Master's thesis

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Water loss in unlined pressure tunnels

A review of the extent, causes and consequences for selected cases

Master's thesis in Hydropower Development Program
Supervisor: Bjørn Nilsen
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Program for MSc-thesis

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In the planning of underground hydropower plants, particular emphasis is placed on locating and designing pressure tunnels in such a way that water loss is minimized. To optimize the location of pressure tunnels and ensuring successful design, hydrogeological, topographical, rock mass and rock stress considerations are key issues. Failure in properly addressing these issues during the design procedure may have severe security and financial consequences.

The main purpose of this MSc is to review water loss for some Norwegian hydropower plants, and based on this discuss the significance of the issue also on a more general basis. A selection of Norwegian hydropower plants are to be included in the review (tentatively Roskrepp, Hatlestad, Tonstad and Tafjord V). Among activities and issues considered to be of special interest for this study, the following are particularly emphasized:

- Collecting existing data and gathering new data from water loss measurements for the selected tunnels (total loss, loss through plugs, loss in rock mass/waterway).
- Evaluation of lost revenue.
- Evaluation of potential loss due to hydraulic "jacking" in case of marginal FoS against hydraulic failure.

To evaluate the issue in a more international perspective and rock conditions different from Norwegian hard rock, a brief comparison with two Indonesian cases is also to be done.

The work shall lead to a suggested best practice on how measures to reduce water loss can be incorporated in pressure tunnel design.

The master-assignment is organized as part of the R&D activities connected to HydroCen – Norwegian Research Centre for Hydropower Technology, with engineering geologist/PhD research fellow Henki Ødegaard as co-supervisor. The thesis work is to be completed by June 11th, 2019.

The Norwegian University of Science and Technology (NTNU)
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This sheet is to be inserted as page 1 of the Master thesis

Thesis Abstract

Water loss in unlined pressure tunnels – A review of the extent, causes, and consequences for selected cases

by

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Unlined pressure tunnel is a cost-saving method that is mostly used in designing a hydropower plant, although the application requires favorable ground condition. Initial assessment in pressure tunnel design often uses Norwegian criterion, which is widely used to provide a basis for lining design and rock cover needed for the underground infrastructure. In some cases, however, the implementation of this approach in pressure tunnel design failed to confine the water tightness and threaten the financial viability. It is compounded with unawareness of water loss from the unlined tunnel, and as consequences, it led into either ineffective design of lining, or the water resources wasted.

This thesis presents a general review on water loss from selected Norwegian hydropower case and proposes a method to assess water loss from the unlined tunnel by using a well-known concept of pressure measurement. In this study, the water loss assessment from unlined tunnel started with the development of tunnel stage-volume curve, continued with the identification of leakage source through water ingress analysis and closed with an assessment of water loss consequences from the unlined tunnel. Three main consequences of water loss are identified in this study; (1) the estimated annual financial loss can be up to 10% of the annual revenue, (2) the reduction of safety factor from the nearby slope due to the increase of groundwater pressure ranging from 10% - 30%, and (3) the increase of water consumption ranging from 7 m^3/MWh – 70 m^3/MWh.

Water loss review conducted in this study shows the evidence from Norwegian rule of thumb limitation. Due to ground condition uncertainty, a new method to optimize lining design by conducting pressure measurement analysis is introduced. The idea is to develop inflatable packer for interval measurement alongside the pressure tunnel. In addition, the idea to use water loss as negotiation tools to reduce environmental flow release is also discussed in this study. To achieve a better confidence level, future study to understand the behavior of water loss through the permeable zone is recommended. Moreover, a probabilistic approach is also recommended in the future to quantify uncertain factors.

Key Words: Hydropower Unlined tunnel, Tunnel leakage, Transition zone,

Preface

Back in 2018, the selection of a master thesis topic is a dilemma for me. Selecting a topic in rock engineering field means I will have a master thesis topic outside my comfort zone. Thankfully, with guidance, encouragement, and patience from Professor Bjørn Nilsen, I am able to finish this thesis. I also want to express my deep gratitude to my co-supervisor Henki Ødegaard, His enthusiasm working in this project has preserved the optimistic attitude and made this thesis possible. This thesis is primarily addressed to fellow hydropower engineers, fellow academics and anyone who interested in hydropower infrastructure. With the spirit of NTNU motto "Knowledge for a better world" I hope this thesis contributes to the development of world knowledge, especially in the hydropower industry.

Beside the supervisors, the writing process of this thesis involves many parties, first of all, I would like to say thank you to Department of Civil Engineering and Environmental and Department of Geoscience and Petroleum for the facilities provided for me to write this thesis. I also would like to say my gratitude to Mr. Thorleiv Hurthi (Sognekraft). Mr. Kaspar Vereide (Sira-Kvina), and Mr. Ole Grønberg Myrold (Tafjord Kraft) for the permission to use data from the respective company.

In this occasion, I also want to say thank you for the loves and support from loves friends and family. I would like to mention my big family in Indonesia, especially Mom, Dad, and my little brothers. I also want to say thank you for hydropower development program batch 2017, and friends from Indonesian Students Unions in Trondheim, which provide me with a wonderful friendship during my journey in Norway. In particular, I would like to mention Irene Hutami for the help in proofreading this thesis, Erik Kleiven Rynning for becoming the most dedicated driver during the journey to Tonstad for the data gathering process, and Anisa Noor Corina for becoming my best friend, flatmate and the best support system ever.

Times really flies while spending past one semester for the writing process for this master thesis. As part of the requirement for a master degree in Hydropower Development Program, I devote myself to provide the best results in this thesis. Nevertheless, I realize this study still be able to be improved. Therefore, my message for the future generations, for the reader of this thesis, thank you for the interest in my work and hope you can continue and enhance this study to provide better knowledge for the world.

Ad maiorem Dei gloriam.

Trondheim, 11 June 2019

Steven Sergij Salim

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List of Symbols and Abbreviations

h	Vertical depth of the point studied
A	Cross-sectional area perpendicular to flow
EEKV	Energy Potential in 1 m3 of Water (kWh / m3)
FoS/FS	Factor of Safety
H	Static water head at the point studied
HPP	Hydropower Plant
i	Hydraulic gradient
K	Hydraulic Conductivity
L	Shortest distance between the surface and the point studied
NGI	Norwegian Geotechnical Institute
NGU	Norges Geologiske Undersøkelse
NVE	Norges Vassdrags-og Energidirektorat
P	Pressure Applied at a Test Stage
P	Power
P_0	Reference Pressure
Q	Flow rate of liquid through the porous
$_uL$	Lugeon Value
α	Inclination of the shaft
β	Average inclination of the valley side
γ	Density of the rock mass
γ_w	Density of water
Δh	Change of hydraulic grade line
η	Component Efficiency (Turbine, Generator, and Transformer)
ρ	Water Density
Ψ	Dimensionless pressure factor

Chapter 1 Introduction

And as for love being zero? Do not make me laugh. Tell the queen of the court the following; "Zero is where everything starts, nothing would ever be born if we didn't depart from there"

-Shinichi Kudo-

Using unlined pressure tunnel is well known as a cost-saving method to develop hydropower plant by reduction of lining needed [1]. An unlined pressure conduit requires rock conditions able to withstand the internal water pressure both with regard to leakages and to deformations that can lead to failures [1]. Nevertheless, within times the development of the hydropower plant in the world tends to have more higher-pressure head than before (Figure 1.1) [2]. Utilizing high head Hydropower Plant subsequently will need an increase on the rock cover needed and as a consequence, the underground infrastructure needs to be put deeper in the mountain or requires extra reinforcement. Extra precaution is needed due to the increase of risk of undesirable events and one of them is the occurrence of leakage from the unlined tunnel system.

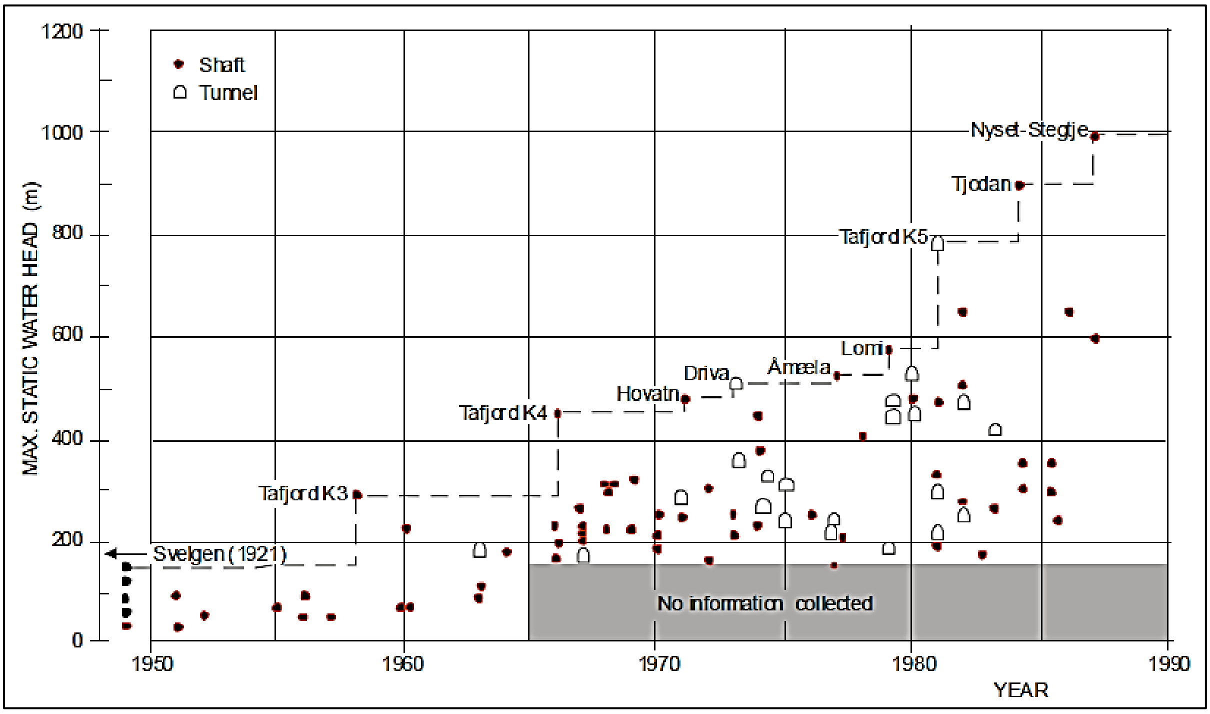


Figure 1.1. Development of Unlined Pressure Tunnels and Shafts [2].

Potential cause that can trigger leakage through the rock mass will be increased due to the increase of uncertainty. The key to minimizing leakage from unlined pressure tunnel is to optimize the location of pressure tunnel against the hydrogeological, topographical, rock mass and rock stress [3]. In Norway, the use of the Norwegian rule of thumb last updated in 1971 by Bergh-Christensen and Dannevig provides a good basis for estimating the rock cover needed for a pressure tunnel [2]. The principle of the Norwegian rule of thumb is by comparing the ability of rock mass to withstand the internal pressure from inside the pressure tunnel based on the topographic of the valley, rock mass and rock stress [2]. Using the Norwegian rule of thumb proven to be satisfactory in most case of pressure tunnel design [4]. Nevertheless, there are some cases it failed due to the role of topography and stress anisotropy. Amadei et al., 1995 argue that the safe alignment of unlined pressure tunnels depends greatly on the extent of tensile regions in valley walls [5]. These findings also in line with Marwa (2004) which argue the needs to conduct field test in determining lining from a pressure tunnel [6], and Amberg et al., which criticize the importance of joint identification before using the Norwegian rule of thumb in assessing hydro jacking [7].

Moreover, the hydrogeological aspects were often neglected in the use of the Norwegian rule of thumbs as mentioned by Basnet (2018) [3]. The influences of hydrogeological condition into leakage from an unlined pressure tunnel are illustrated in Figure 1.2. According to Benson (1989), there are four possibilities of leakage condition depending on the static head, groundwater condition, and permeable zone. The highest possibility of leakage will occur in the condition where the static head is much greater than the groundwater table and with the occurrence of the permeable zone. The principle to put static head lower than the groundwater table is often not considered during the design phase of a hydropower plant. [8]

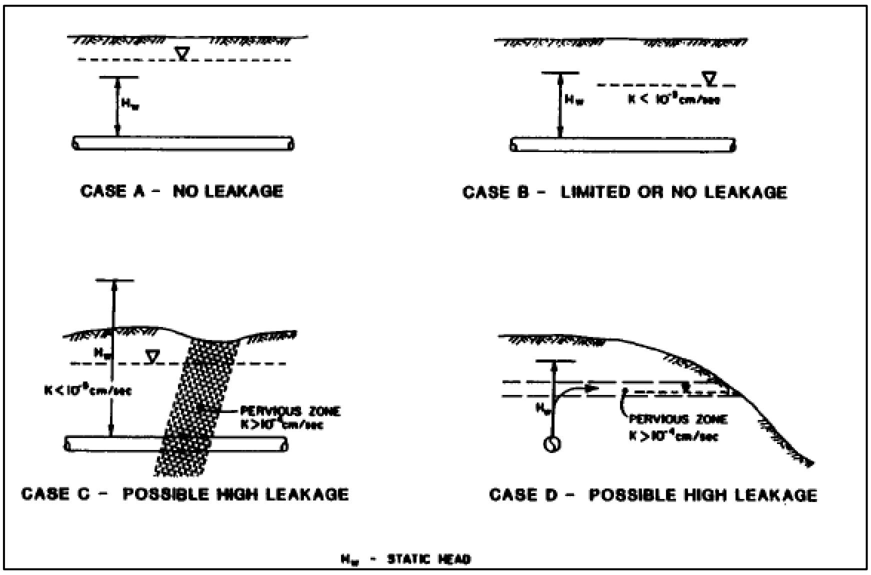


Figure 1.2. Assessment of Leakage Condition in Pressure Tunnel by Benson *[8].*

Previous study observing leakage from unlined pressure tunnel tends to focus on identifying the leakage and did not discuss the severity of leakage from the unlined tunnel. Additionally, in the country with low water scarcity threat, leakage from waterway is often neglected. Regardless leakage from pressure tunnel will increase water consumption from a hydroelectric system and in hydropower plant with high water consumption, it will threaten the reputation of hydropower plant as a renewable source of energy [9]. Therefore, awareness regarding water loss from unlined pressure tunnel needs to be resuscitated.

This study was conducted in order to give a better understanding of the behavior of water loss from the unlined pressure tunnel. The objective of this master thesis is to conduct a review of water loss from several Norwegian hydropower plants and evaluate the loss of revenue due to the loss of water and evaluate the potential loss due to hydraulic jacking in case of a marginal factor of safety against hydraulic failure. At first, data from Roskrepp HPP, Tonstad HPP, Tafjord 5 HPP, and Hatlestad HPP was used as the basis of this review. However, only data from Hatlestad HPP was ready to use for the analysis during this master thesis work. Duge PSP and Solhom HPP were proposed as a replacement, however, the archive review did not give a satisfying result. The problem with data collecting also happens with Cirata HPP and Saguling HPP from Indonesia, the author was unable to retrieve the data due the time and distances difficulties.

To avoid the scope of the study to be broadened, several limitations were set to keep the focus of the study. This study uses the static pressure head, this is based on arguments from Brekke (1987), which states that in unlined pressure tunnel design is typically used static pressure head, while the dynamic head is used for the steel lining design [10]. Limitations also set on several input data. The electricity prices used the historical prices according to Nordpool database and the geological data input mainly based on NGU. This study also used information from NVE to complete the information from several hydropower plants.

Chapter 2 Literature Studies

A thousand words will not leave so deep an impression as one deed.

- Henrik Ibsen -

A literature review was conducted before the start of work with an objective to familiarize the topic and limitations from previous studies. Past study regarding leakage from unlined tunnel did not discuss much of the method to measure leakages and mainly focused on the estimation of leakage. This chapter will discuss background theories that have been used in this work. The literature studies start with a brief introduction on the unlined tunnel principle and the Norwegian rules of thumb. The literature studies continue with hydrogeology effect on pressure tunnel, rock mass permeability, Lugeon test interpretation, and tunnel lining selection. Hydropower potential calculation is also discussed briefly to introduce the relations from water loss into energy production loss.

2.1 Unlined Pressure Tunnel

Unlined pressure tunnel term is used to describe hydropower tunnels where water is in direct contact with the rock. Figure 2.1 Illustrate the development of general layout of hydropower plants in Norway, from the conventional hydro plant until unlined high-pressure tunnel. Unlined pressure tunnel was introduced in 1975 with the construction of Herlandsfoss hydro plant. The use of unlined pressure tunnel significantly reduces the cost and time needed in constructing a hydro plant and proven as a cost-saving method. Due to its popularity, now there are more than 80 unlined pressure waterways in Norway with maximum static head reaching 1000 m. [11]

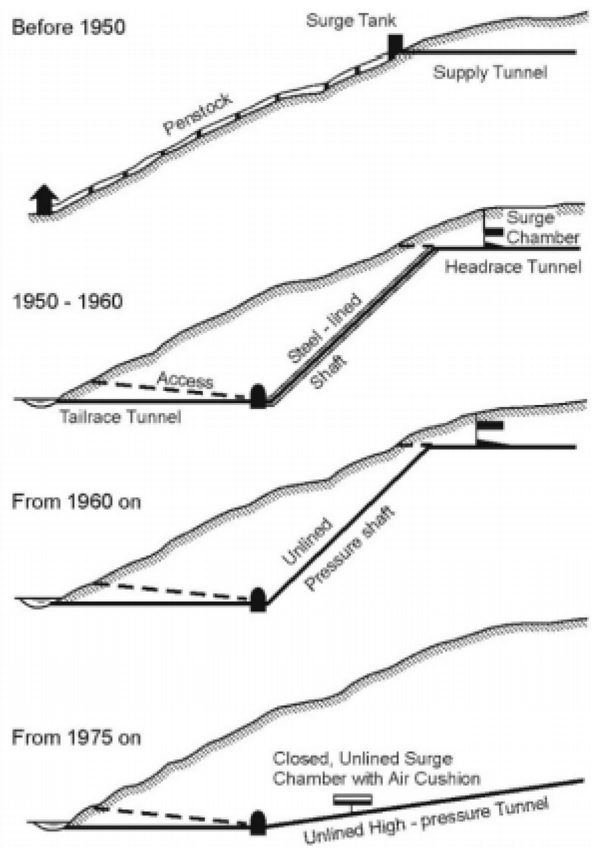


Figure 2.1. Development of The General Layout of Hydropower Plants in Norway [11].

2.1.1 Hydraulic Jacking and Hydraulic Fracturing

Hydraulic jacking and hydraulic fracturing occur when internal pressure from a pressure tunnel exceeds the normal pressure acting on the fracture. This phenomenon will increase the aperture of the joints (hydraulic jacking) or create a new joint opening (hydraulic fracturing), which leads to the increase of leakage from a pressure tunnel due to the joint behaving as an exit way for the flow. [7]

The orientation of a jacking surface relative to a deformable surface is also important [10]. Figure 2.2 illustrates the appliance of hydraulic forces related to the deformability of rock mass due to hydraulic pressure [10]. Assuming that the jacking and deformable surfaces are parallel to each other, the potential for rock mass movements will be beyond if the jacking and deformable surfaces are perpendicular to each other.

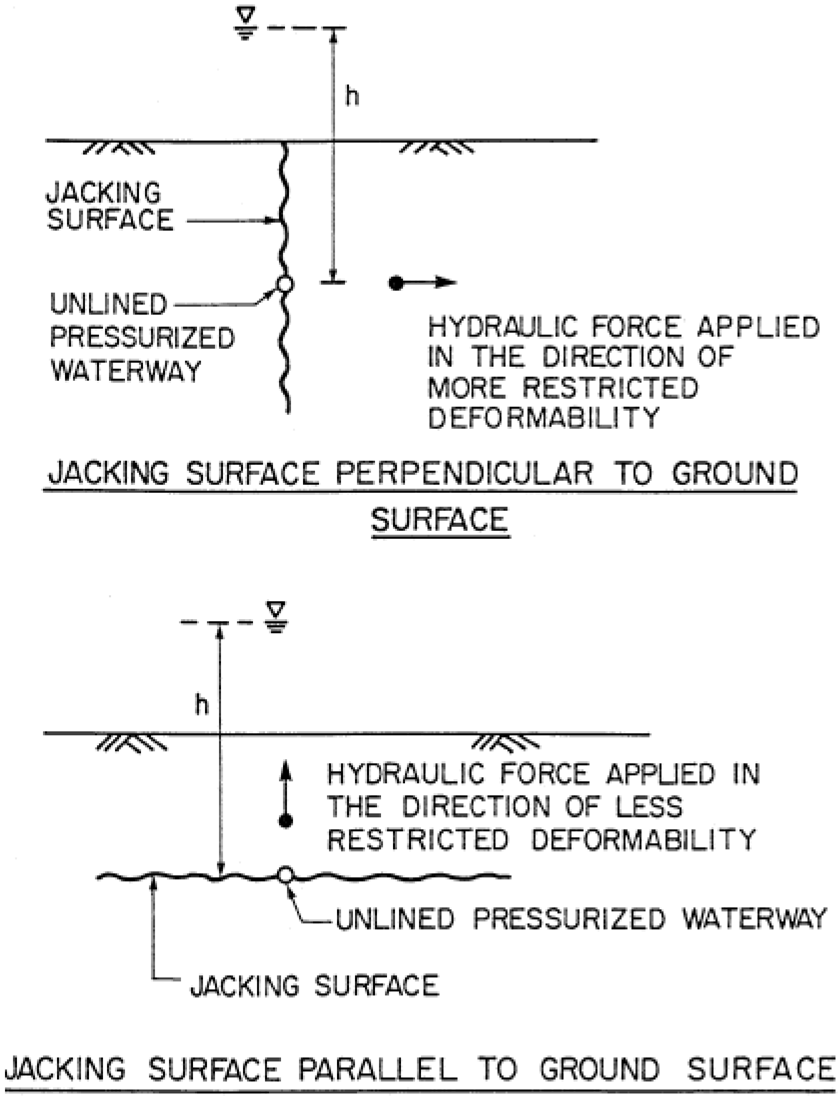


Figure 2.2. The orientation of Jacking Surfaces Relative to Ground Surfaces. [10]

2.1.2 Norwegian Rule of Thumb

The idea of Norwegian rule of thumb was based on the condition that the waterway needs to have an adequate rock cover. Therefore, the internal water pressure was equalized by the rock cover mass [11]. The earliest (before 1968) design criteria is illustrated in Figure 2.3 with the rule was expressed as follows: [2]

$$h > c \times H \tag{1}$$

where:

h = vertical depth of the point studied

H= static water head at the point studied

c = a constant (0.6 for valley sides with an inclination up to 350 and increased 1.0 for valley sides of 600).

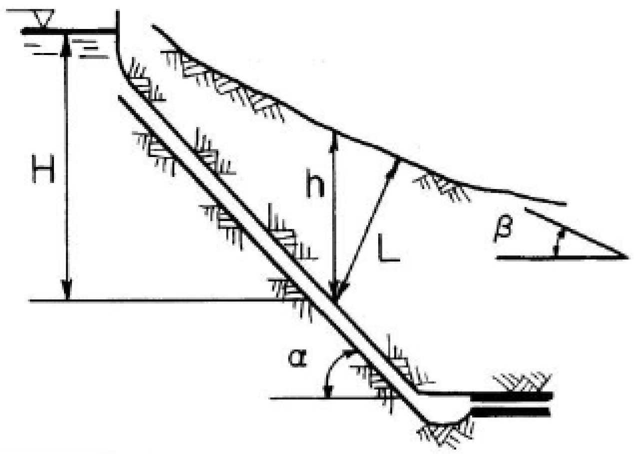


Figure 2.3. Definitions for the Rule of Thumb for Tunnel Design

The use of Norwegian rule of thumb has some revision in 1970, proposed by Selmer-Olson as follows: [2]

$$h > \gamma_w \times H/\gamma \times \cos\alpha \tag{2}$$

where:

γ_w = *density of water*

γ = *density of the rock mass*

α = *the inclination of the shaft*

And the final significant revision is when the inclination of the valley side was taken directly into an account with the revision in 1971 by Bergh-Christensen and Dannevig. The proposed equation as follows: [2]

$$\mathrm{L} > \gamma_\mathrm{w} \times \mathrm{H}/\gamma \times \cos\beta \tag{3}$$

Where:

L = shortest distance between the surface and the point studied

β = average inclination of the valley side

The validity of the Norwegian Rule of Thumb conducted by NGI in 1972 [4] showed a satisfying result for the use of Norwegian rule of thumbs for the preliminary layout in terms of minimum requirements. Figure 2.4 shows the application of Norwegian rule of thumb in several projects and most of the significant leakage occurs in below safety lines. Nevertheless, several hydropower projects still have significant leakage. Although they already fulfilled the Norwegian rule of thumb.

The possible reason for this occurrence is due to the assumption of the Norwegian rule of thumb uses perpendicular orientation between hydraulic jacking occurrence and the least in-situ stress component (σ_3). This simplification, since the jacking pressure could be higher than σ_3 if jacking occurs along surfaces (joints, faults, foliation, etc.) that are inclined with respect to that stress component [5]. Due to this uncertainty during preliminary design, it is recommended to conduct appropriate geological and testing methods to reduce the probability of significant leakage.

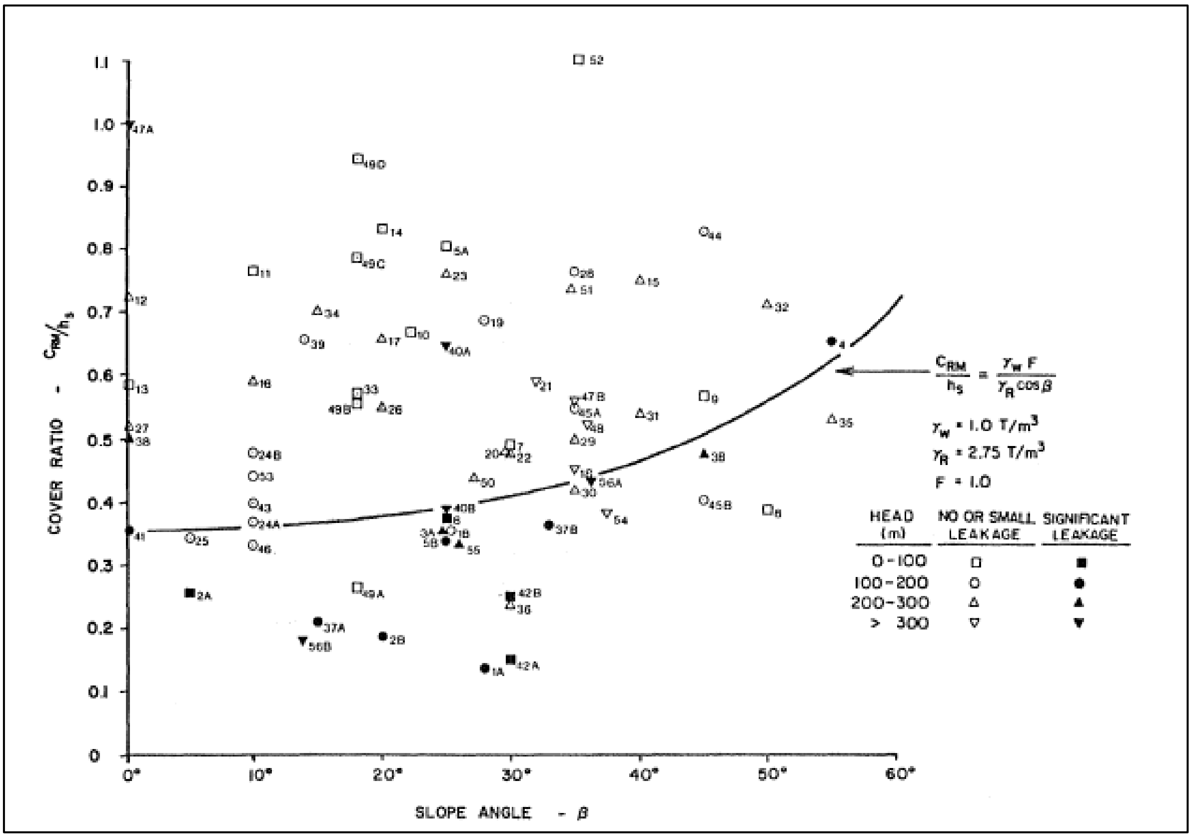


Figure 2.4. Application of the Norwegian Criterion to Selected Projects [4].

2.2 Hydrogeology Effect on Pressure Tunnel

The sources of excessive leakage from pressure tunnels occur in two possible way, the first is by hydraulic jacking and the second source is internal groundwater pressure exceeding the external groundwater pressure on pervious rock [8]. According to Benson (1989), the problem with a pervious rock is more difficult to deal with as there are numerous methods to approach this problem and there is a high possibility of misjudgment. In some cases, the loss of water from a pressure tunnel is allowed depending on the quantity of water losses, the value of water losses, the safety issue of the surrounding terrain and local environmental effect.

Figure 2.5 exemplifies the scenario of leakage which occurs due to groundwater table condition in relation to the internal pressure from a pressure tunnel [8]. Case B, where the groundwater table is higher than the static head the leakage from pressure tunnel is limited or even did not exist. However, the existence pervious zone (case C and Case D) will give a significant possibility of seepage from the pressure tunnel. Standard flow nets analysis can help estimate the leakage from a pressure tunnel. However, the use of flow nets analysis needs extensive data along the tunnel and will increase the time of investigation study. Moreover, the subjectivity of how many investigations needed to be conducted, play an important role as the key to success in unlined pressure tunnel.

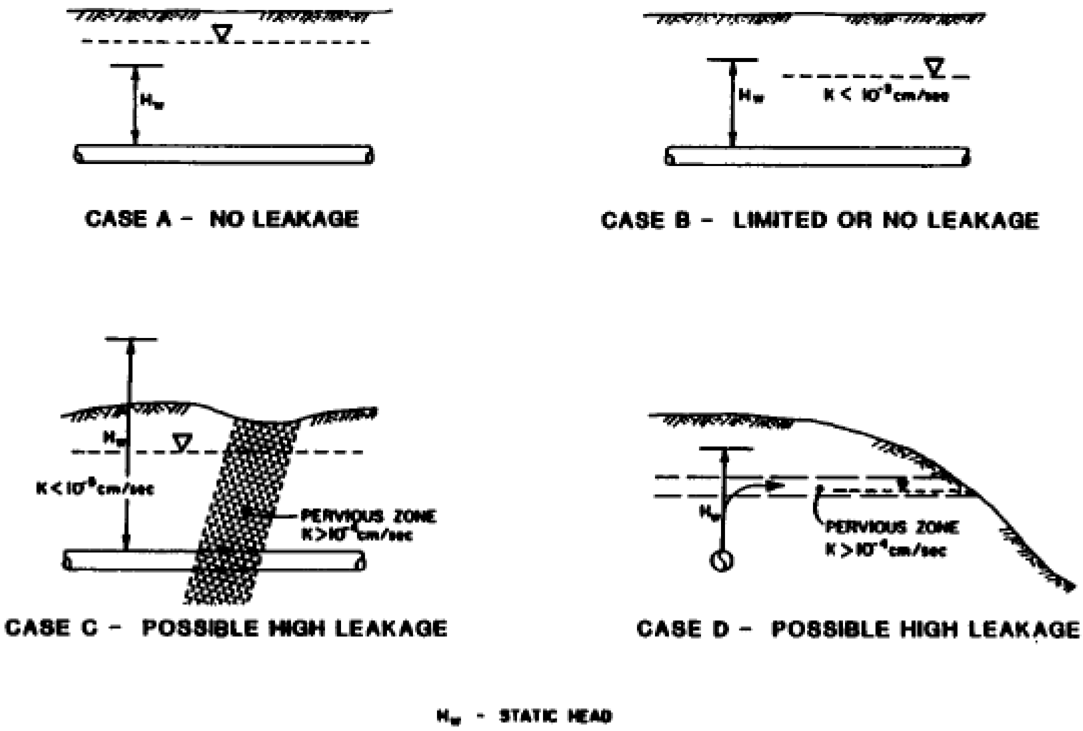


Figure 2.5. Illustration of Leakage Due to Groundwater Table [8]

2.2.1 Groundwater Flow

According to USGS, groundwater is water that exists underground in saturated zones beneath the land surface with the upper surface of the saturated zone called the water table [12]. Rate of flow in groundwater typically has a range from 7 – 60 centimeters per day and during the transport, groundwater will fill the pores and fractures in underground material [12]. It is critical to note that groundwater flows from areas with high hydraulic head to areas of lower hydraulic heads, which means it can flow uphill if the condition fulfilled [13]. The transport rate of groundwater can be addressed using Darcy's Law [14].

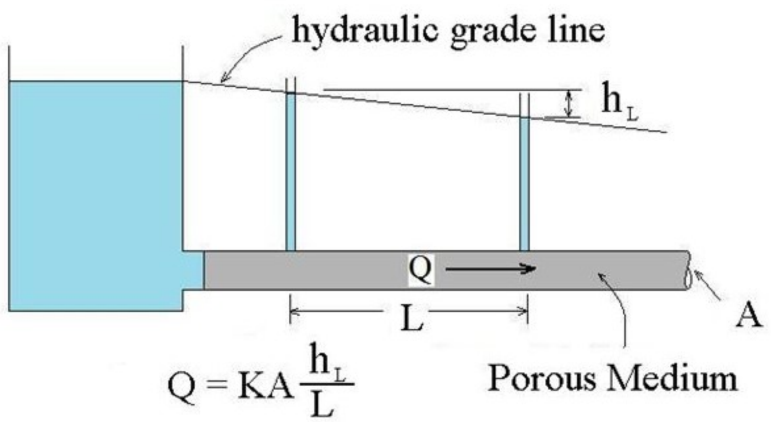


Figure 2.6. Darcy's Law Apparatus [15]

Figure 2.6 illustrates the observations of the flow rate through a porous medium, which become the base of Darcy's law. The equation of Darcy's Law can be written as:

$$Q = K \times A \times i \qquad (4)$$

Where:

Q = Flow rate of liquid through the porous medium (m^3/s)

A = Cross-sectional area perpendicular to flow (m^2)

i = Hydraulic gradient (m/m)

K = Hydraulic Conductivity (cm/s)

Hydraulic gradient (i) in Darcy's equation can be defined as the slope of the hydraulic grade line. The equation of the hydraulic gradient can be written as:

$$i = \frac{\Delta h}{L} \tag{5}$$

Where:

i = Hydraulic gradient

Δh = Change of hydraulic grade line (H1 – H2)

L = Length of the porous medium

Hydraulic Conductivity (K) represents the capability of a media to transport fluid. In the case of groundwater, hydraulic conductivity mainly depends on pore spaces and fractures. Table 2.1 shown the range of K values found in nature using standard viscosity and specific gravity for water at 20 ^{0}C and 1 atm [7].

K (cm/s)	10^2	10^1	$10^0=1$	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
K (ft/day)	105	10,000	1,000	100	10	1	0.1	0.01	0.001	0.0001	10^{-5}	10^{-6}	10^{-7}
Relative Permeability	Pervious				Semi-Pervious				Impervious				
Aquifer	Good					Poor				None			
Unconsolidated Sand & Gravel	Well Sorted Gravel		Well Sorted Sand or Sand & Gravel			Very Fine Sand, Silt, Loess, Loam							
Unconsolidated Clay & Organic					Peat		Layered Clay			Fat / Unweather Clay			
Consolidated Rocks	Highly Fractured Rocks				Oil Reservoir Rocks			Fresh Sandstone		Fresh Limestone, Dolomite		Fresh Granite	

Table 2.1. Hydraulic Conductivity Values Found in Nature [7].

2.2.2 Groundwater Table

Sub-chapter 2.2.1, stated that groundwater is located in the saturated zone of the soil. The boundary layer between the saturated zone and the unsaturated zone is defined as the groundwater table. The location of the groundwater table can change depending on evapotranspiration, surface runoff generations, and climate [16]. According to USGS, the most reliable method to obtain the depth of the water table is by conducting a measurement of the water level in a shallow well or using geophysical methods. In Norway, the database of groundwater table available through NGU-GRANADA database [17].

2.3 Rock Mass Permeability and Lugeon Test

Zeiger (1976) on published technical report on determination of rock mass permeability, conclude that the rock mass permeability is controlled by fissures such as joints, fractures, and bedding planes contained within the mass. Various orientations, spacings, apertures and surficial geometry of fissures sets will cause anisotropic permeability, which can vary the flow from laminar to turbulent. Applied stress can alter the rock mass permeability, which will cause an unstable condition. Therefore, permeability should be measured in an interval during construction and post-construction with seepage pressure should be monitored after construction and compared with the predicted value. [18]

Laboratory measurement on determining rock mass permeability due to individual fissure is the most appropriate way to obtain flow characteristics. However, to obtain representative samples is a problem. Therefore, the best permeability measurement is by conducting field-testing such as water pressure test, pumping test, tracer test, air pressure test, and pressure drop test. [18]

One of the tests to measure rock mass permeability is Lugeon test or packer test. Developed by Maurice Lugeon (1933), Lugeon test is carried out at an interval of a borehole and different locations along the borehole. The schematic of Lugeon test configuration is presented in Figure 3.7. Prior to the beginning of the test, a *maximum test pressure* (P_{MAX}) is defined and the value is chosen such that it does not exceed the *confinement stress* (σ_3) expected at the depth where the test is being conducted. [19], [20]

Equation (6) shown the rule of thumb in determining P_{MAX} value, where D is equal to the *minimum ground coverage*.

$$P_{MAX} = D \times \frac{1\,psi}{ft} \qquad (6)$$

Lugeon test usually conducted in five stages, with particular water pressure for each stage as shown in Table 2.2.

Test Stage	Descriptions	Pressure Step
1	Low	0.50 P_{MAX}
2	Medium	0.75 P_{MAX}
3	Maximum (Peak)	P_{MAX}
4	Medium	0.75 P_{MAX}
5	Low	0.50 P_{MAX}

Table 2.2. Pressure Magnitude Typically Used for Each Test Stage [19]

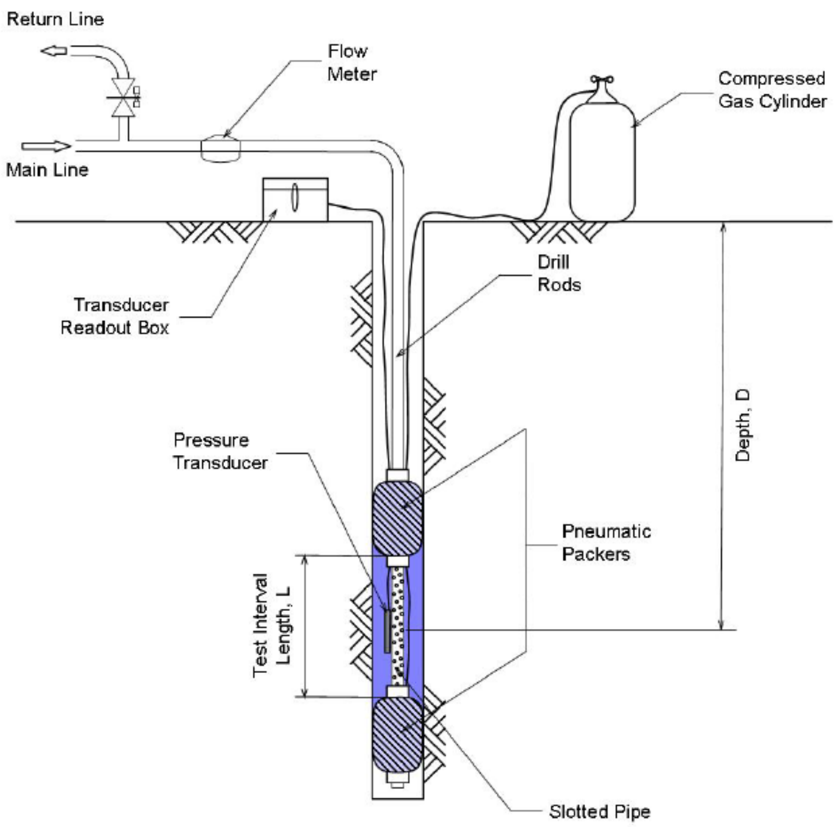


Figure 2.7. Lugeon Test Configurations [19].

Results of Lugeon test gives information on rock discontinuities, which intersect the wall of the borehole in the test section and expressed in Lugeon units (uL or L). A unit of Lugeon (1L) is defined as the water loss of 1 liter/minute per meter length of the test section at an effective pressure of 1 MPa [20]. Table 2.3 describes the conditions from typical Lugeon Value according to Quiñones-Rozo [19]. Here Lugeon value is calculated by using equation (7), where:

q = Water loss (liter/Min)

L = Testing length (m)

P_0 = Reference Pressure (1 MPa)

P = Pressure Applied at a Test Stage (MPa)

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$<1 \times 10^{-5}$	Very Tight	<1
1-5	Low	$1 \times 10^{-5} - 6 \times 10^{-5}$	Tight	±0
5-15	Moderate	$6 \times 10^{-5} - 2 \times 10^{-4}$	Few Partly Open	±1
15-50	Medium	$2 \times 10^{-4} - 6 \times 10^{-4}$	Some Open	±5
50-100	High	$6 \times 10^{-4} - 6 \times 10^{-3}$	Many Open	±10
>100	Very High	$>1 \times 10^{-3}$	Open Closely Spaced or Voids	>100

Table 2.3. Condition of Rock Mass Discontinuities Associated with Different Lugeon Values [19]

$$Lugeon = uL = \left(\frac{q}{L}\right) \times \left(\frac{P_0}{P}\right) \qquad (7)$$

In 2005 Quiñones-Rozo also proposed an update to the Lugeon interpretation practice, which mainly derived from the work performed by Houlsby (1976) by rearranged equation (7) into equation (8).

$$\frac{q}{L} = Lugeon\,Value \times \frac{1}{\alpha} \times \frac{P}{P_0} \tag{8}$$

Changing the last two factors in Equation (8) by defining as a dimensionless pressure factor (Ψ), the flow loss could be express as shown in equation (9)

$$\frac{q}{L} = Lugeon\,Value \times \Psi \tag{9}$$

Therefore, the flow loss could be interpreted as the product of the Lugeon value and the dimensionless pressure factor (Ψ). The purpose is to obtain a "Pressure Loop" from a plotted graph between pressure factor and flow loss [19]. Example of "Pressure Loop" from a typical Lugeon Test is shown in Figure 2.8. Table 2.4 shown the proposed Lugeon interpretation procedure by Quiñones-Rozo (2005). Although it still uses the same behavior categories proposed by Houlsby, using this approach is expected will allow real-time monitoring and interpretation of test data.

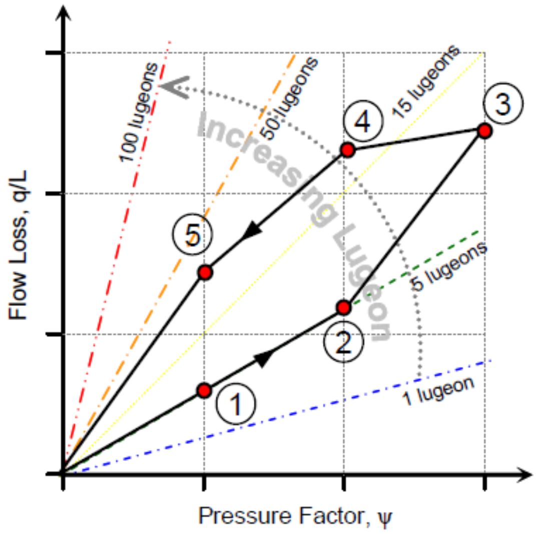


Figure 2.8. Interpretation of Lugeon Test Data in the Flow Loss V Pressure Space [16]

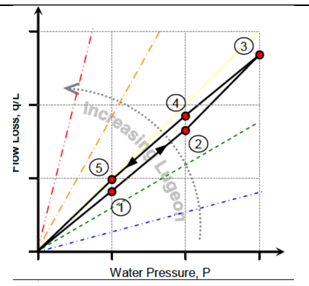
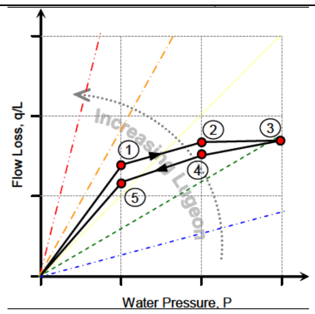
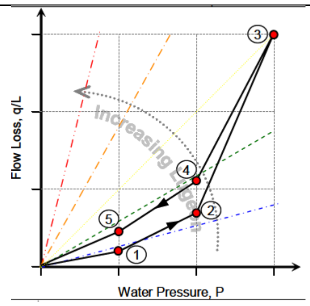
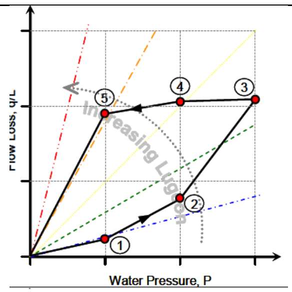
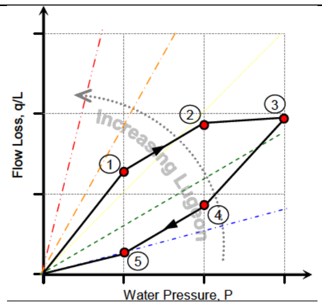
Behavior	Water Loss Vs Pressure Pattern	Description	Representative Lugeon Value
Laminar		All Lugeon values about equal regardless of the water pressure	Average of Lugeon values for all stages
Turbulent		Lugeon values decrease as the water pressure increase. The minimum Lugeon value is observed at the stage with the maximum water pressure	Range of Lugeon values observed at water pressures expected during operation. If water pressure expected during operation is unknown use the value corresponding to the medium water pressure (2nd or 4th stage)
Dilation		Lugeon values vary proportionally to the water pressure. The maximum Lugeon value is observed at the stage with maximum water pressure	Range of Lugeon values observed at water pressure expected during operation. If water pressure expected during operation is unknown use the value corresponding to either low or medium water pressure (1st, 2nd, 4th, or 5th stage)
Wash-Out		Lugon values increase as the test proceeds. Discontinuities infillings are progressively washed-out by the water	Higher Lugeon value recorded (5th Stage)
Void Filling		Lugeon values decrease as the test proceeds. Either non-persistent discontinuities are progressively being filled or swelling is taking place	Use final Lugeon value (5th stage) provided that the presence of non-persistent discontinuities and/or occurrence of swelling is confirmed by observation of rock core.

Table 2.4. Proposed Lugeon Interpretation Procedure Using the Flow Loss vs Pressure Space [19]

2.4 Tunnel Lining Selection

The needs to install lining in a waterway of hydropower plant should be carefully considered due to time-consuming and high cost [21]. Besides the time and cost disadvantage, there are 3 main reasons to use linings in hydropower tunnels and shafts, which is related to hydraulic reasons, stability reasons and containment reasons. The first step in lining design is to select the appropriate lining type according to functional, geology – hydrology, constructability and economic viability [22]. Therefore, it is possible that the design of the lining ends up using different lining systems for a different section of tunnels.

The tunnel lining selection in this sub-chapter will discuss the containment function as main focus without ignoring the other function of linings. According to Norwegian experience, unlined tunnels leak between 0.5 – 5 l/s/km [22]. On some occasion, to achieve an acceptable level of feasibility, lining installations need to be introduced to the tunnels, whose options include the following: Unreinforced concrete, reinforced concrete, a segment of concrete and steel backfilled with concrete or grout [23]. Table 2.5 shows the characteristics from several common lining types according to U.S. Army Corps of Engineers [22]. Due to the different characteristics, the selection of tunnel lining must take into account the cost-effectiveness from each lining type.

Lining Type	Characteristics
Shotcrete Lining	• Provide Ground Support • May improve leakage and hydraulic characteristics of tunnel • Protect the rock against erosion and deleterious action of water • Must be continuous and crack-free in water-sensitive ground
Unreinforced Concrete Lining	• Protect the rock from exposure • Provide smooth hydraulic surface • Acceptable for non-pressurized waterway
Reinforced Concrete Lining	• Provide support for non-uniform loads to protect against nonuniform displacement
Pipe in Tunnel	• Used in small diameter conduits to provide ground support • Used steel/concrete as the material • The void between lining and rock backfilled with lean concrete or cellular concrete
Steel Lining	• Prevent hydro jacking when internal tunnel pressure exceeds the external ground and groundwater pressure

Table 2.5. Characteristics from Several Lining Type [22].

To assess the cost-effectiveness of lining in Norway, NVE develops a cost base curve to estimate the cost for each hydropower components. This manual has been prepared as a tool for calculating average foreseeable costs in relatively quick. In this study, the cost base will be referencing into the price level 1 January 2010. According to NVE, it is important that the cost calculations are performed in such manner and a comparison of cost using different approach is recommended. Figure 2.9 shown an example of NVE cost curve for tunnel lining using steel. [24]

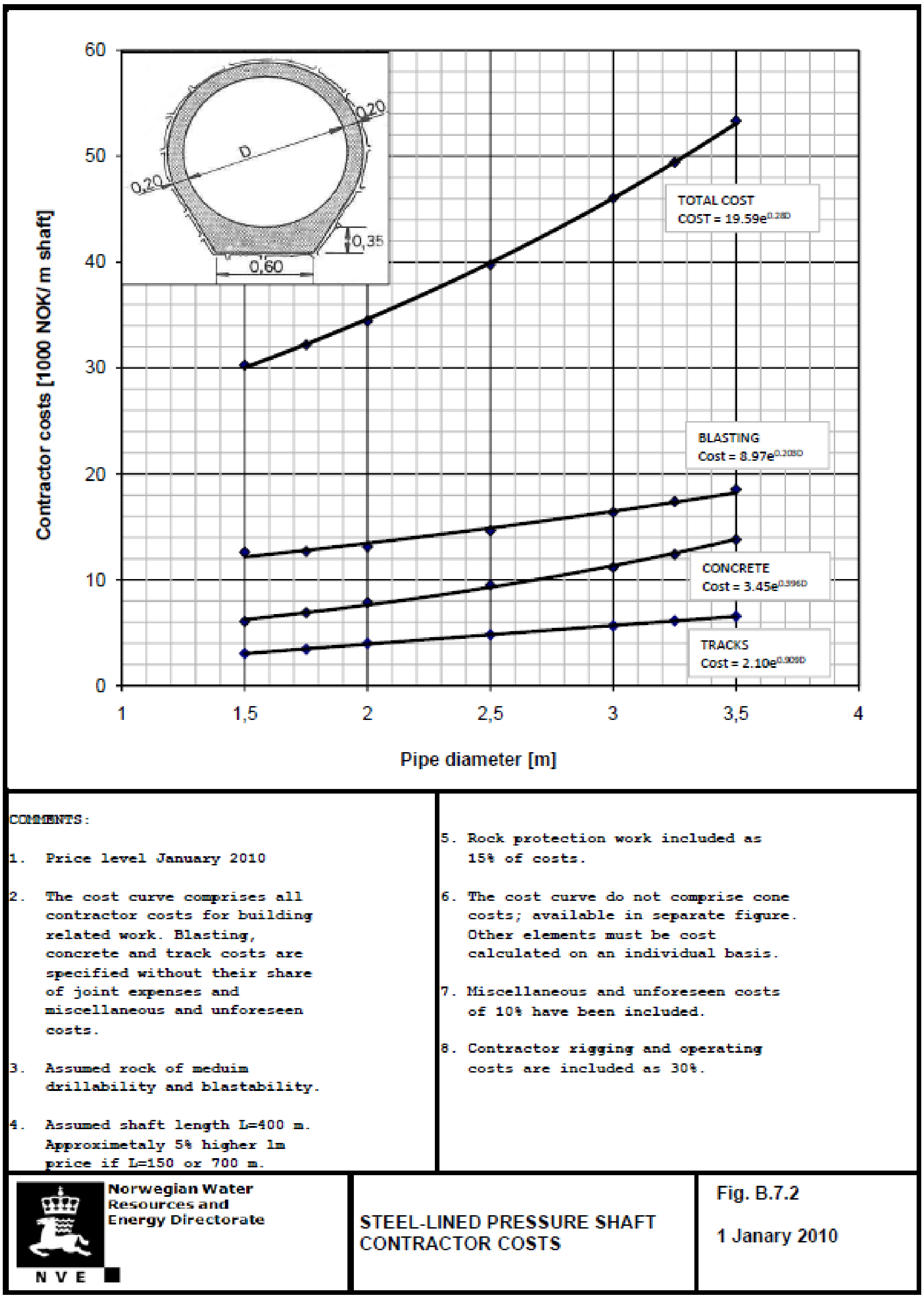


Figure 2.9. Steel-Lined Pressure Shaft Cost Curve [24]

2.5 Hydropower Potential

Hydropower is well known as one of the leading renewable sources for electricity with accounted supply 71% of all renewable energy and 16.4% of the world's electricity from all sources [25]. The main advantage in the use of hydropower as a source of energy is the flexibility to fulfill the base load demand as well as to meet the peak and unexpected demand through pump storage system. The power output from a hydropower plant depends on how much head and flow is available at the site. In term of the equation, the power output from a single hydropower plant can be obtained from Equation (10).

$$P = Q \times \rho \times g \times H \times \eta \tag{10}$$

Where:

P = Power (Watts)

Q = Flow (m^3/s)

ρ = Water Density (Kg/m^3)

g = Gravitational Constant, which is 9.81 m/s^2

H = Drop Head (m)

η = Component Efficiency (Turbine, Generator, and Transformer)

Figure 2.10 illustrates a typical hydropower scheme, general equation of power output from Equation (10). It is important to distinguish between net power output and gross power output from a hydropower power calculation. Net power production from a hydropower plant obtains by reducing the gross head with head loss, which is driven by the size of waterway, length, and roughness of waterway and operational discharge. [26]

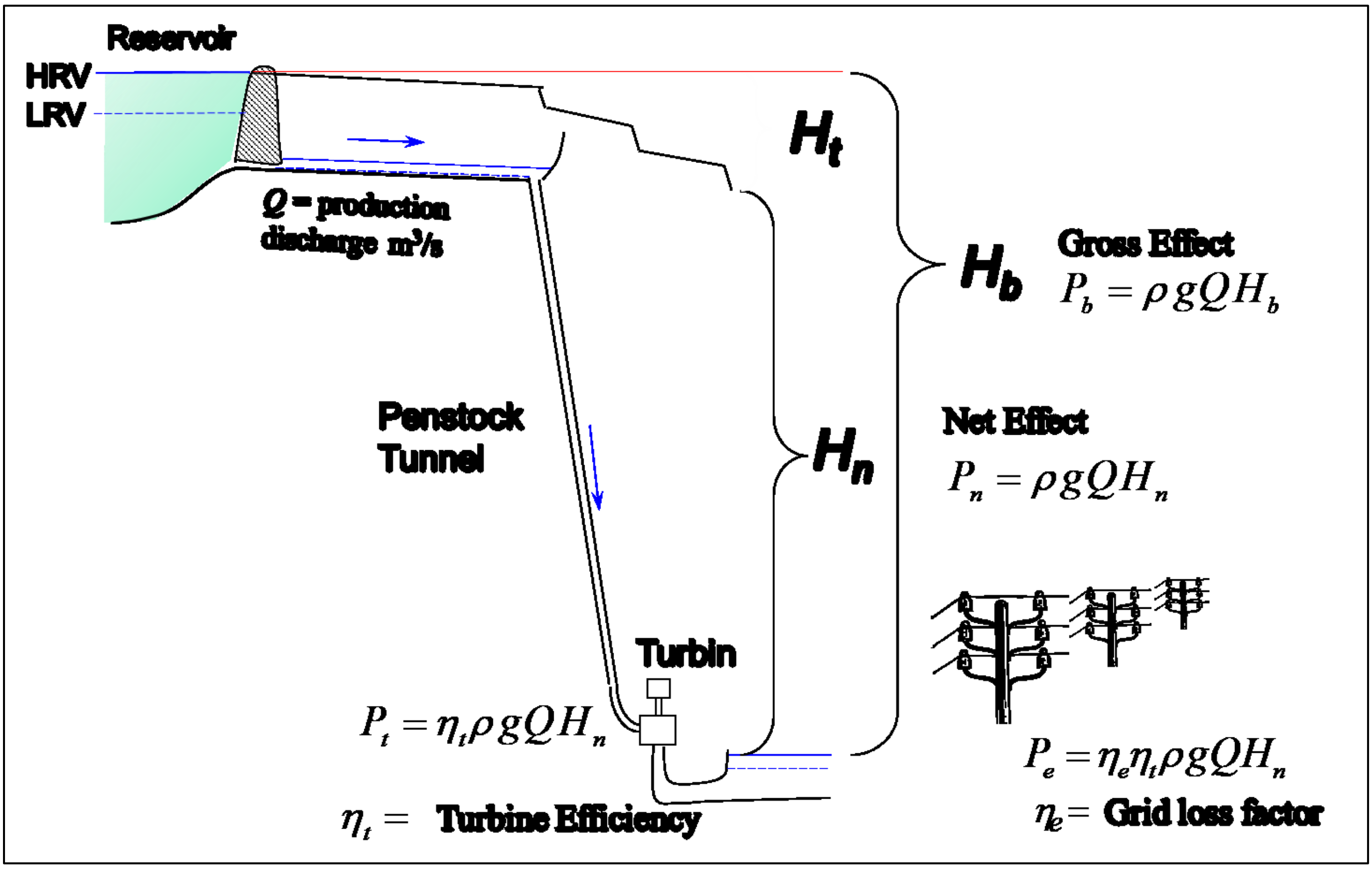


Figure 2.10. Hydropower Power Potential and Losses in Typical Hydropower Scheme [26]

In the planning stage of hydropower plants, the terms of energy equivalent (EEKV) commonly used to determine the energy generated (kWh) each m^3 of volume water. The formula to obtain EEKV from a hydropower scheme can be seen in Equation (11).

$$EEKV = \eta \times g \times \frac{H}{3600} \tag{11}$$

Where:

η = Component Efficiency (Turbine, Generator, and Transformer)

g = Gravity Acceleration (m/s^2)

H = Drop Head (m)

EEKV = Energy Potential in 1 m^3 of Water (kWh / m^3)

Chapter 3 Water Loss from Hydropower Unlined Tunnel

There's a lot of satisfaction that comes from knowing you're doing your best, and there's even more that comes when it begins to pay off.

- Sir Alex Ferguson -

The steps to assess the water loss from an unlined tunnel is illustrated in Figure 3.1. After obtaining the waterway geometry data from each hydropower plant, the waterway stage – volume curve is developed. Afterward, using the pressure drop/increase data with a stage-volume curve, the potential leakage zone, and leakage value can be calculated. Concluding with revenue evaluation and water consumption evaluation, the analysis procedure will produce 4 types of data, which are Potential leakage zone, leakage discharge, revenue loss, and water consumption change. These data will provide a basis of an argument which will be discussed in Chapter 4.

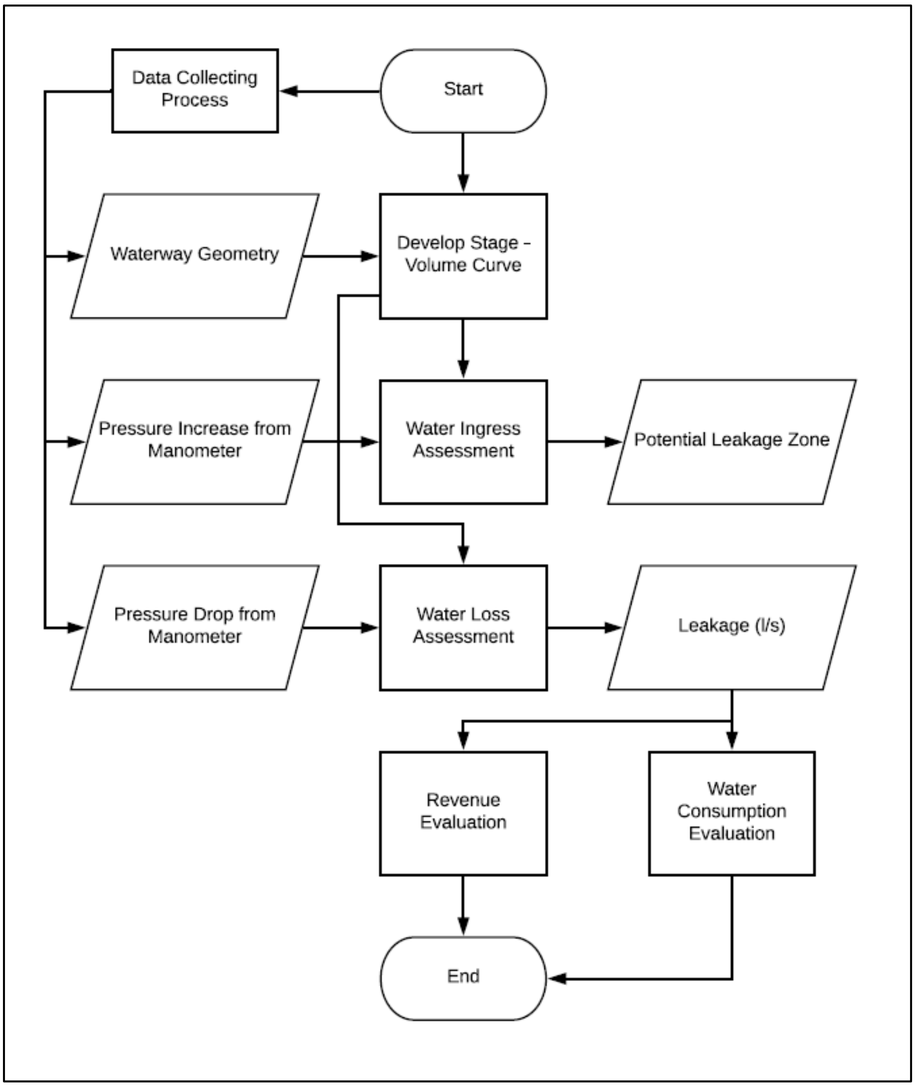


Figure 3.1. Flow Chart of Leakage Analysis in Unlined Tunnels

The first step of analysis is conducted by developing stage-volume curve to show the relationship between the stage (pressure head) and accumulative volume of water inside the tunnel. The purpose of this curve is to simplify the calculation of volume change related to the pressure head change. By using this curve, a certain pressure head value can directly be converted into the volume of water. As an example, in Figure 3.3 is shown a stage-volume curve from a typical waterway of the hydropower plant system. By plotting in this curve, it is easily determined that the volume for 200 m pressure head is 375 m^3.

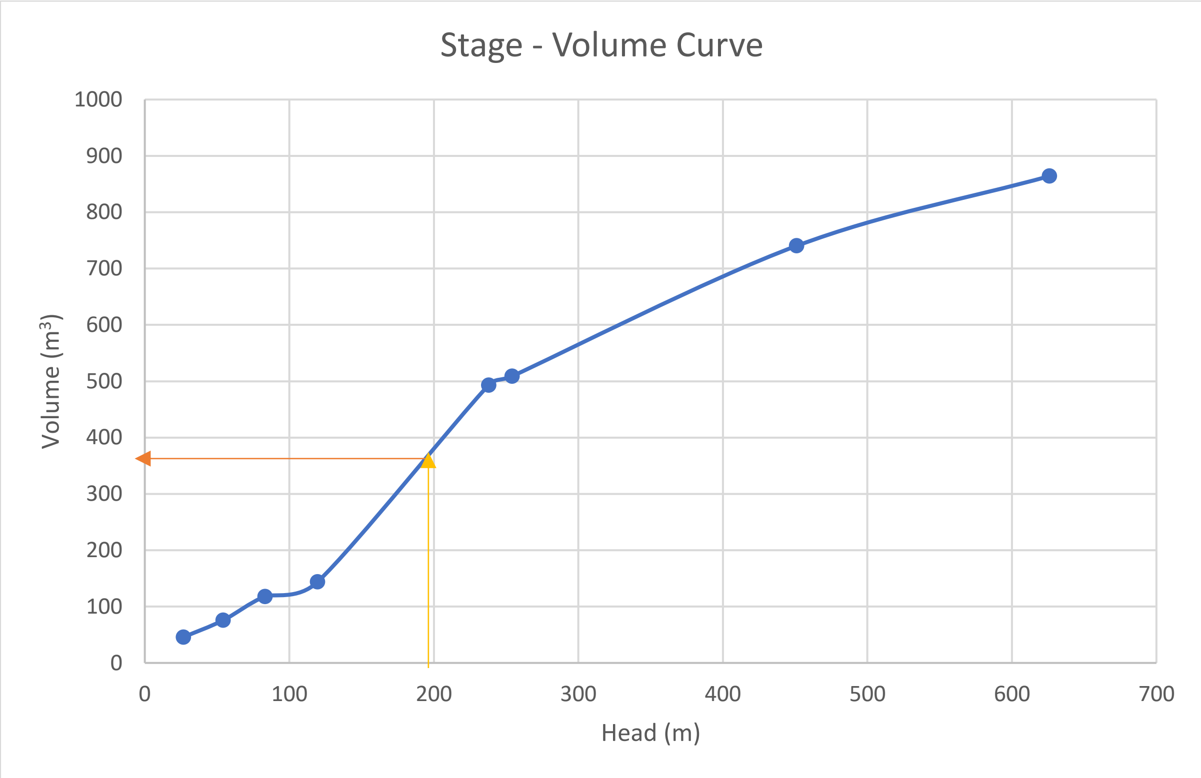


Figure 3.2. Example of Stage - Volume Curve

To develop the waterway Stage – Volume curve, a relationship between the change of pressure head (ΔH) into volume loss was established using a geometric approach as illustrated in Figure 3.3 and equation (12). To achieve the required accuracy, the waterway needs to be divided into several sections according to the geometric differences (such as the angle of inclination and diameter changes). Afterward, the accumulative volume can be calculated by sorted from the most downstream section up to the most upstream section of the waterway.

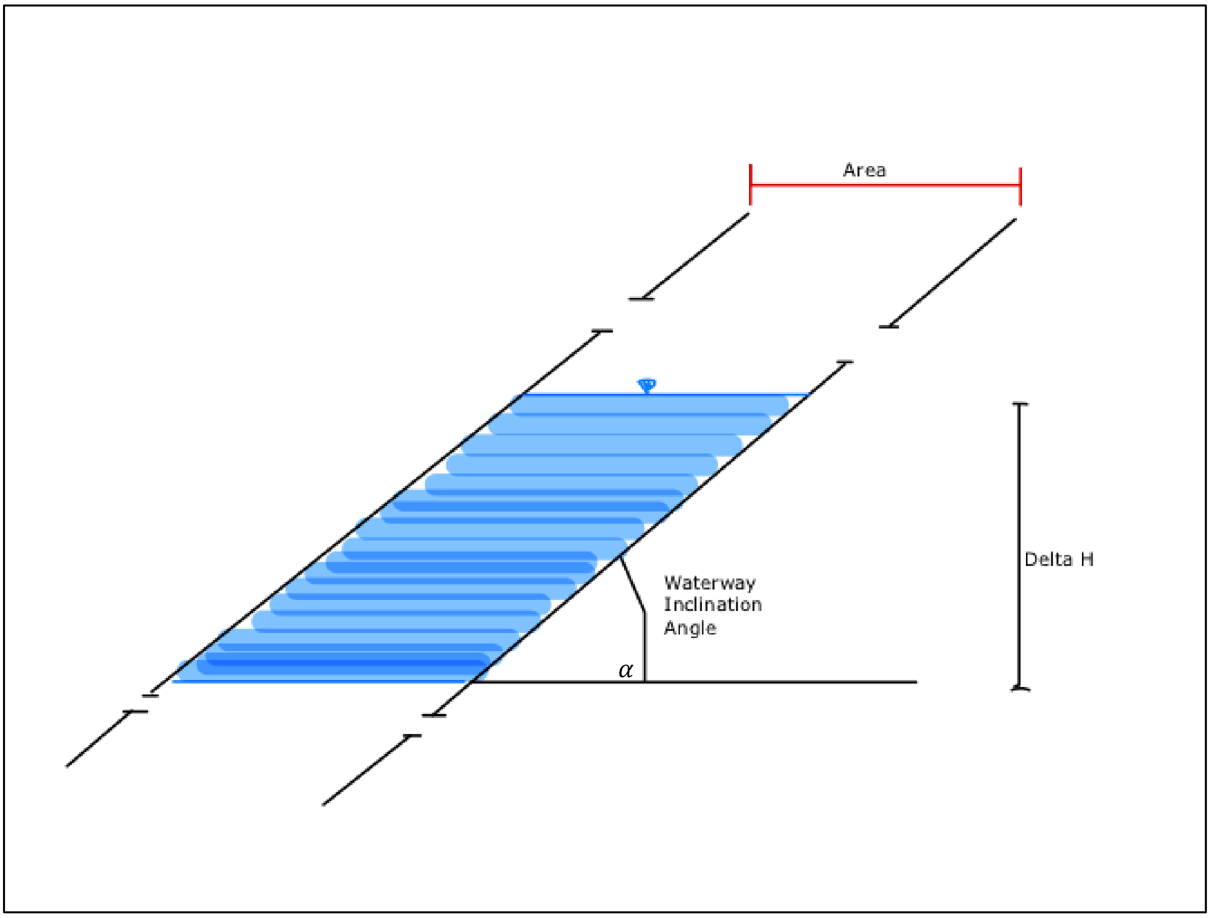


Figure 3.3. Volume - Stage Geometric Relation in Waterway

$$\Delta Volume = Waterway\ Cross\ Section\ Area \times \frac{\Delta H}{\sin \alpha} \quad (12)$$

Equipped with the stage-volume curve for each hydropower plant, the next step of analysis is to calculate the discharge of water loss and water ingress. Equation (13), which is a basic formula to calculate discharge was used to obtain the discharge value. The volume of water used in this formula is the change of volume according to the pressure change. Meanwhile, the time step is the time of pressure measurement conducted.

$$Discharge\ of\ Water = \frac{Volume\ of\ Water}{Time\ Step} \quad (13)$$

The next sub-chapter will discuss the analysis of water loss for Hatlestad HPP, Tafjord 5 HPP, Solhom HPP, Duge PSP, Saguling HPP, and Cirata HPP. From the targeted case study, only data from Hatlestad are successfully collected. The pressure measurement in Tafjord 5 was unable to be conducted due to the maintenance in Tafjord 5. For Solhom HPP, Duge PSP, Saguling HPP, and Cirata HPP the historical data of pressure measurement did not exist and due to time and distance, it is not possible to conduct the pressure measurement.

3.1 Hatlestad Hydropower Plant

Hatlestad Hydropower Plant is located in the west side of the Fjærlandsfjorden in Sogndal with 626 m gross head. According to NVE, Hatlestad has 15.5 GWh annual productions with 4.5 MW capacity and put into operation since 2018 [27]. Location of Hatlestad Power Plant and nearby power plant can be seen in Figure 3.4 and the scheme can be seen on page 65.

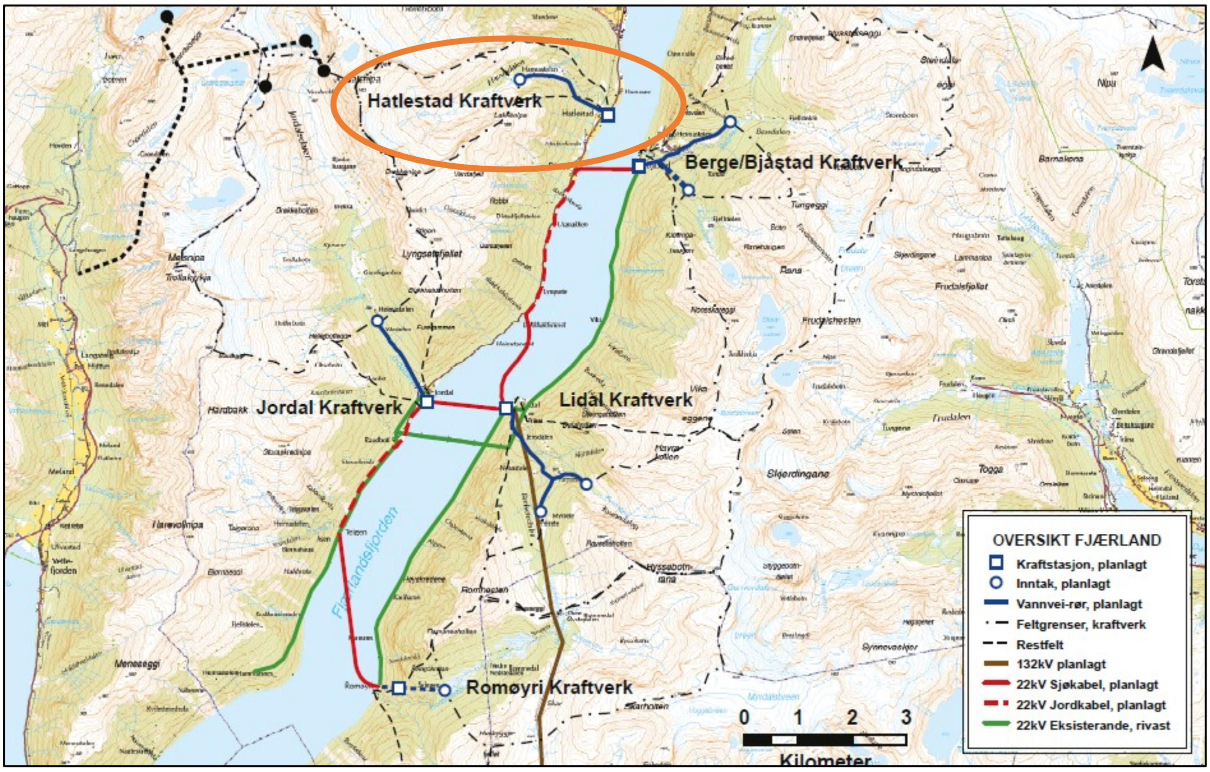


Figure 3.4. Hatlestad Powerplant Location [28]

A banana shape drill hole with 0.77 m diameter is used as an unlined pressure tunnel to carry water from intake to the powerhouse. This drill hole is also equipped with steel line (0.7m diameter) and buried penstock at downstream of a waterway. The main issue in this power plant is the occurrence of significant leakage in the waterway and influence the financial viability of Hatlestad power plant.

From the data collecting process, 8 datasets of pressure measurement (Table 3.1) was obtained in the span of time December 2018 – March 2019. The dataset consists of water ingress data (an increase of piezometric head during static condition) and water loss data (decrease of the piezometric head during static condition) derived during pre-rehabilitation of waterway and post-rehabilitation of the waterway. Furthermore, A drill hole inspection data was obtained through a video recording alongside the unlined section of the waterway.

Dataset	Date of Measurement	Water Ingress Data	Water Loss Data	Notes
1	25/27 March 2019	√	-	Post Rehabilitation; Close Valve Filling
2	26 March 2019	√	-	Post Rehabilitation; Close Valve Filling
3	27 March 2019	√	-	Post Rehabilitation; Open Valve Filling
4	25/28 March 2019	√	-	Post Rehabilitation; Close Valve Filling
5	11/13 December 2018	-	√	Pre-Rehabilitation; Open Valve Filling
6	28 March 2019	-	√	Post Rehabilitation; Close Valve Filling
7	29 March 2019	√	√	Post Rehabilitation; Open Valve Filling
8	7 May 2019	-	√	Post Rehabilitation; Close Valve Filling; Saturated Ground

Table 3.1. Hatlestad Dataset Information

3.1.1 Drill hole Inspection

An inspection using video from inside the drill hole was conducted in Hatlestad HPP. Based on the video, 6 major sources of water ingress in the form of joints are identified and marked in the longitudinal drawing of Hatlestad power plant (Figure 3.5) with the dip angle for each joint shown in Table 3.2. It needs to be noted that the determination of the joints and its dip angle only uses the video as the source, without the assessment in the field. Therefore, the accuracy of the joints needs to be verified in further study.

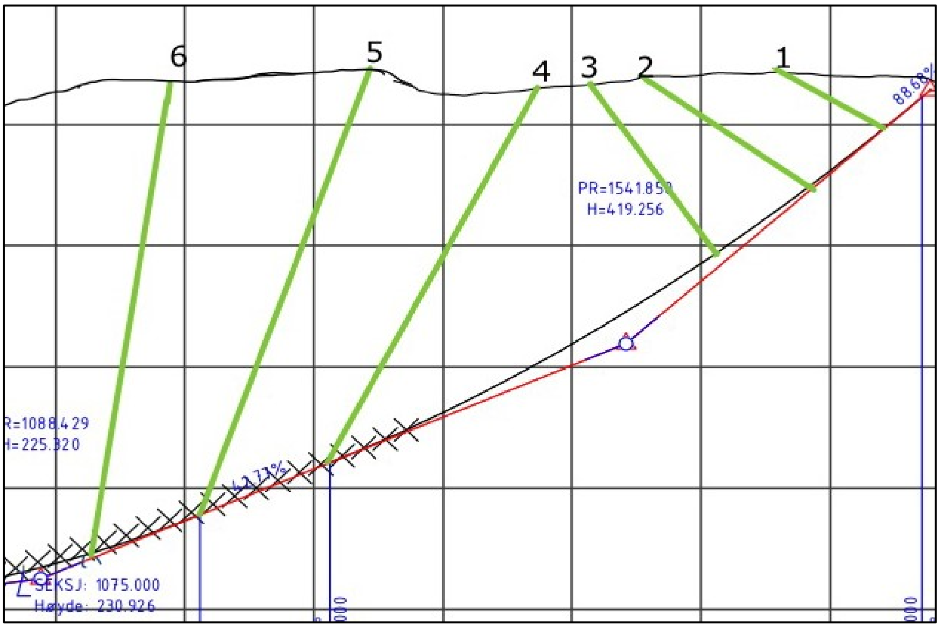


Figure 3.5. Approximate Joint Orientation in Hatlestad HPP Based on Video Recording.

Potential Joint Set	Dip Angle
1	30°
2	40°
3	65°
4	60°
5	70°
6	80°

Table 3.2. Dip Angle of Hatlestad Potential Leakage Zone

3.1.2 Hatlestad Stage – Volume Curve

According to its longitudinal section, Hatlestad Hydropower Plant needs to be divided into 8 Section (Table 3.3 and Figure 3.6) with each section to have a different slope angle or diameter of the drill hole. The accumulative volume calculation for each section is shown in Table 3.4, and as an illustration, the stage – volume curve for Hatlestad Hydropower Plant is illustrated in Figure 3.7. It needs to be noted that this approach has a tolerable error due to several factors, such as tunnel roughness and reading of the drawing.

Section	Slope (m/m)	Station (m)	Elevation (masl)
1	0.23	104	26.76
2	0.378	178	54.09
3	0.272	266	83.11
4	0.65	346	119.52
5	0.131	1106	237.92
6	0.431	1150	254.04
7	0.431	1546	450.99
8	0.875	1786	626.01

Table 3.3. Hatlestad Waterway Section

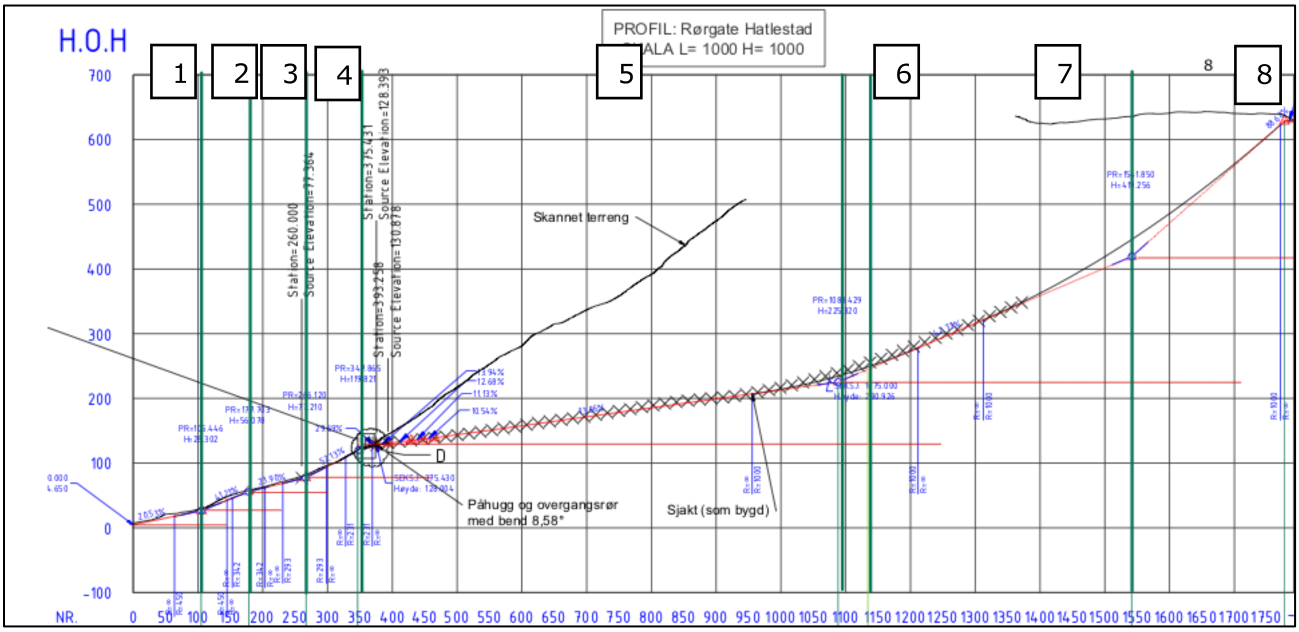


Figure 3.6. Hatlestad Waterway Section

Section	Slopes (Deg)	Pipe Diameter (m)	A (m^2)	Head	Delta Head	Delta Volume (m^3)	Accumulative Volume (m^3)
1	12.99	0.7	0.385	26.76	26.76	45.82	45.82
2	20.73	0.7	0.385	54.09	27.33	29.73	75.55
3	15.26	0.7	0.385	83.11	29.02	42.46	118.01
4	33.02	0.7	0.385	119.52	36.41	25.72	143.73
5	7.50	0.7	0.385	237.92	118.4	349.42	493.16
6	23.36	0.7	0.385	254.04	16.12	15.65	508.81
7	23.36	0.77	0.46585	450.99	196.95	231.43	740.25
8	41.19	0.77	0.46585	626.01	175.02	123.82	864.06

Table 3.4. Stage - Accumulative Volume Calculations

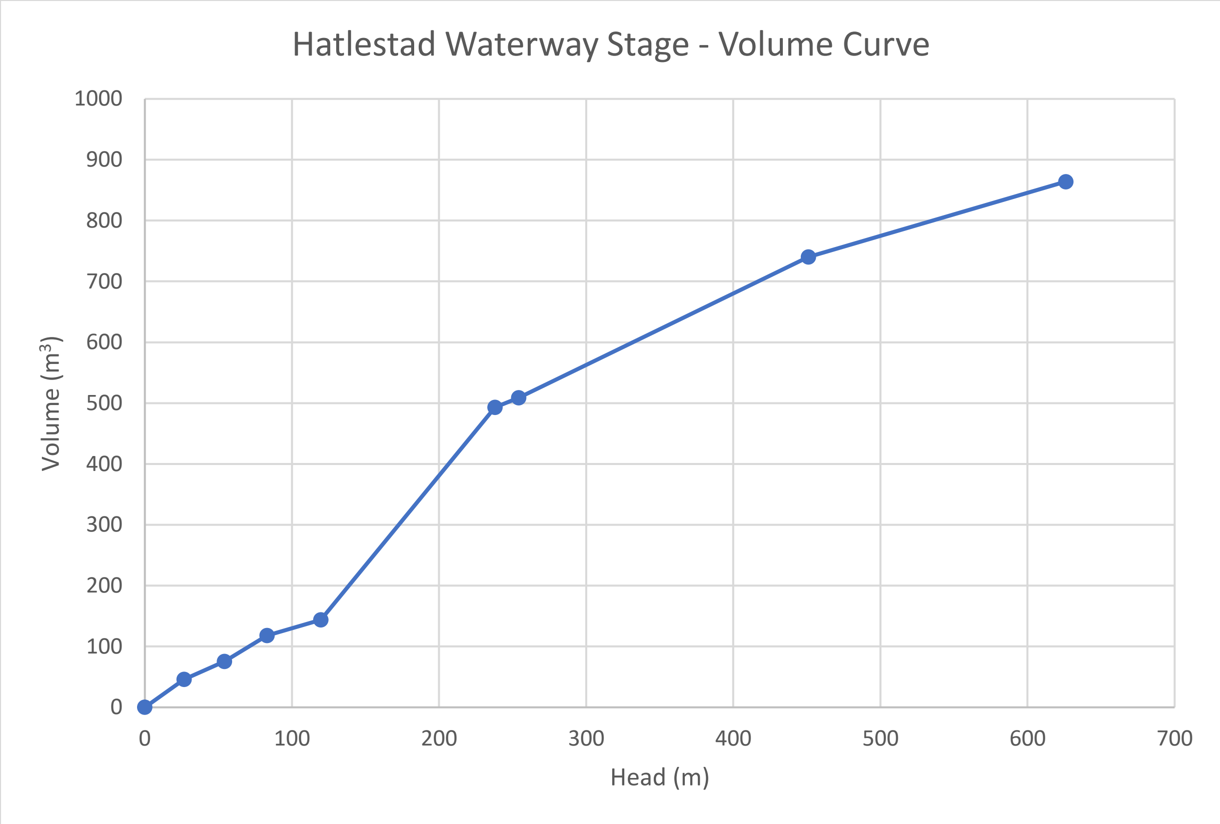


Figure 3.7. Hatlestad Hydropower Plant Waterway Stage – Volume Curve

3.1.3 Water Ingress Data

Water ingress or water entering the tunnel system in Hatlestad can be obtained by measuring the increase of piezometric head over time. 5 Water ingress data with different head elevation available for Hatlestad for the purpose, to identify the location of the leakage in the pressure tunnel. The data of pressure measurement of Hatlestad can be seen in Appendix B and the summary of results presented in Table 3.5.

Dataset	Measurements Date					
1	25/27-March-19	Head (m)		120	225	280
		Water Ingress (l/s)		3.360	4.530	0.164
2	26-Mar-19	Head (m)		125	263.75	285
		Water Ingress (l/s)		3.329	4.170	0.289
3	27-Mar-19	Head (m)		267	286	
		Water Ingress (l/s)		1.706	0.809	
4	25/28-Mar-19	Head (m)		121.3	262.6	286.6
		Water Ingress (l/s)		3.45	3.81	0.23
7	25/28-Mar-19 (11.00 -13.00)	Head (m)		539.32	622.87	
		Water Ingress (l/s)		19.651	25.468	

Table 3.5. Hatlestad Water Ingress

In dataset 3 and dataset 7 (yellow color), the data shows the filling rate due to the opening of the intake valve and not the natural filling from the rock mass, therefore it is not included in the further analysis. Figure 3.8 illustrates the water ingress in Hatlestad waterway and it shows high-value water ingress before reach 270 m elevation in comparison to the elevation afterward. In theory, the reason for the decrease is due to the leaking zone is already submerged with water and the internal pressure of the drill hole already equivalent with the groundwater pressure. Therefore, the groundwater around the drill hole is unable to enter the system. Figure 3.9 illustrates the change of drill hole internal pressure and its effect on water ingress.

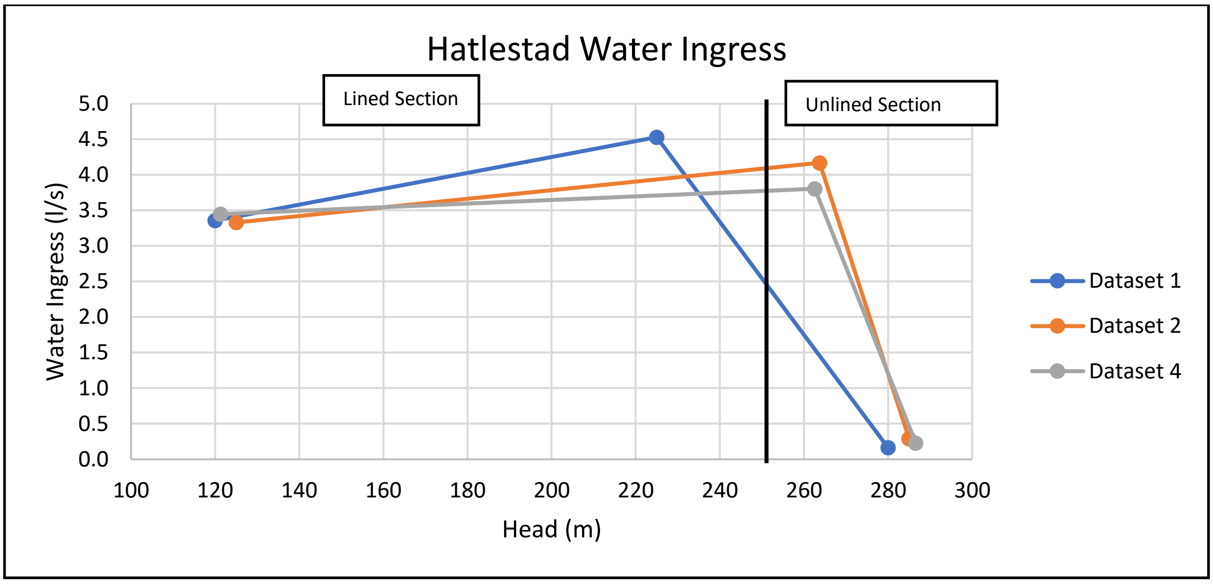


Figure 3.8. Hatlestad Water Ingress

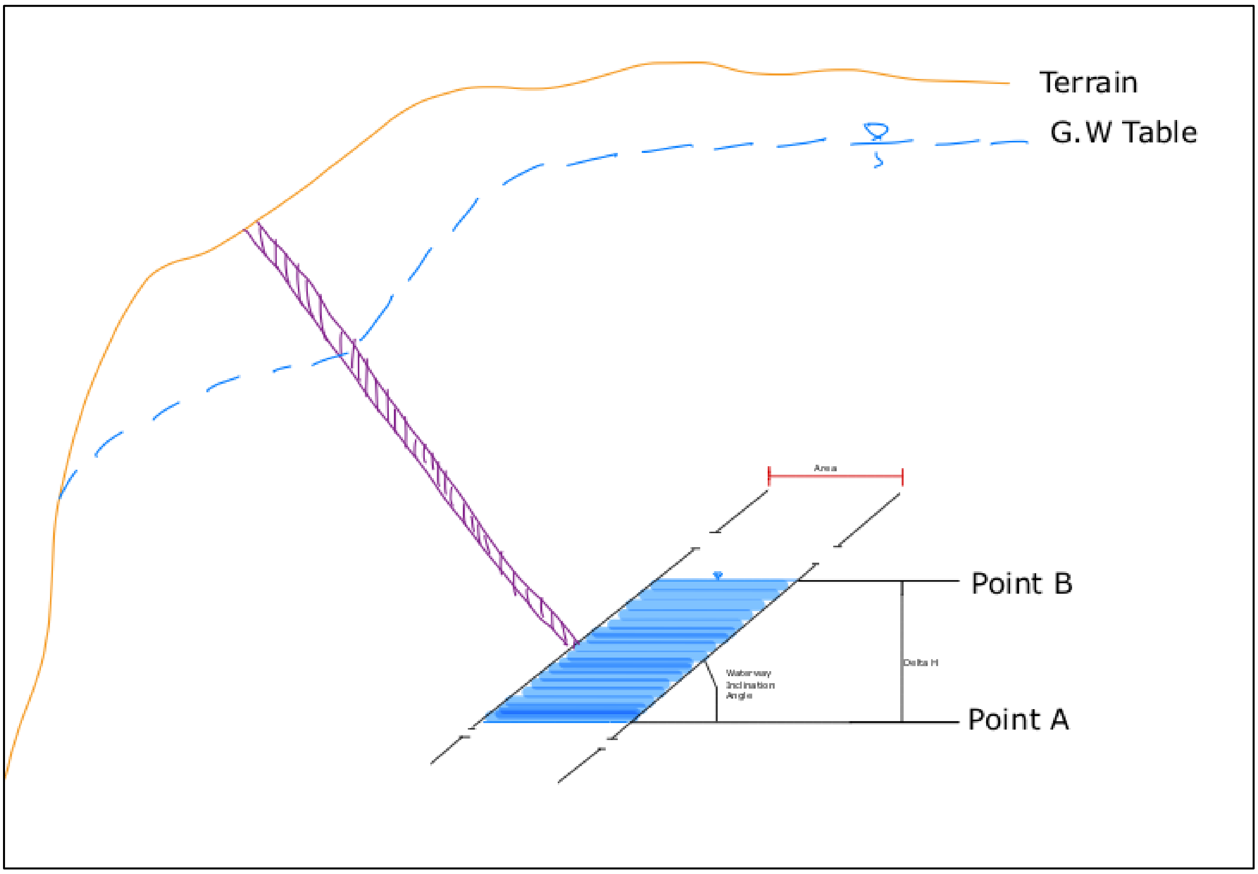


Figure 3.9. Groundwater - Internal Pressure Effect on Water Ingress

Internal pressure for a water tunnel is driven by the water level inside the tunnel. As an example, with the water level in Point A, the intersection of joint and tunnel will be zero. Therefore, the groundwater will enter the tunnel system driven mainly by gravity forces. Contrary, when the water level inside the tunnel located in Point B, the pressure in the intersection of joint and tunnel equalizes with the groundwater pressure. Therefore, with the water level at Point B, water ingress will stop. With relatively low water ingress value after 270 m head, it can be concluded that the source of water ingress is located after the end of still lining (Black dot in Figure 3.10).

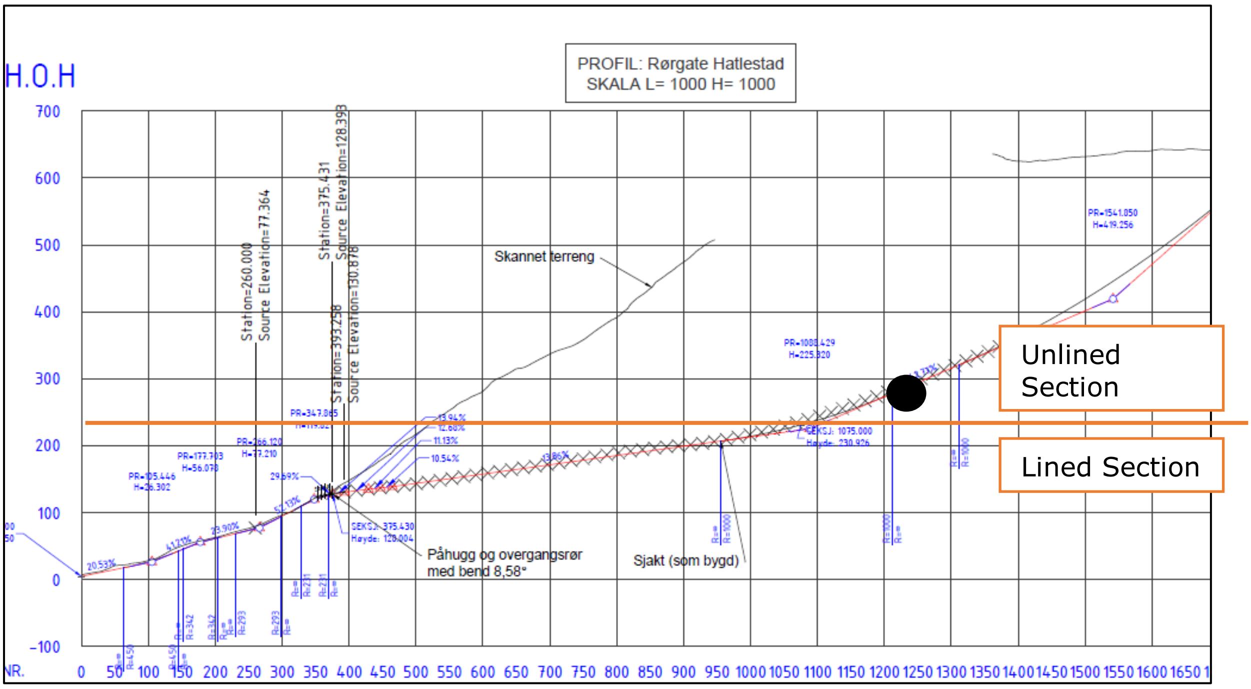


Figure 3.10. Source of Water Ingress

3.1.4 Water Loss Data

Leakage from tunnel system (water loss) is indicated from the pressure drop in the manometer measurement during static condition over time. Figure 3.11 illustrates the drop of pressure due to water loss in a tunnel system. Using the principle that is already discussed earlier in this chapter and in the literature studies, Hatlestad power plant conducted 4 dataset measurement for assessing leakage from its waterway and the results presented in Table 3.6 and Figure 3.12.

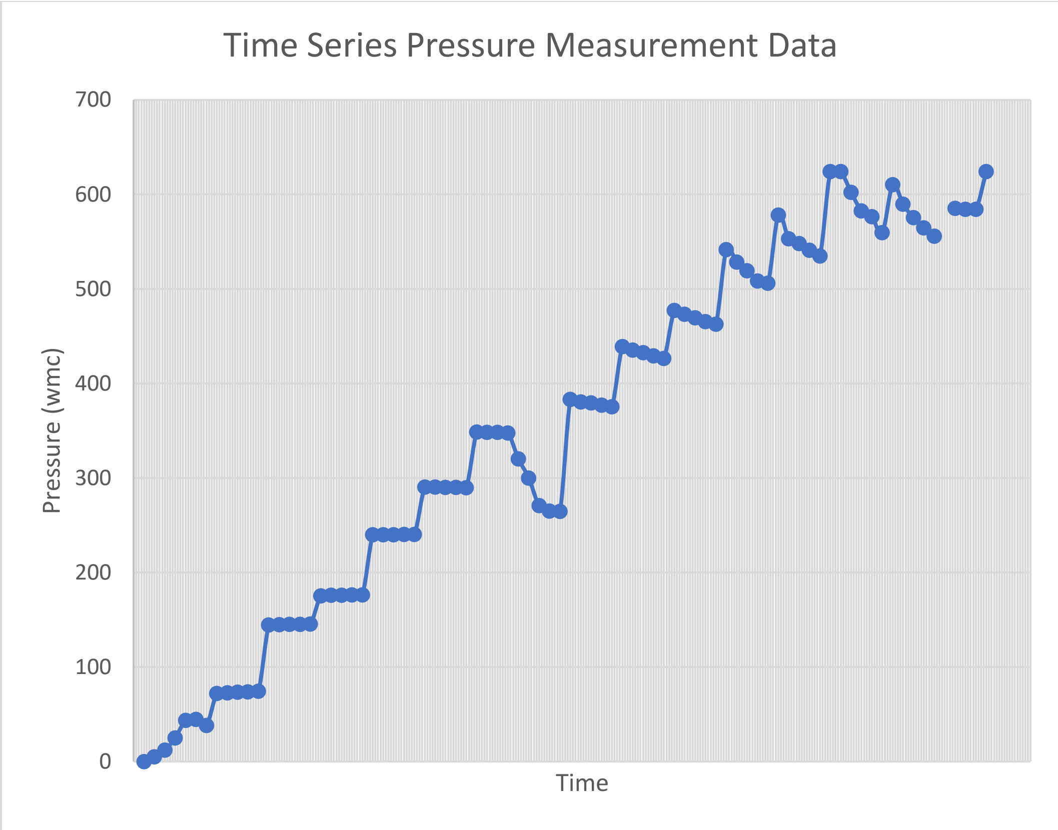


Figure 3.11. Water Loss Indication

Dataset	Measurements Date													
5	11/13-Dec-2018	Head (m)		44.6	290.4	348.6	383.1	438.9	477.3	541.7	578.1	624.3	610.3	585.1
		Water Loss (l/s)		13.30	0.59	1.44	7.27	11.22	8.55	21.11	25.59	34.78	32.13	1.31
6	28-Mar-19	Head (m)		624	623									
		Water Loss (l/s)		15.60	24.60									
7	29-Mar-2019 (13.00)	Head (m)		623.85	622.19									
		Water Loss (l/s)		19.63	25.44									
8	7-May-19	Head (m)		621.8										
		Water Loss (l/s)		3.61										

Table 3.6. Hatlestad Water Loss Summary

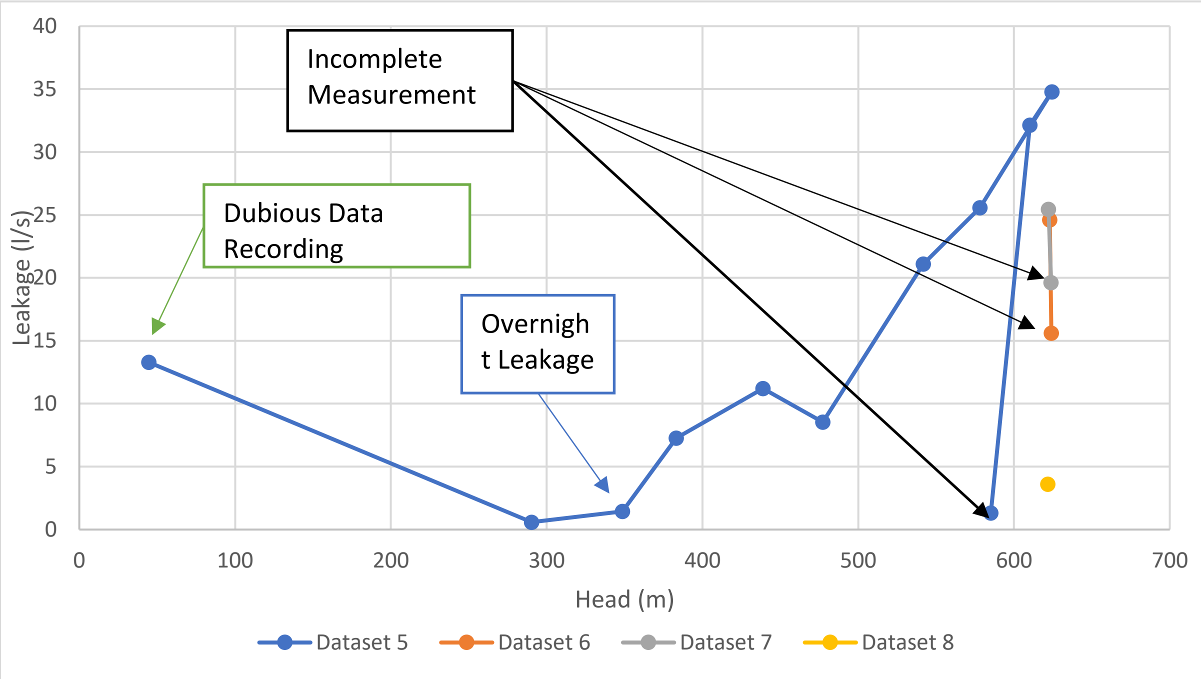


Figure 3.12. Hatlestad Water Loss Summary Graph

Several datasets need to be excluded from the analysis due to incomplete measurement (black color), overtime leakage measurement which leads into an inaccurate value (blue color) and dubious data recording[1] (green color), the normalized result is shown in Figure 3.13. Hatlestad waterway implements a rehabilitation process to its waterway, and dataset 5 was obtained before the rehabilitation process. Meanwhile, dataset 6,7 and 8 were collected after the rehabilitation process. In dataset 8 the leakage value is much lower compare to dataset 6 and 7. The main difference is that the measurement for dataset 8 was conducted during the operational time. Therefore, the groundwater condition has higher water content compared to circumstances during measurement of dataset 6 and dataset 7.

Rehabilitation of waterway significantly reduces the water loss from the waterway from 32.13 l/s until 24.60 l/s; the evaluation of lost revenue will be discussed in subchapter 4.1. The increase of head significantly increases the leakage from the drill hole in Hatlestad with the change as shown in Figure 3.13. There are two arguments regarding inconsistency between 450 – 550 m elevation. First is the possibility of incorrect data recording in which the results are impossible to be validated due to the measurement on that certain elevation only conducted once. The second argument is due to the interpolation procedure using its stage-volume curve was uses two points linear interpolation, which is an incorrect assumption since the drill hole is in banana shape.

[1] Dubious data recording due to inconsistency between leakage calculation and notes on the report. Moreover, the measurement in other steel lining zone indicates zero water loss.

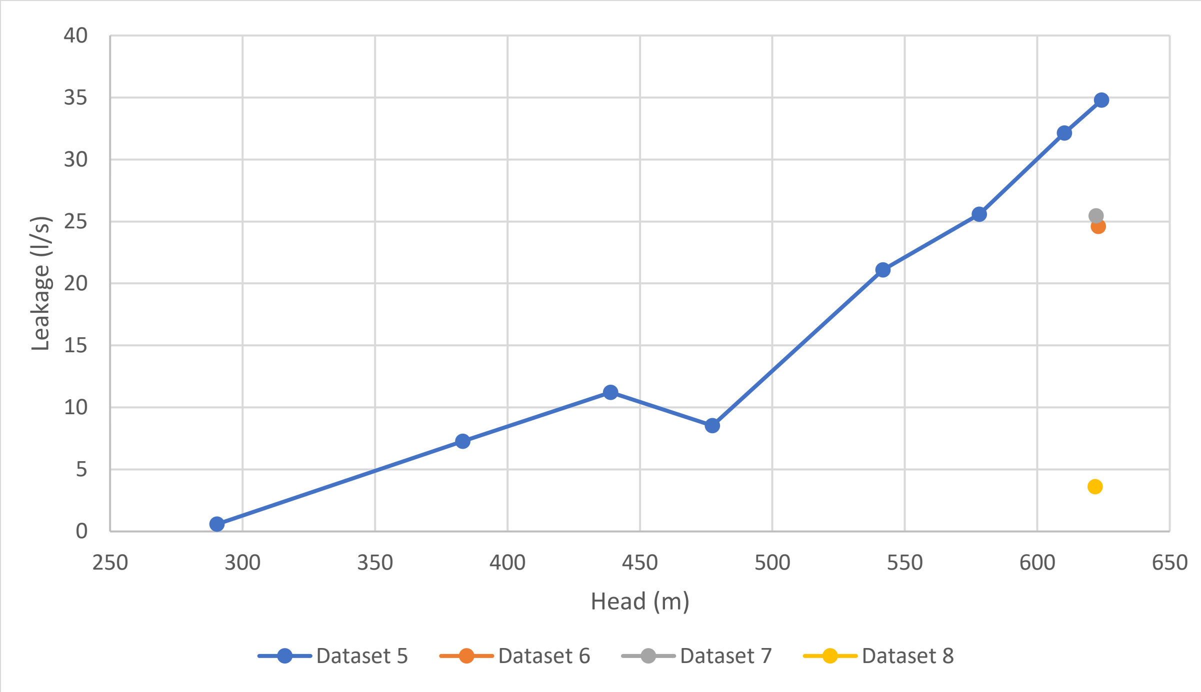


Figure 3.13. Hatlestad Water Loss - Head Relationship

Water loss – pressure head relations curve shown in Figure 3.21 shows similarity with "pressure loop" as the results of lugeon test interpretation proposed by Quinones-Rozo [19]. In Hatlestad HPP water loss results, there is two possible interpretation of "pressure loop" for dataset 5 (Figure 3.14 and Figure 3.15). The difference between these two interpretations is due to inconsistency data on point 3 (P3) and point 4 (P4), which impossible to identify which point represents the true leakage value as the measurements were only conducted once.

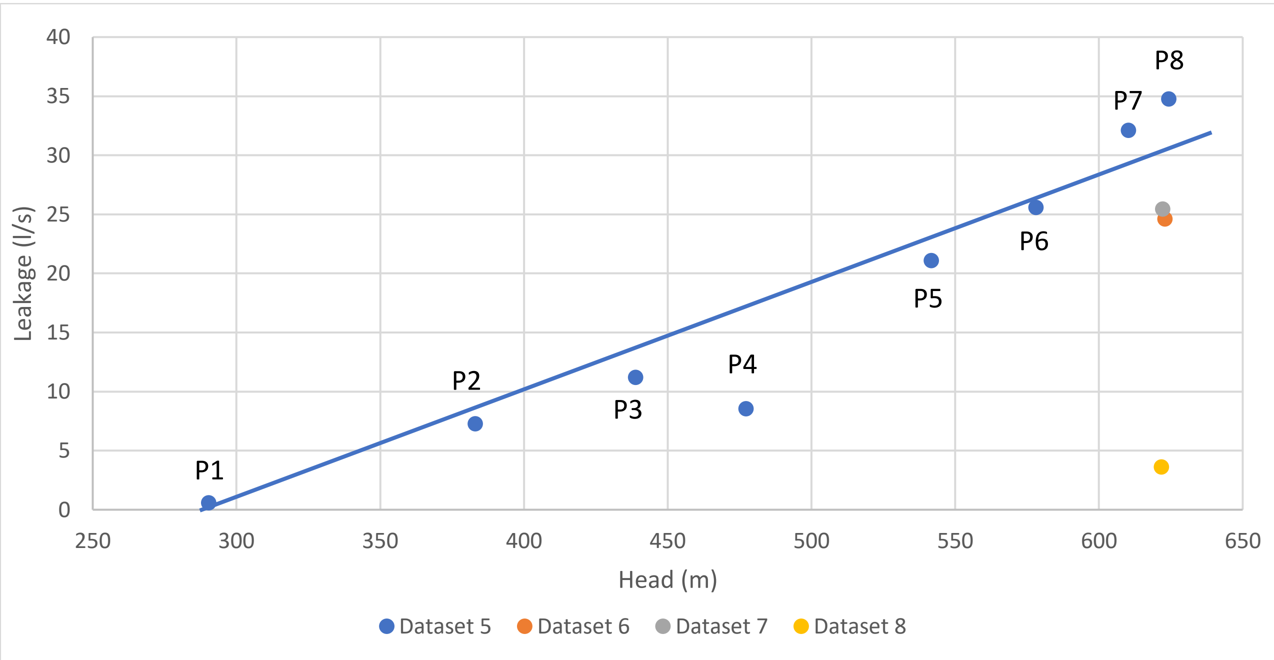


Figure 3.14. Interpretation of Water Loss Data (Possibility 1)

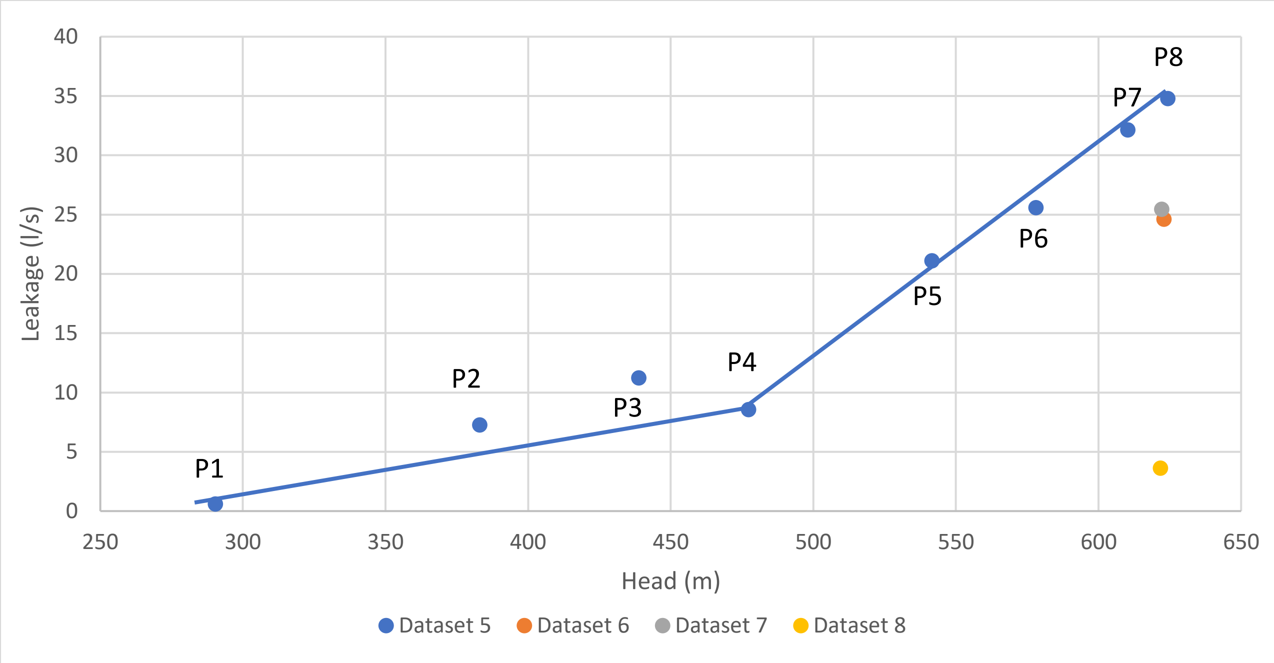


Figure 3.15. Interpretation of Water Loss Data (Possibility 2)

Referring to reference [19], the first possibility of Hatlestad water loss data interpretation shows the laminar behavior, which means the rock mass has low hydraulic conductivity and seepage velocities are relatively small. In contrary, in the second possibility (Figure 3.15) the behavior is between dilation, wash-out or void filling. The exact behavior for possibility 2 is unable to be identified due to the measurement only create half of the "pressure-loop" of Lugeon test. Nonetheless, due to interpretation 2 indicates a critical condition in regards to water loss, the discussion will focus on the second interpretation.

There are two allegations on the behavior of water loss in Hatlestad based on possibility 2. The first allegation is the annulus in the transition zone between unlined – lined tunnel unable to withstand the water pressure after around 450 m elevation and lead into water loss from the tunnel system through the annulus. The second allegation is due to the identified joint 6 which discussed in sub-chapter 3.1.2 and supported by water ingress analysis in sub-chapter 3.1.3. Moreover, based on desk studies a brook (Figure 3.16) was identified which indicate a high groundwater table around the waterway.

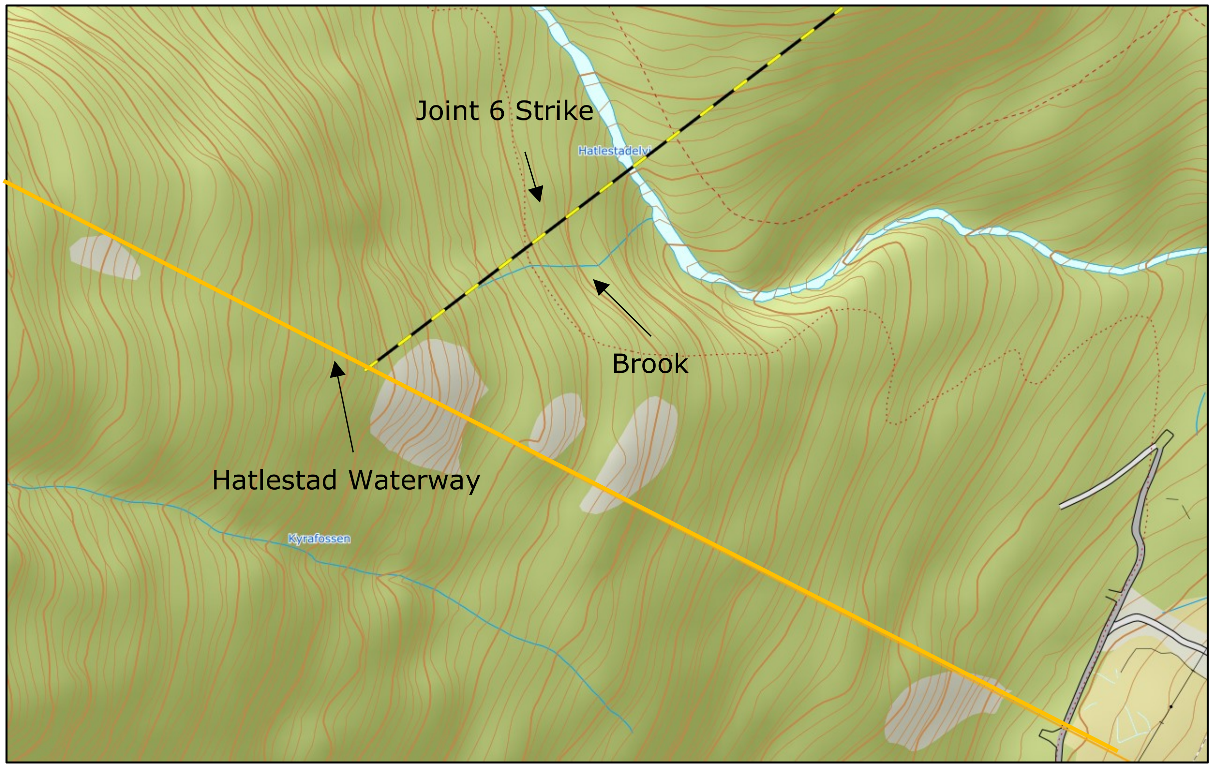


Figure 3.16. Approximation Strike of Joint 6, Brook and Hatlestad Waterway

Figure 3.18 illustrates the cross-section profile from the approximate strike for joint 6, alongside the location of Hatlestad waterway (260 m). In this location, the shortest distance of rock cover (L) was assessed using Norwegian Rule of Thumb according to Bergh-Christensen and Dannevig as shown in equation (14) using the input as follow:

H = 629 – 260 =369 m

γ = 2750 kg/m^3 (Figure 3.17)

γ_w = 1000 kg/m^3

$\beta = \tan^{-1}\frac{161}{233} = 34.64^o$

L $=\sqrt{29.41^2 + 178.5^2}$=180.9 m

$$L > \gamma_w \times H/\gamma \times \cos\beta \quad (14)$$

$$180.9 > \frac{1000 \times (629 - 260)}{2690 \ \times \cos 34.64}$$

180.9 > 166.72

$$FS = \frac{180.9}{166.72} = 1.07 \quad (15)$$

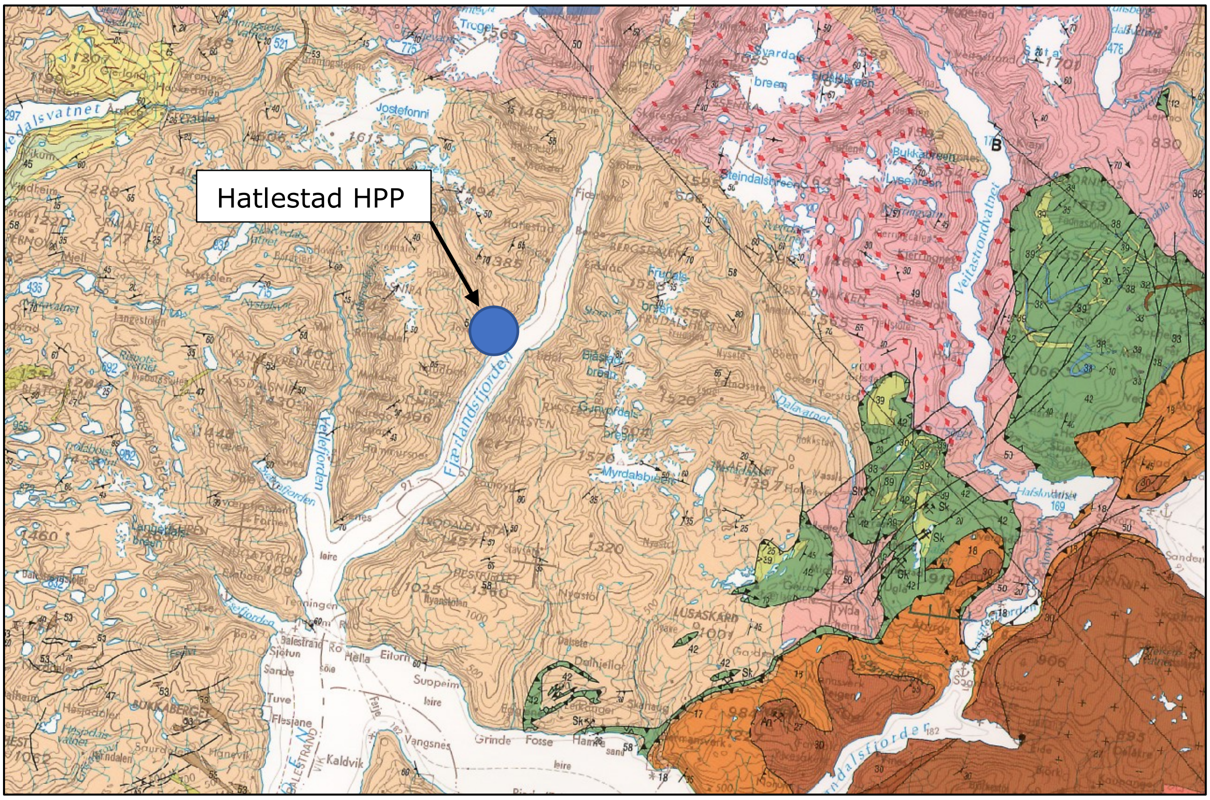


Figure 3.17. Rock Type in Hatlestad HPP Location According to NGU[2] [17]

Initial assessment using the Norwegian Rule of Thumb indicates that the shortest distance between terrain and Hatlestad borehole (L) is adequate. However, the factor of safety is relatively low with a value of 1.07. Considering the assumption in the Norwegian rule of thumb did not take into account stress anisotropy and the uncertainty of several input data such as rock mass density, with a factor of safety 1.07, it is hard to be sure the rock cover is adequate to prevent leakage.

Considering the conclusion on Hatlestad water loss is mainly based on water loss data during pre-rehabilitation, it is wise if arguments are verified using reasonable data from post-rehabilitation. Nevertheless, conducting pressure drop measurement is a time-consuming procedure and stopping the production time is unavoidable. Consequently, it is difficult to conduct measurements due to financial loss during measurement.

[2] Rock type in Light Brown Color Categorize as Quartz Diorite / Tonalite or Trondheimlite with Density approximately 2690 kg/m3 [21].

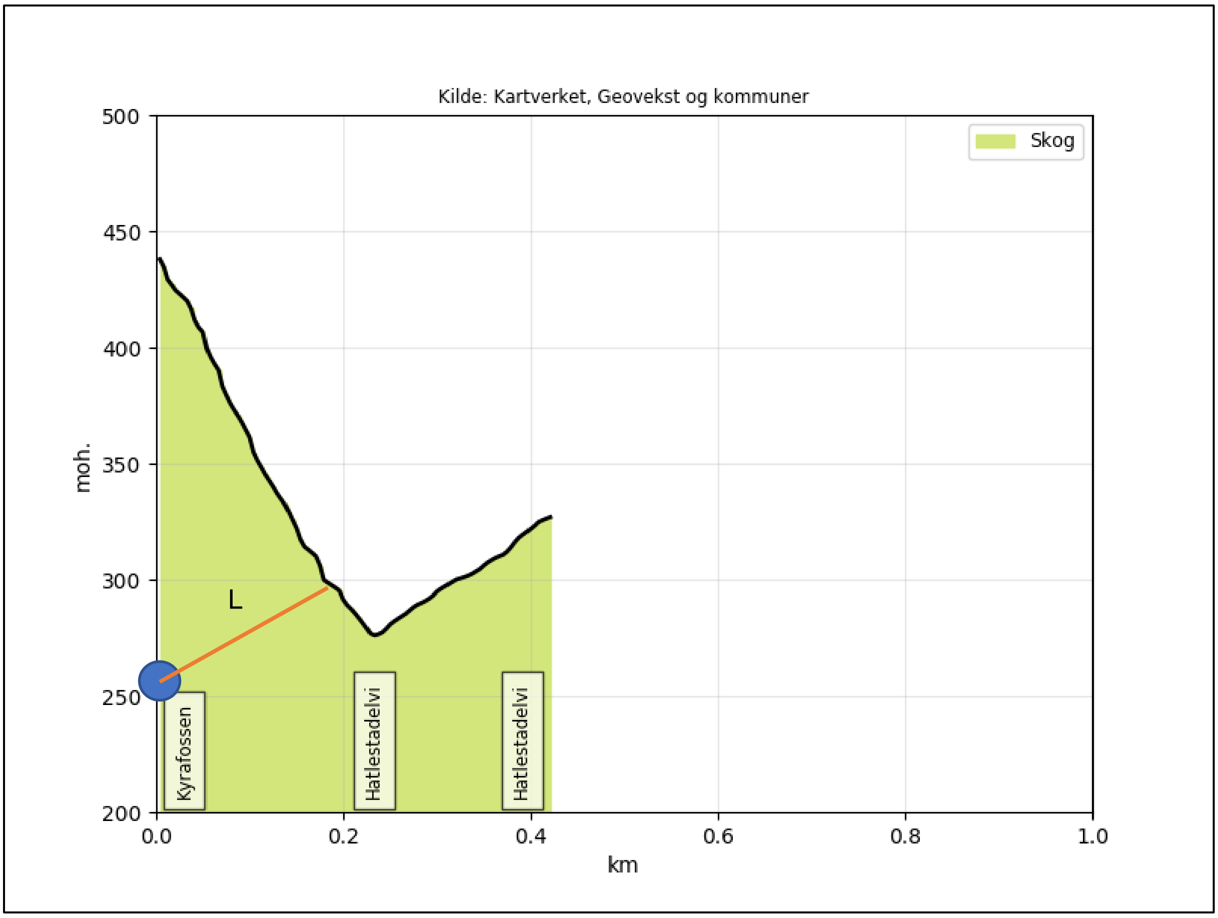


Figure 3.18. Hatlestad Drill Hole Position on Joint 6 Cross Section

Assessment using the Norwegian rule of thumb was conducted alongside the pressure tunnel. In Figure 3.19, the section with green color indicates tunnel section that did not fulfill the criterion. On this basis, the lining installations supposed to be adequate to reinforce the confinement. On the other hand, leakage from the unlined pressure tunnel still happens in Hatlestad HPP.

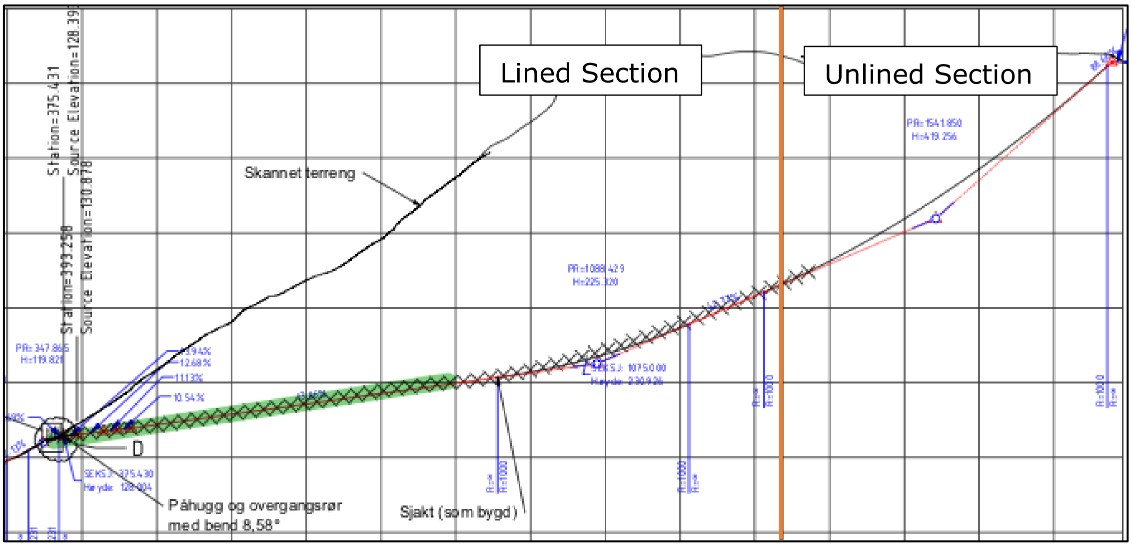


Figure 3.19. Hatlestad Norwegian Rule of Thumb Assessment

3.2 Tafjord 5 Hydropower Plant

Tafjord 5 is a hydropower plant owned by Tafjord Kraft, located in Norddal municipality, Møre og Romsdal. Tafjord 5 is in operation since 1982 with a discharge capacity of 12.8 m^3/s to generate a maximum capacity of 82.7 MW power [27]. According to NVE, Tafjord 5 generates 418.3 GWh average annual production (1981 – 2010 Hydrological data) with an energy equivalent of 1.795 kWh / m^3 and 822.3 m gross head [27]. Due to massive water pressure, Tafjord 5 needs to be located 1,000 m inside the mountains [29].

Figure 3.20 shows the scheme of Tafjord 5 hydropower plant. Zakarias lake functions as a tail reservoir with intake located in Smette Lake (709 m head) and Brusebotn Lake (823 m). In addition, a diversion intake from Upper Hulderkopp lake was constructed. Longitudinal profile of Tafjord 5 presented in Appendix D – Tafjord V Hydropower (page 74). The main focus to assess water loss in Tafjord 5 is in the lower section waterway (after the meeting of Brusebotn waterway and Smette lake). The main reason is to simplify and save time for conducting a pressure measurement. In addition, the author decides to focus more on the effect off surge chamber towards water loss in Tafjord 5 waterway. Nonetheless, it is unfortunate the data collecting process from Tafjord 5 is not complete until the deadline of this study. Still, the data that manages to be collected by the author is presented in this chapter with a purpose to provide the basis for future studies in water loss from unlined tunnels studies.

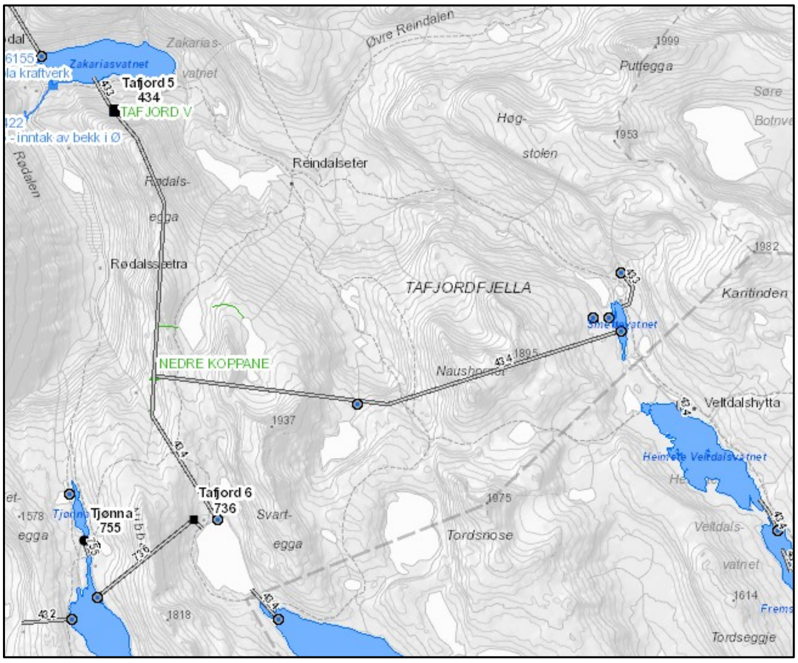


Figure 3.20. Tafjord 5 Top View [27].

To obtain the water loss from Tafjord 5, four stations of pressure measurement is proposed to Tafjord Kraft as the owner of Tafjord 5 (Table 3.7). The location of each station is illustrated in Figure 3.23 with station 1 and station 2 aim to assess the surge chamber leakage and station 3 and 4 assessing the leakage in the lower waterway.

Station	Elevation	Head (m)	Stopping Time (min)	Information
1	450	8.5	20	Leakage Measurement in Surge Chamber
2	480	38.5	20	
3	620	178.5	20	Leakage Measurement in Lower Waterway
4	1060	618.5	20	Leakage Measurement in Shaft

Table 3.7. Tafjord 5 Proposed Pressure Measurement

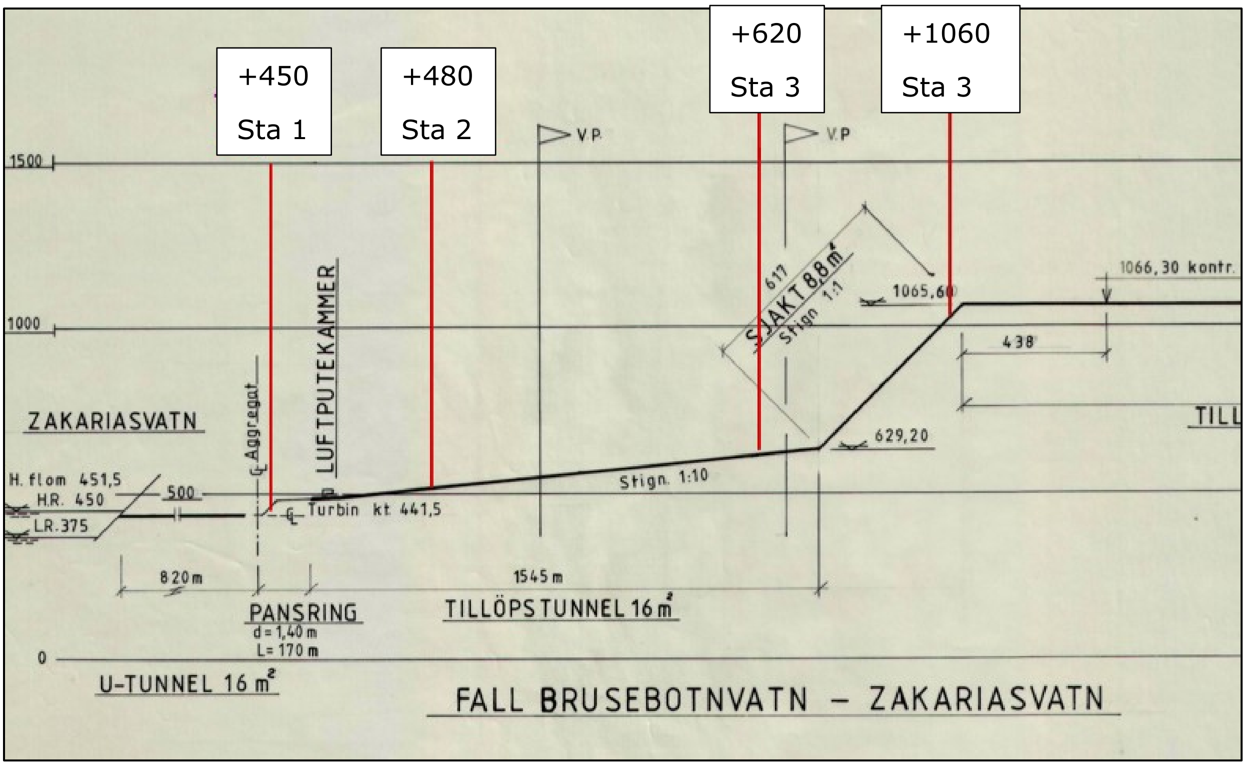


Figure 3.21. Tafjord 5 Proposed Measurement Station

3.2.1 Tafjord 5 Stage – Volume Curve

Similar to Hatlestad, to simplify the analysis of water loss, a Stage – Volume relationship curve was developed using the geometrical principle. To fully understand the Tafjord 5 system, Figure 3.22 shows the schematic flow of water in the Tafjord 5 hydropower plant. Water loss review in Tafjord 5 focuses more on the lower section of the waterway. The stage – volume curve only develops between turbine elevation (441.6 m) until below the Smette lake headrace and Brusebot lake headrace intersection (1065.6 m). Figure 3.23 shows the result of the calculation of stage (head) related to the accumulated volume, which is calculated in Table 3.8.

Initial Elevation	End Elevation	Head	Tunnel Diameter	Area	Slope	Angle	Volume	Accumulated Volume
masl	masl	m	m	m^2	m/m		m^3	m^3
441.5	629.2	188	16	201.14	0.10	5.71	17005.84	17005.84
629.2	1065.6	436	8.8	60.85	1.00	45.00	21403.77	38409.62

Table 3.8. Tafjord 5 Stage - Volume Calculations

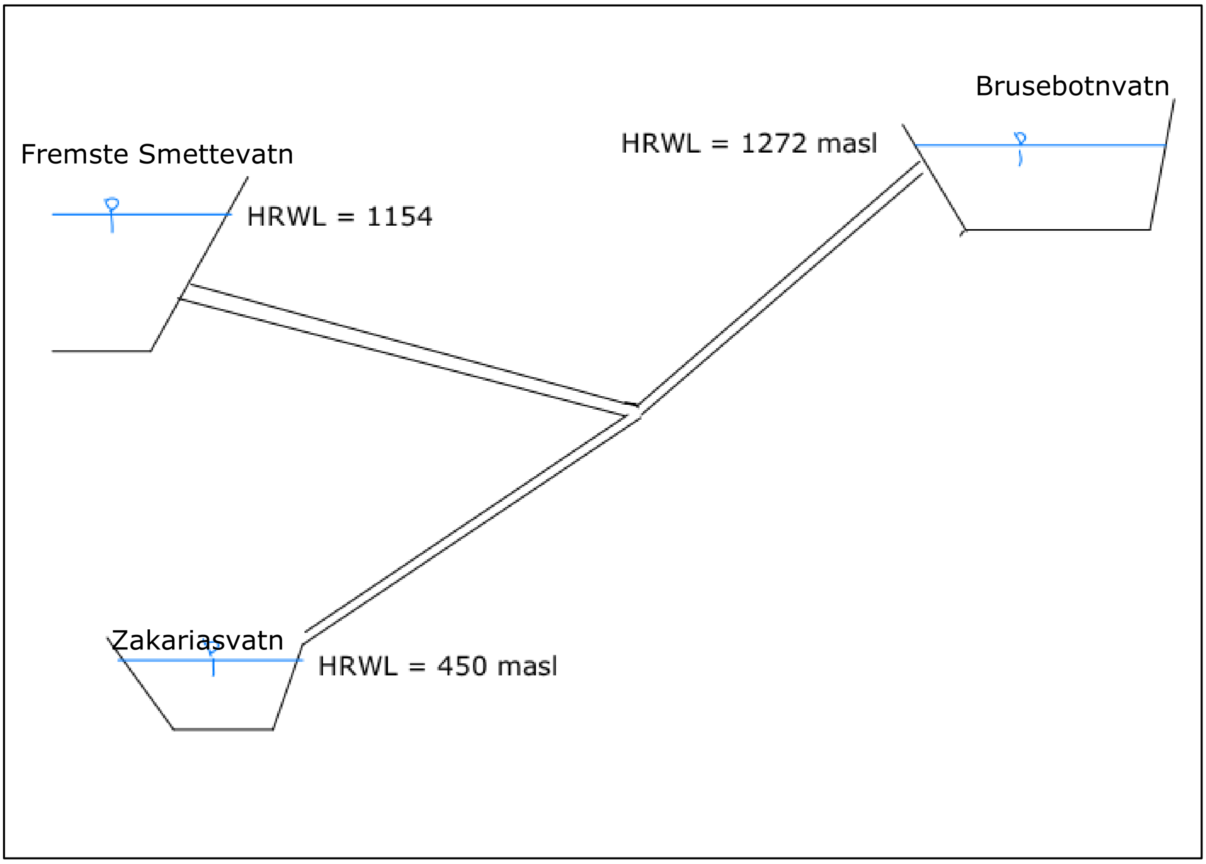


Figure 3.22. Tafjord 5 Water System Schematic Diagram

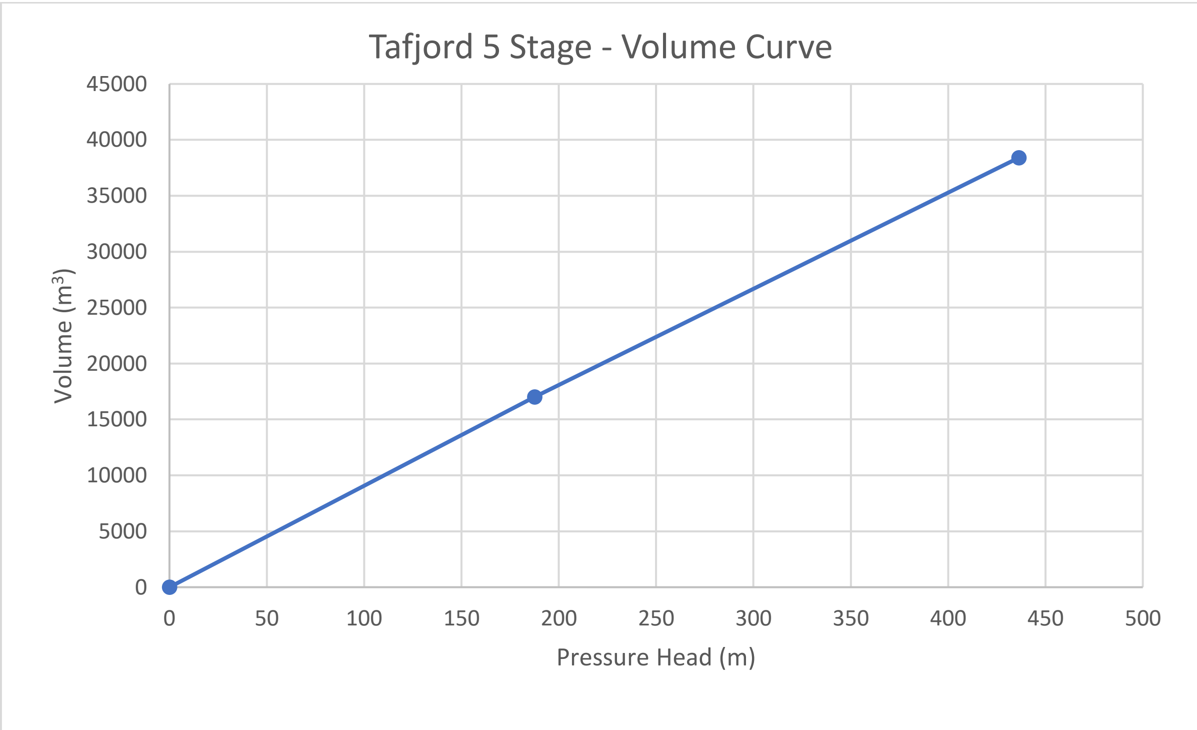


Figure 3.23. Tafjord 5 Stage - Volume Curve

3.3 Other Power Plants

In this study, there are 4 other power plants that are supposed to be included. However, similar to Tafjord 5, the data collecting process from Solhom HPP, Duge PSP, Saguling HPP, and Cirata HPP encounters obstacles. The main obstacle is the pressure measurement data relies on archives data, which makes it hard to retrieve. Moreover, due to time limitations, it is hard to conduct pressure measurement for such hydropower plants. This sub-chapter will discuss briefly the hydropower plant included within the study scope, but not thoroughly analyzed as the data is insufficient.

The data collecting process for Saguling and Cirata HPP is compounded with location problem and permit to use the data. Therefore, data from Saguling and Cirata is unable to be published in this study report. Saguling and Cirata HPP is located in West Java Province, Indonesia and form a cascade system in Citarum River. Saguling HPP has 4×175 MW Francis turbine with installed capacity 700 MW and was in commission since 1986. Cirata HPP is located the downstream side of Saguling HPP, has a capacity of 1008 MW with average annual production 1428 GWh. [30]

With energy equivalent of 0.504 kWh / m^3 and gross drop height 215 m from Homstøl lake until Kvifjorden lake, Solhom hydropower plant generates 695 GWh annually and has a maximum capacity of 200 MW [27]. To fully understand the waterway system in Solhom hydropower plant, an isometric drawing presented in Appendix E – Solhom Hydropower Plant Isometric Drawing and as an addition, Figure 3.24 below illustrates Solhom hydropower plant scheme according to NVE [27].

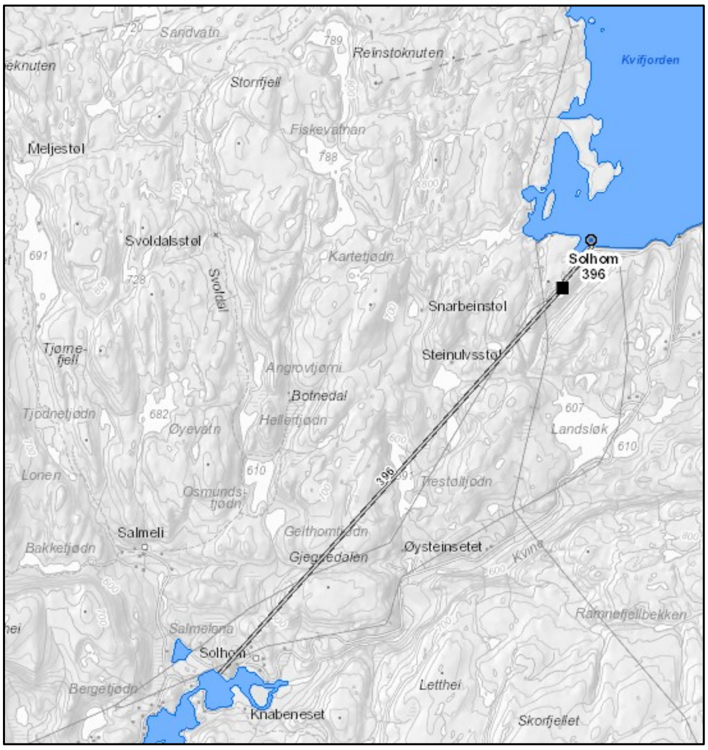


Figure 3.24. Solhom Hydropower Plant [27]

Duge PSP was commissioned in 1979, with a max capacity of 200 MW by taking advantage from the 220 m head. The energy equivalent of Duge PSP is 0.555 kWh/m^3, resulting in 248 GWh of yearly production [27]. Duge PSP intake is located in lake Svartevatn which is dammed with 130 m high riprap dam and is equipped with two reversible vertical Francis turbines; hence it has the capability to pump water to the reservoir from Lake Gravatn through 1200-meter-long tailrace tunnel. Figure 3.25 shows the scheme of Duge PSP according to NVE; the isometric drawing of Duge PSP can be seen in Appendix F.

Similar to Tafjord 5, the data collecting process from Solhom HPP and Duge PSP encounter obstacles. The main obstacle is due to the pressure measurement data relying on past archive which makes it hard to retrieve. Moreover, due to time limitations, it is hard to conduct pressure measurement for Solhom HPP and Duge PSP.

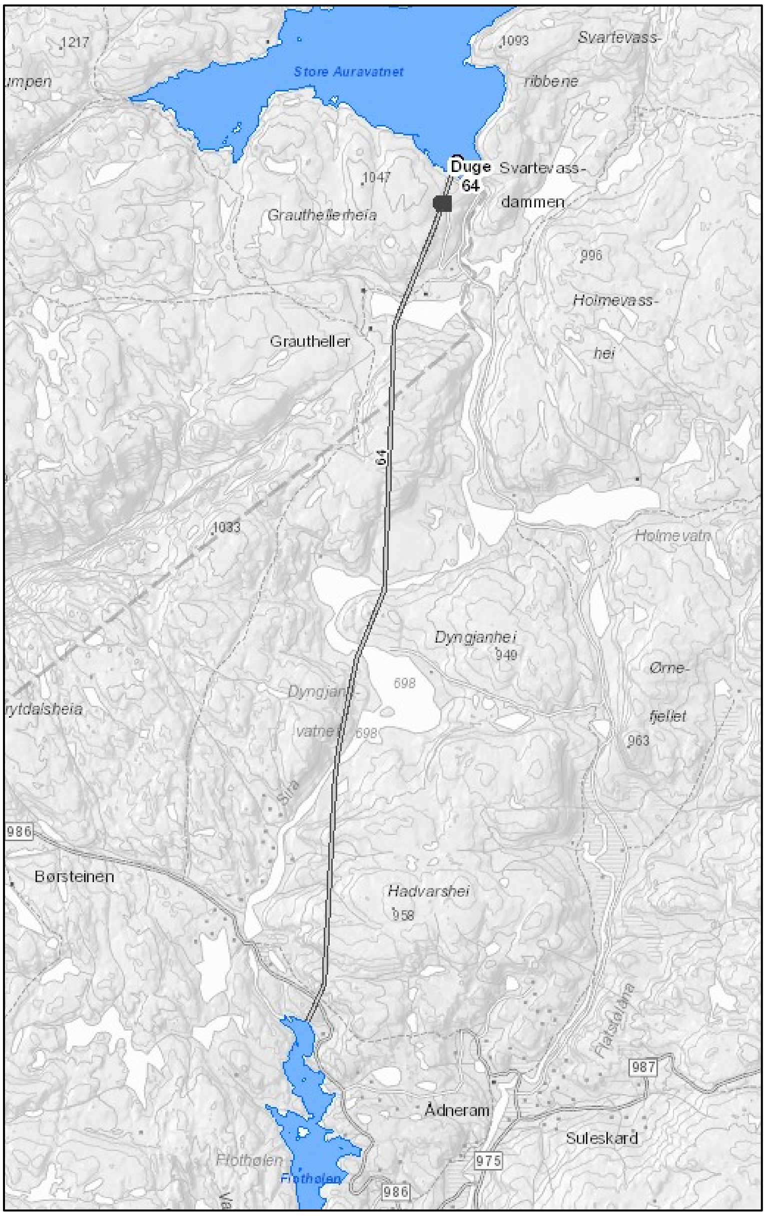


Figure 3.25. Duge Pump Storage Power Plant

Chapter 4 Consequences of Water Loss from Unlined Tunnel

"How often have I said to you that when you have eliminated the impossible, whatever remains, however improbable, must be the truth?"

-Sherlock Holmes-

The evidence from this study suggests that there is significant leakage that leads to water loss from Hatlestad Hydropower Plant. This chapter will discuss the implication of water loss from the unlined tunnel in a general overview. However, these findings are limited by data availability and therefore, it is possible another side effect of water loss occurs on another hydropower plants. The loss of revenue is in plain sight effect due to water loss from the unlined pressure tunnel. Followed by the reduction of a factor of safety on surrounding slopes and water consumption alteration. Specifically, in arctic countries, the ground freezing on surrounding terrain is also possible to generate problems. Using the data from Hatlestad hydropower plant as the main source of conclusion and supported by another power plant data from Norwegian experiences, each consequence of water loss from the unlined tunnel will be discussed in more detail in the next sub-chapter.

4.1 Revenue Evaluation

In principle of using hydropower as a source of energy, the existence of water loss means loss of energy production, which will lead to a loss of revenue. Table 4.1 shows the condition during the measurement of water loss from Hatlestad hydropower plant and Figure 4.1 illustrate the comparison between water loss before and after rehabilitation process in the Hatlestad drill hole during January – February 2019.

Dataset	Measurement Date	Condition
5	11/13-Dec-18	Before Rehabilitation
6	28-Mar-19	After Rehabilitation
7	29-Mar-19 (11.00 – 13.00)	After Rehabilitation
8	7-May-19	After Rehabilitation; Saturated Ground

Table 4.1. Hatlestad Water Loss Measurement Condition

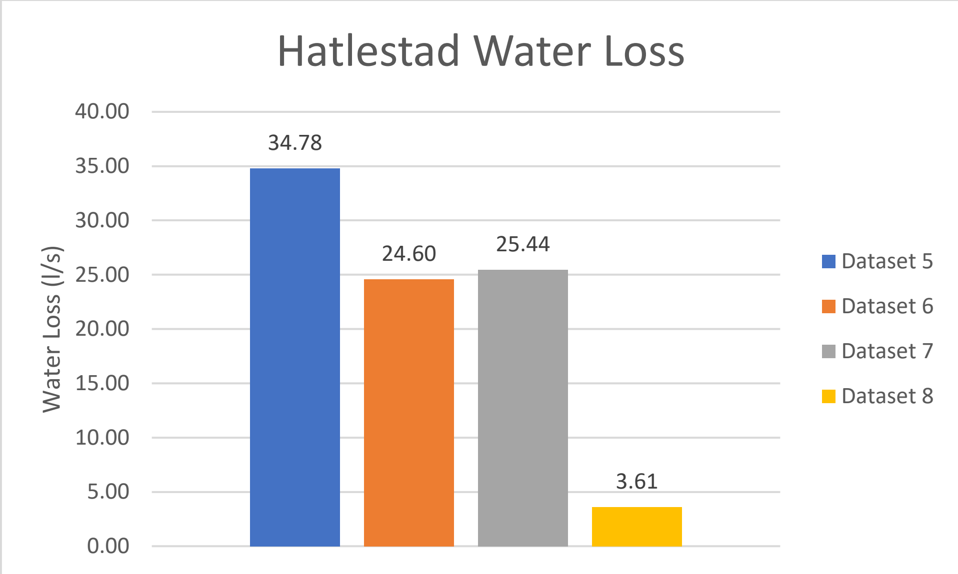


Figure 4.1. Water loss in Hatlestad

As mentioned in the literature review, hydropower energy equivalent (EEKV) is used to convert the volume of water loss in Hatlestad hydropower plant into energy loss. EEKV for Hatlestad is obtained through calculation shown in equation (16) and the annual volume loss from Hatlestad hydropower plant can be seen in Table 4.2 and Figure 4.2.

$$EEKV = \eta \times g \times \frac{H}{3600} = 0.9 \times 9.81 \times \frac{(628.25 - 4.65)}{3600} = 1{,}5312 \; {}^{kWh}/_{m^3} \qquad (16)$$

Dataset	Measurement Date	Water Loss on Maximum Static Head Condition (l/s)	Annual Volume Loss (m^3)
5	11/13-Dec-18	34.78	1,096,894
6	28-Mar-19	24.60	775,882
7	29-mar-19 (13.00)	25.44	802,419
8	7-May-19	3.61	113,872

Table 4.2. Hatlestad Annual Water Loss

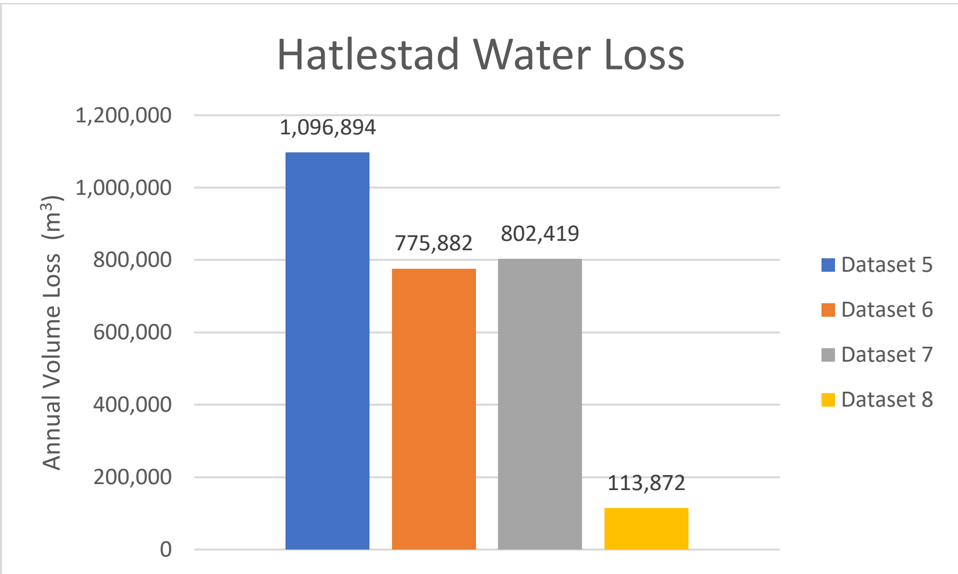


Figure 4.2. Hatlestad Annual Volume Loss

Energy production loss (kWh /year) from Hatlestad is obtained by multiplying the annual volume loss (m^3) with Hatlestad EEKV (kWh /m^3). Subsequently, energy production loss needs to be converted into revenue loss. To obtain the revenue loss, the energy prices used in this study refer to the historical electricity prices according to Nordpool presented in Table 4.3 [31].

The author decides to use three prices to assess the water loss in Hatlestad hydropower plant, which are the maximum price (414.86 NOK /MWh), the average price (287.16 NOK / MWh) and the minimum price (175.99 NOK /MWh). The objective of this price selection is to provide a range of losses value, which response to the unforeseen uncertainty. Due to the location, Bergen prices were selected to assess the revenue loss in Hatlestad hydropower plant. The results of the loss revenue calculation can be seen in Table 4.4. In addition, the comparison of revenue loss can be seen in Figure 4.3.

	Oslo	Kristiansand	Bergen	Molde	Trondheim	Tromsø
2018	419,34	415,47	413,55	423,44	423,44	419,80
2017	270,79	268,93	269,01	275,40	275,40	240,35
2016	242,68	233,24	230,94	266,01	266,01	232,59
2015	176,90	176,63	175,99	189,80	189,80	182,09
2014	228,86	228,03	227,26	263,57	263,57	262,77
2013	292,20	290,44	292,43	303,43	303,43	300,69
2012	221,35	218,32	216,76	235,66	235,66	233,32
2011	362,33	359,78	357,95	370,63	370,63	370,56
2010	434,75	407,14	414,86	465,46	465,46	459,78
2009	295,47	295,47	295,47	310,97	310,97	310,90
2008	324,48	324,48	324,48	421,26	421,26	410,17
2007	206,18	206,18	206,18	236,79	236,79	235,59
2006	396,56	396,56	396,56	394,64	394,64	394,67
2005	233,12	233,12	233,12	235,30	235,30	235,30
2004	246,06	246,06	246,06	243,87	243,75	243,75
2003	293,93	293,93	293,98	290,87	290,46	290,46

Table 4.3. Noordpool Electricity Prices Data [31]

Dataset	Annual Volume Loss (m^3)		Revenue Loss		
		Electricity Prices	414.86 NOK / MWh	175.99 NOK / MWh	287.16 NOK / MWh
5	1,096,894		NOK 697,147.99	NOK 295,740.91	NOK 482,559.81
6	775,882		NOK 493,123.89	NOK 209,190.75	NOK 341,336.09
7	802,419		NOK 509,990.08	NOK 216,345.64	NOK 353,010.71
8	113,872		NOK 72,373.41	NOK 30,701.92	NOK 50,096.25

Table 4.4. Hatlestad Loss of Revenue Calculation

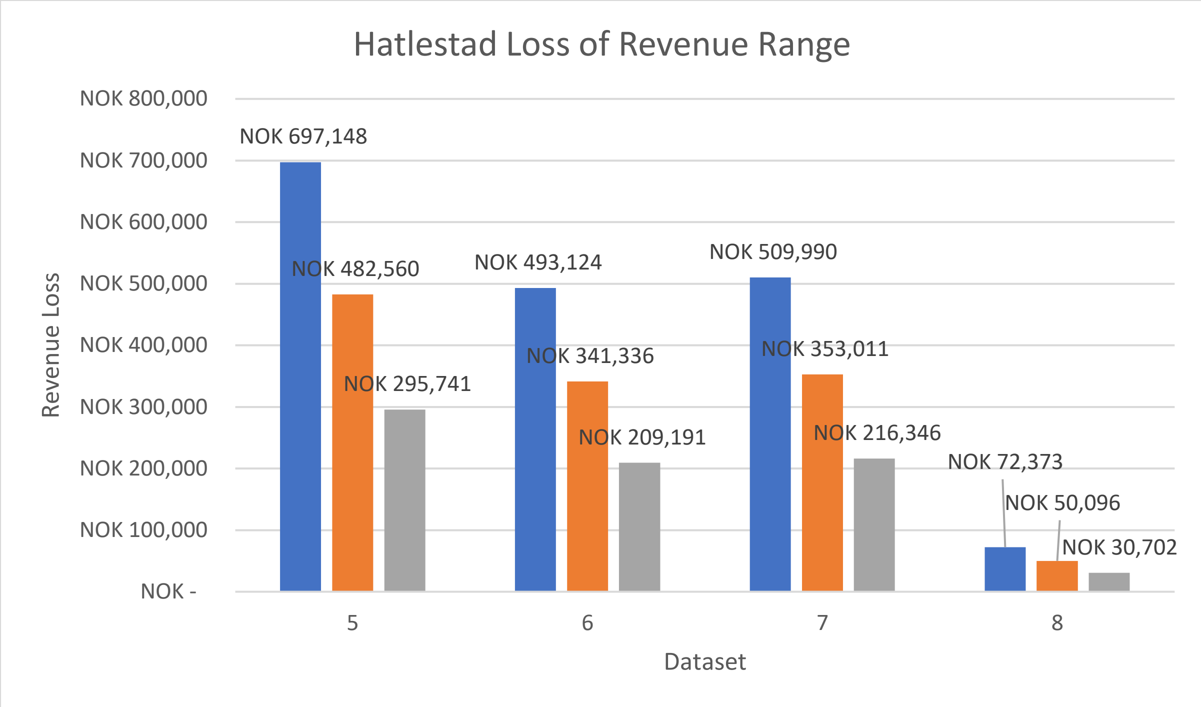


Figure 4.3. Hatlestad Revenue Loss

According to NVE, Hatlestad was estimated to have an annual production of 15.5 GWh, which is equivalent to 4,451,018 NOK/year of revenue (assuming the average electricity prices). Based on this, the ratio of revenue loss in Hatlestad HPP ranges from 1.12% - 10.84% annually. This evidence indicates water loss resulted in financial consequences.

4.2 Factor of Safety Reduction on Nearby Slopes

Besides the quantity of flow, a typical hydropower plant system will take advantage of the drop of the head. Due to this reason, most hydropower plant will be located on steep terrain. Using unlined pressure with high leakage will threaten the slope stability on nearby terrain. As mentioned in the analysis chapter, there are possibilities that the water loss from the unlined tunnel will increase the water table and make the ground fully saturated. Water is not always involved directly in the mass movement of landslides. However, it does play an important role, beside the forces from gravity, earth materials, and slope geometry. The saturated ground will reduce the angle of repose[3] of the materials as the water gets between the grains and eliminates frictional contact between grains. This situation will lead to a reduction in the factor of safety values on the nearby slopes

The effect of groundwater level on slope stability has been discussed a lot in the past. As an example, research conducted by Rahardjo (2010) concludes that the decrease in the factor of safety due to rain actually happens in two case studies in Singapore (Figure 4.4) [32]. A similar study conducted by Choi (2013) to illustrate the severity of landslides due to rainfall and high concentration of developments on hilly terrain in Hong Kong and discusses the prevention and mitigation works [33].

[3] Angle of repose is the steepest angle at which a pile of unconsolidated grains remains stable, it is controlled by the frictional contact between the grains. [41]

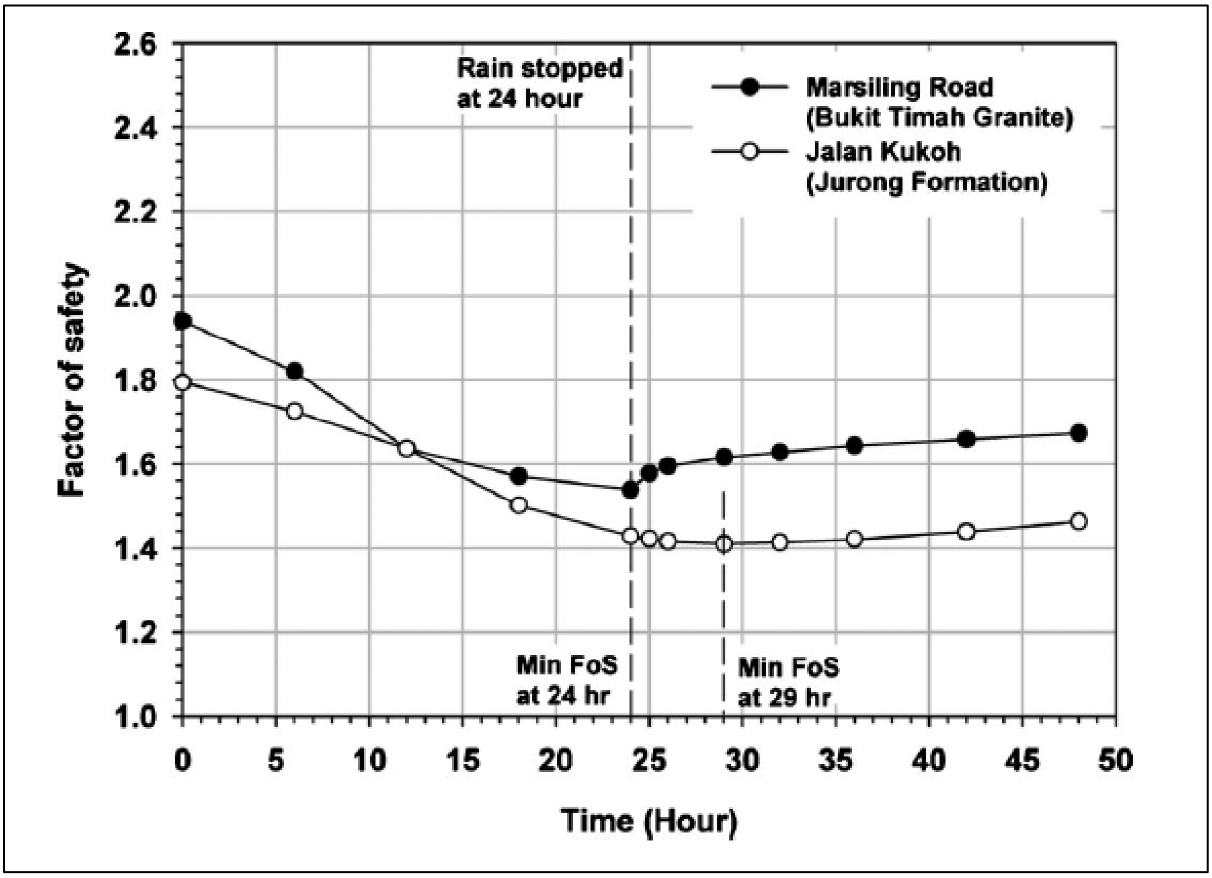


Figure 4.4. Factor of Safety Variations on Slope Due to Rainfall [32]

Figure 4.5 shows the forces that work on a typical landslide failure surface approached using limit equilibrium model[4] [34]. Factor of safety for this typical slope can be obtained using equation (17)

$$F = \frac{cA + (W(\cos\psi_p - \alpha\sin\psi_p) - U + T\cos\theta)\tan\phi}{W(\sin\psi_p + \alpha\cos\psi_p) - T\sin\theta} \tag{17}$$

Where:

$$A = \frac{H}{\sin\psi_p} \tag{18}$$

$$W = \frac{\Upsilon_r H^2}{2}(\cot\psi_p - \cot\psi_f) \tag{19}$$

$$U = \frac{\gamma_w H_w^2}{4\sin\psi_p} \tag{20}$$

U represent the uplift force due to water pressure on failure surface and it can be seen from the equation, the higher the water table will generate higher uplift force and reduce the factor of safety of the slope due to the reduction in resistance force.

[4] Limit equilibrium model has traditionally been used to obtain approximate solutions for the stability problems in soil mechanics. [40]

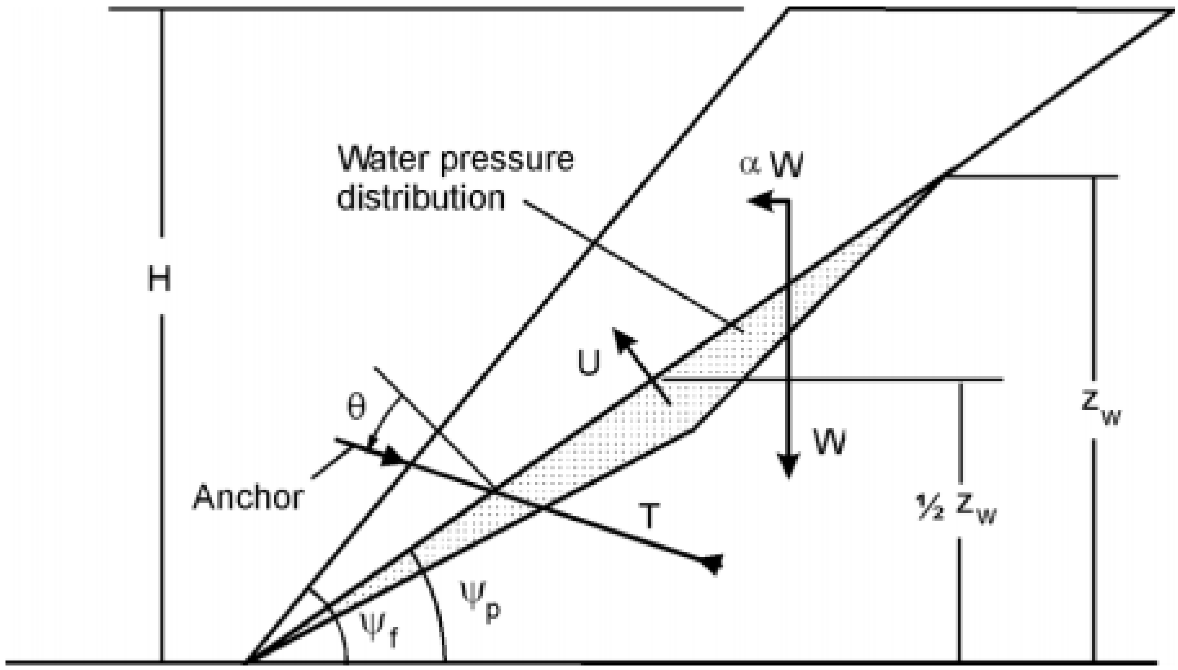


Figure 4.5. Groundwater Pressure Effect on Slope Stability [34]

Using the limit equilibrium method, the change FoS due to ground saturation, slope angle and slope height is illustrated in Figure 4.6. The value of the input variable can be seen in Table 4.5. In this study, the data to assess slope stability is unavailable. However, it is noted that there are several potentials of sliding planes from the dataset. The factor of safety reduction due to the increase of water pressure analysis was conducted using approximation value with simplified geometry of slope similar to Figure 4.5. The data input presented in Table 4.5 and this dataset did not represent any case that had been used in this study.

ψ_f	30°; 45°; 60°
ψ_p	ψ_f – 10°
γ	0.027 MN/m^3
γ_w	0.01 MN/m^3
c	0.6
α	0
ϕ	60°

Table 4.5. Input for Assessing Slope Stability

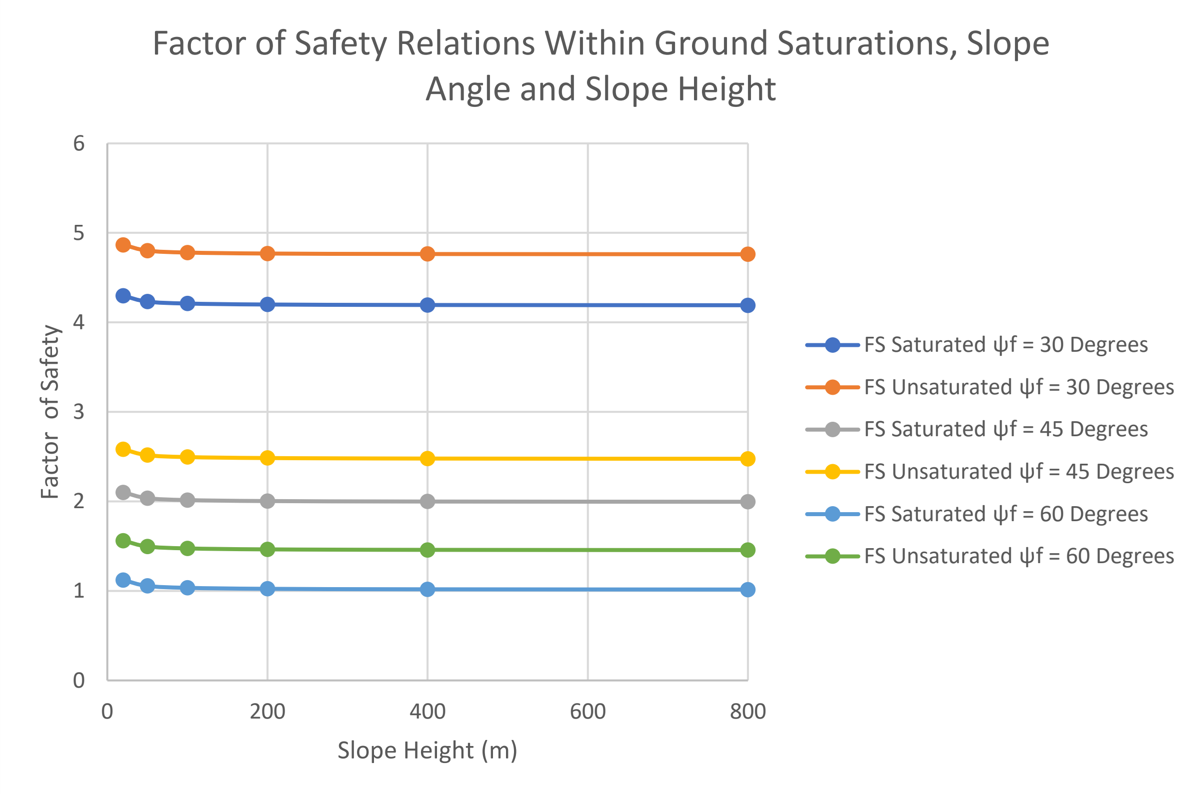


Figure 4.6. Ground Saturations, Slope Angle and Slope Height Affects on Slope FoS

In percentage, the reduction of FoS in the slope presents in Table 4.6 and the trend of FoS reduction can be seen in Figure 4.7. The reduction of FoS due to water pressure will be significant in slope with a steep angle. It should be noted that the analysis uses a simplified version of groundwater pressure and according to Nilsen (2017), using this type of distributions of water pressure only represents the worst case scenario such as long heavy rainfall conditions [35].

Slope Height (m)	FS Reduction (%)		
	$\psi_f = 30^o$	$\psi_f = 45^o$	$\psi_f = 60^o$
20	11.70	18.61	28.21
50	11.86	19.09	29.43
100	11.92	19.25	29.86
200	11.94	19.33	30.08
400	11.96	19.38	30.20
800	11.96	19.40	30.25

Table 4.6. FoS Reduction in %

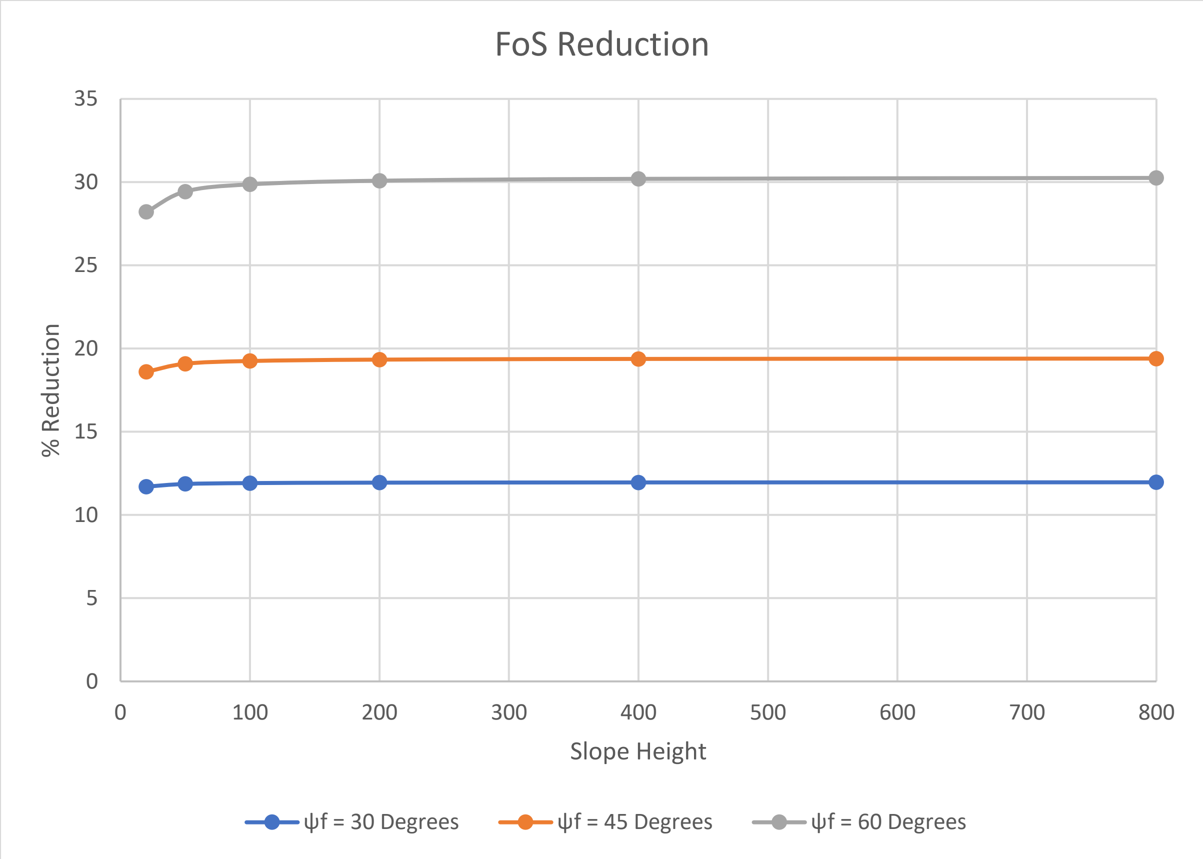


Figure 4.7. Percentage of FoS Reduction

It needs to be considered that the behavior of groundwater pressure towards the potential sliding plane due to leakage from the unlined tunnel is still unknown. Using simplified distribution may lead to higher groundwater pressure in comparison to the assumption proposed by Nilsen (2017) [35]. On the other hand, it is known that leakage from the unlined tunnel will make the ground around the unlined tunnel in saturated condition and this situation may lead to the worst-case scenario of groundwater pressure.

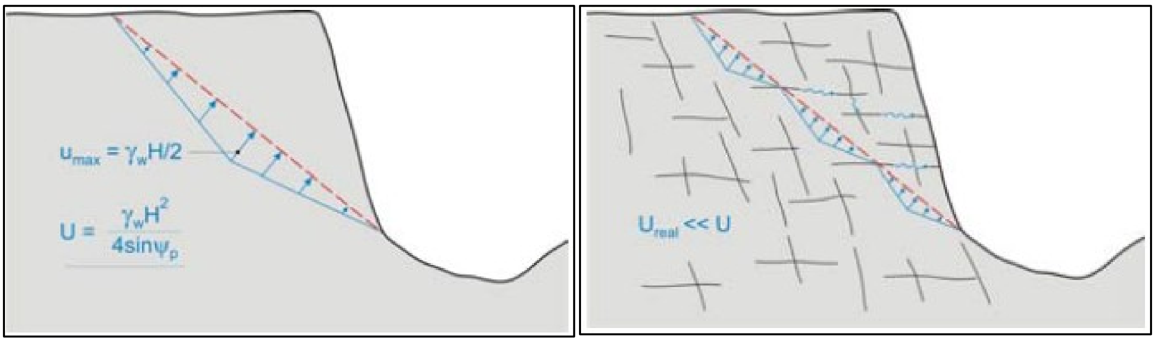


Figure 4.8. Groundwater Pressure Distribution; Simplified Assumption (Left) and More Realistic Assumption (Right) [35]

4.3 Water Consumption Alteration

There are several definitions to define water consumption. According to USGS, water consumption is part of water withdrawn from its source that not return back to the watershed [36]. This definition is also supported by Olsson (2015) by arguing that water is consumed when the control over the water is lost [37]. Using these definitions, water loss from unlined tunnel can be classified as a water consumption due to the production of energy from a hydropower plant.

According to Bakken (2017), there is no agreement on how to calculate the "true" water consumption [9]. For the simplicity, water consumption analysis in this study focuses on the water consumption increase due to water loss from the unlined tunnel, especially in Hatlestad HPP case. The main source of water consumption in typical hydropower plant is the evaporation level from the reservoir [9]. It is well known that for a run-off river hydropower plant without regulations ability, water consumption is admitted as zero. Conversely, using an unlined tunnel as a waterway for run-off river hydropower plant system will threaten the status of zero water consumption, which has pinned to run-off the river hydropower plant.

As discussed in Sub-Chapter 3.1.4, Hatlestad hydropower plant sustains water loss from the waterway ranging from 3.61 l/s until 34.78 l/s. By converting the water loss into annual volume loss using equation (21), water consumption can be calculated by dividing the annual volume loss with annual production, as shown in equation (22). In this study, the annual production for Hatlestad Hydropower plant uses the value according to the report from reference [28] as equal to 15,500 MWh / Year.

$$Annual\ Volume\ Loss = Qloss\ \times 60\ \times 60\ \times 24 \times 365 \tag{21}$$

$$Water\ Consumption = \frac{Annual\ Volume\ Loss}{Annual\ Production} \tag{22}$$

Dataset	Water loss	Volume Loss	Annual Production	Water Consumption	Circumstances
	l/s	m^3/Year	MWh/ Year	(m^3/MWh)	
5	34.78	1096822.08	15500	70.76	Pre-Rehabilitation
6	24.6	775785.6	15500	50.05	Post- Rehabilitation & Unsaturated Ground
7	25.44	802275.84	15500	51.76	Post- Rehabilitation & Unsaturated Ground
8	3.61	113844.96	15500	7.34	Post-Rehabilitation & Saturated Ground

Table 4.7. Hatlestad Water Consumption

Water consumption from Hatlestad HPP ranges from 7.34 m^3/MWh until 70.76 m^3/MWh. Bakken (2017) indicates that gross water consumption rates for hydropower production are in the range of 5.4 - 234 m^3/MWh [9]. Therefore, water consumption in Hatlestad HPP still categorizes as normal water consumption.

Chapter 5 Conclusion

We are in the endgame now.

- Dr. Stephen Strange -

Due to limited data, drawing a general conclusion on water loss on the unlined tunnel from this study will be problematical to be accepted. Collecting historical data of water loss is hard to be done because conducting pressure measurements to measure leakage from an unlined tunnel is uncommon. Furthermore, in a country with low water scarcity threat such as Norway, water loss from the unlined tunnel is often neglected, unless the water loss is threatening the safety and gave significant financial consequences.

Assessing tunnel water loss based on Hatlestad hydropower plant case is in line with arguments from Benson (1989). Although the unlined section of a waterway has already fulfilled the Norwegian criterion, leakage still occurs either through a permeable zone (joint 6) or due to the annulus in transition zone unable to withstand the water pressure. Another important finding is the behavior of leakage – head relationship that shows similarities with Quiñones-Rozo (2005) interpretation of Lugeon value. Based on this evidence, Hatlestad hydropower plants allegedly experience dilation on its permeable zone due to water pressure.

A rehabilitation process was conducted at Hatlestad hydropower plant, and after the waterway rehabilitation process, Hatlestad waterway system still sustains leakage approximately 25 l/s. This value is reduced from approximately 35 l/s leakage before rehabilitation. The leakage value drops significantly into 3.61 l/s in measurement 8, which is conducted during normal operation time. The explanation of this finding is the measurement in the midst of operation time will give saturated ground condition in the area of the waterway system. This circumstance will increase the water table in the surrounding environment and as a consequence, the leakage value will be lower because of lower ground infiltration rate.

Financial consequence due to water loss is unavoidable and can range from 1.12% - 10.84%. Using average electricity prices 287.16 NOK/MWh, financial consequence due to water loss in Hatlestad HPP case approximately NOK 482,560 per year. After the waterway rehabilitation process, this number reduced to approximately NOK 341,336 – NOK 353,011 per year. Financial loss due to water loss in saturated ground condition gives much lower value, approximately NOK 50,096 per year. In this study, it is hard to select a value that describes the most accurate conditions. As already known, hydropower plant has capacity factor approximately 45%, this means hydropower plant will not run in full capacity in one year and have a different situation from time to time, compared to the measurement that had been conducted in measuring water loss. Finding an accurate value will be needed further study with extensive data.

Financial consequences due to water loss also need to consider the cost of improving nearby terrain. Using the limit equilibrium method, FoS on the nearby slope will reduce from 10% - 30% depends on slope abruptness, slope height, and water pressure assumption. Maintain factor of safety at an acceptable level that had been reduced because of water loss from a tunnel system means additional cost which must be incurred by Hydropower plant owner.

Other than that, water loss from the tunnel system also alters the water consumption from a hydropower plant. In Hatlestad case, water consumption due to water loss range from 7.34 m^3/MWh until 70.76 m^3/MWh. Hatlestad hydropower plant is a run-off the river system hydropower plant, which until this time is well known as a power plant with zero water consumption because it does not use a reservoir. Since Hatlestad is a Norwegian power plant, water consumption is not a big issue in Norway and can be neglected since it still categorizes as low water consumption in a country with low water scarcity threat. However, this evidence may lead to water conflict in a country with high water scarcity threat.

Chapter 6 Recommendation

I have fought the good fight, I have finished the race, I have kept the faith.

- 2 Timothy 4:7 -

Using unlined tunnel is a promising method to reduce the capital cost of constructing a hydropower plant and conclusion from this study identify several issues regarding water loss from an unlined tunnel, such as threat the financial viability from a hydropower plant. This chapter will discuss the recommendation that can be done in the future of pressure tunnel design. In addition, based on evidence from this study, several limitations need to be examined further to increase the understanding of water loss from unlined tunnel topics.

Uncertainties in several factors lead to difficulties in predicting the exact value of the implication of water loss. Water loss assessment from this study is unable to give the exact losses and can only provide the range of value. Using a probabilistic approach will be beneficial, as the probabilistic approach will enable variation and uncertainty to be quantified. This study identifies several factors that contain uncertainty, including electricity prices fluctuation, degree of saturation from nearby ground and behavior of water pressure affecting slope stability. The sensitivity towards the cost of water loss has not been identified in this study. Identifying sensitivity will be beneficial to understand the characteristics of water loss from the unlined tunnel and in addressing the problem.

In conducting pressure measurement, there is no standard regarding the time needed to conduct measurement. Measurement time is an important part of this measurement to avoid the bias of measurement. Therefore, it is recommended to study the time factor to propose a good basis of standard in conducting pressure measurement. Other than that, assessing water loss using static pressure head is adequate to identify the tunnel permeable zone. However, it will not give the true value of water loss when the hydropower plant is in operation. To obtain a better understanding of the behavior of water when exiting pressure tunnel through a permeable zone will need a further study, in particular by taking into account the degree saturation on the nearby ground. Through this complexity, the assessment of water loss will need detail in hydraulic level to obtain the detail of fluid flow.

6.1 Lining Optimization Using Pressure Test

Determining the transition zone between unlined and lined section from a pressure tunnel is a design process that is overwhelmed by uncertainty. Estimation procedure has been widely studied in previous studies. However, sometimes it fails to satisfy or to achieve the acceptable level of leakages. The results of this study indicate the needs of a better way to predict the needs of linings in an unlined tunnel. Using a pressure test to identify leakage zone in this study is proven to be used as a solution to tackle uncertainties in determining the transition zone.

The idea to conduct a pressure test is a well-known test in petroleum field with a purpose to identify leakage and well integrity by observing the pressure fluctuations. Using the same principle, this study proposes the use of a pressure test to identify the leakage zone from a tunnel. This finding is also in line with the research conducted by Ødegaard (2017) that recommends the use of this test to give confidence related to rock stress condition [38]. The suggestion of pressure measurement in the tunnel done by treating the whole tunnel similar with a drill whole when conducting a Lugeon test. The interval of pressure measurement can be adjusted by using an inflatable packer. Therefore, the water tunnel can be sealed with an interval and the leakage rate from a certain zone can be identified. Figure 6.1 up to Figure 6.3 present the proposed schematization on pressure measurement. The cost-effectiveness, practicality, and time efficiency from each method need to be studied further to determine the suitability from each method.

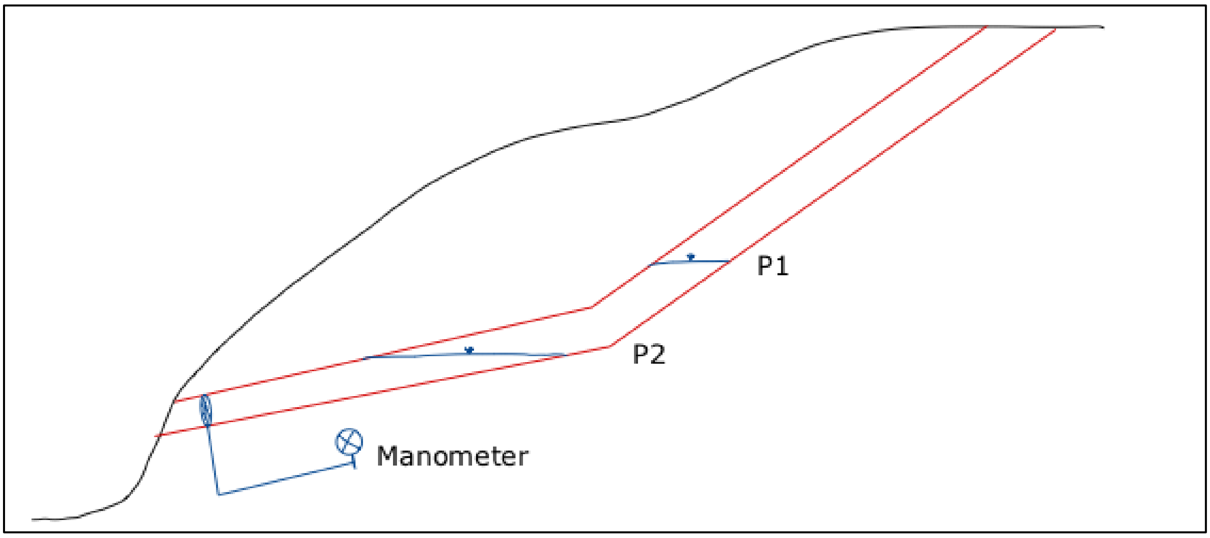


Figure 6.1. Pressure Measurement Without Packer

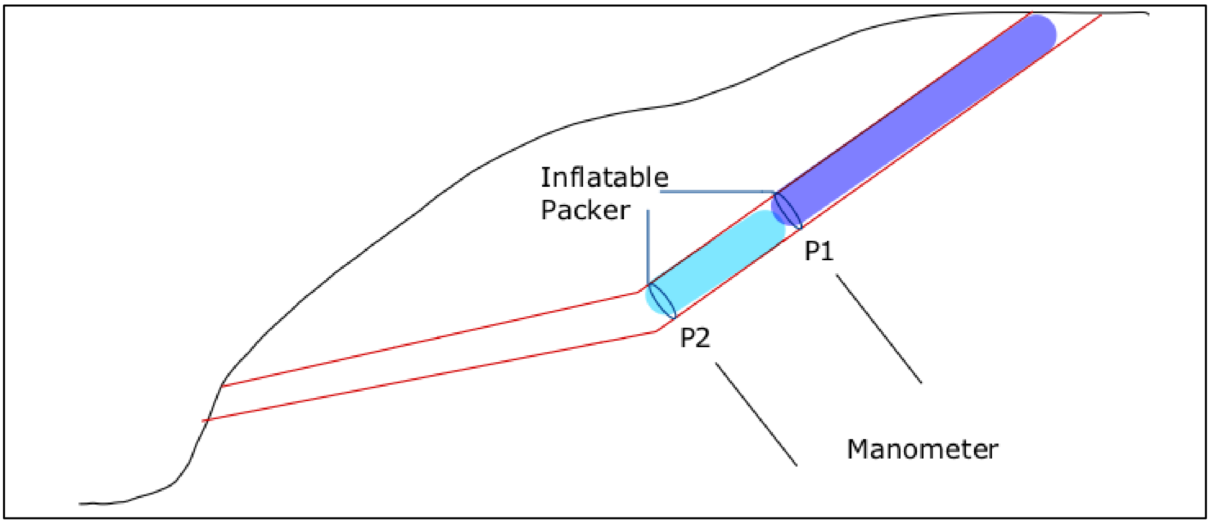


Figure 6.2. Pressure Measurement Single Packer

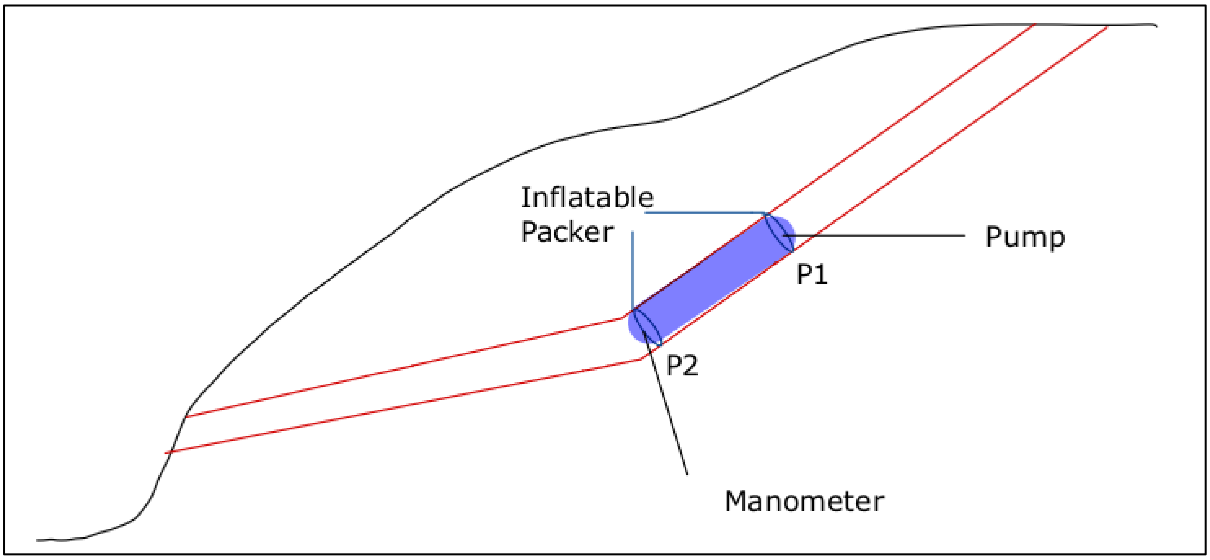


Figure 6.3. Pressure Measurement Using Double Packers

After finding the relations between head/elevation and leakage, tunnel linings can be optimized by using marginal analysis as presented in Figure 6.4. Increasing tunnel lining will reduce the loss of revenue due to water loss from the hydropower system but at the same time will increase the lining cost itself. The optimum tunnel lining length will be the minimum of total cost from tunnel lining installations and cost of water loss. The tunnel lining optimization also may take into account the foreseen cost such as slope mitigation cost.

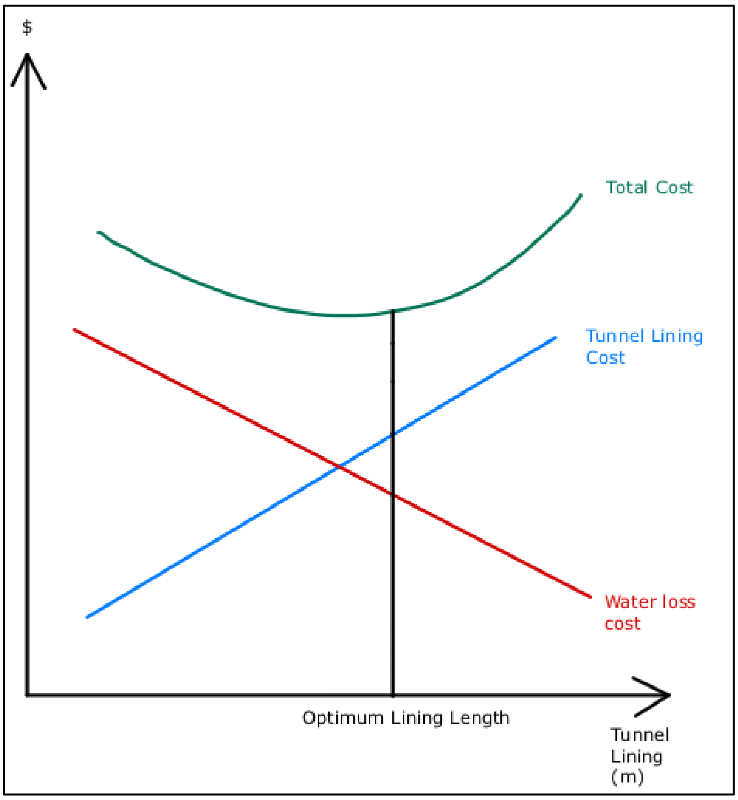


Figure 6.4. Tunnel Lining Optimization

6.2 Taking Advantage of Water Loss

In most circumstances, water loss from pressure tunnel will return to the originating stream. For some cases, the leakage in the terrain is detectable and can be controlled. If the condition allows, the water loss can be used as a negotiation tool in adjusting environmental flow release. Therefore, hydropower operator has more flow to produce energy. This idea can be further developed and argued in its use in the future.

There are several important issues that need to be solved in the use of this idea. Safety on the nearby slope is the most important factor before allowing the leakage from the tunnel system. Moreover, reduction on environmental flow will increase the drying area of some river section before the water from leakage rejoins in the river. In a river with trapezium cross-section shape, reduction on environmental flow will be devastating due to the large increase of dry area. This will be the opposite if the river cross section has a square shape because the drying area will not be as significant as a river with trapezium cross-section. The idea needs to be considered carefully before the implementation in the future, in particular, concern about environmental degradation due to water loss.

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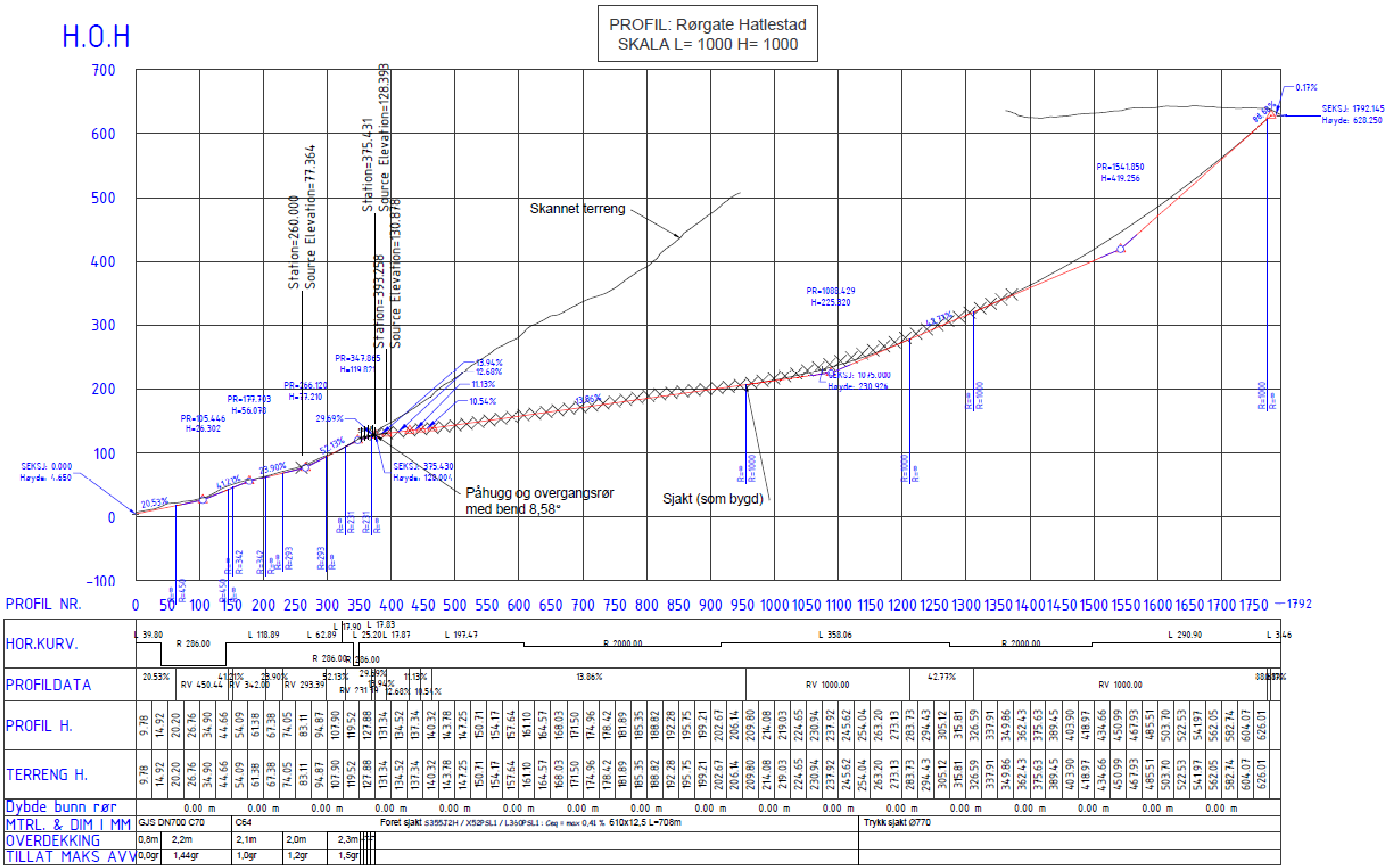
Appendix A – Hatlestad Hydropower Plant Scheme [9]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500
400
300
200
100
0
-100
Station=260.000
Source Elevation=77.364
Station=375.431
Source Elevation=128.393
Station=393.258
Source Elevation=130.878
Skannet terreng
Påhugg og overgangsrør
med bend 8,58°
Sjakt (som bygd)
SEKSJ: 0.000
Høyde: 4.650
SEKSJ: 375.430
Høyde: 128.004
SEKSJ: 1075.000
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SEKSJ: 1792.145
Høyde: 628.250
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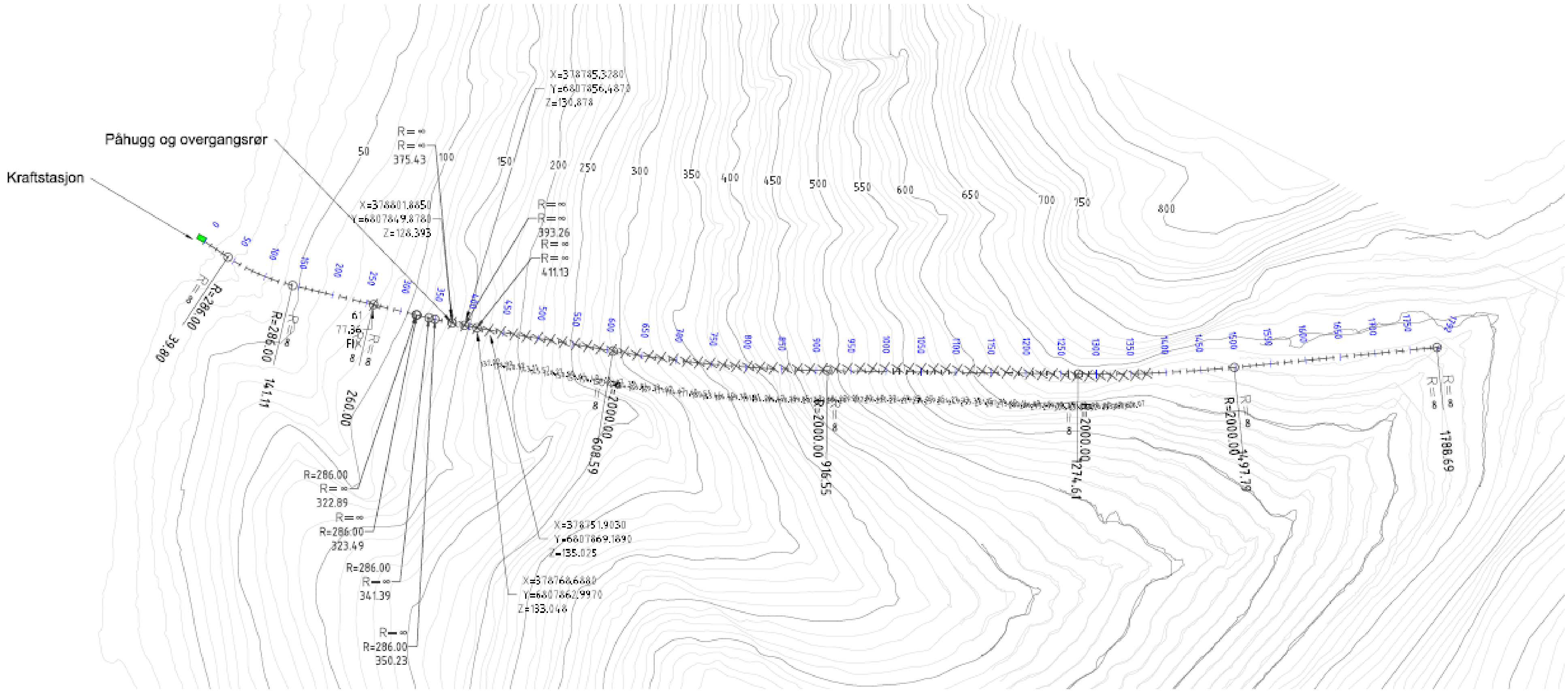
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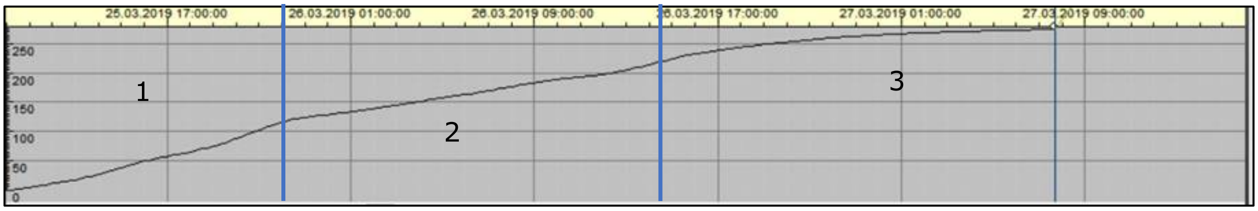
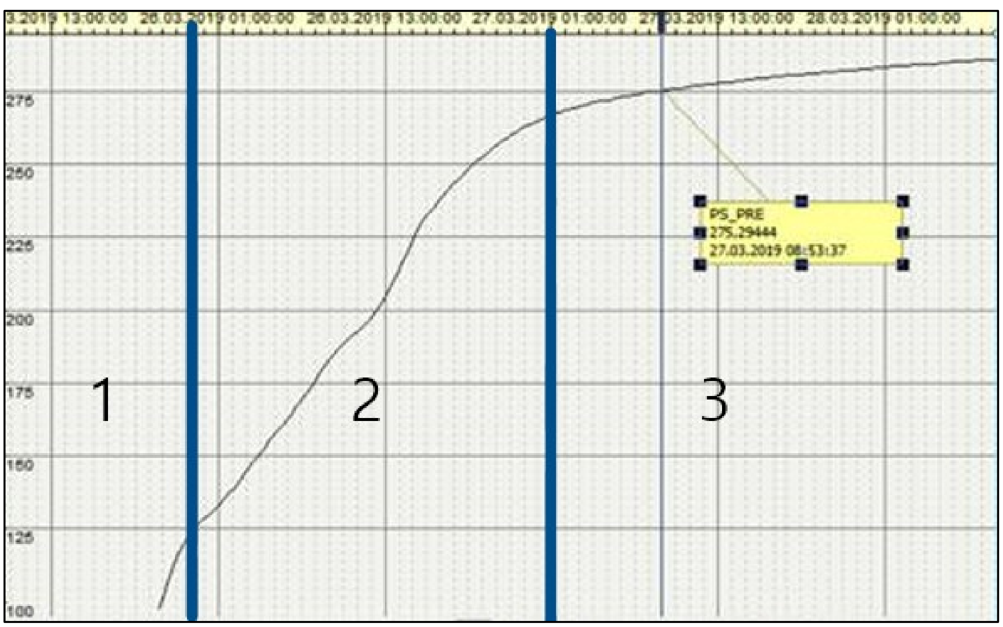
Appendix B – Hatlestad Water Ingress

Dataset 1 –25/27 March 2019



Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	0	120	0.00	145.15	10:00	22:00	145.15	12.00	720	
2	120	225	145.15	455.03	22:00	0:00	309.88	19.00	1140	
3	225	228	455.03	463.88	15:00	8:00	8.85	15.00	900	

Dataset 2 - 26/27 March 2019

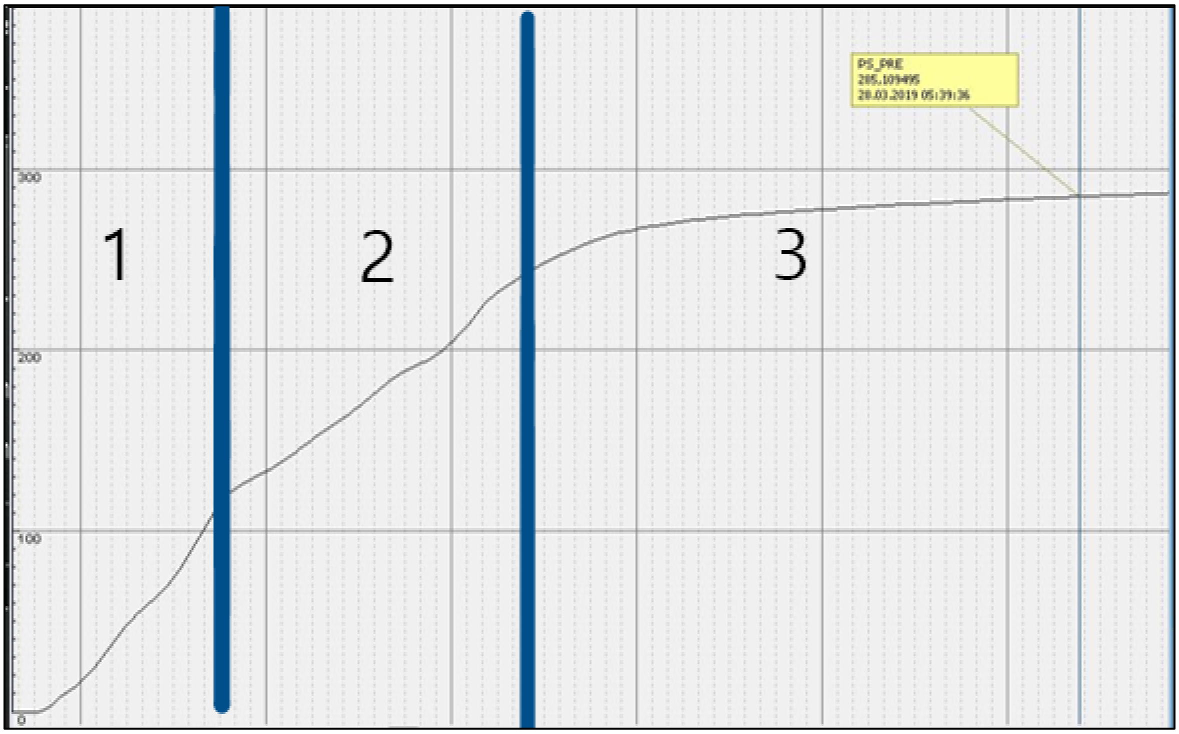


Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	100	125	129.945	159.907	20:30	23:00	29.962	2:30	3.329	
2	125	263.75	159.907	520.224	23:00	1:00	360.317	52:00	1.925	
3	263.75	285	520.224	545.195	1:00	8:00	24.971	70:00	0.099	

Dataset 3 - 26/28 March 2019 (Controlled Filling)

Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	230	267	524.04	469.78	15:30	0:20	54.25	8:50	1.706	
2	267	286	546.36	524.04	0:20	8:00	22.32	7:40	0.809	

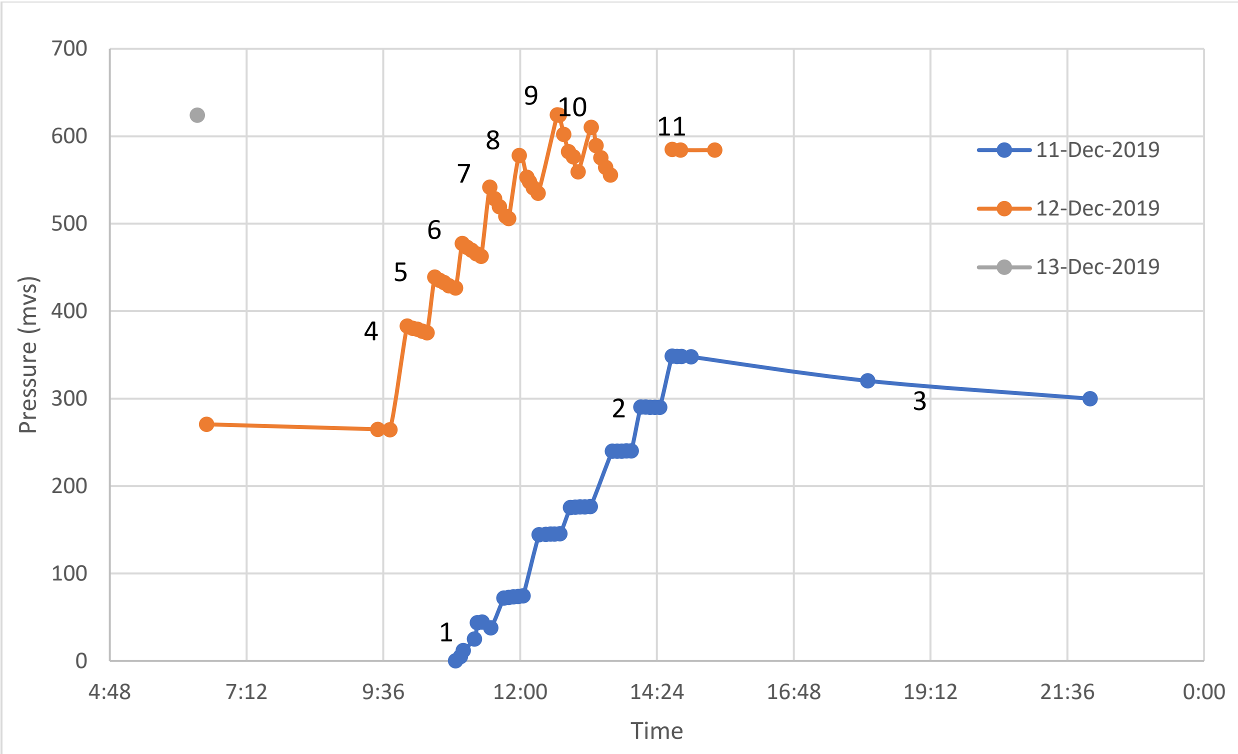
Dataset 4 - 25/28 March 2019



Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	0	121.3	0	148.98	10:00	22:00	148.98	12	720	
2	121.3	262.6	148.98	518.87	22:00	1:00	369.88	27	1620	
3	262.6	286.6	517.12	547.07	1:00	1:00	29.94	36.5	2190	

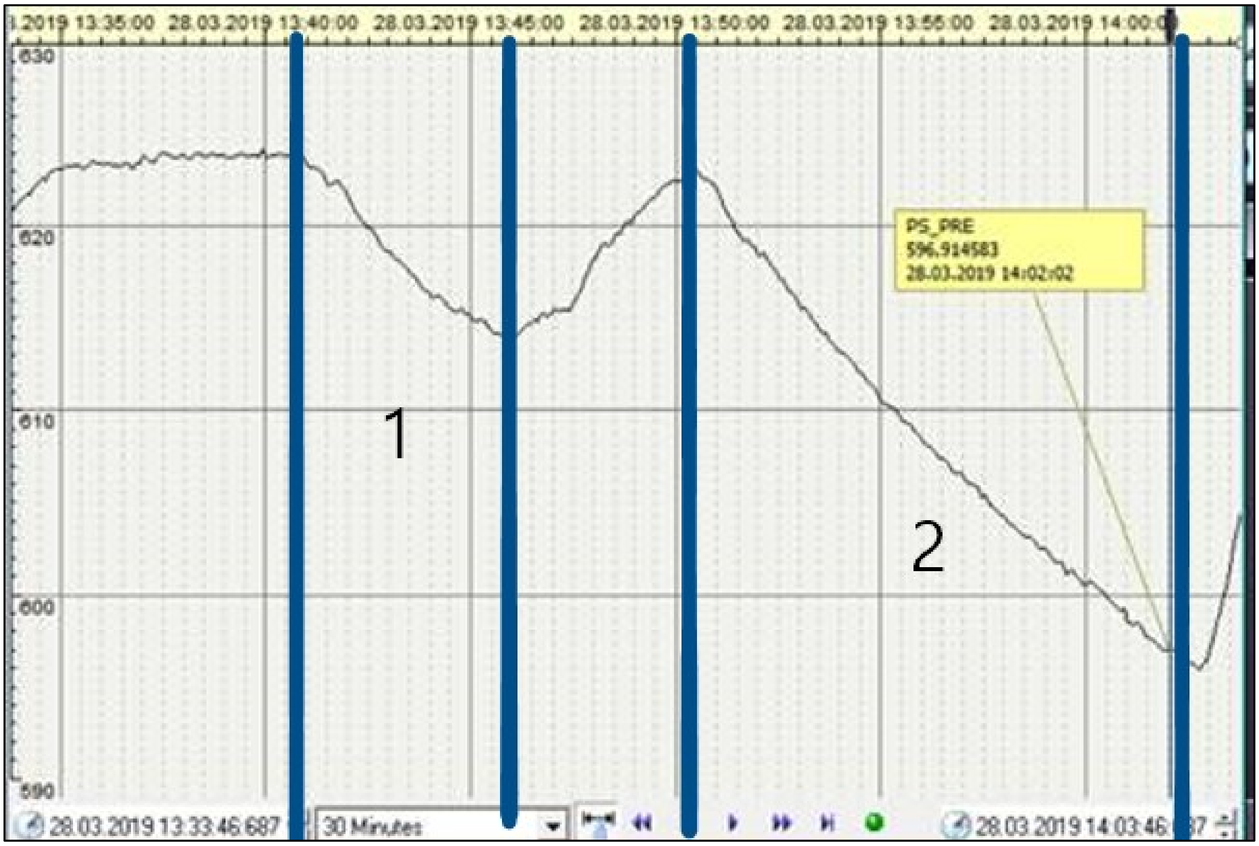
Appendix C – Hatlestad Water Loss

Dataset 5 - 25/27 March 2019



Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	44.6	38	65.226	58.046	11:20	11:29	7.180	0:09	9	Data Inconsistency
2	290.4	289.8	551.540	550.835	14:07	14:27	0.705	0:20	20	
3	348.6	264.5	619.931	521.105	14:40	9:43	98.825	19:03	1143	Overnight Measurement
4	383.1	375.3	660.472	651.306	10:01	10:22	9.166	0:21	21	
5	438.9	426.3	726.042	711.236	10:30	10:52	14.806	0:22	22	
6	477.3	462.8	758.862	748.604	10:59	11:19	10.258	0:20	20	
7	541.7	505.9	804.420	779.094	11:28	11:48	25.326	0:20	20	
8	578.1	534.7	830.171	799.468	11:59	12:19	30.703	0:20	20	
9	624.3	559.4	862.855	816.942	12:39	13:01	45.913	0:22	22	
10	610.3	555.8	852.951	814.395	13:15	13:35	38.555	0:20	20	
11	585.1	584.1	835.123	834.416	14:40	14:49	0.707	0:09	9	incomplete Measurement

Dataset 6 - 28 March 2019

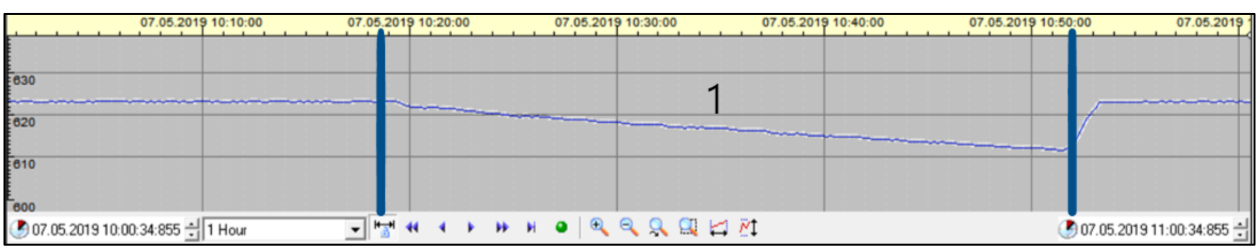


Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	623.85	614.59	862.54	855.99	13:39	13:46	6.55	0:07	15.597	
2	622.19	597.15	861.36	843.65	13:50	14:02	17.71	0:12	24.603	

Dataset 7 - 29 March 2019

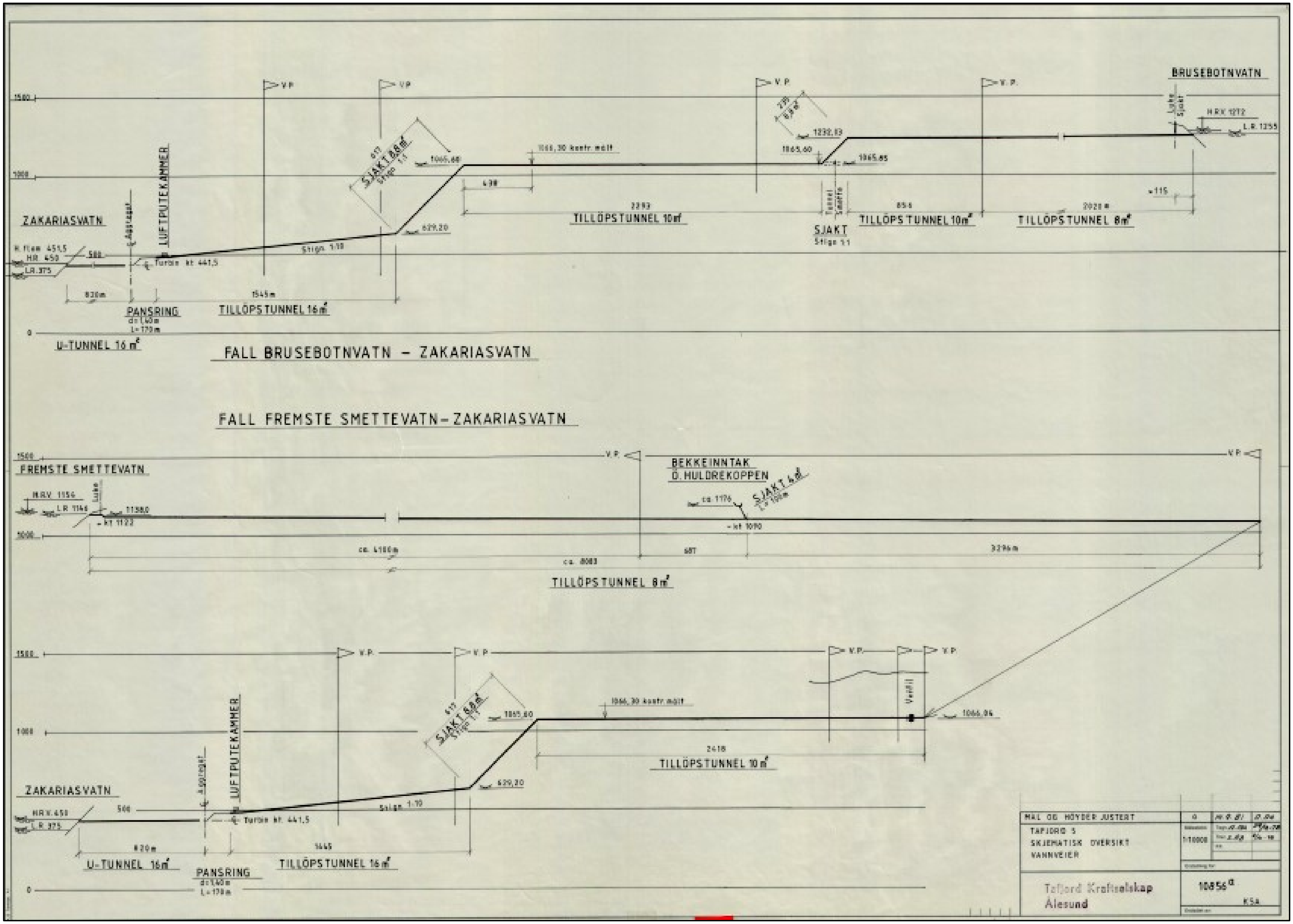
Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	624	614	862.64	855.57	13.40	13.46	7.07	0:06	19.65	
2	623	596	861.94	842.83	13.50	14.03	19.10	0:12	25.46	

Dataset 8 - 9 May 2019

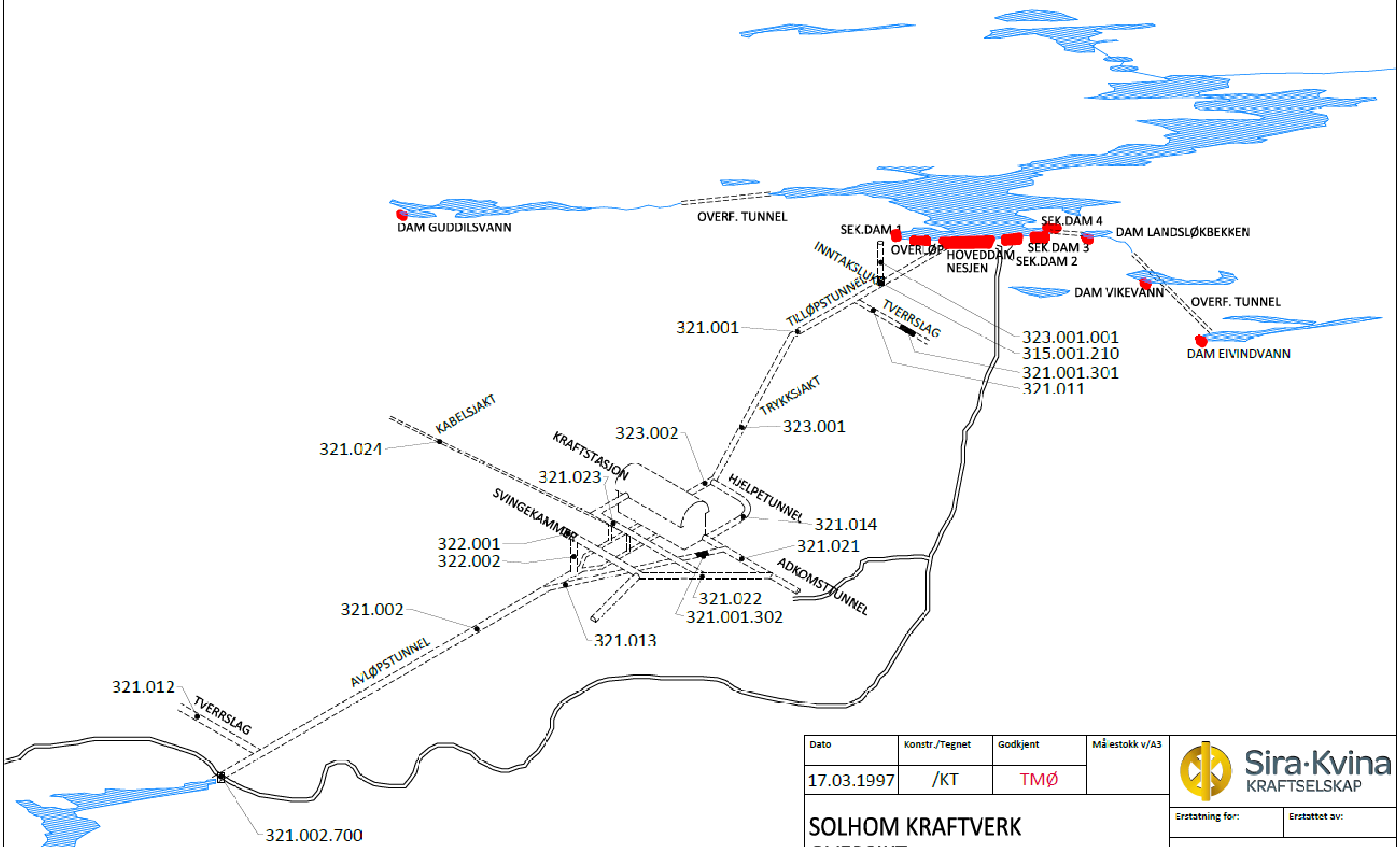


Leakage	Initial Pressure	Final Pressure	Initial Volume	Final Volume	Initial Time	Final Time	Δvolume (m^3)	ΔTime	Leakage (l/s)	Note
1	621.8	612	861.09	854.15	10.19	10.51	6.93	0:32	3.61	

Appendix D – Tafjord V Hydropower Longitudinal Drawing



Appendix E – Solhom Hydropower Plant Isometric Drawing



Dato	Konstr./Tegnet	Godkjent	Målestokk v/A3
17.03.1997	/KT	TMØ	

Sira-Kvina KRAFTSELSKAP

SOLHOM KRAFTVERK
OVERSIKT
ISOMETRISK

Erstatning for:	Erstattet av:

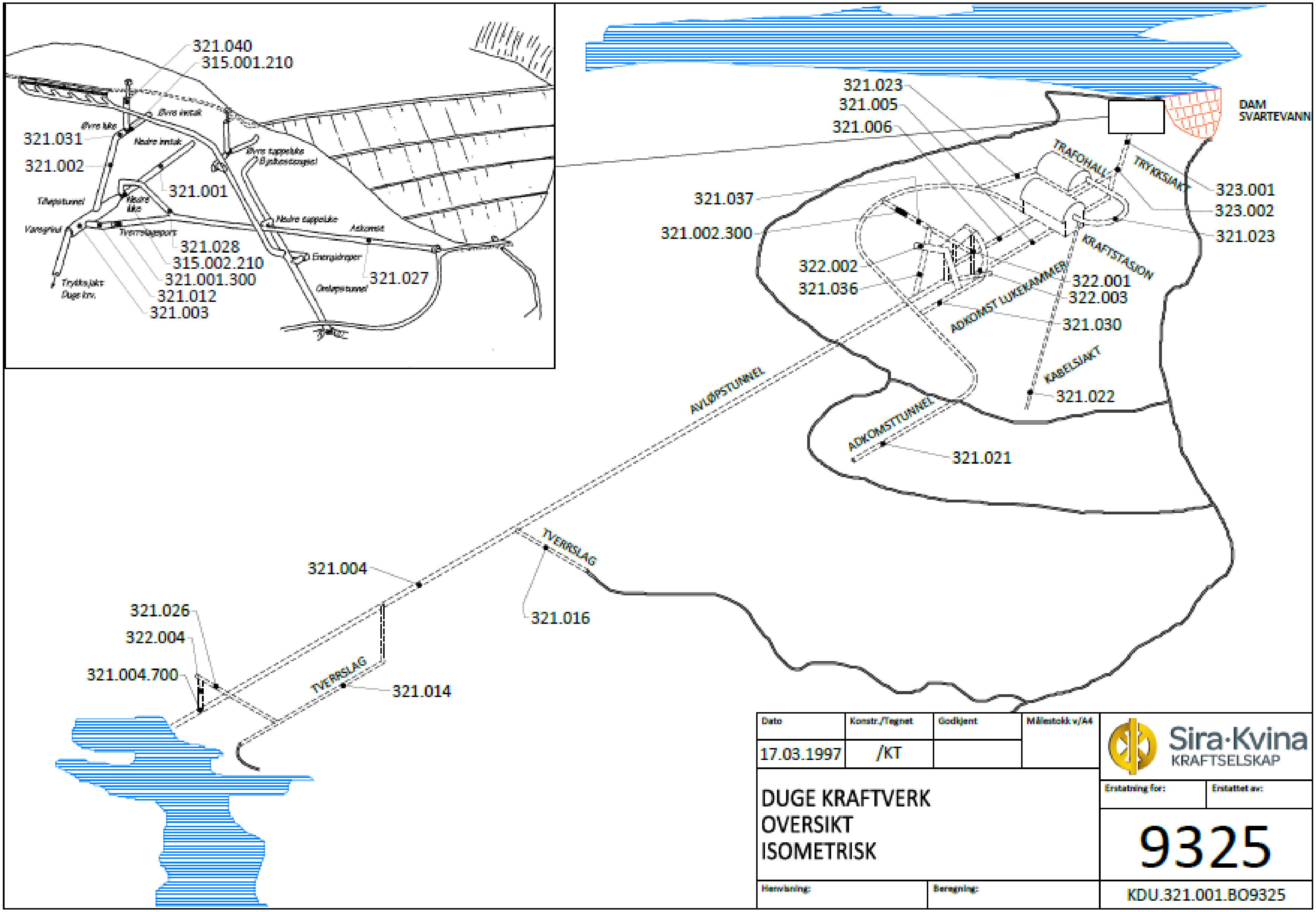
9328

Henvisning:

Beregning:

KSO.321.001.BO9328

Appendix F – Duge Pump Storage Power Plant Isometric Drawing



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