AUTOMATION OF SPILLWAY GATE OPERATIONS BASED ON
RESERVOIR WATER LEVEL MONITORING

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ABSTRACT

AUTOMATION OF SPILLWAY GATE OPERATIONS BASED ON RESERVOIR WATER LEVEL MONITORING

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Spillways are the safety structures of dams against floods. Various types of spillways having different hydraulic characteristics can be considered with variable geometric dimensions. On the other hand, hydrologic reservoir routing is an important tool in the optimization of dam reservoir operations for the purpose of water supply for various needs and electricity production in hydropower plants. Some spillways have gates to control and regulate the flow and to store an optimal amount of water. Other spillways are designed for uncontrolled flow conditions which release excess water spontaneously when the reservoir level is above the spillway crest elevation. In this study, a Gated Ogee Crested spillway is operated to imitate the discharge behavior of Piano Key Weir which functions in uncontrolled flow conditions. This process helps in performing smoother gate operations and keeping water at pre-specified levels in the reservoir. At the end, three scenarios of gate operations are discussed and an empirical formula based on reservoir water level is developed that can serve for hydrologic reservoir routing to assist in automating gate operation. In this way, gates can be opened and closed on time in case of flash floods without knowing the incoming flood hydrograph. At the end of the reservoir routing process, water in the reservoir can automatically be fixed at a pre-defined level.

Keywords: Spillways, PKW, Ogee Crested, Gate Operations

ÖZ

REZERVUAR SU SEVİYESİ İLE DOLUSAVAK KAPAK OPERASYONLARININ OTOMASYONU

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Dolusavaklar, barajların taşkınlara karşı güvenlik yapılarıdır. Farklı hidrolik karaktere sahip çeşitli tipte ve farklı boyutlarda dolusavaklar düşünülebilir. Diğer yandan, hidrolojik hazne ötelemesi farklı ihtiyaçlar için suyun kullanımı ve elektrik üretimi için hazne operasyonlarının optimizasyonunda önemli bir araçtır. Bazı dolusavaklar, akımı kontrol etmek, düzenlemek ve optimum miktarda su depolamak için kapaklarla donatılmıştır. Diğer dolusavaklar, kontrolsüz akım için tasarlanmış olup haznede su seviyesi dolusavak kret kotunu aştığında fazla su kendiliğinden boşalır. Bu çalışmada, kontrolsüz akış koşullarında çalışan Piyano Tuşlu Dolusavak'ın deşarj davranışını taklit etmek için Kapaklı Ogee Tepeli bir dolusavak çalıştırılmıştır. Bu işlem, daha yumuşak kapak operasyonları gerçekleştirmeye ve suyu rezervuarda önceden belirlenmiş seviyelerde tutmaya yardımcı olur. Çalışmanın sonunda, kapak operasyonları için üç ayrı senaryo tartışılmış ve kapak operasyonlarının otomatikleştirilmesine yardımcı olmak için hidrolojik rezervuar yönlendirmesine hizmet edebilecek, haznedeki su seviyesine bağlı bir empirik formül geliştirilmiştir. Bu sayede, ani sel durumlarında gelen taşkın hidrografı bilinmeden kapaklar otomatik olarak açılıp kapatılabilecektir. Rezervuar yönlendirme işleminin sonunda, haznedeki su önceden belirlenmiş bir seviyede otomatik olarak sabitlenmiş olacaktır.

Anahtar Kelimeler: Dolusavaklar, PKW, Ogee Tepeli, Baraj Kapısı Operasyonu

To my family who supported me through my educational journey regardless of all the obstacles and challenges.

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LIST OF ABBREVIATIONS

ABBREVIATIONS

METU	Middle East Technical University
PKW	Piano Key Weirs
OCW	Ogee Crested Weirs
USBR	United States Bureau of Reclamation
USACE	United States Army Corps of Engineering
DSI	State Hydraulic Works
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
GA	Genetic Algorithms

LIST OF SYMBOLS

SYMBOLS

I Inflow (m^3/s)

Q Discharge (m^3/s)

H Hydraulic head (m)

h Depth of Reservoir (m)

L Length of spillway (m)

C_d Discharge coefficient

g Gravitational acceleration (m^2/s)

e Gate opening(m)

A Surface area (m^2)

t Time(s)

L_{eff} Effective length (m)

B PKW Width parallel to flow (m)

W PKW width normal to flow (m)

P Weir height (m)

i notation for inlet keys

O notation for outlet keys

a A constant in the Discharge Coefficient formula of PKW

b A constant in the Discharge Coefficient formula of PKW

c A constant in the Discharge Coefficient formula of PKW

C	Correction coefficient for the newly proposed formula

S	Storage (m^3)

Q1	Discharge for the first opening

Q2	Discharge at the target level

CHAPTER 1

INTRODUCTION

1.1 Spillways

A spillway is a hydraulic overflow structure that is mainly designed for carrying excess amount of water from upstream to the downstream of a dam or a reservoir. A spillway is also meant to prevent the overtopping of a dam, used as a dam safety structure. Spillways are designed so that reservoirs keep functioning above normal operating levels, hence storing as much water as possible. Yanmaz (2018) states that the spillways are consisted of three major parts. The first part accepts water inflow called approach facility. The second part discharges water to the downstream and the third part of it is responsible for energy dissipation. Spillways have different shapes due to hydrologic, geometric, geologic and economic demands. The most commonly used spillway shapes are ogee crested, chute, sideways channel, labyrinth, stepped, shaft, siphon, and piano key spillways. Although the piano key spillway or piano key weirs (PKW) is a newly introduced type, its use is steadily increasing as it keeps reservoir level in a prescribed range without any gates installed. In the past, most of the spillways were designed to have linear crest shapes. However, the recent challenges caused by a warming climate forced the designers of overflow structures to think of different and more hydraulically efficient spillway designs. Due to climate change, an increase of 162% to 507% in the amount of rainfall and maximum probable precipitation (PMP) may result in a 48.9% rise in the probable maximum flood (PMF) in years (2031-2045) as stated by Sammen et al. (2022). This increase in the PMF requires upgraded spillways structures with larger discharge capacities. For this reason, different spillway designs having higher discharge efficiencies have been examined in hydraulic laboratories worldwide. Conforming to Abhash and

Pandey (2021), out of all the dam failures, 23% are only caused by an insufficient discharge capacity of the dam spillways. According to Tullis et al. (2020) nearly 2170 dams only in the U.S need to be rehabilitated due to hydraulic deficiencies. As claimed by the authors, most of these deficiencies occur within the spillways of the dams. To solve the deficiency problem, Anderson and Tullis (2013) suggest three general solutions; increasing weir width, lowering crest elevation and increasing footprints of the weirs.

So far, labyrinth spillways and PKWs have been used as reasonable alternatives to replace old (linear) weirs. As reported by Anderson and Tullis (2013), labyrinth weirs can increase the discharge capacity by 3-4 times compared to a linear weir. Machiels (2012) claims that a PKW can be four times more discharge efficient than an ogee crested weir of the same width and 10% more efficient than a labyrinth weir of the same shape. The advantage of PKW over an ogee crested spillway comes from the extra crest length and the advantage over labyrinth comes from the extra discharge capacity produced by the overhangs of the PKW. Moreover, Paxson et al. (2013) argue that a PKW is 10% more cost-efficient than a labyrinth weir. Although PKWs are more efficient from the perspectives of cost and discharge, a combination of both OCW and PKW can be used in some cases. Aşağı Kaleköy dam, which is used as an example in this study, was designed as a PKW system at first but, later the spillway design has been changed to a gated OCW because of some environmental problems.

According to Sordo-Ward et al. (2016), out of all the large dams that exist around the globe, 30% of them are equipped with gated spillways. For this reason, gate operation plays an important role in storing water in the reservoirs and producing energy.

To perform gate operations, different scenarios can be considered. For instance, partial or full opening of gates may be considered. Gates can also be opened one by one or all at once. The speed with which a gate should be closed or opened may differ from one reservoir to another. In either case, sudden and full opening or closing of gates should be avoided. Sudden opening and closing of hydraulic gates is undesirable due to the safety guidelines of the downstream structures and the gates.

In this study, the discharge capacity and hydraulic characteristics of Piano Key Weirs (PKW) and Ogee Crested Weirs (OCW) are compared based on hydrologic reservoir routing. In case of PKW, uncontrolled discharge is computed from appropriate formula as there are no gates. For the OCW, the gates are operated taking three different scenarios into consideration. In the first case, gates are opened stepwise by 10 cm and 15 cm increments according to the discharge requirements. In the second case, gates are opened or closed to fix the reservoir level at a target value. In the third case, a numerical formula is applied to the reservoir routing to automatically open and close gates at appropriate time intervals. Although the third method is more complicated, the method can be used to develop automatic gate operations.

To conduct the reservoir routing, Runge Kutta and Euler's methods are used. Finally, a mathematical formula is suggested to perform reservoir routing process automatically and store water at pre-specified levels. The formula can be written as a computer code to open gates automatically. In this formula, pre-recorded water level data is used. From the recorded data, rate of change in the reservoir storage and rise speed of the water level can be computed. As a result, gates operations are performed based on the reservoir water level data instead of the inflow data. This mathematical formula assists in automating the gate operations so that the gate will be opened on time in case of flash floods and other unexpected emergency situations such as earthquakes and landslides. This method may prove to be very helpful for dams located in remote areas where reaching the reservoir gates requires longer period of time.

1.2 Flow Routing

Flow routing is a process used for determining time and magnitude of flow while it passes through channels, reservoirs and other hydraulic structures. The flow routing is called flood routing when the inflow values are from a flood event. When the flow goes through a reservoir, the process of routing is called the reservoir routing process. Reservoirs are hydraulic structures mainly used to store water. The reservoir routing process is used for finding design discharge for outflow structures for the safety of dam-reservoir system. Another purpose of performing reservoir routing is to safely transfer water to the downstream in case of a flood while storing as much water as possible. The stored water can be used for energy production, irrigational or other purposes based on design concept of the dam.

Flow routing is divided into two categories of lumped and distributed routing systems (Chow, 1988). In a lumped system the flow is only a function of time while in a distributed system the flow is function of both time and space. The lumped system is also called hydrological routing system while the distributed system is called hydraulic routing system. In this study, we use hydrological reservoir routing, since our inflow values only depend on time. After the reservoir routing process is performed one can determine design discharge of the overflow structure, translation and attenuation intervals of the outflow, surface water level and the volume of stored water.

For the hydrological reservoir routing system, inflow is used as an input variable to compute the amount of outflow, water surface water level and storage. A reservoir is called narrow if its depth and width are small. These types of reservoirs have variable storage-discharge relationship. The storage-discharge relationship is a function of time. In our case, the reservoir is wide and has a larger depth. For this reason, it has invariable discharge-storage relationship. The storage and the discharge are both functions of depth and the hydraulic head respectively. Continuity equation is used to calculate the desired discharge and storage as shown here.

$$\frac{\Delta S}{\Delta t} = I(t) - Q(H) \tag{1.1}$$

where S stands for storage, I for inflow, Q for discharge and H for the hydraulic head over spillway crest. This equation has one known and two unknown variables. To solve this equation different reservoir routing methods have been used. In the Level Pool method, an extra equation relating depth of the dam to the storage is written. After writing the second equation, an iterative process is followed to find maximum values of discharge and water head. Some methods used graphics and interpolation to solve for the desired values between specific time steps. Other methods do not require a second equation to solve Equation (1.1). Instead, small increments of time are used to calculate the rise in the head. These methods make use of differential equations to calculate the speed of water head rise in the reservoir. In this study, two of these differential methods called Euler's and Runge-Kutta methods are used. Euler's method is simpler and easier to use but Runge-Kutta method has various orders of accuracy. Overall, both of these methods are easy to be applied to a reservoir routing process. More details of these methods will be discussed in the following sections.

1.3 Research Aims and Methodology

There have been a lot of research studies regarding PKWs and the gated Ogee Crested spillways. Most of the numerical and physical studies focus on the design, discharge behavior and discharge efficiency of the spillways. While ogee crested spillways have been investigated for a longer time, the PKWs have been introduced to literature recently. Therefore, general design guidelines are still the focus of new research studies. The OCW is mostly designed with gates to regulate the flow of a dam or reservoir. By contrast, the PKW is designed for free flow conditions which releases excess water to downstream whenever a flood occurs. Using hydrologic reservoir routing methods, the amount of discharge flowing through a spillway can be defined by determining the design head and the design discharge for each spillway. While the PKW and all other uncontrolled spillways keep the reservoir water volume at constant levels equal to the height of the dam, for the controlled spillways the water can be stored at any level above the crest. In this study, hydrological reservoir routing method is used to compare gated OCW and PKW.

The gates of an ogee crested spillway can be operated in several ways to transfer water downstream in a safe manner. One of these ways may be opening the gates by an operator in the control room. However, in the case of emergency situations such as flash flood, earthquake and landslides, reaching the gates may not be possible. In such cases, gates may be operated remotely. Another choice may be automatic gate operations based on computer program, which utilize water level data obtained from electronic sensors.

This study aims at investigating possible methods to propose a computer algorithm for automating reservoir gate operations. At first, gates of an ogee crested spillway are operated to imitate the discharge behavior of a PKW to develop algorithms. This way the excess water is discharged safely and the water level is also kept at a required level. In the second step, gate opening-closing operations based on the reservoir water level. Various water level intervals between which the gate opening and closing should start, have been studied. The goal is to keep water levels between normal operating and maximum operating levels.

CHAPTER 2

LITERATURE REVIEW

2.1 Piano Key Weirs

Piano Key Weirs (PKW) are used as an alternative to labyrinth spillways in order to increase discharge capacity and the overall hydraulic efficiency. A PKW increases the discharge capacity 4-5 times compared to a linear weir having the same heads (Hu et al., 2018). The first PKW was successfully designed and implemented in 2006 (Ribeiro et al., 2012). PKW's were introduced after labyrinth weirs but the use of PKW has increased rapidly. To find more discharge efficient spillways, some modifications were introduced to labyrinth weirs to further increase the discharge capacity and the hydraulic efficiency. Labyrinth spillways have larger base area requiring more space on the top of a dam's crest so that the spillway can be placed on it. Since old dams, especially earth-fill dams have small areas on the top of their crests, PKW can be selected as a better alternative. A PKW has smaller bottom, wide front edges and rectangular shape. The overall shape of the spillway resembles to that of a piano and that is why it is called piano key spillway. Due to its small bottom area, it is easier to place a PKW on top of on old reservoir or earth-fill (gravity) dam.

Geometrically a PKW is consisted of two parts. One part is the outlet keys and the other part is the inlet keys. When flow goes through a PKW, it can flow through inlet keys and the side crests of the keys. PKW have higher discharge capacities because of the lateral flow produced by the side crests of the weir. Reportedly, a PKWs is more discharge efficient when the upstream head is low. By contrast, if the upstream head is higher, the side crest flow between inlet and outlet key is greatly decreased, hence the overall discharge efficiency is reduced. Recent research studies use weir height over head-ratio to study effects of hydraulic head on discharge capacity. For this reason, most recent studies use equations in which coefficient of discharge is written in terms of the hydraulic head ratio. There have been a lot of experimental and numerical studies conducted to improve the head-discharge formulas of piano key weirs. Although experimental studies were conducted to write empirical formulas for head-discharge and discharge coefficient relationships, experimental studies are sometimes expensive and prone to scale effects. For this reason, more numerical studies using various software packages have been used recently. A recent study by Koken et al. (2022) found that the empirical equations obtained from experimental studies were greatly affected by experimental parameters and contained error. They used Flow-3D software to conduct a numerical study to investigate the effects of some geometric properties on the discharge behavior of PKWs which will be discussed in details in the next sections.

PKWs are divided into different categories considering their geometry. Considering the location of cantilevered hangovers most common PKWs are; Type-A, Type-B, Type-C and Type D (Abhash & Pandey, 2021). Type-A PKW has symmetric overhangs in the upstream and downstream, Type-B has only upstream overhangs, Types-C with downstream overhangs and Type-D piano key weirs have no overhangs. Important geometric properties of the piano key spillways are discussed in the following section.

2.1.1 Geometric Features of a PKW

A PKW unit can be described by various geometric parameters. Some of these parameters have profound effects on the discharge capacity and overall efficiency while others have smaller or negligible effects.

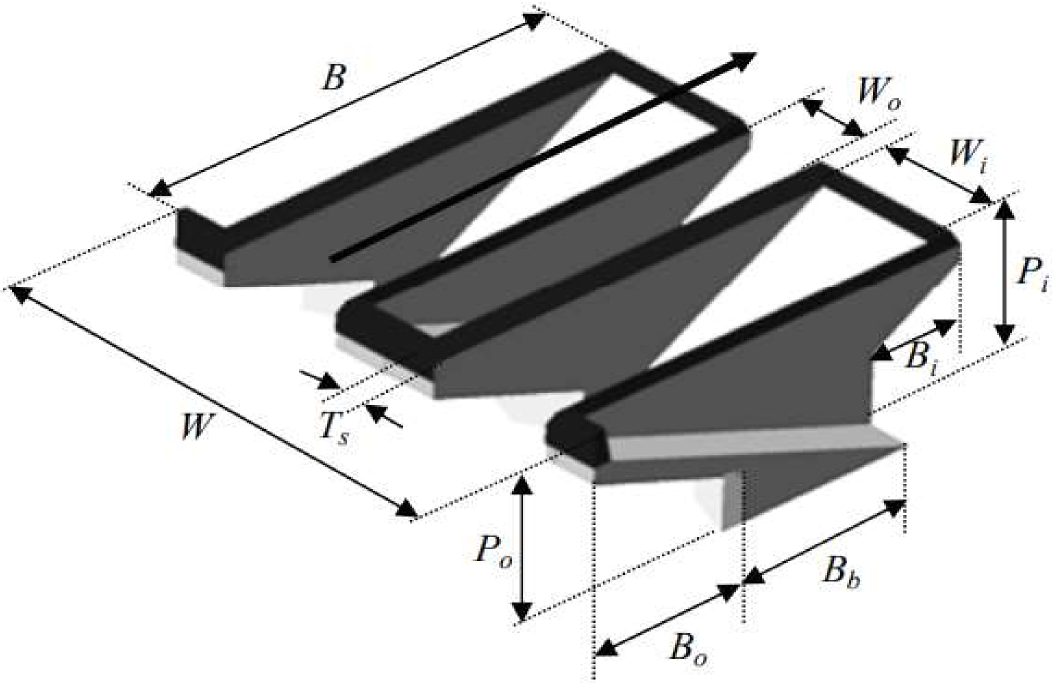

Figure 2.1: Geometric Parameters of a PKW (J. Pralong et al., 2011)

Important geometric elements are defined in Fig. 2.1, where:

i: notation used for inlet keys

o: notation used for outlet keys

B: length of PKW in the upstream-downstream direction

W: total width of the PKW crest

P: height of the outlet or inlet entrances measured from crest level

Ts: wall thickness

There have been a lot of research studies investigating discharge capacity, hydraulic efficiency and geometric properties of PKWs. In order to study discharge capacity and hydraulic efficiency of the weir, one needs to use proper head-discharge relationships. Over the years, different head-discharge formulas with various methods of expressing discharge coefficients have been investigated both experimentally and numerically. Discharge and discharge coefficient values are affected by the geometric properties of weir. For this reason, using an appropriate head-discharge relationship is essential for determining discharge.

The discharge coefficient formula used in this study contains the effects of W_i/W_o ratio, hydraulic head H, weir height P, the overhang ratio Bi/Bo and the H/P ratio on the discharge efficiency of the PKW.

The W_i/W_o ratio is taken as 1.25 to calculate the discharge coefficient using Equation (2.3). Anderson and Tullis (2013) tested PKW models with Wi/Wo ratios of 0.67, 0.8, 1, 1.25 and 1.5. Out of these models, the $PKW_{1.25}$ was reported to have higher discharge efficiency. The efficiency of discharge increases as the W_i/W_o increases relative to $W_i/W_o = 1$, (Anderson, 2011). The discharge efficiency may decrease as the W_i/W_o ratio further increases. Likewise, Kabiri-Samani and Javaheri (2012) stated that a PKW is most efficient when the W_i/W_o ratio is 1.22, after which the value of the coefficient of discharge stays constant. As reported by Koken et al. (2022), the W_i/W_o ratio has no obvious effect on the discharge capacity but for higher water heads the discharge efficiency improves when the value of the Wi/Wo ratio is close to 1. Erpicum et al. (2014) claims that an increase in W_i/W_o ratio improves the overall discharge efficiency by 30% regardless of the value of weir height (P). G. Li et al. (2020) found that at lower heads and a width ratio of $W_i/W_o > 1$, the discharge capacity can improve if the width ratio increases. Machiels et al. (2014) shows that the most reasonable limit for the width ratio is within the interval of 1.29 to 1.57, whatever the value of P.

The overhangs effects on the discharge performance of piano key weirs have also been studied. The discharge equation used in this study is valid only for Type-A PKW which has symmetrical overhangs. PKWs are classified according to the location of the overhangs as Type-A, Type-B, Type-C and Type-D. According to Kabiri-Samani and Javaheri (2012) both upstream and downstream overhangs positively affect the discharge efficiency. In addition, the upstream overhangs were reported to be more efficient according to the authors. Similarly, Anderson (2011) claimed that Type-B piano key weirs perform better than Type-A piano key weirs, since Type-B weirs have larger upstream overhangs. Koken et al. (2022) found that the discharge efficiency increases as the ratio B_i/B_o decreases. According to the authors, as the length of the parameter B_o increases, so does the wetted perimeter of the outlet key and it results in improving the overall efficiency of the weir. Erpicum et al. (2014) concluded that the Type-B PKW was more efficient only at lower heads and the overhang ratio can improve the overall discharge efficiency by 20%.

While the height of a dam has no influence on the discharge capacity as studied by Koken et al. (2022), the discharge efficiency improves if the weir height (P) increases. Erpicum et al. (2014) found that the total discharge efficiency increased as P/W_u increased up to 1.33. For this reason, some of the empirical equations of head-discharge relationships are normalized with the weir height (P). (Olivier Machiels et al., 2014) shows that the discharge capacity improves with the increases in weir height if $P/W_u<1$.

Another important factor that profoundly influence the discharge efficiency of a PKW is the hydraulic head. Generally, a PKW is very effective at lower heads. In agreement with Ribeiro et al. (2012), at higher head ratio, the outlet keys get drowned hence not able to transfer the flow efficiently. Moreover, the jet over crossing caused by the lateral flow decreases the overall discharge efficiency. Guo et al. (2019) argued that the design head ratio in practice is selected as a value between 0.13 and 0.66, otherwise the discharge capacity will be affected adversely. According to the authors when $H/P < 1$, the improvement in discharge capacity is considered to be between 1.2-3.5. Furthermore, Hu et al. (2018) show that the discharge capacity of a piano key weir is 4 to 5 times of an ogee crest or sharp crest weir at lower heads. The authors argue that the change in the direction of lateral crest flow caused by the longitudinal flow, local submergence in the outlet keys and the interference of the two opposing nappes are the main reasons of reduction in the discharge capacity under higher hydraulic heads. According Kabiri-Samani and Javaheri (2012) under smaller head ratios, the flow of a piano key weir can be observed both in form of a jet (longitudinal) and thin a sheet (lateral). At higher heads the two nappes emerge with each other, forming one nappe resulting in the reduction of the hydraulic efficiency. Erpicum et al. (2014) introduced discharge capacity ratio and concluded that the capacity ratio sharply decreased as the head ratio (H/P) increased. Generally, as the head ratio increases the PKW tends to behave as a linear weir, losing its discharge efficiency. Discharge equations contain the head (H) as a dominant parameter that changes the efficiency and hydraulic behavior of the PKWs. Although most of the head-discharge relationship holds when the ratio H/P is between 0.1 and 0.9, S. Li et al. (2020) show when H/P<0.1 the discharge capacity is mainly influenced by wall thickness and shape of the crest of a PKW. To understand the importance of the head in the PKW discharge equations, the following section is dedicated to the head-discharge relationships obtained by prominent scholars. These equations are the results of numerical and experimental research studies conducted by these scholars in the recent years.

2.1.2 Head-Discharge Relationships of a PKW

Discharge over a PKW is a function of spillway crest length, crest width, width and length of outlet and inlet keys, hydraulic head, crest height of the spillway, slope of the inlet and outlet keys, wall thickness, crest shape (half rounded, quarter rounded), fluid density, surface tension and the acceleration of gravity. Lempérière (2009) showed that the unite discharge q can be expressed in terms of water head and weir height.

$$q = 4.3H\sqrt{P_m} \tag{2.1}$$

where H is head and Pm is maximum weir height. Crookston et al. (2018) suggested three predictive methods for finding head-discharge relationships of a piano key weir. The first way is to find suitable length ratios for designing a piano key weir empirically. The second way is to find numerical equations relating discharge with head and discharge coefficient. The third way to create CFD model that can accurately represent behavior of an actual weir while flow is passing through it. The CFD models are said to be in close agreement with the experimental data and are less expensive. The following head-discharge relationship used by Crookston et al. (2018) is shown.

$$Q = \frac{2}{3} C_d L \sqrt{2g} H^{\frac{3}{2}} \tag{2.2}$$

where L is total crest length and H is total hydraulic head and Cd is the coefficient of discharge. After a best line was fitted to the experimental data and the following equation for Cd was obtained.

$$C_d = \frac{1}{a + \frac{bH}{P} + \frac{c}{\frac{H}{P}}} + d \tag{2.3}$$

where a, b, c and d are empirical constants having values of 0.4216, 9.412, 0.1027, and 0.1114, respectively. Equation (2.3) is referred to as Eq. (2) by Crookston et al. (2018) in Figure 2.2 is compared to discharge coefficient curves obtained by Anderson and Tullis (2013).

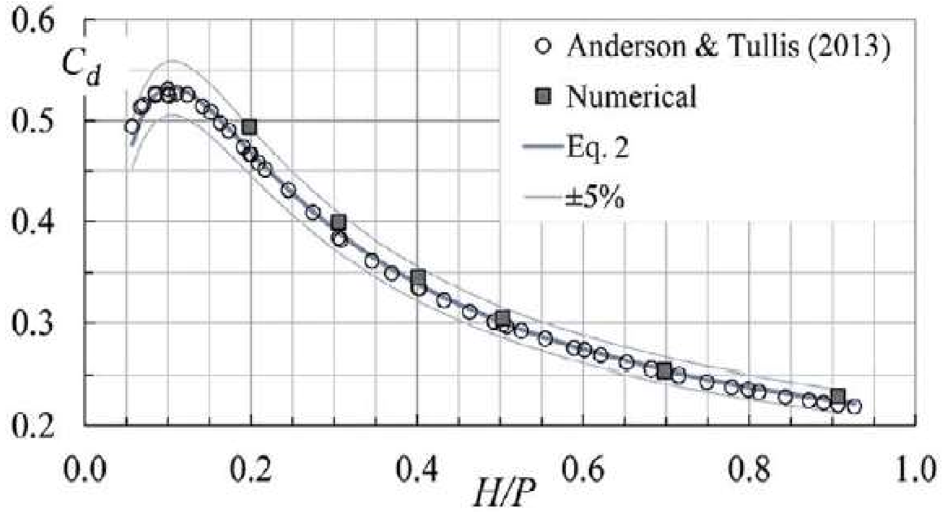

Figure 2.2: Computed Cd Values for Width Ratio Wi/Wo=1.25, (Crookston et al., 2018)

The results of the two studies seem to be in close agreement with each other. The coefficient discharge curves obtained by Anderson and Tullis (2013) for Wi/Wo ratios of 0.67, 0.8, 1, 1.25 and 1.5 are shown below. The authors also provided discharge coefficient curves for a modified piano key weir.

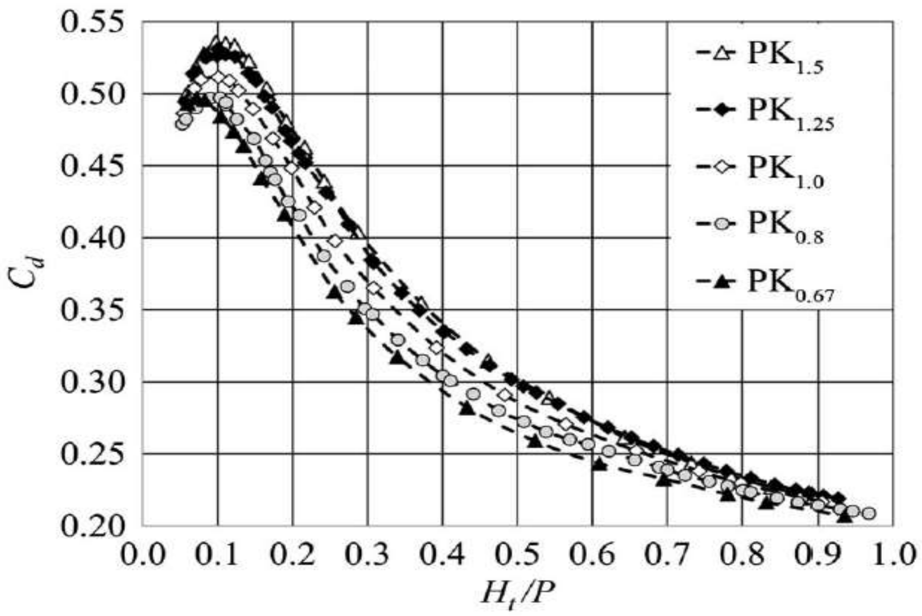

Figure 2.3: Discharge Coefficient Curves for Wi/Wo Ratios (Anderson and Tullis, 2013).

Guo et al. (2019) suggested an analytical formula for expressing the discharge coefficient Cd as written below. The formula was reported to be in close agreement with the original data.

$$C_d = 0.1 + 0.258\left(\frac{L}{W}\right)^{0.45}\left(\frac{B}{P}\right)^{0.1}\left(\frac{Wi}{Wo}\right)^{0.05}\left(\frac{H}{P}\right)^{-0.465} \tag{2.4}$$

Abhash and Pandey (2021) conducted a numerical study on head-discharge relationship of piano key weirs at low heads. The authors compared the numerical study with another study conducted by Kabiri-Samani and Javaheri (2012). The discharge coefficient formula provided by the authors is stated as below.

$$C_d = 0.60 + 0.212\left(\frac{L}{W}\right)^{0.377}\left(\frac{B}{P}\right)^{0.1}\left(\frac{Wi}{Wo}\right)^{0.426}\left(\frac{H}{P}\right)^{-0.675} * \left(\frac{B}{P}\right)^{0.306} * e^{\left(1.504\left(\frac{Bo}{B}\right)+0.093\left(\frac{Bi}{B}\right)\right)} \tag{2.5}$$

According to the authors, Equation (2.5) gives better results when H/P is smaller than 0.9. Kabiri-Samani and Javaheri (2012) stated that Equation (2.5) has max uncertainty of 3% and average uncertainty of 0.7%, hence the equation can be used to predict discharge for free flow conditions efficiently. Similarly, Ribeiro et al. (2012b) as cited by Guo. et al. (2019) proposed a discharge coefficient formula for the discharge capacity of a PKW.

$$C_d = 0.42\left[0.8 + 0.34\left(\frac{L}{W}\right)^{0.7}\left(\frac{P_o}{P_i}\right)^{0.25}\left(\frac{Wi}{Wo}\right)^{0.08}\left(\frac{P_i}{H}\right)^{0.82}\right] \tag{2.6}$$

Since the physical studies are expensive and prone to scale effects, more research works focus on numerical studies involving various software packages. Numerical studies are also reported to be more accurate in terms of predicting discharge performance of a PKW. A recent study by Koken et al. (2022) with the a large PKW model shows that the data obtained from equations proposed by Kabiri-Samani and Javaheri (2012), Anderson and Tullis (2013) and Machiels et al. (2014) diverges from the numerical results by 17.5% (at higher heads), 29% (at higher heads) and 8.6% (at lower heads) respectively. Koken et al. (2022) proposed a new formula for designing PKWs.

$$\frac{h}{L_u} = \exp(-1.92\, ln\, r - 0.918) \tag{2.7}$$

Where h is head over the PKW, Lu length of a unit of PKW and r is discharge capacity ratio relative to linear weir. In this study, Equation (2.2) is used to express head-discharge relationship and Equation (2.3) for computing the discharge coefficient.

2.2 Ogee Crested Spillways

Ogee crested spillways are the most studied and commonly used type of spillways. The ogee crested shape is similar to the shape of S and was reportedly first introduced by Muller in 1908. Since then many other physical features have been introduced to this type of spillway to change its hydraulic properties and maximize its discharge efficiency. Ogee crested spillway is commonly used with gates to control flow going over the spillway. In the past, most of the studies conducted on ogee crested spillways consist of laboratory tests and physical experiments. The most important of all these studies is the design charts provided by USBR and USACE obtained by performing physical experiments. However, the last advancements in the computational field have encouraged researchers to focus more on numerical modelling of the spillway. Numerical modelling of ogee crested spillways are less expensive and free of scale effects. With 1-D and 2-D modelling already in use, more focus is concentrated on 3-D modelling for more accurate predictions of discharge capacity, flow depth and flow velocity. CFD modelling software packages such as FLOW-3D are commonly used. Numerical models are adequately effective to be used in predicting the discharge and pressure distribution over an ogee crested spillway (Savage and Johnson, 2001).

An ogee crested spillway is more effective when the hydraulic head is not higher than the design head. Under high hydraulic head the ogee crested spillway behaves like a sharp crested weir. According to Savage and Johnson (2006), four types of flow can occur when water is transferred downstream by an ogee crested spillway. These types of flows are described as supercritical jet of water, occurring of a true hydraulic jump on the face of the spillway, drowned jump and jet breaking when the ogee crest acts as a broad crested spillway. Imanian and Mohammadian (2019) state, when the head ratio of actual head/design head > 1, the relative pressure becomes negative. As the head ratio further increases the discharge coefficient also increases until it reaches a constant value. The authors argue that further increase in the head ratio causes a sudden decrease in Cd value, since the water layer thickness increases and the ogee crested spillway starts behaving as a sharp crested weir. Furthermore, if water level in a reservoir is higher than the designed level, it leads to suction as negative pressure is created on the surface of the crest. On the other hand, if the local pressure falls below the atmospheric pressure, the risk of cavitation increases. Since cavitation damages the surface of the spillway, improvement of negative pressure should be avoided. The authors also indicate that the discharge coefficient increases until the actual head over design head is 5. After $H_e / H_0 > 6$, the discharge coefficient values decrease as an eddy is formed under the crest of the spillway. This means the flow is separated from the crest resulting in reduced discharge coefficient values. The pressure distribution over the crest of an ogee spillway is shown in Figure 2.4.

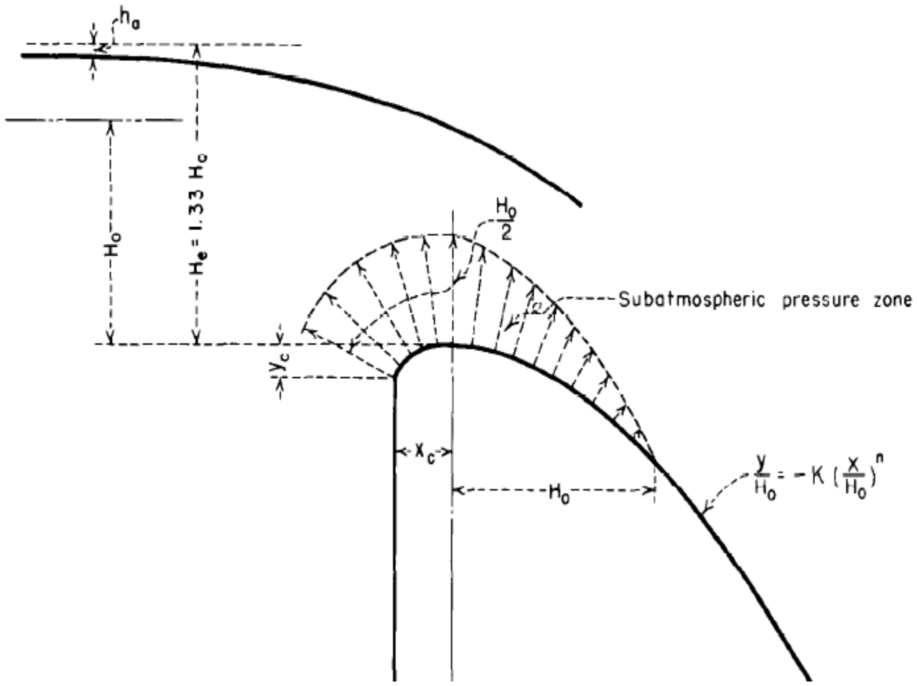

Figure 2.4: Pressure Distribution Over an Ogee Crested Spillway (Design of Small Dams)

The discharge passing through an ogee crested spillway is a function of the weir length and the water head in the upstream. Discharge in controlled conditions where gate is installed for flow regulations, is affected by the gate opening too.

According to Yildiz et al. (2020), the discharge coefficient ratio C_0/C (design discharge coefficient / Design discharge coefficient for other heads) increases as the head ratio (H_0/H_e) increases. The authors argue that the discharge efficiency also improves if negative pressure develops on the crest of the spillway but caution must be taken to avoid cavitation. From the previous research studies, it can be concluded that the discharge efficiency decreases when the head is smaller than the design head as the atmospheric pressure developed on the face of the crest resists the flow. It was stated that the discharge coefficient values of the sloped upstream crest give better results compared with other upstream face shapes.

The discharge coefficient of an ogee crested spillway in free flow conditions is affected by the upstream head, head other than the design head, upstream slope or inclination of the face, and the downstream submergence. Erpicum et al. (2018) conducted a physical study which concluded that the discharge coefficient increases until the head ratio is 5.5. At higher head ratios the discharge coefficient sharply declines as a result of flow detachment from the crest of the spillway. The study also found that the discharge coefficient decreases if the upstream quadrant head is higher than the downstream quadrant head. Moreover, the authors claim that the cavitation risk coming from higher head ratio is not that significant as reported in the previous research studies.

2.2.1 Geometric Properties of an Ogee Crested Spillway

The flow behavior is immensely affected by geometric parameters and pressure distribution over the spillway. For this reason, the geometry of the spillway needs to be studied in a detailed manner. Different geometric shapes of ogee crested spillway are suggested to optimize discharge capacity and hydraulic efficiency. The discharge behavior of an ogee crested spillway mainly depends on head, height of overflow, inclination of the upstream face and pressure distribution. The USBR and USCE also provided different design charts of an ogee crest but these charts are valid under special conditions. Moreover, the charts suggested by USCE have discontinuities in the curvature, hence it cannot be used for computational purposes. Analytically, the lower nappe geometry cannot be determined. USCE (1970) suggested a three-point arc for the upstream and a power function for the downstream part of ogee crested spillway. Working with charts and graphs is difficult when performing iterative calculations. For this reason, empirical equations or mathematical formulas for discharge are used. The details of the three-point arc are shown in Figure 2.5.

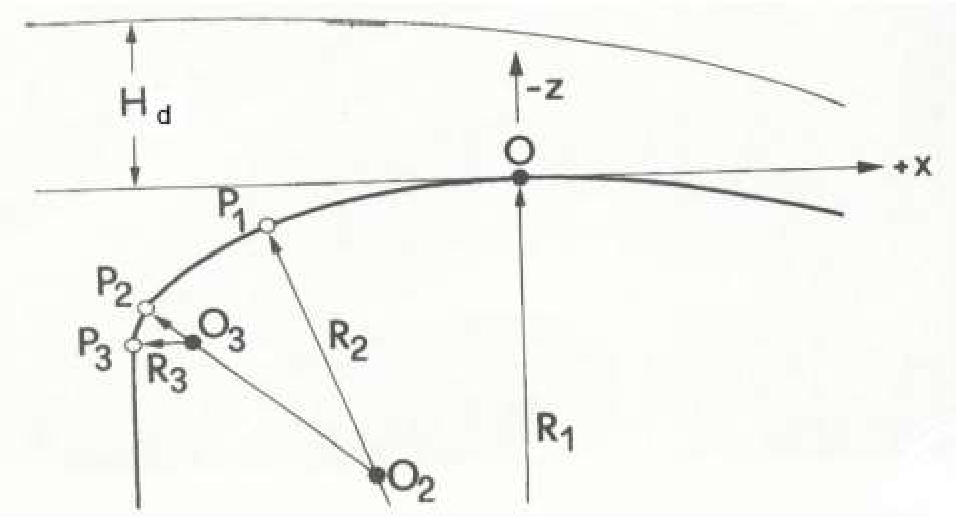

Figure 2.5: Geometric Features of an Ogee crest

Where O1, O2 and O3 are origins of the curvature, R1, R2 and R3 are radii of the curvature and P1, P2 and P3 are transition points of the curvature. The radiuses of the curvature are determined according to the design head of the spillway as shown in the following equations.

$$\frac{R1}{Hd} = 0.5, \qquad \frac{R2}{Hd} = 0.2 \qquad and \qquad \frac{R3}{Hd} = 0.04$$

For the downstream part of the ogee crest the following power function is suggested by Craeger.

$$\frac{Z}{Hd} = 0.5\left(\frac{x}{Hd}\right)^{1.85}, \ for \quad x > 0 \tag{2.8}$$

Where Z and x are origins of the Cartesian coordinates and H_d stands for design head. An ogee crested spillway is consisted of piers and abutments for installing control gates. The length of the spillway should be changed according to the effects of abutments and piers of the spillway. For this reason, abutment and piers effects are included in the effective length of the spillway. More details of the spillway are shown in the Figure 2.6.

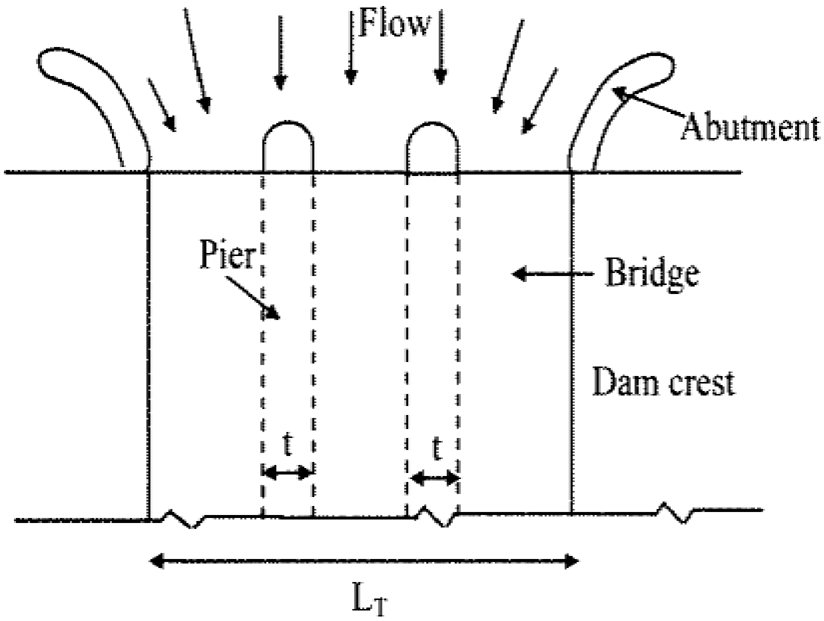

Figure 2.6: Plan View of Spillway (Yanmaz, 2018)

As seen in the Figure 2.6, flow is contracted when it passes through abutments and the piers. K_P and K_a are pier and abutment contraction coefficients determined by using charts provided by Design of Small Dams.

2.2.2 Head-Discharge and Discharge Coefficient Relationship of Ogee Crested Spillway

Head-discharge relationships of ogee crested spillway are different for controlled and uncontrolled conditions. The discharge coefficients also vary for gate and no gates conditions. There have been numerical and physical model studies to predict the discharge of ogee crested spillways as accurate as possible. Firstly, the discharge-relationships for free flow conditions are written here.

The well-known research of USBR (1987) suggested the use of the following head-discharge equation for discharge calculations.

$$Q = C_0 L H_0^{\frac{3}{2}} \tag{2.9}$$

Where C_0 is discharge coefficient for varying head, L is effective length and H_0 is the actual head over the spillway. To calculate the coefficient of discharge for the design head, the following chart was suggested by the Design of Small Dams.

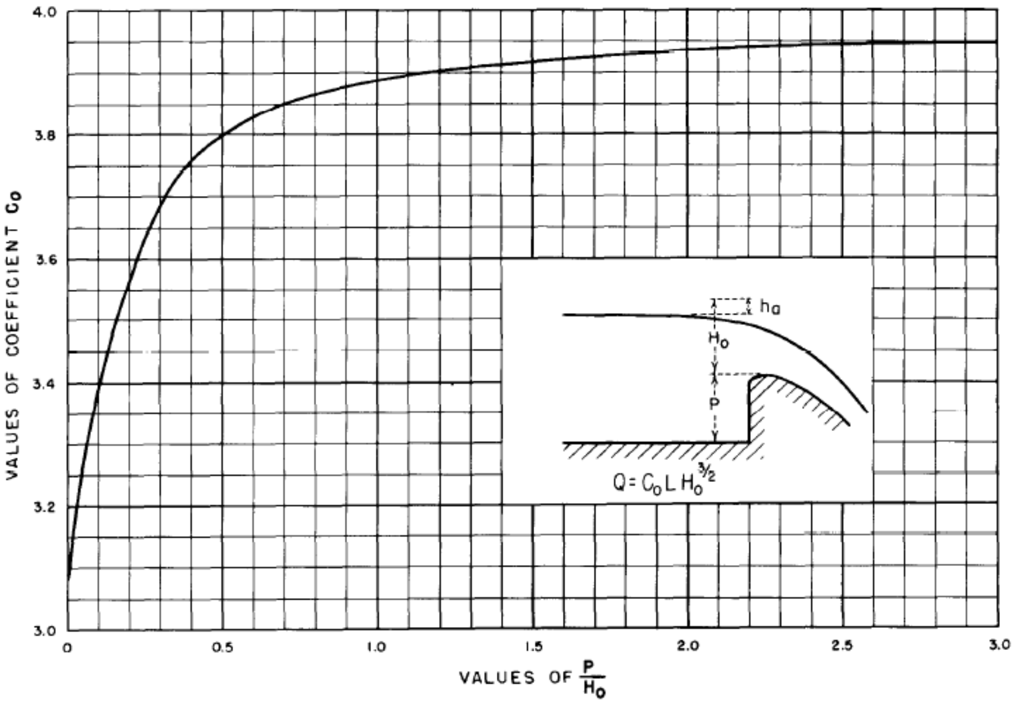

Figure 2.7: Discharge Coefficient for Vertical Faced Crest (USBR, 1987)

The discharge coefficient for varying head is calculated from the chart in the Figure 2.8.

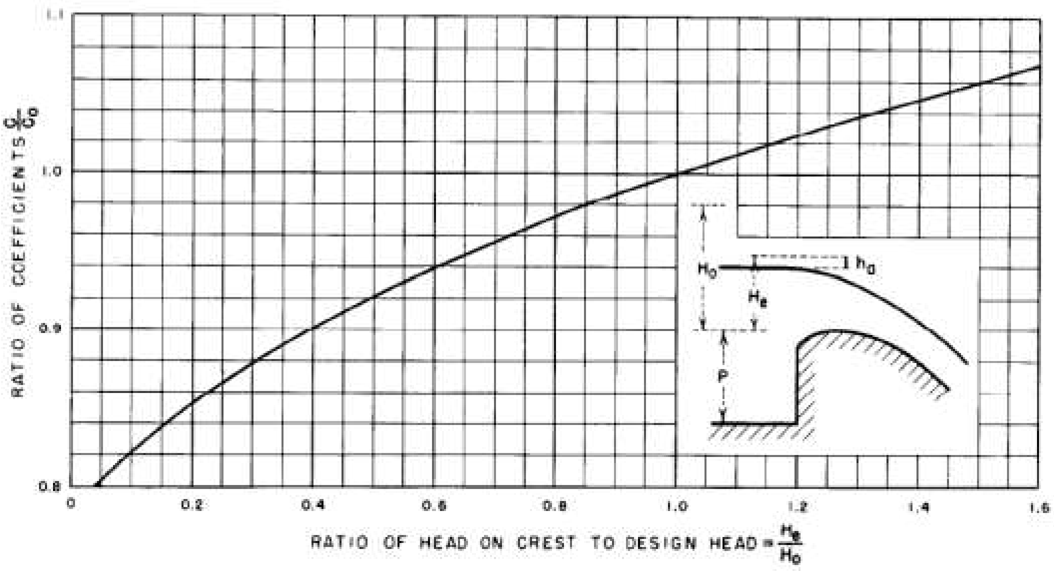

Figure 2.8: Design Coefficient for Head Other Than the Design Head (USBR, 1987)

The authors of the Design of Small Dams also provided charts to include the effects of upstream crest shape and submergence. Yanmaz (2018) used Equation 2.9 for head-discharge and presented the following mathematical equation for calculating the design discharge coefficient of vertical face ogee crested spillway.

$$C_0 = -0.0201 * \left(\frac{P}{H_0}\right)^6 + 0.2148\left(\frac{P}{H_0}\right)^5 - 0.915\left(\frac{P}{H_0}\right)^4 + 1.982\left(\frac{P}{H_0}\right)^3 - 2.3081\left(\frac{P}{H_0}\right)^2 + 1.7719 \tag{2.10}$$

The discharge coefficient for varying heads is written in Equation (2.11).

$$\frac{C_{me}}{C_0} = 0.03\left(\frac{H_e}{H_0}\right)^3 - 0.14\left(\frac{H_e}{H_0}\right)^2 + 0.32\left(\frac{H_e}{H_0}\right) + 0.79 \tag{2.11}$$

The author also presented equations to include upstream inclination, apron and submergence effects. The following equations for computing discharge and discharge coefficient of an Ogee Crested Spillways operating in free flow conditions were used by Design of Hydraulic Structures Course taught at METU.

$$Q = \sqrt{2g} * C_d * L_{eff} * H^{1.5} \qquad (2.12)$$

The discharge coefficient is calculated from the following equation which is valid for H/Hd <=3.

$$C_d = \left(\frac{2}{\sqrt{27}}\right)\left(1 + 4 * \frac{\frac{H}{H_d}}{9 + 5 * \left(\frac{H}{H_d}\right)}\right) \qquad (2.13)$$

Where H is the variable head over the ogee crest and H_d is the design head of the spillways. Bagatur and Onen (2015) also used Equation (2.9) to express the head-discharge relationship and used Equation (2.14) for determining design discharge coefficient.

$$C_0 = \left[1.74 + 9.24\left(\frac{P}{H_0}\right)^{0.25} - \left(\frac{P}{H_0}\right)^{1.25}\right]^{-0.33} \qquad (2.14)$$

The discharge coefficient for varying head is computed from the following equation.

$$C = \left[\left(\frac{H_e}{H_0}\right) - \left(\frac{H_e}{H_0}\right)^{0.333} + 0.967\right]^{0.125} \left(\frac{H_e}{H_0}\right)^{0.0625} \qquad (2.15)$$

The authors use Gene-Expression Programming in their study to demonstrate the above equations. They also included effects of submergence and upstream crest inclination using separate equations which are not written here.

The aforementioned equations are all used when flow is in free conditions and no control gates exist on the crest. However, in case of controlled flow, the parameters of the head-discharge relationship and the discharge coefficient change. The discharge becomes a function of gate opening too. Moreover, the piers and abutments effects are included in the effective length parameter.

$$L_{eff} = L_{net} - 2(NK_p - K_a) * H_0 \tag{2.16}$$

Where L_{eff} is effective length, L_{net} is the total length, K_p is pier contraction coefficient, K_a is abutment contraction coefficient and Ho is the total head. It is important to mention that the total head in the above equations includes approach velocity as well. However, in our case the velocity changes are assumed zero, since the water surface of the dam is very wide. K_p and K_a is calculated using charts provided by Design of Small Dams (1987).

The USBR (1987) provides design charts for calculating discharge coefficients when the flow is controlled by gates. The following equation is used to calculated discharge under the effects of gates.

$$Q = CLD\sqrt{2gH} \tag{2.17}$$

Where C is discharge coefficient, H is head to the center of the gate opening, and D is shortest distance from the gate lip to the crest curve. For the discharge coefficient is calculated form the following chart.

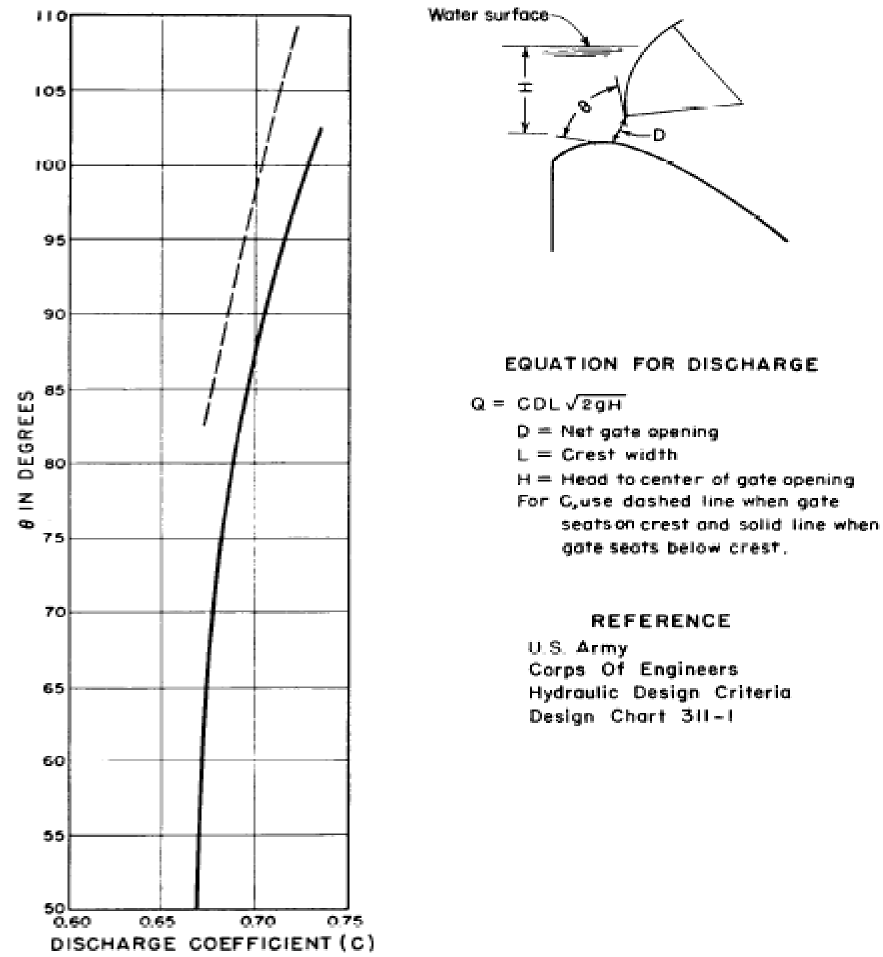

Figure 2.9: Discharge Coefficient (Design of Small Dams, 1987)

However, the Design of Small Dams (1961) used a different head-discharge relationship and discharge coefficient chart.

$$Q = \frac{2}{3}\sqrt{2g}CL(H_1^{1.5} - H_2^{1.5}) \qquad (2.18)$$

Where H1 and H2 are the measurements shown in the following chart for the calculation of the discharge coefficient.

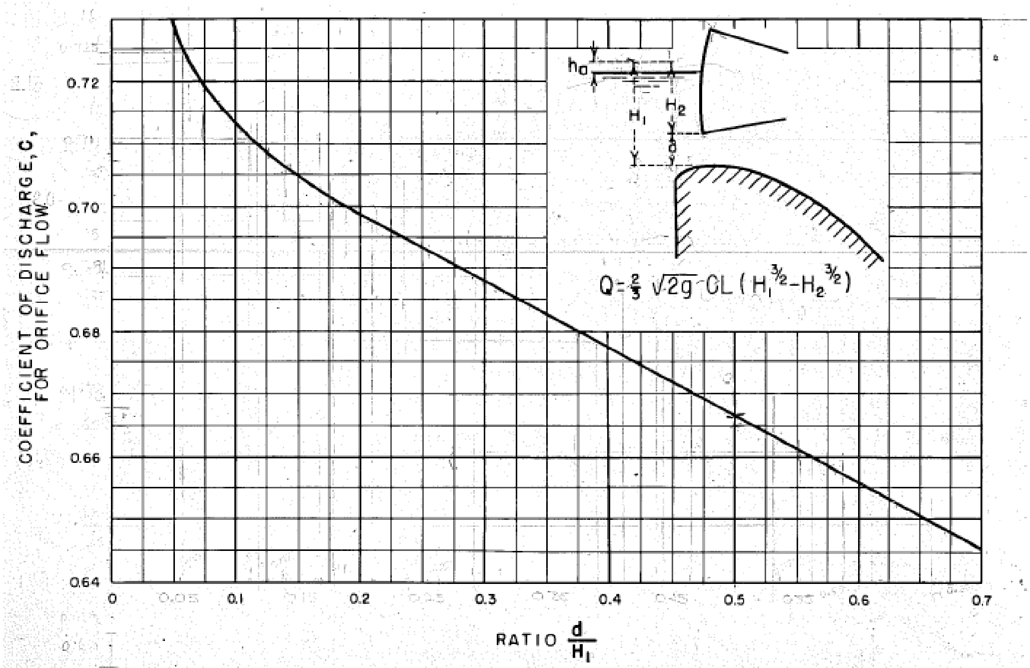

Figure 2.10: Discharge Coefficient (Design of Small Dams, 1961)

Yanmaz (2018) also used Eq. 2.18 for the discharge computation but for the discharge coefficient the following formula is proposed.

$$C = -13.168\left(\frac{e}{H_1}\right)^6 + 29.721\left(\frac{e}{H_1}\right)^5 - 25.295\left(\frac{e}{H_1}\right)^4 + 9.8034\left(\frac{e}{H_1}\right)^3 - 1.538\left(\frac{e}{H_1}\right)^2 - 0.0995\left(\frac{e}{H_1}\right) + 0.7341 \quad (2.19)$$

Where e is gate opening and H1 is shown in Figure 2.10 Bagatur and Onen (2015) conducted GEP and regression model study for determining head-discharge relationship for ogee crested spillway. They used Equation (2.18) for the computation of the discharge values and suggested the following formula for calculating the discharge coefficient of a gated ogee crested spillway.

$$C_d = \left(\left(2*\frac{d}{H}\right)^{0.333} + \left(\frac{d}{H}\right)^2 + 2.11\right)^{-0.333} \quad (2.20)$$

Where d is gate opening and H is water head over the gates. Equation (2.20) underestimated gated flow value when the gates are about to being fully opened.

Tanyeri (2021) conducted a numerical study on the discharge coefficient for flow under radial gates which was validated with a physical model at METU. The author used the following head-discharge equation.

$$Q = C_g C_t d L \sqrt{2gH_C} \tag{2.21}$$

Where C_g is the gate discharge coefficient, C_t is a correction factor for C_g, d is gate opening, L is width of the crest and H_c is head over the radial gate. The discharge coefficient formula for the flow under radial gates proposed by the author is shown below.

$$Cg = 0.0673\left(\frac{d}{H_C}\right)^{1.5} - 0.3142\left(\frac{d}{H_C}\right) - 0.0514\left(\frac{d}{H_C}\right)^{0.5} + 0.7512 \tag{2.22}$$

And the equation for the gate correction factor is written in terms of head and a design parameter X_s.

$$Ct = 0.0202\left(\frac{Xs}{H_C}\right)^{3} + 0.0017\left(\frac{Xs}{H_C}\right)^{2} + 0.1007\left(\frac{Xs}{H_C}\right) + 1.000 \tag{2.23}$$

Where X_s is a parameter related to the position of the radial gate compared to the crest and H_c is head over the gate. X_s is taken as zero, so the value of the correction factor (C_t) is essentially equal to 1. Equation (2.21) and Equation (2.22) predicts the discharge values more precisely compared to other formulas, hence in this study, Equation (2.21) and Equation (2.22) are used for the calculation of flow under radial gate.

2.3 Methods for Gate Operations

Methods for gate operations are selected according to the primary goals of the reservoir routing process. Mainly, gate operations are done so that the excess amount of water produced by a flood event is safely transferred to downstream. Meanwhile, gate operations are also performed to store more water in the reservoir. This could mainly be the case in dry climate areas where the amount of precipitation is low. Generally, reservoir routing and gate operations are performed based on optimization methods having multi-objective functions. Based on the multi-goal optimization methods, not only excess water is safely transferred downstream but also the water is stored at optimal levels. Gate operations are done considering a multi-stage conception. Normally, gate openings are defined differently for the rising and falling limbs of the inflow or flood hydrograph. Although the gate operation is a complicated process, the multi-stage concept is easier to apply to the gated spillways. Salehi et al. (2022) used a 10-staged optimization method and genetic two algorithm methods for gate operations. The GA methods were reported to be more effective. Although more accurate results may be obtained using more stages for the operations, more gate operations are not desirable as it increases complexity of the gate operations. Reported by Sordo-ward et al. (2016), Frenando Giron in 1988 suggested Volumetric Evaluation Method for gate operations. This method is applied to dam operations in Spain. The method decides on storage levels between Flood Control Level and Top of Control Pool. For each water level between the two limits, five different equations are applied to complete the reservoir routing process. Sordo-ward et al. (2017) improved the method by introducing a factor call K to the Volumetric Evaluation Method. The new method is called as the K-Method. The new method is said to be more effective in reducing maximum water levels hence decreasing the damage posed by the flood to the dam structure. This method divides reservoir levels into four different zones. To perform the reservoir routing process, 11 equations are used for the application of the K-Method.

Multiple website pages discuss the electronic process of how gate operations can be done automatically. Dasun (2017) investigated the electronic procedure behind automating reservoir gate operations. However, the author uses only water level sensors to perform the gate operations which is not based on the reservoir routing process.

The two methods used by Salehi et al. (2022) require an initial opening of almost 2 m for the gates. Although in normal operating conditions the gates cannot be opened by 2 m suddenly, the authors do not provide reasons to justify the gate openings. The above methods also don't use a specific criterion to store water at different desirable levels.

In this study, the reservoir gate operations are performed to have smoother gate openings. Furthermore, the gate operations performed in this study include automating the gate operations as well as storing water at pre-defined levels at the end of the flood event.

CHAPTER 3

NUMERICAL INVESTIGATIONS

3.1 Topographic Data

The numerical studies will be conducted on the Lower Kaleköy dam located in Bingöl province of Türkiye. The dam is built on Murat river. The spillway was originally designed as a PKW system but later it has changed to an Ogee Crested Weir with gates to control water level in the reservoir. This gives an advantage to perform reservoir routing computations for the same project using two alternate weir designs so that comparison is more meaningful.

Topographic data which include information about the reservoir bed elevation, reservoir water surface area and reservoir water volume of the dam were taken from Lower Kaleköy dam project conducted by Temelsu (2018) as shown below.

Approach Channel Elevation: 1080 m

Maximum flood level: 1103 m

Maximum operating level: 1102.5 m

Number of gated spillways: 4

Width of a gate: 14 m

Number of piers: 3

Width of a pier: 6 m

Total crest width of spillway: 74 m

The topographic data for elevation, surface area and volume is shown in the table below.

Table 3.1: Topographic Data of Lower Kaleköy Dam

Elevation (m)	Depth(m)	Area (km^2)	Volume (hm^3)
1012.000	0.000	0.000	0.000
1014.000	2.000	0.061	0.061
1018.000	6.000	0.496	1.170
1030.000	18.000	1.588	13.628
1040.000	28.000	2.725	35.248
1050.000	38.000	4.027	69.008
1060.000	48.000	5.535	116.817
1070.000	58.000	7.504	182.013
1080.000	68.000	9.793	268.499
1090.000	78.000	12.330	379.113
1100.000	88.000	15.144	516.482
1105.000	93.000	16.685	596.056

To perform a reservoir routing, water surface area and water volume are expressed as functions of water depth. To find a suitable power function for this data, a best line is fitted through this data. The graphs of the original data and the fitted data are shown in Figure 3.1 and Figure 3.2 given below.

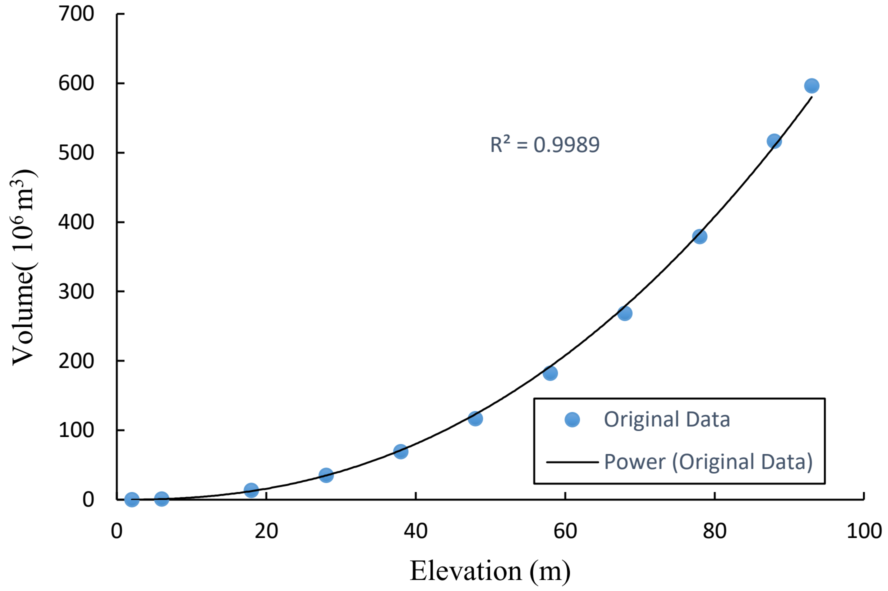

Figure 3.1: Storage Function S(h) Obtained by Fitting a Power Function Through Table 3.1

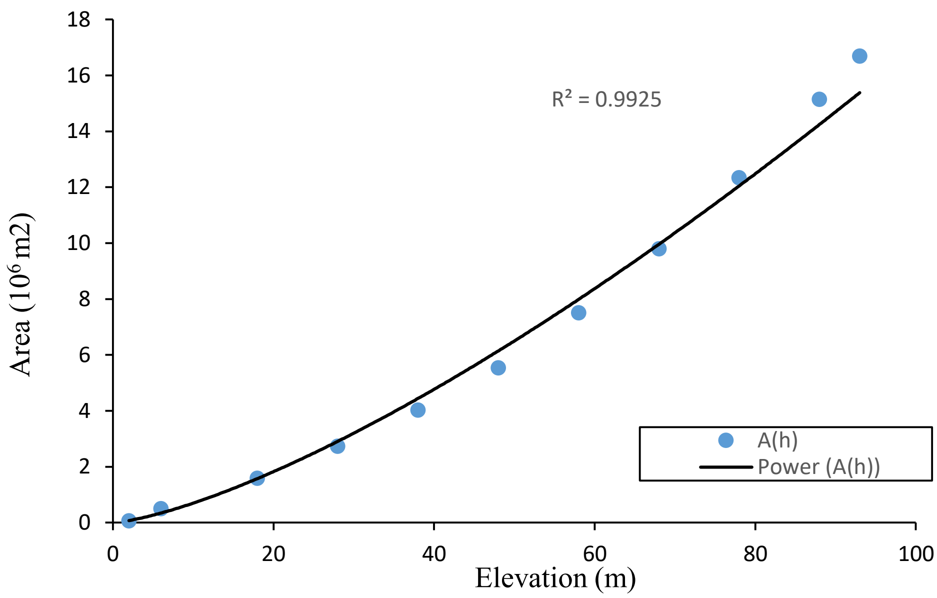

Figure 3.2: Area as a Function of Depth

After fitting a power function through the original topographic data, the following equations for storage and the surface area are obtained.

$$S(h) = 14244 * h^{2.3418} \tag{3.1}$$

$$A(h) = 28477 * h^{1.388} \tag{3.2}$$

3.2 The Inflow Data

Initially real flood data obtained from Aşağı Kaleköy Project by Temelsu (2018) was considered for the flow routing process. The original data was recorded with 24 hour intervals which was not suitable in resolution for a numerical analysis. To overcome this problem, a synthetic data was produced using a mathematic formula used by (Yanmaz, 2018):

$$I = Ip * \left(\frac{t}{t_p}\right)^{3.5} * EXP\left(-3.5 * \left(\frac{t}{t_p} - 1\right)\right) + 100 \tag{3.3}$$

where I is inflow, t is time, t_p is peak time, I_p peak value and 100 (m^3/s) is the value of base flow of the hydrograph. The peak value together with the base flow is 4300 m^3/s. Time interval for the digitized data is $\Delta t = 5$ min. The inflow hydrograph obtained using Eq. 3.3 is plotted in Figure 3.3.

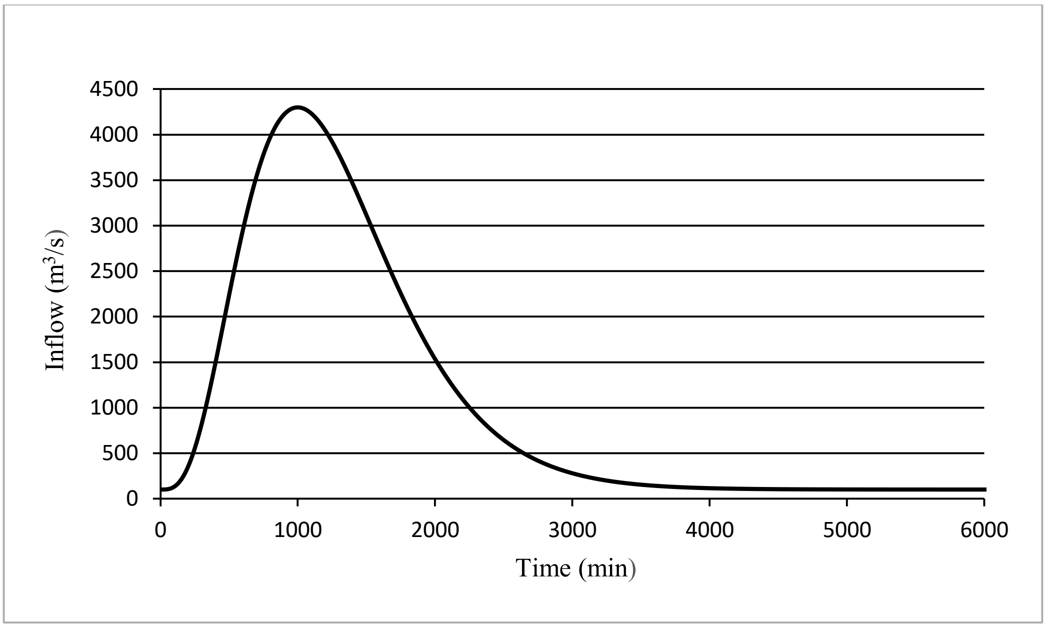

Figure 3.3: Inflow Hydrograph

3.3 Reservoir Routing Methods

As mentioned in the introduction part of flow routing section, there are different types of reservoir routing methods such as Euler's methods, Runge Kutta's method and Level Pool methods. The level pool method may be easily applied to short data sets. It uses interpolation and graphical methods for solving routing problems. In the graphic method, the data points are calculated manually one by one which is not time efficient when the data set is long. Similarly, interpolating between two data points opens door to errors. For this reason, differential methods that are easier to apply to longer data sets are preferred. These methods also minimize error while calculating certain parameters in the flow routing process. The Runge Kutta methods have different orders, the higher the order, the more accurate the results. Higher order Runge Kutta methods are preferred for dynamic problems where a solution stability or sensitivity criterion should be considered. In our case, the events take place in steady state conditions. The calculation steps and the formulas used in Level Pool, Runge Kutta and Euler's methods are described shortly here.

3.3.1 Level Pool Method

The Level Pool method can be used for reservoirs with horizontal water surfaces. The time intervals are divided into small increments. The following relationship is written based on the continuity equation.

$$\left(\frac{2S_{j+1}}{\Delta t} + Q_{j+1}\right) = \left(I_{j+1} + I_j\right) + \left(\frac{2S_j}{\Delta t} - Q_j\right) \quad (3.4)$$

In Equation (3.4) the left hand-side of the equation contains unknowns, while the known variables are on the right hand-side of the equation. Storage and outflow values are known at the j^{th} step with inflow values already obtained from the inflow data. In order to calculate the two unknowns on the left hand-side, a second equation needs to be written. This equation can be established by relating the storage of the reservoir with the outflow or the discharge variable. For this reason, values of $\frac{\Delta s}{\Delta t} + Q$ vs Q (Q on the vertical axis) are plotted. Using the $\frac{\Delta S}{\Delta t} + Q$ vs Q relationship, the unknown on the left hand-side of Equation (3.4) can be calculated either graphically or using interpolation.

3.3.2 Runge Kutta's Method

The Runge-Kutta method is another alternative reservoir routing method but this method is different from the level pool method. Runge-Kutta method does not require an extra relationship between storage and the outflow to solve the reservoir routing problem. The method can be written for different orders, here the third order method is used. It is important to mention that higher orders of the Runge Kutta method have higher accuracy. For a Runge-Kutta method the following storage outflow equation is written.

H stands for hydraulic head of the weir in Equation (1.1) while the small h in previous sections stands for depth of the reservoir. The storage can be expressed in terms of water surface area and the water head.

$$dS = A(H)dH \tag{3.5}$$

Substituting dS in Equation (1.1) we have:

$$\frac{dH}{dt} = \frac{I(t) - Q(H)}{A(H)} \tag{3.6}$$

a- The inflow, outflow and change in head values are calculated in three increments. The following procedure is proposed to route inflow through a reservoir using Runge Kutta 3rd order method. The inflow values are computed by interpolating, for each time increment of $I(t_j)$, $I(t_j + \Delta t/3)$ and $I(t_j + 2\Delta t/3)$. With the initial conditions of the reservoir already known, one can write the following equation to find ΔH1:

$$\Delta H1 = \left(\frac{I(t_j) - Q(H_j)}{A(H_j)}\right)\Delta t \tag{3.7}$$

b- Now the value of $Q(H_j+\Delta H1/3)$ and the value of ΔH2 can be found.

$$\Delta H2 = \left(\frac{I\left(t_j + \frac{\Delta t}{3}\right) - Q\left(H_j + \frac{\Delta H1}{3}\right)}{A\left(H_j + \frac{\Delta H1}{3}\right)}\right)\Delta t \tag{3.8}$$

c- Similarly, the value of $Q\,(H_j + 2\Delta H/3)$ is computed and ΔH3 is calculated as in the following form:

$$\Delta H3 = \left(\frac{I\left(t_j + \frac{2\Delta t}{3}\right) - Q\left(H_j + \frac{2\Delta H2}{3}\right)}{A\left(H_j + \frac{2\Delta H2}{3}\right)}\right)\Delta t \tag{3.9}$$

d- The change ΔH is then calculated as:

$$\Delta H = \frac{\Delta H1}{4} + \frac{3 * \Delta H3}{4} \tag{3.10}$$

e- The final value of H is found as:

$$H_{j+1} = H_j + \Delta H \tag{3.11}$$

The procedure is repeated until the desired values of H are achieved.

3.3.3 Euler's Method

Euler's method is an alternative method used for routing flow through reservoir and channels. The Euler's method is comparatively simpler than the Runge Kutta's method. In this method, Eq. (3.5), (3.6) and (3.7) are used but the time intervals are not divided into smaller parts. In other words, the change in head is calculated only once for each time increment. The following procedure is undertaken when using Euler's method for reservoir routing.

a- An initial value of head is introduced at $t = t_0$

b- The change in head over time is calculated using Eq. (3.6)

c- Change in head is found using Eq. (3.7)

d- The new value of H is found as $H_{n+1}=H_n+\Delta H$

The procedure is repeated until the desired level of surface water is reached.

3.4 Design of PKW and Ogee Crested Spillway

3.4.1 Design of PKW

To design a PKW spillway, various design guidelines suggested by Omuane and Lempérière (2013) in Hydrocoop can be used. It is important to mention that a PKW can be designed with different dimensions considering local conditions and cost effectiveness. Here, the PKW is designed with symmetric overhangs (Type-A). Other geometric properties are selected based on latest literature studies to have a more discharge efficient weir. The design head is later found from reservoir routing process and the weir height is selected so that the discharge coefficient Equation (2.3) which is valid only between $0.1 < H/P < 0.9$ interval, is not violated.

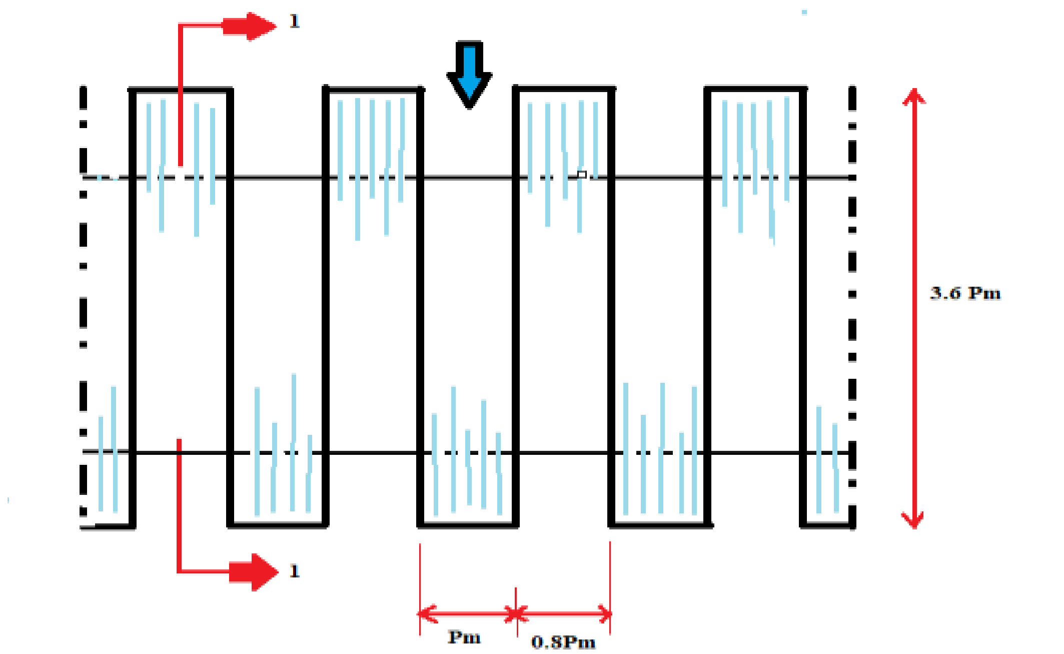

Figure 3.4: Plan View of the PKW

Here P_m stands for the maximum weir height of a labyrinth weir, the ratio $W_i/W_o = 1.25$ as $W_i = P_m$ and $W_o = 0.8P_m$ and the length in upstream-downstream direction is taken as $3.6P_m$.

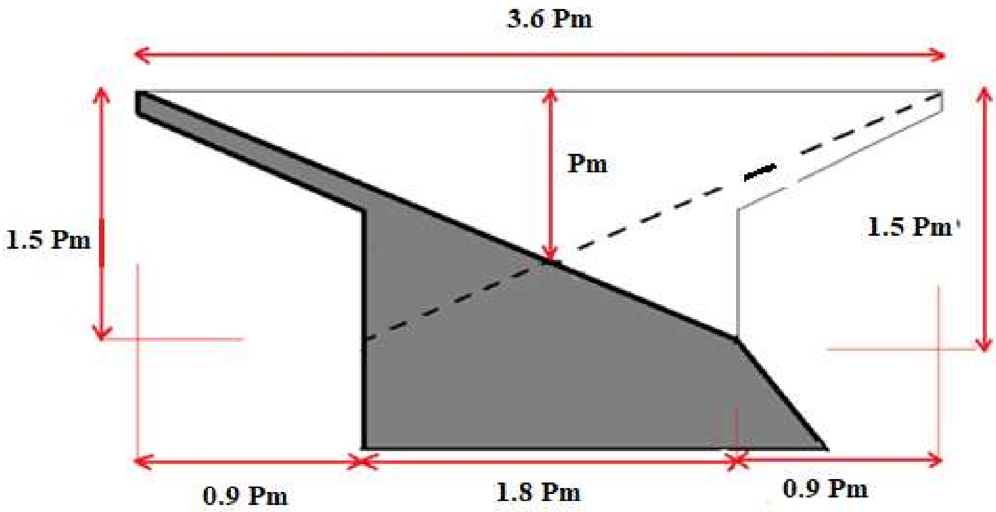

Figure 3.5: Side View of Cross Section 1-1

In Figure 3.5, the length of the overhangs in the upstream and downstream are both equal to each other and taken as 0.9Pm. Hydrocoop suggests the following guidelines for designing a PKW.

a- Pm = 0.5 P

b- P_i = 1.5 Pm

c- Wi/Wo = 1.25 The optimal range of inlet and outlet widths ratios according to Anderson (2011) is [1.25-1.5].

d- 0.1 < H/P < 0.9, according to (B. M. Crookston et al., 2018)

e- Wo = 0.8 Pm, Wi = Pm, Bo = Bi = 3.6 Pm, Overhang lengths = 0.9 Pm for both upstream and downstream (Hydrocoop, France)

f- Thickness of PKW: Ts/P = 0.067 for Goulours Dam and 10 < P/Ts < 22 according to (Ribeiro et al., 2013)

g- Width of one cycle: w = Wi + Wo – 2Ts h- Number of cycles N= total length/ w (one cycle)

i- L = N(Wo+Wi+2Ts+2B) according to Guo et al. (2019) or L = N (Wi + Wo +2B)

In our case the PKW is designed using the following guidelines.

a- P = 5m (taken from Hydrocoop, France), so that H/P limitation in Equation (2.3) is satisfied

b- Design Head (H_d) = 3.2 m, after reservoir operations, 0.1< (3.2/5) =0.64 < 0.9

c- Wi/Wo = 1.25 The optimal range of inlet and outlet widths ratios according to (Anderson, 2011) is [1.25-1.5]. Hence, Wo = 4.5m, Wi = 5.625m,

d- B = 13.25m, Overhang lengths Bo = Bi = 3.313m, overhangs are symmetrical to have a Type-A PKW to satisfy the conditions of Eq. (2.3)

e- Wall Thickness of PKW, Ts = 0.25, Ts/P = 0.067 for Goulours Dam (Lougier, 2009) and 10 < P/Ts < 22 according to (Ribeiro et al., 2013)

f- Width of one cycle: w = Wi + Wo – 2Ts = 4.5+5.625-2*0.25 = 9.625m

g- Number of cycles N= Total Width/ Width (one cycle) = W/w = 240/9.625 ≅ 25.

h- Total length of the PKW, L = N(w+2B) ~ 900m

3.4.2 Design of Ogee Crested Spillway

To design the ogee gated spillway the design discharge of the PKW is considered. To compare the hydraulic performance, the ogee crested spillway should also have the same design discharge capacity so that the two spillway systems are equivalent. The geometric parameters of the spillway are taken from the Lower Kalekoy dam report conducted by (Temelsu, 2018). The following procedure is followed for designing the spillway.

Max operating level: 1102.5 m

Design head =H_{max} = 1102.5 – 1084.40 = 10.437m

We choose 4 gates with three piers. Each gate with is 14m and pier width 6m.

P = 1092.063 – 1080 = 12.063m

To have no cavity risks we can select a Ho value for design head.

For no cavity Hmax/Ho < 1.33, 18.6/Ho <1.33 and Ho is selected as 10.437m.

P/H_0 = 1.15, from Char 3 and after unit conversion, C_0 = 2.153

H_e = 1102.5-1084.4 = 10.4370 m so H_e/H_o = 1.0, C1 / C0 = 1.0 (Chart 4)

P / H0 = 1.15 from this we find, Cinc/ Cver= 1.015 (Chart 5)

Upstream face slope is selected as 1/1.

The discharge coefficient, C = C0 *(C1/ C0) *(Cinc/Cver)

C = 2.153*1.0*1.015 = 2.19

Net width of the spillway, Lnet = 4 * 14.00 = 56.00m

Effective width, L_{eff} = Lnet – 2(N*Kp + Ka) He, Ka = 0.09 from chart-7 but Kp has negative values so it is taken as zero, Ka = 0

Leff = 56 – 2(3*0+0.09) *10.437= 54.122m

Now one can calculate the discharge as,

$$Q = C * L_{eff} * {H_0}^{1.5} = 2.19 * 54.122 * 10.437^{1.5} \approx 4000 \frac{m^3}{s}$$

Equation (2.12) is used to calculate the discharge coefficient of an ogee crested spillway in free flow conditions. Similarly, Equation (2.13) is used for the calculation of discharge values of ogee crested flow. According to Equation (2.13) the discharge coefficient of the design head is calculated as 0.495 and the design discharge is also calculated as 4000 m^3/s.

The above calculations are performed when the gates are fully open. For partial opening of the gates Equation (2.21) and Equation (2.22) are used as discussed previously.

For the geometric design of the upstream quadrant of the spillway the following equation can be used.

$$\frac{X^2}{A^2} + \frac{(B - y)^2}{B^2} = 1 \qquad (3.12)$$

Where X is the horizontal and y is the vertical axis. A and B are one half of the ellipse axes.

For the design of the downstream quadrant the equation is expressed as,

$$\frac{y}{Hd} = \frac{1}{K} * \left(\frac{x}{Hd}\right)^n \qquad (3.13)$$

Where Hd is the design head over the spillway, K and n are constants that can be computed from the design charts. Here, we do not go further into the details of the design of the upstream and downstream quadrants as it is not the focus of this study.

3.5 Reservoir Routing Process

The reservoir routing process as explained in section 3.3 is performed using Euler's and 3rd order Runge Kutta's methods. It is assumed that the reservoir operates in normal conditions when the flood starts entering the reservoir. A 2 m free board exists between the crest level and normal operation level. When the flood enters the reservoir, the water rises until the empty storage provided by free board is filled and then water overflows from the crest of PKW. The surface water rises until the peak values of inflow and then gradually decreases back to the crest level.

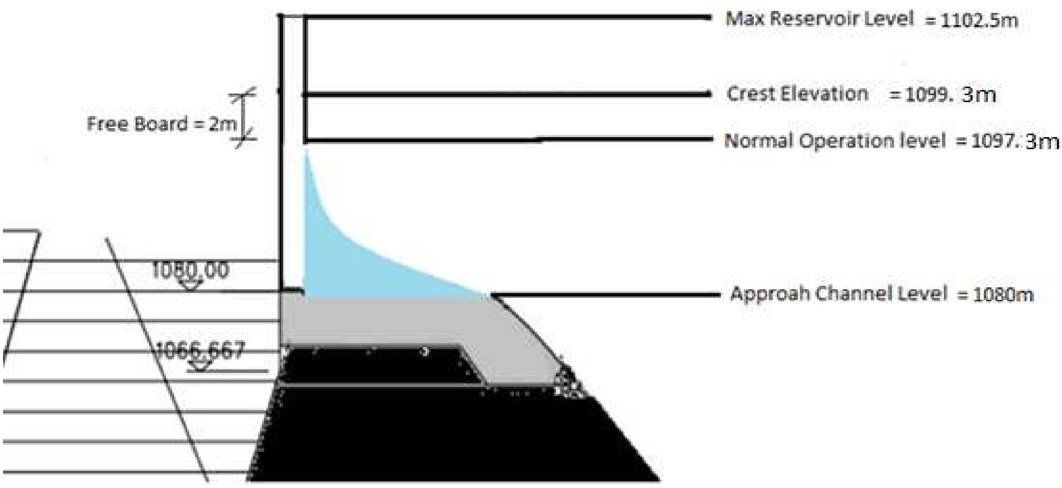

Figure 3.6: Water Levels for the PKW

For an ogee crested spillway the gates are kept closed until water reach maximum operating level (1102.5m). After reaching maximum operating level, the gates can be fully or partially opened. The gates can also be opened one by one or all by once. The partial opening of gates is more suitable to have controlled outflow and safe downstream conditions. This will be discussed in details in the next section. After practicing with Euler's and 3rd order Runge-Kutta methods, negligible differences in the results of the two methods were found. For the peak inflow point of 4300 m^3/s, the water head is computed as 2.9259 m and 2.9286 m for Runge-Kutta and Euler's method, respectively. Being easier and simpler to apply to the reservoir routing equations, for the remaining of the routing processes only Euler's method was used. The following procedure was followed for applying Euler's method for reservoir routing of the systems.

a- The initial value of head over PKW was taken as zero
b- The time interval Δt is taken as 5 min
c- The discharge coefficient is calculated using Eq. (2.3)
d- No outflow is allowed until the surface water level rises above the crest level for the PKW, while for the Ogee Crested spillway outflow is allowed to pass to the downstream only after surface water rising above normal operating level.
e- The change in head over time is calculated using Eq. (3.6).
f- The final value of the head is calculated using $H_{n+1}=H_n+\Delta H$
g- The surface water levels are also calculated accordingly

Table 3.2 shows a part of the calculation process.

Table 3.2: Euler's Method for Flow Routing, PKW

t (min)	I(t) m^3/s	H_n (m)	Q(H) (m^3/s)	A(h) m^2	ΔH/Δt (m/s)	H_{n+1} (m)	Elevation (m)
0	100.000	0.000	0.000	13640505.046	7.33111E-06	0.002	1097.298
5	100.001	0.002	0.000	13640993.254	7.33093E-06	0.004	1097.300
10	100.013	0.004	0.000	13641481.456	7.33157E-06	0.007	1097.302
15	100.055	0.007	0.000	13641969.705	7.33432E-06	0.009	1097.305
20	100.147	0.009	0.000	13642458.142	7.34081E-06	0.011	1097.307
25	100.315	0.011	0.000	13642947.016	7.35287E-06	0.013	1097.309
30	100.586	0.013	0.000	13643436.699	7.37245E-06	0.015	1097.311
35	100.987	0.015	0.000	13643927.690	7.40161E-06	0.018	1097.313
40	101.548	0.018	0.000	13644420.628	7.44243E-06	0.020	1097.316
45	102.297	0.020	0.000	13644916.290	7.49706E-06	0.022	1097.318
50	103.263	0.022	0.000	13645415.596	7.56763E-06	0.024	1097.320
55	104.477	0.024	0.000	13645919.606	7.65625E-06	0.027	1097.322
60	105.965	0.027	0.000	13646429.524	7.76503E-06	0.029	1097.325
65	107.757	0.029	0.000	13646946.692	7.89603E-06	0.031	1097.327
70	109.879	0.031	0.000	13647472.590	8.05125E-06	0.034	1097.329

Table 3.3: Initiation of Flow to the Downstream, PKW

t (min)	I(t) m^3/s	H_n (m)	Q(H) (m^3/s)	A(h) m^2	ΔH/Δt (m/s)	H_{n+1} (m)	Elevation (m)
535	2494.979	2.017	0.860	14090310.870	0.00017701	2.070	1099.315
540	2531.317	2.070	11.800	14102207.422	0.000178661	2.124	1099.368
545	2567.465	2.124	37.021	14114217.821	0.000179283	2.178	1099.422
550	2603.413	2.178	76.711	14126272.919	0.000178865	2.231	1099.476
555	2639.150	2.231	128.987	14138302.784	0.000177543	2.285	1099.529
560	2674.666	2.285	190.988	14150246.567	0.000175522	2.337	1099.583
565	2709.950	2.337	259.732	14162057.126	0.000173013	2.389	1099.635
570	2744.993	2.389	332.614	14173701.550	0.000170201	2.440	1099.687
575	2779.784	2.440	407.586	14185159.340	0.000167231	2.490	1099.738
580	2814.314	2.490	483.155	14196419.710	0.000164208	2.540	1099.788
585	2848.575	2.540	558.297	14207478.931	0.000161202	2.588	1099.838
590	2882.556	2.588	632.346	14218338.089	0.000158261	2.635	1099.886
595	2916.249	2.635	704.895	14229001.377	0.000155412	2.682	1099.933

In the case of the Ogee Crested spillway the details are a little different. For the Ogee Crested spillway the initial head value is taken as 5.237m. For this case, the outflow is allowed to pass to the downstream after the surface water level rises to the maximum operation level (1102.5m).

Table 3.4: Euler's Method and Reservoir Routing for Ogee Crested Spillway

t(min)	I(t) m^3/s	H_n (m)	Q(H) (m^3/s)	A(h) m2	ΔH/Δt (m/s)	H_{n+1} (m)	Elevation (m)
0	100	5.235	0	13890634.3	7.1991E-06	5.237	1097.298
5	100.001	5.237	0	13890796.62	7.1991E-06	5.239	1097.300
10	100.013	5.239	0	13890958.96	7.19989E-06	5.241	1097.302
15	100.054	5.241	0	13891121.34	7.20277E-06	5.244	1097.304
20	100.146	5.244	0	13891283.82	7.20932E-06	5.246	1097.307

After the computation process is completed, the following graph for inflow and outflow of PKW and Ogee Crested spillways is obtained.

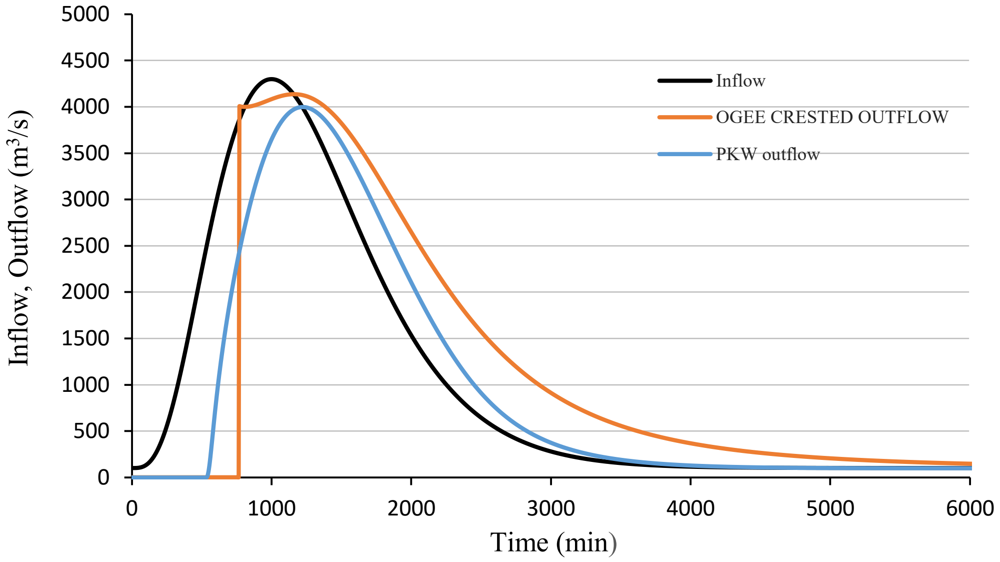

Figure 3.7: Inflow and Outflow Hydrographs of PKW and Ogee Crested Spillway

As seen in the graphs above, the PKW is designed to pass a design discharge of 4000 m^3/s. The Ogee Crested spillway is also designed to pass a discharge of 4000 m^3/s. The water level rises a little and then falls down till the crest level is reached. In order to compare both of the spillway systems, both systems need to be designed for the same discharge values. The starting time is different for both of the weirs just for this initial reservoir routing stage. The gates of the OCW are kept closed until the water level reaches 1102.5 m and then all the gates are opened at once. This is to see the behavior of an OCW and PKW in free flow conditions. For the next reservoir routing processes, the starting time of PKW and gated OCW are the same.

The water level rise and fall is also important in reservoir routing process. Figure 3.8 shows the water levels for PKW and ogee crested spillway. As seen in Figure 3.8, the water level rise starts from 1097.3m for both of the spillways. For the PKW, the overflow happens when the water level passes 1099.3m, but for ogee crested spillway the water rise continues until the maximum operation level is reached and then the gates are fully opened. The PKW water rise continues until 1102.5m and the water surface levels fall back to the crest level at the end of the flood event. On the other hand, for the ogee crested spillway the water level falls beyond PKW crest level reaching to the value of ogee crest level at the end of the flood event. More details of the water level rise and fall are given in Figure 3.8.

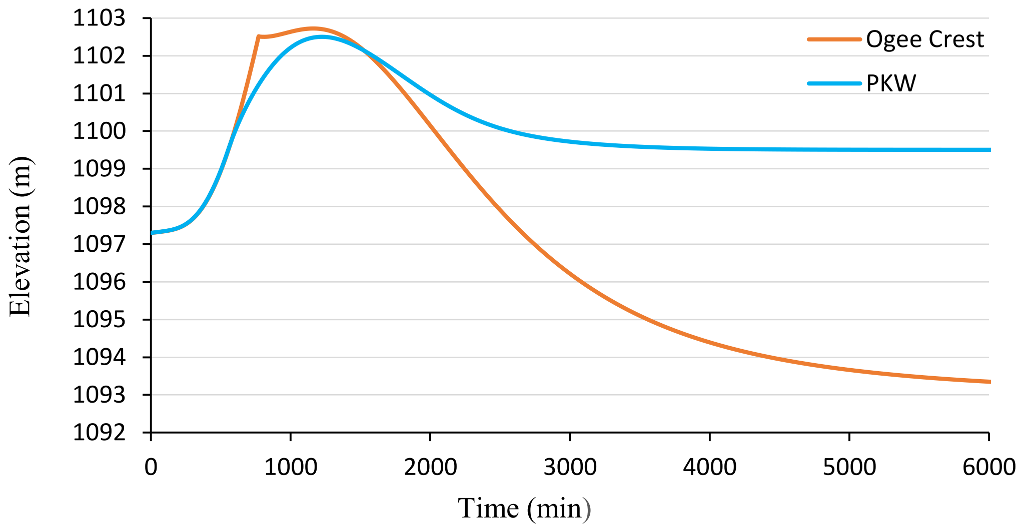

Figure 3.8: Water Level Change for PKW and Ogee Crested Spillway

It is important to mention that the full opening of all the gates at once was only considered for the initial steps of the reservoir routing process. This was to understand the discharge behavior of an ogee crested spillway and the PKW. At the end of the reservoir routing process, attanuation and the time lag for the PKW are comupted. The PKW is designed for 4000 m^3/s discharge, and the attenuation is calculated as 300 m^3/s. The peak inflow point occurs when t = 1000 min, while the peak outflow happens at t = 1225 min. The time lag is computed as 225 min. For the gated OCW the lag time between the peak points of inflow and outflow and the attenuation are the same as the PKW. The reason is that the gate operations are performed so that both of the weirs have similar discharge hydrographs and water level profiles.

3.6 Gate Operations

The ogee spillway is equipped with radial gates. A radial gate has a circular shape, lifted on the sidewalls by its arms. The radial gates are easier to open because of the hydrostatic uplift force acting on the circular gate, hence requiring less external force for lifting the gate. Radial gates are lifted by a hoist mechanism that rotates the gates around their pivot axis using a chain as seen in Figure 3.9.

Figure 3.9: Radial Gates and Its Lifting Mechanism (www.vortexhydradams.com)

The opening and closing speed is an important issue here. For larger inflow values, the gates need to be opened earlier to compensate the inflow in time before reaching the maximum operation levels. Meanwhile, the gates should not be opened when the inflow entering the reservoir is not able to rise above the maximum surface water levels. In other words, water should be kept at certain levels so that the storage is preserved at optimal volumes and the safety of the reservoir is also not endangered.

The rise speed of the water level and the longevity of the flood event are also essential factors affecting the reservoir gate operations. When the rising speed of the water level is high, the gates should be opened at an early stage. Similarly, when the flood event is longer the gates should also be kept open for a longer time.

To open a multiple gates system various opening scenarios may be considered. The gates may be fully opened one by one or all at once, which is not desirable. The other alternative is to open the gates partially one by one or all at once. The discharge and discharge coefficient equations should be used accordingly for each of the above opening alternatives.

In this study, the gates are opened in a manner to have outflow graphs similar to the ones obtained from the PKW. For the first case, the gates are fully opened one by one. It is also important to mention while obtaining similar outflow hydrographs to the PKW, the water levels of the gated spillways should also be maintained at the same level as the PKW.

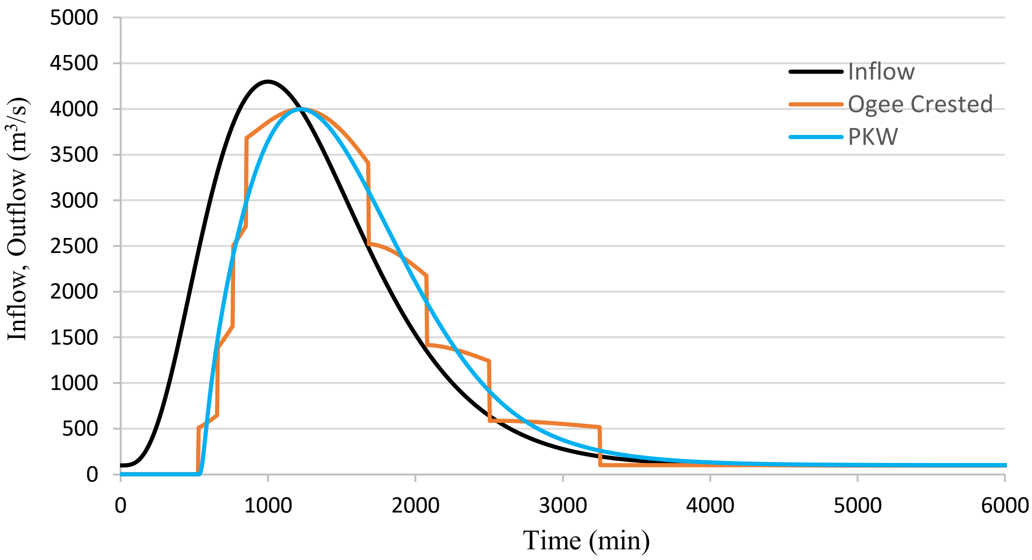

Figure 3.10: Outflow Curves of Gated Ogee Spillway and PKW

The sudden rises and reductions in the gated ogee graphs show the opening and closing steps of each gate. The corresponding water surface profiles are shown in Figure 3.11. The graph related to the gated discharge of the ogee crested spillway shows small changes as the gates are opened and closed. With partial gate openings, the discharge behavior of the PKW is predicted closely.

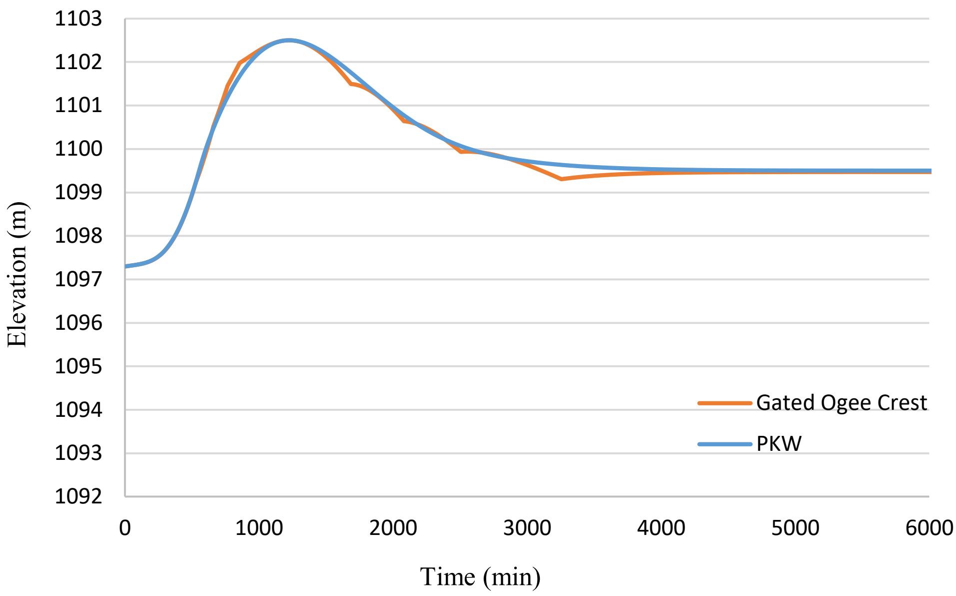

Figure 3.11: Surface Water Levels for Fully Opened Gates

The opening and closing rate of a gate is important when gates are opened partially. According to Kolte (2015) in normal operating conditions, the hoist can be rotated at a speed between 300mm/min-700mm/min. The speed of the hoist rotation can be further increased if larger discharge capacity is needed. Considering this information, the gate opening should not be very big to avoid unrealistic and undesirable results. The partial gate opening can be done in different ways. The first way is to open one gate partially and increase the opening incrementally until the gate is fully open. After this the second gates are opened, the third and finally all the gates are opened. In the second alternative, two gates are opened partially at once. After full openings of the two gates, other gates are partially opened one by one. Similarly, as a last alternative all the gates can be opened partially at once. Here, it is required to find which of the gate opening alternatives predict the discharge behavior of PKW more accurately.

To do this, all of the above alternatives are plotted for the purpose of comparison. The water surface profiles of the different partial gate opening methods is plotted in Figure 3.12.

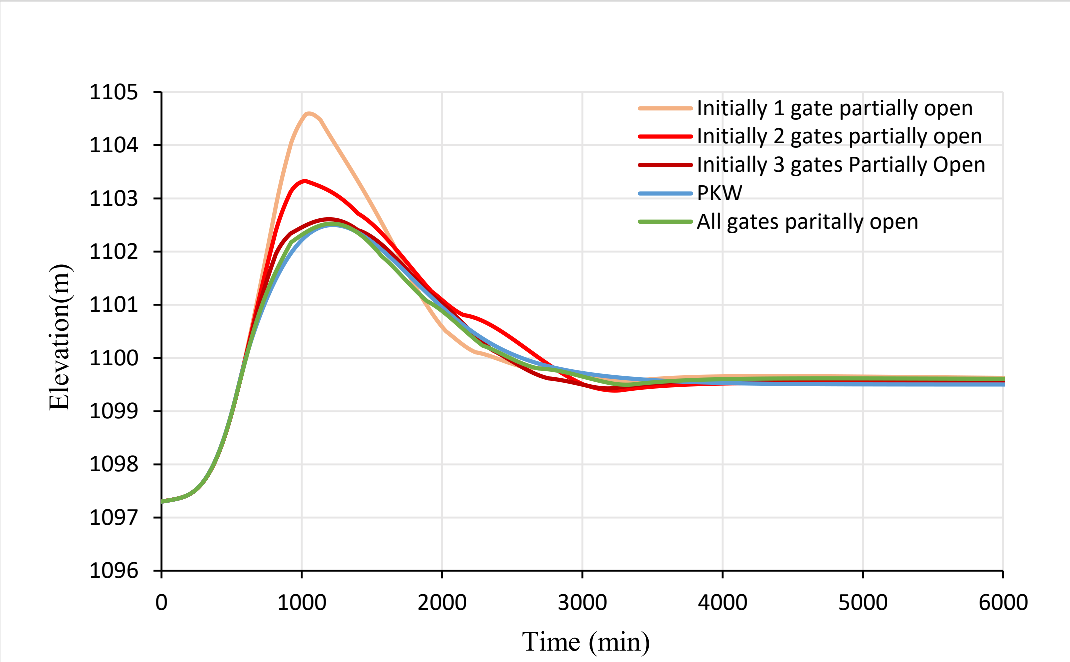

Figure 3.12: Water Levels by Opening Gates Partially

As seen in Figure 3.12, when the gates are opened partially one by one, the water surface level reaches 1104.5 m. This indicates that the first alternative of opening gates one by one partially is not suitable to prevent water from crossing the maximum operation level. Similarly, when two gates are opened partially in the initial stage, the water surface level crosses the maximum operation level and reaches to an elevation of 1103.2 m. In the third case, where three gates are partially opened at once, the water level is kept about the maximum operation levels. The final gate opening method which says all of the gates are partially opened at once, best describes the PKW water surface profile. The last gate opening alternative results in smaller and smoother gate openings as well. Partial opening of one gate requires larger and rapid gate opening to increase the discharge capacity. Therefore, gate operation of this kind requires more work and the large opening of gates (e > 0.5 m) are also not desirable. The last alternative operates all the gates so that even with a small gate opening larger discharge capacity can be achieved.

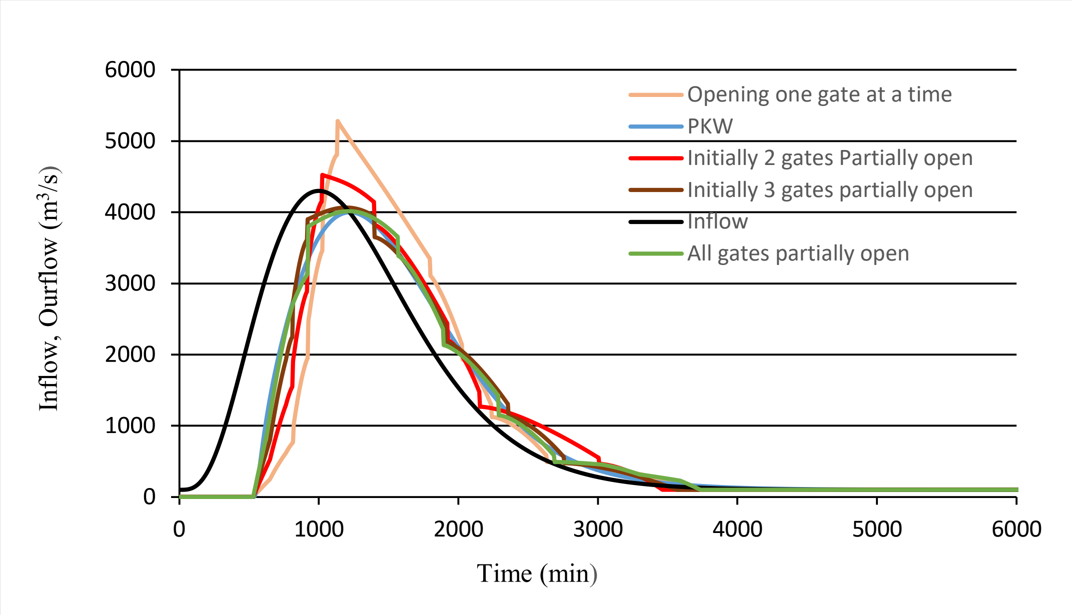

Figure 3.13: Partial Opening of Gates Using Different Methods

From the discharge hydrographs of the opening methods, it can be concluded that the last method which includes partial openings of all the gates at once, predicts the PKW discharge behavior much better. For this reason, the next gate openings will be performed by partially opening all the gates at once.

In the next sections three case studies are considered.

The first study focuses on the gate operation performed stepwise for various gate step sizes. The second method aims at automating gate operations using only reservoir water level as a decision variable. For the third case, a numerical formula is proposed to do the gate operations based on the gate discharge, gate opening and a constant that can be calibrated to include the topographic and inflow variations that affect the computations.

So far, the inflow data was used to route the flow through the reservoir. For the gate operations, only pre-recorded water level data will be used. The pre-recorded water level data is used for computing changes in the reservoir storage, surface area, and water surface levels.

3.6.1 Case Study I: Constant Step Size Gate Operations

The first case study explains constant step size gate operations. When the flood starts entering the reservoir, the flow in the reservoir is regulated considering normal operating conditions. When the water level rises as a result of the flood initiation, the empty storage provided by the freeboard is slowly filled. At this point, the water is about to be over the normal operating conditions. In proportion to the rising speed of the water level, the gates should be opened. Two constant step sizes of 10 cm and 15 cm are considered. As the gates are partially or fully opened, appropriate head-discharge and discharge coefficient formulas are applied. When the water level is observed falling, the gates started to be closed using the same incremental pattern. In other words, every 5 minutes, the gates are closed by either 10 cm or 15 cm, respectively. The opening and closing time of every step operation is assumed to take about 15 seconds. When the water level rises above a specific limit, the gates are opened again. The reservoir level at which gates are opened and closed can be chosen as a point value or as an interval. A point value may be selected for the reservoir level at which we want the water level to stay constant after the reservoir routing process is finished. However, selecting an interval may be more advantageous since the flood volumes can be controlled before rising above the maximum operation level. For this case study, gate operations are performed to keep the water level constant at 1102 m.

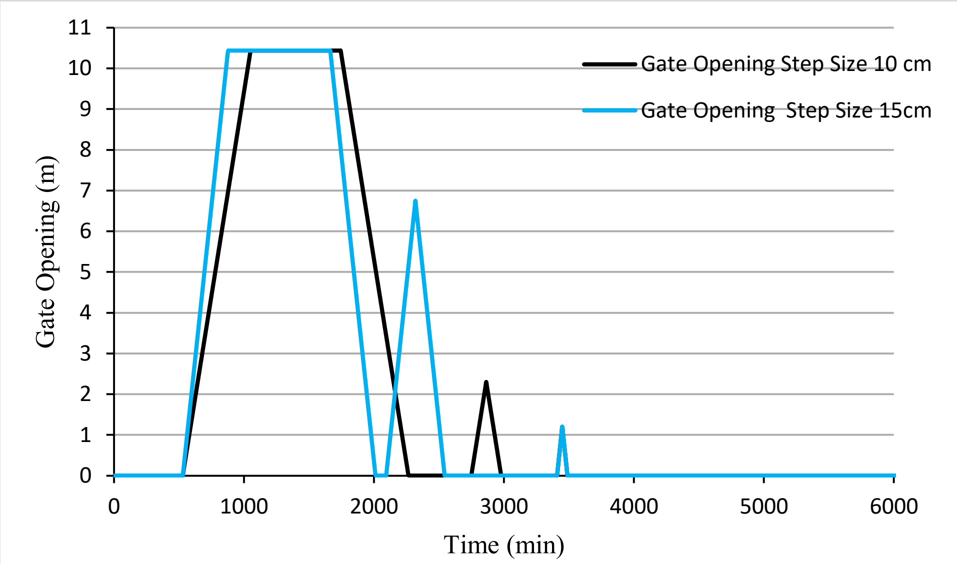

Figure 3.14: Gate Openings with Different Step Sizes

As seen in Figure 3.14, gate operations performed with 15 cm opens and closes gates in a faster manner. Due to the fast closure of the gates, the water level in the reservoir rises up quickly to the target level. As a result, the gates need to be opened again to transfer the excess water downstream.

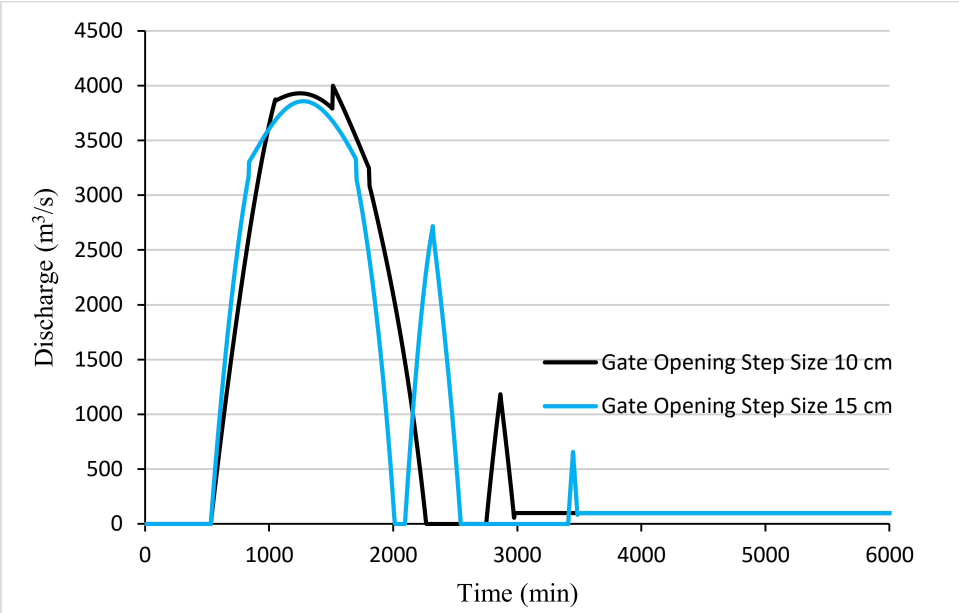

Figure 3.15: Discharge Hydrographs for Each Step Size Opening

The discharge hydrograph related to gate opening step size of 15 cm shows larger values in the initial stage. By contrast, for the 10 cm openings, the discharge values are smaller. At the peak point, the 15 cm opening method discharges more water in the early stages hence the water level does not rise above the maximum operating level. The 10 cm, water level rises above the 15 cm graph indicating the slow opening of the gates. The gate operation with 15 cm increments closes the gates faster, as a result, the water levels rise again quickly. For this reason, gates are closed and opened three times. For the 10 cm incremental gate operation, the gates are closed slowly compared to the 15 cm opening method, hence more water is sent downstream and the water level does not rise quickly. As a result, full gate opening and closing operations are performed only twice. The surface water graphs indicate more details about the two opening methods.

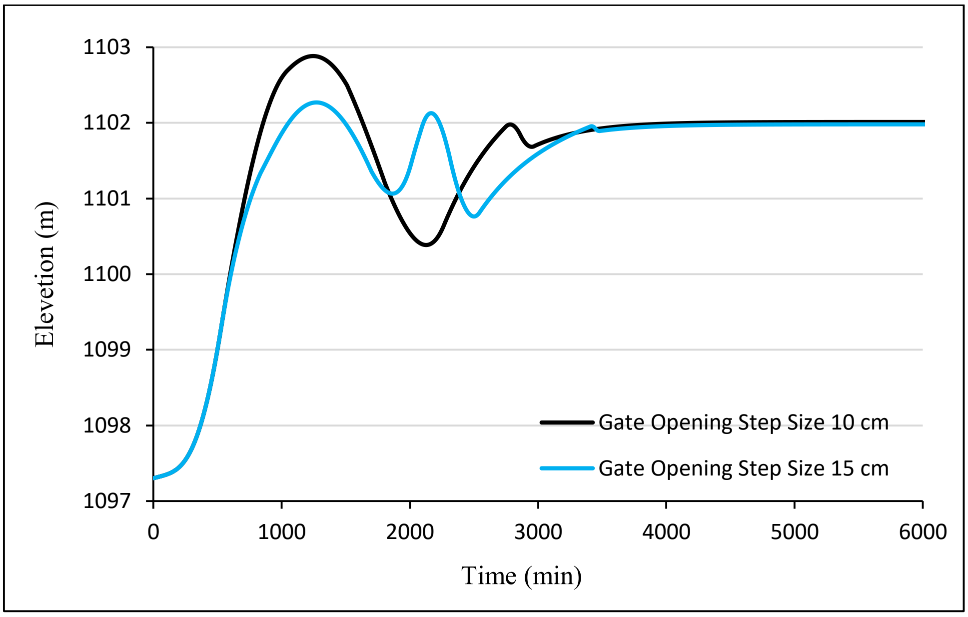

Figure 3.16: Water Level Profiles for Each Step Size Opening

Figure 3.16 indicates that a 15 cm incremental opening restricts water before reaching the maximum operation level of 1102.5 m. The gate operation with 10 cm increments cannot open the gates fast enough to prevent water from passing the maximum operation level. The water surface profile shows that the water levels related to 10 cm openings pass well beyond the maximum operation level. As a result, for this specific inflow and topographic data, the dam gates should be incrementally opened at least by 15 cm so that the water does not cross the maximum operation levels. The advantage of the gated Ogee Crested spillways is that they can be used to store water at different elevations of the reservoir. However, more work is needed to make the gate operations smoother which will be discussed in the next sections. Since the 15 cm opening method keeps water below the maximum operating level, next gate operations will be done with incremental openings of at least 15 cm to satisfy the main conditions of the reservoir routing process.

3.6.2 Case Study II: Pre-set Minimum and Maximum Target Levels

For this case study, the gates of the dam are operated automatically based on the water levels in the reservoir only. When to open or close the gates is a decision that should be made considering the speed of rising or falling of the water levels in the reservoir. A better decision may be made, after gaining experience and practicing the gate operation with previously recorded data. To automatically perform gate operations based on the water level in the reservoir, two scenarios for decision-making may be considered. The first way is to open and close gates after the water reaches a specific water level in the reservoir. For example, gates are partially opened when the water level is above a pre-specified maximum level and the gates are partially closed when the water level falls below a pre-specified minimum water level. Selecting this interval may also be related to the speed of the water level rise in the reservoir.

As mentioned in the previous section, gates can be opened and closed considering different gate opening increments. For this case study, 15 cm increments are selected as it prevents water from passing the maximum operation levels. In this case study, the gate operations are done to keep the water levels at 1101m, 1102m and 1103m in the reservoir as three different test cases.

The gate operation in this case has two primary goals. The first goal is to transfer excess water downstream in a safe manner so that the flood does not breach the maximum operating levels. The second goal is to keep the water levels between the normal and the maximum operation levels at the end of the reservoir routing process. With this being said, the water stored in the reservoir should be kept at optimal levels. At the end of the reservoir routing process, water can be stored at desired levels in the reservoir based on appropriate gate operations.

To contain water between normal and maximum operating levels, the gate opening should begin when the water level exceeds the normal operating level (1097.3m). The gates are opened with an initial value of 15 cm. The opening of the gates by increments of 15 cm continues as the water levels rise. The gates are finally fully opened. The gates are left in a fully open state until the water level falls below the target level.

The following calculation steps were considered for the above process. First, the recorded data of the head rise of water in the reservoir was used to calculate surface area A(h) using Eq. 3.2, the speed of head rise f (H, t) = $\Delta H/\Delta t$ and the change in the storage $\Delta S/\Delta t = (\Delta H/\Delta t) * A(h)$.

Table 3.5: Initial values of the Recorded Data and Calculation of A and $\Delta H/\Delta t$

Time (min)	A(h) m^2	$\Delta H/\Delta t$ (m/s)	Recorded H(m)	$\Delta S/\Delta t$ (m^3/s)
0	13640505.05	-	5.237	100
5	13640993.25	7.33093E-06	5.239399	100.001208
10	13641481.46	7.33157E-06	5.241598	100.0134301
15	13641969.7	7.33432E-06	5.243798	100.0545505
20	13642458.14	7.34081E-06	5.246001	100.1467183
25	13642947.02	7.35287E-06	5.248206	100.314825
30	13643436.7	7.37245E-06	5.250418	100.5856032
35	13643927.69	7.40161E-06	5.252639	100.9869998
40	13644420.63	7.44243E-06	5.254871	101.5477069
45	13644916.29	7.49706E-06	5.257121	102.2967958
50	13645415.6	7.56763E-06	5.259391	103.2634237
55	13645919.61	7.65625E-06	5.261688	104.4765939
60	13646429.52	7.76503E-06	5.264017	105.9649597

The discharge coefficients for the partial and full gate openings are calculated based on Equation (2.22) and Equation (2.13) respectively. The discharge values are subtracted from the $\Delta S/\Delta t$ column. The new values of the head in the reservoir are updated after outflow released from the reservoir. The upgraded values of water head in the reservoir are used for the calculations of surface area and water level in the reservoir.

Table 3.6: Calculated Values

Time (min)	Gate Opening(m)	C_d(Partial)	C_d(Full)	Q (m^3/s)	$\Delta S/\Delta t - Q$ (m^3/s)
0	0	-	0.4520	0	100.000
5	0	0.7512	0.4520	0	100.001
10	0	0.7512	0.4521	0	100.013
15	0	0.7512	0.4521	0	100.055
20	0	0.7512	0.4521	0	100.147
25	0	0.7512	0.4521	0	100.315
30	0	0.7512	0.4522	0	100.586
35	0	0.7512	0.4522	0	100.987
40	0	0.7512	0.4522	0	101.548
45	0	0.7512	0.4522	0	102.297
50	0	0.7512	0.4522	0	103.263
55	0	0.7512	0.4523	0	104.477
60	0	0.7512	0.4523	0	105.965

The updated values of H, A, effective length and the water level based on ΔH is shown in Table 3.7. ΔH is the change in the head rise due to the recorded water head data and the discharge. Since no discharge is allowed until the freeboard volume is filled, the value of ΔH only includes effects of the recorded data. As soon as the outflow starts, the ΔH will also include the reduction in the head rise due to the initiation of the outflow.

Table 3.7: Updating Parameters Based on ΔH

Time (min)	Effec. L(m)	ΔH (m)	H updated (m)	A (h) (m^2)	Water Level(m)
0	55.057	0.0022	5.239	13641437.367	1097.302
5	55.057	0.0022	5.241	13641925.558	1097.304
10	55.057	0.0022	5.244	13642413.795	1097.307
15	55.056	0.0022	5.246	13642902.221	1097.309
20	55.056	0.0022	5.248	13643391.084	1097.311
25	55.055	0.0022	5.250	13643880.754	1097.313
30	55.055	0.0022	5.252	13644371.734	1097.315
35	55.055	0.0022	5.255	13644864.661	1097.318
40	55.054	0.0022	5.257	13645360.311	1097.320
45	55.054	0.0022	5.259	13645859.605	1097.322
50	55.053	0.0023	5.261	13646363.603	1097.324
55	55.053	0.0023	5.264	13646873.509	1097.327

Table 3.8: Initiation of Outflow from the Reservoir

Time (min)	Gate Opening(m)	Q (m^3/s)	ΔH (m)	Updated H (m)	Water Level (m)
530	0	0	0.052386	7.254	1099.317
535	0.15	75.279	0.051517	7.306	1099.369
540	0.3	148.353	0.050693	7.356	1099.419
545	0.45	220.569	0.049886	7.406	1099.469
550	0.6	291.802	0.049097	7.455	1099.518
555	0.75	362.109	0.048325	7.504	1099.567
560	0.9	431.536	0.047569	7.551	1099.614
565	1.05	500.116	0.046827	7.598	1099.661
570	1.2	567.878	0.0461	7.644	1099.707
575	1.35	634.842	0.045385	7.689	1099.752
580	1.5	701.028	0.044683	7.734	1099.797
585	1.65	766.451	0.043993	7.778	1099.841
590	1.8	831.125	0.043315	7.821	1099.884
595	1.95	895.062	0.042647	7.864	1099.927
600	2.1	958.272	0.04199	7.906	1099.969
605	2.25	1020.763	0.041342	7.947	1100.010

In this case study, the gates are closed after the water level falls 0.5 m below the target level. The gates are opened if the reservoir level is 5 cm lower than the target level. Based on this concept, the following gate opening graphs are obtained. To have automatic gate operations using this method, a simple computer code is written in terms of the water level involving an upper and a lower water level interval. This is the simplest method for automating gate operations. The limits for gate openings and closings considering the target levels of water may be chosen differently. In this case study, a possible alternative for automating gate operations depending only on the water level in the reservoir is provided.

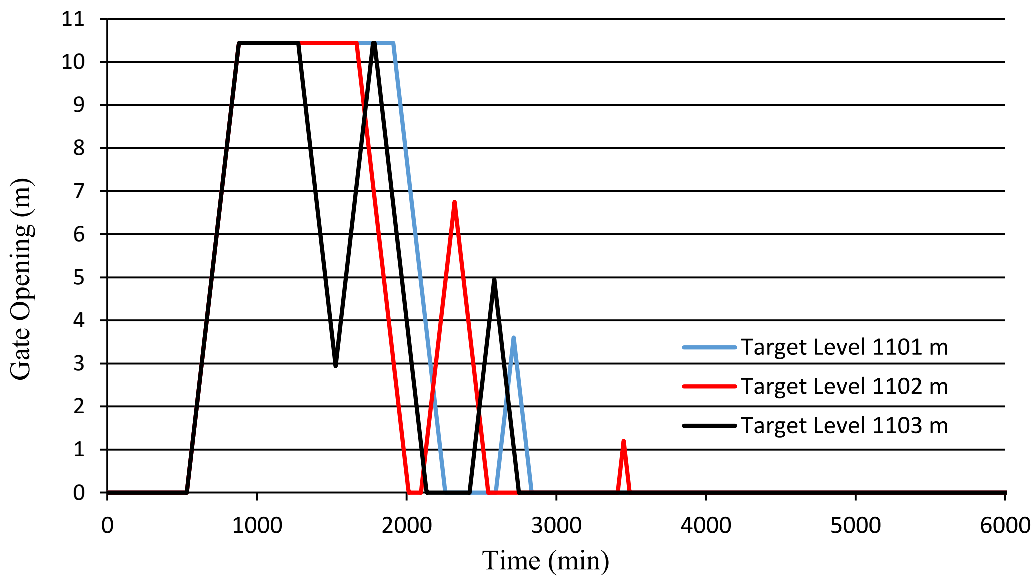

Figure 3.17: Gate Operations Graphs for Different Target Levels

The initial gate openings are the same for all the target levels until the gates are fully opened. For the 1101 m target level the gates stay fully opened for a longer time compared to the target levels of 1102 m and 1103 m. Since the target level of 1101 m is lower than the other target levels, more water is needed to be released. For this reason, gates are kept fully opened for a longer time to reduce the water level to 1101 m as seen in Figure 3.17. As a result of larger volumes of water being released in the initial stage, fewer gate openings and closings are required to store water at the target level of 1101 m. For the target levels of 1102 m and 1103 m, the gate operations are repeated three times. Meanwhile, the second opening of the target level 1103 m is higher than that of 1102 m. Since less water release is required to bring the water level to 1103 m, the water level rises back more quickly than in the case of 1102 m.

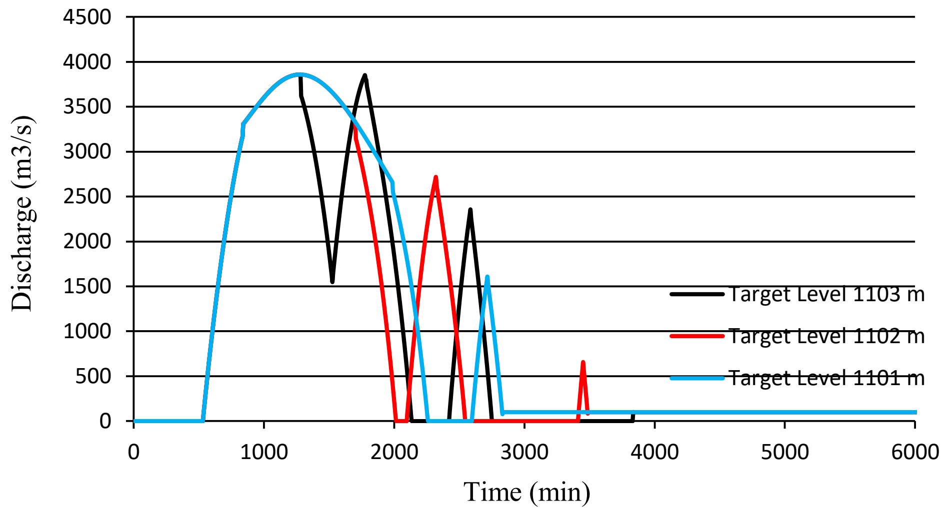

Figure 3.18: Discharge Comparison of the Target Levels

From the discharge hydrograph and gate operation graph, it is concluded that keeping water at higher levels requires more gate operations. To keep water at 1103 m, the gates should be closed faster so that the extra water loss is prevented. Since most of the flood is remaining, the inflow values are still high and the water level rises to the elevation of 1103 m again. Therefore, gates are opened again to release the excess water. As seen in Figure 3.18, the water is released for the third time as well. However, less water is released in the third gate operation as most of the flood event is completed. To keep water at 1102 m, more water is released in the initial stage of the gate operation. Most of the flood event is passed in the initial stage of the gate operation. For this reason, the volume of the water released in the second gate opening is less than that of the 1103 m. The third gate operation releases a very small amount of water to keep the water level at 1102 m. Finally, in the case of target level 1101 m, most of the water is released in the initial stage until the water level falls to 1101 m. When the water level is at 1101 m, the flood event is almost completed, hence two gate operations are enough to keep the water level at 1101 m. More details of the gate operations are shown in the water surface profile below.

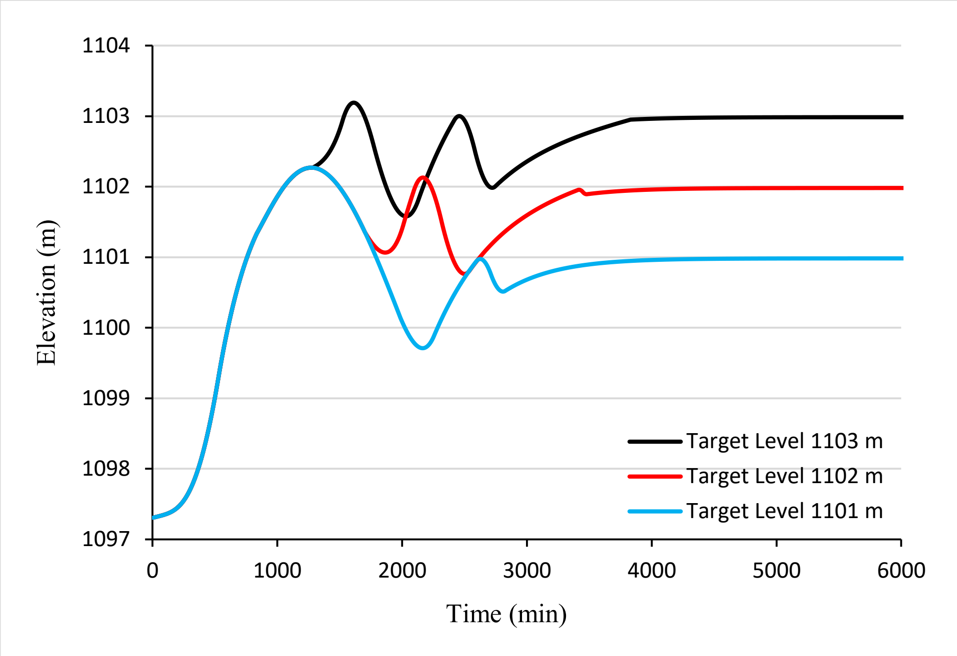

Figure 3.19: Surface Water Profiles for Different Target Levels

Figure 3.19 indicates that the water surface rise until 1102.2 m is same for all target levels. Since the target level 1103 m is above the maximum operation level, the gates should be closed for a while so that the water level reaches to 1103 m. The ups and downs in the water surface profile shows closing and openings of the gates respectively. At the final stages of the flood event where the speed of the head rise is lower, the gates should be closed earlier to store more water.

3.6.3 Case Study III: Empirical Formula Based Gate Operations

In this case study, the gate openings are formulated by an empirical expression in terms of gate discharge, gate opening, discharge at target level, a coefficient for calibrating the reservoir geometry, the storage rate and the speed of the water rise.

$$\Delta e = \frac{de}{dQ}(Q1 - Q2) * C \tag{3.14}$$

where Q1(h(t), e) is the discharge value equal to the storage rate at time t. In this formula e is the gate opening at time t, Q2 is the gate discharge at the target level for the same gate opening as that of Q1, Q2(h_0, e). To find the gate opening for Q2, at best a second order polynomial is fitted into the artificially produced data.

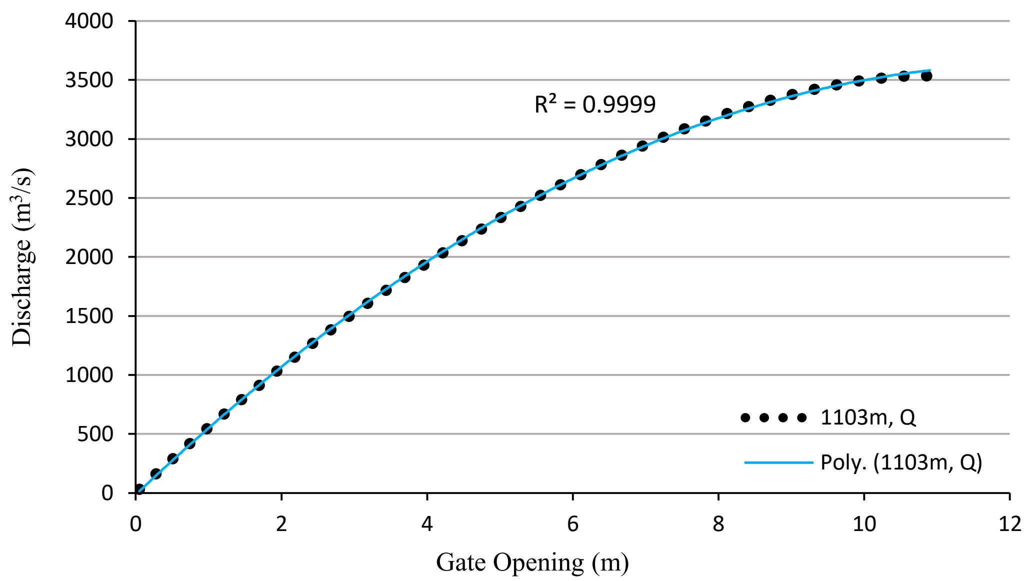

Figure 3.20: Gate Discharge vs Gate Opening for 1103m. Water Level

Figure 3.20 displays values of the outflow from the gates plotted against the gate opening. The head of water is kept constant at 1103 m and the curve is plotted for different gate openings. The best function is written in the following form.

$$Q_2 = -23.786e^2 + 589.28e - 15.573 \tag{3.15}$$

After obtaining Eq. (3.15), the curve for de/dQ is plotted as shown in Fig. 3.21.

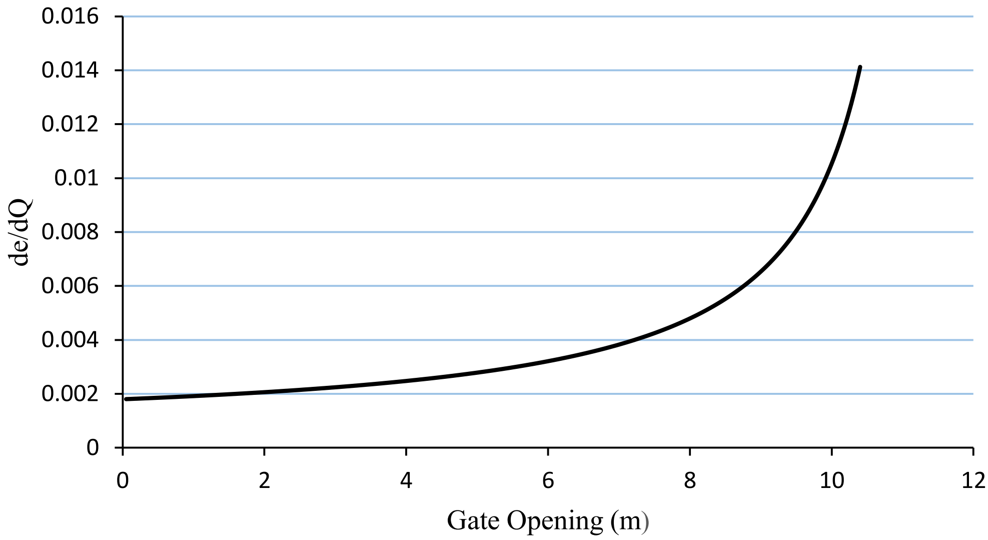

Figure 3.21: de/dQ vs gate openings for water level 1103m

Values of de/dQ are used in Eq. (3.14). The equation is calibrated for the constant C by trial and error.

In this study, the discharge curve is divided into three parts. The rising part of the curve where water levels rise till gates are fully open. The second part includes data points on the falling side of the curve but the rate of the head rise is still significant. The third part is included the in falling limb of the graph of change in the storage but throughout this part, the rate of head rise is marginal or very small. Different values of C are applied for each of the stages and the target level. For the 1103 m target level, C is taken as 0.23 when the rate of the storage change is positive. The gate opening starts from the normal operation condition and the gates are being partially opened until full openings. When the rate of change in the storage is negative, the C coefficient is taken as 0.13. The coefficient is selected as 0.23 when the speed of head rise is less than 10^$^{-4}$ m/s. The gates are closed when the gate opening is zero and 100 m^3/s of the incoming volumes of water is released through the turbine. This means the flood event is finished and the gate operations of the reservoir are performed in normal operation conditions. For the target level of 1102 m, C is taken as 0.33, 0.01 and 0.154 for the three stages of the gate operations. Similarly, C is taken as 0.35, 0.08 and 0.1 for the target level 1101 m. The following procedure is followed for computing the discharge and surface water levels. For the initial stage, absolute value of (Q1-Q2) in Equation is considered so that gates will be opened until the maximum value of gate opening (10.37 m) is reached.

- First, gate discharge at the desired level is plotted against gate opening to get the gate opening curve.
- From the above graph de/dQ is plotted against e
- For the first opening at t=to, Q1 = Q(spill), which is computed using the recorded data and the amount of gate opening e is found. Q2 is also calculated for the same gate opening using Figure 3.20
- For t+Δt, we obtain h(t) value from the data and compute Q1, Q2 respectively.
- After this de/dQ is calculated form the graph in the second step
- then Δe is calculated from Equation (3.14).
- The constant C is selected according to each of the routing stages discussed above
- New gate opening is found by Δe + e

The calculation steps explained above are shown in the tables below.

Table 3.9: Recorded Data and Calculations of Gate Opening and Cd

time (min)	ΔH/Δt (m/s)	H(recorded) (m)	Gate opening(m)	Cd (Partial)	Cd (Full)
535	0.000145	6.960	0.500	0.780	0.468
540	0.000147	7.004	0.517	0.724	0.468
545	0.000149	7.049	0.543	0.723	0.468
550	0.000151	7.095	0.569	0.723	0.469
555	0.000153	7.141	0.596	0.722	0.469
560	0.000155	7.187	0.624	0.721	0.469
565	0.000157	7.234	0.653	0.720	0.470
570	0.000159	7.282	0.682	0.720	0.470
575	0.000161	7.330	0.713	0.719	0.471
580	0.000163	7.379	0.744	0.718	0.471
585	0.000165	7.429	0.777	0.718	0.471
590	0.000166	7.479	0.810	0.717	0.472
595	0.000168	7.529	0.844	0.716	0.472
600	0.000170	7.580	0.879	0.715	0.472

The calculation of Q1, Q2 and the water surface profile from the recorded data is shown in the following table. As mentioned in the Case Study II, the Δs/Δt is calculated form the recorded data. ΔS/Δt- Q1 is computed in the next step. The head values are updated according to the change in the record by the initiation of the discharge. Similarly, water level and other parameters are calculated.

Table 3.10: Calculation of Q1, Q2, Change in Storage and Water Level

Q1 (m^3/s)	ΔS/Δt (m^3/s)	ΔS/Δt-Q1	updated H (m)	Water Level(m)	Q2(m^3/s)	de/dQ
247.509	2038.793	1791.284	6.957	1099.020	288.545	0.00185
238.250	2068.209	1829.959	6.996	1099.059	297.978	0.00185
250.256	2097.472	1847.216	7.035	1099.098	311.704	0.00186
262.556	2126.572	1864.016	7.075	1099.138	325.826	0.00186
275.253	2155.503	1880.249	7.115	1099.178	340.365	0.00186
288.352	2184.253	1895.901	7.156	1099.219	355.327	0.00187
301.859	2212.817	1910.958	7.197	1099.260	370.717	0.00187
315.781	2241.185	1925.404	7.238	1099.301	386.540	0.00187
330.122	2269.349	1939.227	7.279	1099.342	402.798	0.00188
344.888	2297.302	1952.414	7.320	1099.383	419.496	0.00188
360.084	2325.037	1964.953	7.362	1099.425	436.638	0.00188

The gates operation performed this way displays smoother graphs as shown in the Figure 3.18.

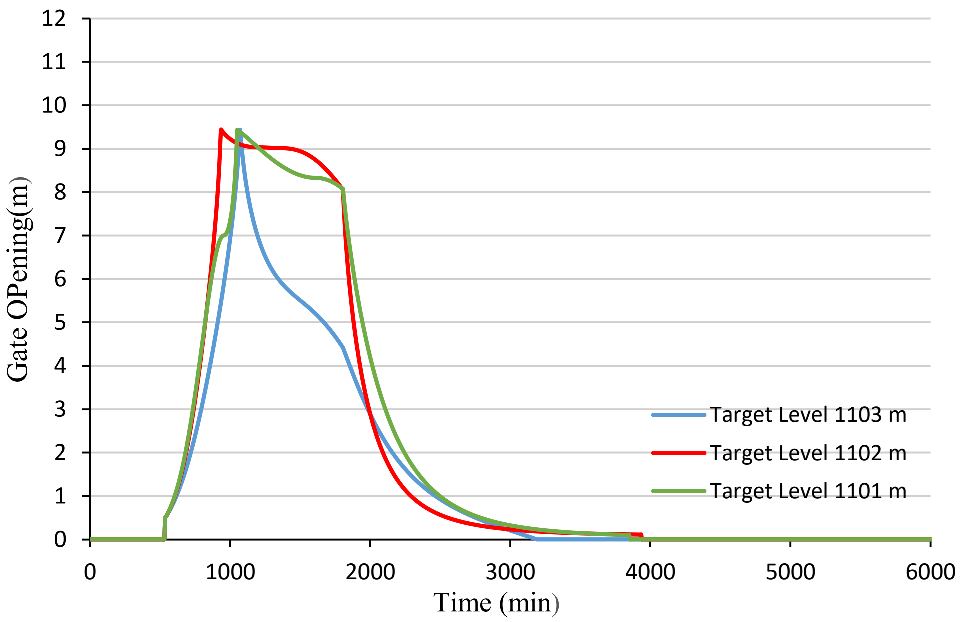

Figure 3.22: Gate Openings for Different Target Levels

As seen in Figure 3.22, the amount of water released for the target level 1103 m is less compared to the other target levels. The outflow graph of 1103 m overlaps the inflow graph for a short period of time. However, in the case of target levels 1101 m and 1102 m the outflow graphs show larger amount of water released to reduce the water level and store water at the desired levels. As seen in Figure 3.22, the gate openings are performed only once and the opening increments are decided by Equation (3.15). Moreover, the gate operations are smoother comparted to Case Studies I and II.

The discharge hydrograph obtained using the numerical formula is shown in Fig. 3.23.

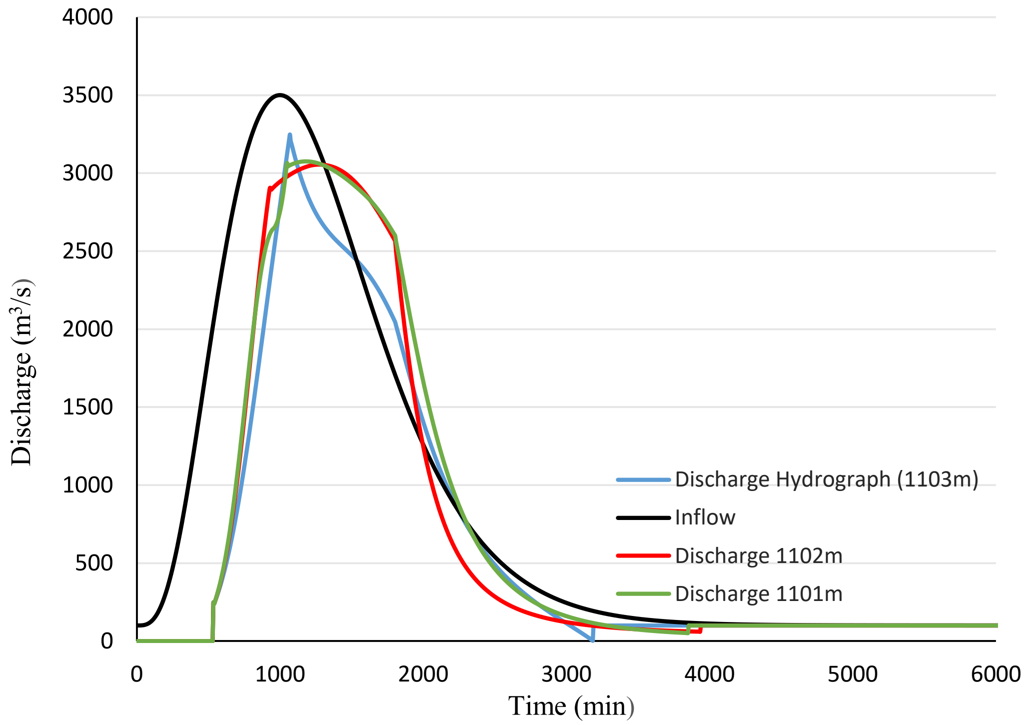

Figure 3.23: Discharge Curves of Produced Based on the empirical Formula

Figure 3.23 confirms the gate operation process performed by the empirical formula. As discussed previously, to store water at the target level 1103 m, less amount of water is released compared to Case I and Case II. The water surface profiles show more details of the gate operation process based on the new equation.

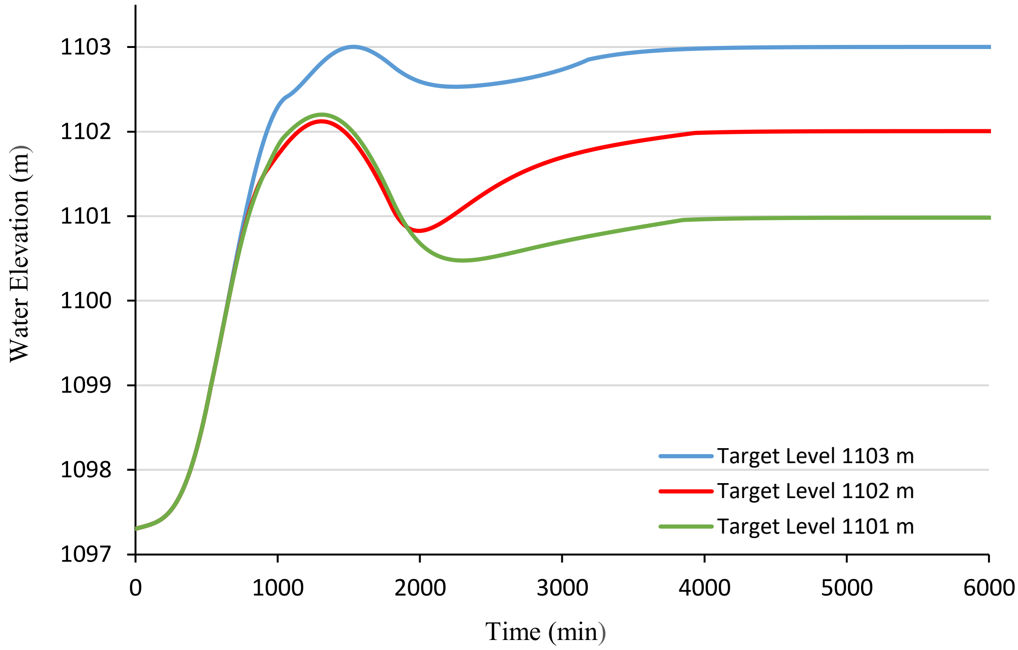

Figure 3.24: Profiles of the Target Water Levels

The current gate opening method perform the gate operations to get water near the maximum operating level which is also the designed water level for the reservoir. After that the water is stored or released based on the location of the target level. For target levels higher than the maximum operation level, the gate is partially closed to rise the water level to the desired level. On the other hand, if the target level is lower than the maximum operation level, the gates are opened to release water and lower the water level. It may be a better practice not to allow water levels go beyond the target levels. In such a case, the gated spillway should be designed with a larger discharge capacity to keep water at each target level. Logically, it may not seem reasonable to store water level beyond the maximum operation or the designed water level. In this specific case, different water levels below and above the designed level were studied for the sake of practice. Equation (3.14) can be used for any reservoir but the values of coefficients C hold true only for this reservoir.

CONCLUSIONS

In this study, automation of the spillway gate operations based on the reservoir water levels using hydrologic reservoir routing is investigated. As a first step, the gate operations of an ogee crested spillway were arranged to imitate the discharge behavior of an equaivalent PKW for a given flood hydrograph. Then some test cases were considered to operate the gates to keep the reservoir level at a pre-determined value by observing the water level changes in the reservoir wihout knowing the inflow hydrograph. The first test case was the automation of gate operations based on a constant step size for the gate opening. In the second test case, automated gate operations are investigated based on reservoir water level monitoring. In the third test case, an empirical formula is applied to the reservoir routing process. Based on this formula, the gate operations are automated to store water in the reservoir at pre-specified levels.

Major findings of this study can be listed as follows:

1) It is possible to imitate a PKW system by spillway gate operations using a computer algorithm when the inflow hydrographs are known as illustrated in this study.
2) Water level monitoring of the reservoir can be used for gate automation when the inflow hydrograph is not available. However, this requires development of an empirical formula which may involve determination of some parameters by calibration for each specific reservoir-spillway system.
3) The automated gate operations conducted by computers without human interferences may improve safety of dams against flush floods when there is not enough time for preparation for an emergency case.

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