needs of the runner. The control of Kaplan turbine is done by adjusting the guide vanes and the angle of rotor blades. The maximum efficiency of Kaplan turbines is approximately 94 % (Dixon et al. 2014).

The Finnish terrain is flat and therefore the head of a plant is usually low. Thus Francis and especially Kaplan are mostly used turbine types in Finland. Kaplan turbine efficiency curve is very flat and therefore it is suitable for peaking and cycling production. Suggestive efficiency curves are shown in Figure 4 (Antila 1997).

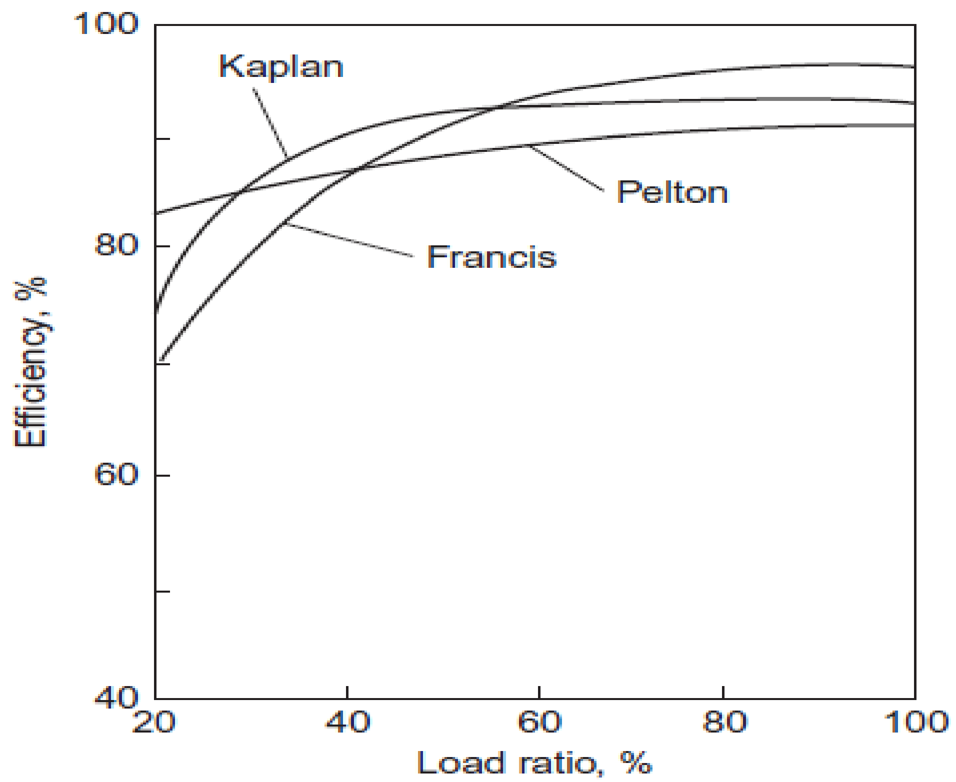

Figure 4: *Efficiency curves of three hydropower turbines mainly in use (Dixon et al. 2014).*

2.3 Hydropower production planning

Hydropower production can be offset when it is mostly needed and energy can be stored to reservoir if available. Amount of water available has seasonal and yearly variations. If a reservoir exists, there is a possibility to smoothen seasonal and yearly variations in available water. The main object of hydropower production planning is scheduling the offset during times when production is most profitable. Hydropower production planning can be generally divided into three different planning periods which are separated according to time frame.

Long-term hydropower planning is longest planning period of these three planning periods and it contains both expansion and seasonal planning. Seasonal planning period is from one year to five years ahead, whereas expansion planning time period is in the range of tens of years. Compared to seasonal planning, expansion planning has to take into account many other prevailing uncertainties like changes in power production system, climate change or political decisions. Long-term seasonal planning is mainly based on forecasts and different scenarios of power demand and water inflow to the system. Both power demand and water inflow are difficult to predict and this means that models used in long-term planning should be capable of handling uncertainties. The nature of long-term planning depends from controllability of hydropower unit. If there is no reservoir above the plant (e.g. ROR-units), expansion and seasonal long-term planning highly depends only from the inflow forecasts. When the hydropower plant is completely controllable and the water can be discharged freely within constraints of the seasonal reservoir and the only short-term constraint is the maximum discharge of the unit (Antila 1997; Dixon et al. 2014).

Mid-term planning horizon is from two weeks to one year ahead with weekly resolution. The purpose of the mid-term planning is to give endpoint to storage description in the form of incremental water values for reservoir. The target reservoir content at the end of planning period is given by long-term planning model. The reservoir content at the beginning of the planning period is known. The input of mid-term model is given from inflow, network flow programming, and market price forecasts. Such as long-term modelling, inputs for mid-term modelling are also uncertain, thus mid-term models should also be stochastic (Dixon et al. 2014).

Short-term planning covers time frame from days to few weeks ahead. The objective of short-term planning is to maximize the value of hydro in the selected time scale. The input to short-term hydro planning comes from mid-term model, inflow and market price forecasts. In short-term planning inflow and market price can be predicted with sufficient accuracy. Predicted values can be generally assumed to be correct and when the forecasts change, the model can be recalculated. Therefore, short-term hydropower planning can be treated as a deterministic problem. Constraints of short-term modelling are maximum discharge of hydropower unit and reservoir levels. Short-term modelling is more “precise” than mid-term or long-term modelling: for example, short-term model should include turbine efficiency curves and other features of hydropower unit. Short-term planning provides also intraday, manual frequency regulation, and automatic frequency regulation markets (Crona 2012; Antila 1997; Dixon et al. 2014).

The nature of hydropower production planning highly depends from the controllability of hydropower unit. The physical constraints, like maximum discharge of unit, reservoir minimum and maximum levels or other constraints defined in a Governance rule must be taken into account. Hydropower production planning is a continuous process and the models are recalculated continuously.

In addition to physical production planning, similarly to any other production form, also hydropower companies are hedging their production against price risks in day-ahead or intraday markets with financial contracts. The financial contracts are made in different market place. The trading in financial markets is done anonymously through the clearing house NASDAQ OMX. Financial contracts do not fulfil to physical delivery, only cash settlement. Thus, financial market trading is not bounded by technical constraints nor does it have direct influence to power system’s physical situation. In practice, the above-mentioned means that financial markets have more players than physical markets, mainly because participation to financial markets does not obligate any links to physical power markets. The production hedging is part of long-term and medium-term planning (Scharff et al. 2014).

The operational decision making process of hydropower companies in Nordic electricity markets are combination of participation to all market places by utilizing trading capacity and energy. Description of operational decision-making process is presented in Figure 5 (Scharff et al. 2014).

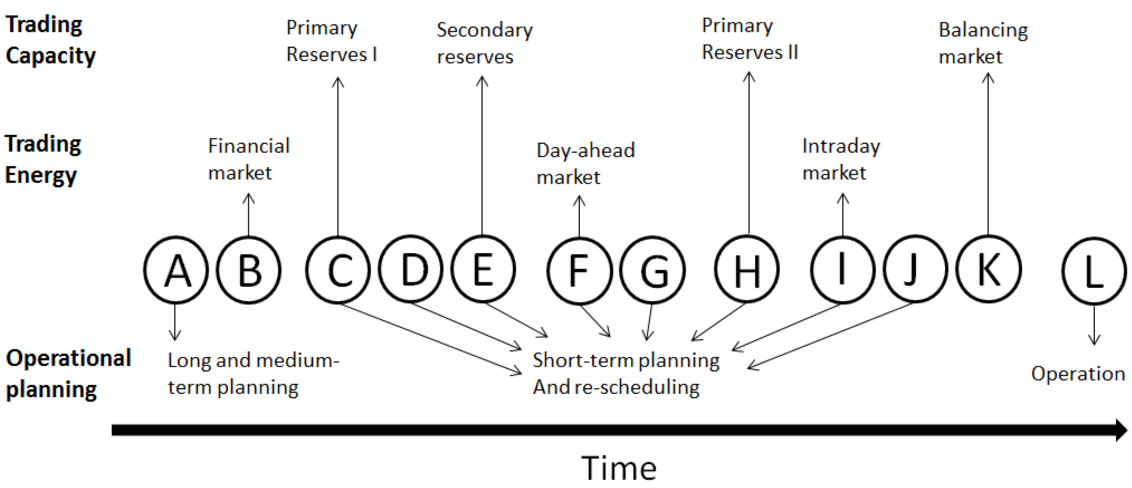

Figure 5: *Decision making points of hydropower companies in Nordic electricity market.*

A. Long-term and medium-term production planning
B. Trading on financial markets
C. Tendering process for yearly primary reserves
D. Short-term production planning, weeks ahead
E. First tendering process for secondary reserves
F. Bidding on day-ahead market
G. Short-term production planning, days ahead
H. Second tendering process for hourly primary reserves
I. Intraday markets
J. Self-balancing
K. Balancing market
L. Real-time operation.

Hydropower production planning contains many different steps in different marketplaces, as the list above shows. The results of this thesis can be utilized in short-term planning and re-scheduling.

2.3.1 Objectives

Flexibility of output, low variable costs combined with the ability to storage water, are the defining characteristics of hydro planning. For hydropower producer, water can be considered to be a scarce resource and thus hydropower companies strive to maximize the value of usable water by allocating generation to hours which receive the highest price for it. The future price of electricity is exposed to many uncertainties, like availability of water, temperature-driven demand, and fuel prices of other power generation sources. Hydropower producers generally, especially in short-term planning, encounter the following question in planning: should the production be sold today, or save it for tomorrow with the expectation from higher price for produced energy. In addition, hydropower can participate in different market places because of its specific characteristics, which must be take account in short-term planning. Thus, the marginal cost of hydro production comes from the price expectations, or water values, rather than nominal variable cost of production (Kauppi 2009; Olsson 2005).

The optimal generation scheduling is based on inflow and market price forecasts, which both have seasonal variations. These form the hydro scheduling problem. Consumption, reservoir levels, and inflows are significant price drivers in hydro-dominant Nordic electricity markets. During time when inflow predictions are high and reservoir is full, it is more profitable to offset the production with low electricity price than save the water and ran to the situation, where is necessary to spill water through the turbines (Vilkko 1999; Bye 2008).

The near future electricity prices have a significant role in short-term planning while the main question is, how the generation given by mid-term planning is scheduled to different hours. Thus, the near future price forecasts, combined with near future inflow predictions, are main drivers in short-term planning. In short-term planning, which is mainly on focus in thesis, the real effects of generation should be modelled in production planning. This requires that behavior of reservoirs, intake and tail water levels, turbine efficiencies and scheduling, and other characteristics of water system, for instance delays in the water system, can be modelled in the right way. The modelling of real effects of generation is presented in sections 2.3.2 – 2.3.5 (Vilkko 1999; Bye 2008).

2.3.2 Head effect

The power generation of the hydropower plant is not a linear function of the rate of discharge through the turbine. The power generation function increases until a peak is reached. After the peak, power generation can even decrease mainly because the head of the hydropower plant is reduced at high discharge levels and the efficiency of the turbine deteriorates, which cannot be avoided unless the another generator is started. The decrease in head is due to decreasing water level above the dam and increasing tail water level. The variation of water specific energy with a function of discharge through the hydropower plant is presented in Figure 6.

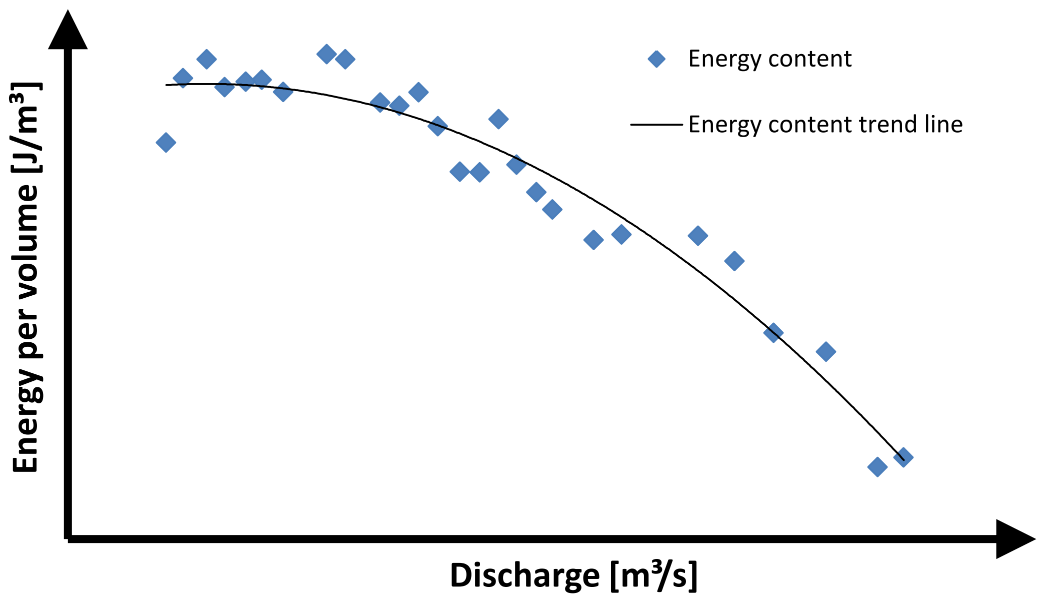

Figure 6: *The variation of water specific energy with function of discharge through the hydropower unit.*

It is advisable to run the generators on the optimal production zone and high intake water level because the specific energy of water is at its highest and more energy from the same amount of water is received. Commonly hydropower units contain more than one turbine. When there is more than one turbine on the site, the production curve of the hydropower plants is a combined production curve of turbines. An example of a combined production curve is shown in Figure 7.

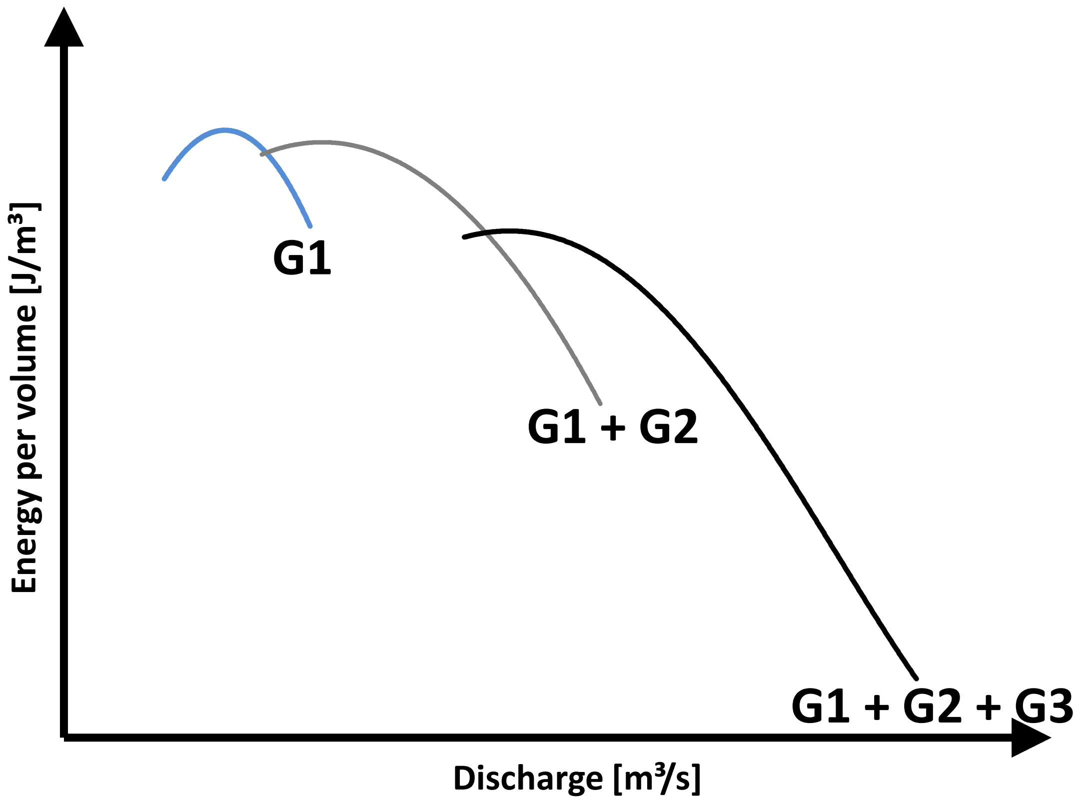

Figure 7: *Three generators combined production curve shown as a function of discharge.*

2.3.3 Tail water

The part of the river right after hydropower plant is called tail water. The river usually resists the flow caused by discharge through the turbines and flood gates, and therefore the tail water tends to increase when the discharge increases. Practically, higher tail water level will reduce the head of the hydropower plant and make the efficiency of the turbine worse (Vilkko 1999). A conventional method to model tail water behavior is a function of a total discharge of hydropower plant. In Figure 8, there is shown typical linearization of tail water level in according to discharge.

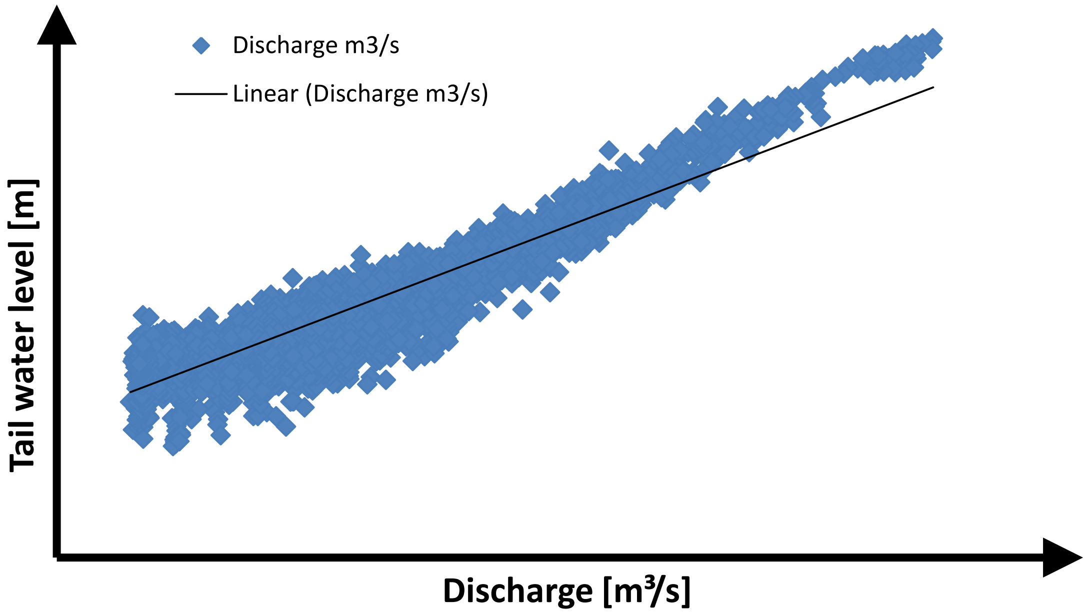

Figure 8: *Linearized tail water level.*

In water systems, where is many hydropower plants located one after another, the tail water could be located in subsequent hydropower plants reservoir. In these circumstances, the downstream reservoir level influence to tail water level of previous plant. In Figure 9 there is an example, how subsequent reservoir level influence to previous plant tail water. Data is from hydropower plants which are located in river 20 km away from each other.

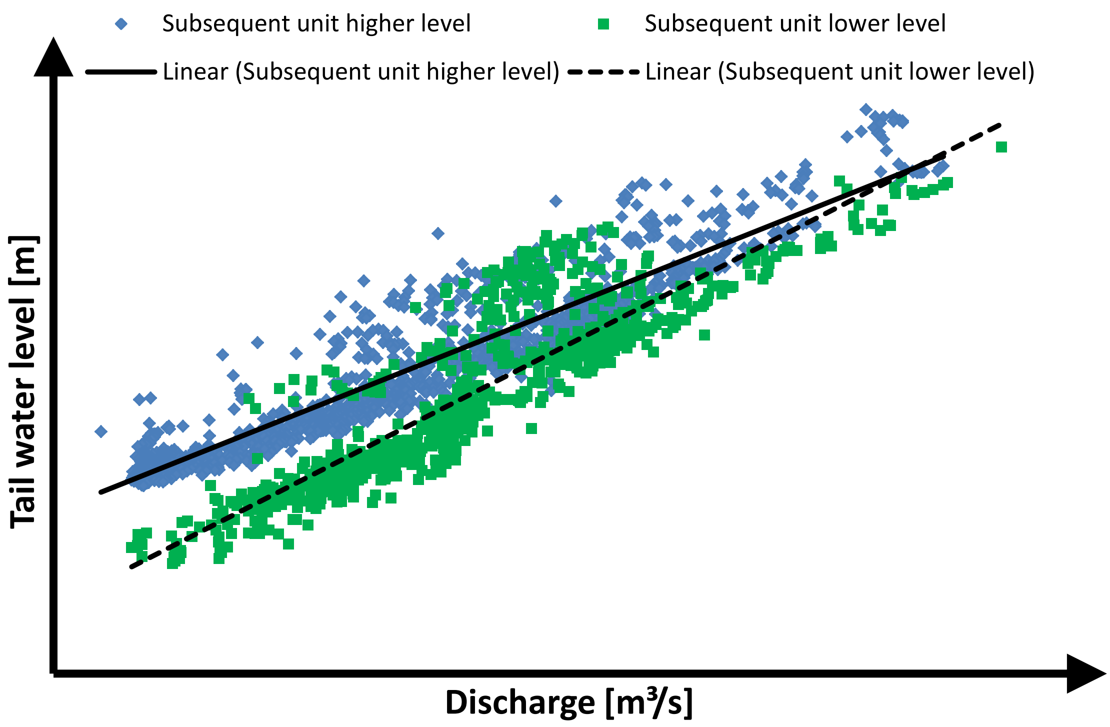

Figure 9: *Reservoir level located in downstream influence to previous hydropower plants tail water.*

In practice, the change in tail water is a dynamic process, where tail water level increases or decreases with due time after changes in discharge. In reality, tail water level response is slower than a simple linear regression model implies. If the model does not take this into account, the reduction in calculated energy affected by tail water is too low or too high. The reduction in energy caused by the tail water increase can be handled for example with exponential smoothing, which allows one to calculate a slower changing to be used with the linearized tail water curve.

The standard exponential smoothing can be represented as follow:

$$S_f(t) = \alpha \cdot x(t) + (1 - \alpha) \cdot S_f(t-1) \tag{7}$$

Where smoothing parameter α $(-)$ is restricted as following:

$$0 < \alpha < 1 \tag{8}$$

In equation (7), $S_f(t)$ is a weighted average from previous smoothed value $S_f(t-1)$ and the most recent observation $x(t)$. The smoothing parameter can be selected as following:

$$\alpha = \frac{2}{1 + l} \tag{9}$$

Where l is a number representing the period of smoothed values (Gardner 2006; Räsänen 2014).

The smoothed tail water behavior is caused by ramped increase of discharge is presented in Figure 10.

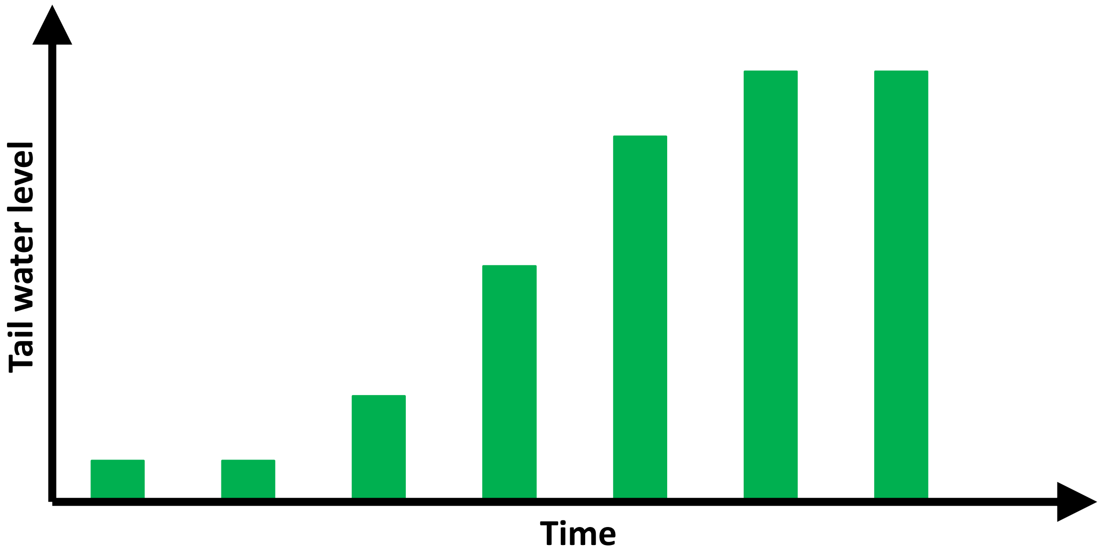

Figure 10: *Smoothed tail water behavior caused by hydropower plants ramped increase of discharge.*

2.3.4 Delays

For hydropower stations located in same riverbed, the release from upstream reservoir ends to the latter station as inflow. In this situation hydropower units located in same riverbed are said to be hydraulically coupled to each other. This type of hydropower is very common and it is known as run-of-the-river hydropower (ROR).

As previously mentioned, ROR units are located in same riverbed as a chain and units are hydraulically coupled to each other, thus the generation of subsequent unit depends strongly from upstream hydropower unit operation. If there is no ability to store water above the hydropower unit, in other words reservoir above the plant does not exists or it is very small, hydropower unit must discharge the same amount of water as the inflow from the upstream hydropower unit. This kind of coupling reduces maneuverability of the hydropower system.

Hydropower planning requires understanding of the water movement. This stands out especially in units without a reservoir, where the operator must know how much water is available and when it is available. The time delay that water travel within a channel from upstream unit to subsequent unit must be known to keep subsequent unit hydro balance on acceptable level and make acceptable production plans. Well-known water movement contributes to better hydropower scheduling and helps avoid unfavorable production imbalance.

The water flow in system could be simulated with Saint-Venant equations which are based on Navier-Stokes equations. There are computer programs developed to model water flow which are partly based on Saint-Venant equations. One of these programs is HEC-RAS which performs one-dimensional steady flow and one and two-dimensional unsteady flow calculations. These programs and Saint-Venant equations require dimensional model of the water system which are not readily available without extensive measurements and thus out of scope of this thesis (Sharkey 2014).

In some cases, the water delay approximation as a constant can be an acceptable level. Even though such approximation can be acceptable, there are also regions where such water delay cannot be modeled as a constant value, because of the stream flow routing behavior between upstream and downstream plants. The water movement from plant j to i with constant delay is presented in Figure 11.

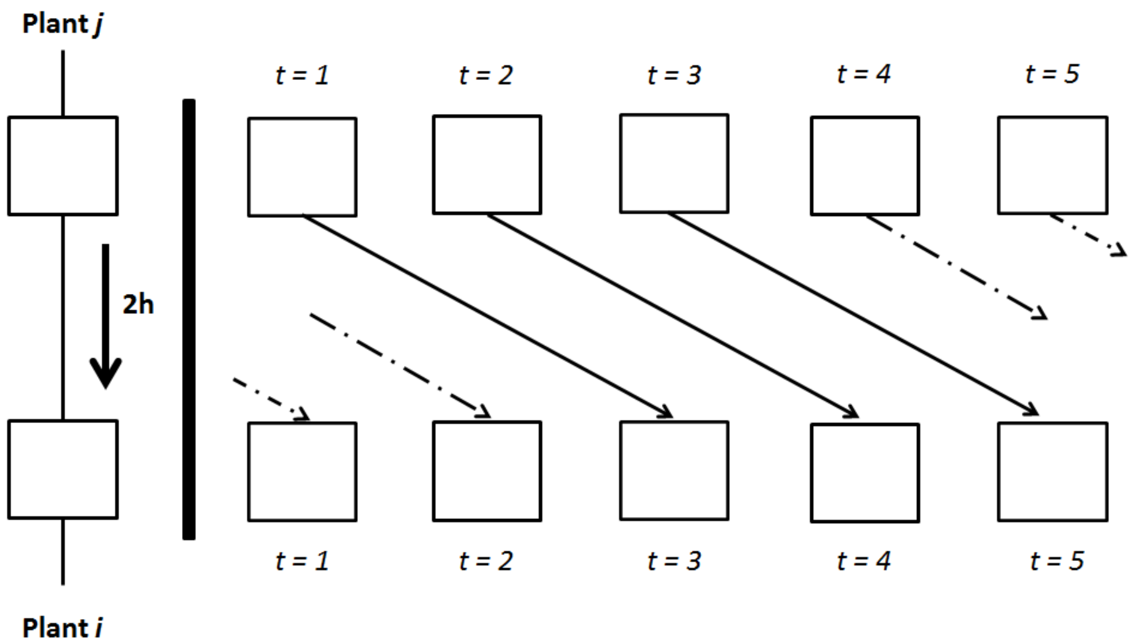

Figure 11: *Constant water delay between hydropower plants. The arrows present portions of moving water in river.*

The biggest assumption with a constant water delay is that the whole amount of water will reach at subsequent hydropower unit in time step, which is not true in practice due to the stream flow routing. In this situation, the mathematical formulation of water balance is represented as follow:

$$V_i^t + (Q_i^t + S_i^t) - \sum_{j \in M_i} (Q_j^{t-\tau_{ij}} + S_j^{t-\tau_{ij}}) = V_i^{t-1} + I_i^t \tag{10}$$

, where

V_i^t is storage of hydro plant *i* at the end of time step t,

Q_i^t is turbine outflow of hydro plant *i* at the time step *t*,

S_i^t is spillage of hydro plant *i* at the time step *t*,

M_i is set of hydropower plants upstream plant *i*,

τ_{ij} is water delay time in hours between plants *j* (upstream) and plant *i* (downstream),

and I_i^t is natural inflow to hydro plant *i* at time step *t* (Souza 2012).

As mentioned before, a constant water delay time is not useful in certain regions because water moves as a wave above the river bed in an open channel and thus all the influence of the increased inflow upstream does not effect to downstream power plant in one hour, especially when there is long distance between hydropower units. In this case, different portions of water released by upstream hydropower unit at time step t_p reach subsequent hydropower unit in different hours, ranging from τ_{ij}^{min} to τ_{ij}^{max}. This situation, caused by stream flow routing, is presented in Figure 12 (Souza 2012; Chanson 2004).

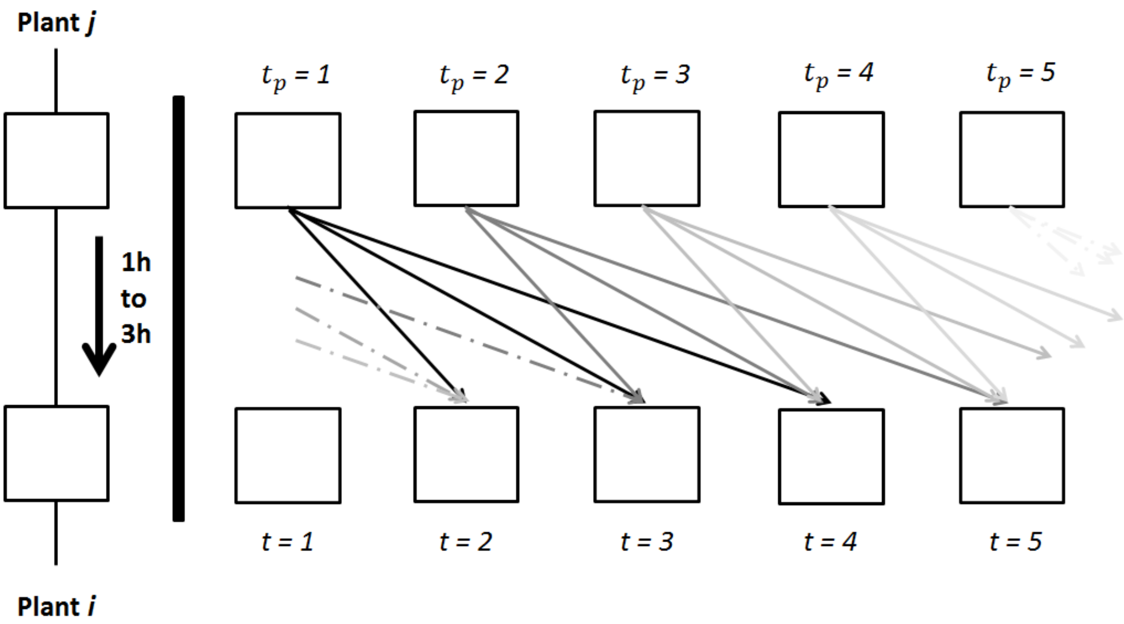

Figure 12: *Stream flow routing effect between two hydropower units with non-constant water delay. The arrows present portions of moving water in river.*

In the example shown in Figure 12, the different portions of released water of hydropower plant *j* at hour t_p reach subsequent hydropower plant from time steps $t = (t_p + 1)$ to $(t_p + 3)$, so in this case τ_{ij}^{min} is one hour and τ_{ij}^{max} is four hours. The delay can be also presented as in stream flow routing curve. The stream flow routing curve shows cumulative percentage of water released from upstream hydropower unit *j* at time step t_p that reach subsequent unit up to each time step $t = (t_p + k)$ in the future, where *k* ranges from τ_{ij}^{min} to τ_{ij}^{max} (Souza 2012). The stream flow routing curve based on statistic data is shown in Figure 13.

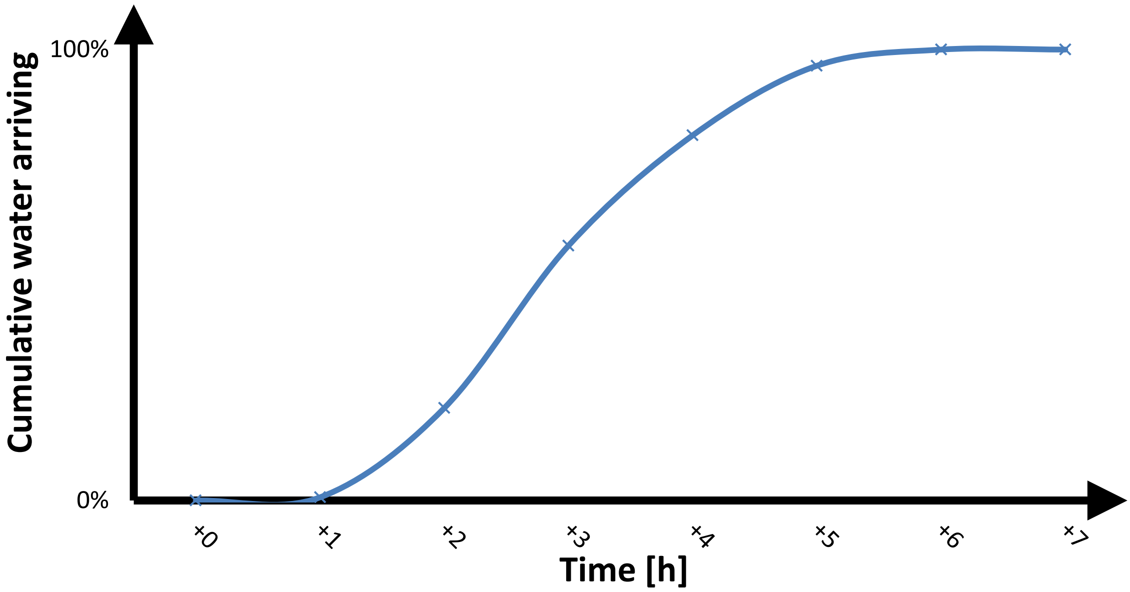

Figure 13: *Example of stream flow routing curve based on statistical data, where τ_{ij}^{min} one hour and τ_{ij}^{max} is six hours.*

Table 1 shows the delay of water share to each hour. For simplicity, Table 1 shows how much of the water arrives during each hour to subsequent hydropower unit.

Table 1: *The amount of water shares to each hour based on Figure 13.*

hour	+0	+1	+2	+3	+4	+5	+6	+7	Σ
p.u.	0	0.20	0.34	0.21	0.14	0.07	0.04	0	1

Based on Table 1, we can define a participation factor $\kappa_{ij,k}^{t}$, where t is water release time from upstream unit j to subsequent unit i and k is time difference from water release time to water arrival time to subsequent unit. Now water balance equation for hydropower unit i considered stream flow routing can be written as following:

$$V_i^t + (Q_i^t + S_i^t) - \sum_{j \in M_i} \sum_{k=0}^{\tau_{ij}^t} \kappa_{ij,k}^t \left(Q_j^{t-k} + S_j^{t-k}\right) = V_i^{t-1} + I_i^t \quad (11)$$

It has been noticed that stream flow routing curve changes in different river conditions. It can vary according to season of the year and operating condition of the river channel. The delay depends also from river flow: the delay is larger with lower river flow (Souza 2012; Vassenden et al. 2016).

The most important thing in water delay time is the distance between hydropower units. If the distance between two units would be very small, the delay can be modeled as a constant. Most of regions water delays cannot be modeled as a constant, because of stream flow routing between upstream and downstream units.

2.3.5 Hydropower reservoir

The reservoir above a hydropower plant allows for storage of water. This possibility to store water is a unique feature of hydropower compared to other renewable energy sources. If there is available volume in reservoir, the inflow to hydropower plant does not restrict plants discharge at all. The reservoir allows an opportunity for allocation of energy to periods with higher demand and therefore production can be timed when it mostly needed. The amount of energy and the amount of time it can be stored is related to the size of available reservoir (Crona 2012; Vilkko 1999).

The size of reservoir is defined by its highest regulated water level and lowest regulated water level. Allowed water levels are traditionally dictated in power plant permits which are unique for each hydropower plant. The difference between the highest regulated and the lowest regulated water level in the reservoir can be almost 100 meters. There can be also several more constrains of the regulating reservoir, for instance, the limits how the discharge has to be ramped and forbidden operational intervals. Limits are also described as soft and hard constraints: soft constraints are allowed to be broken in certain situation but hard constraints are strict (Crona 2012; Statkraft 2016).

When a reservoir can store a significant amount of the annual inflow it is called seasonal reservoir. The seasonal reservoir capacity is remarkably greater than a typical plant reservoir capacity. Seasonal reservoirs are usually geographically greater and the boundaries for water level variation are larger. The production of hydropower is practically completely controllable when it located between two seasonal reservoirs. In these types of hydropower plants, the constraint in energy production comes from maximum discharge of turbine and the annual availability of water in the upper reservoir. A seasonal reservoir has the ability to absorb the changes in inflow during different seasons. Typically, the reservoir is filled during the flood time and it is unloaded during dry seasons (Vilkko 1999; Antila 1997; Kemijoki Oy 2016)

There are different methods to estimate the volume of lake or reservoir. The estimation can be based, for example on statistical data or hydrographical measurements. Hydrographical measurement utilizes depth-sounders and the method brings out the total volume and bed shapes of the reservoir or lake (Furnans 2008). For a less laborious way of estimation for hydropower is calculating the water balance of the reservoir. Change in the reservoir volume must correspond to the combined incoming and outgoing water volumes. These volumes are discharge, inflow and spillage and these variables have the unit m^3. Volumes are usually big and inspection time is hours or days, so it is more user-friendly to use million cubic meters Mm^3. The water balance equation is:

$$x(t) = x(t-1) - q(t) - s(t) + q_{upstream} + w(t) \tag{12}$$

Where $x(t)$ is current reservoir content, $x(t-1)$ is previous reservoir content, $q(t)$ is the discharge, and $q_{upstream}$ combined with $w(t)$ is an inflow. $w(t)$ includes evaporation and other variables that are difficult to measure (Crona 2012).

When hydropower plants are in located in same riverbed they are hydraulically connected to each other. Usually, in a situation like this, the reservoir is controlled by first hydropower plant in upstream and subsequent plants reservoir does not exists or reservoir is small. If a small reservoir exists above the hydropower plant it could storage enough water for hourly allocation of electricity (Vilkko 1999).

The size of the water reservoir may change in the long run because of transported sediment. The accumulated sediment is detrimental to the lake or reservoir because it displaces the storage of water and reduces the surface area. It is problematic especially in reservoir designed for hydropower production: the useful storage volume is lost, increasingly only the inflow is available and this may be insufficient during low-flow periods. In areas of extremely high sediment yields, smaller lakes and reservoirs may fill completely, but this is relatively rare. The coarser particles are more rapidly deposited in lake or reservoir while the finer particles are transported farther depending on the velocity and dynamics of the water.

Hydro reservoir levels affects heavily to the available supply in Nordics. Hence, reservoir levels and hydro-inflow has significant role to Nordic electricity price, because half of the joint of production is hydro-based. Negative deviation in hydro reservoirs causes an increase in electricity price in Nordic markets (Vehviläinen et al. 2005; Turcik et al. 2012).

2.3.6 Measuring water flow in hydroelectric power plants

The control of water masses is the key thing in hydropower operation. The amount of available water can for example be based on run-off forecasts, reservoir levels or information about the discharge of the previous hydropower unit. Known discharges upstream are usually the main source of the level of inflow for a hydropower unit located in chain.

Water flow is not directly measured in hydroelectric power plants. The water flow through the turbine can be calculated for example from head level (difference between intake water level and tail water level), measured hydroelectric power from generator combined with known turbine efficiency curve. The efficiency curve is defined experimentally by the turbine manufacturer, possibly with a miniature version of the turbine. If one of above-mentioned measurements is incorrect, the discharge calculation is also incorrect.

In practice, this can be noticed in two successive hydroelectric power plants in the river, where there are no noteworthy inflows between the plants. Figure 14 presents such a situation. There is not remarkable inflow between hydropower units and the discharge from previous unit has been constant long enough or in other words, the sequential unit discharge had reached the stable level.

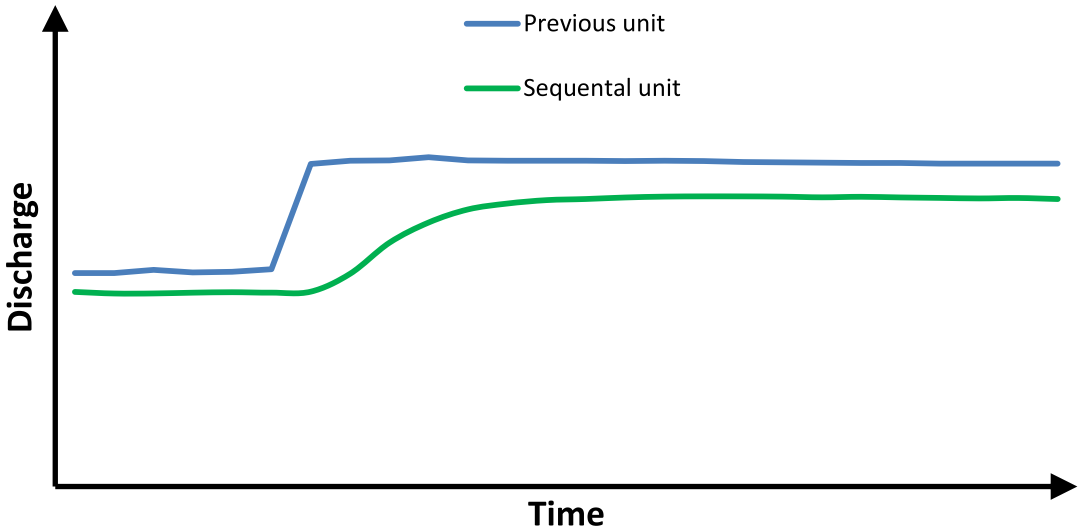

Figure 14: *The different between two subsequent unit discharge calculation.*

As Figure 14 shows, there is a continuous difference between two sequent units discharge. If both measurements are accurate, it would mean that water is disappearing or in other words, run-off should be negative. However in reality, there is an error in either or both hydropower unit's water flow measurement. The main reason is most likely incorrect efficiency curves in either one or both hydropower plants.

This kind of error is easy to handle while there is no significant inflow in between the hydropower units. The situation can be annoying when there are more than two hydropower units in chain and there are remarkable inflows between units: there is no knowledge of the correct discharge of hydropower units and thus the inflow between the units is hard to determine. This situation is harmful for predicting inflows when there is no valid data for predicting. Incorrect inflow forecasts are tedious for hydropower planning and can lead to suboptimal hydropower planning. This may have economic implications.

3. DETERMINING WATER DYNAMICS IN RIVER SYSTEM

The catchment area of the studied water system is 27 046 km^2 and 11 % of this area are lakes. The catchment area consist larger and smaller lakes and river links between these. The catchment area erupts to the sea through the main channel. The main focus of this thesis is the main channel which should be understood as the river part between last significant reservoir and tail water of last hydropower unit. There are four hydropower plants located in the main channel and it is illustrated more clearly in Figure 15.

3.1 River system

The power plants in the studied river system of this thesis are owned by multiple companies. The river system is run by different parties with different interests to plan their own hydropower units. However, hydrological balance must be kept within certain boundaries, so there are commonly agreed rules about water usage. This guideline is called Governance Rule, which gathers all the commonly agreed rules by the producers as well as limitations from environmental permits together in single instructing document.

The Governance Rule assists the different parties to co-ordinate their hydropower operations and follow the environmental conditions easier while there are many parties operating in same water system. It illustrates the river system which consists of different hydropower units, reservoirs and streams and gives an overview of the relevant catchment area with its specific characteristics. The Governance Rule starts with an overview from water system and its different areas. It is an important to be acquainted with overview of water system before more specific rules and regulation can be included to the model of system. The Governance Rule continues with an elaborate specification of regulation limits, which in general varies according to season of the year or discharge of the water system. The commitment of minimum discharge and other specific characteristics for each hydropower unit is defined in specific description of each unit. Finally, it defines rivers usage and directives during special circumstances like spring flood time or ice cover creation case-by-case.

3.1.1 Hydropower units and reservoirs

Simple illustration of the river system examined in this thesis is presented in Figure 15. Hydropower units are identified with P and water reservoirs with R. There are eight hydropower units in river system and water is stored to five reservoirs in total. Power stations *P1*, *P2*, and *P3* is operated by player *A*. Power station *P4* is operated by player *B* and rest of power stations are operated by player *C*. The player who operates the power station below reservoir takes care of regulating of the reservoir located above. Because power stations *P1* – *P4* and reservoirs *R1* – *R4* are operated by third party, those are not target of study at all in this thesis.

The operation of upstream hydropower unit has high influence to downstream units and reservoir contents. Hence, upstream unit parties are involved to send their average discharge plans to downstream parties. In practice, the regulation of *R1* inform the daily average to regulator of *R3*, while regulators of *R2*, *R3*, and *R4* inform their average discharge as for to regulator of *R5*. The run-offs to reservoirs and between the hydropower units are forecasted by Finnish Environmental Institute. The planned amount of water released of each reservoir is the sum from upstream unit discharge, run-off forecasts, and wanted change of reservoir content.

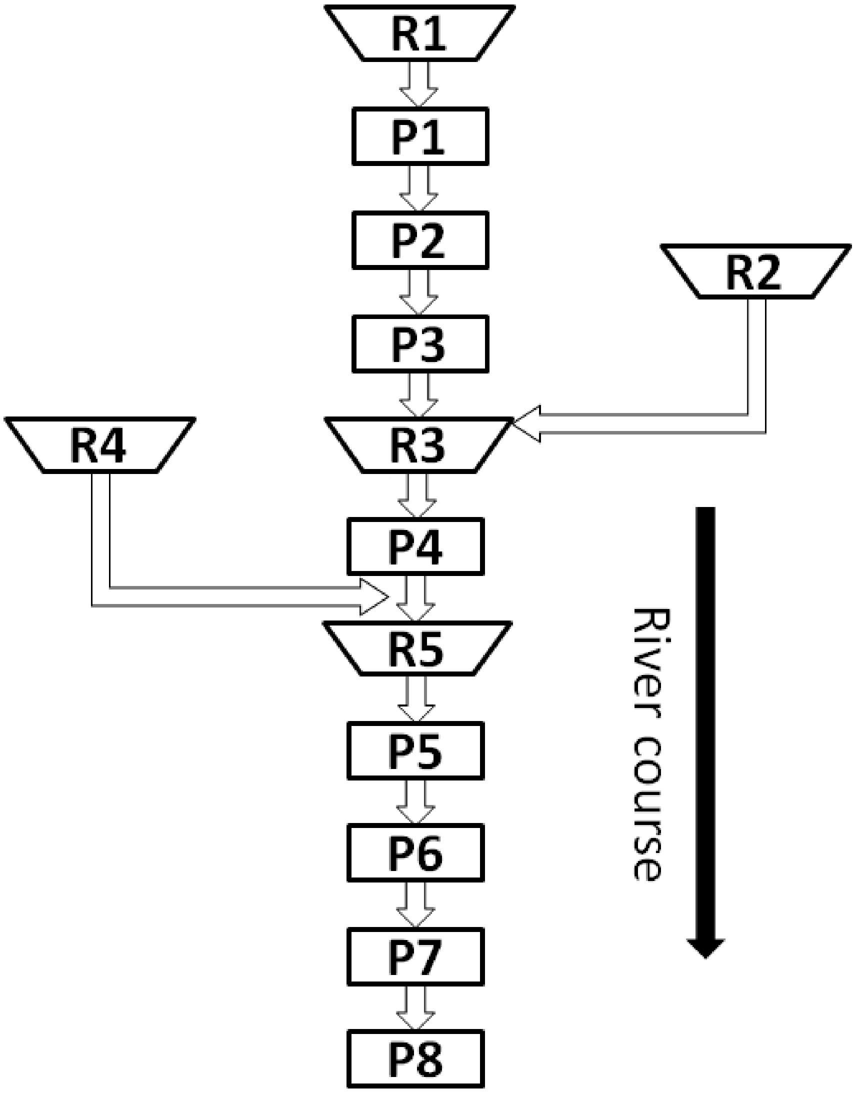

Figure 15: *The examined water system of this thesis.*

The *P2* and *P3* are ROR units and follow the discharge of conventional hydropower *P1*, P4 is located between two reservoirs and it is planned and operated as conventional hydropower unit. The regulating of *R5* is made with conventional hydropower unit *P5* where the water flows through the ROR units *P6* – *P8*. The station *P6* is purely ROR unit and it follows the discharge of *P5* with a few hours delay. Units *P7* and *P8* are presented as ROR units in Figure 15 but both stations have small storage capacity which allows temporary short-term regulation.

The main focus of this thesis is the river section between hydropower units *P6* and *P7*. The distance between these two units is approximately 45 kilometers along the river side. There are also noteworthy run-offs between these two units which must be taken account when the production is planned. Roughly half of the run-off is estimated with a water level-to-discharge –curve, which in practice is not very accurate, especially when the river is covered with ice. The hydropower plant's *P7* vicinity is presented in Figure 16.

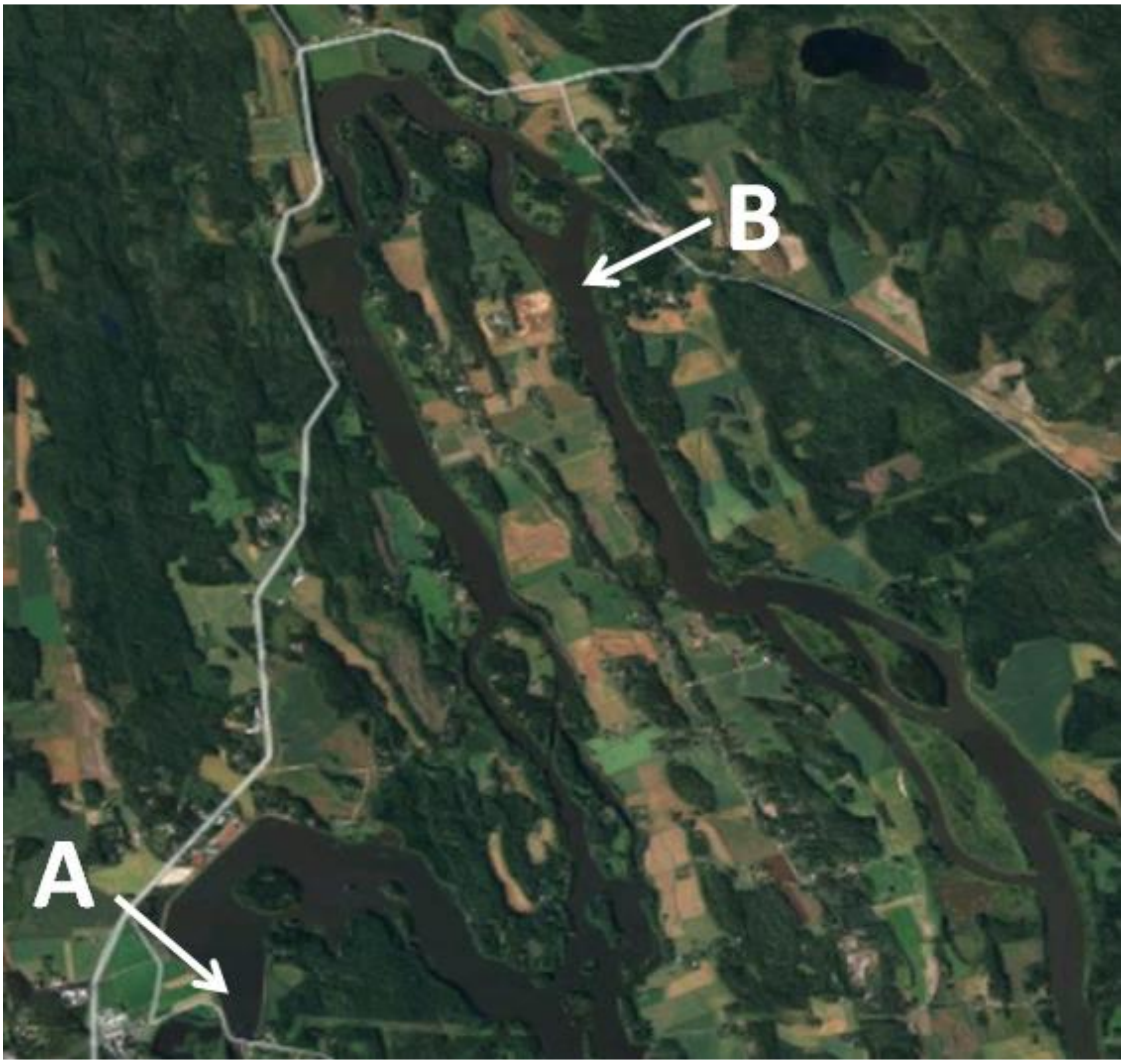

Figure 16: *Vicinity of hydropower plant P6. A) Hydropower plant and intake level measurement. B) Further water level measurement, where plants permit is also linked.*

In Figure 16, the hydropower plant's *P7* intake is marked with *A* and further water level measurement with *B*. The distance between measuremt's is approximately 11 km along riverside.

3.1.2 Planning period

The planning process has many different purposes and those can be divided into economic and environmental considerations. Main functions of the planning process are presented as following:

1. Take into account hydropower plants maintenances
2. Flood damage mitigation
3. Mandate for short-term regulation
4. Long-term forecasts for hedging and economic reporting.

During maintenance, the amount of discharge through the turbines is restricted which may lead to uneconomical spillage. Spillage can be minimized by utilizing reservoir volumes and storing water to reservoirs combined with good scheduling. A well planned maintenance is performed during a period when the river flow is low enough and hydropower station can operate without the unit that is not available. The restrictions in discharge can be take account when river system plans are calculated. However, maintenance in station means limitations in flexibility which is unavoidable (Crona 2012).

Hydropower and its regulation enable water management. Hydropower has significant role in flood management. For example, areas near riverbanks were often flooded before regulation took place in water management. Removing high discharge peaks from water flow in rivers is possible with reservoirs. Water level in reservoirs may be lowered with anticipatory regulating and planning. This makes room for flood waters. As is typical in hydropower production planning, the inflow predictions of high accuracy are also important in flood predicting. The higher precision of inflow forecasts would help the hydropower operator to better prepare to flooding which would improve flood mitigation (Mill et al. 2010).

In the studied water system, flood control has been written into the original targets of regulation. Flood protection is a continuous process which is carried out during both winter and open water times. During flood and other problematic circumstances, the whole river conditions are observed and all regulating assets in the water system are utilized. For example of above-mentioned, upper water limit of *R1* can be broken if the discharge of *P7* is in danger to exceed over 500 ${m^3}/{s}$ and *R5* can break its seasonal upper water limit for mitigating flooding in the lower reaches of the river. Flood mitigation is extremely important in this particular river because the highest flood risk area of

Finland is located in the delta of the river and smaller, but still significant, risk area between hydropower units *P6* and *P7*. Flood risks are especially high during winter time when potential frazil ice flood could cause major economic damage. Water system's strategic utilization is the most important way of reducing the financial risks of flooding. Common targets stand out especially during divergent hydrological situation or remarkable jeopardy (UPM Energy Oy; Flood 2015; Finnish Environmental Institute 2016).

The planning of the studied river system consists of mid-term and short-term planning. Mid-term planning is based on rolling one year planning procedure and it starts from next calendar week until one year forward with one week resolution. As inputs the mid-term planning uses target levels, price forecasts, inflow forecasts and discharge restrictions or other unavailability of hydropower units. As the river system is operated by many parties, the planning cannot be made by one party alone. All the parties give their own regulating suggestion of reservoirs. Suggestions contain possible outages of hydropower units or other restrictions. The suggestions of all parties are gathered together and the final plan of regulating is formed as an weighted average of regulating share of ownership. In summary, all parties do their own mid-term plans and the final plan is formed from these plans.

Mid-term planning is the basis for short-term planning. Short-term planning can see to the end of the prevailing week forward in this study. The short-term planning is re-made every day and the planned production of hydropower units is sold to day-ahead-markets to Nordic electricity exchange. The time span of day-ahead-markets is 24 hours from 1 a.m. to 1 a.m. in EET time zone. The river system could be planned less frequently than every day. However, the benefits from using latest inputs (e.g. run-off or price forecasts) are greater than the hinder caused by daily re-planning. Also, the production plans and water levels behavior is much more accurate when the river production is planned with latest and best obtainable forecasts.

The short-term planning is modelled more accurately compared to mid-term planning. Short-term planning model includes delays between the hydropower units, efficiency curves for each turbine, higher resolution price forecasts, and energy losses according to intake and tail water level with one hour resolution. The result of short-term planning is discharges, water levels, and energy's converted with resolution of one hour. The planning procedure starts with controlling water level of *R5* with taking account run-offs and discharge from *P4* and *R4*. The run-offs during riverside and possible unavailability's of prevailing hydropower units *P6* – *P8* have to be taken into account when the decision of discharge level of *P5* is made to avoid a possible spillage. After this the hydropower units *P7* and *P8* are planned with utilizing the small storage capacities above intake. The planning is made by Physical Trader who utilizes unique optimization model developed particularly to this water system.

3.1.3 Constraints

Every reservoir has its own highest and lowest water level limits. These regulation limits can vary according to seasonal conditions. The regulation of a reservoir must be carry out in such way that limits are not infringed. The river course between *P5* – *P8*, which is on main focus in this thesis, has its own restrictions. The minimum average discharge from *P5* should not go below 30 $m^3/_s$ and the *R5* has its minimum and maximum water regulation levels as well. As previously mention, *P6* is a pure ROR unit and follows the discharge of *P5* with a few hours delay. *P7* does not have discharge constraints but its permit is tied to two water level measurement located in different places. First measurement is located in the intake immediacy (measurement *A*) and the second one is eleven kilometers to upstream from the power plant (measurement *B*). Both water levels have their own upper limits and intake level has also the lower limit because of physical constraints in hydropower units (UPM Energy Oy).

The operating principles of different units are different. In the studied river system, the operation principle depends on plant's head, in other words, the specific energy of water. In the studied river system, there are two different operation styles: plants are operated by planned energy or by planned discharge. While the plant is operated according to discharge, there might come balance error from the energy output. If the plant is operated according to planned energy, the error might come from plants discharge. If the modelling of hydropower plant's intake level, tail water level and efficiency is faultless, there should not occur any errors in energy produced or water discharged. However, in practice, this is impossible. If the plants behaves very differently from modelled, according to equation (6), hydropower unit needs more or less water, depending from separations sign, to produce same energy as planned. Thus, when the hydropower unit operation basis is chosen, we have to observe the energy balance error or error in discharged water caused by hydropower unit. If the specific energy per cubic meter is low, the caused balance error depend from production modelled is not remarkable if the plants is operating according to discharge. If the plants specific energy per cubic meter is big, the deviations from modelled production can be remarkable if the plants is operating according to discharge. While the plant is operated according to planned production, the water discharged could be more or less, which could have significant influence to plants intake and tail water level. For example, while the plant head is flatter than planned, the water needed to produce planned energy is more, which cumulates in the long run if it is not accounted for.

In the studied water system, hydropower unit *P5* is operated with planned discharge. The drop of *P5* is smallest from hydropower units which are regulated. If the plant were operated according to energy, it would cause remarkable harm in downstream plants water levels. The possible balance error of *P5* caused by this operating style is not significant. The hydropower unit *P6* is not manually controlled and the control circuit keep the constant intake level, which means that *P6* is operated as per arriving discharge. Hydropower units *P7* and *P8*, whose reservoir enables daily regulation, are operated by planned energy. This means, that there are deviations between hydropower plants planned and realized discharges. These errors could cause a significant cumulative error in hydropower unit intakes.

3.1.4 Planning uncertainty and problems

Major challenges in the planning of the river system between *R5* and *P8* are the unknown delays between *P6 – P7* and run-off forecasts. Delays are modelled as a constant shared delay profile at the moment. However, this is disproved by experimental knowledge. Experimental knowledge indicates that the delay is not constant and it varies in different situations. It seems that water delay is different in different base flows of river.

The run-off between *P6 – P7* is hard to determine. Mainly because discharge measurement in hydropower units *P5* and *P6* are not calibrated which brings into question are the discharge measurement of unit *P7* equal with *P5* or *P6*, or neither. Thus, determining the accurate run-off between *P6 – P7* is hard.

Sudden change in weather like torrential rain is problematic to take account in production planning. This requires accurate weather forecasts, which are not readily available. Torrential rain can cause the level of *R5* to rise very quickly if it hits right above the reservoir. Also, the run-offs between *P6 – P7* can increase quickly if torrential rain hits above the river. This is mainly seen in discharge of the small sidestream which connects to the main channel between *P6 – P7*. This is the biggest single run-off of the river. This river is approximately half of run-offs as rule of thumb. Most of the year this river is not a problem in production planning or operating but it becomes tricky during flooding or rainy seasons. This river can “burst” suddenly which is not easily seen while production plans are made on previous day.

Making production plans for the river system is hard especially during the time when the discharge changes of the hydropower units are big and the base flow of the river is high. Especially behavior of intake level of *P7* is hard to predict because its size and volume is not well-known at the moment. The intake level must be kept low during high discharges because the upper limit of water level eleven kilometers away reduces it. *P7* is operated according to energy output, so the errors made in intake level modelling cumulate in operation. This mainly occurs because the planned energy needs more or less water during low or high intake level. The hydropower unit *P7* is planned and operated like conventional hydropower but the production cannot be planned optimally because the models in use are not accurate enough. In practice, this means that the content of the storage above the intake level cannot be planned full enough before it starts to drain during higher electricity demand.

The size of the *R5* is big enough to implement daily regulation with *P5* during the week. The amount of discharged water from *P5* roughly determines whole river daily production level. The operator of *P4* gives rolling preliminary discharge plans with six hours resolution to operator of *R5* and *P5*. These discharge plans are preliminary and thus those may change substantially which makes the typical optimization of regulating of *R5* more difficult or even impossible. There are three different water level measurements in *R5* and the measurement located nearest of hydropower unit *P5* is mainly used while the water content of *R5* is approximated. When the discharge from *R5* declines it causes leveling of the lakes which make up the reservoir which can be noticed from the decreased difference of three different water level measurements in *R5*. This typically comes up during weekends when the water level measurement located nearest hydropower unit *P5* increases more than modelled. This phenomenon is hard to model and it is problematic because it makes the optimization of *R5* harder when the behavior of water level is not known very well.

Hydropower unit *P7* is co-owned hydropower plant. Operating, planning, trading and owning involve multiple companies. It is operated and planned by service provider, but the energy produced is traded to Nordic electricity markets by the owners. However, *P7* is placed on one of its owner's electricity balance who is not planning or operating the hydropower plant. This make more difficult to fix possible errors in production planning. The market-based fixing cannot be made on intraday markets or regulating markets effectively because the operator cannot utilize changes in markets so easily. The production plans of *P5 – P8* are made before 10:30 a.m. which in turn means that not all input forecasts are necessarily updated. The day-ahead production of *P7* is sent to the owners at 10:30 a.m. which means that the next day production plans cannot be altered easily anymore before market closure. In practice, this means that all changes in energy production compared production plans made before 10:30 a.m. is sold or bought from markets with higher price uncertainty than spot-markets.

The river production plans are made with a unique optimization program, which gives results of every units discharge with one hours resolution. The optimization is made in external server, which solves the optimal result from given start and stop water levels. The physical characteristics of the river are modelled to optimization. After the optimization result is given, a person responsible for trading may alter it manually. Manual corrections are made to better accommodate uncertainties in the input data and the non-linear nature of production curves. The main reason for changing discharge levels manually is the manual operation of hydropower units. Changing production level at every hour is laborious and economically questionable with small price differences and starting costs of units.

The river is planned every single day of the year and those plans are made by at least three different persons. The modelling of the river is still a simplification and the model does not fully replicate the physical behavior of the river, thus every Trader has own views of water levels behavior. When a Trader is making plans to next day, he's inspecting current day's plans as well. The behavior of water level for every hour is fed to the planning system straight from the model, so the planner cannot know how much previous day Trader has taken own view to the water level behavior. This impairs the continuity of planning desk (Nyrhinen 2016).

3.2 Tests

The objective of the water system development tests was to find out the water delay between *P6* and *P7* and to clarify the size of the water storage capacity of unit *P7*. To determine both, the delay and the size of the storage were devised their own types of tests. These tests were carried out between May and August. The size of the storage capacity was decided to be examined first because without it, the delay would be hard to determine. The presumption was that the knowledge of storing capacity was supposed to facilitate the determination of the water delay between *P6 – P7*.

During the tests, the production of river was normally planned the previous day in concert with the Trader and the production of the river was sold to the Nordic electricity exchange. The test runs required the production planning to deviate from normal, so the tests caused economical loss. The economic losses were comparatively minor and were further minimized by timing tests on weekends. The tests were always performed by setting river conditions as topmost priority. Also, no environmental limits set by power plant permits were broken because of tests.

The tests were performed under circumstances while the river section between *P5 – P7* was in full control and stable. The most favorable timing for the tests was during the time when there should not be a remarkable change in run-offs. This would decrease the possibilities for errors in the data analysis.

The test data was collected from the hydropower operating system or from the energy management system which in turn gathered it from the hydropower operating system. The data gathered from the hydropower operating system is available in multiple time spans up from three minutes. When data was imported, a three minute time span was used because the time of change in each variable was easier to notice. Discharge and height data is available in two decimal accuracy with units $m^3/_s$ and m. The accuracy of discharge data was sufficient but during the data analysis it was noticed that the height data could have been more accurate.

All tests' data were not classified as successful. During some of the tests it rained, which caused changes in run-offs during tests and therefore the accurate inflow to $P7$ was hard to determine. Also, one test had to be cancelled because of unexpected changes in the river system. The hydropower plant $P7$ participates also in primary regulating market which causes fluctuations in discharge based on local frequency deviation from 50 Hz. The capability to frequency control is individual to each generator and naturally during higher discharge there are more generation units involved in automatic frequency control and thus the amount of discharge varies more.

3.2.1 Water reservoir tests

The storing capacity tests of the hydropower unit $P7$ were performed during May and June 2016. During May, the average flow of the river was substantially above normal, which provided difficult circumstances to perform storing capacity tests, but also, it admitted the possibility to make tests during high flow of the river. The river flow decreased in later May and June which admitted favorable circumstances to perform tests with larger scale of base flow. The preconception was that a discharge level may affect to the results of the tests, thus utilizing different circumstances in the river was important. Although, the hydropower unit $P7$ regulating during high flow in the river is not sensible, the results were exploitable.

The main idea of storing capacity tests was to make a step increase or decrease of discharge of hydropower unit and measure the changes in water level. Before the tests were performed, there was a thought that water levels could behave differently during decent and rise. Thus, the tests were performed to both directions. In order to perform tests successfully, reasonably the accurate knowledge of inflow was required. Thus, the discharge of $P5$ and $P6$ should be held constant long enough, so that all the influence of discharge changes could be seen. To determine accurate run-off between $P6$ – $P7$, hydropower unit $P7$ discharge should be as much as the inflow to power plant is. This was made by the operators who manually looked for discharge level where upper water was kept stable. The operator made this before and after test, because the information from run-offs change was required as well. After all conditions were favorable, the stepped increase of discharge was started in $P7$. The duration of storing capacity tests varies between 5 hours 8 hours.

Before the tests were performed, there was an assumption that water levels could behave differently during descent and ascent. Thus, the tests were performed to both directions. Also, there was a preconception that the discharge level may affect the results of the tests, thus the tests were performed in several different discharge levels.

The main idea of tests is presented in Figure 17. The duration of test was five hours and the time span of data using in Figure 17 is seven hours.

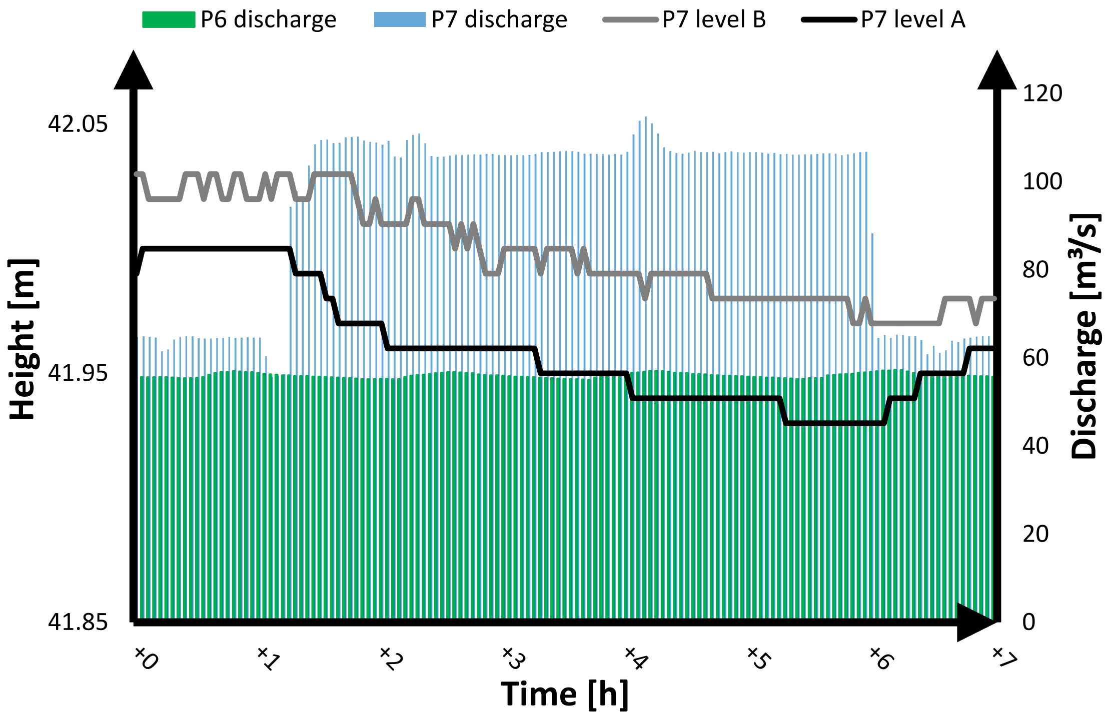

Figure 17: *The portrayal of storing capacity tests. P7 level B is located 11 km upward from plants intake.*

As seen from Figure 17, the discharge of previous hydropower unit was sufficient because the minimum discharge of *P6* during test was 1.75 % below and the maximum 2.28 % above the average flow of *P6*. The discharge range of *P6* during test converted to water cubic meters cannot be perceived noteworthy. The inflow to *P7* can be perceived as well-known in this test. The upstream water measurement was constant before increased discharge combined with the acceptable constant discharge of *P6* and thus the run-offs between these two plants could be calculated. The stable discharge, duration and size of the step of all test runs are presented in Table 2.

Table 2: *The duration, stable discharge, and the size of the step increase. Stable discharge is the discharge when hydropower unit P7 intake is stable.*

	Time of step increase h	***Size of step increase*** m^3/s	***Stable discharge during test run*** m^3/s
Test run 1	*6.0*	*-61.0*	*176*
Test run 2	*6.0*	*-47.4*	*182*
Test run 3	*6.0*	*41.0*	*176*
Test run 4	*5.0*	*46.1*	*118*
Test run 5	*4.8*	*42.1*	*65*
Test run 6	*7.9*	*30.7*	*150*
Test run 7	*4.9*	*28.3*	*157*
Test run 8	*6.0*	*-29.3*	*153*
Test run 9	*5.7*	*48.0*	*259*
Test run 10	*7.0*	*22.9*	*417*
Test run 11	*7.2*	*25.7*	*415*

When storing capacity tests were analyzed, it was of primary importance to determine the accurate run-offs between hydropower plants *P5 – P7*. *P5*'s measurement was decided to use in place of *P6*'s measurement. When analyzing the storing capacity tests data, one of the main presumptions was that the discharge measurement of *P5* and *P7* are equal compared to each other. The run-offs were determined by utilizing equation (12). Because the upper water measurement was constant through requisite time span before stepped increase or decrease was performed, in other words, $x(t) - x(t-1) = 0$, the equation (12) can be remodeled to studied river system as following:

$$w_{run-offs}(t) = q_{P7}(t) + s_{P7}(t) - q_{P5}(t) - s_{P5}(t) \tag{13}$$

The data from discharges were gathered with unit $m^3/_s$, and the volume in SI-units grows to become a very large number very quickly. For this reason, it is easier to utilize something called an Hour Unit (*HU*), which is defined as the volume summed up by a discharge of 1 ${m^3}/{s}$ during one hour: $1\ HU = 1\frac{m^3}{s} \cdot \left(1h \cdot \frac{3600}{1} \cdot \frac{s}{h}\right) = 3600\ m^3$.

The storage content unit, which is mainly in use in this study and easy to understand, describes how many cubic meters in seconds the difference should be between inflow and outflow so that it would cause one centimeter change in reservoir level. When the difference between inflow and discharge calculated in *HU*s is divided with the reservoir level change, it results is reservoir storing capacity shown as following: $storing\ capacity\ factor = \frac{\Delta q}{\Delta y}\frac{HU}{cm}$.

A cumulative difference between inflow and outflow and cumulative water level change was observed when the storing capacity was calculated. Because the data from upper water measurement was obtainable with only one centimeter accuracy, the division results between cumulative water attrition and accumulative water level development were not smooth. The results were smoothed by taking the average from the ten last results. This can be seen from Figure 18, where the ten last results are marked with black.

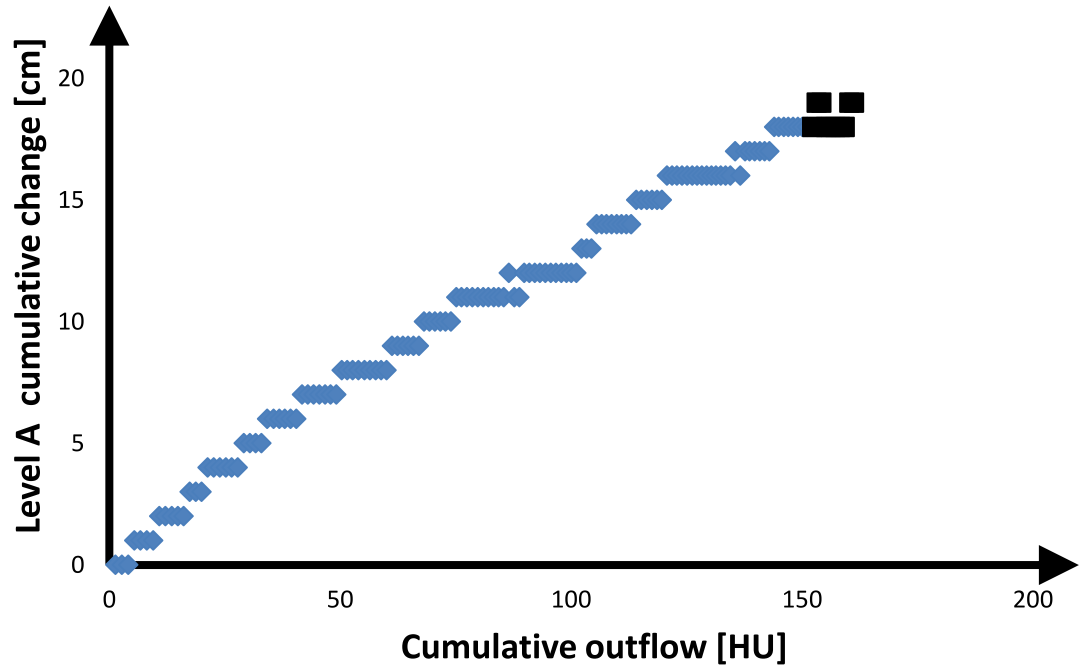

Figure 18: *The results of the test run 10. Cumulative outflow is the remainder between calculated inflow and hydropower unit discharge. It is shown as hourly cubic meters.*

The 1 cm reporting accuracy and the issues it causes in the storing capacity estimation is seen in Figure 18. According to storing capacity equation, the storing capacity factor is the slope between cumulative outflow and cumulative water level change.

3.2.2 Water delay tests

Water delay tests were performed during July and August 2016. The water delay tests were not easily implemented during high river flow because controlling and forecasting of the river is more difficult during that time. Also, during high flow of the river, run-offs are higher and the variation is bigger. During July and August, inflows to water system were on a favorable level, and the tests could be performed. As mentioned before, the analysis of water delay tests requires knowledge of storing capacity volume and thus delay tests were performed after storing capacity tests. The quantity of water delay tests performed was less than of storing capacity tests.

The main idea of water delay tests was quite similar compared to storing capacity tests. The stepped increase or decrease of discharge was made from the hydropower plant *P5* and measured water receiving to *P7*. The amount and the time of water received to *P7* were estimated from water level accumulation right above plant's intake. Similarly as in storing capacity tests, the hydropower plants *P5* and *P7* discharge and water level of *P7* were approximately constant enough long before the test, so that exact run-offs during the tests could be determined. The discharge level of *P7* when upper level was stable was searched by the operator.

Figure 19 shows the main idea of the water delay tests. The data shown in Figure 19 is a sample taken from the test run 12.

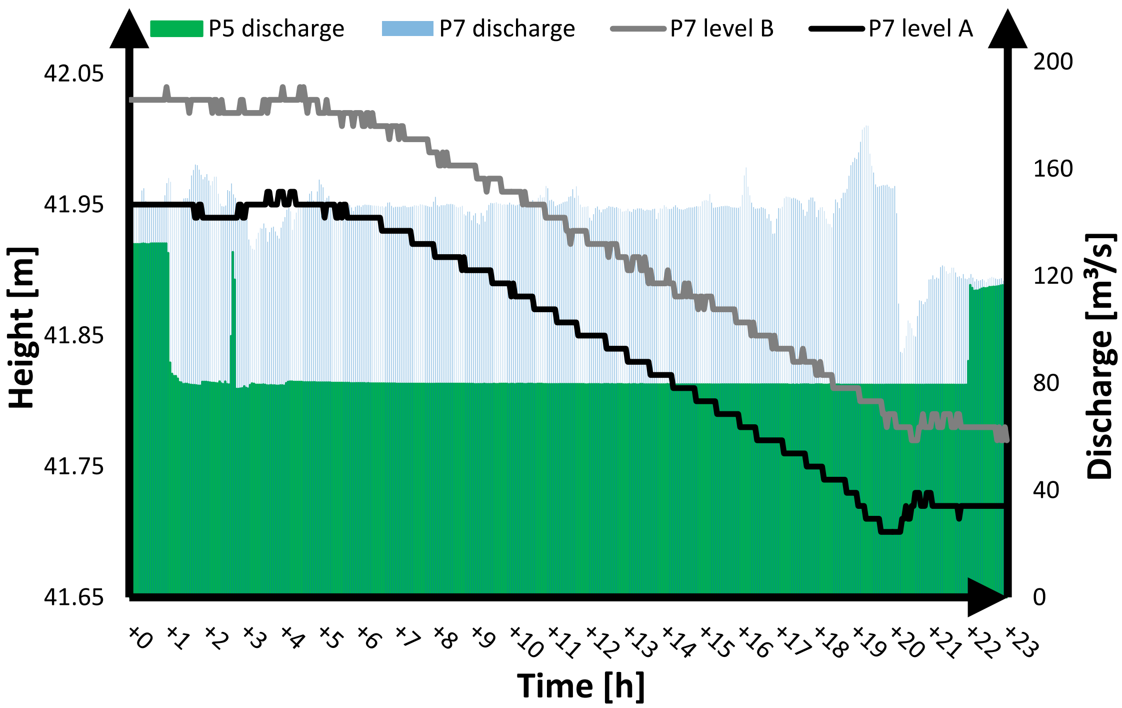

Figure 19: *Statistical data from water delay tests. In this test a stepped decrease in discharge was made. The data in the Figure is taken from test run 12.*

Figure 19 shows how water level in P7 starts to decrease after some delay as a result from decreased discharge in *P5*. The level of water in *P7* decreased until the operator started to search for a new stable position. When a stable position was found, the test was finished. The discharge increase of *P5* at the end of the test does not have influence to inflow of *P7* during the test, because the time delay between *P5* and *P7* is several hours. As seen in Figure 19, the discharge of *P5* is stable enough excluding the peak which was reached quite soon after the discharge decrease. The peak is a result from the switch between the generators, which caused higher discharge temporarily. The peak could not be perceived noteworthy in the test performed because the duration of the peak was not significant and that amount of water was also taken into account when the data was analyzed. The bigger problem in this test was the instability in *P7* discharge, which was mainly caused by automatic frequency control. Changes in discharge caused immediate change in upper water which hindered solving delay from the data. The maximum P7 discharge was 19.8 % higher than its average and the minimum was 88.3 % from its average during the test.

In water delay tests, the run-offs were determined similarly as in storing capacity tests. Like in the storing capacity tests, the discharge measurement of *P5* was used in calculations in place of *P6* discharge measurement and one of the main assumptions was that the discharge measurements between *P5* and *P7* are equal compared to each other. Similar as in storing capacity tests, also the water delays were solved from *P5* to *P7* in place of *P6* to *P7*. The assumption was that the water delay should be divided like a wave to different time spans. The assumption is based on a reference (Chanson 2004) and experimental knowledge of water journey between *P5* and *P6*.

Table 3 presents the relevant information from the delay test runs. It shows the constant discharge of *P5* before tests, size of step increase from *P5*, and stable discharge of *P7* during test run.

Table 3: *The statistics of the water delay tests. Figure 19 presents the test run 12.*

	P5 constant discharge m^3/s	***Size of step from P5*** m^3/s	***P7 Stable discharge*** m^3/s
Test run 12	*131*	*-51*	*146*
Test run 13	*109*	*45*	*146*
Test run 14	*98*	*55*	*102*
Test run 15	*59*	*44*	*71*

When solving the water delay between hydropower plants, the development in water level measurement *A* was used. The divided share of increased or decreased inflow to each time period could be solved with knowledge from *P7* discharge, accurate run-offs, and discharge of *P5*. The inflow in one hour accuracy is required in this thesis. The water delay from *P5* was solved with utilizing equation (11). When the equation (11) is converted to suitable in this river system, it can be presented as following:

$$\Delta V_{P7}^{t-(t-1)} = I_{P7}^{t} - (Q_{P7}^{t} + S_{7}^{t}) + \sum_{j \in M_i} \sum_{k=0}^{\tau_{P7,P5}^{t}} \kappa_{P75,k}^{t} \, (Q_{P5}^{t-k} + S_{P5}^{t-k}) \tag{14}$$

Where $\Delta V_{P7}^{t-(t-1)}$ can be presented as following:

$$\Delta V_{P7}^{t-(t-1)} = V_{P7}^{t} - V_{P7}^{t-1} \tag{15}$$

The final equation used in calculating forecasted upper water level can be presented as following:

$$V_{P7}^{for} = V_{P7}^{t-1} + \frac{I_{P5}^{t} - (Q_{P7}^{t} + S_{P7}^{t}) + \sum_{j \in M_i} \sum_{k=0}^{\tau_{P7,P5}^{t}} \kappa_{P75,k}^{t} \, (Q_{P5}^{t-k} + S_{P5}^{t-k})}{V_{P7,storing\ capacity}} \tag{16}$$

To clarify, the calculated forecast to upstream water level is an equation which consists of previously forecasted water level, remainder between inflow and discharge, divided with known storing capacity. The change in reservoir content from previously forecasted value should be equivalent with the difference between inflow and discharge. The first forecasted value used previously realized water level observation in place of previously forecasted value V_i^{t-1}. The shared delay factory κ formed as many variables as the assumption from water delay in hours was.

The constraints used while solving equation (16) are presented as follows:

$$0.96 < \sum_{k=15}^{\tau_{P75}^{15}} \kappa_{P75,15}^{15} < 1.04 \tag{17}$$

$$\sum_{k=1}^{\tau_{P75}^{1}} \kappa_{P75,1}^{1} < 0.01 \tag{18}$$

$$\sum_{k=3}^{\tau_{P75}^{3}} \kappa_{P75,3}^{3} < 0.02 \tag{19}$$

In other words, hour $t + 0$ share is restricted below value 0.01, hours $t + 1$ and $t + 2$ are restricted below value 0.02. In addition, the sum of all released water from *P5* could vary ± 4 % mainly because different hydropower plants water discharge measurements diverge from each other, and there is no clear view, which hydropower plants discharge measurement is nearest to the actual.

Regression analysis was used in solving water delays. The method used in regression analysis was the sum of least squares method where the sum of squared differences between the calculated upstream water level and realized water level was minimized by changing the delay factors. The sum of water the shared water delay values κ practically affects to the amount of water arriving from *P5* to *P7*. Before the data was analyzed, it was known that the water delay between *P5* to *P7* should be divided somehow from four hours to sixteen hours. The knowledge of water delay range was the base of constraints of the κ values. The target was to create a physical adapter model from the results.

3.2.3 Data analysis

Data analyses were performed with Microsoft Office Excel 2010. In water delay analyses there were also Excel Solver Add-In utilized to solve mathematical problems. The data of each test was imported from the hydropower controlling system to an Excel file and it was archived for possible future reference.

3.3 Forecasting tool targets

The modelling of *P7* water levels turned out to be tricky and challenging both experimentally and modelling-wise. Hydropower plant *P7* has been operated for a relatively short time by the current operator. Different forecasting models have been created to model hydropower units *P7* water levels but the results in predicting have been unsatisfactory. The river section between *P6* and *P7* has been difficult to model and its unforecasted behavior affects *P7* water levels and controlling. Therefore, this thesis is mainly focused on this river section. The problematic behavior and the lack of workable forecasting tools cause error in *P7* production balance. Furthermore the water level behavior is sometimes hard to forecast even in few hours away which causes unplanned operation in *P7* electricity production. This, in turn, might cause economic losses, because of the way of the production imbalance is priced. The planning errors in *P7* water levels also cause re-planning in below, in other words, in hydropower plants *P8* intake, which possess the biggest head in the hydropower system. Therefore, water management and economic risks extend not only to *P7* but also to *P8* (Fingrid Oy 2016).

The objective of this thesis is to study water movements’ and water levels’ behavior between the river section *P6* and *P7* more precisely. To reach this objective, the results of unique tests of the river and its historical data are utilized. A new forecasting tool based on these research results was created, which aims to be used in the hydropower controlling center. The objective of the new forecasting tool is to predict the behavior of both water levels of *P7*. Before this study, there was not forecasting model to water level *B*. This should simplify planning and operating of river system.

A forecasting tool was created from several physical adapter models, which try to simulate water levels and water movement in the river system. One of the main objectives of this thesis is to find out, is it sensible and profitability to model this hydropower system as this way. If the results of this study are not satisfactory, other techniques could be used, but they require economical investment.

4. RESULTS AND DISCUSSION

4.1 Hydro reservoir

The upstream and water levels measurements locations of hydropower plant *P7* are presented in Figure 16. When the research was started there was no certainty is the storing capacity of hydropower plant *P7* located in water level *B* measurement or right above hydropower plants intake (measurement *A*). Experimental knowledge has shown that hydropower plants intake does not behave like a conventional reservoir. Both water level measurements have their own environmental limits.

The assumption of *P7's* storing capacity was that it owns unambiguously constant ability to store water. Of course, there would not be exact constant results from tests while the quality of data and circumstances in river changes, but assumption was that results from different test runs should be similar to one another. The volume of the storing capacity had not been determined with this accuracy before the tests, because circumstances to determine it are challenging during normal river production. Of course, there was a rule-of-thumb value for storing capacity volume, which was in use.

All the storing capacity tests were analyzed with the same method presented in section 3.2.1. Most of the tests were performed before analysis was started. Against expectations, the storing capacity tests did not give equal results while tests were analyzed. The storing capacity tests results were analyzed from both water level measurement locations. The results, calculated for either *B* or *A* water level measurements, did not give reasonable estimates about the ability to store water. The range of results varies a lot. The maximum value calculated from water level *A* measurement was 385 % bigger than the smallest one. The results, starting points, and changes in reservoir levels are presented in Table 4.

Table 4: *Results of the test runs. Starting points are presented in centimeters from below the max height in the environmental permit.*

	Level A storing capacity factor	Level B storing capacity factor	Level A starting point	Change in level A	Level B starting point	Change in level B
Test run 1	22.8	33.6	-19	-16	-15	-11
Test run 2	25.3	35.7	-16	-11	-12	-8
Test run 3	17.3	24.3	-10	14	-6	11
Test run 4	22.2	32.2	-13	10	-14	7
Test run 5	27.6	36.4	-10	7	-14	6
Test run 6	29.9	36.4	-6	8	-4	7
Test run 7	18.8	28.1	-11	8	-9	5
Test run 8	24.3	40.7	-16	-7	-14	-5
Test run 9	10.7	16.1	-21	26	-7	15
Test run 10	8.5	17.6	-66	19	-4	9
Test run 11	7.7	24.8	-58	24	-3	8

The results vary a lot. Portrayal and data analyzes from storing capacity factor results extremes, test runs 11 and 6, are presented as following. In test run 11, the result of storing capacity was small compared to the test run 6. The portrayal of the test run 11 is presented in Figure 20 and storing capacity analyze in Figure 21.

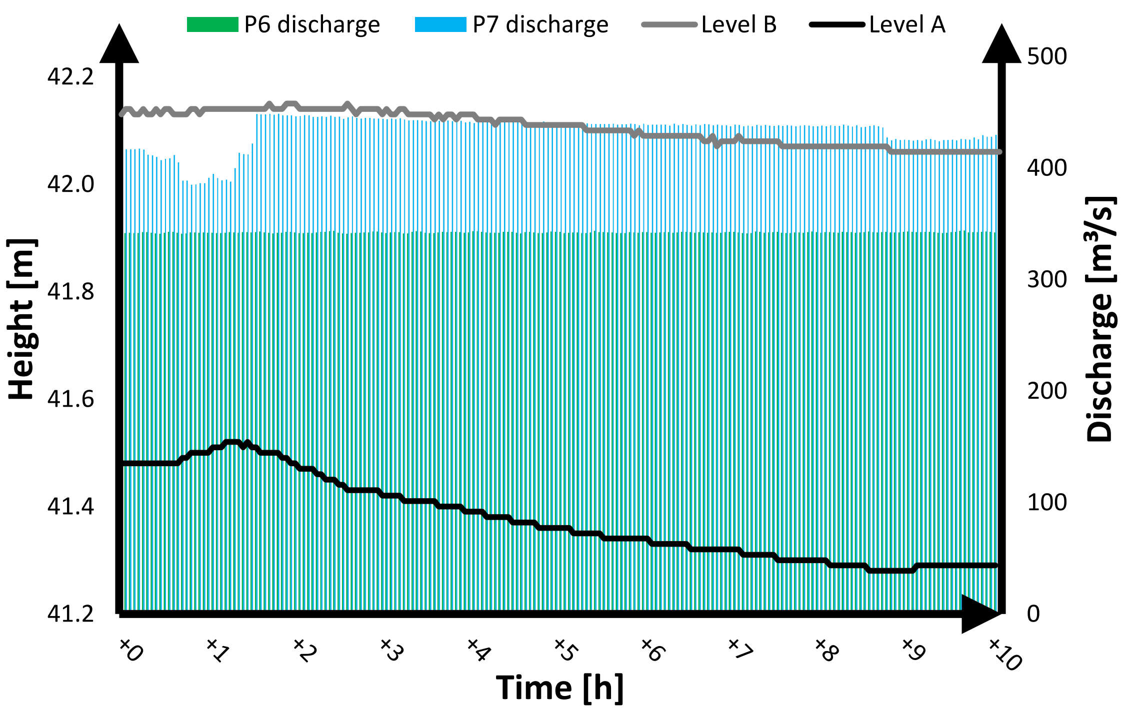

Figure 20: *The portrayal of storing capacity test run 11. The flow in the river was high during the test run and the intake level was not exactly stable before the step increase was performed. During the high flow of the river, the stable discharge is hard to find out.*

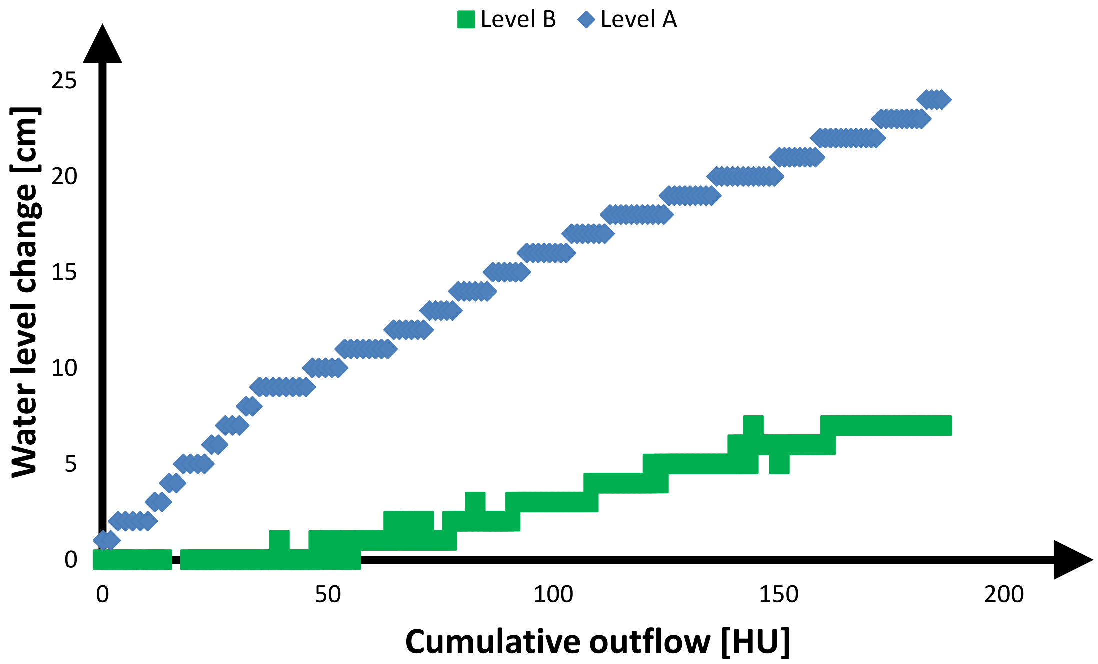

Figure 21: *Cumulative water level change and outflow of the test run 11. Cumulative outflow is shown as hourly cubic meters.*

The change in water level *A* is greater than in water level *B*, thus the calculated storing capacity factor of water level *B* is higher than from water level *A*. This can be explained mainly due to higher influence of discharge change in water level *A*. As mentioned in section 3.2.1, the result of storing capacity factor is an average from ten last calculated values. Figures 22 and 23 presents the portrayal and storing capacity analyzes of the test run 6, in which results were highly different than test run 11.

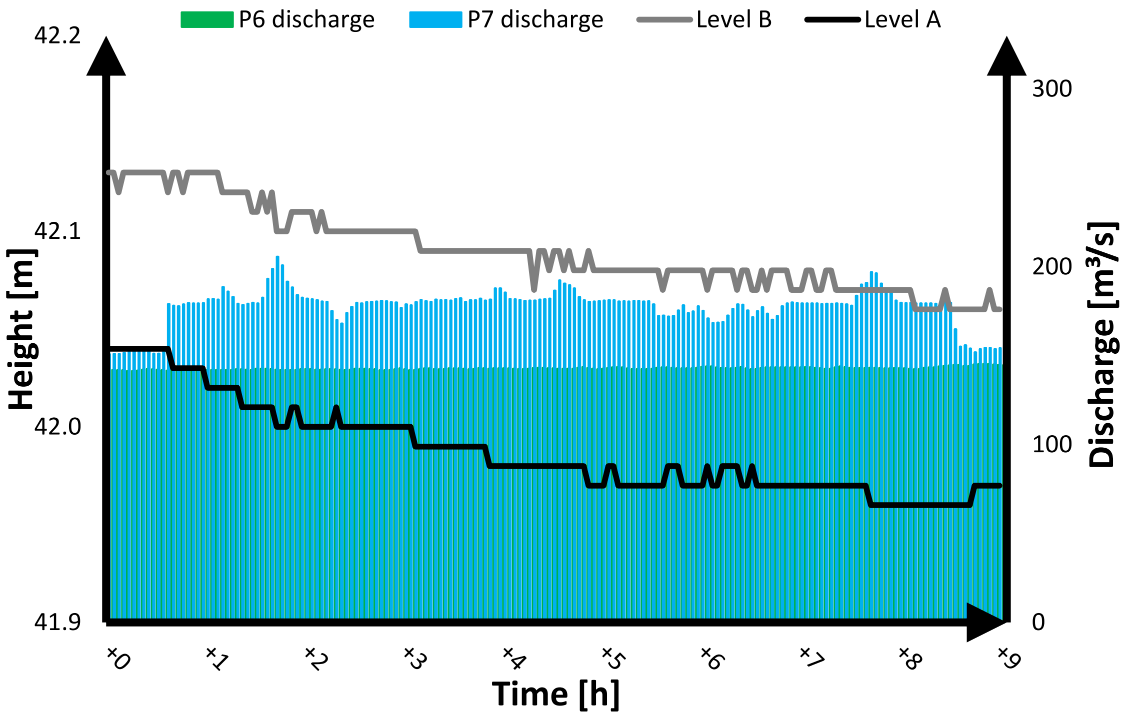

Figure 22: *Test run 6 portrayal. The discharge of P6 was less than half compared to the test run 11.*

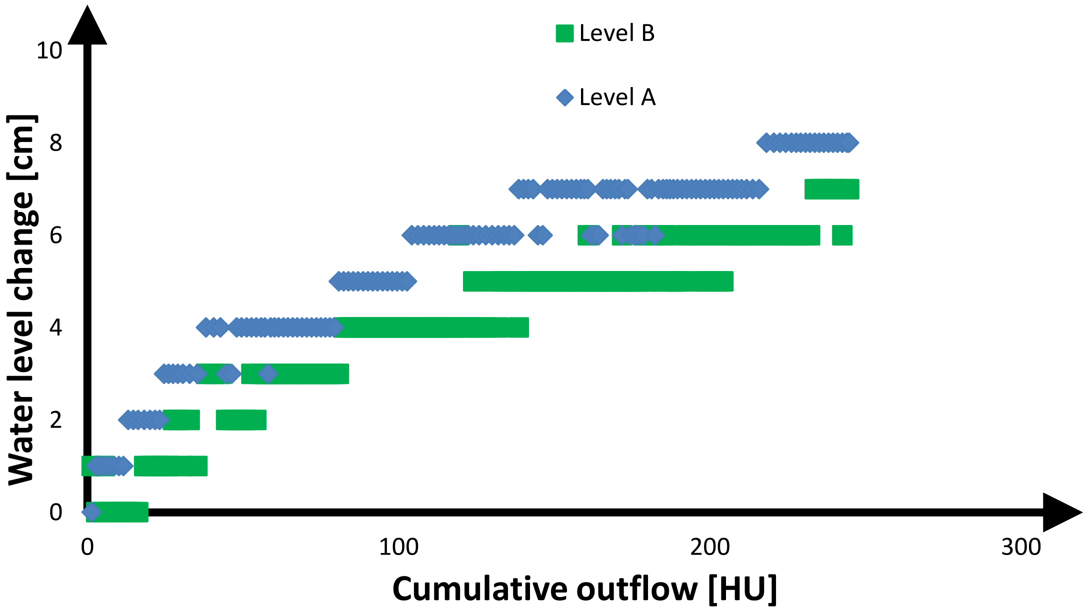

Figure 23: *Cumulative water level change and outflow of the test run 6. Cumulative outflow is shown as hourly cubic meters.*

As seen from Table 4 and Figure 24, the results of storing capacity tests *P7* are not constant and the result of each test varies a lot. When the results were gathered together, a negative correlation between storing ability and discharge was found. According to the test results, the storing capacity seems to be dependent on discharge level: while the stable discharges during tests were higher, the storing capacity factors were smaller and vice versa.

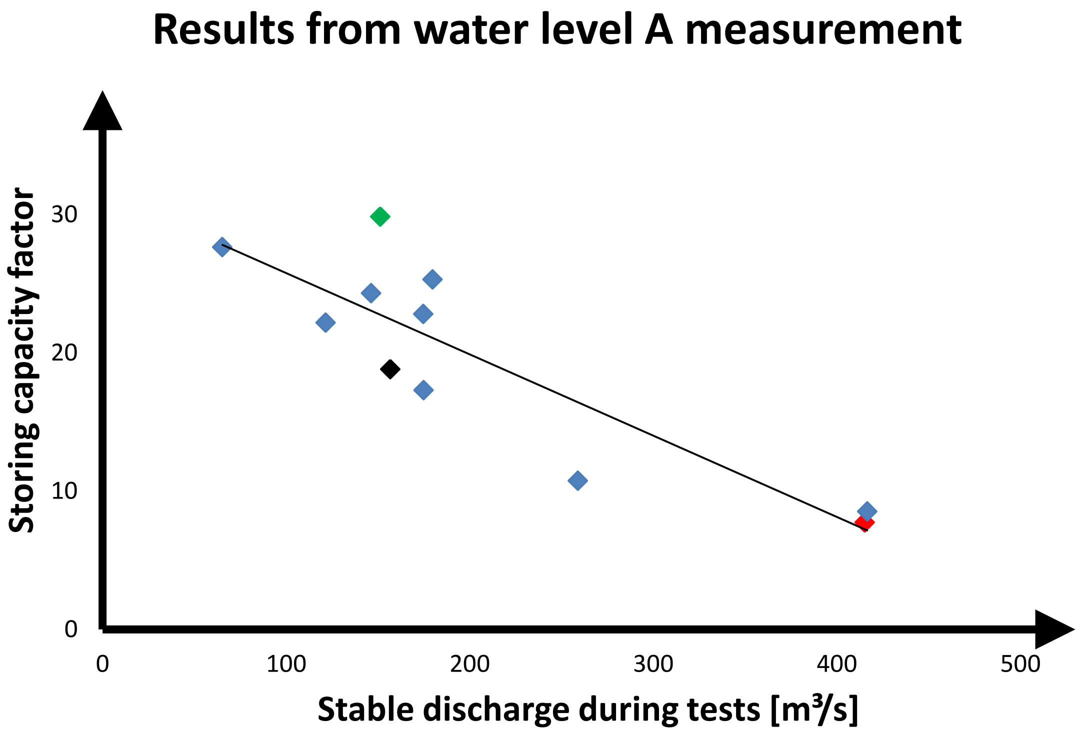

Figure 24: *The test results of storing capacity volume in a function of a stable discharge during tests. The results are calculated from intake water level measurement. The added trend line is presented in Table 5. Black marker is the result of test run presented in Figure 19, red marker from test the run 11 (Figure 20), green marker from the test run 6 (Figure 23), and blue ones represents the results from other tests.*

This phenomenon occurred also from the test results which were calculated from water level *B* measurement.

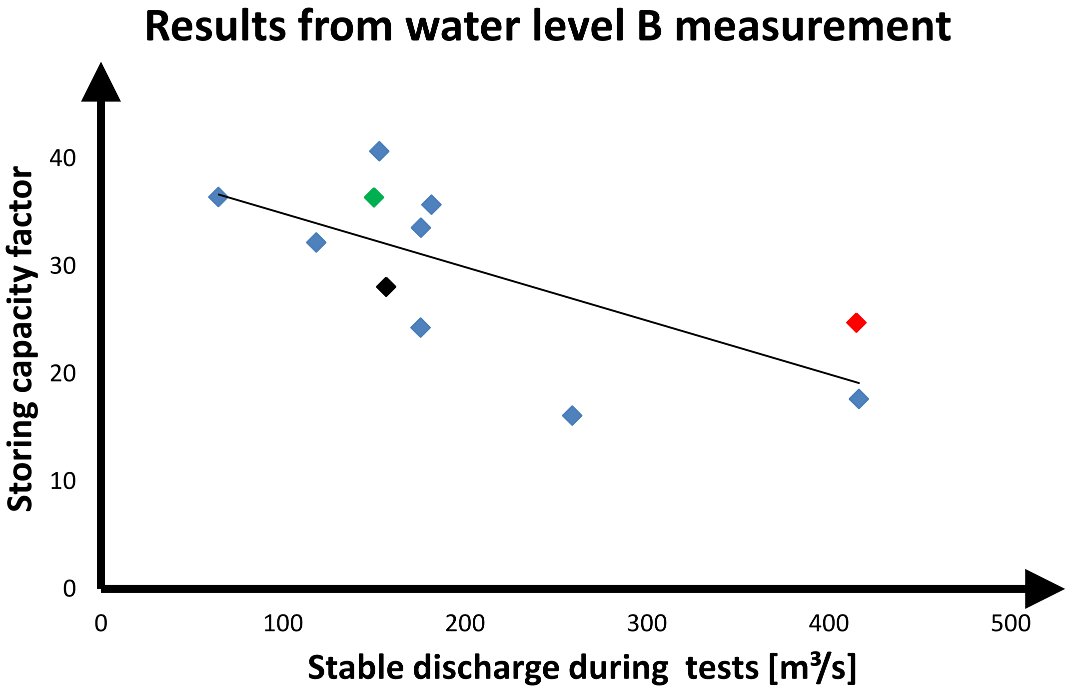

Figure 25: *The test results of storing capacity volume calculated from water level B measurement in a function of a stable discharge during tests. The added trend line is presented in Table 6. Black marker is the result of test run presented in Figure 17, red marker from test the run 11 (Figure 21), green marker from the test run 6 (Figure 22), and blue ones represents the results from other tests..*

In both test results, loci of data points can be seen. In Figure 24, the locus of data points is centered near the rule-of-thumb value used in operative work before thesis. The three smallest values calculated from intake level measurement were not even in the same scale compared to the estimated reservoir size before the tests.

From the test results, there could create a trend line between discharge of *P7* and storing capacity factor. The trend line was made with Microsoft Excel by utilizing regression. The trend line statistics from water level *A* is presented in Table 5 and from water level *B* in Table 6.

Table 5: *Water level measurement A regression statistics from fitted trend lines.*

Level A measurement	***Linear***	$R^2 = 0.7598$		***Second order***	$R^2 = 0.7657$
	x	*Intercept*	x^2	x	*Intercept*
Trend line	-5.880E-02	31.6565	5.5162E-03	-8.7808E-02	34.6405
T-stat	-5.3350	12.3535	0.4529	-1.3490	4.8688
P-value	4.7159E-04	6.0104E-07	0.6627	0.2143	1.2418E-03

Second order trend line variables x^2 and x have big P-values so those are statistically insignificant. Thus, it is reasonable to use linear trend line.

Table 6: *Water level measurement B regression statistics from fitted trend lines in test runs data.*

Level A measurement	***Linear***	$R^2 = 0.4895$		***Second order***	$R^2 = 0.5188$
	x	*Intercept*	x^2	x	*Intercept*
Trend line	-4.9717-02	39.8581	1.2607E-02	-0.1159	46.6636
T-stat	-2.9378	10.1120	0.6969	-1.2004	4.4127
P-value	1.6545E-02	3.2612E-06	0.5056	0.2643	2.2483E-03

The regression results show, that there is statistically significant possibility that the storage coefficient does change with respect to the average flow rate of the river. Moreover the value of the storage coefficient decreases with higher flows. This phenomenon is demonstrated in Figure 26, which shows the estimated storage coefficient as a function of average discharge level of *P7*. The figure also shows the relative size of the linear estimate against the rule-of-thumb value used in the prior forecasting tools.

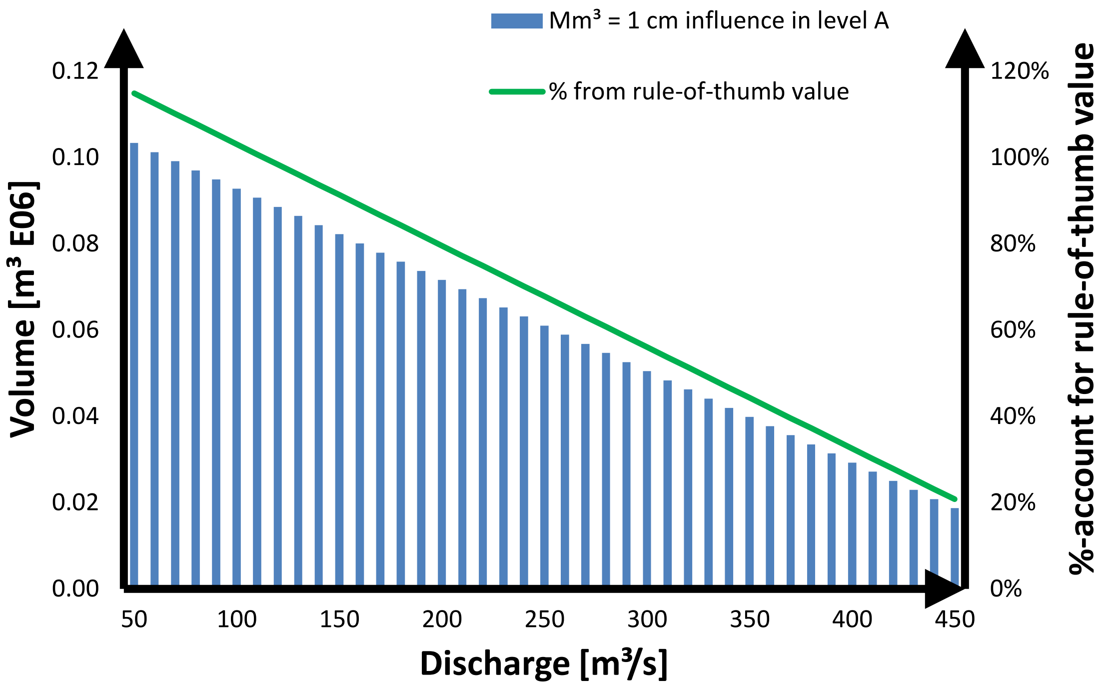

Figure 26: *The amount of water needed to 1 cm change in intake water level. The modelled storing capacity percentage account from rule-of-thumb value is shown in right-hand y-axis.*

For example, if the hydropower plants discharge is 110 $m^3/_s$, the remainder between inflow and outflow should be 90700 m^3 or 25 *HU* that it would cause 1 cm change in water level *A*. If the discharge is 190 $m^3/_s$, the remainder should be only 73700 m^3 or 20 *HU*. The difference between storing capacities is not very big, but it still is 23 % from the water mass. What is also visible from Figure 26 is that the rule-of-thumb value is approximately correct only for small average flow.

During the time the storing capacity tests were performed, a relation between the height difference of water levels *A* and *B* was spotted. The difference between these two water levels was higher when the discharge of *P7* was higher. The relation is clear. The difference between water level *A* and *B*, presented in Figure 27, depends highly from discharge, as clearly seen from Figure 27.

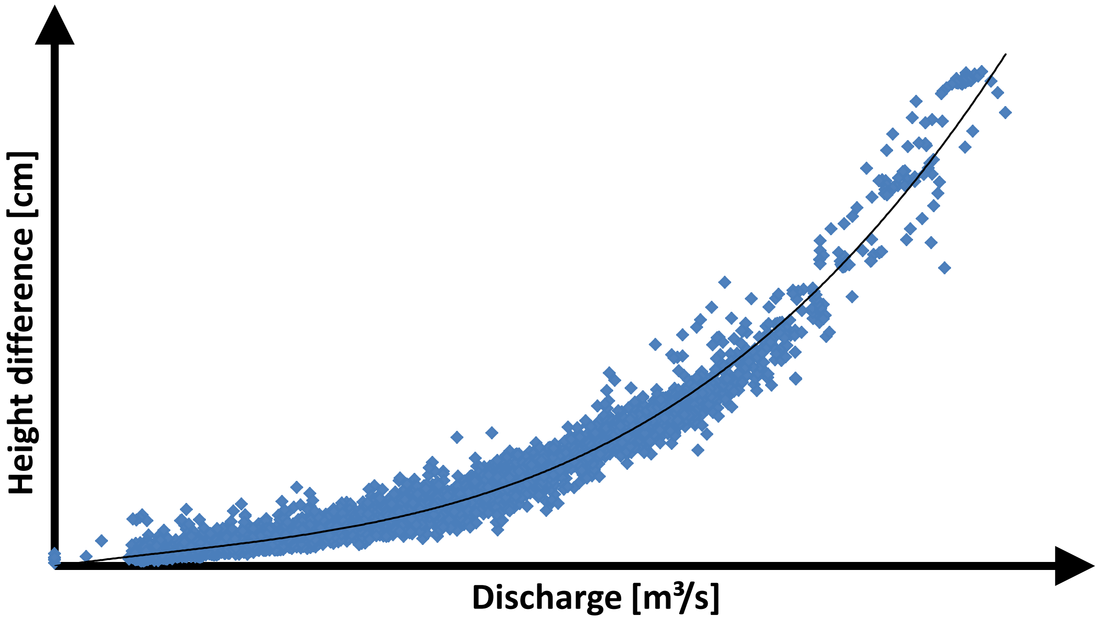

Figure 27: *The difference of water level B and A with a function of discharge. The figure shows a third-degree polynomial trend line in with R^2-value of 0.9874.*

However, the difference between these water levels does not increase or decrease immediately right after discharge is raised or reduced. It takes awhile before reaching a new equilibrium. The intake level reacts more powerfully to changes due to temporary blockage or emptying of water in hydropower plants intake. This is mainly due to slower changes in river bed channel. In addition, water level *B* does not react immediately and in a same way as water level *A*, because the influence caused by discharge change moves along slowly to further upstream. The temporary state after discharge change cannot be modelled with an adapted function. The behavior of intake water level right after discharge change is presented further in Figure 37.

Because the influence of discharge changes does not occur immediately like the adapted model assumes, a smoothed value was taken in use. The smoothed value is calculated as in equation (7) and smoothing parameter α is restricted as same way as in equation (8). In equation (7), $S_f(t)$ is simply weighted average from current hour modelled in adapted function value $x(t)$ and previous hour smoothed value $S_f(t-1)$. The smoothing parameter was calculated with solver by utilizing hourly historical data. There was 11745 hours in data and the data was only taken between April and November, in other words, ice related issues were omitted. The best fit was given with smoothing parameter 0.42527. In addition, with the smoothed value, the height difference of the level *A* and level *B* was modelled as a function of two, three and four hours discharge average of *P7* and weighted average of three hours.

Because the plants permit is confined to both, water level *A* and *B*, the comprehension of both water levels behavior is important. The water level *B* maximum limit is 0.07 m

higher than level *A* maximum limit, thus after certain discharge level, instead of water level *A*, the water level *B* becomes a limiting factor, which generally limits the intake level to lower than its maximum level is.

The results of the step-response test were hypothetical, but the correlation between discharge and storing capacity were much higher than expected. In reality, the storing content of the water level *A* cannot vary according to discharge. The fluctuating storing capacity factor just models the observable behavior better than a constant value. In reality, the storing capacity might vary according to water level, because the reservoir might be shaped like semi-circular. This can be studied with depth sounding. In case of semi-circular shaped storing content, it is sensible that fluctuating storing capacity models the intake level behavior better: during higher discharge, the water level *A* is on lower level because the maximum limit of the water level *B* reduces it.

4.2 Water delays

The solving method of water delay is presented in section 3.2.2. To solve the delay profile, the inflow to hydropower unit *P7* was calculated from hydropower plant *P5* discharge and run-offs forecasts or run-offs calculated from tests. There were three main questions before the test results were analyzed:

1. How long it takes to see whole share of water arriving to *P7*?
2. In which hour the first influence of arriving water is seen?
3. When the peak is reached?

The portrayal of one performed water delay test run is shown in Figure 28.

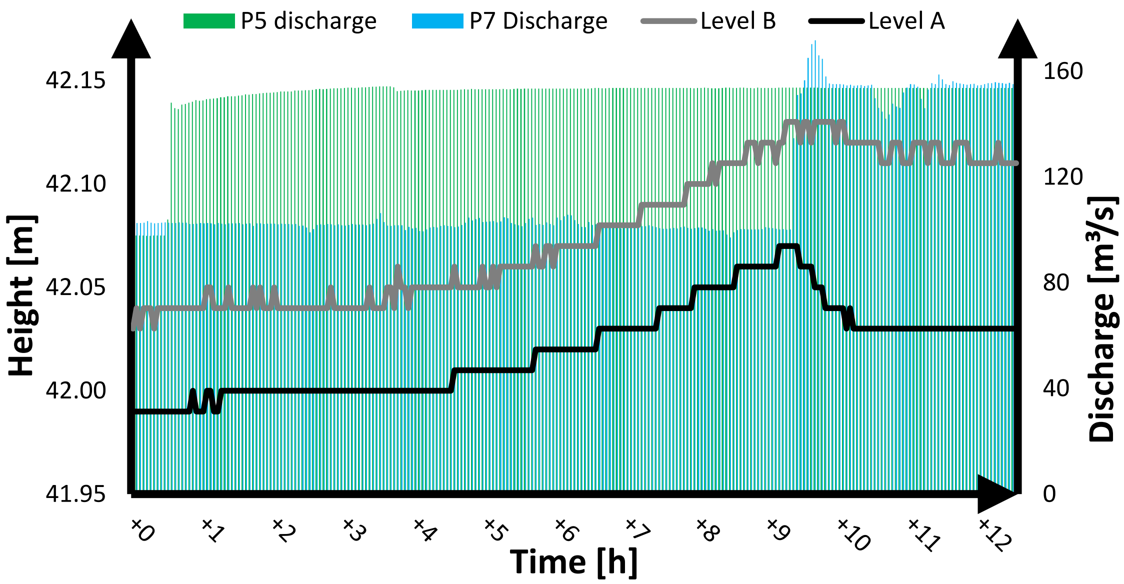

Figure 28: *The portrayal of the water delay test runs. The influence can be seen first in water level B. The discharge of P7 is increased during hour +9 because water level B*

approaches near its maximum level. The discharge of P5 has been stable more than 14 hours before the step increase was made. The time span of this Figure is 12.5 hours.

The data of test runs presented in Figure 28 shows that the influence of increased discharge in hydropower unit *P5* can be seen in *P7* water levels within a delay. Water level *B*, which located 11 km to upstream from *P7*, reacts naturally earlier to increased discharge. During this test run, first changes in water level *A* occurs four hour after and in water level *B* three hours after. This knowledge can be utilized in delay profile calculations when constraints (18) and (19) were set.

When the tests were analyzed first time, a set of sixteen variables was used, in other words, the presumption was that the water delay is divided into next sixteen hours including the releasing hour from *P5*. From the data of one test, the first influence caused by stepped increase in *P5* was seen in water level *B* measurement after three hours and ten minutes and in the intake level measurements after four hours. Utilizing this information, additional constraints can be fed to the solver. In other words, hours $t + 0$ until $t + 2$ can be restricted to zero. Also, the presumption was that all the water released from *P5* is arriving to hydropower unit *P7*, so the sum of water delays should be approximately one.

The results given by sixteen variable set were not satisfactory. The presumption from shape of the water delay was, that it will increase started from hour $t + 2$, until it will reach a peak, and after the peak, it will start to decrease until the value zero is reached. The solver did not give realistic values and the results were not consistent with the physics of water movement in an open channel. Adding constraints to solver was problematic, because the timing of peak was unknown. Also, the different constraints just only moved the values to the limits given by constraints. The optimal solution of one test run is shown in Figure 29. There were no constraints added in the optimal solution visualized in Figure 29.

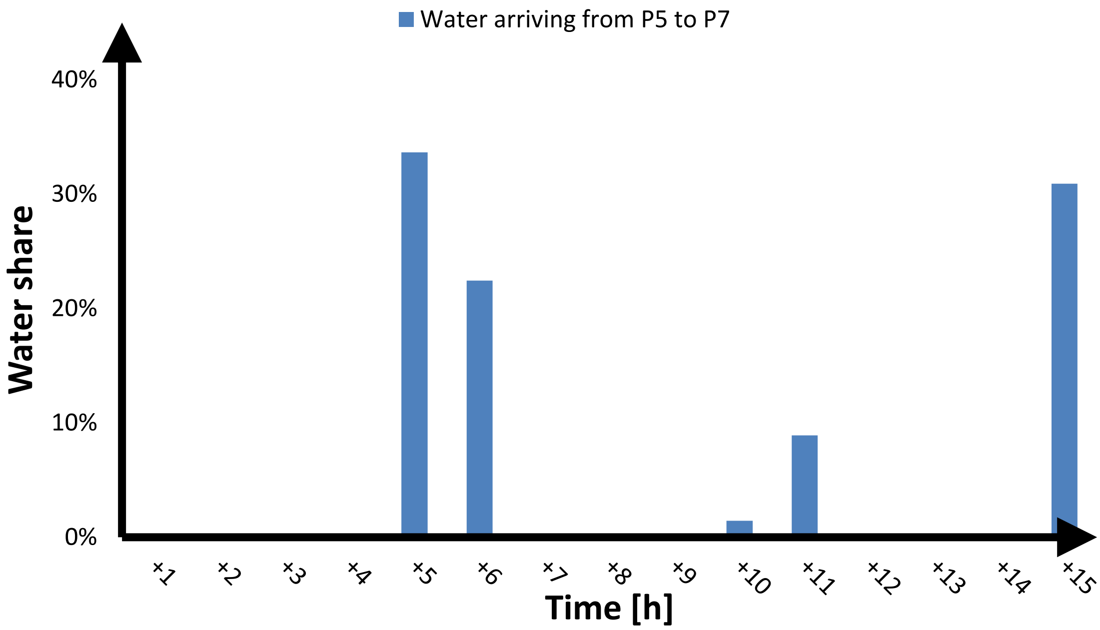

Figure 29: *The optimal solution of water delay profile of one test run. The optimal solution is not supported by physics of water movement in open channel. The storing capacity is modelled as a constant in this solution. The solution where the storing capacity is modelled as in a function of discharge does not deviate from above presented, because during delay test runs the discharge of P7 is approximately constant. The fluctuating storing capacity value during this test run was approximately same as a rule-of-thumb value.*

The water delay profile should to be in shape of a wave because it supports the physics of water movement, and thus the gamma-distribution was taken into use. The pros of the gamma-distribution were that the shape would be wave-liked and there are only two parameters to change: shape parameter α and scale parameter β. In addition, the minimum of a non-linear problem is easier to solve with two variables than with sixteen variable. Because of the wanted delay profile shape, and characteristics of gamma-distribution, it was not possible to restrict first four hours to value zero. Thus, the constraints (18) and (19) are not restricted to zero.

Eighteen data sets were analyzed and fitted with gamma-distribution. The time period of analyzed data series were 114 hours. The results are presented in Figure 30.

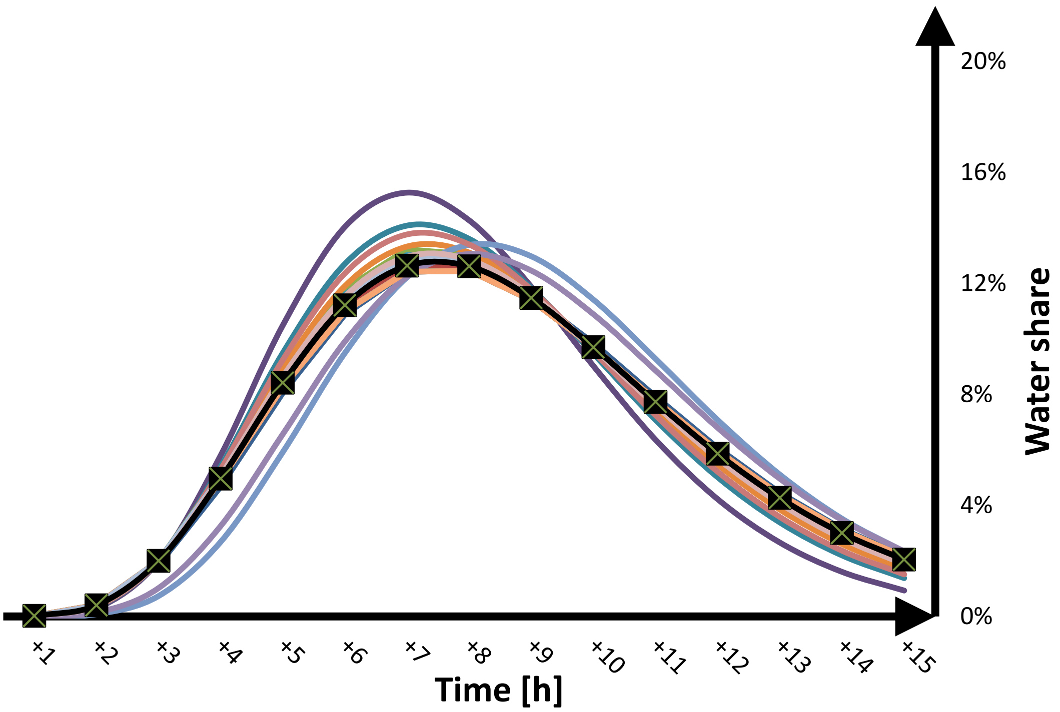

Figure 30: *The results of shard water delay tests. The black line with markers represents common solution of all data series.*

Figure 30 shows that water delays fitted with utilizing gamma-distribution are acceptable in common ground. Additionally, majority of solutions reaches a peak in same hour. Cumulative water arriving, in other words, stream flow routing curve is presented in Figure 31.

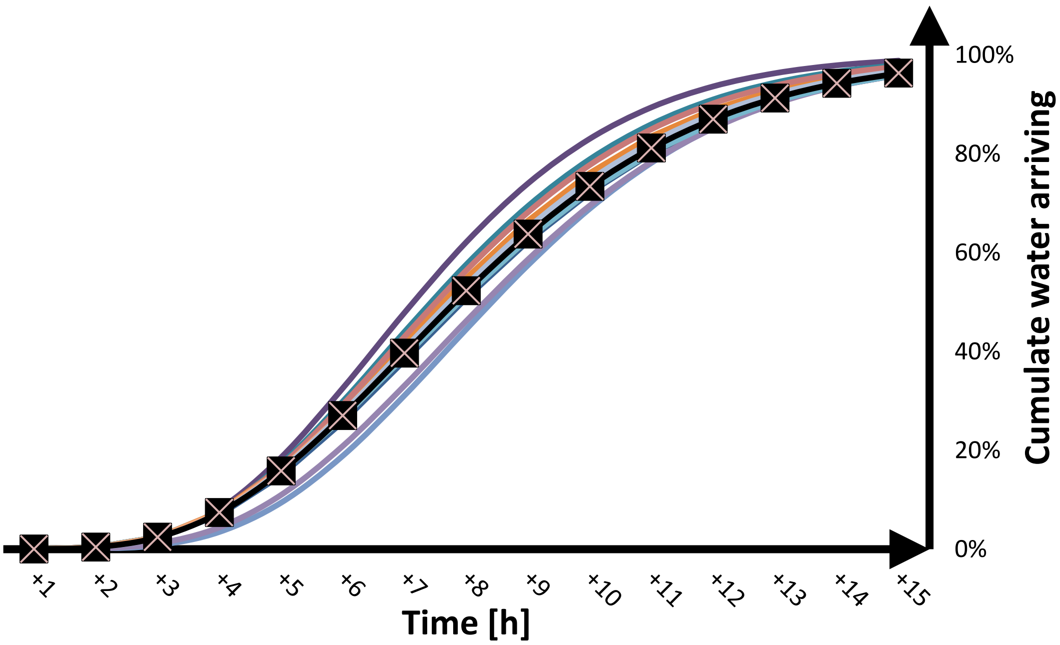

Figure 31: *Stream flow routing curve. The black line with markers represents common solution of all data series.*

Regardless of constraint (17), none of the cumulative water delays rise above value 1. In $\frac{7}{19}$ cases the bottom boundary of constraint (17) were limiting the cumulative sum of water delay. In addition, most of data series were close from bottom boundary. This could mean, that run-off forecasts are usually too high, or the discharge measurement of *P5* is too high, or discharge measurement of *P7* is too low. The alternative possibility is that there was too short water delay time used in solving, but cumulative sum of arriving water was no different with longer water delay.

When the delays were calculated, the storing capacity of *P7* was modelled with the rule-of-thumb value. In addition, the common solution of all data sets with different value of storing capacity factor and without constraints was calculated. The storing capacity was modelled with four different values: the smallest and the highest values acquired from storing capacity tests, with the rule-of-thumb-value and with a fluctuating storing capacity factor according to linear trend line in a function of a discharge. The solutions are presented in Figure 32.

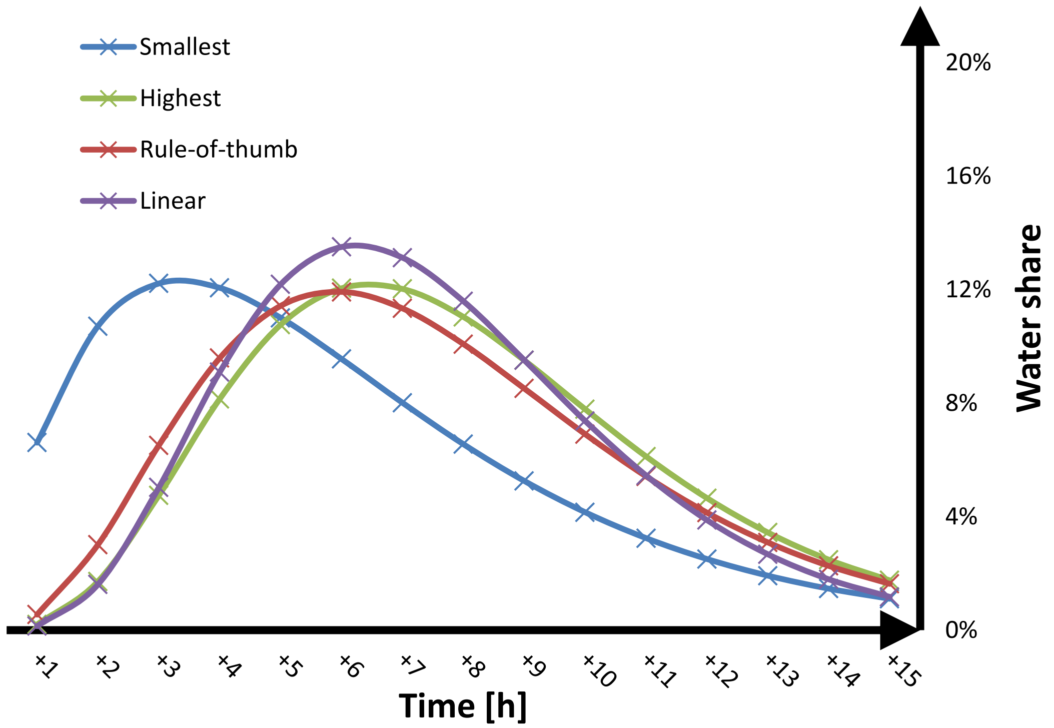

Figure 32: *The common solution of eighteen data sets with different storing capacity factors.*

The solutions with different storing capacity factor values are similar one to another, excluding the solution where the storing capacity of *P7* was modelled with the smallest value taken from the storing capacity tests. This delay profile can be totally ignored. Water level *A* prediction with this delay profile and small storing capacity factor was mostly poor in the data sets. The intake level reacts too strongly with small storing capacity factor.

The data series were divided by average discharge of *P5* through the analyzed time period. From some of the solutions, the shape of water delay was smoother when the average discharge of *P5* was higher. In other words, the peak value of delay profile was on lower level and the water arriving to *P7* was divided more smoothly. Thus, the delay profile was made dynamic and it changes as a function of *P5* average discharge. Only two different delay profiles were used, which form the borders to the dynamic delay profile. Because there are only two variables in gamma distribution, the dynamic profile variables differs between α_{min} and α_{max} and between β_{min} and β_{max}. The interpolation of both values is linear with a function of *P5* average discharge through eighteen hour time span.

As mentioned before, there is difference between two different water delays cumulative values and this also occurs in dynamic delay profile. Figure 33 shows a stream flow routing curves from *P5* to *P7* with three different water delay profiles when the simulated step increases from *P5*.

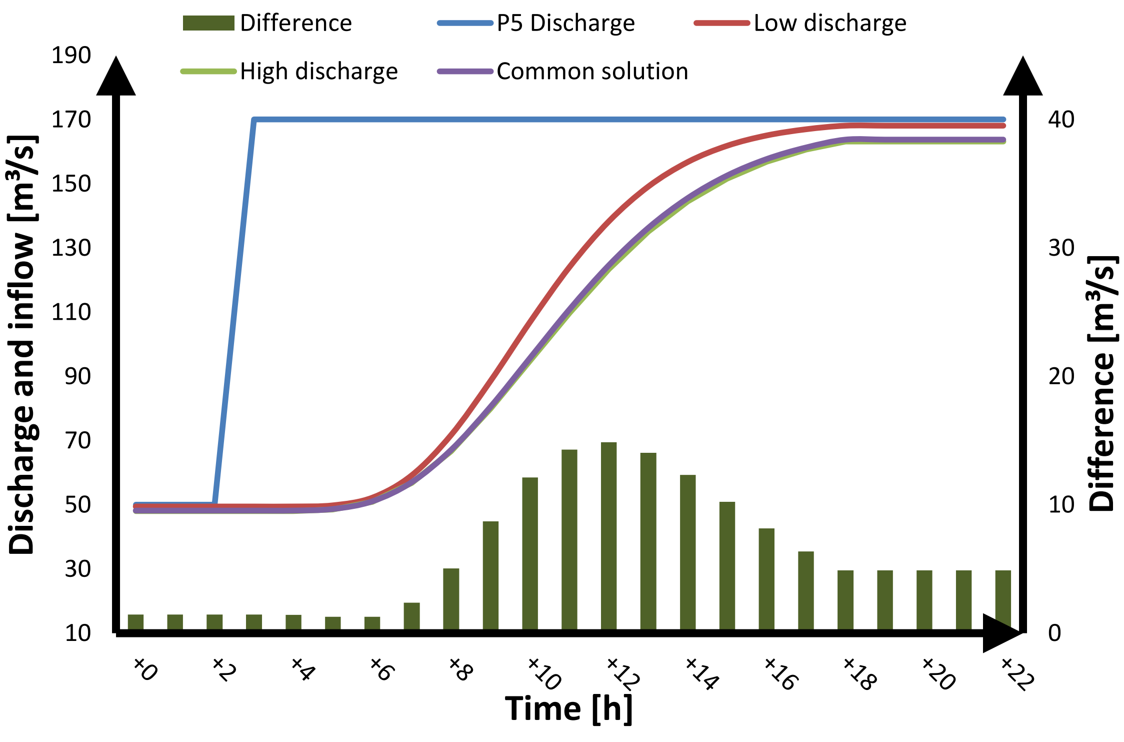

Figure 33: *The difference in arriving water by using three different stream flow routing curves. The separation shown in figure is between high and low discharge average of P5.*

In this 22 hour comparison time span the difference caused by cumulated value difference between water delays during high and low discharge of *P5* could cause 5.7 cm difference in intake level of *P7*, when the storing volume is modelled as a constant. The simulated water arriving from *P5* to *P7* can be seen in Figure 34.

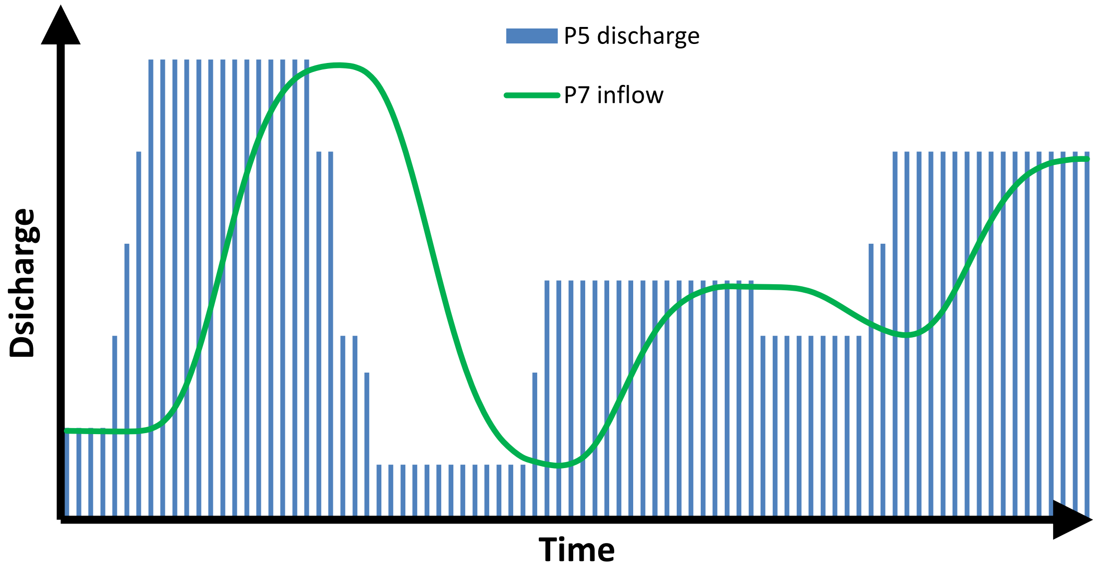

Figure 34: *Hydropower unit P5 discharge modelled arriving to P7.*

4.3 Planning tool

The planning tool was made by utilizing results given from this study. The planning tool predicts hydropower plants *P7* water level *A* and *B* to 81 hours forward starting from current hour. The calculation of planning tool was implemented with Microsoft Excel and it was inserted into the energy management system. The planning tool utilizes realized values of *P5* and *P7* discharges and the latest *P7* intake and tail water level measurements. These are taken from the energy management system and the values are updated every third minute. Discharge plans are used for future water level calculation. Because hydropower plants *P7* and *P8* are highly hydraulically coupled to each other, also the *P8* intake level calculation was inserted to planning tool. This will facilitate whole hydropower system controlling and planning, because hydropower units cannot be planned separately. The intake level calculation of *P8* was taken from previously used planning tool without any modifications.

4.3.1 Planning tool objectives

The main objective of the planning tool is to forecast the behavior of water level measurements *A* and *B* of hydropower unit *P7*. Better water level forecasting accuracy should especially simplify the river planning and reduce the need for manual control during high discharges or flood, which are the most challenging times in the river operation. The river controlling during challenging circumstances is laborious and time-consuming for the operator and well-working forecasting tool would save operator’s time. The saving of the operator's time would enable the operator to attend to other issues. In addition, better water level forecasting would also help the operator and the planner to sell hydropower plants production more market-based in both, day-ahead and intraday markets.

However, the planning tool behavior and accuracy will be tested experimentally first and if the results are satisfactory, parts of the planning tool calculation could be utilized in the river production planning optimization. The day-ahead planning optimization is also built on top of Excel and thus the calculation is quite simple to copy in to the optimization logic.

4.3.2 Parameters

The planning tool is a result from models which try to illustrate actual water behavior in the river system. The models are physical adapter models which are created from historical data or data gave from the planned water system tests. In the following examples, the forecasting tool uses modelled inflow and realized discharge of *P7*. The modelled inflow is sum from delayed discharge of *P5* and run-off forecast. The delayed discharge of *P5* is presented section 4.2. The water level *A* storing capacity is modelled with a linear trend line as a function of discharge and it is smoothed by using the average value from last four hours. Examples of the water level *A* predictions can be seen in Figures 35 to 45. The good and worse predictions are shown.

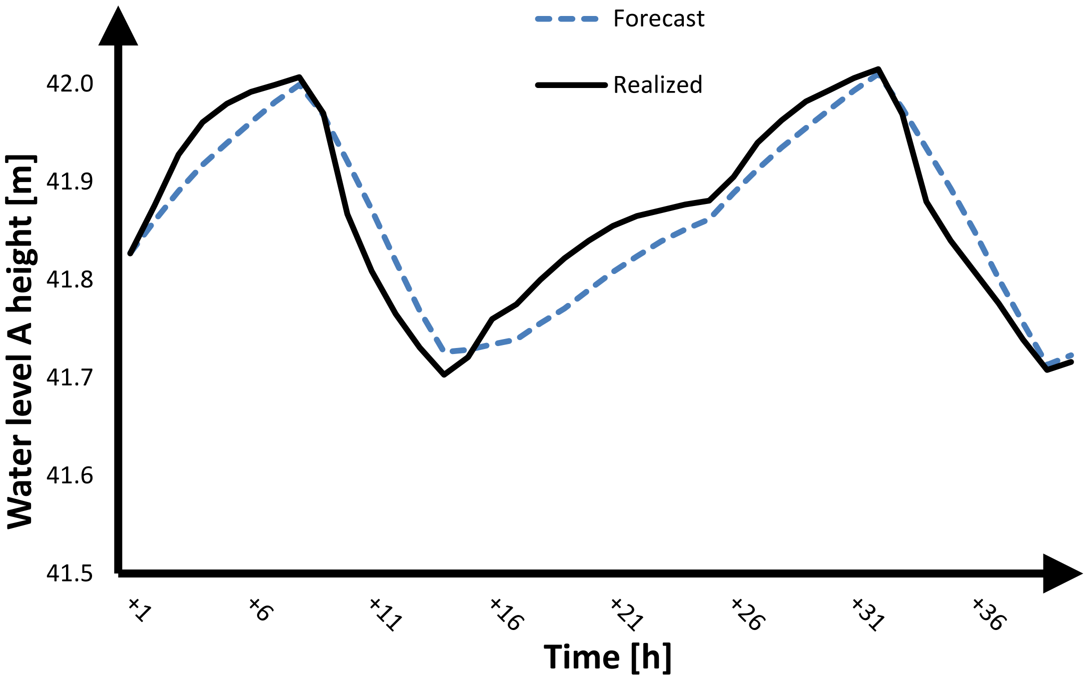

Figure 35: *An example of the forecasting tool prediction. The forecast is in good agreement with the realized values. The SSE of this prediction is 0.0448. The time span in this figure is 40 hours.*

The prediction presented in Figure 35 is good. The forecasting tool predicted the levels and timing of peaks well and there is not remarkable error between peaks. In addition, the direction of prediction is always equal to realized water level and there is no wandering off. During this time span, the circumstances in the river were normally and the average discharge of *P7* was approximately 160 $^{m^3}/_{s}$. An example of a failed forecast is presented in Figure 36.

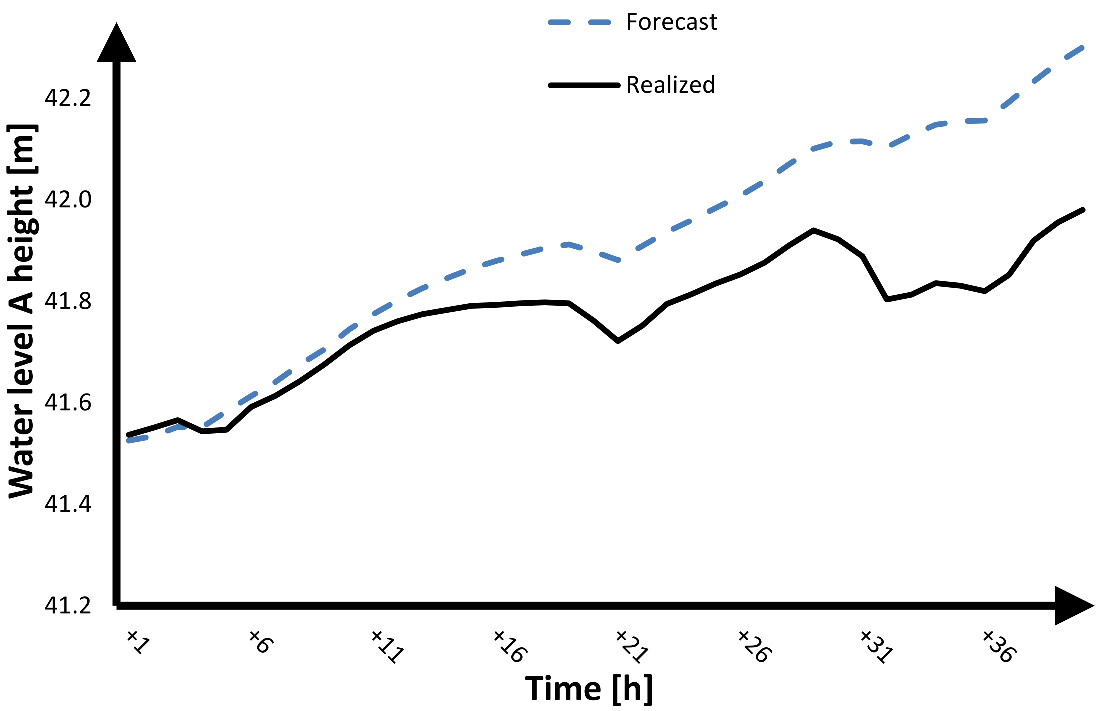

Figure 36: *A failed prediction of the forecasting tool. The SSE of this prediction is 1.304. The time span is 40 hours.*

The prediction presented in Figure 36 is bad because it wanders off very badly. Wandering off can be due to discharge measurement of *P5* and run-off forecasts. In addition, the size of changes downwards in water level *A* are underestimated.

In the latest configuration of the forecasting tool, the change in water level *A* due discharge changes is modelled. Thus the forecasted intake level is sum from the reservoir level and discharge change influence to the intake level. The adaptor is shown in Figure 37.

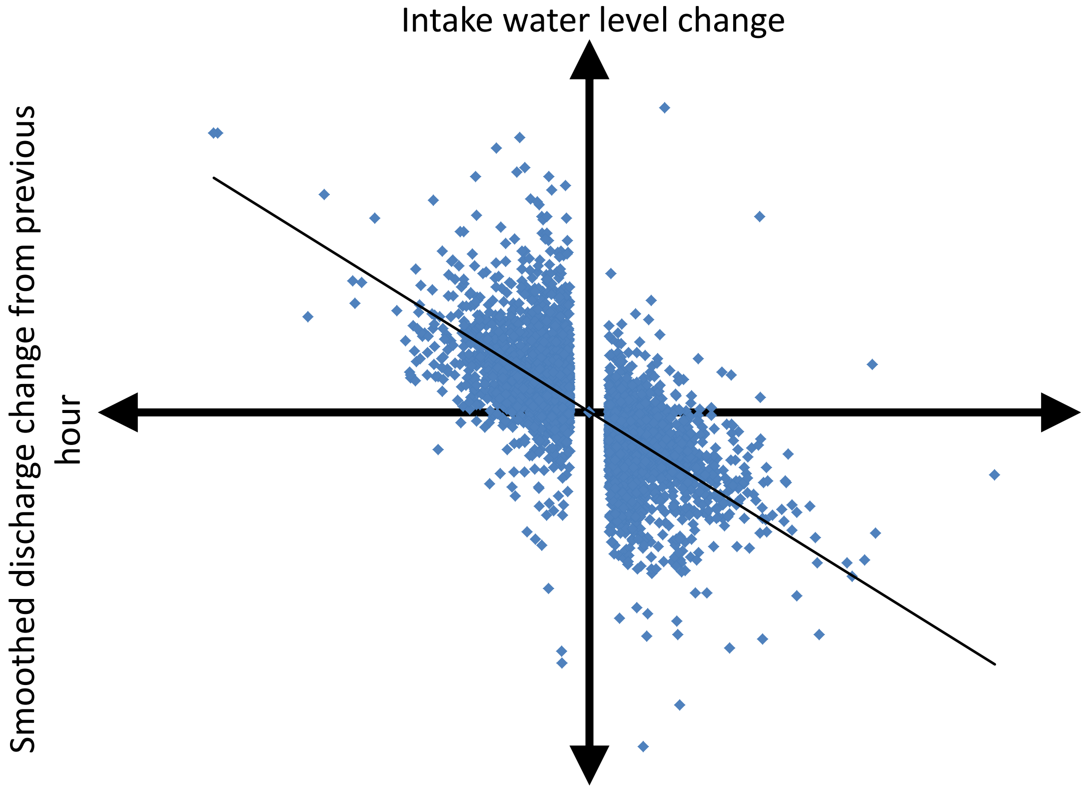

Figure 37: *The sudden changes in hydropower plants intake immediately after discharge changes. Fewer than 10* m^3/s *discharge changes are ignored from data. The discharge is smoothed from previous hour and current hour similar as in equation (7) with smoothing factor 0.2. There is added trend line with* R^2*-value 0.4533.*

Experimental knowledge shows that using the adapted function as it is, was not working. Using function as it is causes too strong fluctuation compared to real affects in hydropower plants intake level. The problem was solved with smoothing the influence of the function. The smoothing was performed with exponential smoothing with using smoothing factor 0.2. Exponential smoothing inhibits the fluctuating and smooth the effect of trend line function responding more realized influence. In predictions, presented in Figures 35 and 36, the sudden change due discharge changes was not taken account. In Figures 38 and 39, there are shown predictions when change in water level *A* due discharge change is included.

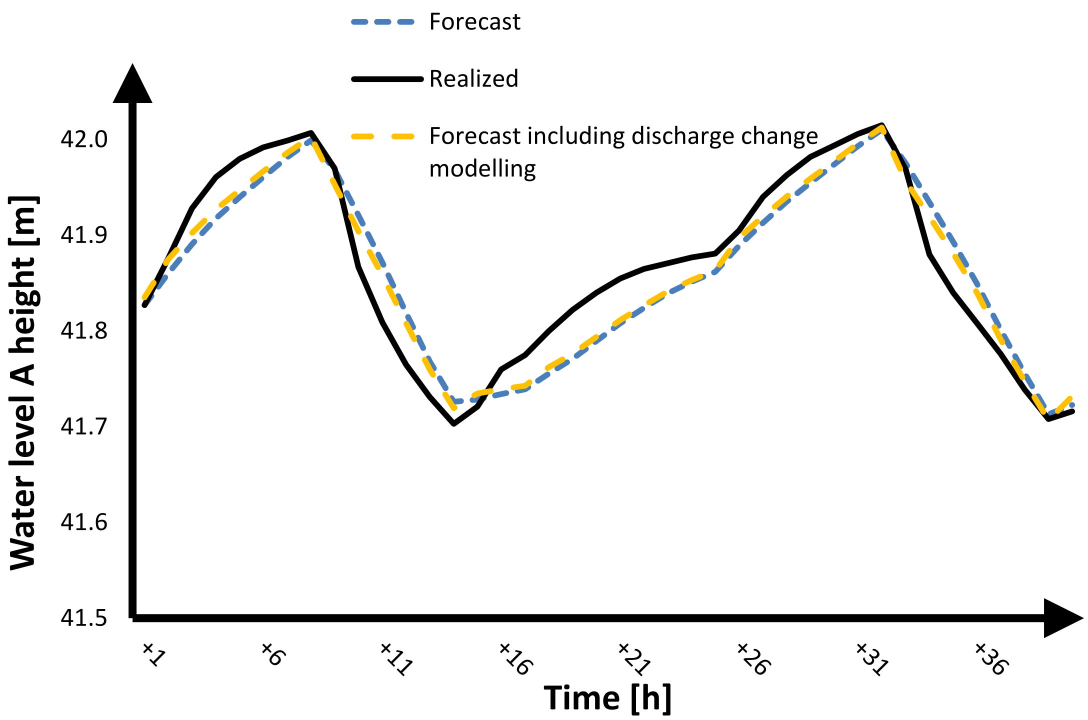

Figure 38: *The forecast including discharge change modelling. The SSE is 0.0301.*

The modelling of discharge changes does not change the behavior of the forecast fundamentally. The main idea in discharge change influences modelling was fix the error which appears after the peak or bottom is reached. As seen from Figure 38, the water level *A* increase or decrease after peak or bottom more than model shows. In Figure 39 there is shown a failed prediction with a discharge change influence modelling.

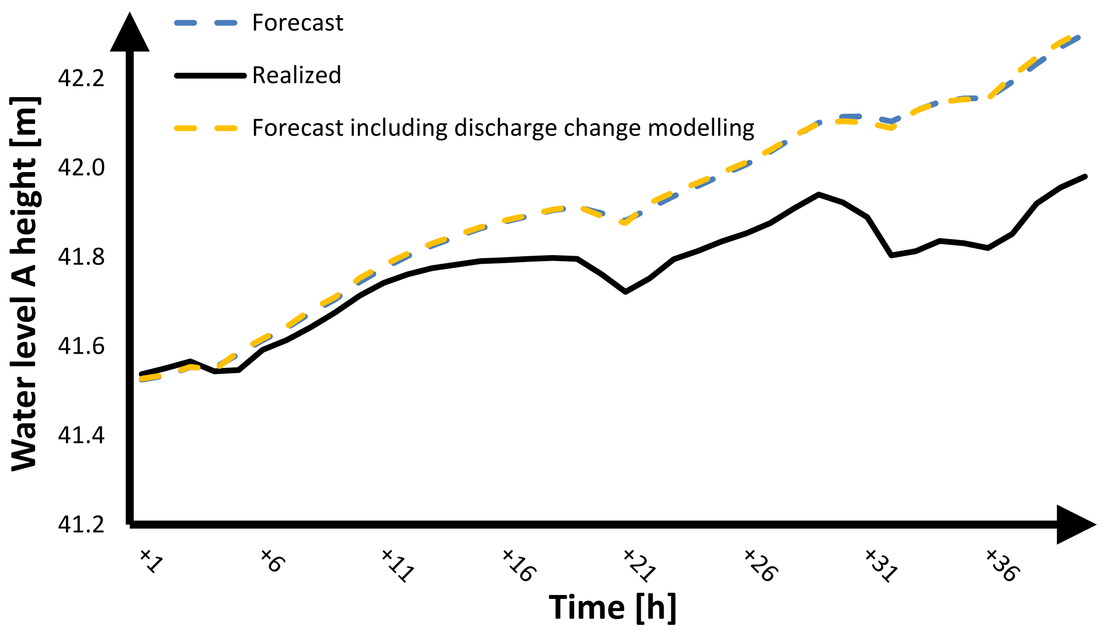

Figure 39: *A failed prediction of the forecasting tool. The SSE of this prediction is 1.328. The time span is 40 hours.*

The difference between predictions with and without the discharge change influence modelling would be bigger, if the calculation worked differently. Now, the current calculation works so that the water level *A* reservoir content is modelled separately, and after that the influence of discharge changes is summed to it. To clarify, the next hour calculations are started from previous hour reservoir content instead of modelled intake level. Additionally, the influences of discharge changes are highly smoothed to avoiding fluctuating in water level *A* predictions which reduce the modelled influence of discharge change even more.

In real forecasting process, the planned discharge is in use instead of realized discharge. Or more accurately, a calculated discharge is used, because the discharge is a derived value in hydropower units. Thus, the discharges used in the planning tool are calculated values, not exact values. The hydropower unit *P7* is operated by planned energy instead of planned discharge. According to equation (6), the efficiency curves and tail water were modelled in the planning tool, which enables the water requirement modelling. Tail water modelling and calculations presented in section 2.3.3 are adapted into the planning tool as such. The combined efficiency curve of hydropower plant *P7* is presented in Figure 40.

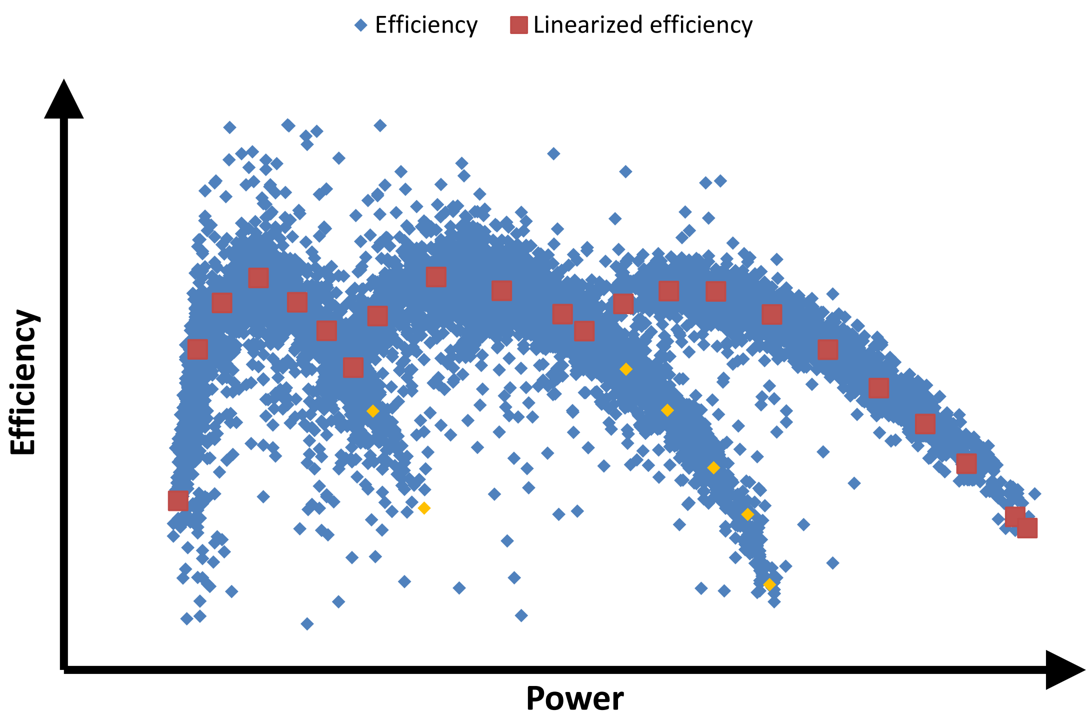

Figure 40: *The efficiency curve of hydropower unit P7. Data points with red color are in use in linearization. Yellow marked data points present unfavorable operation which forecasting tool is not taken account.*

The efficiency curve is linearized to be operable in planning tool. The data points used in linearization is marked with red color in Figure 40. The efficiencies of generator units 1 and 2 are modelled as equal, but generation unit 3 is different. The modelled water requirement replaces the discharge plans in planning tool and it is calculated like in equation (6). It uses energy forecasts, intake and tail water level forecasts and linearized efficiency curve presented in Figure 40. The combined efficiency curve in the planning tool assumes certain operating order of generators, which can generate error in water requirement modelling. In addition, the linearized efficiency curve does not take account unfavorable operation, for example divergent operating order or times when two generators are operating close to their maximum power and the efficiency is worse than operating with three generators. This causes error in amount of water needed which causes error in forecasted water levels. Also, part of the error comes from the linearization.

The forecasting tool prediction which calculates the discharge from planned energy is shown in Figure 41.

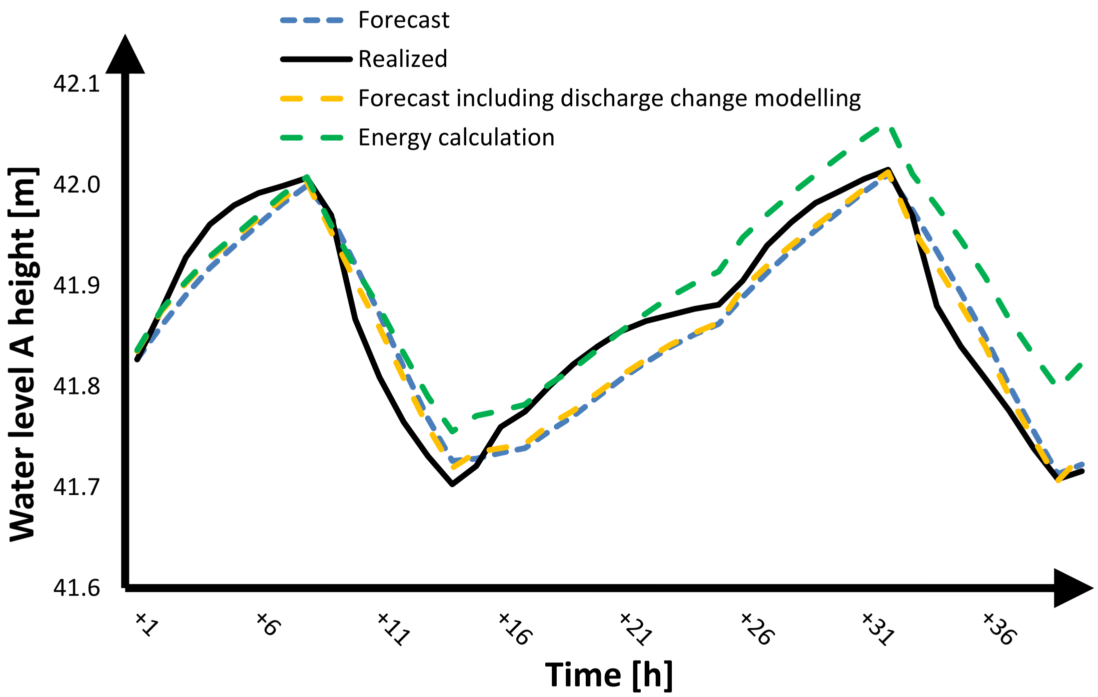

Figure 41: *The forecasting tool predictions with three divergent modelling. The SSE of the forecasting tool with energy calculation is 0.1049.*

In this data set, the energy calculation makes the forecasts worse. The failed forecast of forecasting tool with energy calculation can be seen in Figure 42.

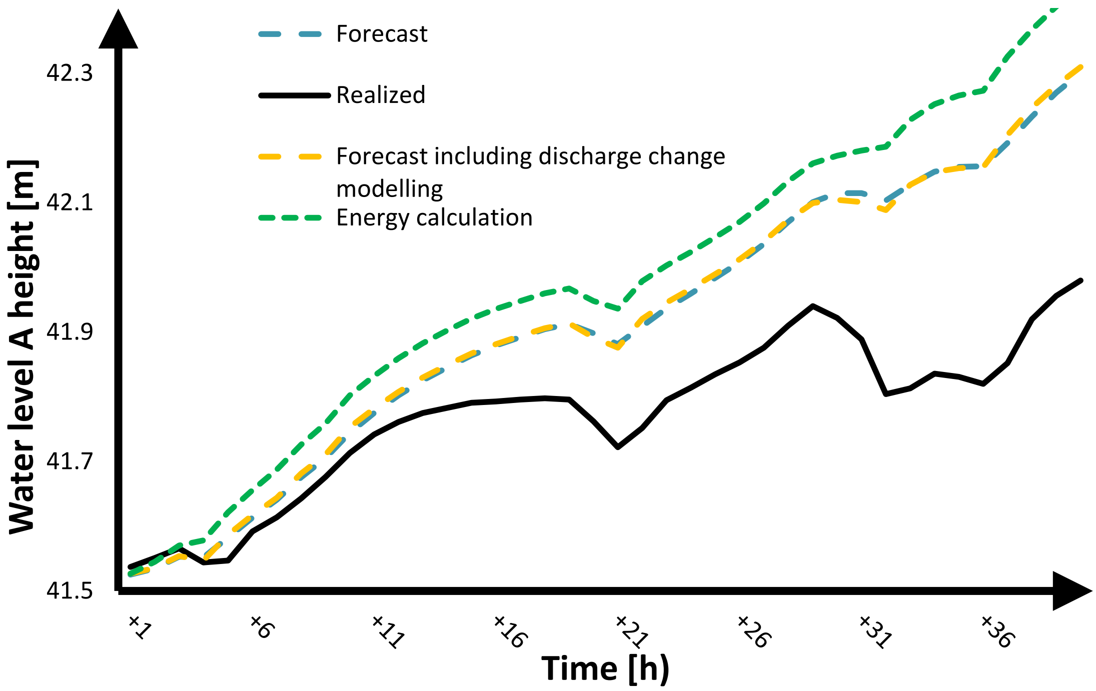

Figure 42: *All the forecasts, regardless used calculation, failed this data set forecasting. The SSE of forecast made with energy calculation is 2.2522. This is mainly due modelled discharge: it was systematically lower than realized which causes the higher forecast of water level A predicting.*

In Summary, the planning tool is a result based on physically adapted models from water level *A* storage volume, water level difference function according to water levels *A* and *B* of *P7*, discharge changes effects in *P7*, stream flow routing curve from *P5* to *P7*, and *P7* water requirement to produced energy which planned. It uses the latest realized values. The planning tool adaptors contain smoothing, because the changes in river system are not as clear as the adaptors assume.

4.3.3 Forecasting tool results and comparison

Besides the forecasting tool *A* created in this study, there are two different water levels forecasting tools in use. First one, forecasting tool *B*, is in use in river production day-ahead optimization and planning. Second one, forecasting tool *C* is more in use on intraday desk, but also manually on day-ahead production planning. Forecasting tool *C* is not used in optimization. During this thesis, another forecasting model, planning tool *D*, was made, which takes a totally different approach, because it is not based on physical modelling of water movement. The planning tool *D* is a so called “black box” model, which utilizes historical data and machine learning. The black box is a computer program where users enter information (e.g. historical data of water levels and discharges) and system utilizes pre-programmed logic to return output (e.g. water level forecasts) to the user.

These four tools were compared to one another with thirteen 40 hours' time span data series in the planning tool comparison. The data series were chosen at random. The difference in comparison to real process of future water levels forecasting, realized discharges were used instead of planned discharges with each planning tool. For comparison of planning tools, common scale-depended measures were chosen. These are useful for comparing methods on one data series with are in same scales. Methods are presented as following:

- Sum of Squared Errors (SSE)
- Mean Absolute Error (MAE)

Also, a maximal error was recorded, which descripts the maximum error of forecasted and realized water levels, which can be considered critical measurement according to water level forecasting. To comparison each planning tool to created planning tool in this study, there are also the ratio of two planning tools squared error in use. This descripts which of planning tools forecasted data series better measured with SSE.

Because there are forecasts of run-offs use in river production planning, it is apparent that it causes error in the water level modelling. In addition, there are no regular comparison according to realized and forecasted run-offs, thus profound analyzes and better forecasting models for run-offs is not created.

Sum of squared errors of each forecasting tool are presented in Table 7. The best forecasting model is bolded.

Table 7: *Sum of squared errors of each planning tool.*

Sum of squared errors

	FT A	FT B	FT C	FT D
Day 1	1.956	0.032	0.097	**0.016**
Day 2	**0.023**	0.144	0.084	0.039
Day 3	1.550	0.822	1.087	**0.474**
Day 4	0.076	0.861	**0.059**	0.165
Day 5	0.224	1.109	**0.069**	0.081
Day 6	0.032	0.403	0.057	**0.030**
Day 7	0.054	0.756	0.070	**0.013**
Day 8	0.039	0.544	0.074	**0.007**
Day 9	0.123	0.524	**0.079**	0.106
Day 10	**0.033**	0.211	0.093	0.040
Day 11	**0.013**	0.034	0.081	0.016
Day 12	0.067	0.741	0.089	**0.011**
Day 13	**0.060**	0.258	0.802	0.715

The different forecasting tool predictions during Day 10 are shown in Figure 43.

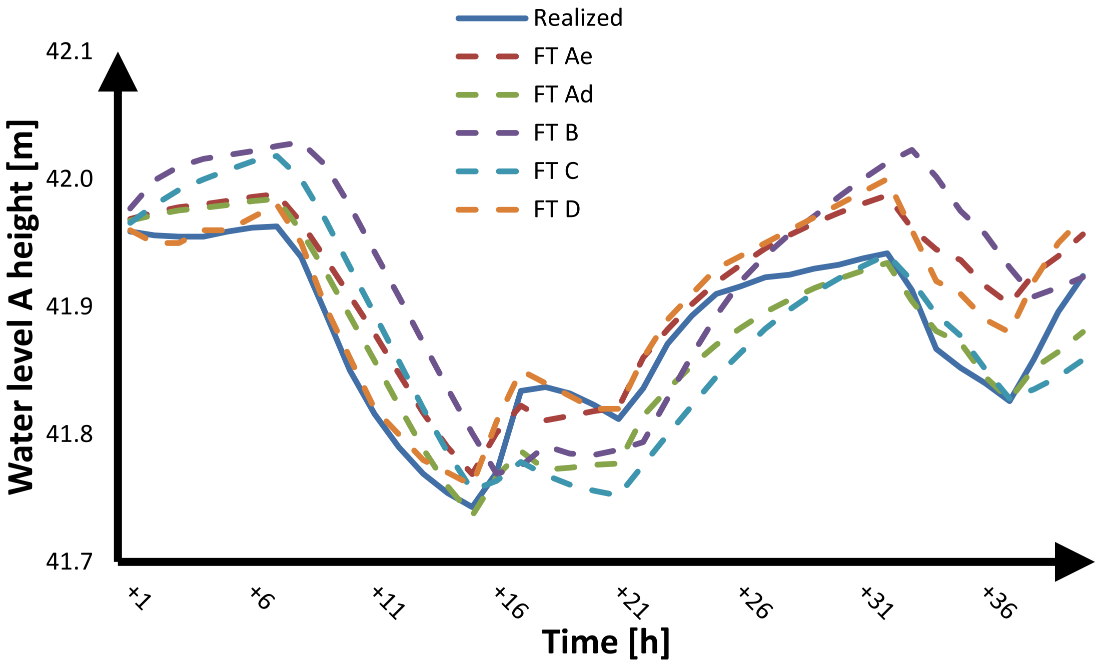

Figure 43: *Forecasts during Day 10. FT A_E is with an energy calculation and FT A_D is with a discharge calculation.*

The average from absolute differences between forecasted and realized water levels, in other words, mean absolute error of each forecasting tool through the compared time span can be seen in Table 8.

Table 8: *Mean average errors of each forecasting tool.*

MAE

	FT A	FT B	FT C	FT D
Day 1	20.16	2.22	4.53	**1.68**
Day 2	**2.04**	5.61	3.82	2.70
Day 3	16.00	11.11	15.75	**9.08**
Day 4	3.59	12.66	**3.14**	5.39
Day 5	7.11	15.44	3.48	**3.43**
Day 6	2.39	8.63	2.91	**2.34**
Day 7	2.92	11.87	3.27	**1.47**
Day 8	2.74	9.09	3.99	**1.00**
Day 9	5.12	9.99	3.86	**4.02**
Day 10	**2.44**	6.18	4.19	2.46
Day 11	1.55	2.51	3.93	**1.50**
Day 12	3.82	12.67	3.95	**1.45**
Day 13	**3.01**	7.09	11.68	11.52

The maximum difference between forecasted and realized water level are shown in Table 9.

Table 9: *The maximum error in centimeters of each forecasting tool due the comparison time span.*

MAX error

	FT A	FT B	FT C	FT D
Day 1	31.79	6.30	7.06	**4.10**
Day 2	**4.73**	8.90	8.66	5.60
Day 3	37.36	31.00	24.49	**22.80**
Day 4	7.84	26.10	**7.35**	13.10
Day 5	11.12	29.10	**8.76**	10.30
Day 6	5.30	18.30	8.08	**4.90**
Day 7	7.37	25.00	8.05	**3.30**
Day 8	5.74	23.40	6.43	**3.60**
Day 9	9.90	19.70	**7.64**	12.40
Day 10	6.47	13.50	8.11	**6.10**
Day 11	4.04	5.50	9.15	**4.00**
Day 12	6.70	21.30	9.20	**3.30**
Day 13	**13.24**	13.80	26.54	22.70

Table 10 shows the sum of squared errors ratio between forecasting tool *A* and other forecasting tools. If the value is less than 1, the forecasting tool created from results of this study has forecasted water levels better than other planning tool.

Table 10: *Forecasting tool A comparison to other forecasting tools with ratio of sum of squared errors.*

Ratio of sum of squared errors

	$^{\text{FT A}}/_{\text{FT B}}$	$^{\text{FT A}}/_{\text{FT C}}$	$^{\text{FT A}}/_{\text{FT D}}$
Day 1	61.768	20.126	119.938
Day 2	**0.161**	**0.276**	**0.591**
Day 3	1.886	1.426	3.273
Day 4	**0.089**	1.288	**0.463**
Day 5	**0.202**	3.247	2.777
Day 6	**0.080**	**0.564**	1.086
Day 7	**0.072**	**0.775**	4.246
Day 8	**0.072**	**0.527**	5.786
Day 9	**0.235**	1.567	1.162
Day 10	**0.158**	**0.358**	**0.837**
Day 11	**0.392**	**0.162**	**0.837**
Day 12	**0.091**	**0.758**	6.074
Day 13	**0.233**	**0.075**	**0.084**

As seen from Table 10, regardless the same input values, the forecasting tool *A* has failed forecasting Day 1 very badly. During the Day 1, there was a very high flow in the river. The error was not caused by errors in run-offs forecasts, because other planning tools do not give same kind of error. Those are also using same run-off forecasts. Possible error could cause by the discharge measurement of *P5*, because all the other planning tools use the discharge of *P6* in inflow calculations to *P7*. The discharge measurement of *P5* is systematically higher than discharge measurement of *P6*. The difference between hydropower units *P5* and *P6* water discharge measurements increases while discharge increase. In addition, all the other planning tools use constant water storing capacity factor which are remarkable bigger than fluctuating storing capacity factor of planning tool A during Day 1 high inflow. The different forecasting tools predictions during Day 1 are presented in Figure 44. Figure 44 includes also the forecasting tool *A* with energy calculation, which comparison is presented later in this section.

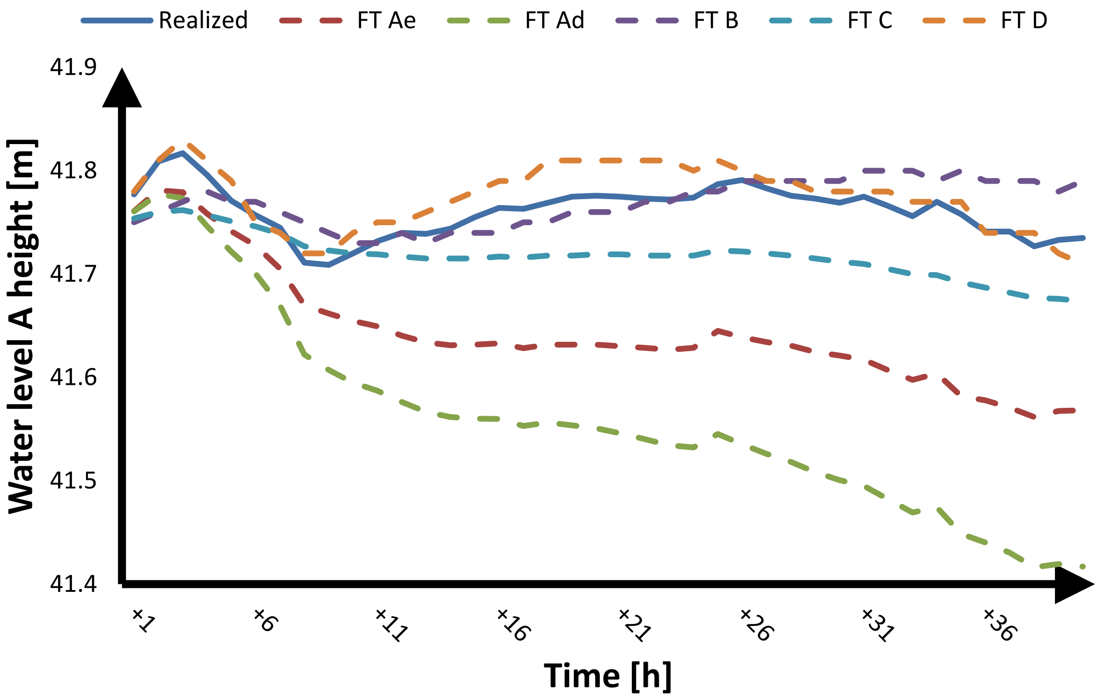

Figure 44: *Different forecasting tools Day 1 forecasts.*

The error between realized and forecasted water level *A* affects to plant's head, which in turn affects to received energy. If the difference in hydropower plant's head between realized water level and water level forecasted with $FT\ A_D$ is converted to energy, the modelled energy in Figure 44 time span would be 1.73 % higher when realized water level is used. Realized tail water and linearized efficiency curve was in use in comparison.

The latest configuration of forecasting tool *A* starts predicting from planned or realized energy which is noteworthy difference to other forecasting tools. The latest configuration from forecasting tool *A* was also tested with same comparison time spans and the results are presented in Table 11.

Table 11: *Forecasting tool A results with energy calculation in same comparison time spans.*

Forecasting tool A with energy calculation

	SSE	MAE	Max error
Day 1	0.653	11.744	17.612
Day 2	0.043	2.929	5.570
Day 3	3.298	24.252	52.478
Day 4	0.368	8.334	17.487
Day 5	0.976	14.425	24.225
Day 6	0.132	5.022	10.774
Day 7	0.288	7.404	15.341
Day 8	0.205	5.809	14.227
Day 9	0.029	2.150	6.585
Day 10	0.067	3.510	8.475
Day 11	0.006	1.100	2.254
Day 12	0.152	5.750	10.052
Day 13	0.486	8.927	23.102

Measured with SSE, the results of forecasting tool *A* with energy calculation was worse than with discharge calculation. The forecasting tool with energy calculation was better at predicting water levels in three days from thirteen than planning tool with discharges. The result was expected, because the error increases because of modelled efficiency curves and the tail water. The forecasting tool *A* with energy calculations was compared with all the other forecasting tools with ratio of SSE, including also forecasting tool *A* with discharge calculation. The results are shown in Table 12.

Table 12: *The SSE ratio of FT* $\mathrm{A_E}$ *compared other forecasting tools.*

Ratio of sum of squared errors

	$\mathrm{FT\ A_E}/\mathrm{FT\ A_D}$	$\mathrm{FT\ A_E}/\mathrm{FT\ B}$	$\mathrm{FT\ A_E}/\mathrm{FT\ C}$	$\mathrm{FT\ A_E}/\mathrm{FT\ D}$
Day 1	**0.334**	20.613	6.717	40.026
Day 2	1.863	**0.300**	**0.515**	1.100
Day 3	2.128	4.014	3.035	6.965
Day 4	4.810	0.427	6.194	2.225
Day 5	4.351	**0.880**	14.126	12.083
Day 6	4.092	**0.328**	2.307	4.443
Day 7	5.321	**0.381**	4.123	22.593
Day 8	5.233	**0.377**	2.757	30.281
Day 9	**0.239**	**0.056**	**0.374**	**0.278**
Day 10	2.009	**0.318**	**0.719**	1.680
Day 11	**0.452**	**0.177**	**0.073**	**0.378**
Day 12	2.259	**0.205**	1.714	13.722
Day 13	8.095	1.882	**0.606**	**0.680**

During Day 1, Day 9, and Day 11 the forecasting tool with energy calculation forecasted the intake level behavior better than forecasting tool with discharges. This cannot be explained by different discharge levels in the river during comparison time span, because during these three days average discharges in the river differ substantially from each other. During Day 1, the error in efficiency curve can compensate the error caused by discharge measurement of *P5*. In other words, the discharge calculated from efficiency curve in those power output areas is smaller than realized discharges, which eliminates the error in *P5* discharge measurements error in during high flow. During Day 9, the forecasting tool A_E was sufficient in predicting water levels. Additionally, the discharges in the river varied a lot through this time span, which makes forecasting water levels more challenging. There was no remarkable error between realized and calculated *P7* discharge during Day 9 which means, that the errors in tail water modelling cancelled out the errors caused by efficiency curves. All the forecasting tools had predicted hydropower units *P7* intake level behavior precisely during Day 11. During Day 11, the flow in the river was quite low. During these circumstances, the predicting of water levels is usually easy, mainly because there are no remarkable changes which affects to the water levels. The predictions of all forecasting tools during Day 11 are present in Figure 45.

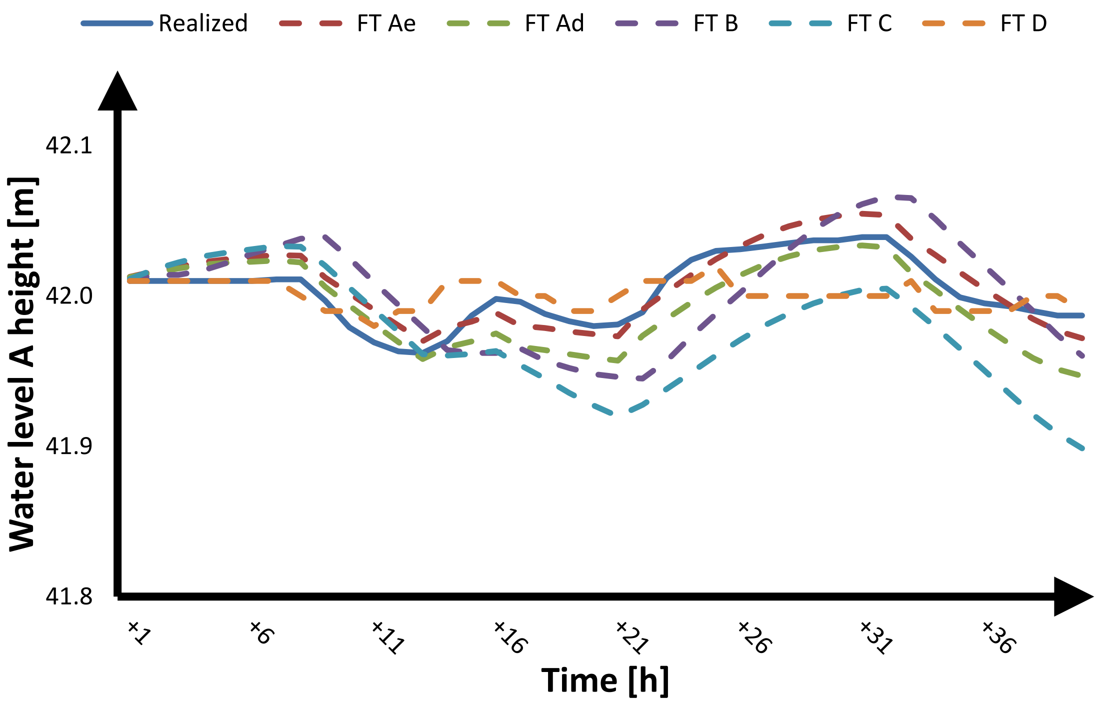

Figure 45: *Different forecasting tools Day 11 forecasts.*

In case of Figure 45, the energy production modelled with realized water level *A* does not differ significantly from production modelled with water level *A* forecasted with $FT\ A_D$. The modelled production with realized water level was only 0.08 % higher than modelled production with $FT\ A_D$ forecast. Realized tail water level and linearized efficiency curve was in use in comparison.

The error in *P7* intake level caused by uncertainty in run-off forecasts is ambiguous, because of the fluctuating storing capacity factor in the forecasting tool *A*. The error in intake level is greater during high discharges. A cumulative error in hydropower plant *P7* intake through 24 hours planning period, caused by 10 $m^3/_s$ uncertainty in run-off forecasts, is presented in Figure 46.

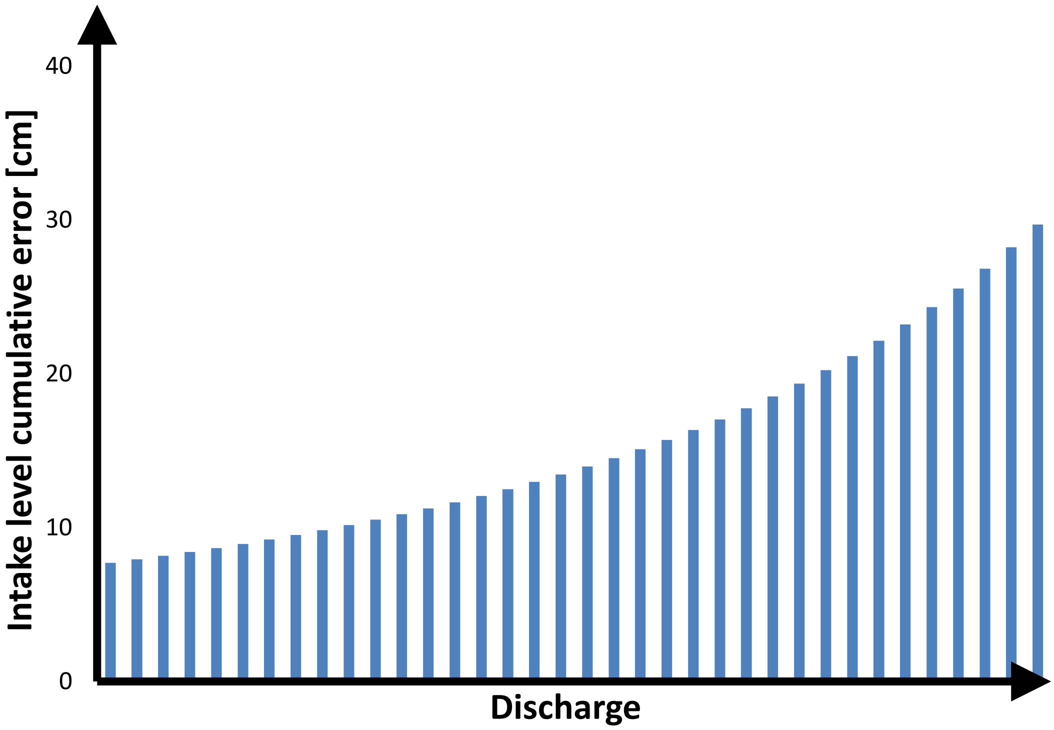

Figure 46: *A Cumulative error during 24 hour time span caused by flaw in run-off forecasts.*

With results of this study, the cumulative error in hydropower plant *P7* intake caused by errors in run-off forecasts depends from *P7* discharge. The error is higher during high discharge of *P7*, because of fluctuating storing volume.

An analysis from hydropower plant *P7* losses was made with the Forecasting tool A. The water level behavior was forecasted like in $FT\ A_D$ and the energy output of each hour was converted like in equation (6) with an efficiency-discharge –curve. There were compared three different planning styles:

1. The discharge of *P7* is as much as the inflow is to the hydropower plant
2. The reservoir of *P7* is utilized to moderate day-night regulating
3. The hydropower unit *P7* production varies highly according to electricity price and the reservoir capability is highly utilized.

The average discharge and of each planning style over analyzed period are equal. In planning style 1, the water level *A* could be kept as high as possible to avoid head-losses. Compared to planning style 1, there occur head-losses in planning styles 2 and 3 during day-time because of higher discharge, whereas the head of the plant is higher due lower discharge on night-times. The discharges and scaled head of the hydropower plant *P7* are presented in Figure 47.

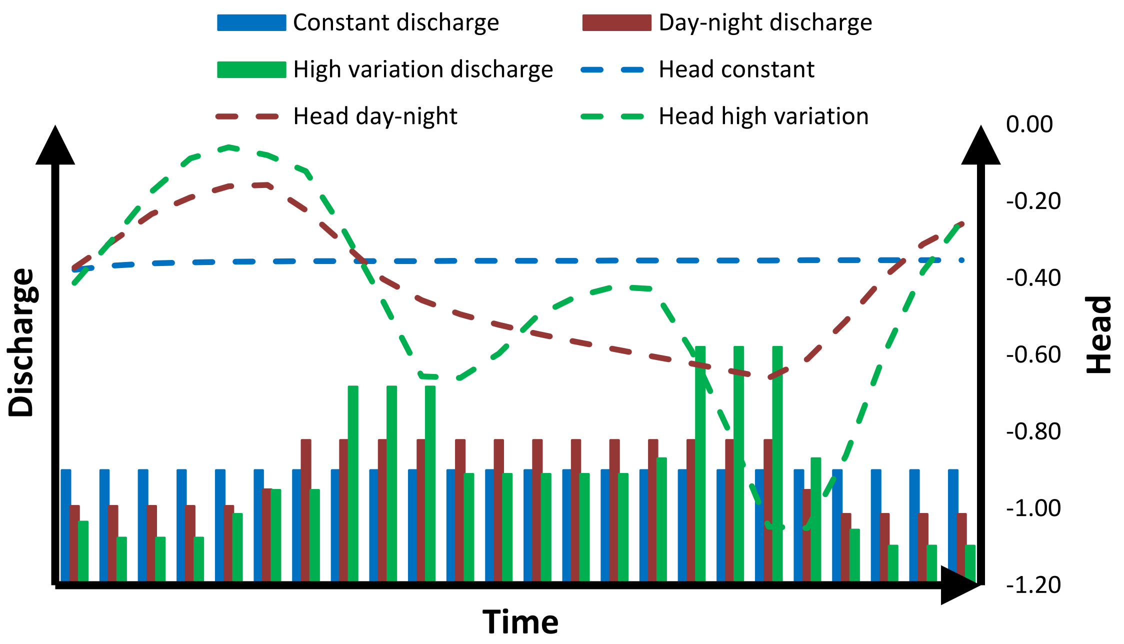

Figure 47: *The discharge and scaled head of the hydropower plant P7. The head of the plant is lowest during highest discharge. This causes head-losses in power production.*

The head of the hydropower plant *P7* is constant during planning style 1, because the intake level and tail water level are stable. When the reservoir utilized and discharge is temporarily higher than the inflow to the plant is, the head-losses are bigger due reduced intake level and increased tail water level. The reduced head causes losses in power production.

The losses of planning styles 2 and 3 were compared to planning style 1. Because the of the fluctuating storing capacity factor in calculation, the change in reservoir over the analyzed period is not same. The change in reservoir content was valued with rule-of-thumb-value and added to the losses. Compared to planning style 1, the daily power production of planning style 2 was 3.36 % less and planning style 3 was 4.30 % less.

4.3.4 Forecasting tool applicability

Creating an accurate forecasting tool based on physical adapted models with available data and inputs is difficult. The river seems to behave differently under different circumstances, which is challenging to model. More accurate models could be created if certain improvements would be performed.

The weakness of the forecasting tool comes from uncertainty of the inputs. The uncertainty forms from two issues:

1. The instability of the data when the adapters of the model are created
2. The instability of the input data during forecasting.

The forecasting tool is very sensitive to errors in inputs. The errors in inputs are not very damaging when only single hour forward is predicted, but the errors cumulate in this model. The forecasting tool treats all the inputs as truths, which is harmful especially in relation of run-off forecasts or hydropower plant discharges. The river behavior depends on season and it is different for example during ice cover, which causes variation on the model accuracy. The data used in model is gathered during open water time.

The directions of the change in predicted water levels are parallel with realized for most of the time, but this model suffers from increasing errors especially in further forecasting, which can be seen in Figures 42 and 44. Also, the model does not "catch" all the spikes, which occur in the intake level. For example, the bounces in intake level are usually more intense than the model predicts. Forecasted water level behavior is usually more directly than actual, which usually behaves smoother. This is not remarkable problem, because there does not appear to be major error during then. The lack of bounces in forecasted intake level and more direct behavior of forecasted water levels can be seen in Figure 43. The model suffers from wandering off and it is most intense during high flow of the river. This can be a result from run-off forecasts or the hydropower unit *P5's* discharge measurement. Water level *B* forecasting with discharge-difference –function is problematic in certain situations when there is substantial change in discharge: despite of substantial decrease of level *A* during strong increase of discharge, the model shows that water level *B* increases, which in turn is not true. In practice, the change in water level *B* is parallel with water level *A*.

The daily production can be modelled precisely when the water level predicting with $FT\ A_D$ is good, but if the model is wandering off, there occurs noteworthy error between realized and forecasted amount of water needed to produce same energy. The requirement of water needed highly depends from head of the plant. The modelled errors in received energy with good and bad forecasts are presented under Figures 44 and 45. The error in water level *A* increases the error in water balance which may lead to situation where the production deviate from planned and causes error in electricity balance. The error in electricity balance may lead to unfavorable operation where energy must be bought or sold with unfavorable prices which have economic impacts. Thus, it is important to forecast the water levels well. Good water level forecasting also minimizes the risk of breaching to environmental limits.

Referring to section 4.3.3, the results of the forecasting tool *A* with discharges made in this study are the best produced by a physical adaptor model. The forecasting tool *A* predicts water levels more precisely than prior forecasting tools. Especially, forecasting tool *A* forecast water level *A* much more precisely than forecasting used in optimization configuration. Also, it forecasts water level *A* more precisely than antecedent forecasting tool mainly used, forecasting tool *C*. Thus, it is sensible to use forecasting tool *A* instead of other forecasting tools.

The reservoir behavior and the effect in water level *A* and *B* are now better known and the forecasting tool *A* has been an improvement in water level forecasting. Thus the reservoir's regulating volume can be utilized better and the lost value evaluation can be improved. However, the model is not accurate always and drifting can occur. The results of model should be observed by user, who's making the last evaluation of forecasted water levels plausibility. Experience of the water level behavior could give some help while the tenability of the model results is observed. The user might have better insight to the inputs, for example run-offs, and the user can revise inputs of the model quite easily while using.

5. CONCLUSION

In this thesis, the step-response tests were implemented in the river system and the river system was modelled with the acquired data. The step-response tests present a new method of gathering data which can be used to model the water dynamics in the river system. This was the first time in the studied river system, where appropriate tests were carried out with the only object of clarifying the water dynamics. The new forecasting tool, created in this thesis, is configured differently from prior tools. For example, the size of the reservoir of the hydropower unit *P7* was modelled as a constant in prior tools. The new forecasting tool forecast the water levels better and improves the possibility to more market-based operation with *P7*. In addition, the lost value evaluation can be improved better. Moreover, this study results clarifies the objects of further development of studied river system.

Creating accurate models which strive to model the physical movement of water in the studied river proved to be challenging during this thesis. This could be a result stemming from the nature of the input data. There are many X-factors in data, for example accurate knowledge of the hydropower unit discharges and the run-offs between the units.

Some uncertainties can be eliminated, but these require economical investments. The discharges of hydropower units could be calibrated, which might improve the quality of the data. The calibration would not involve all of the units, only hydropower units *P6* and *P7*. The river sections between *P5 – P6* and *P7 – P8* are easier to handle because there are no remarkable run-offs in those sections. The calibration could be performed by utilizing external company and the efficiency curves of each generator can be remodeled by the river operators. Also, more accurate delays between hydropower units could be determined at the same with discharge calibration. With knowledge of exact discharges, the accurate storing capacity could be determined. Also, it could be determined with depth sounding. The depth sounding should be performed at least from *P7* water level *B* measurement to the intake level. Depth sounding of the water level *B* vicinity should simplify its forecasting, especially during times when the reservoir is filled and the behavior of the water level *B* is hard to forecast. The depth sounding provides also a possibility to model the water movement with HEC-RAS, but then the whole river section between *P6* and *P7* should be depth sounded. This might not be sensible, because it would require heavy economic investments and a lot of work combined with uncertainty of its functionality.

At the moment, hydropower unit $P7$ control circuit does not provide an opportunity to operate the plant with an automatic discharge adjustment which would keep the intake level stable. With such new control circuit, more accurate information about the delays could be determined, because the discharge of $P7$ would be able to follow the inflow to the plant. Also, the remodeling of the control circuit would open up a lot of new possibilities. The new control circuit could continuously keep up the water levels while fulfilling a production requirement and propose changes in production plan. At the moment, the water levels are looked after by the operator, whom carries out changes to ensure that environmental permits are not breached. The remodeling of the control circuit would decrease the consumed time by the river operation. In addition, such a control circuit does not require the accurate knowledge of the water movement in the river. Similar river level control circuit remodeling could be done to all units in the river, which would be capable handling all the units including production plans and water levels. Regardless of the hydropower plants automation controlling, the final responsibility about the water levels and produced energy is laid upon the operator and the operator still needs to know the river behavior in case of problems without plant automation.

At the moment, the developed forecasting tool $FT\ A_D$ is the best tool that uses physical adapter modelling to forecast water level A. The $FT\ D$ predicts the water levels better, but its behavior is unpredictable and it is not suitable to be used in optimization. $FT\ A_D$ model's issues with forecasting errors cumulating and the unpredictability of $FT\ D$ can be mitigated with combining the predictions of these two. The near future forecasting can be made with $FT\ A_D$ and further forecast could be gradually combined from the forecasts of these two models.

All parts of the $FT\ A_D$ configuration are not suitable to linear optimization used in river production planning, but the parts which are suitable, should be added to the calculation. Due to partly adding, the functionality should be confirmed beforehand and the configuration can be first added to the "after calculation" of the optimization. Also, it is sensible to observe the behavior of the optimization with new configuration before it is possibly taken into use in the river production planning.

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