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Optimal operation of multi-reservoir system for flood control: Application to the Vu Gia Thu Bon catchment, Vietnam

Thanh Hao Nguyen

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THÈSE DE DOCTORAT

Gestion optimale d'un système multi-réservoirs pour le contrôle des crues

Application au bassin versant du Vu Gia Thu Bon,
Vietnam

Thanh-Hao NGUYEN

Polytech Lab

**Présentée en vue de l'obtention
du grade de docteur en**
Contrôle, Optimisation et
Prospective

d'Université Côte d'Azur

Dirigée par :

Pr. Philippe GOURBESVILLE

Soutenue le : 17 Juin 2020

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Invité

Abstract

The multi-reservoir system in the Vu Gia Thu Bon catchment plays an important significant role in the production of annually alternative electrical and mitigation of flood damage. However, the reservoir system usually faces severe flooding that results from natural characteristic and hydro-climatic conditions within the basin. Therefore, optimization of the operation of the multi-reservoir system is a problem of close interest to the owners of these hydraulic facilities.

The main objective of the current research is to control flood flows and flood levels at various locations at the downstream of the Vu Gia Thu Bon catchment. Due to the characteristics of the system and the targeted optional objectives, a flood control operating strategy has been developed based on coupled simulation-optimization to reduce downstream flood damage of the multi-reservoir system by using spillway gates. The objective function is minimizing the total damages during the flood events that can be expressed as a function of water surface elevations at the inundation zones.

The proposed method is based upon combining of three major components: (1) a hydraulic 1D model that allows simulating the flows in the river including the reservoir system, (2) an operation reservoir module adopted for simulation of the multi-reservoir considering physical constraints of the system as well as operation strategies, and (3) an optimization model (Shuffled Complex Evolution algorithm) applied to determine the best set of spillway gates levels, which specify the reservoir release.

The method has been successfully implemented for the multi-reservoir system in the Vu Gia Thu Bon catchment. Three flood events in 2007, 2009 and 2017 were selected for demonstration. In order to assess performance of the approach and for comparison purpose, three developed scenarios that are representing operations the reservoir system in the historical, the current rules and the proposed model have been used. The results indicate that the proposed model provides much better performance for all scenarios in terms of reducing the peak flow as well as reducing the maximum water levels at selected downstream control points compared to the rest scenarios.

Keywords: hydropower, flood control, multi-reservoir operation, multipurpose reservoirs, optimization methods, river-reservoir system, distributed deterministic hydrological modeling, 1D hydraulic modeling, Vu Gia Thu Bon catchment, Vietnam.

Résumé

Les différents réservoirs dans le bassin versant Vu Gia Thu Bon jouent rôle essentiel pour production d'électricité ainsi que pour la réduction des risques d'inondations et des dommages dans les secteurs aval. Cependant, les réservoirs sont confrontés à de crues extrêmes qui résultent des caractéristiques naturelles du bassin versant et des conditions hydro-climatiques. L'optimisation de la gestion des différents réservoirs est une priorité pour les responsables de la sécurité civile.

Ce travail de recherche a comme objectif de développer les méthodes d'évaluation des opérations des retenues, nécessaires à la protection contre les crues du bassin versant Vu Gia Thu Bon. La stratégie de contrôle des crues est basée sur un modèle qui associe simulation-optimisation. La fonction objective consiste à minimiser les dégâts totaux d'inondation qui dépend des débits ou des hauteurs d'eau dans les secteurs aval.

La méthode proposée comporte trois composants majeurs : (1) la simulation des débits et des niveaux d'eau réalisée par un modèle hydraulique 1D ; (2) la simulation des opérations pour la production hydroélectrique réalisée par un module d'opération de structure ; (3) un modèle d'optimisation (algorithme Shuffled Complex Evolution) destiné à obtenir les règles optimales d'opération pour les retenues.

La méthode a été mise en œuvre avec succès pour le système multi-réservoirs dans le bassin versant du Vu Gia Thu Bon, Vietnam. Les performances du modèle d'opération et optimisation pour gestion des crues ont été évaluées la base des crues historiques de 2007, 2009 et 2017. Les résultats obtenus indiquent que les stratégies proposées par le modèle offrent de bien meilleures performances pour la réduction du débit de pointe et sur la diminution du niveau maximal de crue dans les secteurs aval. La méthodologie peut donc être transférée pour la gestion opérationnelle du bassin versant du Vu Gia Thu Bon.

Mots-clés : système multi-réservoirs, contrôle des crues, production hydroélectrique, Shuffled Complex Evolution, multi objectifs, optimisation, stratégies de gestion optimisé, bassin versant du Vu Gia Thu Bon, Vietnam.

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Chapter 1

Introduction

The current research targets the optimization of multi-reservoir system regarding the concurrent objectives of energy production and downstream flooding management. This chapter begins with a brief overview of problems related to hydropower project development and their influences on hydrological dynamic in rivers. The identification of key questions to be addressed and research objectives are presented. The overall organization of the thesis is concluding this first chapter.

1.1. Background

Humans have been harnessing water to produce energy and perform work for thousands of years (Viollet, 2017). Hydroelectric power is considered as one of the oldest energy sources to generate mechanical and electrical energy. The main advantages of hydropower are a renewable power source, low operating costs in comparison with other power sources, a clean fuel source to the air, a flexible source since the hydroelectric plant can go from zero power to maximum power quickly to adapt energy grid demands. In addition to a sustainable fuel source, hydropower project construction may produce additional benefits such as irrigation, water supply for public and industrial activities, shipping and navigation, flood control, recreational activities and aquaculture. However, uses conflicts may appear among all of those activities and many recent examples have underlined this type of situation (Bene, 2018; Hess & Fenrich, 2017; Kalair, 2012).

In the past two decades, Vietnam economy has been developing remarkably. According to the World Bank (WB, 2016), the country is one of the most dynamic emerging countries in the East Asia region. The energy sector plays an essential part in fostering the socio-economic development of the country. Vietnam has a dense river system with a lot of large river basins (ADB, 2016). According to the assessment of energy potential, the total hydropower capacity is about 26,000 MW, with a total produced about 100 billion kWh (ICEM, 2008). Progress of total primary energy supply grew at 4.7% for the period time 2007-2017, in which hydropower experienced the highest growth at 14.5% (EREA & DEA, 2019). The role of hydropower for sustainable energy development is important (Kaygusuz, 2009), but problems related to the operating efficiency of existing reservoirs is still a considerable challenge.

Vietnam is located in the tropical monsoon climate in which rainfall amount is abundant but strongly variable flow in time and space. The situation of too much wet season water and too little dry season water is causing many reservoir operating difficulties (Richaud et al., 2011). On the other hand, due to typhoons with heavy rains and complicated topography, floods become a constant threat to livelihoods located in the downstream areas. In such complex situation, reservoir operation is an intricate problem that involves multiple decision variables, numerous objectives, as well as significant uncertainties and risks (Anand et al., 2018; Oliveira & Loucks, 1997). In the Vietnamese context and even more globally, flood control and mitigation are

a high concern for authorities who are responsible for reservoir operation in flood seasons and population safety. Obviously, each river system and catchment request a specific flood control system and ways of tackling floods may differ from one river to others. Therefore, designing optimal operation of multi-reservoir systems with multiple objectives is a major issue and represents a challenging task. Each decision taken for one reservoir would have significant impacts on the rest of other reservoirs in the system, as well as on flood conditions in the entire river basin (Che & Mays, 2017).

1.2. Problem description

The Vu Gia Thu Bon river basin is the fourth largest in terms of potential hydroelectric capacity in Vietnam after the Da, Dong Nai, and Se San river systems (ICEM, 2008). This basin plays a significant role in the social and economic aspects of the central region of Vietnam. The Government of Vietnam has planned eight large-medium hydropower projects on Vu Gia Thu Bon catchment in the seventh National Power Development plan with a total power capacity of approximately 1,100 MW (Government of Vietnam, 2011). Besides undeniable benefits, operating of the hydropower reservoir system still has some limitations and the project is frequently judged to have increased natural disasters in recent years. Flood damages caused by hydropower operation could elicit public outrage, leading to increases stress for decision-makers in performing the flood control operation (Chou & Wu, 2013). During the flood event in 2013, all hydropower reservoir operators stated that they had complied correctly with operational regulation. Still, the residents who suffered the severe flooding consequences did not absolve the responsibility of the operators (Luu et al., 2014). In such difficult and conflicting situations, the analysis of multi-reservoir system operation typically with optimization and simulation models can provide quantitative information to improve operational water management.

1.3. Research questions

The research area of this study is the reservoirs system on Vu Gia Thu Bon catchment. The four major reservoirs are vital engineering systems for flood mitigation and electricity generation. This research work focuses on the management and mitigation of flooding at the downstream area of the Vu Gia Thu Bon catchment. In this context, the main challenge of the research work is the optimal operation of the multi-reservoir system to mitigate flood

damages in the downstream area. The following research questions are addressed:

- What is causing flooding in the Vu Gia Thu Bon catchment, and what is the role of the hydropower reservoirs in reducing flood damages?
- What is the operating procedure in multi-reservoirs for flood mitigation, and is it optimized?
- What is the recent development in numerical operation reservoir simulation, and can it be used for multi-reservoir system in the Vu Gia Thu Bon catchment?
- Could a novel method be proposed to deal with the optimal operation of the multi-reservoir system?
- What are the adaptation strategies of reservoir operation for the Vu Gia Thu Bon multi-reservoir system?

To answer the above questions, a list of research goals has been identified and is addressed within this research project.

1.4. Research objectives

The overall objective is to identify the implications of optimal operation of the multi-reservoir system for flood control in the Vu Gia Thu Bon catchment and to propose a novel simulation-based optimization model that could be used for operational operation.

Following the overall objective, the specific goals of this research include:

- Analyzing general practices in flood control of reservoir system and operation rules of the existing multi-reservoir system of the Vu Gia Thu Bon catchment.
- Modeling and simulating the existing multi-reservoir operation according to the current regulations.
- Combining simulation and optimization models in reservoirs system operation to optimize flood regulation strategy in the Vu Gia Thu Bon multi-reservoir system.
- Using the simulation-based optimization technique to optimize the reservoir operation for flood control and validating implementation of the defined strategy with the recorded historical flood events.
- Proposing strategies for optimal operation of the Vu Gia Thu Bon multi-reservoir system.

1.5. Methodology

The current research aims to propose an overall method for calculating a multi-reservoir release strategy, which is implemented before, during, and after flood events. The problem is described as an optimal control problem, in which decision variables are a function of spillway gates openings.

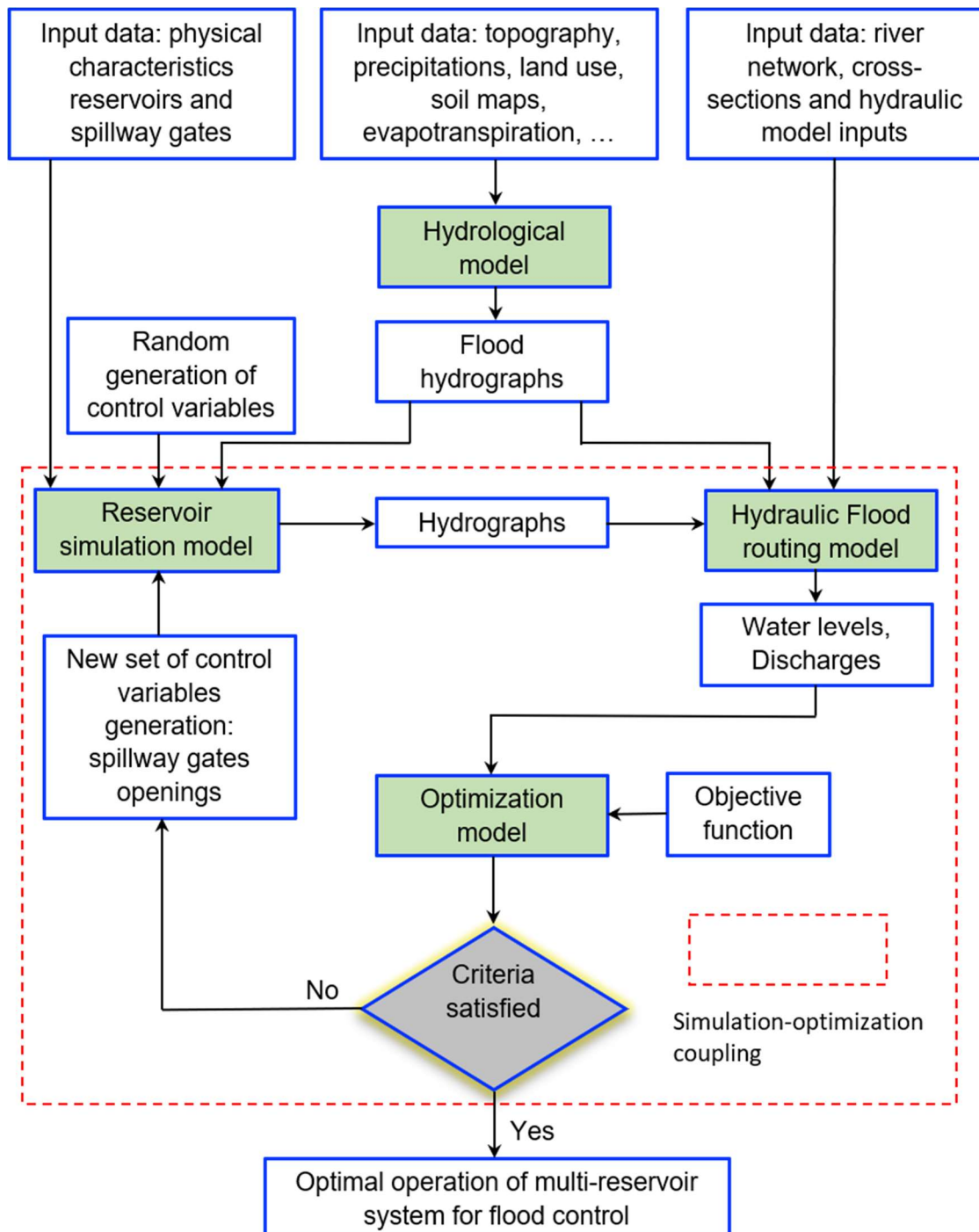


Figure 1.1. Framework of the proposed methodology for optimal operation of multi-reservoir system.

Fig. 1.1 shows the flowchart of the proposed approach for optimal operation of the multi-reservoir system in the Vu Gia Thu Bon catchment. The optimization-simulation model for multi-reservoir operation consists of three major components: (1) a hydraulic 1D model (Mike 11) that allows simulating the flows in a river system with different hydraulic structure (including the reservoir systems), (2) an operation reservoir model (Mike 11 Structure Operation (SO)) adopted for simulation of the multi-reservoir considering physical constraints of the system as well as operation strategies, and (3) an optimization model (the Shuffled Complex Evolution algorithm) applied to determine the best set of spillway gates levels, which specify the reservoir release.

The first component is the operation reservoir model (Mike 11 Structure Operation (SO) module) that requires the characteristics of reservoirs, the operation rules and reservoir inflows in order to start a reservoir operation simulation. The reservoir characteristics curves and operation rules are readily available at these reservoirs, however, the inflows are not available. In fact, in the Vu Gia Thu Bon catchment, there are only two stations (i.e., Nong Son and Thanh My stations) for measuring discharges. These stations are located at the middle of these branches (Fig. 1.2). Hence, the inflows must be estimated based on a rainfall-runoff model.

Vo (2015) has been successfully built a deterministic distributed hydrological model based on Mike SHE model and the efficiency of the model is also verified by the capability of predicting extreme peak flow and baseflow. Mike SHE is a deterministic modeling system based on physical laws (Jajarmizadeh et al., 2012). Most of the parameters used are physical variables that develop within a set of values that can be described by physical processes (Vo & Gourbesville, 2016a). Furthermore, the potential of overcoming the vulnerability and the lack of systematic data is one of the positives of the deterministic distributed models. With regard to the particular problems in the Vu Gia Thu Bon catchment, Mike SHE model represents an appropriate alternative for hydrological modeling in a situation of data scarcity.

Therefore, in the current research, reservoir inflow series are extracted from the hydrological Mike SHE model. Once the reservoir hydrographs are obtained, they are entered into the SO module as inputs. After that, this module computes gates openings and released outflows from the reservoirs

based on the operational rules. The released hydrographs of the reservoirs are entered into the hydraulic model as boundary conditions.

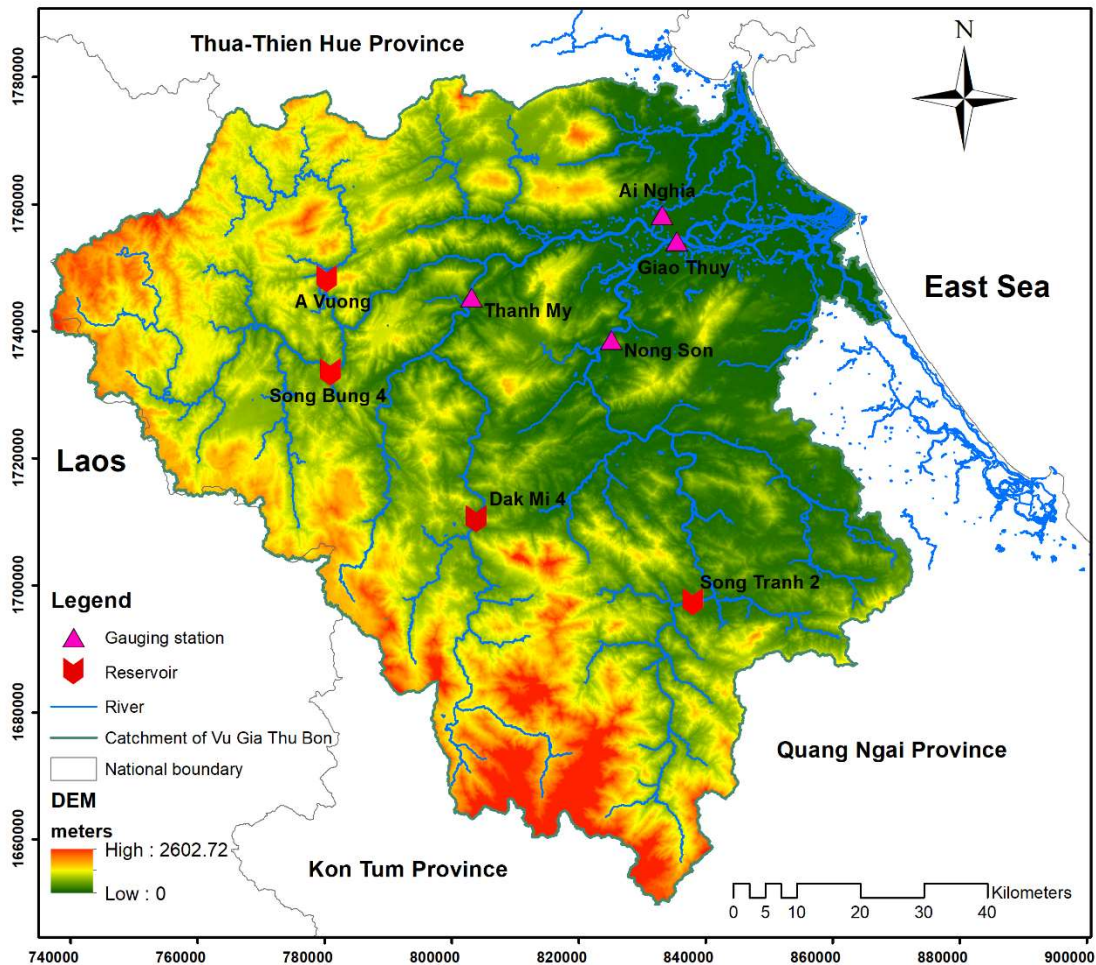


Figure 1.2. Multi-reservoir system in the Vu Gia Thu Bon catchment.

The second component is the hydraulic 1D model (Mike 11) that allows users to perform one-dimensional unsteady flow condition for the Vu Gia Thu Bon catchment. The 1D hydrodynamic model offers a much simpler model setup and faster computational time compared to those of complex hydrodynamic models like 2D or 3D models and the ability to use them in optimization models. This hydraulic model also computes the flow discharges and water levels in the river network considering the physical constraints of the system. The maximum water levels at the downstream control points are necessary to estimate the objective function of the optimization model.

The last component is the optimization procedure based on the Shuffled Complex Evolution (SCE) algorithm interfaces the simulation model to calculate gates levels during the operation of the multi-reservoir system. The SCE algorithm synthesizes the best features of several existing algorithms,

including GA, and the complex shuffling This algorithm is one of the techniques that are robust optimization techniques to find the global optimum solution of complex problems with many functions such as non-convex, non-differentiable and multi extrema functions (Ngo et al., 2007). In the proposed methodology as shown in Fig. 1.1, first chromosomes are randomly generated in the feasible parameter space and entered into the operation reservoir model. After running the hydraulic model, the water surface elevations are obtained and can be used to estimate the objective function. Each decision variable is spillway gate levels in multi-reservoir system and these parameters are generated in the range of parameter space in the SCE optimization model.

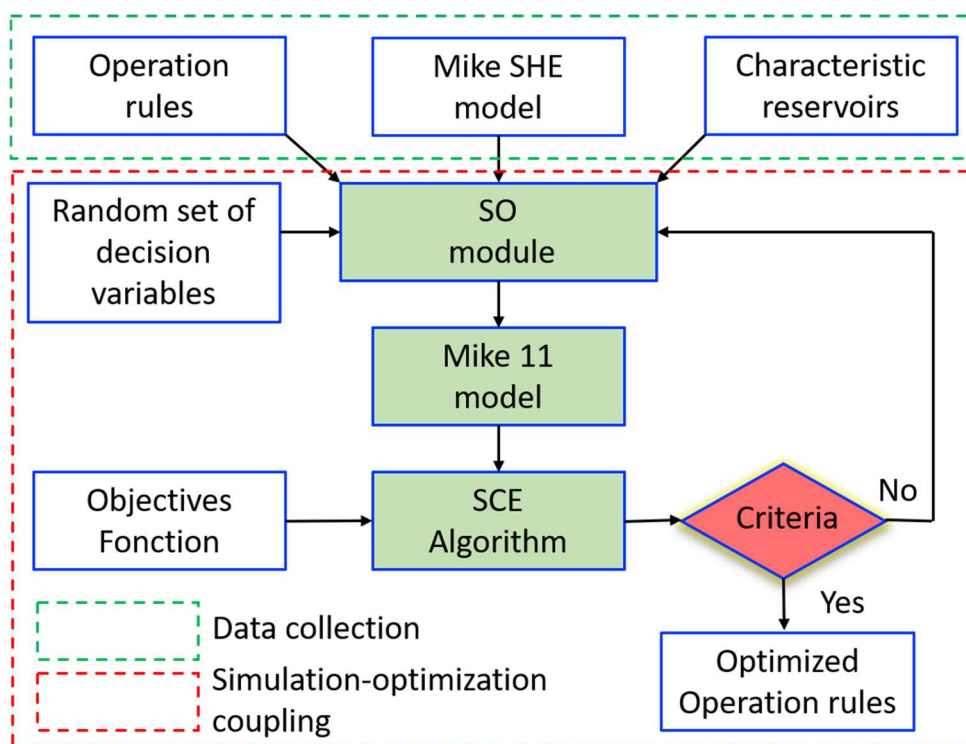


Figure 1.3. Interconnection of the components for optimal operation model of multi-reservoir system in the Vu Gia Thu Bon catchment.

Using this approach, the main objective of the method is to control flood flows and flood levels at various locations (i.e., control points) at the downstream of the Vu Gia Thu Bon catchment. The objective function is minimizing the total flood damages that can be expressed as a function of water surface elevations at the inundation zones - at Ai Nghia and Giao Thuy stations (Fig. 1.2).

Fig. 1.3 describes a brief description of the interfacing of the components for optimal operation of the multi-reservoir system in the Vu Gia Thu Bon catchment. In the first step, the SCE optimization model produces a random

set of decision variables, which are the input of the SO module such as spillway gates levels. Then the operation reservoir module SO computes gates openings and releases of reservoirs in the multi-reservoir system. Once sets of feasible solutions are determined, the releases are entered into the Mike 11 hydraulic model to compute the flow discharges and water levels of the river network considering physical constraints of the system. After that, the optimization model evaluated the objective function based on the selected results from the simulation model. Another set of control variable is then generated through the optimization algorithm. This process is repeated until the stopping criteria are satisfied. These include: the iteration number reaches a certain predefined, or the value of the objective function improves insignificantly over some iterations (i.e., less than 1%).

1.6. Structure of the thesis

Chapter 1 describes the scope of this research, research questions, research objectives and the organization of the thesis.

Chapter 2 provides a detailed introduction of the research area of Vu Gia Thu Bon catchment, the general processes and regulating principles of the reservoir operation for the Vu Gia Thu Bon catchment.

Chapter 3 describes the hydraulic model for river routing, the reservoir system operation model and their application for the multi-reservoir system of the Vu Gia Thu Bon catchment.

Chapter 4 provides a literature review of the application of optimization models for flood control, selection of optimization techniques and a brief overview of the shuffled complex evolution (SCE) algorithm.

Chapter 5 shows the application and results of the optimization-simulation model for the Vu Gia Thu Bon catchment using historical flood events in 2007, 2009 and 2017.

Chapter 6 introduces several approaches in dynamic control Flood Limit Water Level (FLWL) and presents optimal results of the FLWL for parallel reservoir system with multi-objective for the Vu Gia Thu Bon catchment.

Chapter 7 summarizes the main findings and provides conclusions and perspectives regarding the proposed strategy and its operational implementation.

Chapter 2

Vu Gia Thu Bon catchment context and challenges

2.1. Vu Gia Thu Bon catchment overview

The Vu Gia Thu Bon is the biggest river basin in the central region of Vietnam which extending from 14°54'N to 16°13'N and 107°12'E to 108°44'E (Figure 2.1). The catchment borders on the Cu De basin to the north and the Eastern Sea to the east. It shares borders Tra Bong basin to the south, with the Mekong basin to the west. The total catchment area is 10,350 km², which 70% mountainous, and 30% is foothill and plain, located in the major area of Quang Nam province and Danang city as well as small parts of Kontum province.

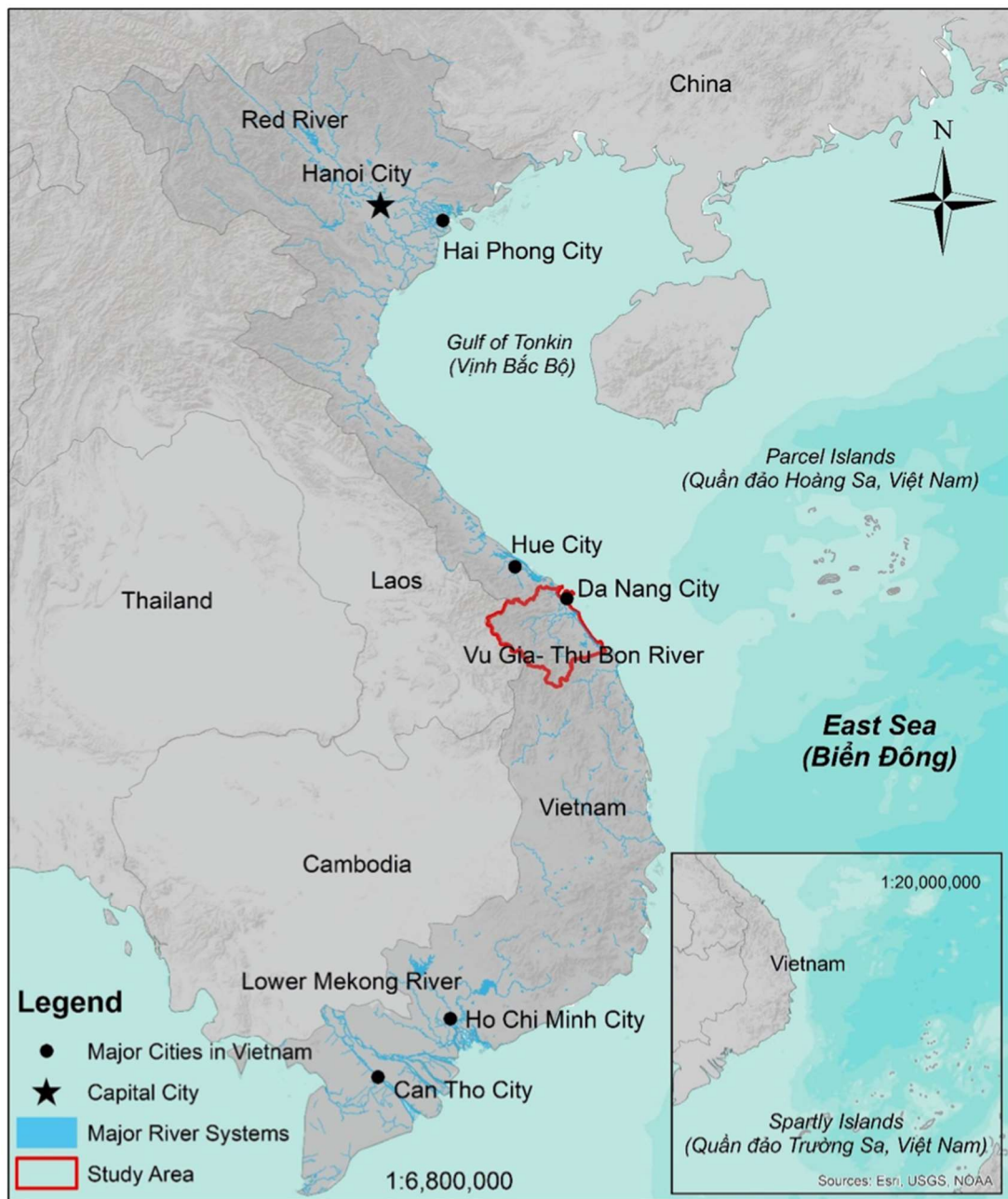


Figure 2.1. Location of the Vu Gia Thu Bon catchment (Tran, 2018).

The catchment’s natural topography divides the area into three major landscapes, the highlands, midlands, and lowlands (Fig. 2.2). The Truong Son mountain distinguishes the topography in the highlands with the highest elevation at over 2000 m and the Komtum mountain with mount Ngoc Linh as the highest mountain at 2598 m. On the one hand, the highland area presents steep sloping topography. The river is short and steep with narrow valleys, steep riverbanks, and many waterfalls and rapid flow. On the other hand, the midlands have lower hills ranging from 200 m to 800 m in comparison to the highlands (Fink et al., 2013), the river beds widen and shallow.

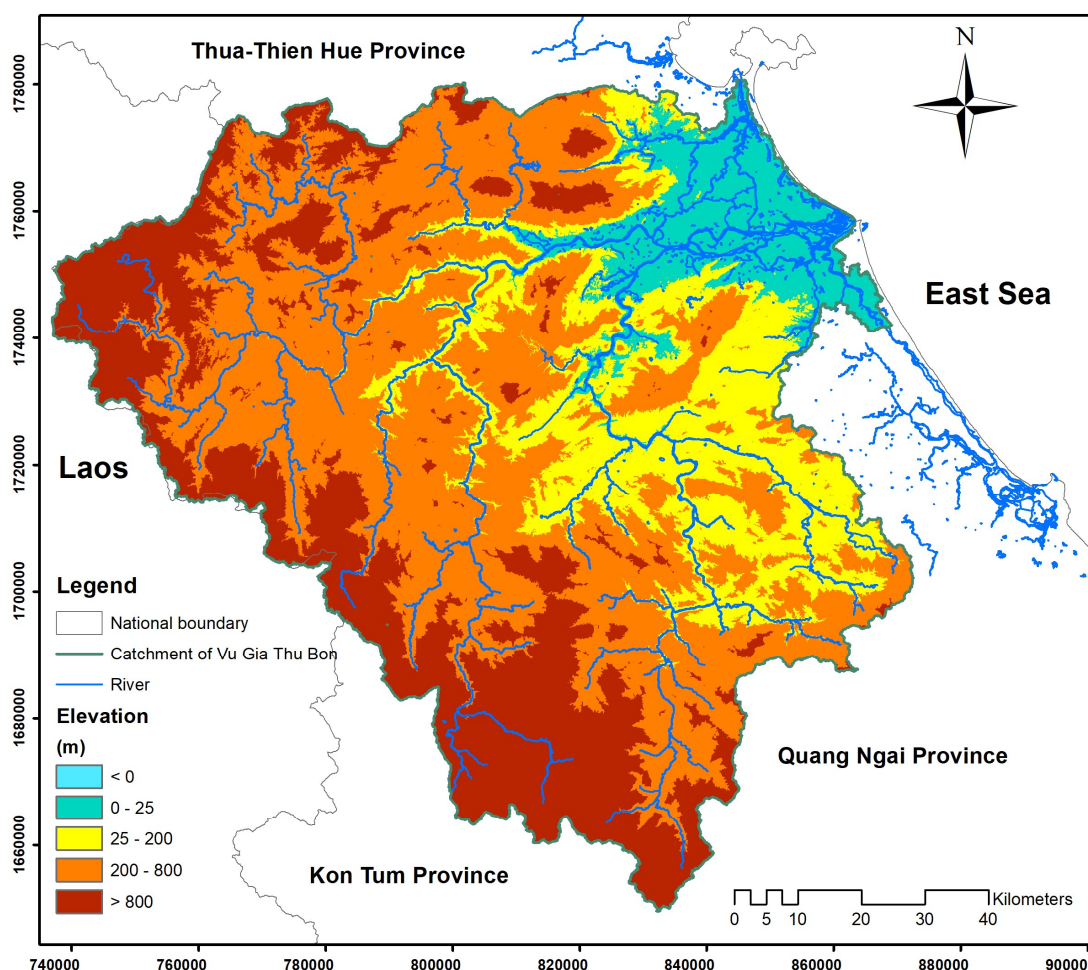


Figure 2.2. Topography of the Vu Gia Thu Bon catchment.

The topographic conditions of this area are advantageous for the hydropower project development, as is the case where a large number of hydropower facilities have been built in recent years ago. Areas below 25 m characterize lowlands; the riverbanks become low, allowing overflows into the fields and villages during the flood season. In the lowlands, the river system has many different connected branches by natural and artificial canals.

The population of Quang Nam province reached 1,501,100 in 2018. In addition, the average annual growth rate of the population in this region was mentioned with 0.5 % between 2006 and 2014 (QSO, 2014). By contrast, the figure in Da Nang city was about five times higher in the same period and in the period of 1995-2005, at 2,6 % and 2,4 %, respectively. (DSO, 2014). In 2019, the total population of Da Nang city increased to 1,134,310 persons with 86.9 % of people in the city living in urban areas compared with 32.2 % in Vietnam (DSO, 2019). The population in the Vu Gia Thu Bon catchment is densely concentrated in the lowlands, particularly in the cities, where the density is over 5,000 persons per km² (Nauditt & Ribbe, 2017).

The economy of the Vu Gia Thu Bon basin is diverse, including a changing primary sector consisting of agriculture, forestry, fishery and handicrafts. The economy of the region is rapidly changing from agriculture-based to industry and service-based, with an annual growth rate of over 10 % in the last decade (Trinh et al., 2017). Most economic centers are located in the lowlands, where posse high population density, crowded industrial and service zones and intensive agricultural activities supporting the dynamic growth.

In the period of 2011-2015, the national electricity consumption on average rose by 10.6 % per year. The electricity demand is predicted to be continued going up by about 8 % annually on average until 2035, corresponding to the additional requirement of 93 GW of power generation capacity during the period (MOIT & DEA, 2017). Hydropower is responsible for about 23.1 % of the nation's electricity generation in 2020 (Government of Vietnam, 2011). Government of Vietnam has identified nine priority river basins for future hydropower development and the Vu Gia Thu Bon catchment is the fourth largest in terms of potential hydroelectric capacity in Vietnam after the Da, Dong Nai, and Se San river systems (ICEM, 2008).

2.2. Hydrological characteristics of the Vu Gia Thu Bon catchment

2.2.1. River network

The Vu Gia Thu Bon river network, originating from the eastern side of the Truong Son mountain range near the border to Laos, at an elevation of 2598 m. Flows through 17 districts of Quang Nam province and Danang city, the network drains to the East Sea at Han estuary in the Danang city and Cua

Dai estuary near Hoi An ancient town. The Vu Gia Thu Bon catchment has two mains rives, the Vu Gia and the Thu Bon rivers (Fig. 2.3).

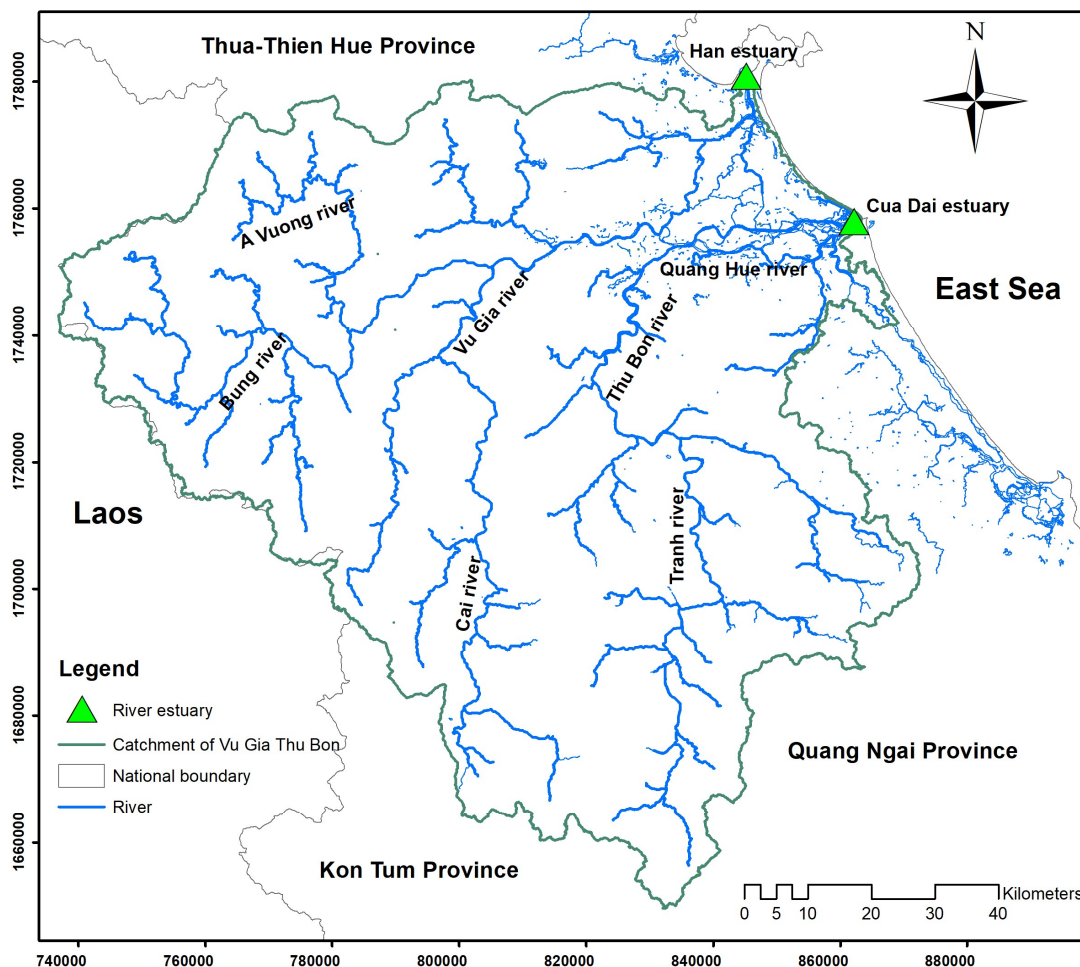


Figure 2.3. River network in the Vu Gia Thu Bon catchment.

The Vu Gia river is 204 km long and originates in the north-western of the Truong Son mountain range. This river has many tributaries, the Dak-Mi (or Cai river) from the south, the Bung river from the west, the A-Vuong river from the northwestern and the Con river. From upstream to Ai Nghia, the Vu Gia river comprises an area of 5180 km².

The Thu Bon river is 152 km long and originates at the high peaks of Ngoc Linh mountain. The catchment area is 3825 km² to Giao Thuy. Analogous to the Vu Gia river, the Thu Bon river has tributaries such as Tranh river, Khang river, and Truong river.

There is a link between the two rivers through the Quang Hue river in the downstream part of the catchment. In the flood season, this river transfers flow from the Vu Gia river to the Thu Bon river. Approximately 16 km from the Quang Hue river to downstream, the Vinh Dien river transports water

from the Thu Bon river and returns to the Vu Gia river in flood season. In addition to the flow exchange, additional water from other branches is also supplied to the mainstreams such as Ly Ly river to Thu Bon river; meanwhile, the Tuy Loan river contributes to the flow of the Vu Gia river.

2.2.2. Rainfall and runoff

Based in the south of the Thua Thien-Hue province and east of Laos, the climate in the Vu Gia Thu Bon catchment shows the typical weather in the south-central Vietnamese (Fink et al., 2013). It has relatively warm winters, hot summers, and a strong rainy season caused typhoons, from September to December.

Precipitations over the area are spatially variable. The average annual precipitation in the upland areas of the catchment is approximately 3000-4000 mm. This is roughly 50-100% higher than annual rainfall in coastal areas (about 2000 mm per year). It should be noted that this area is steep with narrow bends of the river. So most of the rainfall here is not held upstream but flows downstream and into the estuaries. Precipitation is also a temporary variable in the catchment. The rainy season begins in September and lasts until December, while the dry season is from January till August. About 65-80% of the amount accounts for the rainy season (Souvignet et al., 2014).

In terms of flow, a significant difference exists between the flood season and the dry season. The annual average of discharge of the Vu Gia river is about 120 m³/s, while the largest flow recorded about 7230 m³/s on 29th September 2009 at Thanh My gauging station. Similarly, the annual mean discharge of the Thu Bon river is about 270 m³/s (Vu et al., 2017), while the largest flow recorded about 10,600 m³/s and occurred in successive years, 1998 (December) and 1999 (November) at Nong Son gauging station.

In the Vu Gia Thu Bon catchment, there are only two stations (i.e., Nong Son and Thanh My stations) for measuring discharges. These stations are located at the middle of these branches (Fig. 2.4). Hence, the inflows must be estimated based on a rainfall-runoff model. Vo (2015) has been successfully built a deterministic distributed hydrological model based on Mike SHE model and the model efficiency is likewise confirmed by the capacity to predict extreme peak flow and baseflow. Therefore, reservoir inflow series are extracted from the hydrology Mike SHE model (see the Appendix).

2.3. Floods and flood damages

2.3.1. Floods

The major causes of floods in the Vu Gia Thu Bon catchment related to intense rainfall events are:

- Direct influence by a storm or tropical depression hitting the watershed or moving along its seashore;
- Intense Northeastern monsoons head towards the catchment, combined with the circulation of a storm or tropical; this complicated weather causes the heavy rain;
- When the tropical depression zone appears in the South of the East Sea, simultaneously, in the North, a monsoon or Northeast wind is moving towards the South.

The collected data shows that out of 174 storms and tropical depressions taking place in the East Sea between 1997 and 2009 which 26 storms and 12 tropical depression affected directly to the Vu Gia Thu Bon catchment (Vo et al., 2011).

The water level is observed at nine stations along two main branches of river system, Vu Gia and Thu Bon rivers (Fig. 2.4). The annual maximum water levels observed at various gauging stations downstream are shown in Fig. 2.5. The data is supported by the Mid-Central Region Centre for Hydro-meteorological Forecasting.

During 22 years (from 1995 to 2016), there were ten years that maximum water level reaches above warning level 3. The maximum level recorded occurred in 2007 and 2009. In recent years, flood water often rises quickly and more unpredictably, which becomes a growing concern of local people about increasing flood risks in flooding season.

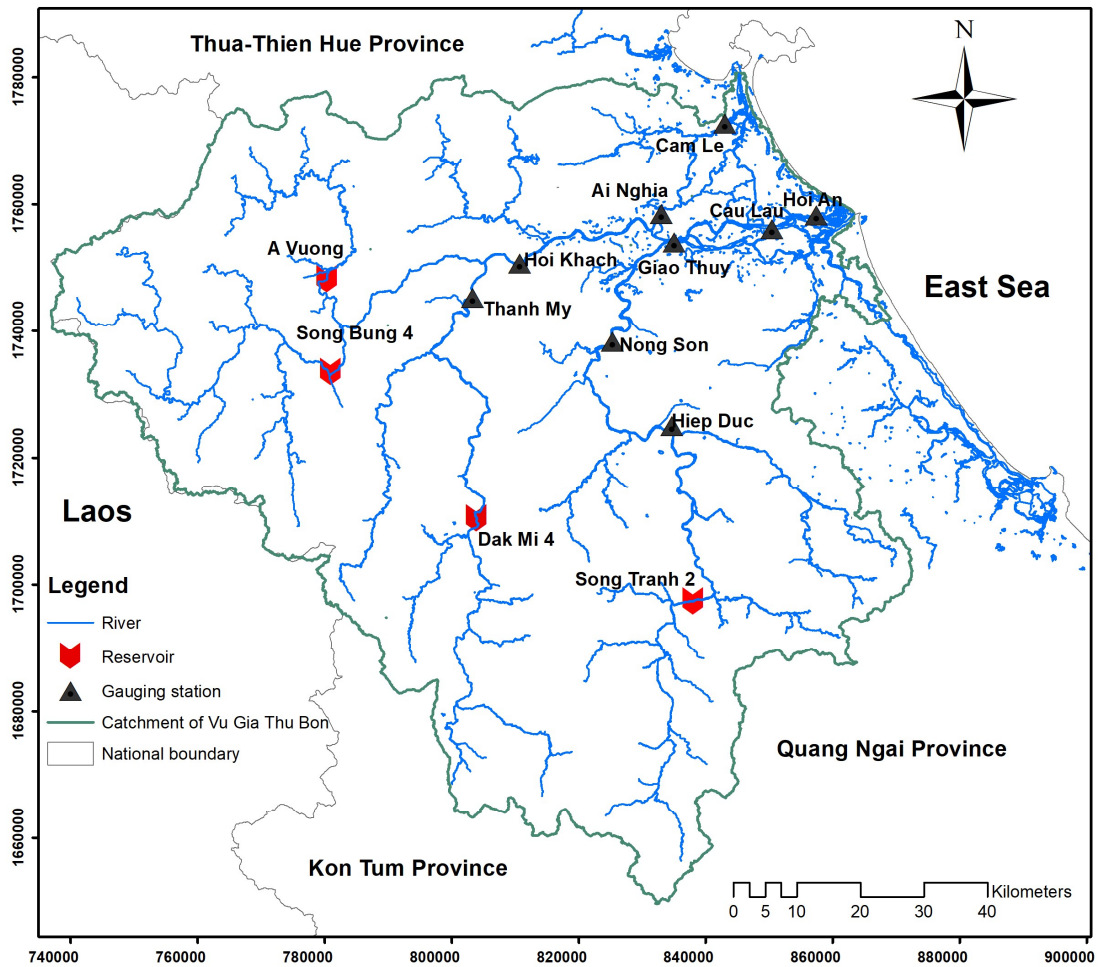


Figure 2.4. Gauging stations and four major reservoirs in the Vu Gia Thu Bon catchment.

There is a significant change in flood characteristics in recent years. Flood flow is stronger, the water level is higher, water recedes likely slower, and at the time, flood recession period occupied a longer time compared to the previous year from 2000. Previously, floods had risen slowly and receded quickly; local people had been able to forecast the intensity, hit time, and duration, which helped them prepare for the event.

The rainy season usually begins from September and ends in December, during which time heavy rain often happens and leads to an increase in flood risk in the catchment. Heavy rain and floods continued beyond the end of the rainy seasons, and irrigation and hydropower dams in affected areas reached maximum capacity, resulting in controlled water discharges that further intensified the flood impact downstream of the reservoirs.

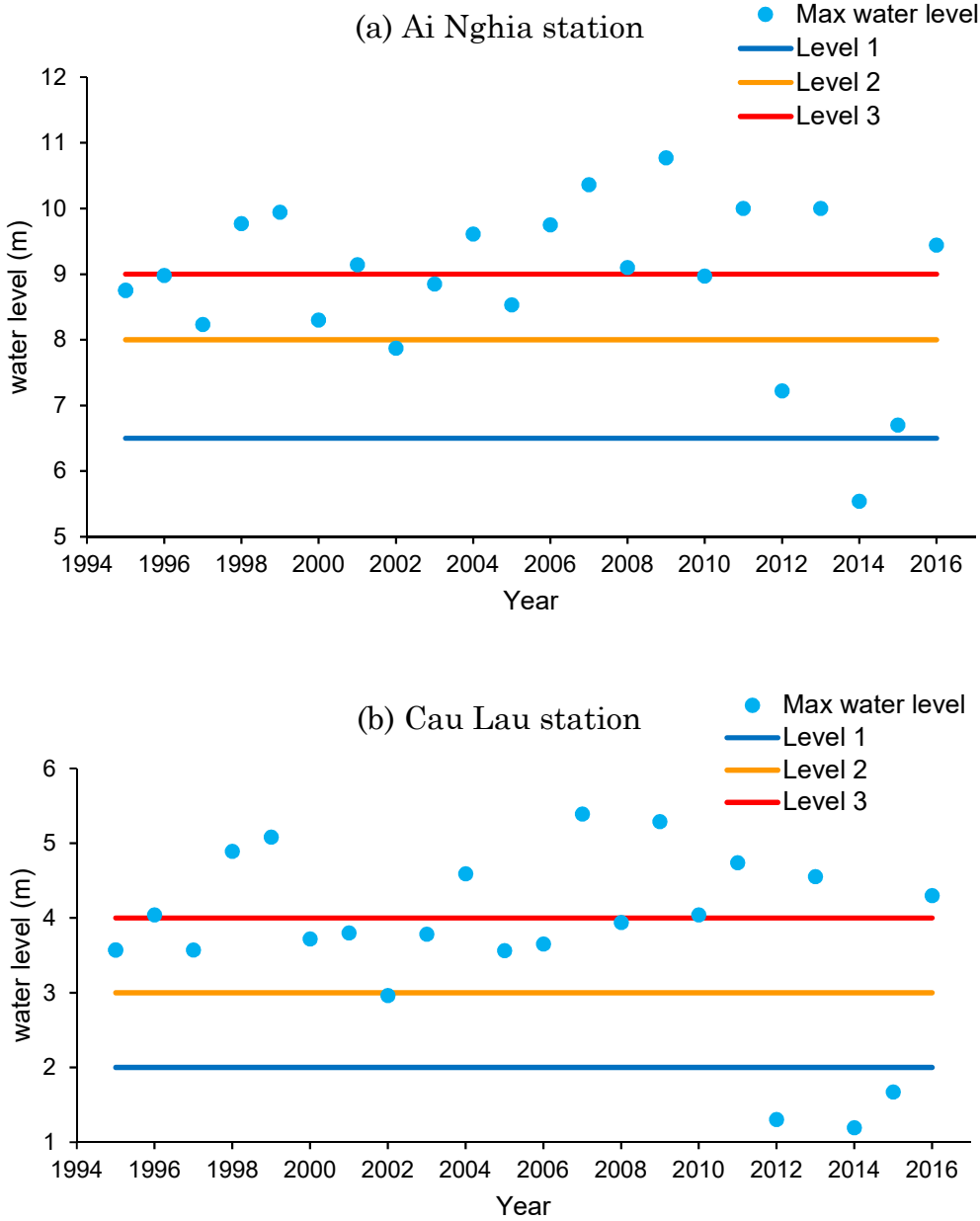


Figure 2.5. The annual maximum water level at downstream: (a) Ai Nghia station and (b) Cau Lau station

2.3.2. Flood damages

Vietnam is among those at the highest risk from natural disasters, with floods, droughts, severe storms, landslides, and forest fires having substantial economic and human impacts annually (Nga et al., 2015). Over the last two decades, natural disasters in Vietnam have caused more than 13,000 deaths and property damage over US\$ 6.4 billions (World Bank, 2010). Annually, disasters cost Vietnam an average of up to 1.5 percent of its gross domestic product (GDP).

In the Vu Gia Thu Bon catchment, the flood is the most destructive hazard causing major damage in properties and livelihood among the other natural disaster (Satriagasa et al., 2014). The loss of human lives and property losses caused by floods in the period 1997-2009 was provided by Quang Nam provincial steering committee for storm and flood control, given on Tab. 2.1.

Although hydropower reservoirs in the Vu Gia Thu Bon catchment are not designed to provide complete protection against all possible floods, efficient operation of these reservoirs can reduce flood levels and prevent major flood disasters.

Table 2.1. Flood damages in Quang Nam province in the period 1997-2009.

Year	Death (Persons)	Missing (Persons)	Injured (Persons)	Damages costs	
				(Billions VND)	(Millions US\$*)
1997	33	0	0	100	5.4
1998	54	1	36	390	21.1
1999	118	0	339	758	41.1
2000	13	0	0	139	7.5
2001	13	1	9	76	4.1
2002	0	0	0	2	0.1
2003	32	2	5	91	4.9
2004	19	23	13	156	8.4
2005	12	5	24	110	6.0
2006	176	1	562	1901	103.0
2007	47	0	339	2000	108.3
2008	33	0	3	155	8.4
2009	52	0	220	3700	200.4

* US Dollar exchange rates for 31 December 2009: 1 USD to VND = 18,465.

2.4. Hydropower reservoirs system

2.4.1. Hydropower projects development

Over the last two decades, Vietnam is a dynamically developing economy with a relatively high growth rate. According to the World Bank (WB, 2016), the country is one of the most dynamic emerging countries in East Asia. The energy sector plays an important part in fostering the socio-economic development of the country. Vietnam has a dense river system with a lot of large river basins. As a cheap and available source of energy, hydropower is

a key component of the national energy mix (Nguyen-Tien et al., 2018). According to estimations, the total technical hydropower capacity is about 26,000 MW, with a total produced about 100 billion kWh. Progress of total primary energy supply grew at 4.7% between 2007-2017, in which hydropower experienced the highest growth at 14.5% (EREA & DEA, 2019).

After the Da, Dong Nai, and Se San river systems, the Vu Gia Thu Bon river basin ranks fourth in potential hydroelectric capacity in Vietnam (ICEM, 2008). This basin plays a significant role in terms of social and economic points of view for the central region of the country. According to the Seventh National Power Development Plan, the Government of Vietnam has listed eight large-medium hydropower projects (i.e., more than 30 MW) for the Vu Gia Thu Bon catchment with total power capacity is approximately 1,100 MW (Fig. 2.6). The major characteristics in terms of 8 hydropower projects, are listed in Tab. 2.2.

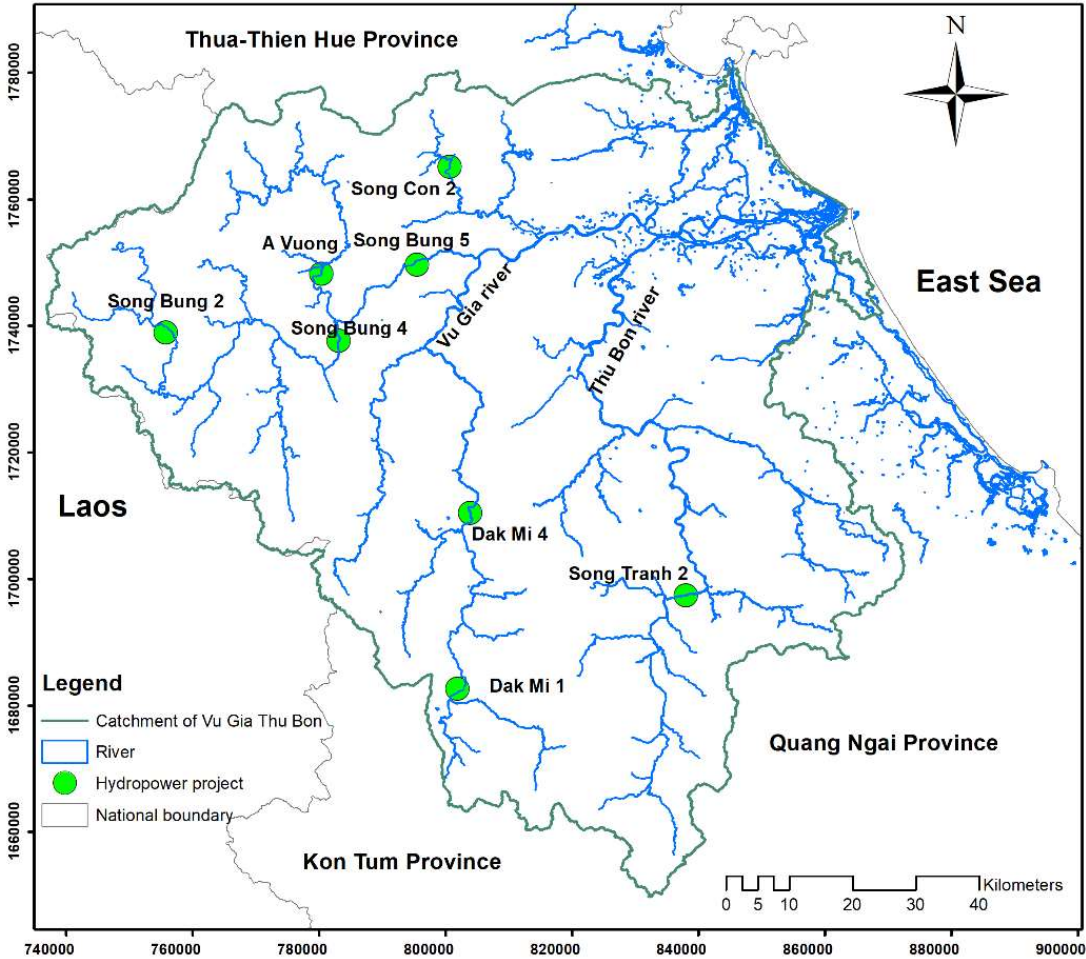


Figure 2.6. Hydropower projects in the Vu Gia Thu Bon catchment.

Table 2. 2. Characteristics of hydropower project in the Vu Gia Thu Bon catchment

		Song Bung 2	Song Bung 4	Song Bung 5	Dak Mi 2	Dak Mi 4	Song Con 2	A Vuong	Song Tranh 2
Basin area at dam	Km ²	334	1448	2369	445	1125	250	682	1100
Mean annual flow	m ³ /s	18.7	73.7	118	28.4	67.8	13	39.8	114
Normal water level	m	605	222.5	60	630	258	278	380	175
Dead water level	m	565	205	58.5	624	240	276	340	140
Active storage	10 ⁶ m ³	73.9	233.99	2.45	0.692	158.26	0.7	266.48	521.1
Installed capacity	MW	100	156	57	98	148	50	210	190
Turbine discharge	m ³ /s	34.5	166	239.24	44.36	128	25	78.4	245
Dam height	m	96	114	41.5	30	90	16.5	80	96

2.4.2. Reservoirs system general description

The steep slope of mountainous topography greatly limits the capacity of reservoirs in the central region of Vietnam in general and of reservoirs in the Vu Gia Thu Bon catchment in specific. Most projects are using dams for the impoundment of the river and using potential heads of the rivers to build a system of hydropower reservoirs cascade. All of these large hydropower reservoirs in the Vu Gia Thu Bon catchment are used a guiding channel for transferring water from the reservoir to the hydropower plant. Since 2015, eight large-medium sized dam projects have been constructed on the mainstream of the river basin. However, there are only four hydropower reservoirs with capacity flood control, including A Vuong, Dak Mi 4, Song Bung 4 and Song Tranh 2 (Fig. 2.4). These four reservoirs play the most important role in flood control in the Vu Gia Thu Bon catchment.



(a) A Vuong reservoir



(b) Dak Mi 4 reservoir



(c) Song Bung 4 reservoir



(d) Song Tranh 2 reservoir

Figure 2.7. The four major reservoirs in the Vu Gia Thu Bon catchment.

A Vuong reservoir (Fig. 2.7a) is located on the A Vuong river within Hien district in Quang Nam province. A Vuong river is a tributary of the Bung river, which originated northwest mountain with elevation 1400m. The

catchment area is 682 square kilometers downstream to the A Vuong dam site. The dam height will be 80 meters, and it will create a reservoir with an area of 9.09 km² at the normal water level. The hydropower plant has 210 MW with two units. The design head is 300 m, and the project has to mean an annual energy potential of 815 million kWh.

Dak Mi 4 reservoir (Fig. 2.7b) is to built and operates a hydropower plant with an accumulation reservoir in the Phuoc Son district in Quang Nam province. This project produces 752 million kWh per year with an installed capacity of 190 MW. The project contains an upper cascade, Dak Mi 4a, with a capacity of 148 MW and a lower cascade, Dak Mi 4b, with a capacity of 42 MW. The dam built in the Cai river, one of the major tributaries of the Vu Gia river. The upper dam has a reservoir area at the normal water level of 10.44 km². This project started into operation in 2012.

Song Bung 4 reservoir (Fig. 2.7c) is located on the Bung river, a tributary of the Vu Gia river, within Nam Giang district of Quang Nam province. The dam sites on the boundary of Zuoih and Ta B'Hing commune and the reservoir inundate part of these two communes. Water from the reservoir is diverted to a tunnel of length 3.2 km and penstock to a power station located about 5 km downstream of the dam. The hydropower facility has 156 MW and has a mean annual energy potential of 222 million kWh.

Song Tranh 2 (Fig. 2.7d) reservoir is a large reservoir of 730 million m³ volume, making it one of the most significant reservoirs in the central of Vietnam. The hydropower project is located on the mainstream of the Tranh river, which is a part of the Thu Bon river, with a capacity of 190 MW, having annual energy electricity of 679 million kWh. This project was constructed in 2006 and was completed in 2011.

Mitigation of flood damages is the first priority for operational reservoirs system during flood season (from September to December) in the Vu Gia Thu Bon catchment.

2.4.3. Spillway gates description

The four reservoirs have gated spillway to control reservoir outflow during flood events that protect the safety of their dams and the headwork. The spillway gates sized to safely pass floods equal to or less than the probable maximum flood. The operation of the spillway gates depends on the state of the reservoir, reservoir inflow, and operation strategy. Type of the spillway

gates of four reservoirs in the Vu Gia Thu Bon catchment is the radial gate (Fig. 2.8). Main characteristics of four spillway gates givens in Tab. 2.3.

Table 2.3. Characteristics of spillway gates.

	A Vuong	Dak Mi 4	Song Bung 4	Song Tranh 2
Crest level of weir (m)	363	242.5	210.5	161
Number of gates	3	5	6	6
High of gate (m)	17.5	16	12	14
Wide of gate (m)	14	14	12	14
Capacity of passing* (m ³ /s)	5720	8584	6363	9035

* maximum discharge capacity corresponding the normal water level.



Figure 2.8. A radial gate of A Vuong spillway.

2.5. Reservoir operational regulation

2.5.1. Basic terminology

This research work focuses on the optimal multi-reservoir operation for flood control that essentially means determining operation strategies, defining a schedule of spillway gates to fulfill objectives best. An operational regulation

is a set of rules for determining the amount of water released during flood season from reservoir systems under various conditions. Regulation of the multi-reservoir in the Vu Gia Thu Bon catchment based on reservoir inflows, downstream conditions, and storage levels. Different storage levels include (Fig. 2.9):

- Dead water level;
- Normal water level;
- Design flood level;
- Exceptional (check) flood level;
- Flood limit water level.

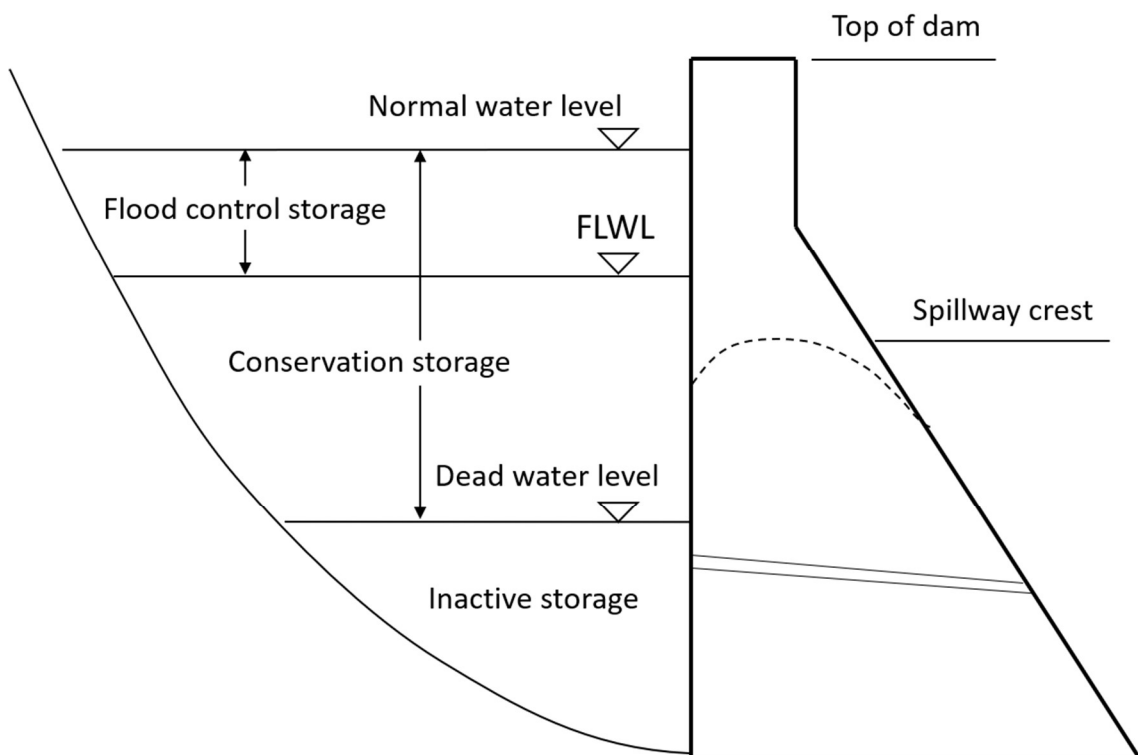


Figure 2.9. Design characteristics of a reservoir.

Flood Limit Water Level (FLWL) is an operational level designed to reserve adequate flood control capacity. The general rule about FLWL is that the storage level of the reservoir is not allowed to exceed FLWL during the flood season to offer adequate storage for flood prevention. According to the operational regulation of the multiple reservoirs system in the Vu Gia Thu Bon catchment, the reservoir levels can fluctuate between the upper limit and the lower limit of the dynamic control of FLWL. Different storage levels and flood control characteristics of four major reservoirs in the Vu Gia Thu Bon catchment are listed in Tab. 2.4.

Table 2.4. Reservoir characteristics in terms of flood control.

Reservoir	A Vuong	Dak Mi 4	Song Bung 4	Song Tranh 2
Exceptional (check) flood level (m)	382.2	260.33	228.11	178.51
Normal water level (m)	380	258	222.5	175
Dead water level (m)	340	240	205	140
Upper bound of FLWL (m)	376	255	217.5	172
Lower bound of FLWL (m)	370	251	214.3	165
Reservoir storage (10^6 m ³)	343.55	312.38	510.8	729.2
Active storage (10^6 m ³)	266.48	158.26	233.99	521.1
Flood control storage (10^6 m ³)	35.14	31.07	75.3	61.45

2.5.2. Multi-reservoir operational regulation

Regulation for flood control

Hydropower reservoir development and management are achieved within a complex system of organizations, laws and encompassed also traditions. In addition to the agency that owns and operates the reservoir, numerous other public agencies, officials, project beneficiaries, interest groups, and concerned citizens play significant roles in determining operating policies (Wurbs, 1991). According to the decision of the Government of Vietnam, the Steering Committee for Disaster Prevention, Search and Rescue of Quang Nam province takes responsibility for the operation of the reservoir systems in the Vu Gia Thu Bon catchment during the flood season.

The four major reservoirs in the Vu Gia Thu Bon catchment include A Vuong, Dak Mi 4, song Tranh 2, and song Bung 4, which were operated in the years 2009, 2011, 2012 and 2014, respectively. Following the operational regulation, the flood season from 1st September to 15th December of every year. In the flood season, reservoirs system is operated in the following order of priority:

- Strictly ensuring the safety of the dams: the maximum water levels do not exceed the exceptional (check) flood level for all floods that provide by Tab. 2.4;
- Taking part in reducing downstream floods;
- Ensuring efficiency in hydropower generation.

The storage levels in the reservoirs should be kept below the upper bound of flood limit water level (Fig. 2.10) to provide sufficient storage for flood mitigation.

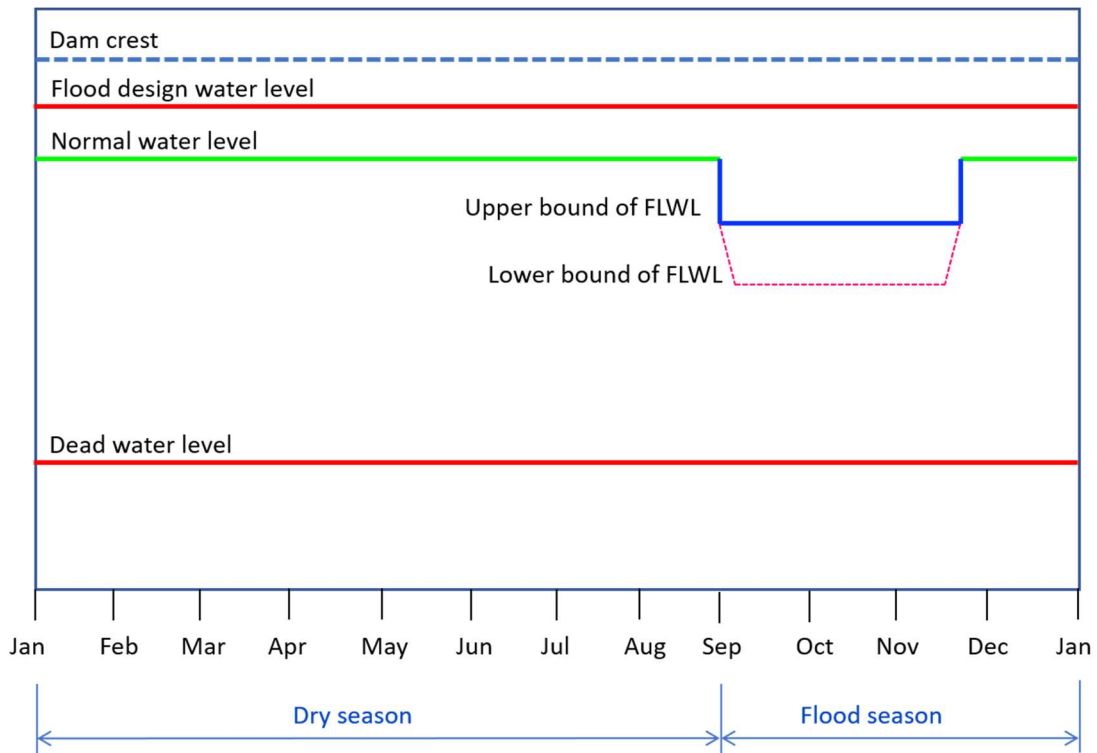


Figure 2.10. Time-varying reservoir stage for the rule curves of the reservoir.

Table 2.5. Varying downstream warning water level for the operation rules of the reservoirs system

	Hoi Khach	Ai Nghia	Nong Son	Cau Lau
Level 1 (m)	14.5	6.5	11.0	2.0
Level 2 (m)	15.5	8.0	13.0	3.0
Level 3 (m)	16.5	9.0	15.0	4.0
H ₃ (m)	16.0	8.5	14.0	3.5

The regulation of operation for flood mitigation consider the key parameters follow in the flood season:

- The reservoir stages in reservoirs system;
- The water level at the downstream control point: operation of A Vuong, Dak Mi 4 and Song Bung 4 reservoirs define by the water level at Hoi Khach and Ai Nghia stations while the operation of Song Tranh 2 defines by the water level ay Nong Son and Cau Lau stations (Tab. 2.5);

- Hydrological forecast information: 24-48h forecasts of the inflows information.

Operational regulation of multiple reservoirs in the Vu Gia Thu Bon catchment defines three regulation periods to reduce floods hitting lowlands during flood season as follows:

The procedure of pre-release: the objective of this procedure to release a proportion of the usable volume of reservoirs before flooding occurs and allows the reservoirs to reduce the water level. The release discharge should be greater than the inflow discharge to reduce the reservoir stage to lower bound of FLWL with conditions following:

- The reservoir stage is above the lower bound of FLWL;
- The water level at the downstream control points is lower than the water level 2 (Tab. 2.5);
- Vietnam Meteorological and Hydrological Administration (VMHA) forecast heavy rainfall, flood in the next 24-48h or reservoir inflows are greater than the discharge value defined by Tab. 2.6:

Table 2.6. Defined discharge for the procedure of pre-release.

	A Vuong	Dak Mi 4	Song Bung 4	Song Tranh 2
Discharge (m ³ /s)	450	550	550	900

During the pre-release procedure is effectuated; if the water level at the downstream control points is greater than the water level 2, the release discharge should be equal the inflow discharge to maintain the reservoir stages.

Procedure during a flood event: the objective of this procedure is to reduce major downstream floods in the Vu Gia Thu Bon catchment. The reservoirs have to store water to reduce the flows to downstream with one of the conditions following:

- The water level at the downstream control points is greater than the water level of H₃ (Tab. 2.5);
- Reservoir inflows are greater than the discharge value defined by Tab. 2.7:

Table 2.7. Defined discharge for the procedure of flood control

	A Vuong	Dak Mi 4	Song Bung 4	Song Tranh 2
Discharge (m ³ /s)	600	700	700	1500

The release discharge should be less than the reservoir inflow so that the reservoir stage will be rising. When the reservoir stage reaches the normal water level, this procedure will be changed to the procedure for the safe protection of the dam.

Procedure for safe protection of the dams: in the case where the water level at downstream control points is greater than the value of H_3 (Tab. 2.5), the reservoir stages reach the normal water level and there are floodings in the catchment that are increasing, the procedures are changed to dam protection. Based on the reservoir stages and reservoir inflow, a step-by-step opening the spillway gates to control release discharge equal to reservoir inflow.

Procedure of post-flood: the post-flood procedure is a method for ensuring the reservoir stage is descending to the upper bound of FLWL (Tab. 2.4) for the next flood event. The total discharge release greater than the reservoir inflow during the period from 24 to 72 hours with conditions following:

- The water level at the downstream control points is lower than the warning water level 1 (Tab. 2.5).

For a detailed description of the regulation, refer to Decision 1537 of (Government of Vietnam, 2015).

Regulation for hydropower generation

The reservoir in the Vu Gia Thu Bon catchment operates for seasonal regulation of the flows. Hydrological characteristic includes the wet season or the flood season (September to December) and the dry season (January to August).

Table 2.8. Typical monthly stage in the Song Bung 4 reservoir.

	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug
Reservoir stage (m)	205	213	219	222.5	221	219	216	213	209	205	200	195

The reservoir stages will be filled up to the normal water level during the wet season and be drawn down to the dead water level at the end of the dry season. An example of the typical variation of reservoir stages during the year of the Song Bung 4 hydropower reservoir, shows in Tab. 2.8.

Rule curves generally maintain regulation for hydropower generation. The operating rule curves enable visual expression of the operating strategy (Chang et al., 2005) and are derived from different operating guidelines curves. As illustrated in Fig. 2.11, the operation guide curve was defined with the fewest possible variables. In this study, hydropower generation is the second objective of the reservoir system in the flood season. Therefore, the operation to get as much hydropower as possible within the constraints of the flood reduction strategies. It consists of three zones, as shown in Fig. 2.11 to define how much water is supplied in the model for hydropower generation.

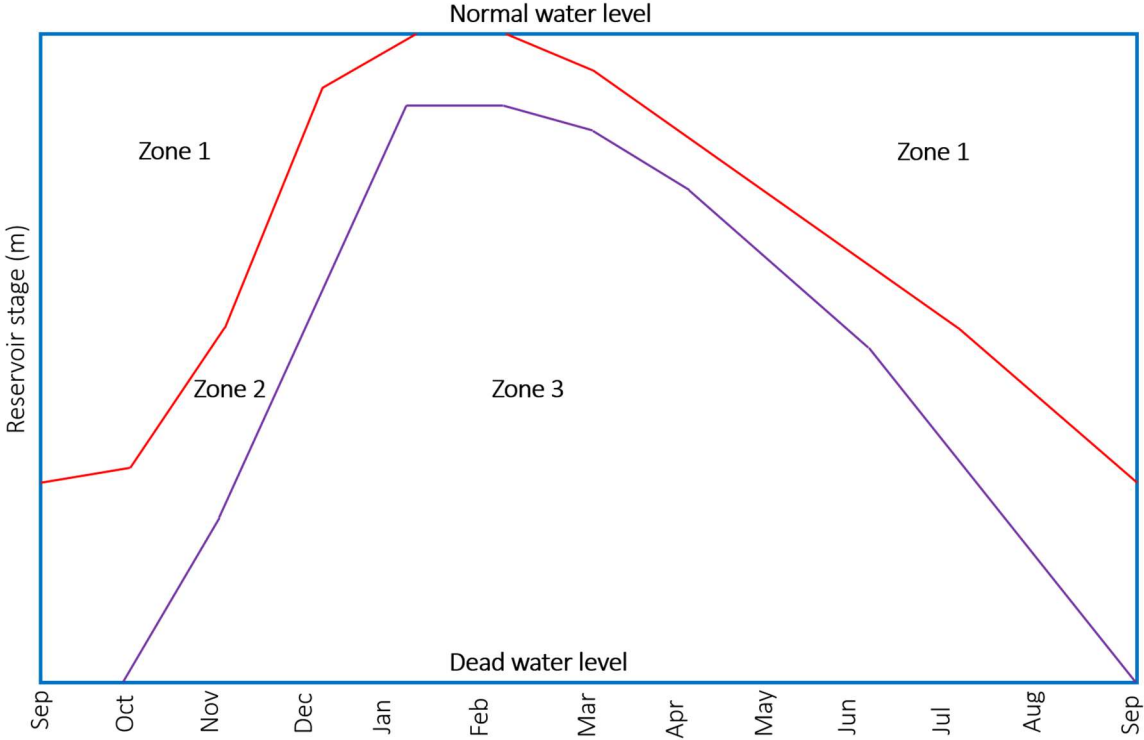


Figure 2.11. Operating rule curves of A Vuong reservoir.

Chapter 3

Modeling and simulation of the river-reservoir system

The objective of this research is the optimal operation of the multi-reservoir system in the Vu Gia Thu Bon catchment for flood mitigation. These are two steps. The first step is to simulate the operation of the reservoir system using a 1D hydraulic model (Mike 11). The second step is the coupling simulation model with the optimization model to the optimal operation of spillway gates for flood mitigation during the flood event. This chapter describes the operation reservoir simulation model and application in the Vu Gia Thu Bon multi-reservoir system.

3.1. River-reservoir system modeling problem

Construction of reservoirs is important to use water resources for many beneficial purposes such as public and industrial water supply, irrigation, hydroelectric power generation, and navigation. The usefulness of reservoirs is a more likely emphasis for a river basin's which extremely variable water flow. Dams and systems of appurtenance also control rivers to mitigate flood damage.

The simulation of river-reservoir system modeling is very challenging. The problem of operation reservoir system may result from involves many decision variables complicated, uncertainties, and multiple objectives such as inflows, return flows, storages, water supply demands, flood control (Oliveira & Loucks, 1997; Rani & Moreira, 2010). In addition, the conflicting objectives in the multi-objective reservoir system present major challenges for operators when making operational decisions (Ngo et al., 2008).

Operation strategy is a set of guidelines for estimation of the amount of water to be stored or releasing flow at several reservoirs in the system under different conditions. Most of the reservoir systems are still being operated by a constant rule curve, and these curves are usually presented in the form of graphs or tubular (William, 1985). The rule curve guides the discharge release of the reservoir according to the storage level, reservoir inflow, and time of the year.

In order to evaluate and analyze the river reservoir system, a framework for representing operating rules or decision criteria must be integrated into the simulation model. The fact that operation reservoir system models contain various mechanisms for making release discharge decisions within the schema of user-specified operating regulations. Operation of reservoir system can be categorized as (Wurbs, 2005):

- Operations during normal hydrologic conditions from the perspective of optimizing the present day-to-day, seasonal, or year-to-year use of the reservoir system.
- Operations during normal hydrologic conditions from the perspective of maintaining storage capabilities for responding to infrequent hydrologic extremes expected to occur at unknown times in the future.
- Operations during extreme hydrological situation: operations during flood events or low flow or drought conditions.

A wide variety of operational strategies are currently used in reservoir projects around Vietnam and around the world. Hydropower reservoirs in the Vu Gia

Thu Bon catchment are also subjected to the aforementioned operations conflict objectives, which are hydropower generation and flood mitigation. A simulation model will support investigating each operational strategy through this research. Before developing the model that can used for analyzing and evaluating obtained simulation results, the fundamental reservoir equation is introduced below in details.

3.1.1. Reservoir regulation equation

A major component for the model of a river reservoir system is the reservoir operation model. The reservoir operation model is based on the principle of conservation of mass for a control volume. When inflows to a reservoir are computed from available stream flows, they have been estimated as the change in storage plus measured outflow, as shown in equation follows:

$$\text{Inflow}(t) = \text{Storage}(t) - \text{Storage}(t-1) + \text{Outflow}(t) \quad (3.1)$$

The conservation of mass for a reservoir can be written in mathematical form as:

$$\frac{d}{dt}R = \sum in - \sum out \quad (3.2)$$

where:

$\frac{d}{dt}R$ is the accumulation of mass in the reservoir;

$\sum in$ is the total inflow of mass through the control structure;

$\sum out$ is the total outflow of mass through the control structure.

The conservation of mass can be changed to display the volume flow rate instead of the mass flow rate, as shown below:

$$\frac{d}{dt}V = \sum Q_{in} - \sum Q_{out} \quad (3.3)$$

where:

$\frac{d}{dt}V$ is the volume changes in the reservoir;

$\sum Q_{in}$ is the total volumetric inflow through the control structure;

$\sum Q_{out}$ is the total volumetric outflow through the control structure.

Equation (3.3) is the basis of the reservoir operation model. The schematic of a reservoir with components of inflow, outflows, and reservoir storage shows in Fig. 3.1.

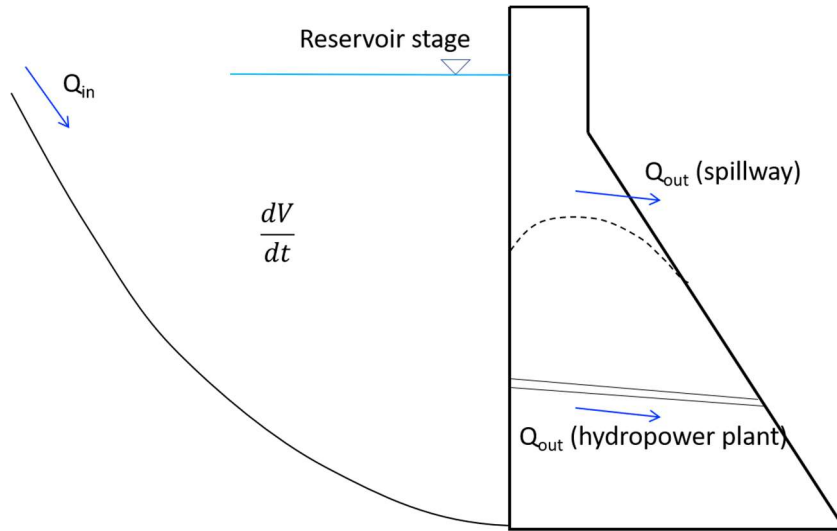


Figure 3.1. The schematic of reservoir operation.

In general, the computation is performed by converting the above equation (Eq. 3.3) into a finite difference, and the solution is applied small time step, of a length of time Δt , and input discharge sequence for a given period time as following:

$$V_{t+1} - V_t = \left(\frac{Q_t^{in} + Q_{t+1}^{in}}{2} \right) \Delta t - \left(\frac{Q_t^{out} + Q_{t+1}^{out}}{2} \right) \Delta t \quad (3.4)$$

In the reservoir operation model, various strategies determine the discharge released from the reservoir using spillway gates and gates for turbines. Definition operating strategies for structures is explained in these next sections.

3.1.2. Simulation of operating multi-reservoir system: a review

Modeling river and multi-reservoir system research have developed in recent years to include a wide range of river basin management hydrology, physical infrastructure such as outlet hydropower plant, spillway gate. Also, many research literature on water resources addressing the problem of developing and testing reservoir operating rules for flood control includes the use of models for mathematical simulation and optimization. William (1985), Wurbs (1991) and Rani & Moreira (2010) offered comprehensive lists of references and in-depth analyses of the use of these models for the different study of the reservoir system. Adeyemo (2011) provides a detailed discussion and formulation of reservoir simulation models and optimization techniques using evolutionary algorithms.

In general, the simulation model of the river and reservoir system aims to compute water level in the reservoir, discharge release from the reservoir, water

level, and discharge of the river network. These parameters are calculated based on a given set of hydrological inputs, system demands, and operating regulation. While a pure simulation model does not include an algorithm for specifically determining an optimal solution, a typical approach is to make multiple runs with alternative operational rules in the search for an optimal or near-optimal solution (Wurbs, 1991). For many years, water resources engineering agencies responsible for the preparation, construction, and operation of reservoir projects have regularly used simulation models (Loucks, 2017). In recent years, ineffective reservoir operating strategies have been analyzed. In a systematic setting, the implications of individual decisions and contradictory cost/benefit calculations have been also investigated (Labadie, 2004).

Although many reservoir system operation simulations models have been developed by using hydraulic models, only a few of them address the concern regarding flood control operation in widely used modeling systems such as HEC-5, HEC-ResSim, Mike 11 (Nguyen et al., 2018). Ngo et al., (2008) have used a 1D hydrodynamic model based on Mike 11 hydrodynamic model to analyze the operation rules curves for the Hoa Binh reservoir in Vietnam. An alternative strategy has been evaluated for the main purposes of flood control and hydropower generation. Yazdi & Salehi Neyshabouri (2012) have used the Mike 11 simulation model to compute and assess the potential damages of different flood scenarios. The estimation of flood damages was assigned the water depth of floodplains under the various combinations of structural and nonstructural measures.

HEC-5, which developed by the US Army Corps of Engineers, is capable of simulating operations of reservoir systems with multiple objectives such as flood control, hydropower generation, and water supply. The Decision Support System (DSS) was designed by Ford & Russell Killen (1995) to provide real-time information for improved flood prevention and control in the Trinity river basin in Texas within HEC-5 used for simulation reservoir operation. Bayat et al. (2011) utilized the HEC-5 for operation policies applying gate regulation curve under flooding conditions.

The HEC-ResSim software tool, developed by the Hydrologic Engineering Center of the US Army Corps of Engineers, is a useful modeling system and is widely used. The software can support engineers and planners, performing water resources studies, in predicting the behavior of reservoirs and to help reservoir operators to plan releases in real-time, during day-to-day and

emergency operations (Klipsch & Hurst, 2013). HEC-ResSim was applied by Uysal et al. (2016) to study the operation of Yuvacik Dam in Turkey during a flood event. The model can be applied to determine the magnitude and timing of spillway release discharge with the objective of water supply and flood control downstream of the dam.

Some of the most frequently used multi-reservoir tools have been benchmarked during the current research to determine capabilities and performances. An effective simulation model can provide to a decision-maker the results for examining the effects of different scenarios over a river reservoir system. For the current research objective and after reviewing the various available tools, the operation of the river multi-reservoir system in the Vu Gia Thu Bon catchment has been simulated with a Mike 11 model. This modeling system is adopted for flow simulation in the river system and includes the multi-reservoir system.

Mike 11 is a software package developed by the Danish Hydraulic Institute (DHI, 2014) for simulation of fully dynamic, one-dimensional flows, sediment transport, and water quality in estuaries, rivers, and irrigation systems. The core of the Mike 11 system consists of a hydrodynamic (HD) module that is able to simulate unsteady flows in a network of open channels. The Mike 11 HD can be used for many applications, including flood forecasting and reservoir operation, simulation of flood control measures, operation of irrigation and surface drainage systems, or design of channel systems, etc.

The general process related to the river-reservoir system simulation is presented in Fig. 3.2. This process consists of several major components. The first component is the hydraulic model that allows users to perform one-dimensional unsteady flow computation for the Vu Gia Thu Bon river network. After calibration and validation of the hydraulic model, it is possible to analyze the multi-reservoir operation strategy. The second component is the reservoir operation model for spillway gate operation using the Structure Operation (SO) module in Mike 11 modeling system. Analyzing results present operation rules are performed using flood elevation and discharge in the river reservoir system, calculated by the model.

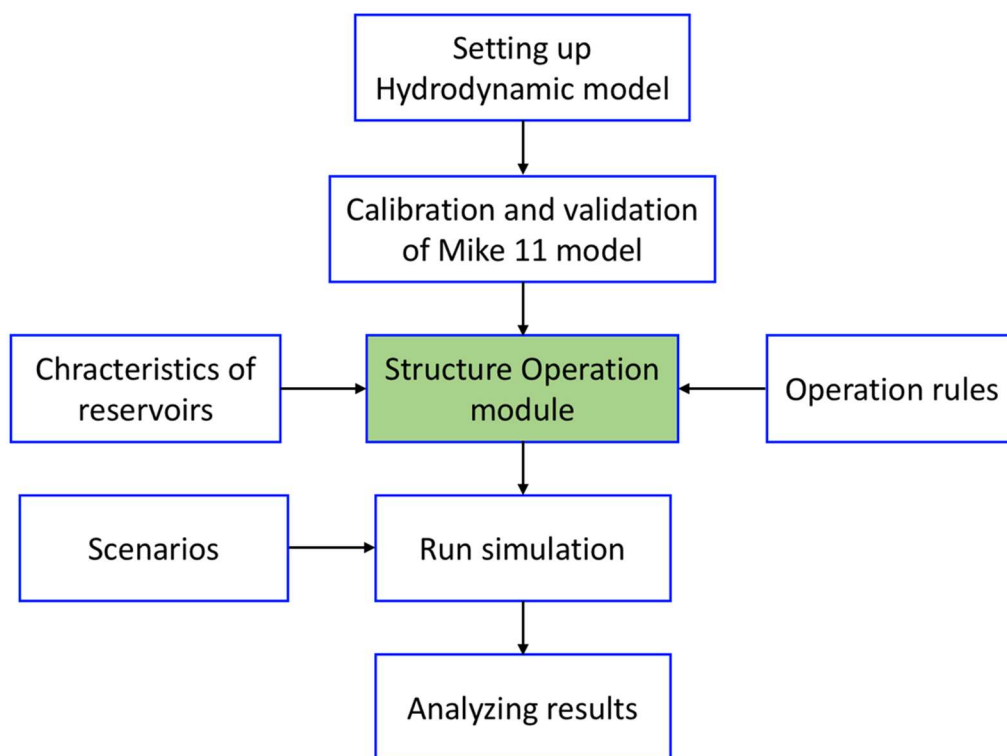


Figure 3.2. Methodology flowchart for modeling of reservoir system operation.

3.2. Mike 11 modeling system

3.2.1. Mike 11 HD model description

The Mike 11 hydrodynamic modeling system (DHI, 2016) is an implicit, finite-difference model for computing the steady flow of rivers. This modeling tool can describe both the subcritical and the supercritical flow conditions applying a numerical scheme that adapts to the local flow conditions.

Mike 11 HD applies the dynamic wave description solving the vertically integrated equations of conservation of continuity and momentum (Barré de Saint Venant equations/shallow water equations). The derivation of the equations of continuity and momentum, as used in Mike 11, the resulting equations are below:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (3.5)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(\alpha \frac{Q^2}{A} \right)}{\partial x} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0 \quad (3.6)$$

where:

Q is discharge (m³/s);

- A is the cross-sectional area (m^2);
- q is the lateral inflow ($\text{m}^3/\text{s}/\text{m}$);
- h is the stage above datum (m);
- C is the Chezy resistance coefficient ($\text{m}^{1/2}/\text{s}$);
- R is the hydraulic or resistance radius (m);
- α is the momentum distribution coefficient (s^2/m^3);
- g is the gravitational acceleration (m/s^2);
- x and t are the distance (m) and time (s), respectively.

Both the continuity and momentum equations are simultaneous, quasi-linear, first-order, partial differential equations of hyperbolic type. The solution of these equations is based on an implicit finite difference scheme known as a six-point Abbott scheme developed by Abbott and Ionescu (Abbott & Ionescu, 1967). After solving the model derived discharge, Q (or velocity, V) and water elevation, h in all cross-sections of simulated river systems.

For the Vu Gia Thu Bon catchment, the hydrodynamic model was calibrated using the historical hydrographs recorded in the hydrometric gauges of the river basin.

3.2.2. Development of 1D hydrodynamic (HD) model for the Vu Gia Thu Bon river network

The 1D hydrodynamic model has been developed for modeling operation multi-reservoir system in the Vu Gia Thu Bon catchment. After calibration, the one-dimensional model has been used to simulate to predict the opening of spillway gates during the flood season. In the HD model, the simulation editor provides a link between the network editor and the other Mike 11 editors, as illustrated in Fig. 3.3.

(a) River network

The Vu Gia Thu Bon river network has two mains rives, the Vu Gia and the Thu Bon rivers. The Vu Gia river has many tributaries, such as the Bung branch, the A Vuong branch. The length of the river Vu Gia to the Han estuary is 204 km. The Thu Bon river is 152 km long to the Cua Dai estuary. This model is developed on 17 rivers and linking branches (Fig. 3.4). After the preparation of the base model, the four reservoirs have been placed in the reservoir branches of the network, and the spillway gates have been incorporated into the model.

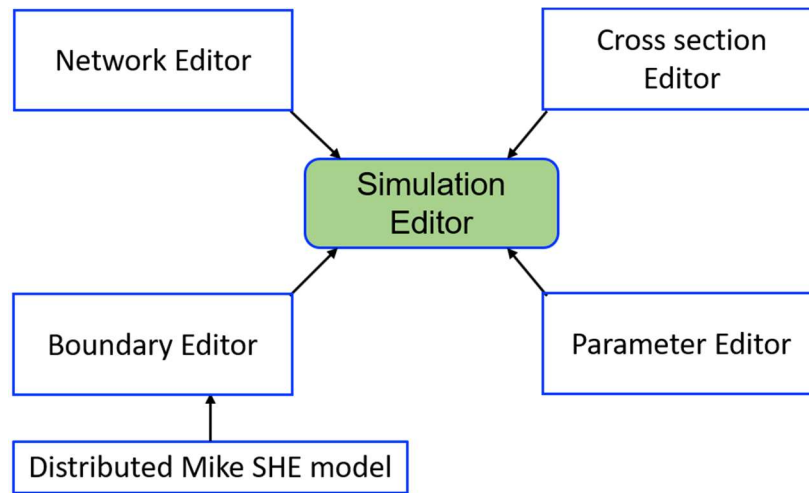


Figure 3.3. The Mike 11 simulation editor.

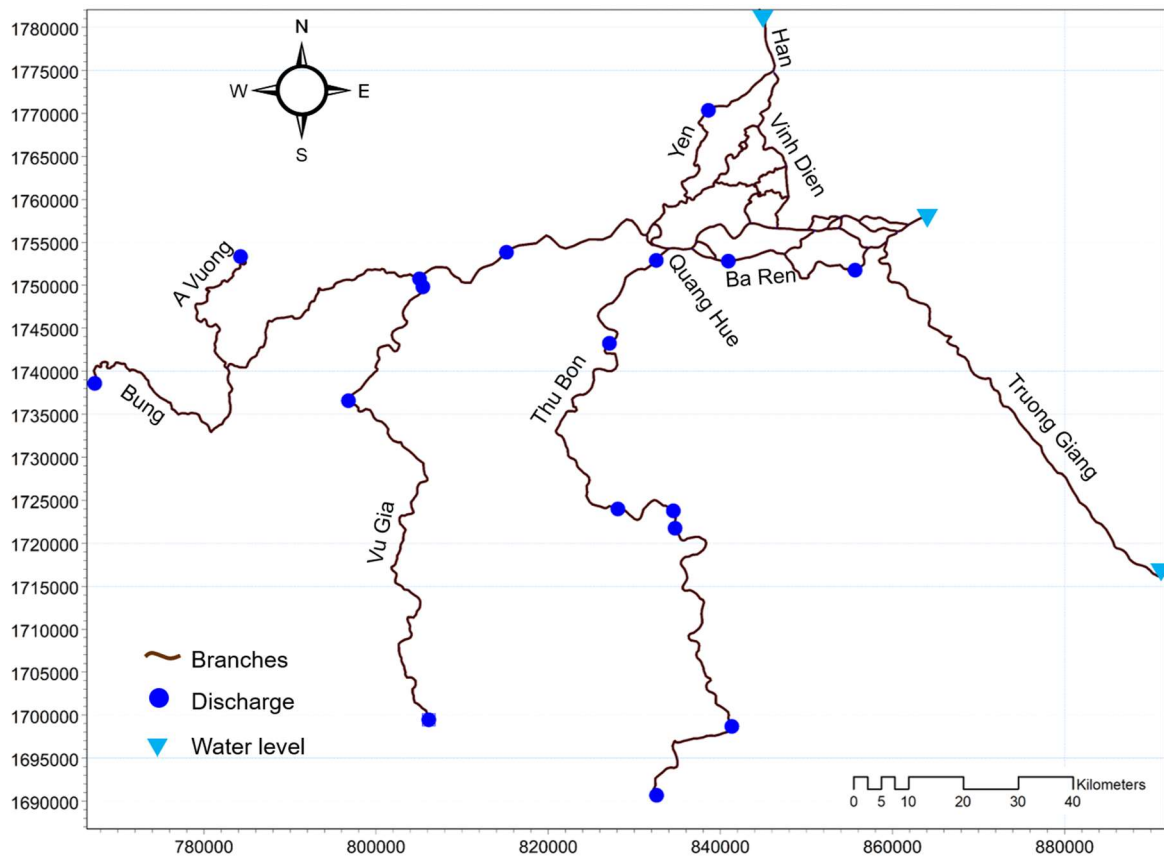


Figure 3.4. River network of the Vu Gia Thu Bon catchment in Mike 11 model.

(b) Cross-sections

The geometry of each river branch is specified via cross-sections from measurements at the downstream area, and the other is extracted from a Digital Elevation Model (DEM). The DEMs benefit from the research project LUCCi (Land Use and Climate Change interactions in central Vietnam) which is funded by the German Federal Ministry of Education and Research. This data presents

topography at the catchment with 15 m solution and covers overall Vu Gia Thu Bon catchment. More than 1000 river cross-sections at different sections of the river were used to define the geometry of the river and the four reservoirs. Fig. 3.5 shows a graphical representation of the elevation versus widths of the river at a few selected locations of the Vu Gia river branch. It is observed from Fig. 3.5 that the widths of the river get wider towards the downstream end. In high flow conditions, the capacity of the cross-sections is not found to be sufficient enough to control the flood. Therefore, during flood season, the flow was mainly overtopping into the floodplain is more likely common.

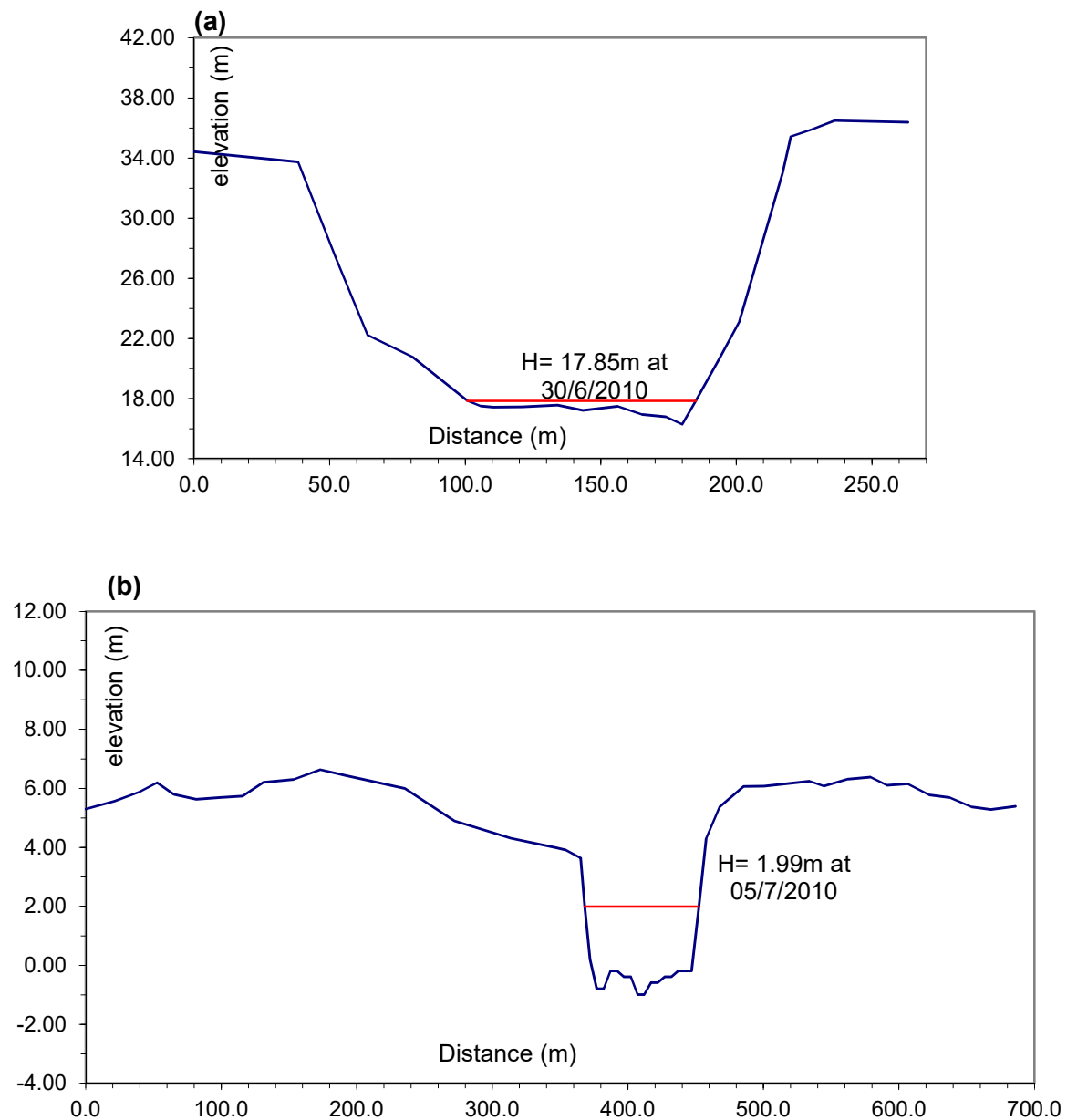


Figure 3.5. Typical cross-section of the upper (a) and downstream (b) Vu Gia river.

(c) Boundary conditions

The hydrodynamic model contains various upstream boundaries and three downstream boundaries. In the Vu Gia Thu Bon catchment, there are only two stations (i.e., Nong Son and Thanh My stations) for measuring discharges. These stations are located at the middle of these branches (Fig. 3.7). Hence, the inflows must be estimated based on a rainfall-runoff model. Vo (2015) has been successfully built a deterministic distributed hydrological model based on Mike SHE model and the model efficiency is likewise confirmed by the capacity to predict extreme peak flow and baseflow. Therefore, the upstream boundaries were the discharge time series inflow four reservoirs and lateral flow that are extracted from the Hydrological Mike SHE model established for the full catchment. The three downstream boundaries of the hydraulic model are the water level at the estuaries of Han, Cua Dai, and Cua Lo. These levels are defined as sea levels at Son Tra and Hoi An stations.

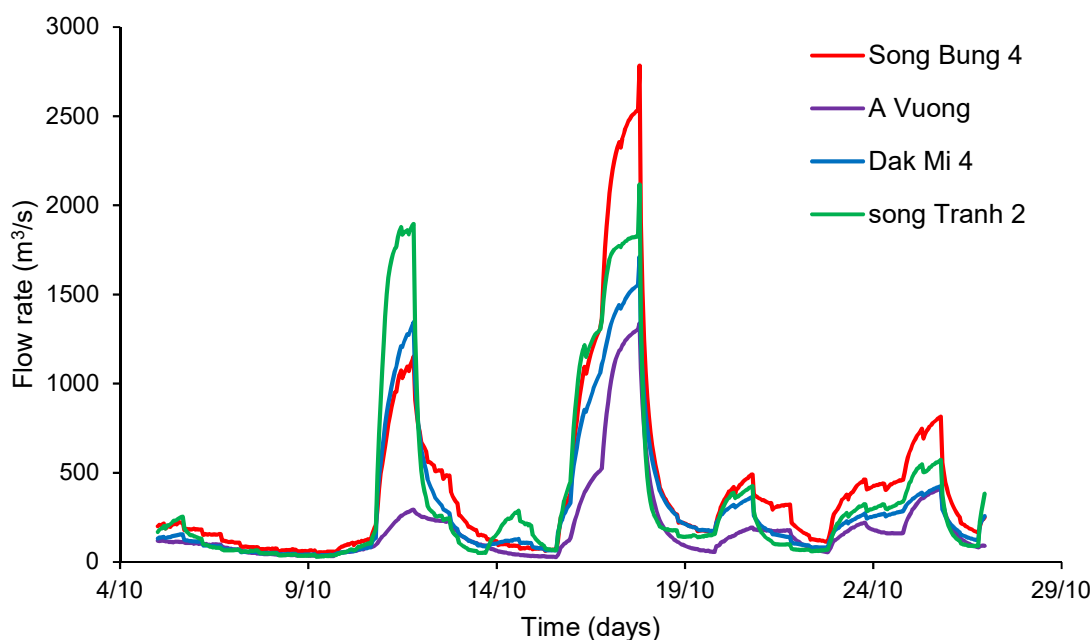


Figure 3.6. Inflow reservoir hydrographs extracted from Mike SHE model.

(d) Hydrodynamic parameters

There are many parameters that can influence the outputs of the simulation and the quality of results. Among parameters, after topographical accuracy, flow roughness is one of the major parameters for the calibration of the 1D HD model. The flow roughness is represented via Strickler roughness coefficient M (inverse of Manning's roughness n). In the model setup, the high order fully dynamic wave approximation is appropriate to simulate the flood flow in the research.

Computational time steps Δt and grid size Δx have been chosen appropriately to make the computation stable by keeping the Courant number less than unity.

3.2.3. Calibration of 1D HD model

The model calibration is the most important part of the model running. The hydrodynamic unsteady flow models must be calibrated and verified. The calibration and validation processes require two independent and statistically reliable sets of data. One dataset is used to establish the optimum values of the ‘free’ coefficients, and the second dataset is utilized to verify the calibration model (Fr nch, 1985). As described above, the flow Strickler roughness coefficient M (inverse of Manning’s roughness n) for the range of flows associated with the previously observed floods was selected using a trial-error method to obtain the best comparison between observed and simulated flow parameters. A range of flow roughness values selected during the trails was based on the conditions of the channel (Singh, 2017).

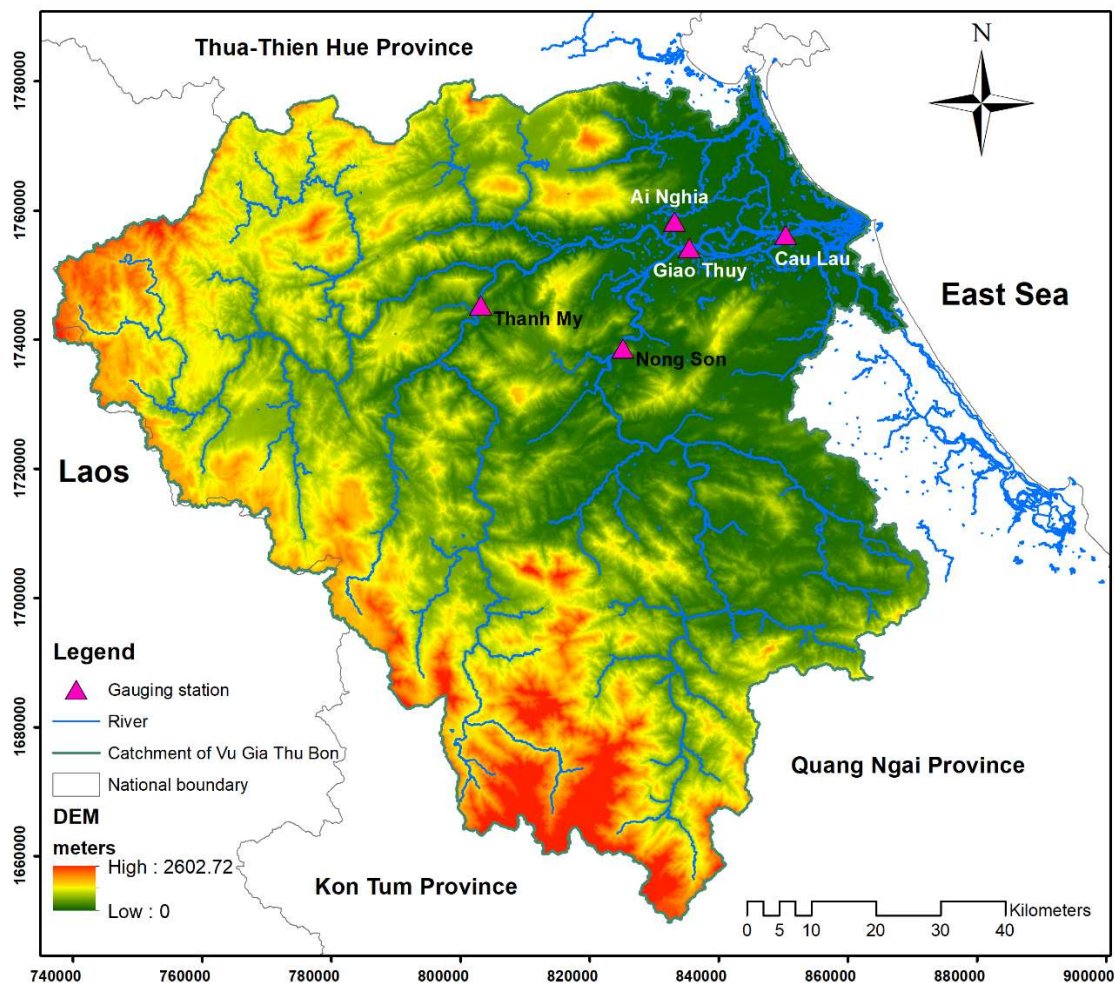


Figure 3.7. Gauging stations in the Vu Gia Thu Bon catchment.

The hydrological year of 2009 has been used for the calibration with different values of roughness coefficient M . Water levels measured at downstream monitoring gauges such as Ai Nghia, Giao Thuy, and Cau Lau stations (Fig. 3.7) have been compared with simulated water levels. The correlation coefficient (R), root means squared error (RMSE), and Nash-Sutcliffe coefficient (NSE) were utilized to evaluate the correlation between observed and simulated values. The formulas and their performance levels were shown as follow:

$$R = \frac{\sum_{i=1}^n (H_{obs,i} - \bar{H}_{obs})(H_{sim,i} - \bar{H}_{sim})}{\sqrt{\sum_{i=1}^n (H_{obs,i} - \bar{H}_{obs})^2 \sum_{i=1}^n (H_{sim,i} - \bar{H}_{sim})^2}} \quad (3.7)$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (H_{obs,i} - H_{sim,i})^2}{n}} \quad (3.8)$$

$$NSE = 1 - \frac{\sum_{i=1}^n (H_{obs,i} - H_{sim,i})^2}{\sum_{i=1}^n (H_{obs,i} - \bar{H}_{obs})^2} \quad (3.9)$$

where:

$H_{obs,i}$ and $H_{sim,i}$ are observed and simulated water levels values at the time step i ;

\bar{H}_{obs} and \bar{H}_{sim} are average of observed and simulated water levels values.

Table 3.1. Performance criteria for model evaluation (Wang et al., 2012).

Performance indicator	Excellent	Good	Fair	Poor
R	> 0.95	0.85-0.95	0.75-0.85	< 0.75
NSE	> 0.85	0.65-0.85	0.5-0.65	< 0.5

Initially, a constant Strickler roughness coefficient M for the bed resistance of 30 was applied to the Mike 11 HD model. However, during the initial runs, this value resulted in water levels being too high at downstream, which was subsequently taken care of by decreasing the local value of M . Several trial simulations models have been undertaken for various Manning's M at different locations of the river and were used to update the global values of M .

The calibrated global Strickler roughness coefficient M was found to be 60, and the local values were observed to be within the range of 30-50 in the upstream of the Vu Gia Thu Bon river system, whereas 60-80 is observed for the

downstream of the river branches. The peak water level was over-predicted, and the difference in the peak of the flood between simulation and observation was found to be 0.22 m, 0.16 m, 0.05 m at Ai Nghia, Giao Thuy, and Cau Lau gauging stations during calibration, respectively (Tab. 3.2).

Hydrograph of 1h water level in Fig. 3.8. demonstrates the capacity of the 1D HD model to reconstitute the flooding process occurring downstream of the Vu Gia Thu Bon river system.

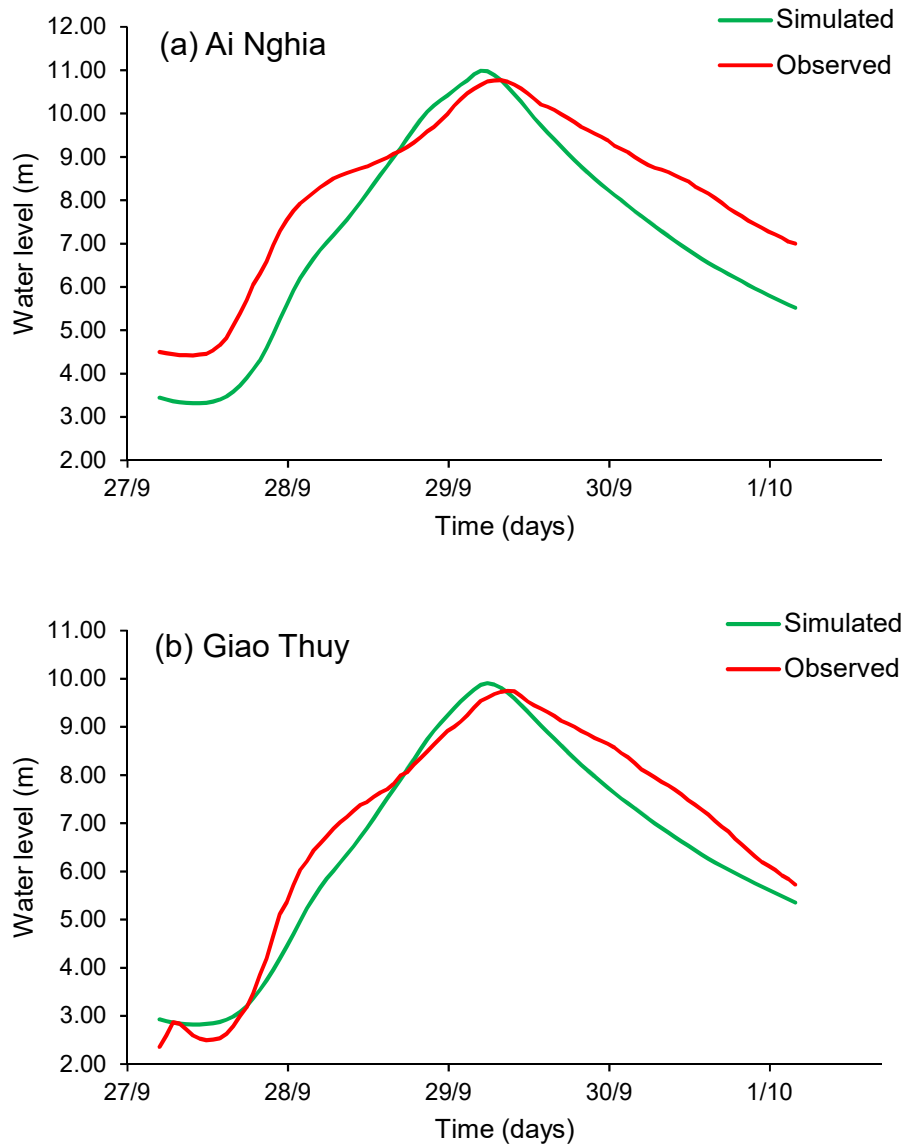


Figure 3.8. Comparison of temporal variation of observed and simulated water level at Ai Nghia (a) and Giao Thuy (b) gauging stations during calibration of Mike 11 model.

3.2.4. Validation

The calibrated hydrodynamic model has been validated for the floods from November 10th to November 15th, 2007 that is one of the most significant flood events observed within the last decade. The simulated water levels were compared with the observed water levels at downstream gauging stations to measure the performance of the model graphically (Fig. 3.9) and performance criteria (Tab. 3.2).

Table 3.2. Statistical indices of Mike 11 model at downstream gauging stations.

Simulation	Year		Stream gauging stations		
			Ai Nghia	Giao Thuy	Cau Lau
Calibration	2009	R	0.97	0.98	0.91
		RMSE	1.20	0.63	0.75
		NSE	0.98	0.99	0.96
		ΔH_{\max} (m)	0.22	0.16	0.05
Validation	2007	R	0.92	0.93	0.85
		RMSE	1.37	1.21	1.19
		NSE	0.98	0.98	0.92
		ΔH_{\max} (m)	0.07	0.24	0.41

In general, from Fig. 3.9 it can be seen that maximum observed values are systematically higher than the maximum simulated water level at different gauging stations. Still, the difference between the highest simulated water level at these stations and observations is not significant (Tab. 3.2).

The hydrographs were seen that the simulated peak flows arrive earlier than the reality and this tendency occurs similarly at all stations. The difference in times appearance of peak water level between simulation and observation is just less than 2 hours. The cause might result from the inaccuracy of the hydrological model, which unable to reach 100% performance while representing the complicated hydrological process of the catchment. The difference between the simulated peak and observed peak was found to be 0.07 m, 0.24 m, 0.41 m at Ai Nghia, Giao Thuy, and Cau Lau gauging stations during validation, respectively. The correlation coefficients R between simulation and observation reach relatively high, R index passed 0.85 in all stations.

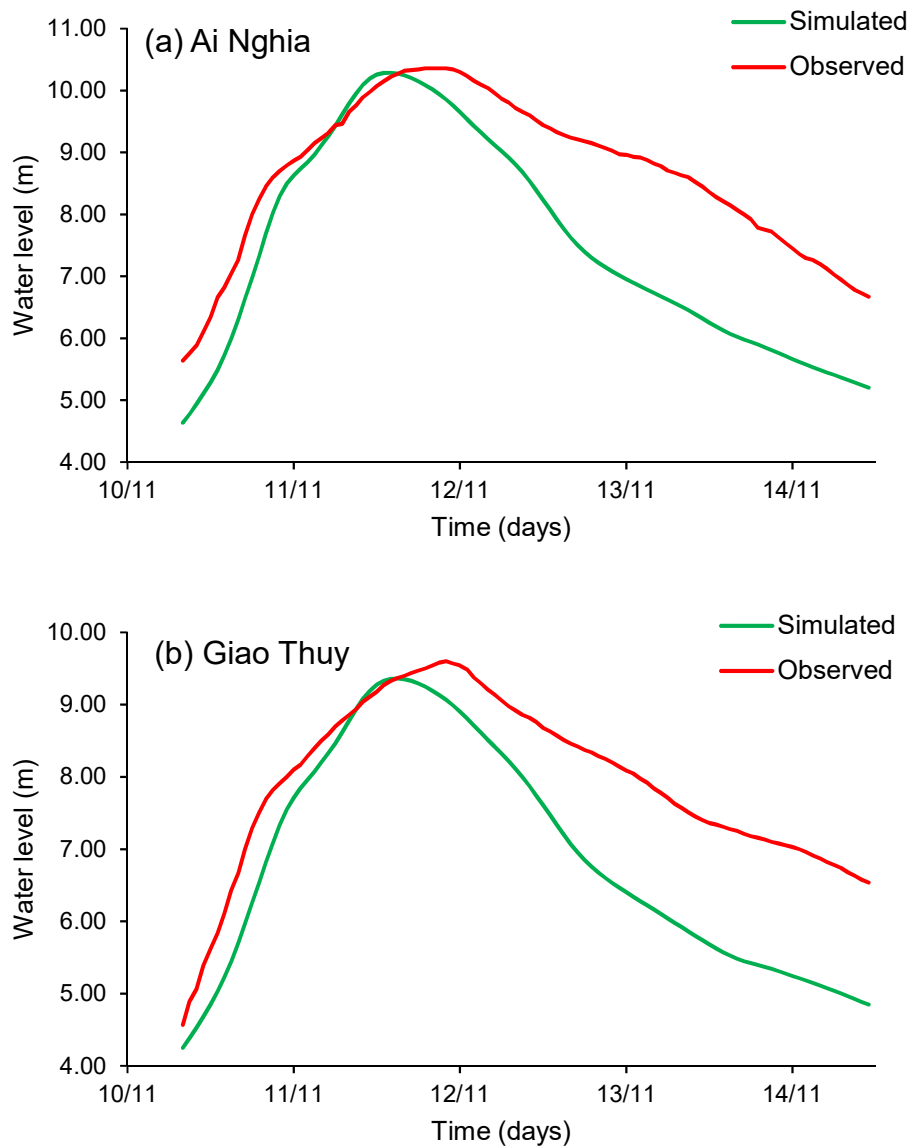


Figure 3.9. Comparison of temporal variation of observed and simulated water level at Ai Nghia (a) and Giao Thuy (b) gauging stations during validation of 1D HD model.

Fig. 3.9 shows that, in general, the simulated hydrographs capture the observed data at both stations pretty well, especially the rising limbs and the flood peaks as well as the peak arrival time. The recession limbs, however, are underestimated as well as the total flow volumes for both stations. The differences in the hydrograph patterns could be due to the uncertainties of the boundary conditions as well as the lacking of some physical processes in the model setups. In particular, the upstream boundary conditions were not based on the observed data (since they were unavailable) but based on the simulated results of the distributed Mike SHE model (see the Appendix). These simulated flow series could be lower described with the observed flow series. Furthermore,

the use of the 1D model could not capture the complicated hydraulic processes in the downstream areas of the Vu Gia Thu Bon catchment with a complicated river network. Indeed, these areas are flat and the flows are highly turbulent (Fig. 3.7). There are also shear stresses due to the momentum exchanges between the flows in the main river and the flood plain areas. These characteristics could be better described with a 2D model or a coupling of 1D and 2D models rather than only the 1D model. Therefore, once approaching the flood peak and the water in the flood plain area gradually returns to supplement to the main river flow, the simulated recession limb tends to decrease much faster than the real process. Consequently, this results in an underestimation and lower accuracy when simulating the hydraulic processes in these areas with only the 1D model (Vo & Gourbesville, 2016b).

However, the current research mainly focused on the maximum water level at the downstream control points. The results indicated that the differences between the simulated and observed water stage peaks were insignificant. In addition, the 1D model offers a much simpler model setup and faster computational time compared to those of a 2D model. For the above persuasive shreds of evidence, the use of a 1D HD model was more applicable and more preferable in simulating the water levels at the downstream areas of the Vu Gia Thu Bon river system.

This chapter consists of two main parts. The first part presents the calibration and validation of the one-dimensional unsteady flow hydraulic model. This model allows users to simulate and compute the hydraulic processes and interest hydraulic variables (i.e., water stages) for the Vu Gia Thu Bon river network. The second part describes the modeling of the reservoir operation with the Structure Operation (SO) module. A brief overview of the SO module and analyze the multi-reservoir operation strategy are provided in the next section.

3.3. Structure Operation modeling

According to the above description, the computational core of the Mike 11 model consists of a hydrodynamic simulation engine and a wide range of additional modules (DHI, 2016). Structure Operation (SO) is one of the add-one modules that can be used to define operating strategies for structures such as sluice gates, overflow gate, radial gate, pumps, and reservoir release, which may be included in the river network.

Control structures may be used whenever flow through a structure is to be regulated by the operation of movable gates, which forms part of the structure. They can also be used to control the flow directly without considering the moveable gate into consideration. Which is note as the simulation of a pump. With the SO module, control structures may be operated by choosing among an arbitrary number of different control strategies, which are presented as a sequence of 'IF-THEN' statements.

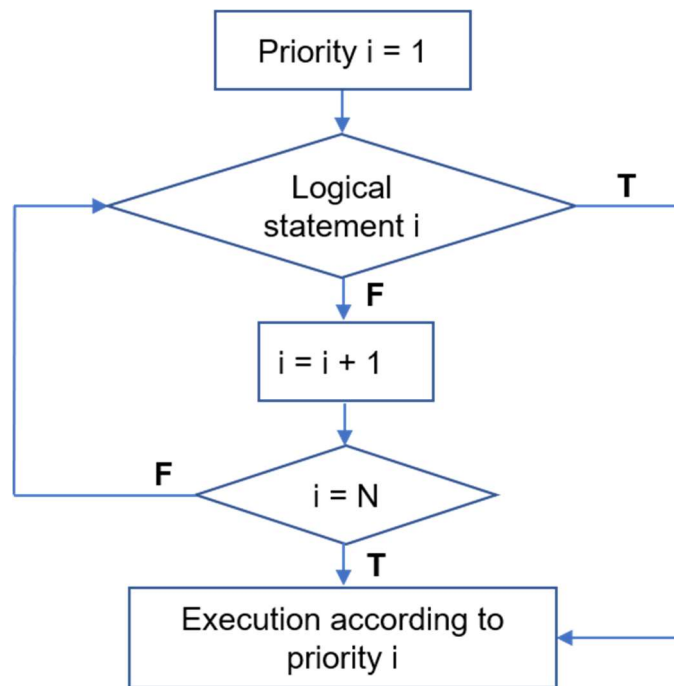


Figure 3.10. Operational procedure on SO module.

The calculation of the gate operation is determined from a control strategy. A control strategy describes how the gate level depends on the value of control points. For a specific gate, it is possible to choose among an arbitrary number of control strategies by using a list of “IF” statements (DHI, 2014). For each of these statements, it is possible to define an arbitrary number of conditions that all must be evaluated to True if the “IF” statement is to evaluate to True (Fig. 3.10). It is hereby to made probable to use different operating policies depending on the actual flow regime, reservoir stage, the water level at a control point, and time of the year, etc.

A control strategy consists of two parts: (1) conditions that must be fulfilled for the strategy to be executed and (2) a control strategy itself. The control strategy itself is a relationship between an independent variable (the value of the control points) and a dependent variable (values of the target points).

As mentioned above, it is possible to make Mike 11 model choose among an arbitrary number of control strategies. The control strategy belonging to the first of these statements that are evaluated to True will be executed (Fig. 3.10). Thus, it is important for the user to define which “IF” statement that is evaluated first, second, third, and so on. In this research, a simulation model that simulates the releases from the four reservoirs in the Vu Gia Thu Bon catchment, through the operational structures spillway gates, specified in Mike 11 model as a control structure.

3.3.1. Calculations required for reservoirs system operation

* Radial gate

This gate type corresponds to a tainter gate. In contrast to the other gate types, a radial gate does not need any information about head loss factors (DHI, 2016). Fig. 3.11 is the definition sketch of a radial gate.

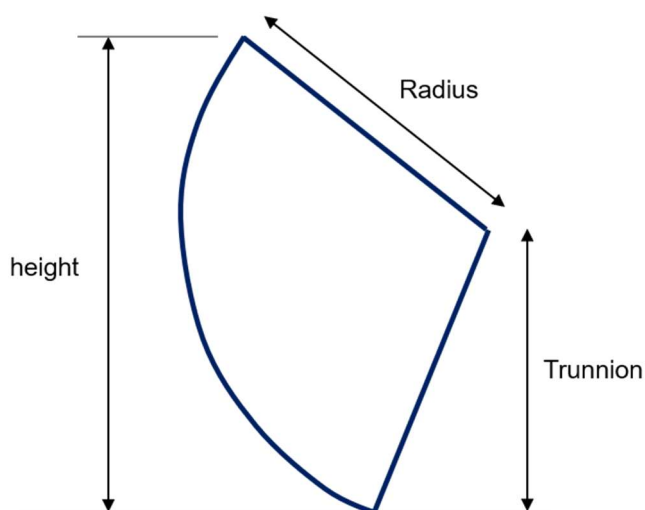


Figure 3.11. Definition of a radial gate.

Height: height above the sill level of the overflow gate crest when the gate is closed.

Radius: radius of the gate.

Trunnion: height above the sill level of the center of the gate circle.

Table 3.3 shows the characteristic of four radial gates in the Vu Gia Thu Bon reservoir system.

Table 3.3. The characteristic of four radial gates

	A Vuong	Dak Mi 4	Song Tranh 2	Song Bung 4
Number of gates	3	5	6	6
Width of the gate (m)	14	14	14	12
Sill level	363	242.5	161	210.5
Height (m)	17	15.5	14	12
Radius (m)	19.5	17	15.5	13.5
Trunnion (m)	7.5	6.9	7.1	8.1

3.3.2. Reservoir definition - Calibration of reservoir capacity curve

Modeling of reservoirs includes several considerations such as used types of structure, modeling of the structure. An appropriate definition of structure type could enhance the outcome quality of the model. However, another very important task when modeling reservoirs is that ensuring consistent reservoir state-volume between modeling and natural situation. If the reservoir stage-volume relation in the model is not correct, then reservoir levels following specific inflow events to the reservoir will not be correct.

In the Vu Gia Thu Bon reservoir system, the capacity curve of each reservoir is defined based on available its bathymetry (pairs of water levels and corresponding storage values), which are available from the technical reports produced at designing time. In this model, more than 400 cross-sections upstream reservoirs are used to define the storage capacity of these four reservoirs.

Based on enough long series of inflows to four reservoirs (discharge inflow reservoir is $500 \text{ m}^3/\text{s}$), the rules of model determined for fulfilling of the total capacity of the reservoir and the water level in each reservoir is calculated. By modifying the cross-sections of the upstream reservoir, the reservoir capacity curve is more likely to adapt to the real curve. As a result, the similarity of both curves of measured and estimated is showed on Fig. 3.12.

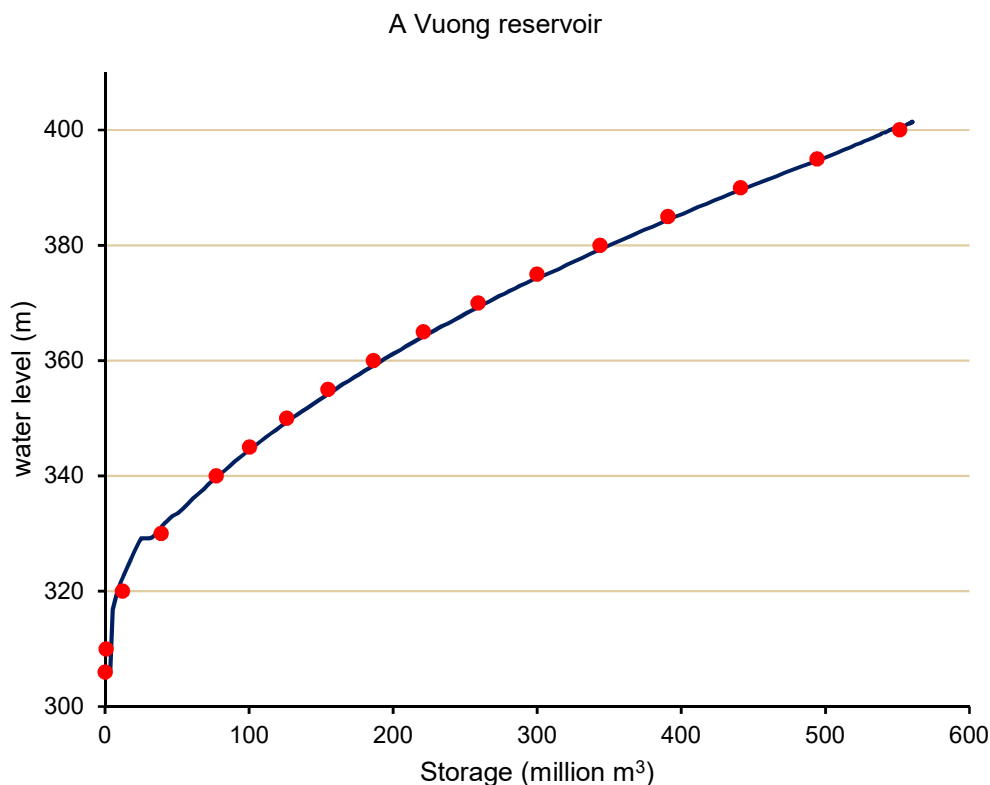


Figure 3.12. The relation water level with storage of A Vuong reservoir; measured (solid) and estimated (red dots).

3.3.3. Calibration discharge factor

This gate type corresponds to a tainter gate. In the Mike 11 model, radial gates are automatically into an underflow part and an overflow part. The way the flow through the underflow part is calculated depends on the flow regime. Under free-flow conditions, the discharge is calculated to follow Bijankhan et al., (2011) as equation 3.10 below:

$$Q_{free,underflow} = \tau \frac{\delta}{\sqrt{1 + \frac{\delta w}{y_1}}} a \sqrt{2gy_1} \quad (3.10)$$

where:

τ is a discharge calibration factor;

g is the acceleration of gravity;

y_1 is the upstream water level;

w is the vertical gate opening;

a is the flow area through the gate (vertical gate opening multiplied with the width of the gate);

δ is the contraction coefficient computed as:

$$\delta = 1 - 0,75 \left(\frac{\theta}{90^\circ} \right) + 0,36 \left(\frac{\theta}{90^\circ} \right)^2 \quad (3.11)$$

where:

θ is the inclination angle of the gate relative to the canal bottom.

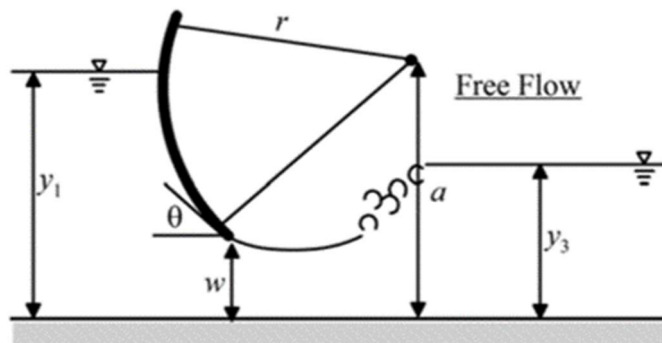


Figure 3.13. Definition sketch for radial gate – free flow.

The under submerged flow conditions, the discharge is calculated as following the suggestion of Clemmens et al., (2003):

$$Q_{submerge,underflow} = \tau \frac{\delta}{\sqrt{1 - \left(\frac{\delta w}{y_1} \right)^2}} a \sqrt{2g(y_1 - y_2)} \quad (3.12)$$

where:

y_2 is the downstream water level.

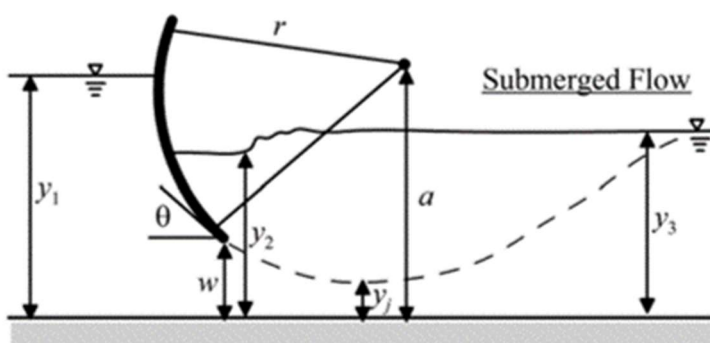


Figure 3.14. Definition sketch for radial gate – submerged flow.

If the upstream water level is higher than the top of the radial gate, flow above the radial gate will occur. The overtopping flow is calculated as a function of the downstream water level conditions.

When the downstream water level is lower than the top of the radial gate, the discharge is calculated using:

$$Q_{overflow} = b\alpha y_1^\beta \tag{3.13}$$

where:

b is the width of the gate;

α and β are the user-defined coefficients ('weir coeff' and 'weir exp' in the model).

To determine the value of the discharge factor, the mathematical model results with the physical model results for various flow conditions are compared. Based on physical model results, four different discharge factors have been used to cover the entire spillway gates flow in the Vu Gia Thu Bon reservoir system based. These values are given in Tab. 3.4.

Table 3.4. Discharge calibration factors and maximum discharge capacity of four spillway gates.

Spillway gates	Discharge calibration factor (τ)	Maximum discharge capacity (m ³ /s)
A Vuong	1.07	5720
Dak Mi	1.04	8584
Song Tranh 2	1.02	9035
Song Bung 4	1.05	6363

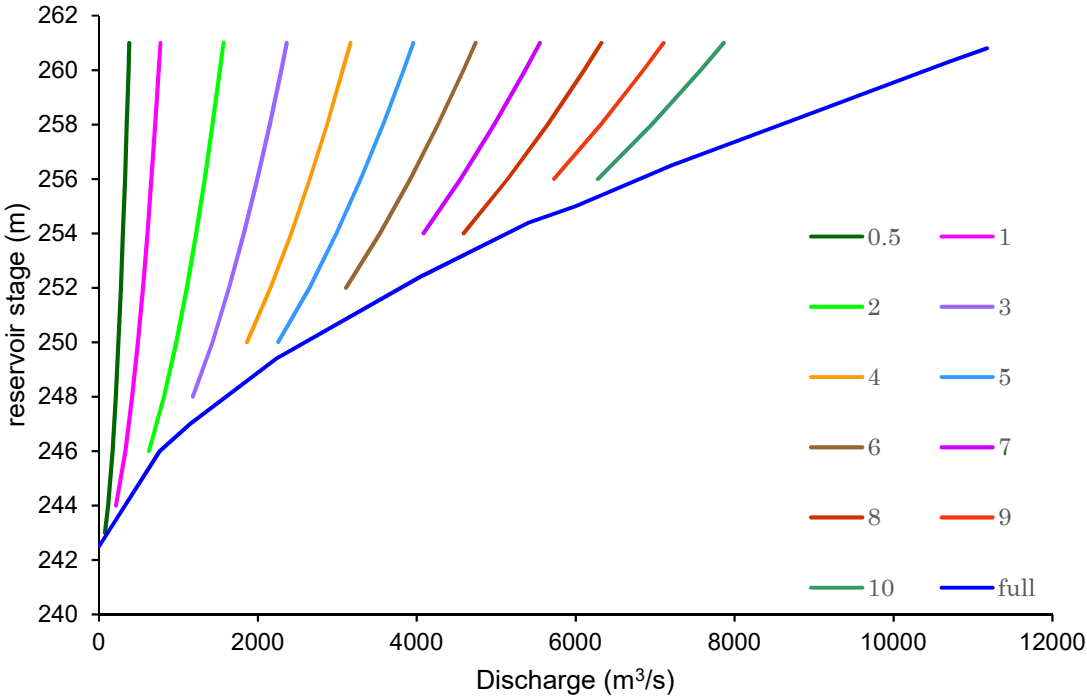


Figure 3.15. Relations of spillway gate opening coupled with water level vs. water release.

3.4. Modeling operation of the multi-reservoir system in the Vu Gia Thu Bon catchment

3.4.1. Regulation for flood control

The reservoir operation guidelines in this section are based on the Decision 1537 (Government of Vietnam, 2015). The general framework of the operation strategy includes pre-release, flood control, and prefill procedures. The opening of the spillway gates depends on the reservoir stages, inflow, water level at the control points, and time of the year. Before formulating, three flood stages are defined and demonstrated in Fig. 3.16. A summary of release guidelines regarding each of the flood stages and the corresponding conditions as following:

Stage 1: pre-release procedure (before flood arrival): $[t_0, t_1]$

Pre-release procedure in flood event is considered as an effective measure in flood management. The principle of this procedure reduces the risk of flood by limiting volume in this reservoir system for storage inflow (Chou & Wu, 2013). These operations discharge a proportion of the usable volume of reservoirs before occurring of flood with considering inflow forecasting. As a consequence, the strategy allows the reservoir to reduce water level accommodates with required volume for flood control downstream while the water level is

maintained at the highest level at the end of flooding which can meet other activities demand in the next dry season such as hydropower generation, irrigation.

Although this strategy effectively controls flooding impact, there are many risks to operation in flood control and negative impacts on water demand. This is because this strategy strongly depending on meteorological conditions and hydrological forecasting. On the one hand, in cases of forecasted flooding that is much larger than reality, the pre-release amount is enlarged, resulting in insufficient water sources for the next dry season. On the other hand, if the forecasted flooding is much smaller than the happening flood, the pre-release volume is not adequate to store the total volume of the flood event that may lead to a large amount of water discharge to downstream and causing damages to residents and infrastructure.

In this stage, the reservoir stage is in the bound of Flood Limit Water Level (FLWL), there are estimated the following two policies:

- Policy 1: if the water levels at the control points are lower than alarm level 2, the release amount should be greater than the inflow discharge to reduce the reservoir stage, reaching to lower bound of FLWL (Fig. 3.16 b).
- Policy 2: if the water levels at the control points are on the alarm level 2, the release discharge equals inflow to maintain the current stage at these reservoirs.

Stage 2: flood control procedure (the large flood stage): $[t_1, t_4]$

There are three principle policies in this stage.

- Policy 1: if the water levels at the control points are higher than alarm level 2 and smaller than alarm level 3, the release amount equals to inflow to maintain the current reservoir stage (the lower bound of FLWL).
- Policy 2: if the water levels at the control points are greater alarm level 3 and the reservoir stage less than the normal water level, the reservoir releases are less than the reservoir inflow, store water in these reservoirs. In order to mitigate flood reduction, these releases are classified by several intervals of both storage level and reservoir inflow.
- Policy 3: when the reservoir stage reaches the normal water level, the total release discharge equals to reservoir inflow.

Stage 3: post-flood procedure: $[t_4, T]$

The post-flood procedure is the method for ensuring the reservoir stage is descending to the upper bound of FLWL. When the water levels at the control points are below the alarm level 1, the total discharge release is greater than reservoir inflow during the period from 24 to 72 hours.

In the current situation, a simulation model specified in Mike 11 as a control structure that presents the releases from the four reservoirs according to the operation guidelines above. The gate operation (gate type is a radial gate) methods are determined from a control strategy. The control strategy for the gates structure is described in the Tab. 3.5. The control strategy describes how the gate opened level depends on the value of the control points, such as the reservoir stage, the downstream water levels, reservoir inflows and the time of the year.

Table 3.5. Control definition for the spillway gates

Control definition	Condition	Gate operation
Stage 1	IF $H_{res} > L\text{-FLWL}$ and $H_{downs} < H_2$	THEN Raise Gate Level
	ELSE IF $H_{res} > L\text{-FLWL}$ and $H_{downs} \geq H_2$	THEN Keep present gate position
Stage 2	ELSE IF $H_2 \leq H_{downs} \leq H_3$ and I is increasing	THEN $R = I$
	ELSE IF $H_{downs} \geq H_3$ and I is increasing	THEN Change gate level for $R < I$
	ELSE IF $H_{downs} \geq H_1$ and I is decreasing	THEN Lower gate level for $R = I$
Stage 3	ELSE IF $H_{downs} \leq H_1$	THEN Raise gate level
	ELSE	Close gate

H_{res} is the reservoir stage; H_{downs} is the water level at a downstream control point; I and R are the reservoir inflow and release discharge; H_1, H_2, H_3 are the

alarm level 1, alarm level 2, alarm level 3 at the control point, respectively. L-FLWL is the lower bound of flood limit water level.

In this setup, the target points are the gates level of the four radial gates in their reservoirs, and more than three hundred of the logical statement is supplied to the model.

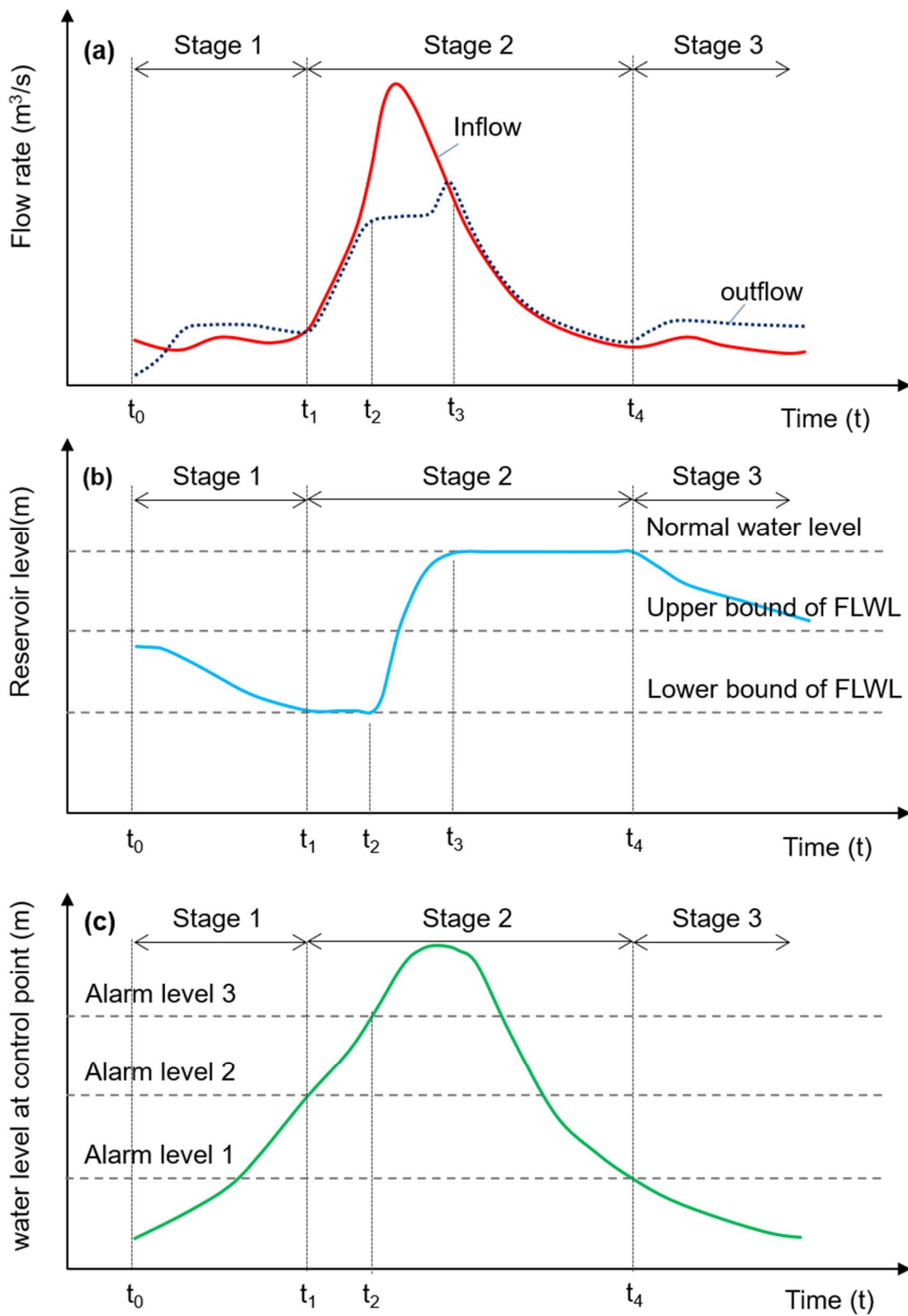


Figure 3.16. Definition of the three flood stages during a flood event.

3.4.2. Regulation for hydropower generation

Regulation for hydropower generation is generally maintained by rule curves. The operating rule curves enable visual expression of the operating strategy (Chang et al., 2005) and are derived from different operating guidelines curves. As illustrated in Fig. 3.17, the operation guide curve was defined with the fewest possible variables. In this study, hydropower generation is the second objective of the reservoir system in the flood season. Therefore, the operation aims to get as much hydropower as possible within the constraints of the flood reduction strategies. It consists of three zones, as shown in Fig. 3.17 to define how much water is supplied in the model for hydropower generation.

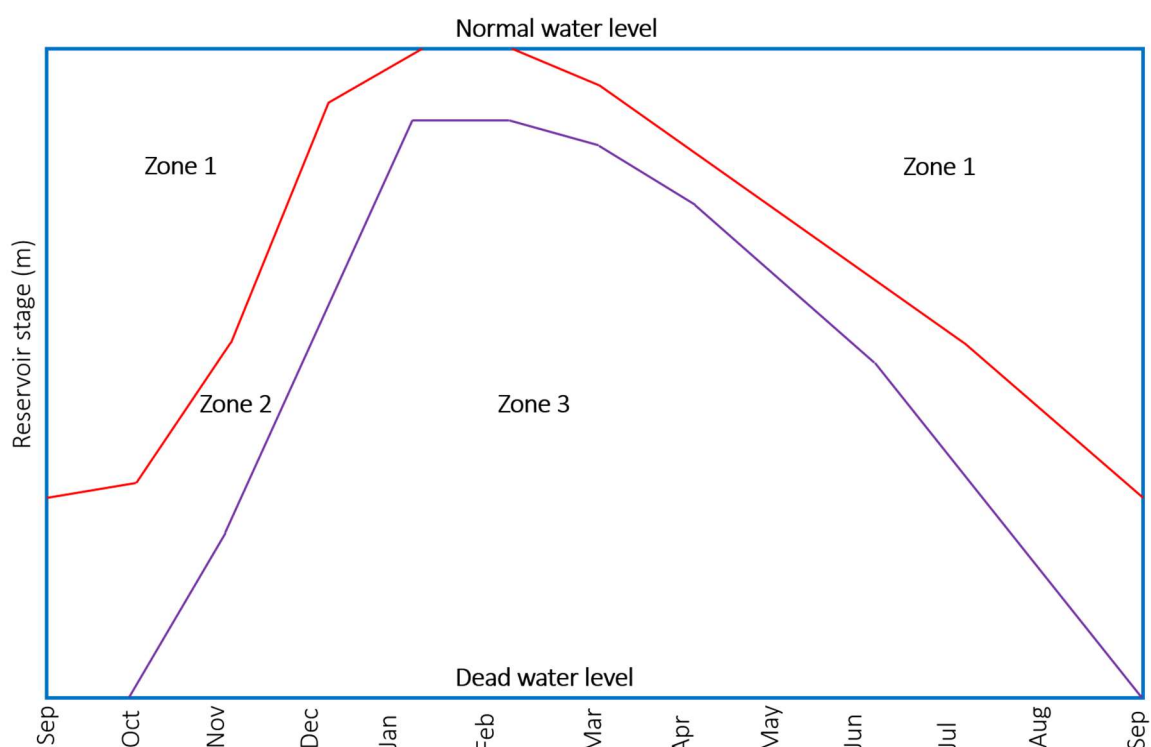


Figure 3.17. Operating rule curves of A Vuong reservoir.

The reservoir system operation for hydropower generation in the flood season according to the rule curves are described as follows:

- When the reservoir stage in zone 1, hydropower generation is operated with maximum discharge through turbines. The maximum discharge values of four hydropower plants shown in Tab. 3.6;
- When the reservoir stage in zone 3, hydropower generation is operated with maximum discharge through turbines that meets the minimum discharge values (Tab. 3.6);

- When the reservoir stage in zone 2, hydropower generation is operated with maximum discharge through turbines between the maximum and minimum discharge values.

Table 3.6. The maximum and minimum discharge values of four HP

Hydropower plant	A Vuong	Dak Mi 4	Song Tranh 2	Song Bung 4
Max discharge	78.4	128	245	166
Min discharge	15.68	25.6	49	33.2

3.4.3. Application scenario

Historical data of seven flood seasons, including normal and large flood conditions were utilized to evaluate control strategies (Tab. 3.7). The reservoir’s initial storages are also set to the dead water level at the beginning flood season (September 1st every year).

Table 3.7. The maximum reservoir inflow based on historical data from 2003-2009.

No	Year	Maximum reservoir inflow			
		A Vuong	Dak Mi 4	Song Tranh 2	Song Bung 4
1	2003	1893	3123	3265	4021
2	2004	1540	2540	4674	3271
3	2005	1707	2102	3348	2560
4	2006	2346	2590	900	3657
5	2007	2112	3695	5299	4488
6	2008	1556	1866	2471	2641
7	2009	3707	4713	3995	4155

After setting up the calibration and validation of the 1D hydrodynamic model, operating the reservoir system with two cases:

- Case A: without the reservoir system. In this case, each reservoir’s release will discharge equal to the inflow in the reservoir; it can be said that the flow in the river system is natural. The original historical event caused a significant loss of properties and humans in the floodplains. At the time, four hydropower projects had not been built yet. It can help us understand the magnitude of the flood before any human intervention and demonstrate an important role in reducing the flooding downstream.

- Case B: current rules. The control strategies are elaborated in Mike 11 model following the present regulation rules. The control strategy describes how the gate level depends on the value of the control points. In the flood control procedure, the water levels at the downstream control points are the most important parameters for flood control rule. The present regulation rules define these values are 8.5 m and 3.5 m at Ai Nghia and Cau Lau stations, respectively. The objective of this case is to evaluate the current rules during flood season concerning flood control and hydropower generation.

3.4.4. Results

The figures 3.18-3.19 shows the Dak Mi 4 reservoir flowrates following operation corresponding to the control strategy in case B in the 2007 flood season. These figures show time series of reservoir inflow and release (Fig. 3.18 a), reservoir stage (Fig. 3.18 b) and water level at the control points (Fig. 3.18 c). The reservoir operates by defining gate levels of the spillway and releasing discharge to the turbines. As an example, during the 2007 flood season, three stages of operation were corresponding to three procedures:

- Firstly, pre-release procedure: at 0:00 a.m. 8 November 2007, the reservoir stages were higher than the L-FLWL, so the reservoir released an amount of 400 m³/s during 48h for decreasing the water level in the reservoir.
- Secondly, flood control procedure: from 10th to 11:00 a.m. 11th November, the water level at the downstream control point was lower than the $H_3 = 8.5$ m, the release amount equaled reservoir inflow to maintain the reservoir stage (Fig. 3.19 a). At 11:00 a.m. 11th November, the water level reached 8.5 m (Fig. 3.19 b), so the release discharge was less than the reservoir inflow, and the reservoir stage was rising to normal water level.
- Finally, dam safely operation procedure: at 10:15 p.m. 11th November, the reservoir stage exceeds normal water level (258 m), the spillway operated according to the dam protection operational with release equal reservoir inflow.

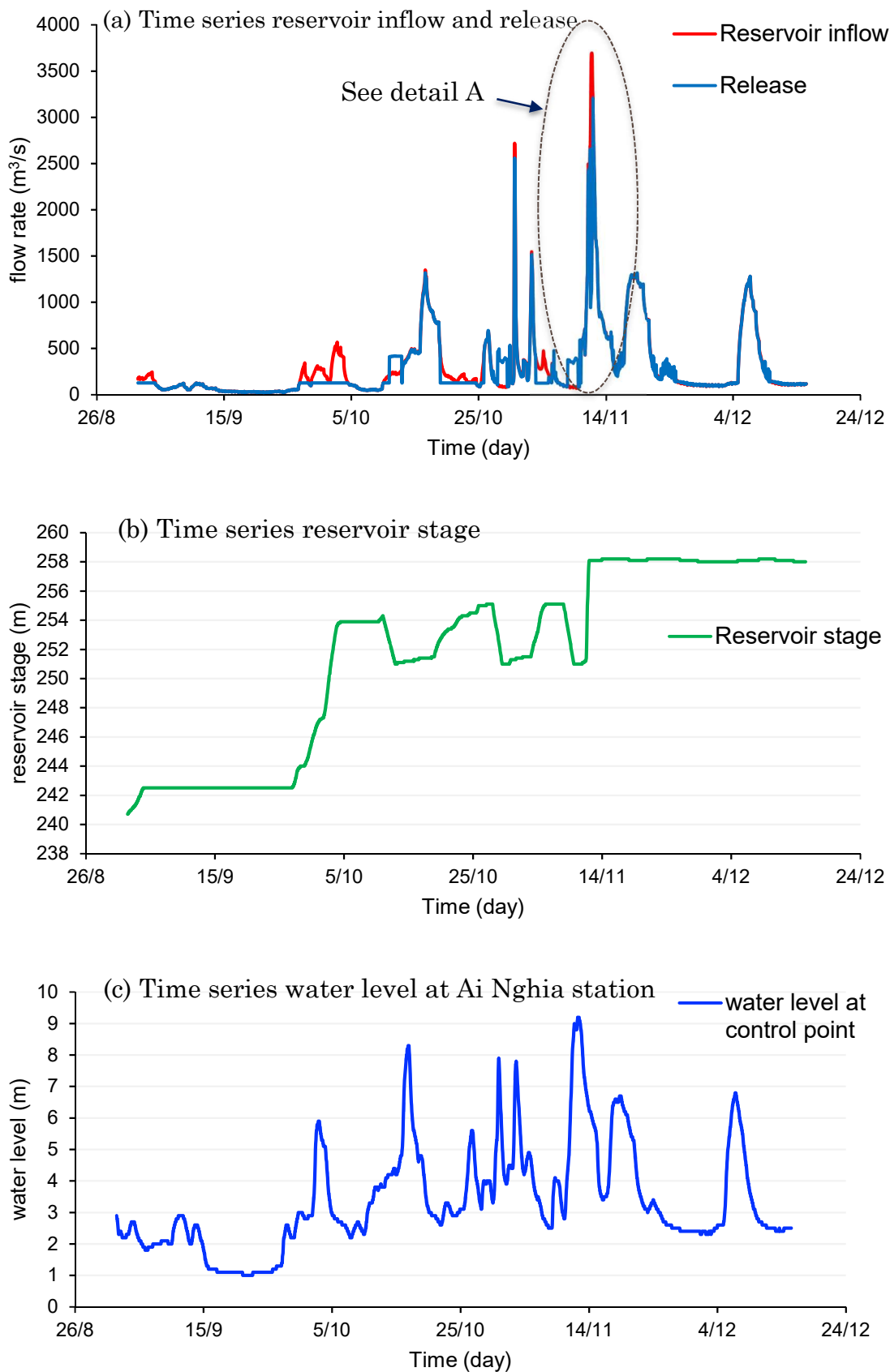


Figure 3.18. Dak Mi 4 reservoir operation in the 2007 flood season.

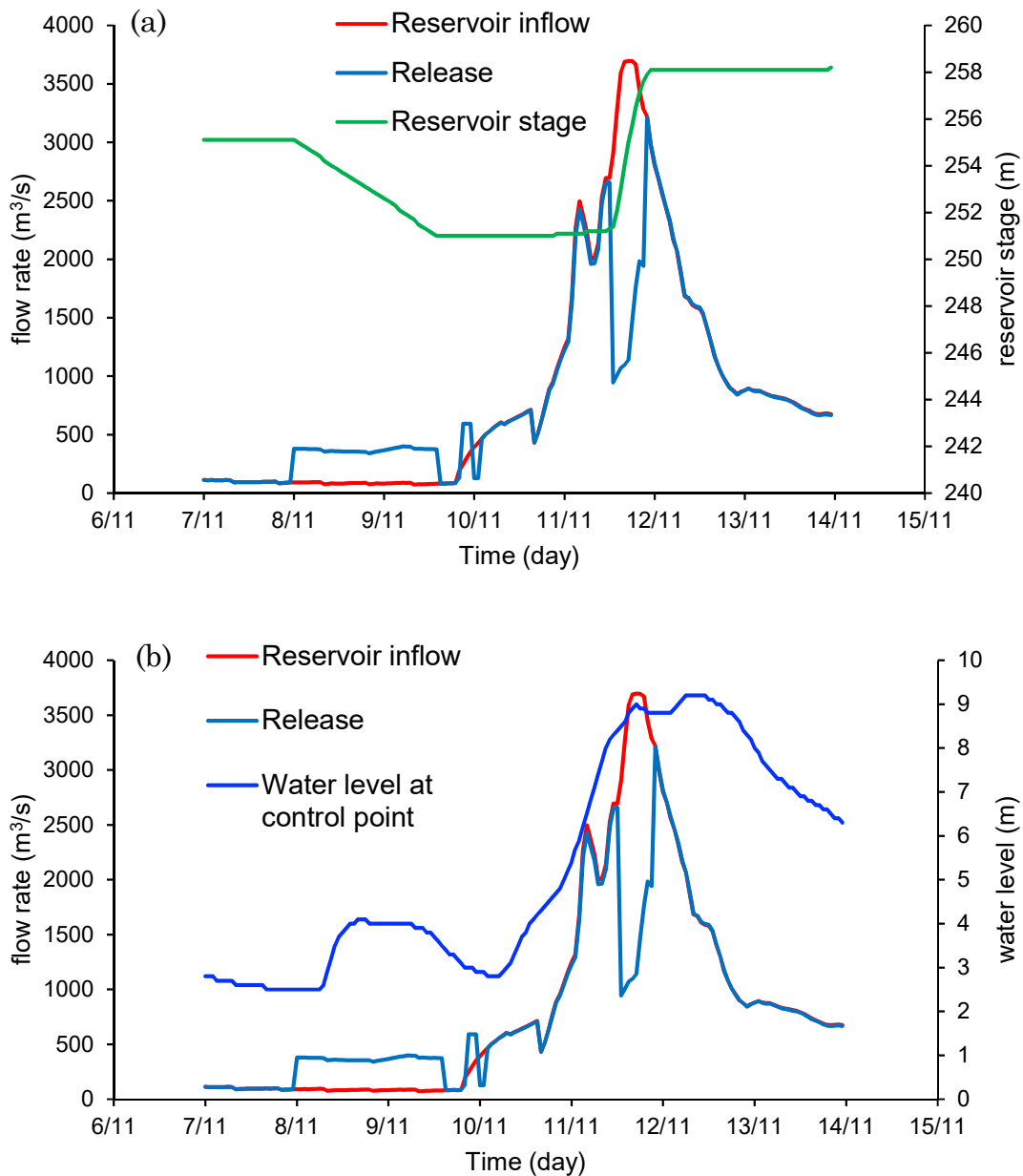


Figure 3.19. Flood control procedure for Dak Mi 4 reservoir in the 2007 flood season (detail A in Fig. 3.18 a).

The most important issues of reservoir system operation in flood season are to reduce downstream flood peak stage and store floodwaters for future uses. Tab. 3.8 summarizes the results of the four reservoirs in reducing maximum water levels and the reduction rate of peak discharge at two control points. For the large flood events (in 2006, 2007 and 2009), the maximum water levels decreased 0.96 m and 0.92 m on average at Ai Nghia and Giao Thuy, respectively. For medium flood events (in 2003, 2004, 2005 and 2008), the reservoir system operated according to the current rules, which are reduced the maximum water level. Still, the effectiveness of the reservoir system on flood reduction was not significant. Because of the current regulation define the release discharge equal reservoir inflow if the water level at downstream is lower than the H_3 (Fig. 3.20).

Table 3.8. Results for the floods event considered.

Year	Scenario	Max water level (m)		Flow peak (m ³ /s)	
		Ai Nghia	Giao Thuy	Ai Nghia	Giao Thuy
2003	WO Res	8.92	7.61	2 320	11 100
	Current rules	8.69	7.41	2 150	10 300
2004	WO Res	9.42	8.64	2 640	15 650
	Current rules	9.11	8.40	2 465	14 600
2005	WO Res	8.13	7.42	1 770	10 266
	Current rules	8.08	7.32	1 728	9 850
2006	WO Res	9.45	7.91	2 815	12 403
	Current rules	8.34	6.81	1 917	8 220
2007	WO Res	10.29	9.36	3 421	19 592
	Current rules	9.22	8.44	2 389	14 654
2008	WO Res	8.41	7.58	1 871	10 788
	Current rules	8.22	7.42	1 764	10 171
2009	WO Res	10.69	9.55	3 973	20 787
	Current rules	10.0	8.8	3 161	16 714

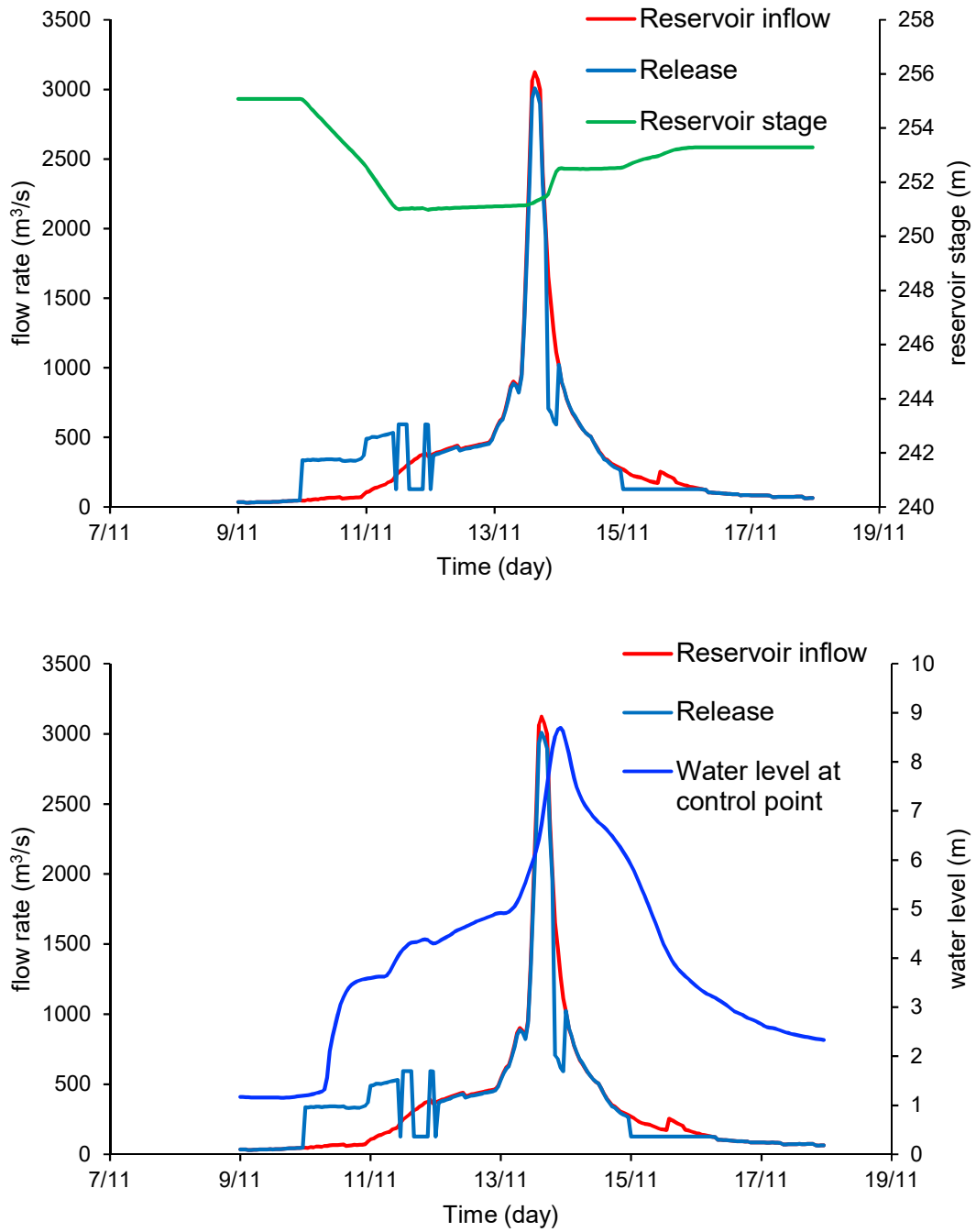


Figure 3.20. Flood control procedure for Dak Mi 4 reservoir in the 2003 flood season.

3.4.5. Conclusion

In the Vu Gia Thu Bon catchment, as the urban areas are located on the coastal plain and in the downstream areas of the hydropower reservoirs, the management of reservoirs can generate an increased risk of flooding during the rainy season.

A novel strategy for flood mitigation in a specific reservoir system with multiple reservoirs and control points is proposed. The simulation results show that the reservoir system operation can reduce efficiently the maximum water level downstream.

In the next chapter, the optimal operation of reservoir systems will be carried out, in which the multi-objective will be considered by using a simulation-optimization framework.

Chapter 4

Optimization models for flood control

The optimization of multi-reservoir system operation is a complex, multi-purpose optimization problem with different objectives, such as irrigation, industrial water supply, hydropower generation, navigation and flood control.

In this research project, the optimal strategies for flood control of the multi-reservoir system in the Vu Gia Thu Bon catchment are defined as core objectives. The problem statement, the state-of-the-art of methods and the selection of optimization techniques are discussed in the current chapter.

4.1. Introduction

Optimization of the operation of a single reservoir or multi-reservoir system is a problem of close interest to the owners of these hydraulic facilities. Many optimization procedures have been developed to analyze the performance of future reservoir systems or to optimize the performance of the existing reservoir systems. Despite the multitude and specificity of each river reservoir system network, the formulations of the associated optimization problems are often in common.

The primary objectives of reservoir operation are irrigation, water supply, flood control, hydropower generation with the variables being the discharge released through spillways or valve devices. These objectives are generally expressed as a function of released discharge. It is, therefore, critical to determining the flow rate for each reservoir of the system and at each time step. In summary, there are concrete objectives targeted by the operation of the reservoirs and their associated variables:

- Maximizing the hydropower generation and/or water supply;
- Minimizing the deviation of hydropower generation from the demand;
- Minimizing flood damages;
- Maximizing reservoir storage at the end of the optimization horizon;
- Minimizing the volume of water released for navigation needs;
- Ensuring minimal discharges for maintaining environmental quality in the downstream area.

Therefore, optimization of reservoir operation with multiple purposes is a complex problem because of the need to tackle various interactions and trade-offs between these objectives. In addition, they are sometimes competitive or conflicting (Lin & Rutten, 2016). For example, in the case of a multi-purpose reservoir, which serves only the hydropower and flood control as key purposes, the operation may be needed for maximum benefits from the generation of hydropower, while the release must be restricted to prevent downstream flooding. The reservoir operators also need to consider several potential trade-off solutions before deciding the one that best suits his needs (Reddy & Kumar, 2006).

One of the main objectives of the research work is the development of a methodology for optimizing for the operations of multi-reservoirs systems under flooding conditions. From a mathematical point of view, an

optimization problem is defined by different components: a model of the system to be optimized, the variables, the constraints, and one or more objective functions. An optimal solution is a vector containing a set of values of the decision variables that satisfy the constraints and yields an optimal value of the objective function.

Flood control and mitigation are significant concerns for the authorities responsible for the reservoir operation in the flood seasons. How to operate the reservoirs so that flood control and mitigation could be maximized are considered a key technology strategy in the flood protection management. Thus the focus is on optimizing the effective use of the available reservoir storage during flood events in particular and during flood seasons in general. Although reservoirs are not primarily built to provide full flood protection, effective use of the reservoir storage capacity can help to reduce the downstream flood levels and mitigate flood damages as well as to prevent major flood disasters.

The optimal operating strategy for a reservoir depends not only on the flood control decision of each reservoir but also on the respective content of each other reservoirs in the network (Christensen & Soliman, 1986).

In this current research, the optimal strategies for flood control of the multi-reservoir system in the Vu Gia Thu Bon catchment are considered. The problem statement, state-of-the-art and selection of optimization techniques are discussed in the next sections.

4.2. Problem statement

The optimization problem contains two main parts: (i) the objective function and (ii) the constraints. Here, the objective function is to minimize the total damages during the flood events that can be expressed as a function of water surface elevations at the inundation zones. For the operating strategy of the multi-reservoir system under flooding conditions, the optimization model is developed to minimize the objective function that is described by minimizing the total maximum water levels and/or discharges. The constraints include hydraulic constraints, physical constraints and operational constraints. The optimization model for the operation of a multi-reservoir system under flooding conditions can be formulated as follows (Unver & Mays, 1990):

4.2.1. Objective function

$$\text{Minimize } z = f(h, Q) \quad (4.1)$$

where:

h and Q are the flow depth and flow discharge at the damage zone, respectively.

4.2.2. Constraints

(a) The hydraulic constraints defined by the Saint-Venant equations for one-dimensional gradually varied unsteady flow and other relationships such as upstream, downstream, and internal boundary conditions in one hand, and in the other hand, initial conditions that describe the flow in the different components of a river-reservoir system:

$$g(I_t, R_t, h_{x,t}, Q_{x,t}, \dots) = 0 \quad (4.2)$$

where:

$I_t, R_t, h_{x,t}, Q_{x,t}$ are inflow, release discharge from the reservoir, flow depth, and flow discharge, respectively. With respect to space x and time t , these variables and their partial derivatives are interrelated via a nonlinear function f .

(b) The bounds on discharges are defined by the minimum and maximum allowable reservoir releases and flow rates at specified locations:

$$\underline{Q} \leq Q \leq \bar{Q} \quad (4.3)$$

(c) The bounds on elevations are defined by the minimum and maximum allowable water surface elevations at specified locations (including reservoir levels):

$$\underline{h} \leq h \leq \bar{h} \quad (4.4)$$

(d) The physical and operational bounds on gate operations are:

$$0 \leq \underline{r} \leq r \leq \bar{r} \leq 1 \quad (4.5)$$

(e) Other constraints, such as operating rules, target storages, storage capacities are:

$$W(r) \leq 0 \quad (4.6)$$

where:

The variables h and Q are the water surface elevation and the discharge at the computational points, respectively.

r is the gate setting, all given in matrix form to consider the time and space dimensions of the problem.

For a particular variable (e.g., h), the bars above (\bar{h}) and below (\underline{h}) denote the upper and lower bounds for that variable, respectively.

4.3. Coupled Optimization-Simulation model

According to the previous description the optimization of multi-reservoir system operation is a complex, multi-purpose optimization problem with different objectives such as irrigation, industrial water supply, hydropower generation, navigation and flood control. In order to achieve multiple objectives, procedures based on coupling simulation models with optimization algorithms have been proposed in many previous studies. Fig. 4.1 shows the framework of the simulation model coupled with the optimization model. The optimization of the operation of a multi-reservoir system during flood events can be formulated as a combination of a simulation model and an optimization model (Bayat et al., 2011).

In this method, the hydrodynamic model is adopted for the simulation of the flow of the river network considering physical constraints of the system as well as operation policies. The optimization algorithms are applied to determine the best set of decision variables, such as reservoir release and storage. For the optimization of the multi-objective function, the optimization tool searches for the set of non-dominated or Pareto-optimal solutions according to the trade-offs between the various objectives (Pedersen et al., 2007).

First, the optimization model produces a release discharge hydrograph based on generating randomly a set of the decision variable (e.g., the method considers reservoir release as a decision variable that needs to be optimized). Next, the simulation model computes the flow discharges and water levels in the river network. After that, the optimization model evaluates the multi-objective functions based on the selected results from the simulation model. Then another set of control variables is generated through the optimization algorithm (Fig. 4.1). This process is repeated until the stopping criteria are satisfied. These include: the iteration number reaches a certain predefined,

or the value of the objective function improves insignificantly over some iterations (i.e., less than 1%).

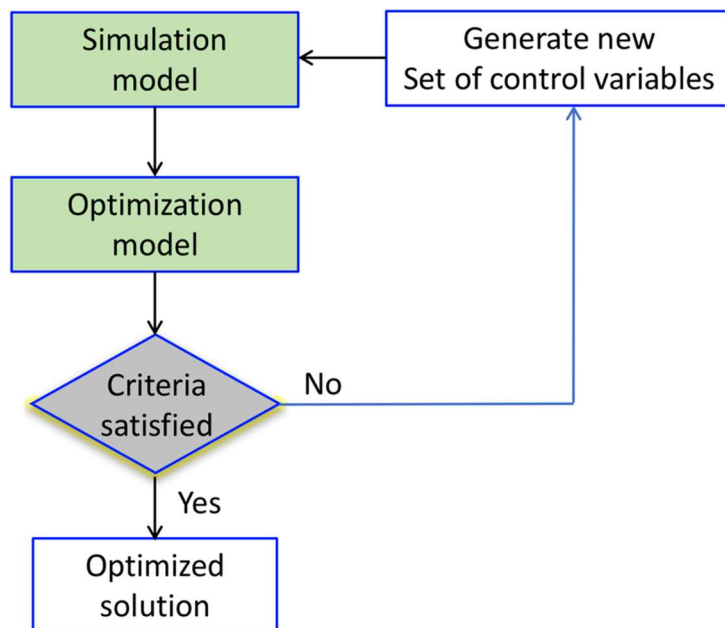


Figure 4.1. Flowchart of the simulation and optimization procedure.

4.4. State-of-the-art: a review

This section presents an overview of the simulation and optimization models that are frequently applied in dealing with the optimal operation of the multi-reservoir system. Many studies have been used in the simulation and optimization models in solving this problem. Examples of works based on simulation models, HEC-5 and Hec-ResSim are to be used for reservoir system simulation. These tools are able to simulate the operation of the reservoir, including water supply, hydropower generation, and flood control. However, hydrodynamic models such as Mike 11, Hec-Ras, and Sobek are more suitable for the comprehensive simulation of flood control operations, to simulate complex changes in water levels of the river network. Many approaches are used in the literature to operate various reservoir system in which combined effectively simulation-optimization modeling for better system performance (Lin & Rutten, 2016).

For the operation of reservoir systems, a variety of optimization models were developed (Adeyemo, 2011; Fayaed et al., 2013; Labadie, 2004; William, 1985). Several researchers have made efforts to control the floods of reservoir systems using optimization techniques, including Linear Programming (LP), Dynamic Programming (DP) and heuristic programming such as Genetic

Algorithm (GA), Non-dominated Sorting Genetic Algorithm II (NSGA-II), Shuffled Complex Evolution (SCE), fuzzy logic and Artificial Neural Networks (ANN).

Linear Programming (LP) refers to a process optimizing a linear objective function subject to a given set of linear constraints. This has been used in determining optimal operating policies and flood control (Needham et al., 2000; Wei & Hsu, 2008). Hsu & Wei (2007) developed a real-time operation model for resolving the optimal release during a typhoon in the Shihmen reservoir system in Taiwan. This model had the objective of minimizing the peak flow at the downstream control points as well as maximizing the reservoir storage at the flood ending for all scenarios.

Dynamic Programming (DP) has been used in the optimization of the reservoir system (Bhaskar & Whitlatch, 1980; Chen et al., 2013). The popularity and success of this technique based on its ability to deal with the non-linearity and stochastic features of the water resource systems. However, the dimension of decision variables is a significant obstacle to solve optimization by using LP and DP (Reis et al., 2005). The limitations that arise from the complexity of simulation models have been overcome by combining simulation models with heuristic search strategy as well as using advances computational techniques.

Genetic Algorithms (GA) and Non-dominated Sorting Genetic Algorithm II (NSGA-II) are stochastic search algorithms based on the principles of natural evolution originated from the Darwin's evolution theory. These two are global search techniques for a solution to non-linear reservoir optimization problems, such as operating rules of the reservoir (Hassaballah et al., 2012; Hinçal et al., 2011; Prakash et al., 2015; Yazdi & Salehi Neyshabouri, 2012a; Zargar et al., 2016), real-time reservoir operation (Li Chiu Chang & Chang, 2001; Che & Mays, 2017; Fallah-Mehdipour et al., 2012).

Li Chiu Chang (2008) proposed a reservoir control flood optimization model using a genetic algorithm (GA) as a search engine for finding the rational release and desired storage during flood events. A penalizing strategy was used for each parameter to guide the GA searching process. The objective function is set to minimizing the released peak for alleviating the downstream damages while the final water level is aimed of approaching the target level.

A modeling methodology was developed by Malekmohammadi et al. (2010) for the real-time optimal flood management of river reservoir systems. This method was based on the interfacing Genetic Algorithm (GA), which, based on the optimization model for reservoir operation, estimates the optimal hourly reservoir release, with the unsteady module of Hec-Ras flood routing using Hec-GeoRAS. The model's objective function was based on minimizing flood damages in a system of cascade reservoirs. The optimization model was formulated for the operating rules in real-time using the K-NN algorithm that searches through the historical flood and optimal reservoir operation determined by the optimization model to find a similar situation. The real-time model was successfully applied to the Bakhtiari and Dez river reservoir systems in the southwest of Iran.

A real-time model using simulation-optimization was developed by Che & Mays (2015) for determining reservoir release schedules before, during, and after an extreme flood event. This model included a hydraulic model (HEC-RAS) for one-dimensional unsteady flow routing, a reservoir release operation model and a short-term rainfall forecasting model. The reservoir release schedules for a flood event for a multi-reservoir system were formulated and solved using a genetic algorithm to minimize the flow rate of control points downstream.

Kim & Heo (2006) applied the second-generation evolutionary multi-objective technique - NSGA-II - to find the Pareto optimal solutions to a multi-reservoir system optimization problem with multi-objectives in the Han river basin in Korea. The objective functions are defined as minimizing storage and release for each reservoir regarding the water supply downstream.

Yazdi & Salehi Neyshabouri (2012b) proposed a simulation-optimization framework by coupling the Mike 11 simulation model and the NSGA-II multi-objective optimization model to optimize the design of multi-reservoirs for flood control. The Mike 11 hydrodynamic model was used for the simulation of different flood scenarios with different return periods in the rivers. The NSGA-II algorithm was applied for developing a Pareto front trade-off objectives between the investment cost and flood damage cost. The proposed model suggested the optimal designs can efficiently reduce construction costs, flood peaks corresponding damage costs at the downstream.

A multi-objective simulation-optimization framework for reducing damage in a system of two parallel reservoirs and control points was proposed by

Prakash et al. (2014). This framework took into account different combinations of objectives for flood mitigation, such as maximizing the empty space available in the reservoirs, minimizing the cumulative flood volume over the channel capacity at control points as well as minimizing the peak flow at the control points. The NSGA-II algorithm was used as the solution technique to optimize the operating strategy for the multi-reservoir system.

Amirkhani et al. (2017) applied NSGA-II to optimize the operation of spillway gates for the Karkheh reservoir in Iran. This method proposed that the opening of the gates is proportional to the water level of the reservoir for minimizing downstream damages while reducing the probability of dam overtopping.

Particle Swarm Optimization (PSO) is a simple model that uses the mechanism of learning swarm behavior in flocking birds and fishing to direct the particles in their quest for optimal solutions globally (Wahab et al., 2015). An optimization-simulation model was developed by Bayat et al. (2011) for short-term reservoir operation under flooding conditions. The problem was formulated as a combination of particle swarm optimization algorithm and a hydraulic simulation model of river flood routing such as Muskingum, Muskingum-Cunge, and Saint-Venant algorithms. The purpose of the model was to determine the optimal reservoir releases for minimizing flood damages.

Keophila et al. (2018) studied the multi-objective optimization for flood control and hydroelectric generation of two cascade hydropower plants in central Laos. The operation model computed the water release using Hec-ResSim and multi-objective optimization for minimizing flood damages while maximizing electricity production using the Multi-Objective Particle Swarm Optimization algorithm (MOPSO).

The Shuffled Complex Evolution (SCE) algorithm synthesizes the best features of several existing algorithms, including GA, and the complex shuffling. This algorithm has been applied in calibrating various hydrology models (Duan et al., 1994) and in finding reservoir optimal operation solution (Bakhtiari Nezhad et al., 2018; Nezhad et al., 2018; Richaud et al., 2011). Ngo et al. (2007) was proposed a combination of simulation and optimization models to optimize control strategies for reservoir operation. The Mike 11 simulation model was established for control strategies and the SCE algorithm was adopted for optimizing strategies operation. The purpose of the

model was to minimize the downstream flood peak and maximizing hydropower generation during the flood season as well as reservoir water elevation at the beginning of the dry season. The results showed a Pareto front on the trade-off between flood control and hydroelectric power generation for the Hoa Binh reservoir, in Vietnam.

The fuzzy logic provides an effective solution for nonlinear and partially unknown processes. A fuzzy controller is formed by input and output fuzzy sets assigned over the controller input (storage level, estimated inflows, demands) and output variables (release from a reservoir) and a collection of fuzzy rules. Karaboga et al. (2008) used an operation method based on the fuzzy logic control for the operation of reservoir spillway gates during floods. A model designed for optimal operation of reservoirs system by Mousavi et al. (2005) using a dynamic programming fuzzy rule-based. Chang et al. (2002) developed a grey fuzzy stochastic dynamic programming for determining reservoir operating strategies. This approach is used to improve the operations of the Shiman reservoir in Taiwan.

An artificial neural network (ANN) is “a massively parallel-distributed information-processing system that has certain performance characteristics resembling biological neural networks of the human brain” (Haykin, 1994). One of the main uses of ANN in the optimization of reservoir systems is to determine optimal rules from implicit stochastic optimization (Labadie, 1997). Chang et al. (2010) established a real-time reservoir operational model based on fuzzy and neural network systems for flood control.

The Model Predictive Control (MPC) has been developed for real-time control of a dynamic system. Lin et al. (2018) used the MPC strategy for real-time control of a reservoirs system in Myanmar. A simulation-optimization framework was developed with three main components: the SACramento Soil Moisture Accounting model (SAC-SMA) for inflow prediction, MPC approach for real-time control of a multi-reservoir system and SOBEK hydrodynamic software package for flow simulation of a river system.

Over the years, many optimization models for reservoir operations have been proposed in the literature, including flood control as one of the objectives. The reservoir flood control studies mentioned in those papers concentrated only on the optimal operation aspect of a single or multi-reservoir system. However, none of these models have capabilities like the model that is proposed in this research.

In the current research, due to the characteristics of the system and the targeted optional objectives, a flood control operating strategy has been developed based on coupled simulation-optimization to reduce downstream flood damage of the multi-reservoir system by using spillway gates. This model is applied to the optimal operation of a parallel reservoirs system with limited storage capacity in the Vu Gia Thu Bon catchment. The Mike 11 hydrodynamic model is utilized for flood routing in the reservoirs systems and the Shuffled Complex Evolution (SCE) algorithm for the optimization of operating spillway gates.

The SCE algorithm is one of the techniques that are robust optimization techniques to find the global optimum solution of complex problems with many functions such as non-convex, non-differentiable, and multi extrema functions (Ngo et al., 2007). In this research, the SCE algorithm, as implemented in the AutoCal (DHI, 2017) software, is adopted for optimizing the multi-reservoir system operation in the case study. A brief overview of the SCE algorithm is provided in the next section.

4.5. SCE algorithm

4.5.1. SCE algorithm description

The shuffled complex evolution method is a global searching algorithm developed by Duan et al. (1992) at the University of Arizona. This algorithm is based on the genetic algorithm and introduced the new concept of complex shuffling. The method combines four concepts: “combination of deterministic and probabilistic”; “systematic evolution of a complex of points spanning the parameter space”; “competitive evolution” and “complex shuffling” (Duan et al., 1993).

The SCE method first generates an initial sample of the size $s = p \times m$, where p is the number of complexes and m is the number of points in each complex. A sample of points are randomly generated from the feasible parameter space defined by the lower and upper limits of each parameter, and the objective function value is calculated. After that, the sample of s points is sorted in order of increasing objective function. Then, the s points are divided into p complexes and each sub-complex is evolved according to the simplex algorithm (Nelder & Mead, 1965). The next step, shuffling and combining the points in the evolved complexes into a single sample population. The partition

and shuffling method are repeated until one of the criteria for termination are satisfied.

The SCE algorithm is illustrated in Fig. 4.2 with the following steps:

- **Step 1.** The algorithmic parameters initialization: p and m , where p is the number of complexes ($p \geq 1$), and m is the number of points in each complex ($m \geq n + 1$, where n is the number of parameters sets or sets of decision variables). Compute $s = p \times m$ where s is the sample size.
- **Step 2.** Generate sample: a sample s points of $x_i \{x_1, x_2, \dots, x_s\}$ in the feasible space and compute the objective function value f_i at each point x_i .
- **Step 3.** Rank points: sort the s points in order of increasing objective function value so that the first point represents the point with the smallest function value (best point) and the last point represents the point with the largest objective function value. Store them in an array $D = \{x_i, f_i, i = 1, 2, \dots, s\}$.
- **Step 4.** Partition into complexes: partition D into p complexes A_1, A_2, \dots, A_p , each complex containing m points, such that the first complex contains every $p(k-1) + 1$ ranked point, the second complex contains every $p(k-1) + 2$ ranked point, and so on, where $k = 1, 2, \dots, p$.
- **Step 5.** Evolution: evolve each complex A_k according to the competitive complex evolution (CCE) algorithm.
- **Step 6.** Complex shuffling: replace A_1, A_2, \dots, A_p , into array D , such that $D = \{A_k, k = 1, 2, \dots, p\}$. Sort D in order of increasing objective function value.
- **Step 7.** Check convergence: if one of the criteria for termination is satisfied, stop; otherwise, returns to step 4.

The SCE method is illustrated graphically in Fig. 4.3 by using a two-dimensional example. The function includes one global optimum located at (4,2) and one local optimum located at (1,2). The sample population includes s points (in this case, $s = 10$) divided into two complexes; each complex contains m ($m = 5$) members, marked by * and •, respectively. In the first evolution cycle, one community is converging toward the local optimum, and the other is converging toward the global optimum. The two evolved complexes are shuffled according to the procedure in step 6, and the new memberships of the two evolved complexes after shuffling are displayed in

Fig. 4.3c. In the end, the second evolution cycle, the locations of the members of the two evolved complexes are shown in Fig. 4.3d. The description of the evolution in step 5 taken by each complex is illustrated graphically in Fig. 4.4 (Duan et al., 1994).

One of the two methods of optimization implemented in the AutoCal software is the SCE algorithm. This tool is introduced in the next section.

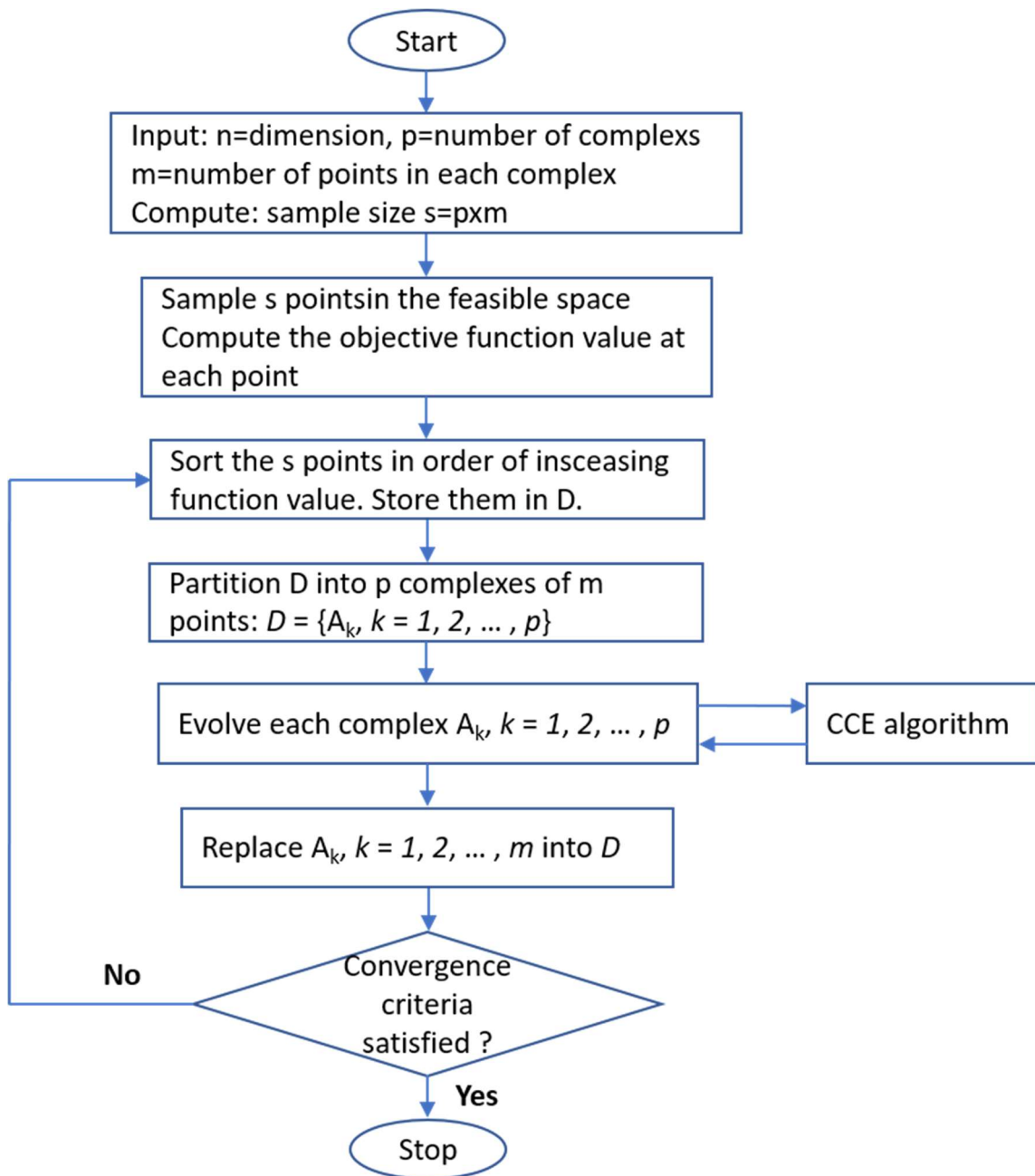


Figure 4.2. Flowchart of the SCE method (Duan et al., 1993).

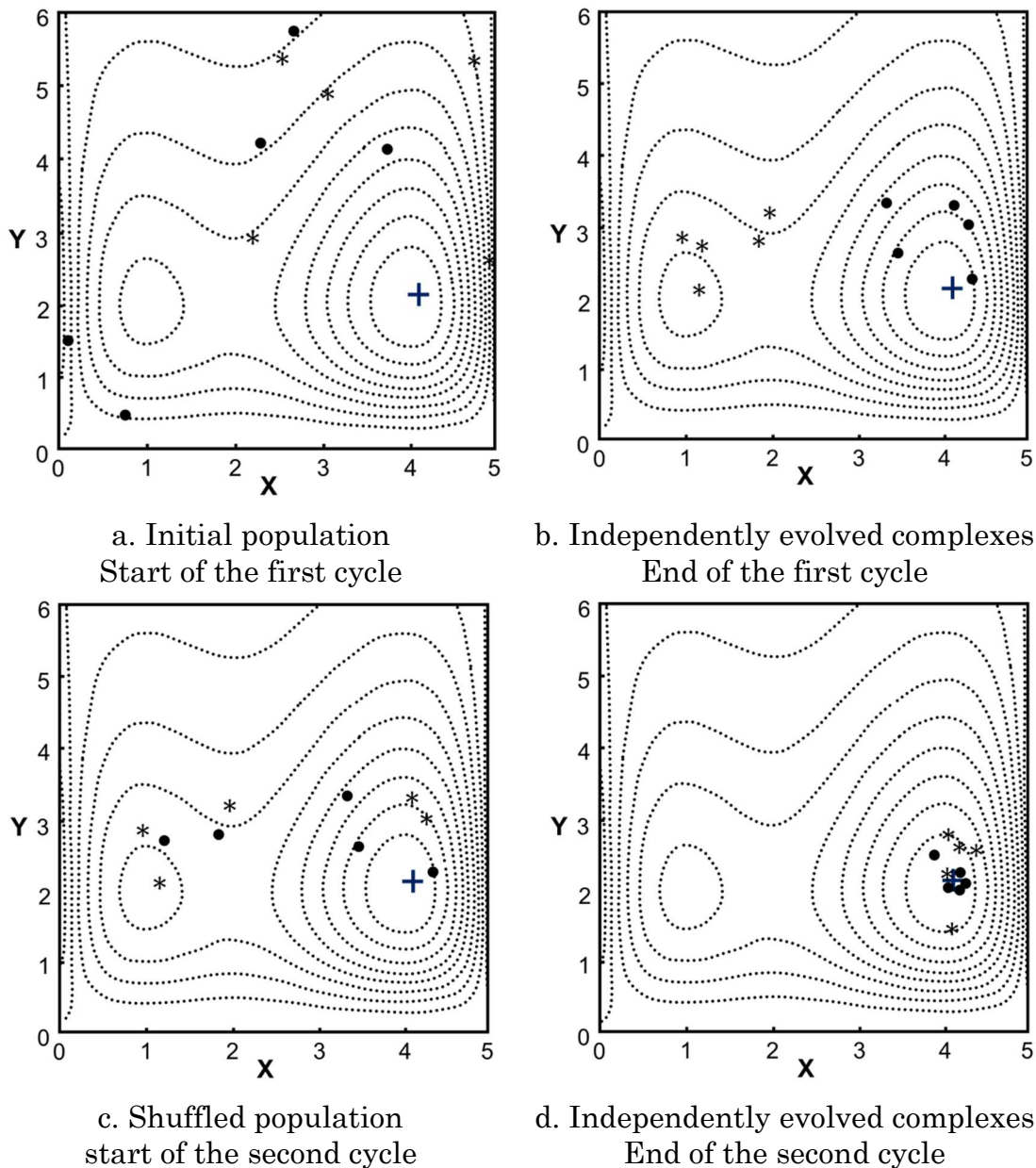
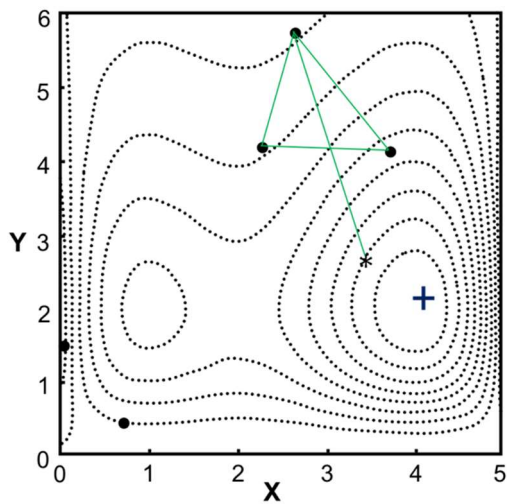
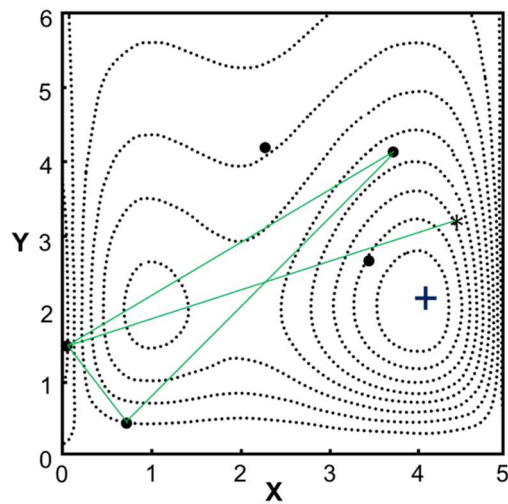


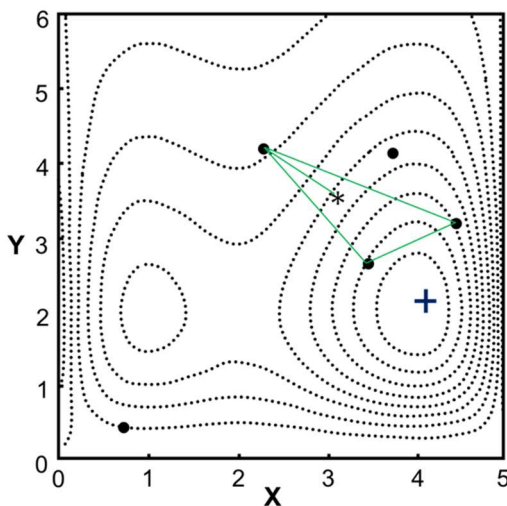
Figure 4.3. Illustration of the SCE algorithm. Extracted from (Duan et al., 1994).



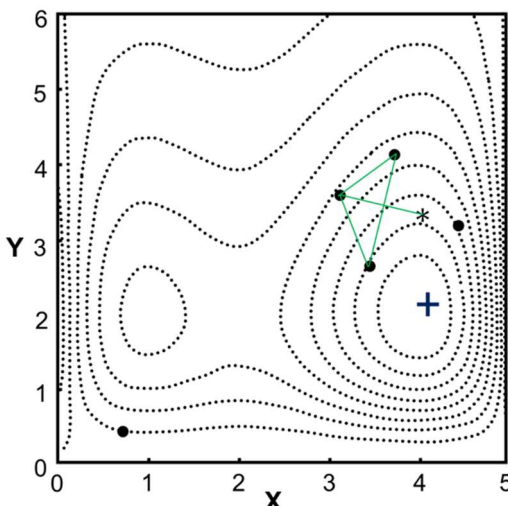
a. Generation of the first offspring



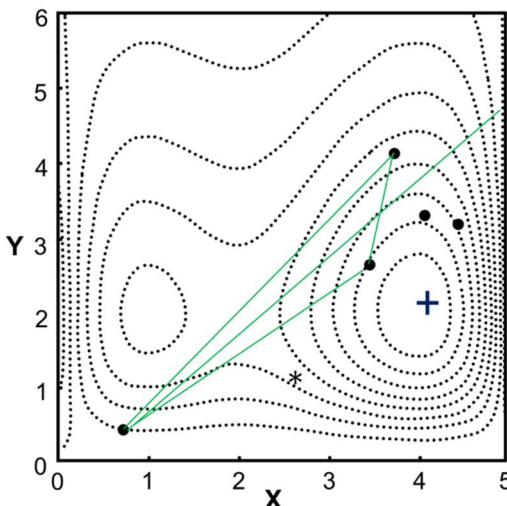
b. Generation of the second offspring



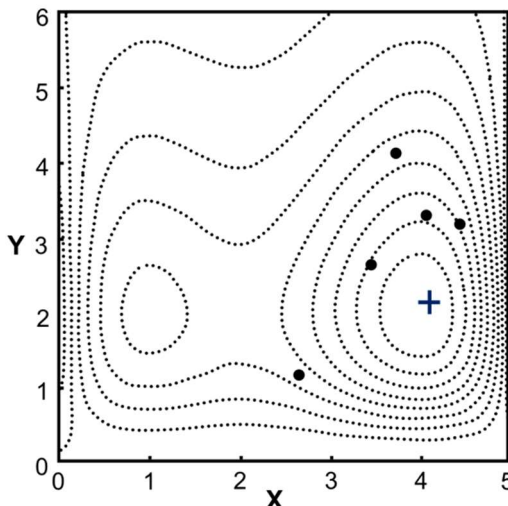
c. Generation of the third offspring



d. Generation of the fourth offspring



e. Generation of the fifth offspring



f. Evolved complex after the end of five steps

Figure 4.4. Illustration of the SCE algorithm. Extracted from (Duan et al., 1994).

4.5.2. AutoCal description

AutoCal (Auto Calibration) software is a generic tool for performing automatic calibration, parameter optimization, sensitivity analysis, and scenario management of the numerical modeling engines. The framework of the AutoCal tool process is shown in Fig. 4.5.

In the first step, all files in the simulation model that include model parameters to be performed in the AutoCal should be specified. The simulation model executed using the set of parameters which restricted to feasible parameter space. Whether AutoCal is used for parameter optimization, sensitivity analysis, or scenario management, the performance of the model simulation given the specified parameter set should be assessed by calculating statistical performance measures. These measures are typically statistics that compare measurements or, in general, target value with corresponding simulated values. After that, the objective function is evaluated by checking the convergence criteria. If one of the criteria for termination is satisfied, stop; otherwise, return to execute the simulation model with a new set of control variables generating by the SCE algorithm. The algorithmic parameters of the SCE algorithm, their feasible range, and recommended values (Duan et al., 1994) are shown in Tab. 4.1.

Table 4.1. Algorithmic parameter for the SCE algorithm.

Parameter	Description	Range	Recommended value
p	Number of complexes	$p \geq 1$	-
m	Number of points in a complex	$m \geq 2$	$2n + 1$
q	Number of points in a sub-complex	$2 \leq q \leq m$	$n + 1$
β	Number of evolution steps taken by each complex before shuffling	$\beta \geq 1$	$2n + 1$

(n is the number of optimization parameters).

The algorithmic optimization runs until one of the stopping criteria is satisfied. Three stopping criteria are defined as presented below:

- The maximum number of model evaluation;
- Convergence on objective function space. In this case, the optimization model will terminate if the objective function of the best

control variables has not changed more than a user-defined minimum value in a given number of shuffling loops;

- Convergence in parameter space. In this case, the optimization model will terminate if the range of parameter values of the entire population in the parameter space is less than a given value.

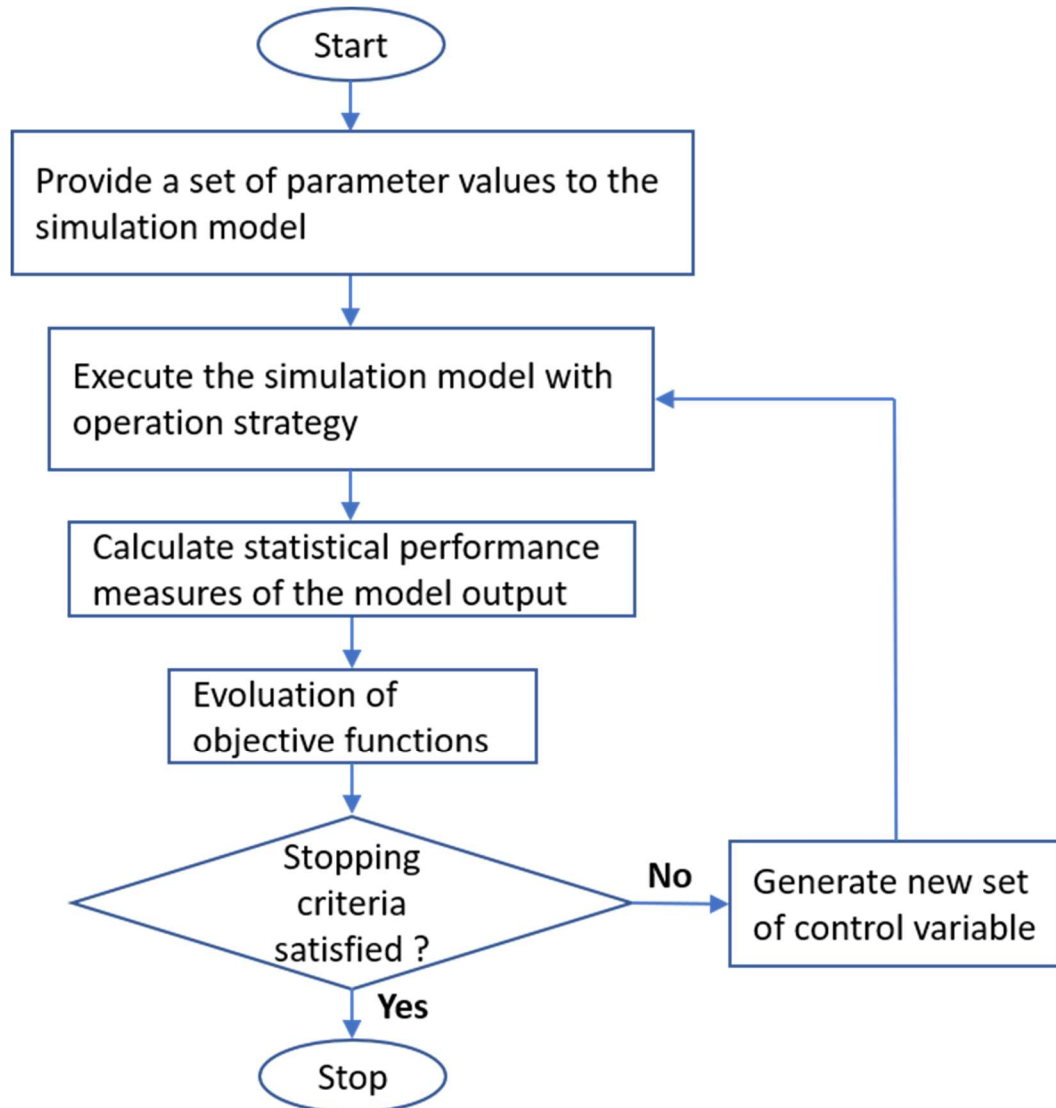


Figure 4.5. Flowchart of the AutoCal software.

In AutoCal, the statistical performance measures require that simulation results and the corresponding observations are given as time series. This software includes three basic comparison statistics:

- Average error (Avg. Error):

$$AE = \frac{1}{n} \sum_{i=1}^n w_i (Sim_i - Obs_i) \quad (4.7)$$

- Root mean square error (RMSE):

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n w_i (Sim_i - Obs_i)^2} \quad (4.8)$$

- Standard deviation of residuals (St.Dev.):

$$STD = \sqrt{\frac{1}{n} \sum_{i=1}^n w_i (Sim_i - Obs_i - AE)^2} \quad (4.9)$$

Where Obs_i and Sim_i , $i = 1, 2, \dots, n$ are the observed and the corresponding simulated time series, respectively. The index i presents the time step, and n is the total number of time steps. A user-specified weight is w_i .

The statistic AE is a measure of the general offset between targets and simulations (bias), whereas STD is a measure of the dynamical correspondence. RMSE is an aggregated measure that includes both bias and dynamical correspondence.

Besides the basic statistics, AutoCal includes two event-based statistics:

- Error of maximum value (Error of max.):

$$ErrMax = w_i (Max\{Sim_i\} - Max\{Obs_i\}) \quad (4.10)$$

- Error of minimum value (Error of min.):

$$ErrMin = w_i (Min\{Sim_i\} - Min\{Obs_i\}) \quad (4.11)$$

The maximum and minimum observed and simulated values are extracted in the period defined in the target file.

AutoCal uses three different functions for aggregation of the performance statistics:

- Weighted sum:

$$F_{pool,i} = \sum_{j=1}^n F_j \quad (4.12)$$

- Weighted sum of absolute values:

$$F_{pool,i} = \sum_{j=1}^n |F_j| \quad (4.13)$$

- Weighted sum of squares:

$$F_{pool,i} = \sum_{j=1}^n F_j^2 \quad (4.14)$$

Where F_j is the output measure or comparison statistic and n is the number of measures or statistics that are pooled.

The multi-objective optimization problem explores the entire Pareto front between the objective functions by performing several optimizations runs using different weights. The weight allocated to the objective function in the combination of the various objective functions to be transformed into one aggregate calculation. Depending on the specific model application being considered, the assigned weights should reflect the relative priorities given to the different objectives.

A simulation-optimization framework for the optimization of a multi-reservoir system in the Vu Gia Thu Bon river basin is introduced in the next chapter.

Chapter 5

Optimal operation for a multi-reservoir system in the Vu Gia Thu Bon catchment

In the previous chapters, a simulation model and an optimization tool were presented. The performance of the simulation model has been demonstrated through its application to the river reservoir system in the Vu Gia Thu Bon catchment, but the optimization model has not yet been applied within this catchment.

The main objective of this chapter is to present the functioning of coupling between simulation and optimization models for operating the reservoirs system in this catchment. Also, it demonstrates the effectiveness of the model applied to four major reservoirs hydroelectric that is validated by comparison with the current rules method.

5.1. The optimal model of multi-reservoir for flood control in the Vu Gia Thu Bon catchment

5.1.1. Introduction

The multi-reservoir system in the Vu Gia Thu Bon catchment plays an important significant role in the production of annually alternative electrical and mitigation of flood damage. However, the reservoir system usually faces severe flooding that results from natural characteristic and hydro-climatic conditions within the basin.

This catchment is often hit by typhoons and inshore tropical depression, which brings extreme precipitation. Due to typical characteristics of the rives such as shorts and steeps with narrow valleys, steep riverbanks with many waterfalls, so as soon as a typhoon strikes, the upstream watershed receives voluminous rainfall in a short time that quickly converges downstream when storm landing. Hence it can easily lead to flooding in the lowland area causing considerable economic losses and casualties.

Terrain conditions limit flooding prevention capacity of hydropower plant reservoir in the Vu Gia Thu Bon river basin. The fact that the steep slope of the mountain topography significantly limits the storage capacity of the reservoirs in this catchment area. The total active storage capacity of four major reservoirs is 1,180 million m³ while the maximum 5-day flood volume is 2,890 million m³ at Nong Son station and 1,176 million m³ at Thanh My station (the flood event in 2009). As a consequence, the optimization of the multi-reservoir operation in flood season plays a significant role in flooding control in this catchment.

In the flood season, the multi-reservoir system is operated in the following order of priority: safe protection of the dams, flood mitigation and hydropower generation. The operational problems of the rivers and multi-reservoir system are more likely to increases which result from developing conflict objectives and the number of reservoirs. The occurrence of flooding damage will easily elicit public outrage, which further increases the stress of decision-makers during operating flood control measures. The analysis of multi-reservoir system operation typically involves optimization in simulation models that can provide quantitative information to improve operational water management.

There are four major hydropower reservoirs of A Vuong, Song Bung 4, Dak Mi 4, and Song Tranh 2 (“song” mean river in Vietnamese) in the river catchment. Fig. 5.1 illustrates the schematic of the Vu Gia Thu Bon river multi-reservoir system. The runoffs from each river basin are directly discharged into their perspective reservoir. The releases from the reservoirs are then routed through two main rivers – the Vu Gia and Thu Bon rivers. Towards the downstream area, there is a link between the two rivers through the Quang Hue river in the downstream part of the catchment.

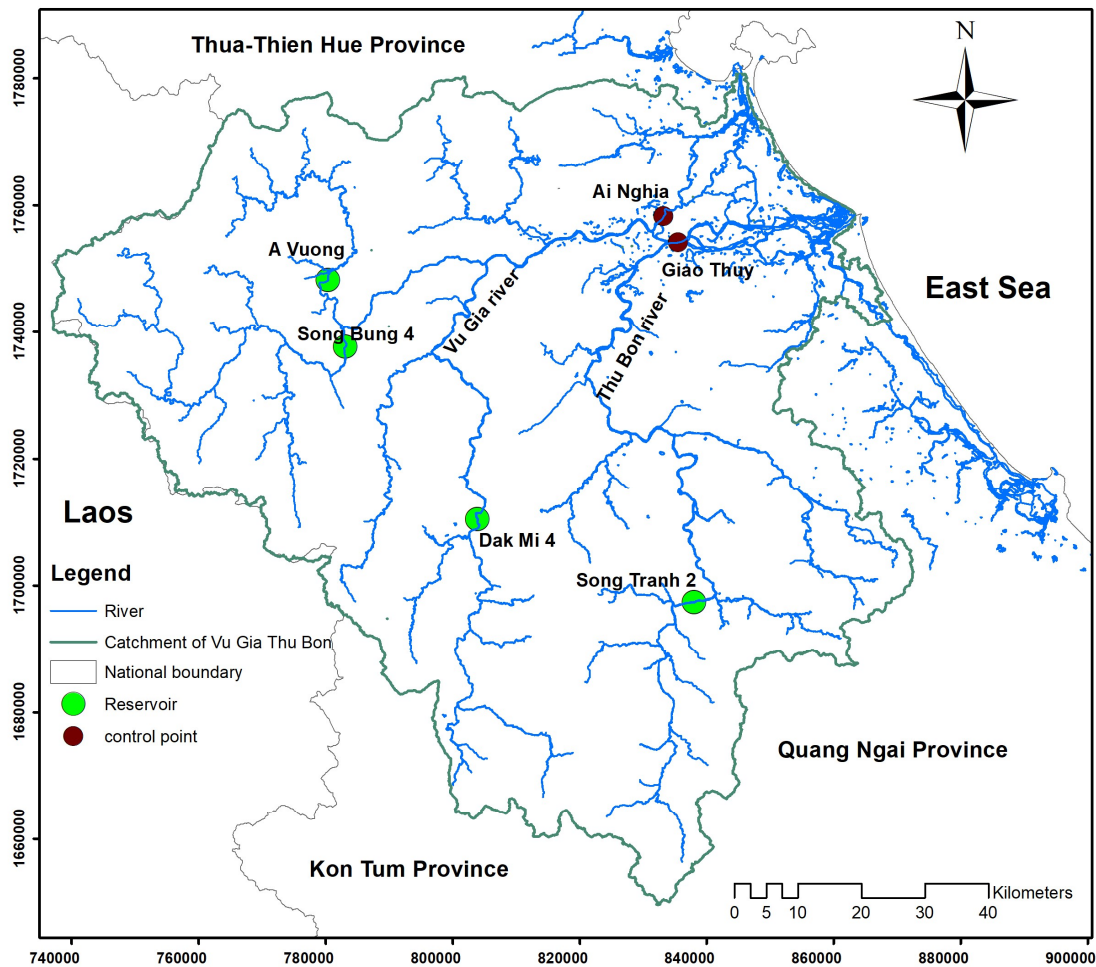


Figure 5.1. Description of the Vu Gia Thu Bon river-reservoir system.

During flood events, the operation of spillway gates is one of the main issues in dams management in the Vu Gia Thu Bon catchment. On one hand, if spillways opened larger than it should be, a large amount of flow resulting in damage will be released to downstream of the reservoir. On the other hand, if they are not opened sufficiently, the dam’s safety can be seriously threatened (Zargar et al., 2016). This research’s objective is to establish the

multi-reservoir operation strategy during the flood season, considering the optimal operation of spillways.

5.1.2. Three-flood-stage operation strategies

Research concerning the operation of the flood control reservoir considers the operation of spillway gates have been developed in recent years (Alvaro et al., 2016; Amirkhani et al., 2017; Haktanir et al., 2013; Karaboga et al., 2008). Some of the most challenging problems in reservoir operation are also optimum regulation of spillway since the optimal results of reservoir operation are primarily in the form of a release discharge, which is seldom used to specifically guide the reservoir operation (Liu et al., 2017). The optimum operating strategy for a reservoir depends not only on the flood control content of each reservoir but also on the corresponding content of each other reservoirs in the system (Christensen & Soliman, 1986). This research focuses on defining stages of spillway gates for the specified stage level in the four reservoirs in the Vu Gia Thu Bon catchment.

According to the reservoir operation guidelines for the multi-reservoir in the Vu Gia Thu Bon catchment, the general framework of reservoir release includes three flood stages: pre-release procedure (prior to flood arrival), flood control procedure (preceding flood peak) and post-flood procedure (post-peak). These steps regulation of reservoir system are described as chapter 3. The guidelines for the first and third stages are clear, while the second stage is difficult for operators.

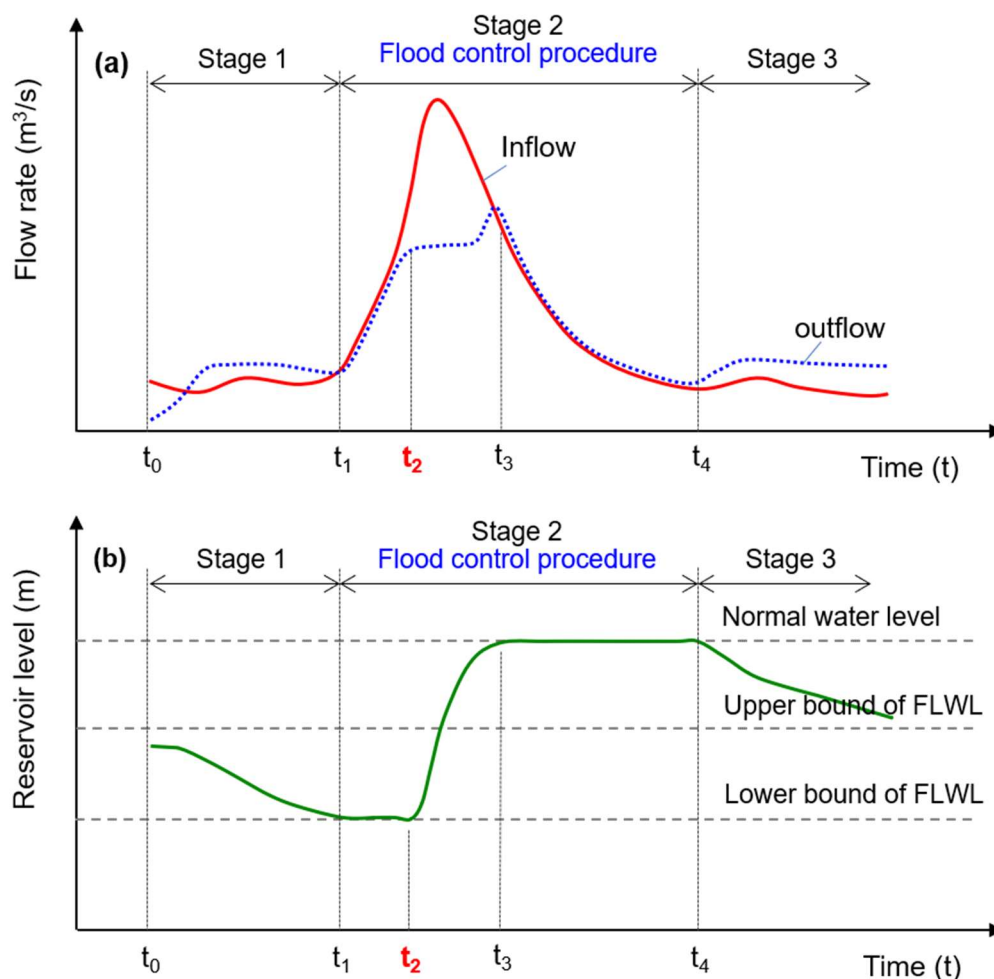


Figure 5.2. Definition of the three flood stages.

Following these guidelines, in the second stage - flood control procedure, the released amount is less than the reservoir inflow. Also, this procedure would begin when the water level at downstream control points greater than a value of specified H_3 level at the time t_2 (Fig. 5.2).

In the current research work, the optimization model provides an operation strategy of reservoir systems with multiple spillway gates to minimize flooding at the control points downstream.

To describe the methodology, let RL_{ij} be a critical reservoir level, in which the subscripts i and j represent reservoir number and critical level number, respectively. In the present study, the number of reservoirs is four of A Vuong, Dak Mi 4, song Bung 4, and song Tranh 2, respectively ($i = 1, 2, 3$, and 4). The number of critical levels in each reservoir depends on various Flood Limit Water Level (FLWL) and the normal water level of their reservoir ($j = 1, \dots$,

k). The increments of the critical level in all reservoirs defined to be equal to 1 meter ($e = 1$).

After selecting of critical reservoir levels, the optimization model defined stage levels of spillway gates (control variables) GL_{ij} to release discharge (Fig. 5.3). These control variables use a set of parameters that restricted to feasible parameter space by user-defined. To describe the beginning time of regulating flood control, let H_m be water levels at control points downstream corresponding T_m (where m represents control points at Ai Nghia station and Giao Thuy station).

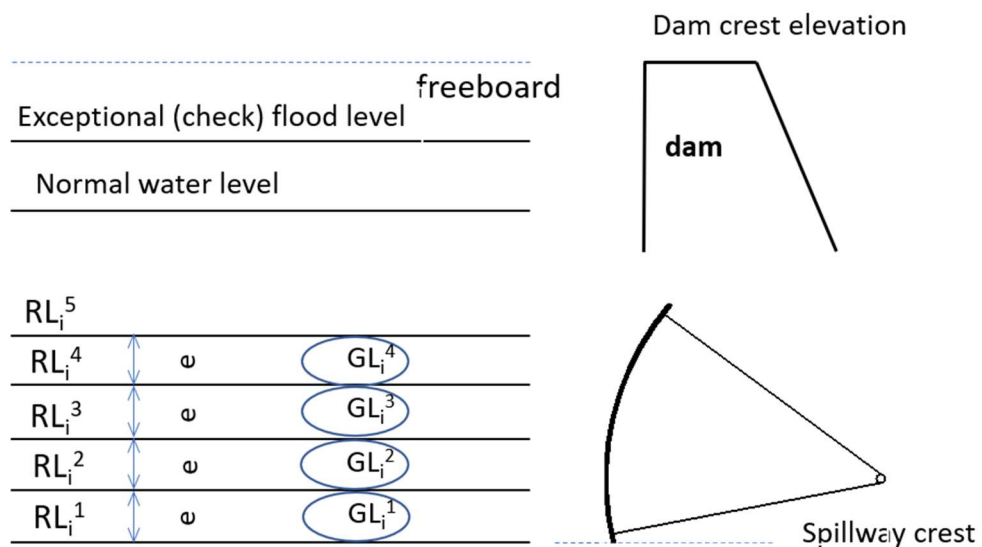


Figure 5.3. Multistage parameters method scheme.

The main priority in determining these parameters is to reduce floods hitting downstream meanwhile prevent water level in reservoir not exceeding the water level checked for all flood.

5.1.3. Architecture of simulation and optimization coupling

One of the main objectives of this research is the development of a methodology for optimizing the operation of the multi-reservoir system with multiple spillway gates in the Vu Gia Thu Bon catchment during flood events. Using different spillway gates has a significant impact on reservoir operation during the flood events (Liu et al., 2017). The problem is formulated as an optimal control problem in which spillway gates levels of four major reservoirs represent the decision variables. Forty-two decision variables are used in the model to represent spillway operation.

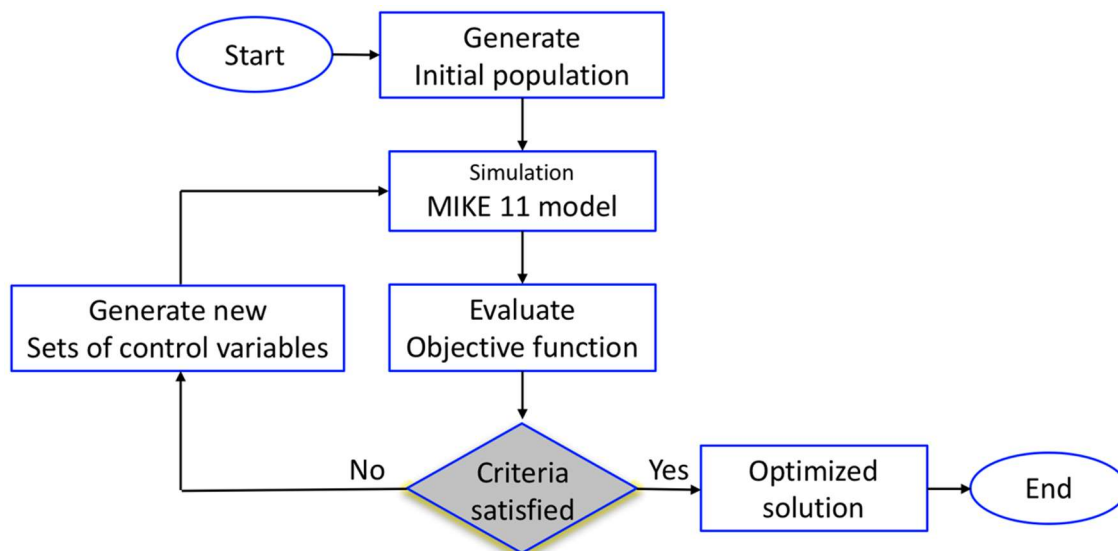


Figure 5.4. Basic steps of the Simulation-optimization framework.

The simulation-optimization framework that is adopted for determining the spillway gates operation strategy of the multi-reservoir system in the Vu Gia Thu Bon catchment during flood events is illustrated in Fig. 5.4. Once the sets of feasible solutions (spillway gates levels) are determined, decision variables are inputted into the Mike 11 hydrodynamic model to simulate the floods in the river systems. The main objective of the method is to control the flood elevations at critical locations downstream of a river-reservoir system (at Ai Nghia and Giao Thuy stations). The maximum water levels at downstream control points are objective functions for minimizing the vulnerability of flood in the downstream area. If the objectives were not met, the model would repeat its optimization process using the SCE algorithm. Which means that a new set of control variables is generated for the simulation model. The processes repeat and continue until the stopping criteria are satisfied.

5.1.4. Stopping criteria

In the SCE algorithm, the number of complexes p is the most important parameter (DHI, 2017). Sensitivity tests show that the dimensionality of the optimization problem is the primary factor determining the proper choice of p . In general, the larger value of p is chosen for higher probability of converging into the global optimum, but at the expense of a larger number of model simulations (the number of model simulations is virtually proportional to p), and vice versa. In this case, the number of complexes p defined to two

complexes for trade-off balancing between the robustness of the algorithm and computing time.

Stopping criteria of the SCE algorithm should be selected carefully because these parameters influence in convergence conditions and calculating time. These parameters are found by a trial and error procedure represented as the Tab. 5.1, where:

- The maximum number of model evaluations allowed in the optimization: nb model;
- The number of loops of convergence. The number of iteration loops in which the objective function value of the best parameter set has not changed more than “the minimum relative change in objective function value”: nb of loops;
- The minimum relative change allowed in the best objective function value in the last “the number of loops of convergence”: relative change.

In order to decide the proper parameter for the stopping criteria, a different kind of criteria concerning the objective function value is used in this study. Tab. 5.1 shows the objective function for different model evaluations performed during the optimization with alternative stopping criteria considered. The objective function value is generally smallest when the number of model evaluations is 2000, and even when the number of model evaluations is increased (up to 3000), there are no improvements significant in the sense of objective value (Fig. 5.5).

Table 5.1. Results for the stopping criteria considered.

N ^o	nb model	nb of loops	Relative change	Objective function
1	1000	3	0.1	150.66
2	2000	3	0.1	149.93
3	2000	3	0.05	149.72
4	2000	3	0.01	149.41
5	2000	5	0.05	149.41
6	2000	5	0.01	149.06
7	2000	10	0.01	149.06
8	3000	10	0.01	148.93

The SCE parameters following: maximum number of model evaluations is 2000; the number of iteration loops is 3; minimum relative change in the objective function is 0.01, were selected. Each routing model or fitness evaluation lapses about 3 minutes (using a computer – 3.6 GHz CPU speed and 16 GB RAM). Therefore, the total run time for the model with 2000 was about 100 hours.

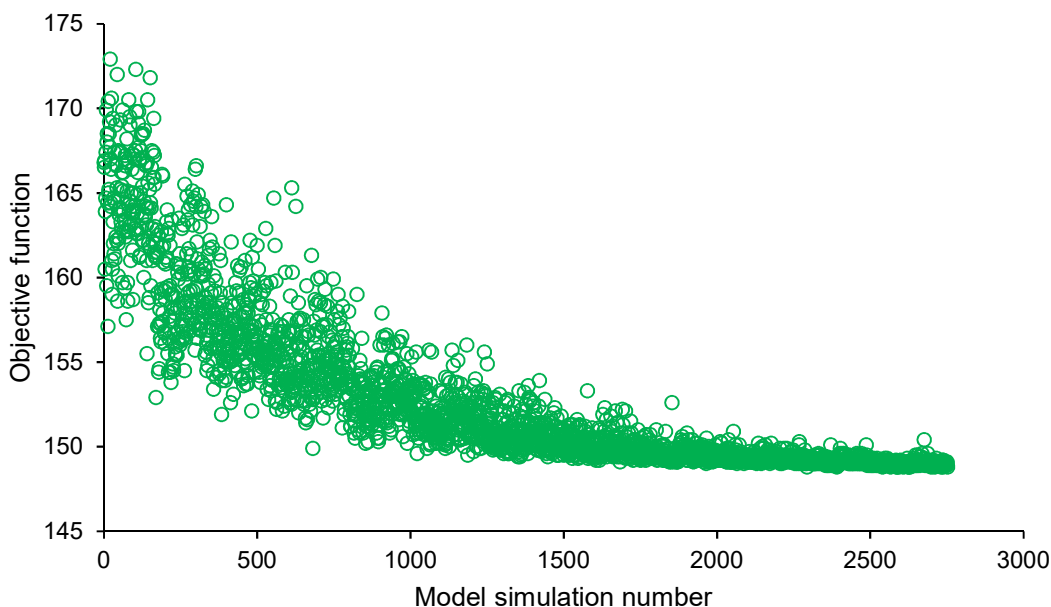


Figure 5.5. Objective function for the different model evaluations.

5.2. Objective function

The major aim of this study is evaluation flooding mitigation capacity of the system, which regulated by flood peak at critical downstream gauging points. Hence, the objective is to minimize the flood peak at the control points that can be expressed by:

$$\text{minimise } F = \max \left(\sum_{j=1}^2 H_j^2 \right)_{t \in [t_o, T]} \quad (5.1)$$

where:

H_j is the flood peak that occurred at j station at the time step t of the flood event.

The subscripts j represents the number of the control points at Ai Nghia station and Giao Thuy station, respectively (Fig. 5.6).

T is the total number of time steps for flood events.

t_0 is the initial operating time.

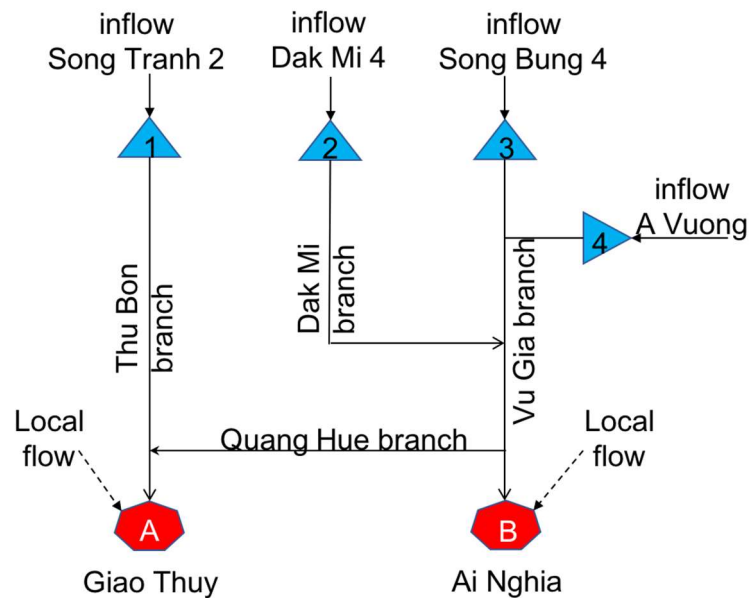


Figure 5.6. Description of the four reservoirs and two control points in the Vu Gia Thu Bon catchment.

5.3. Flood event of 2007

The flood event of November 10th – 16th, 2007 caused severe damages in the Vu Gia Thu Bon catchment especially on some of its main downstream. The damage caused by this disaster amounted to 2.384 billions VND, 47 victims.

The rainfall recorded in localities from November 11th to the afternoon of November 12th lasts for a long time to 300-400 mm. There are many places in which extreme rainfall occurred, such as Nam Tra My mountainous area reaching up to 662 mm. Prolonged heavy rains cause a significant amount of flow generation, resulting in rising rapidly in floods in the river network. Water level on the Vu Gia and Thu Bon rivers exceeded the alert level III of 1-1.5 m at 10 am on November 12th that beaten the historical flood in 1999. The hydrological consequences of this event were the rare floods observed in Nong Song to 10,600 m³/s and Thanh My to 5,280 m³/s (see Fig. 5.7).

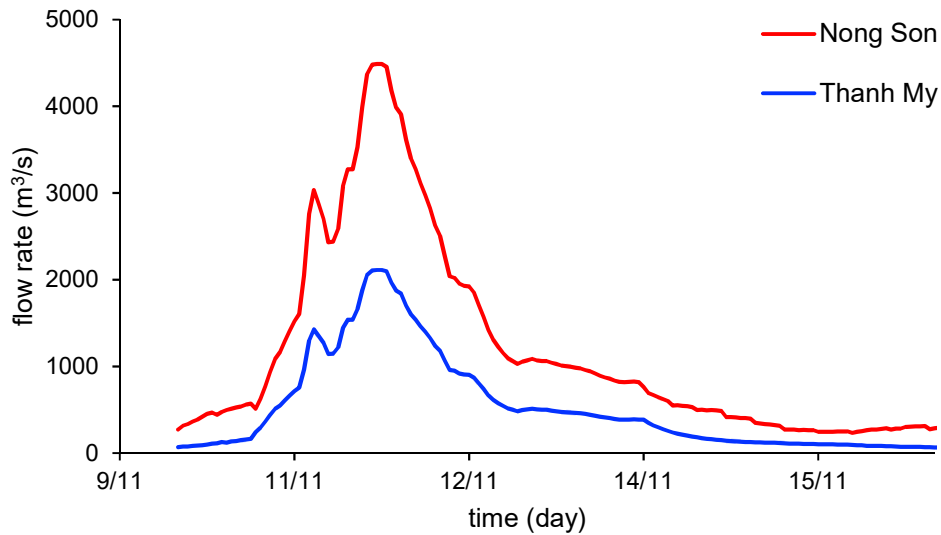


Figure 5.7. Hydrograph at Nong Son and Thanh My gauging stations in the 2007 flood event.

5.3.1. Boundary and initial conditions

The Vu Gia Thu Bon is a big river system but observed data of this river system are still not sufficient to give a good overview of the hydrological situation. There are only two discharge stations in this catchment that are Nong Son and Thanh My stations. The reservoir's inflow of the system reservoir using in this research is extracted from the Mike SHE model (Vo & Gourbesville, 2016).

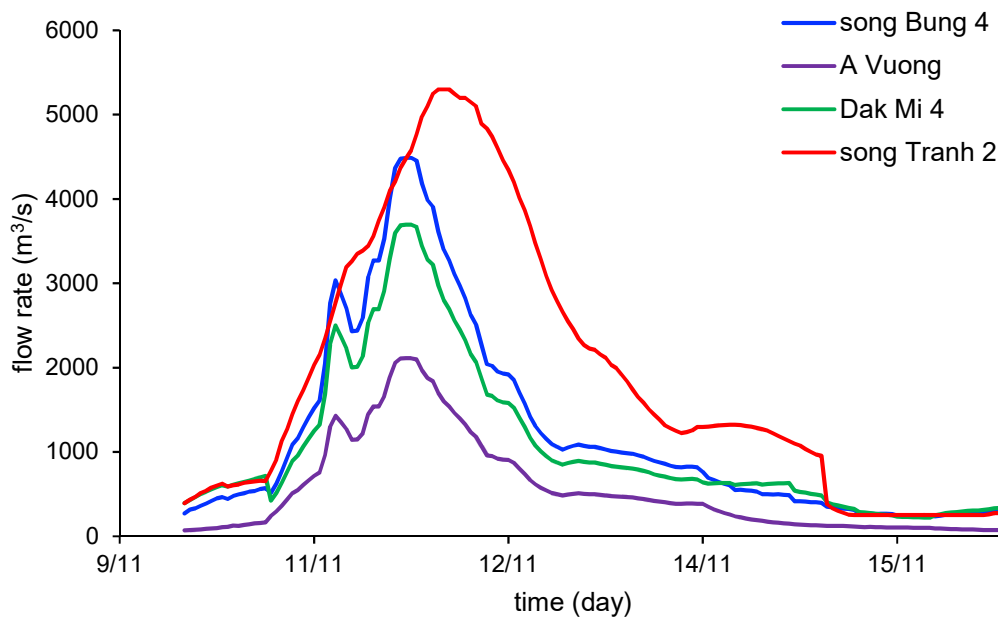


Figure 5.8. Inflow hydrographs entering four reservoirs (Vo & Gourbesville, 2016).

Mike SHE obtained hydrographs of the historical flood events in 2007 are represented in Fig. 5.8. The peak inflow of these four reservoirs, the A Vuong, the Dak Mi 4, the Song Tranh 2, and the Song Bung are 2112 m³/s, 3696 m³/s, 5300 m³/s, and 4488 m³/s, respectively. The flooding events, in this case, corresponds to a return period of 25 years, based on design flood analysis. The volume inflow of four reservoirs during flood events is larger than the designed active storage capacity, specifically 290 against 266.48 million m³ the A Vuong reservoir, 534 against 158.26 million m³ the Dak Mi 4 reservoir, 1010 against 521.1 million m³ the Song Tranh 2 reservoir, and 636 against 233.99 million m³ the Song Bung 4 reservoir.

The initial storages of four reservoirs are extracted from simulation model in chapter 3, that are 258.72 million m³ in the A Vuong reservoir, 243.57 million m³ in the Dak Mi 4 reservoir, 608.43 million m³ in the Song Tranh 2 reservoir, and 413.78 million m³ in the Song Bung 4 reservoir.

5.3.2. Scenario operation

The flood-reducing capacity is the most significant variable of the Vu Gia Thu Bon reservoirs system that need to be considered in flood season (Nguyen et al., 2018). Three methods are proposed to figure out a general picture of the operation of the multi-reservoir system and demonstrative effective optimal operation strategies:

Method 1: no regulation of flood control

In this method, each reservoir's releases an amount equal to the inflow to the reservoir. So, it can be said that the flow in the river system is likely original flood flow conditions in the year of 2007. The fact that four hydropower projects were have not built yet in 2007. So, this scenario can help us understand the magnitude of the flood before any human intervention and demonstrate an important role in reducing the flooding downstream.

Method 2: operation associated with the current rules

In the event of a flood, the main objective of responsibility for civil security is to reduce human losses and material damages. The various hydroelectric reservoirs in the Vu Gia Thu Bon catchment upstream represent an opportunity in this regard and can serve as an effective lever for flood control in the river network downstream. Reservoirs can store a certain volume of water during flood events based on the free volume of the reservoir, that is

named “flood control storage”, and the water level of reservoir corresponding named “flood limit water level”.

According to the current rules during flood control procedure, if the water level at the control point of Ai Nghia station greater 8.5 m for A Vuong, Dak Mi4, Song Bung 4 reservoirs and 3.5 m at Giao Thuy station for Song Tranh 2 reservoir, the reservoir releases are less than the reservoir inflow. Therefore, reservoir’s release will be reduced into downstream during the flood (Fig. 5.9).

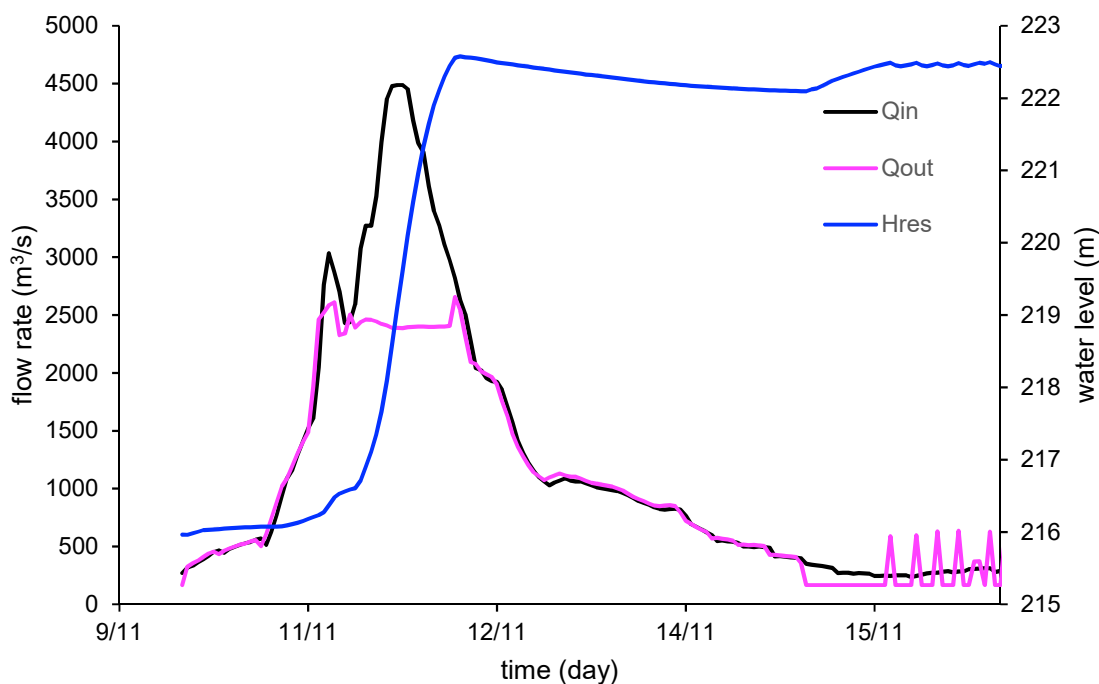


Figure 5.9. Flood control procedure.

Method 3: optimal operation

The main operational reservoir activities should be focused on reducing the peak of water level downstream and optimized the hydrographs of four reservoirs so that the flooding threat to the downstream is reduced to the minimum. The objective of global optimization consists of determining stage levels of spillway gates adapted to a reservoir considering its influence on the control points downstream and also to these other reservoirs. Parameters of the optimal model are a range of gate levels in four spillway gates and the water level at the control points. The total number of parameters to be optimized is 42. The feasible parameter space for these parameters is shown in Tab. 5.2.

Table 5.2. Feasible parameter space for these parameters in the SCE algorithm

Reservoir	GL_i^1	Reservoir stage (m)	Lower bound (m)	Upper bound (m)	Optimal value (m)
A Vuong	GL_1^1	370	363.65	371	365.83
	GL_1^2	371	363.59	370.13	366.76
	GL_1^3	372	363.55	368.86	364.82
	GL_1^4	373	363.51	368.15	364.37
	GL_1^5	374	363.48	367.74	364.03
	GL_1^6	375	363.45	367.74	364.18
	GL_1^7	376	363.44	367.04	364.49
	GL_1^8	377	363.43	366.87	364.06
	GL_1^9	378	363.42	366.87	364.32
	GL_1^{10}	379	363.34	366.65	363.92
	GL_1^{11}	380	363.34	366.56	363.98
Dak Mi 4	GL_2^1	251	244.6	248.4	246.16
	GL_2^2	252	244.5	248	246.56
	GL_2^3	253	244.4	247.5	246.47
	GL_2^4	254	244.3	247.3	244.83
	GL_2^5	255	244.2	246.9	245.43
	GL_2^6	256	244.1	246.7	245.55
	GL_2^7	257	244.1	246.6	244.89
	GL_2^8	258	244	246.4	245.15
Song Tranh 2	GL_3^1	165	166.35	168.4	167.29
	GL_3^2	166	166.35	168.4	166.67
	GL_3^3	167	164.96	168.4	165.70
	GL_3^4	168	164.42	168.4	164.89
	GL_3^5	169	164.09	168.4	164.85
	GL_3^6	170	163.87	167.58	164.43
	GL_3^7	171	163.65	166.64	164.54
	GL_3^8	172	163.52	166.39	164.37
	GL_3^9	173	163.43	166.21	163.84
	GL_3^{10}	174	163.32	165.88	163.62
	GL_3^{11}	175	163.21	165.58	163.90
Song Bung 4	GL_4^1	214	214.64	217.63	216.73
	GL_4^2	215	214.64	217.63	216.26
	GL_4^3	216	213.82	217.63	216.45
	GL_4^4	217	213.35	217.63	215.43
	GL_4^5	218	213.08	217.63	215.57
	GL_4^6	219	212.88	217.63	214.92
	GL_4^7	220	212.73	216.84	213.43
	GL_4^8	221	212.61	216.32	213.62
	GL_4^9	222	212.49	215.92	212.82
	GL_4^{10}	222.5	212.44	215.75	213.17

5.3.3. Results

To verify optimal strategies, derived non-regulation, the simulation of operation in flood event 2007 was conducted to compare the released amount associated with the current rules and optimal operation. The results for optimized outflow discharges of the multi-reservoir system according to the proposed framework are presented in Fig. 5.10.

Fig. 5.10 depicts the simulated flood control operation during the flood event, using the optimized and current rules. The hydrograph of non-regulation that is in the black line is also shown for comparison. The result shows that the optimal simulated maximum release at the end of the peak is 1548 m³/s, 2694 m³/s, 2545 m³/s, and 4611 m³/s while the corresponding the current rules is 1577 m³/s, 3214 m³/s, 3250 m³/s, and 4897 m³/s in A Vuong, Dak Mi 4, Song Bung 4 and Song Tranh 2, respectively. Afterward, the operation proceeds into the post-flood stage and the gates of spillways are held open during the recession period, the water level of the reservoir is steadily kept around the normal water level.

From the simulation of the 2007 flood event, the obtained result can show that the proposed rules consistently achieve water levels closer to the target levels at the end of each stage. The ideal optimal solution reduces the maximum reservoir releases to 27.49% of the inflow peaks, and this performance of peak reduction degenerates to 11.39% with current rules operation.

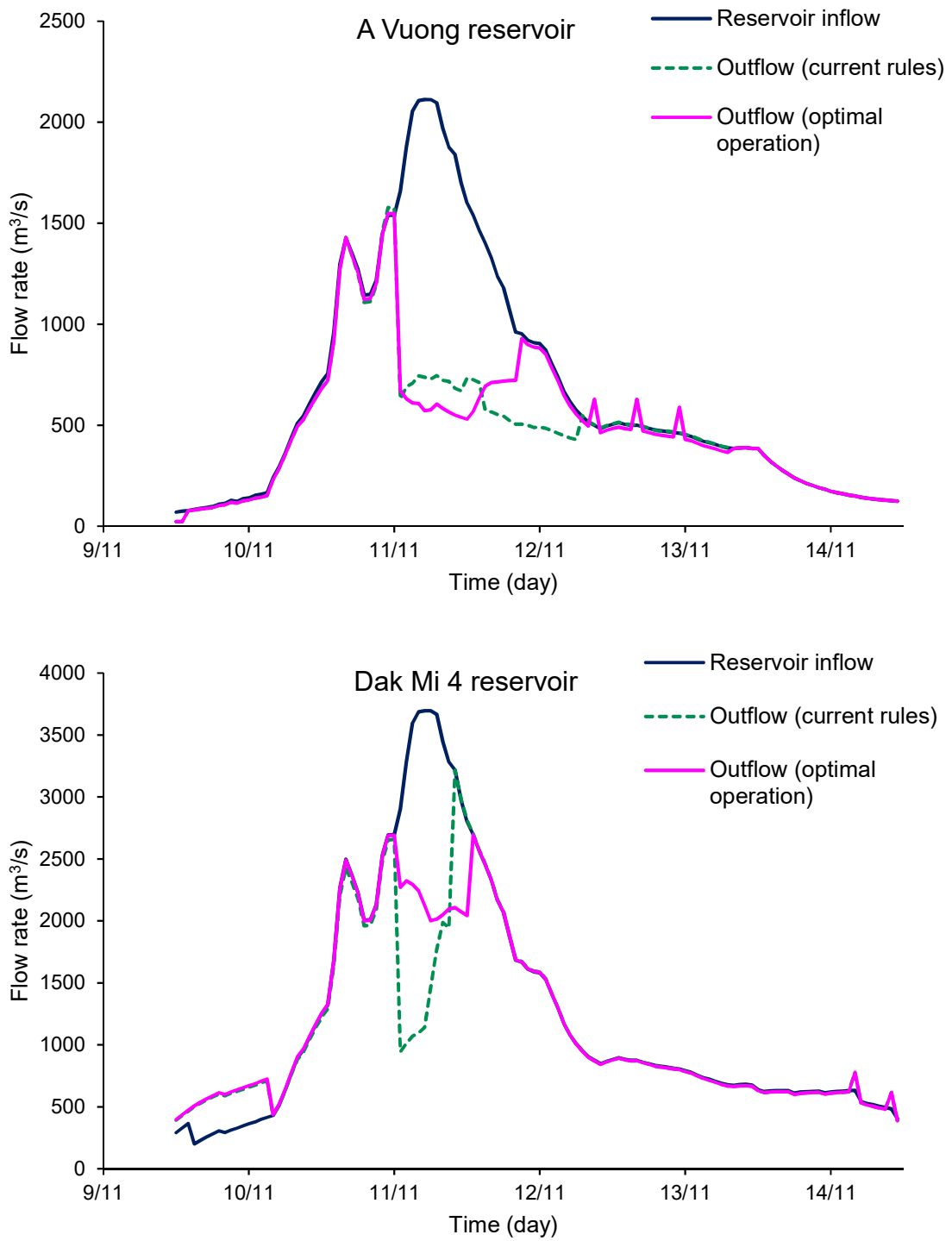


Figure 5.10. Comparison of operation process between three scenarios.

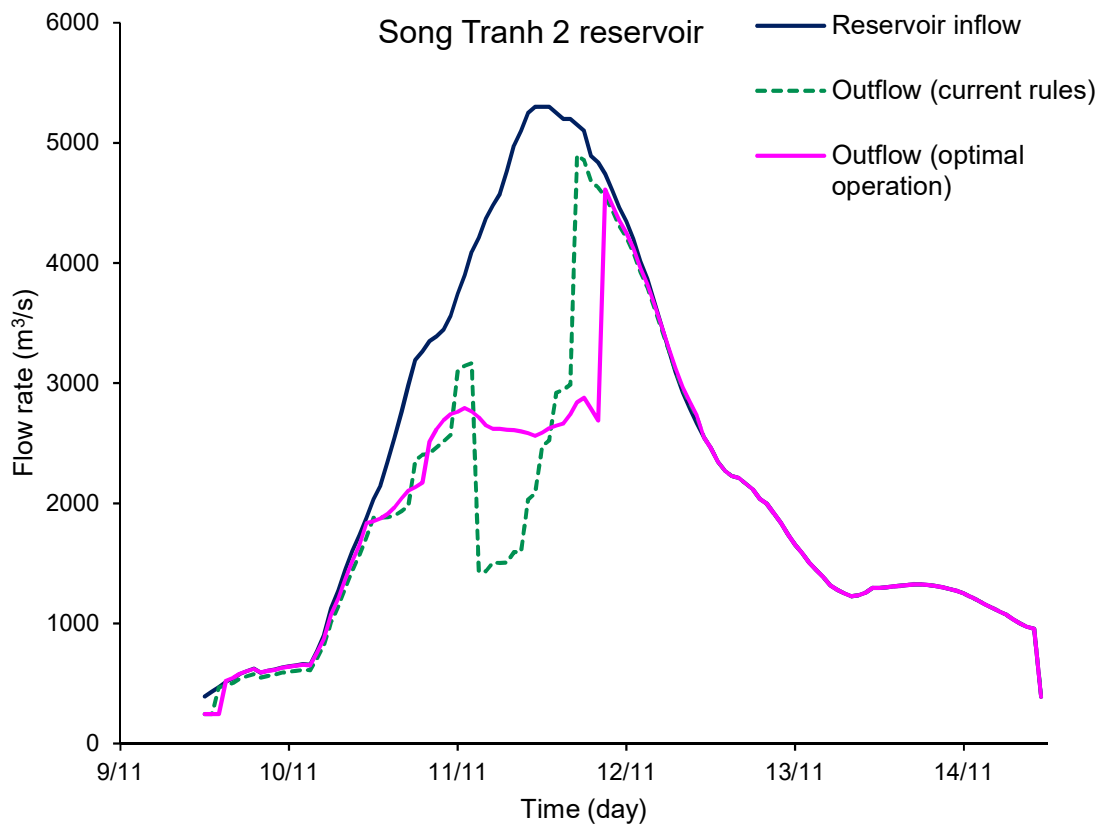
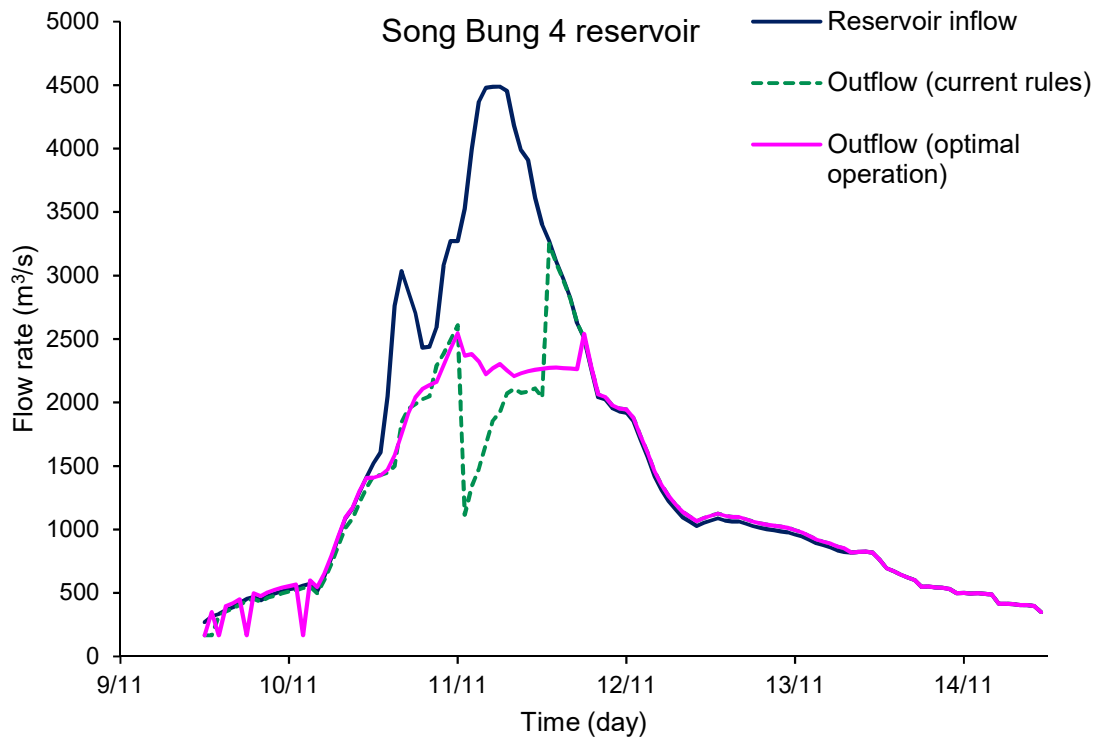


Figure 5.10 (continued). Comparison of operation process between three scenarios.

The water level conditions at downstream Ai Nghia and Giao Thuy stations, defined as model control points, are the most important factor of the optimization model. Fig. 5.11 shows the flood stage at Ai Nghia and Giao Thuy stations during the flood event in 2007 and the condition under when optimization operating rules were implemented.

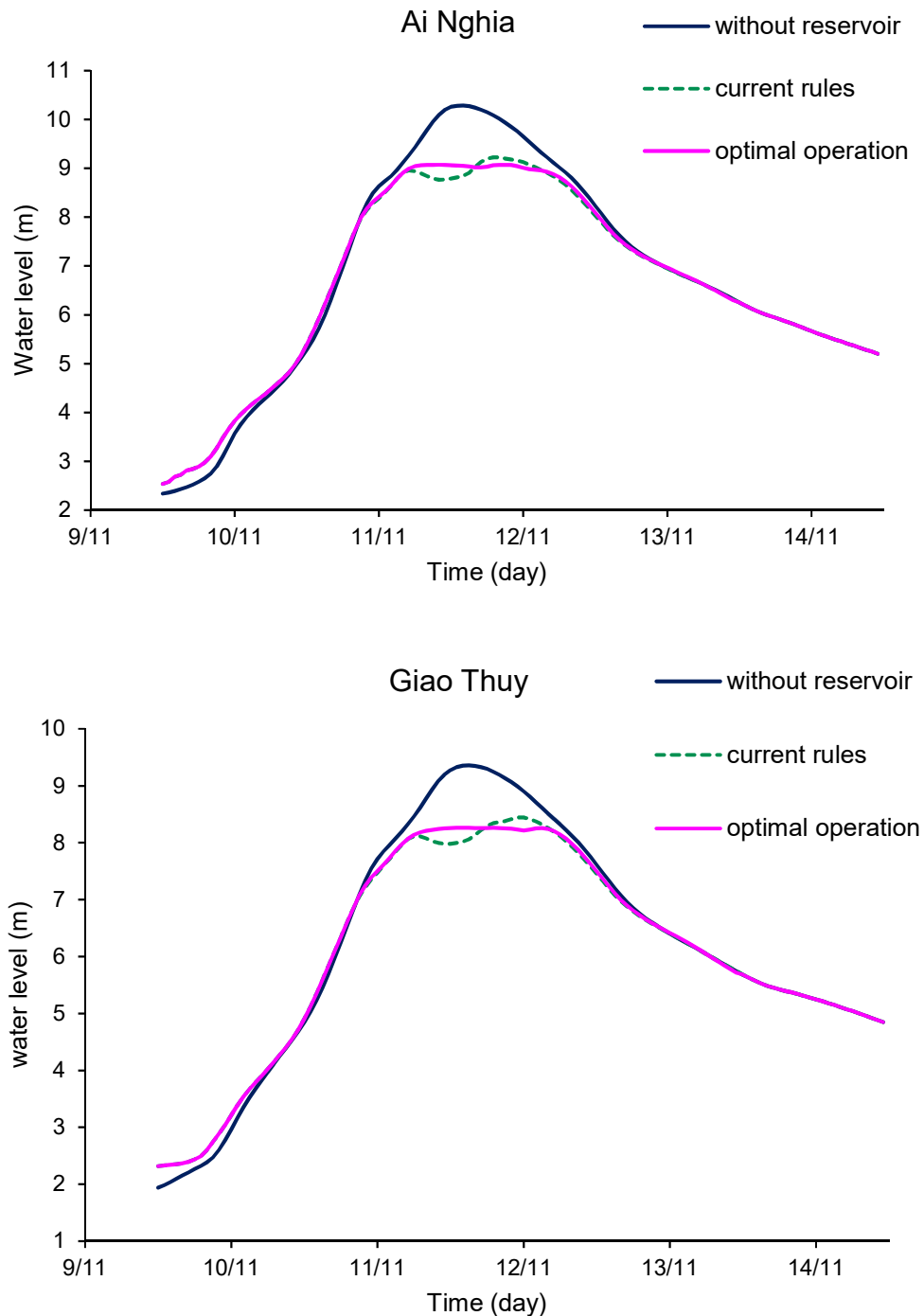


Figure 5.11. Comparison of water level at the control points downstream.

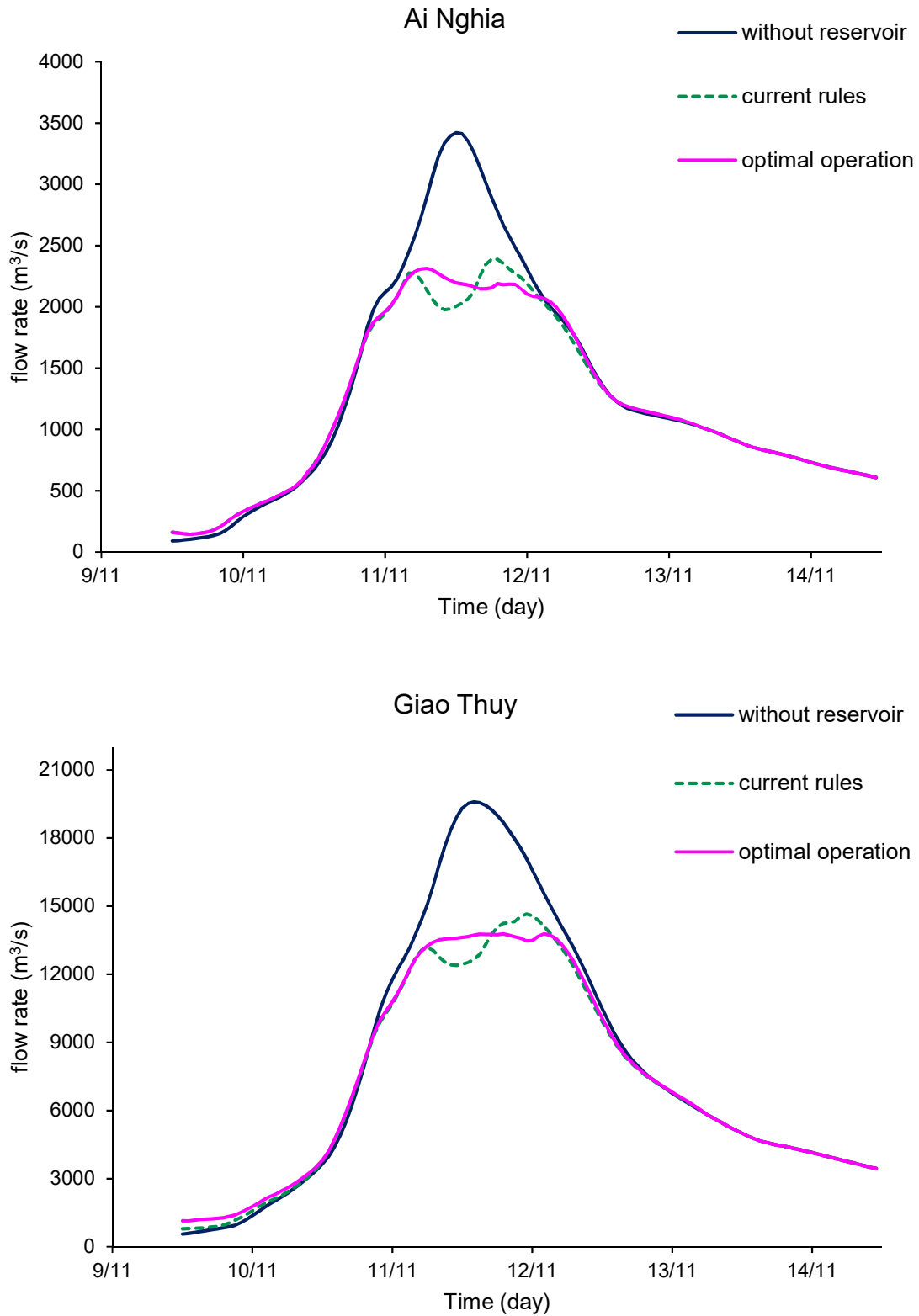


Figure 5.12. Runoff hydrographs of flood events at the control points.

From Tab. 5.3, once can find that the optimal operation rules procedure much better performance among these three rules, in terms of reducing the downstream floodwater at control points. At the Ai Nghia station, the simulated maximum water level was reduced from 9.22 m following current rule to 9.07 m according to optimization simulated, while observation in 2007 flood (without constructed reservoir) recorded the highest water level at 10.29 m. Similarly, the maximum water level in the Giao Thuy station decreased by 0.18 m between the current rule and optimization model.

Table 5.3. Results for the three scenarios considered

Scenario	Maximum water level		Flow peak	
	(m)		(m ³ /s)	
	Ai Nghia	Giao Thuy	Ai Nghia	Giao Thuy
Without reservoir	10.29	9.36	3 421	19 592
Current rules	9.22	8.44	2 389	14 654
Optimal operation	9.07	8.26	2 313	13 780

5.4. Flood event of 2009

The typhoon Ketsana was the second-most devastating tropical cyclone of the 2009 Pacific typhoon season, named “number 9” in Vietnam, caused \$500 millions damages and 53 fatalities in the Vu Gia Thu Bon catchment. The recorded rainfall in three days is 610 mm at Hien, 425 mm at Thanh My, 401 mm at Ai Nghia, 570 mm at Nong Son stations. The heavy rains were causing massive flood surges in the total downstream catchment. Record high water levels were reported at Ai Nghia, Giao Thuy, and Cau Lau stations, with 10.5 m, 9 m, and 4.5 m, respectively.

In 2009, only the A Vuong hydropower dam was built and started operation in April 2009. The flood event occurred from September 29th to October 2nd, 2009 and has caused historical damages. The A Vuong reservoir has suddenly released 2700 m³/s for 4 hours, causing a significant change in flooding situation in the downstream area (Fig. 5.13).

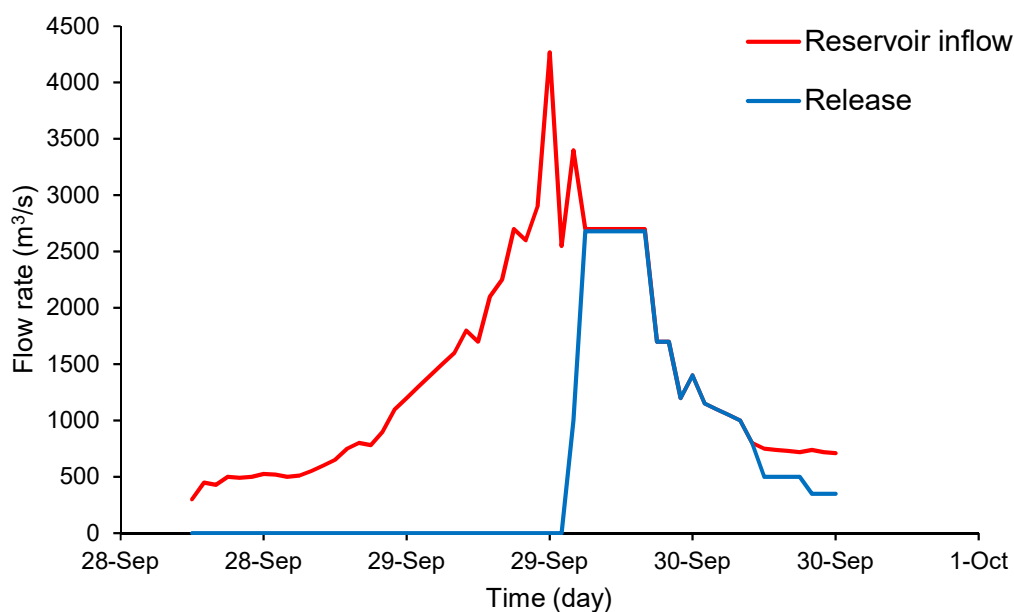


Figure 5.13. Operation of A Vuong reservoir in the 2009 flood event.

There was a complicated situation in A Vuong reservoir operation. On one hand, the flood was being increased and reached flood peaks in the downstream area. On the other hand, in upstream area, floodwaters were continuously forced to discharge to ensure the safety of the reservoir and dams. It caused overlapping floods (artificial floods). This phenomenon has resulted in an increase in water levels in the downstream areas, resulting in increased flood depth and prolonged flood time.

5.4.1. Boundary and initial conditions

The flood event happened from 27 September to 3 October 2009, reaching significant historical monitoring data at gauging data. At the moment, precipitations on Nong Son and Thanh My stations reached over 800 mm for both stations. As a result, the entire Vu Gia Thu Bon catchment was under significant flood with a maximum inflow of 3707 m³/s, 4713 m³/s, 4155 m³/s, 3996 m³/s in A Vuong, Dak Mi 4, Song Bung 4 and Song Tranh 2 reservoirs, respectively (Fig. 5.14). Interestingly, flood duration is different between these reservoirs with the peaks that did not occur at the same time.

It is a relatively large flood with origination in four upper streams of the four reservoirs. The return period of the flood is higher than 20 years in terms of the peak discharge of the natural runoff recorded at the Nong Song and Thanh My stations.

The reservoirs' initial storages are set to the historical values that are 251.16 million m³, 243.57 million m³, 457.22 million m³, 452.5 million m³, corresponding reservoir stage 369 m, 251 m, 219 m, 160 m, in A Vuong, Dak Mi 4, Song Bung 4 and Song Tranh 2 reservoirs, respectively.

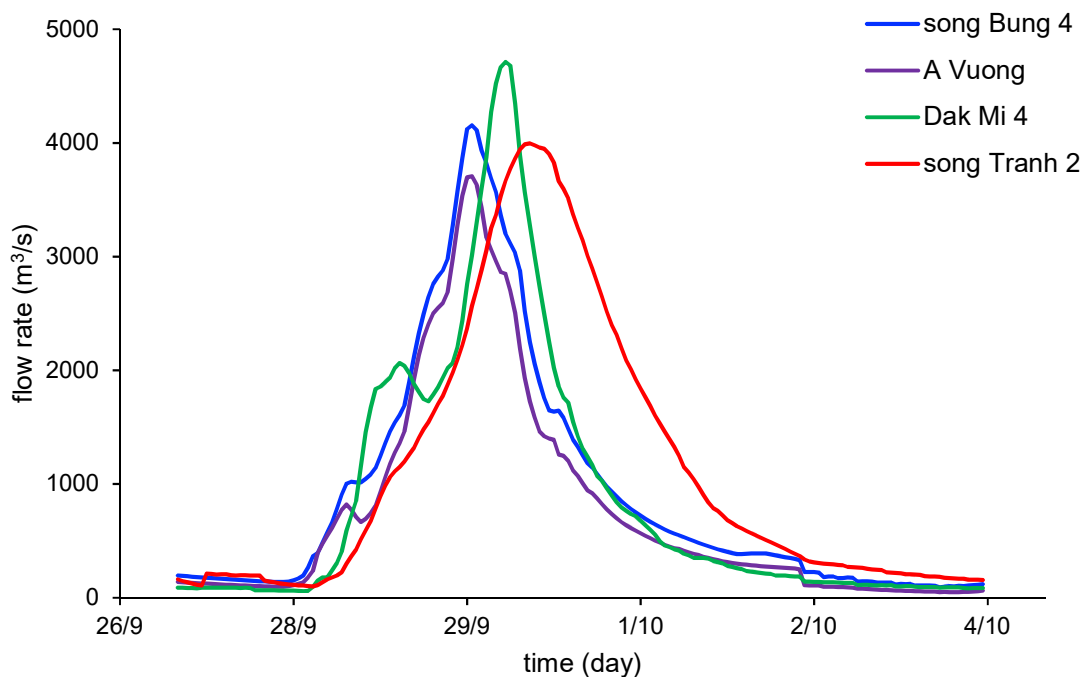


Figure 5.14. Inflow hydrographs of four reservoirs in flood event 2009 (Vo & Gourbesville, 2016).

Table 5.4. The time information of the given time series

Reservoir	Peak discharge (m ³ /s)	Total volume (10 ⁶ m ³)
A Vuong	3 707	450.59
Dak Mi 4	4 713	531.70
Song Bung 4	4 155	547.20
Song Tranh 2	3 996	665.90

5.4.2. Results

Obtained results have been categorized into two parts: (1) describing the operation of the four biggest reservoirs following actual operation of A Vuong in the flooding 2009, (2) comparison of simulated results according to three following scenarios:

- Simulating operations of four reservoirs following A Vuong reservoir operation in 2009 as a pilot study (historical). This is because A Vuong reservoir was only a hydropower plant operated at that moment. Also, operation rules for the four most significant reservoirs had not been formulated.
- Operating of the four most significant reservoirs according to recent operation regulation (current rules).
- Simulating operations of these most significant reservoirs following coupled optimization-simulation model (optimal operation).

During the actual event, the operator did not release until 1 pm September 29th, 2009, at the A Vuong reservoir because flood control storage is not proposed in design. The fact that the spillway gates only operate if the water level in the reservoir is higher than the normal water level. So, at 1 pm on September 29th, the reservoir stage in four reservoirs reached the normal water level; the operator makes the decision to release floodwater with an amount of discharge equal to the reservoir inflow (Fig. 5.15). As a result, the water levels downstream of these reservoirs suddenly changed, causing flooding and riverbank erosion.

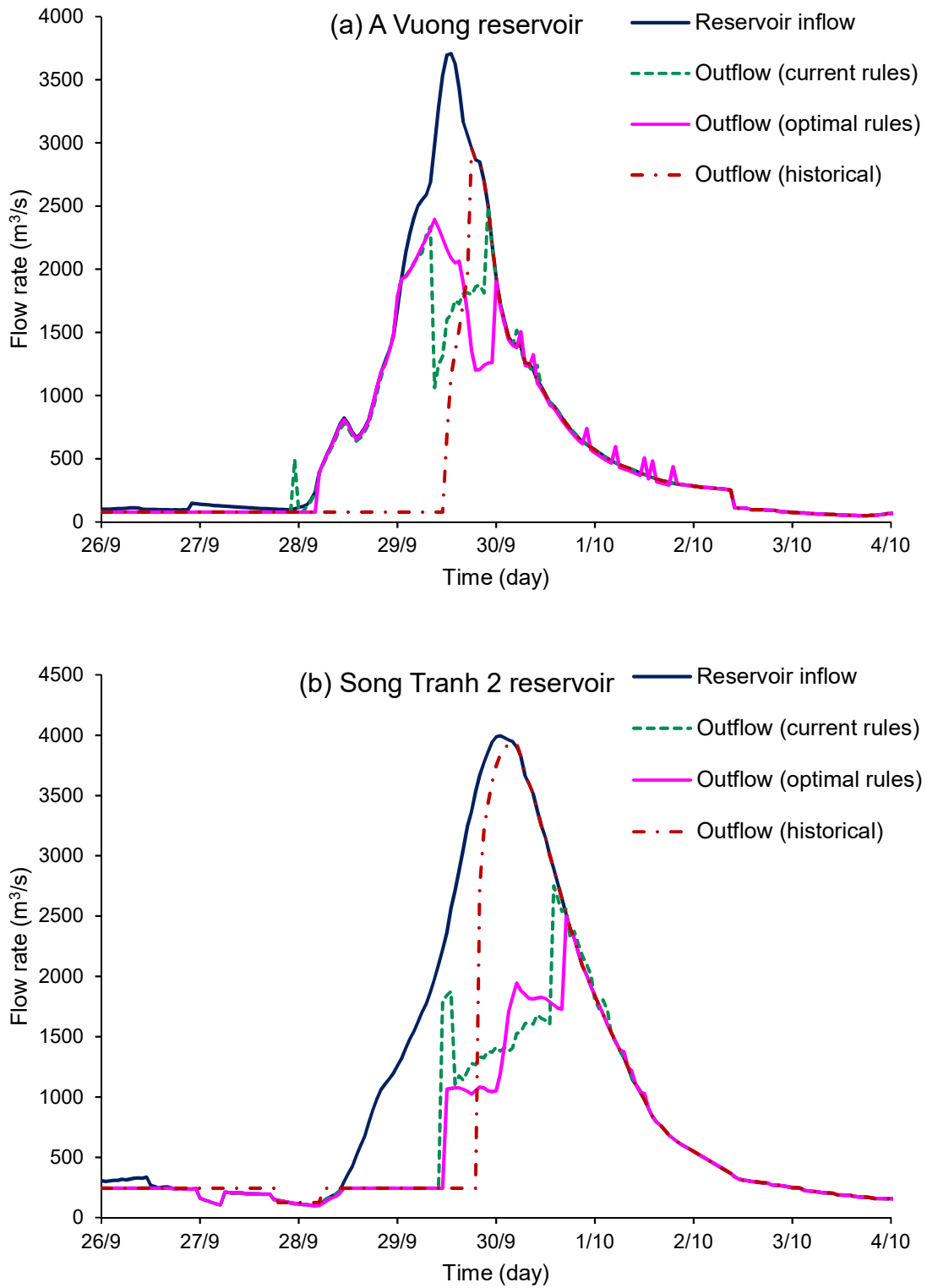


Figure 5.15. Comparisons between the current rules with the optimal operation for A Vuong and Song Tranh 2 reservoirs.

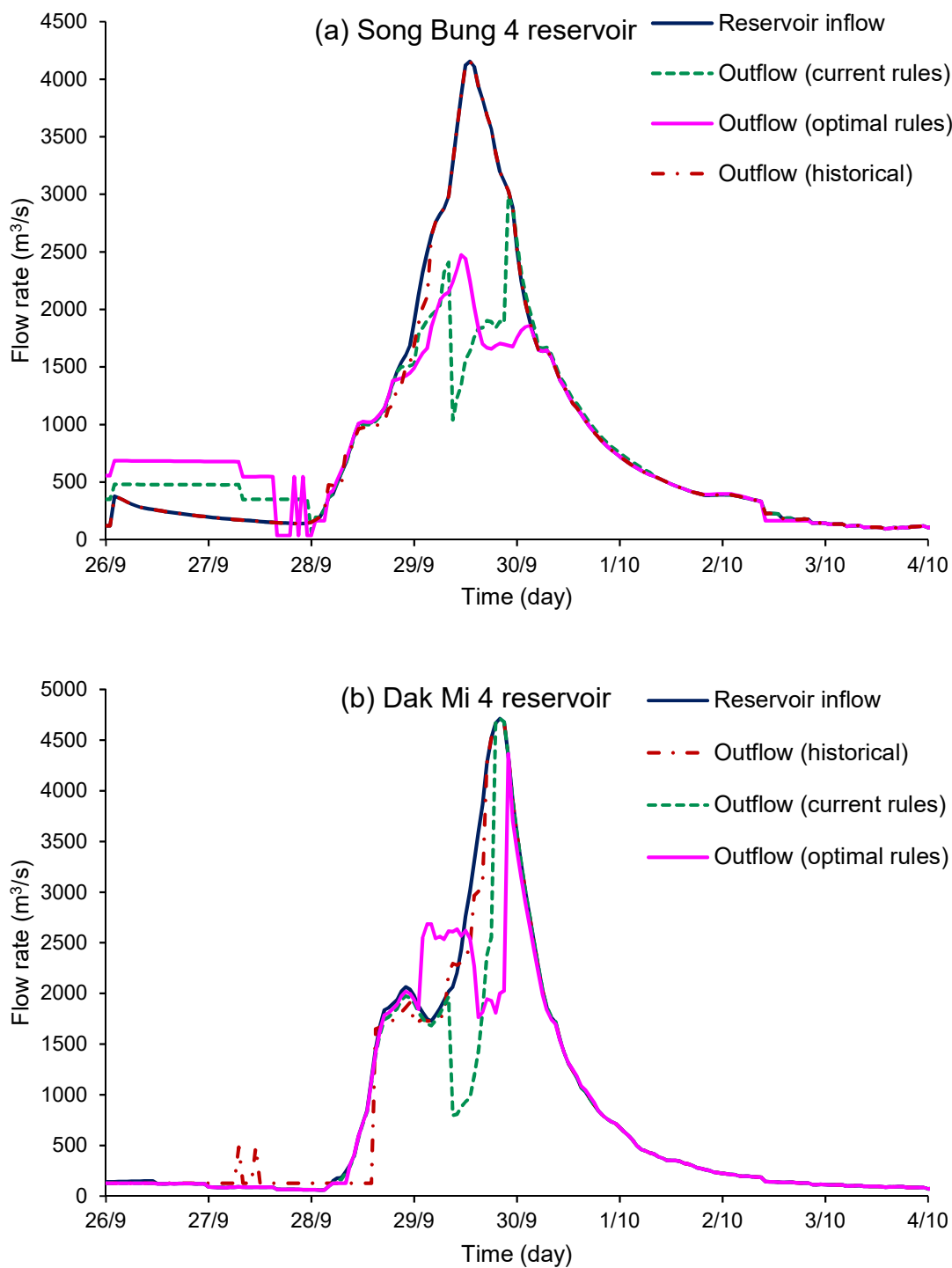


Figure 5.16. Comparisons between the current rules with the optimal operation for Song Bung 4 and Dak Mi 4 reservoirs.

Variation of reservoirs storage and flood hydrographs at reservoirs results from the operation associated with current rules and optimal operation are compared in Fig. 5.15 and 5.16. The water level in the reservoir system was already higher than the upper bound flood limit water level before flood arrival (except the Song Tranh 2 reservoir). When the flood was entering the

reservoirs, the release rate was increased gradually in order to prevent any sudden change in storage volume. Initially, the discharge amount and inflow rates were almost the same. Then the inflow rate exceeded the release rate, causing storage increase up to maximum value and the reservoir stays its normal water level for a while before the reservoir draws down.

In the Song Tranh 2 reservoir (Fig. 5.15 b), the reservoir water level was 160 m at 1 a.m. on 26 September. That means that the water level was lower than the lower bound flood limit water level by approximately 5 m and the dam released water only for power generation. When the reservoir stage reached the lower bound FLWL, it released water for performance flood control procedure.

Table 5.5. Results for three scenarios considered

Control point	Scenario	Downstream control-point			
		Maximum water level (m)	HR (%)	Peak runoff (m ³ /s)	QR (%)
Ai Nghia	Historical	10.40	2.68	3 578	9.93
	Current rules	10.00	6.42	3 161	20.44
	Optimal rules	9.50	11.10	2 625	33.91
Giao Thuy	Historical	9.10	4.71	18 088	12.98
	Current rules	8.80	7.85	16 714	19.59
	Optimal rules	8.40	12.04	14 380	30.82

In order to assess the performance of the three scenarios, two criteria are taken into account (Hsu & Wei, 2007), defined as follows:

- **Downstream control point peak runoff reduction rate (QR):**

$$QR(\%) = \frac{Q - Q^{max}}{Q} \times 100 \quad (5.3)$$

where:

Q^{max} is the computed peak runoff at the control point for any of the scenarios;

Q is the unregulated peak runoff at the control point.

- **Downstream control point maximum level reduction rate (HR):**

$$HR(\%) = \frac{H - H^{max}}{H} \times 100 \quad (5.4)$$

where:

H^{max} is the control point maximal level during the flood;

H is the control point maximal level based upon supposing no upstream building reservoir.

Generally, the higher of the criterion leads to greater results. Under three scenarios, Tab. 5.5 lists the results of the flood event in 2009. From Tab. 5.5, the downstream peak runoff reduction rate (QR) and maximum level reduction rate (HR) derived from the optimal operation are greater than from both historical operation and current rules.

Fig. 5.17 illustrates water levels obtained from the simulation at gauging stations. At the Ai Nghia station, the simulated maximum water level was reduced from 10.0 m following current rule to 9.5 m according to optimization simulated. At the same time, historical operation according to the method of the A Vuong reservoir recorded the highest water level at 10.4 m. Similarly, the maximum water level in the Giao Thuy station decreased by 0.4 m between the current rule and optimization model. The release rules derived by optimal algorithm demonstrate their effectiveness in flood mitigation.

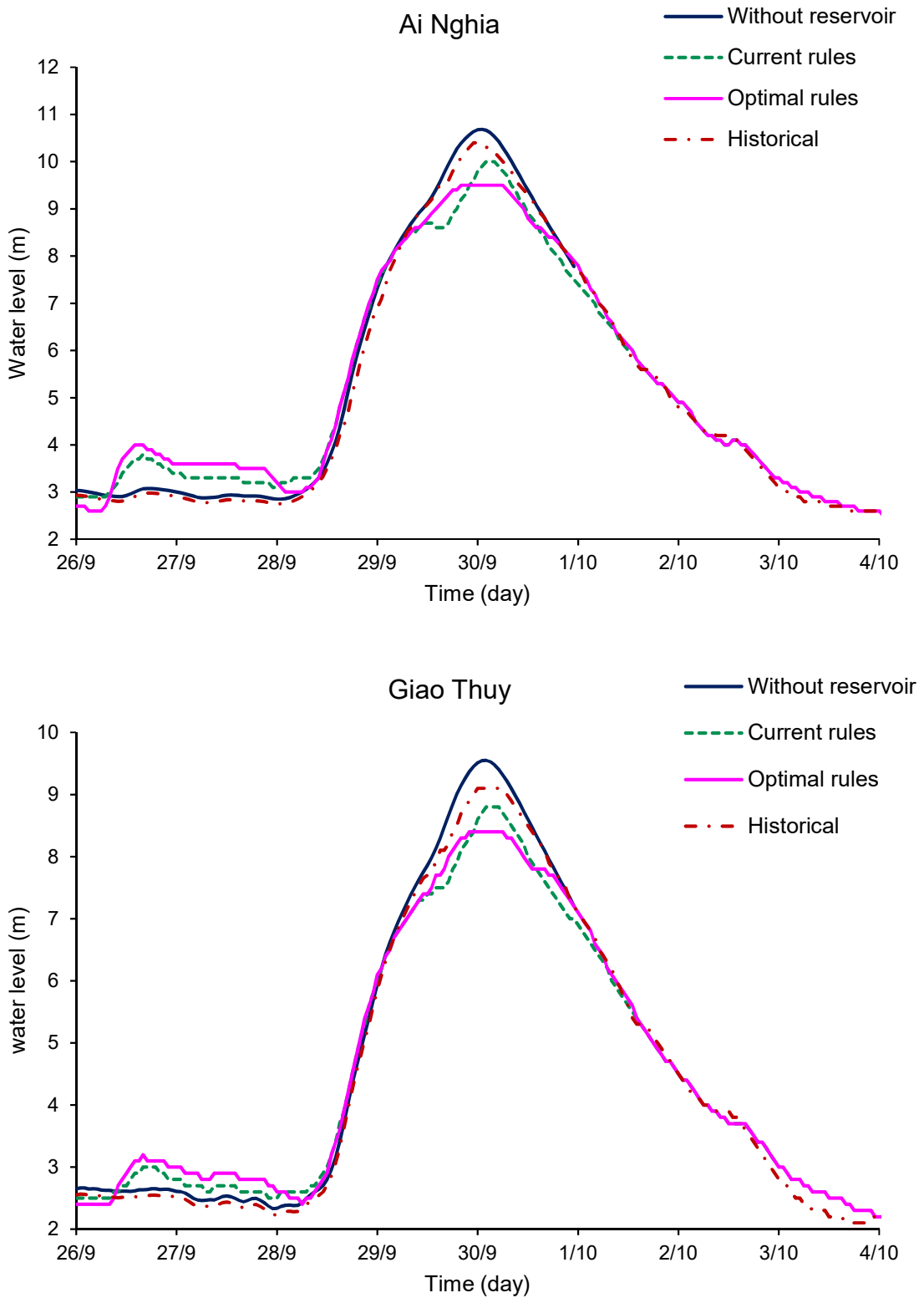


Figure 5.17. Water level hydrograph of three scenarios at the control points.

5.5. Flood event of 2017

On 4th of November 2017, typhoon Damrey (the typhoon n° 12 in Vietnam) made landfall in the southern province of Vietnam, bringing with it extreme rainfall. The rainfall recorded 804 mm in Tra My in 48 hours between 3rd and 5th of November. At least eight other locations in the Vu Gia Thu Bon catchment recorded rainfall amount at around 500 mm during this period and consequently severe inundations appeared in the year of 2017.

According to Vietnam’s Central Steering Committee for Disaster Prevention, Search and Rescue, the 2017 flooding caused significant damages to the Vu Gia Thu Bon river basin. Statistic data confirmed 13 people had lost their lives, and ten were missing. At least 10,620 people have been evacuated across affected areas. The damage caused by this disaster amounted to 542 billions VND.

At the time, four major reservoirs in the Vu Gia Thu Bon catchment have been operated according to the regulation of the Government.

5.5.1. Boundary and initial conditions

The recorded data of the inflow hydrograph in four reservoirs (A Vuong, Dak Mi 4, Song Bung 4, and Song Tranh 2) are shown in Fig. 5.17. Tab. 5.6 gives general information about them.

The time information of the given time series of a flood event is:

- Starting time: 10:00 a.m. 4 November 2017.
- Ending time: 1:00 p.m. 7 November 2017.
- Time step: 1h.

Table 5.6. The flood event in 2017

Reservoir	Peak discharge (m ³ /s)	Total volume (10 ⁶ m ³)
A Vuong	897	138.49
Dak Mi 4	4 075	534.64
Song Bung 4	4 887	372.97
Song Tranh 2	6 402	903.77

From Tab. 5.6, it can be seen that the runoff coming have a significant volume and peak discharge, in which the return periods corresponding are 25, 25, 20

years in Dak Mi 4, Song Bung 4 and Song Tranh 2 reservoirs, respectively, while a small volume in A Vuong reservoir.

The initial reservoir storages are also set to the historical levels, that is 243.59 million m³, 243.57 million m³, 534.81 million m³, 413.78 million m³, corresponding reservoir stage 368 m, 251 m, 165 m, 216 m in the A Vuong, Dak Mi 4, Song Tranh 2, and Song Bung 4 reservoirs, respectively.

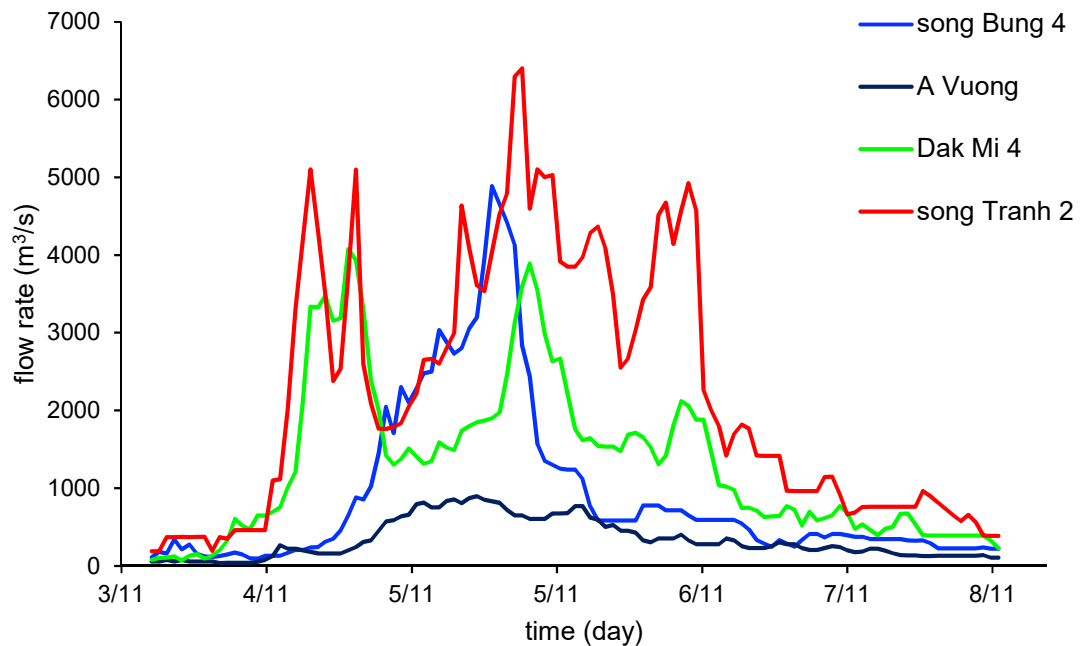


Figure 5.18. Inflow hydrographs of four reservoirs in flood event 2017.

5.5.2. Results

In order to show the significance of the simulation-based optimization model, the optimization strategies are applied simultaneously for the multi-reservoir system in the Vu Gia Thu Bon catchment. Here, comparisons with the downstream control points peak runoff and the maximum water level are made among two scenarios: (1) the historical, and (2) the optimal operation. The simulated optimal gate operation of four reservoirs is compared to the actual rules during the flood event. Inflow and outflow hydrographs released for all reservoirs performed by the historical operation and proposed model are shown in Fig. 5.19 and 5.20.

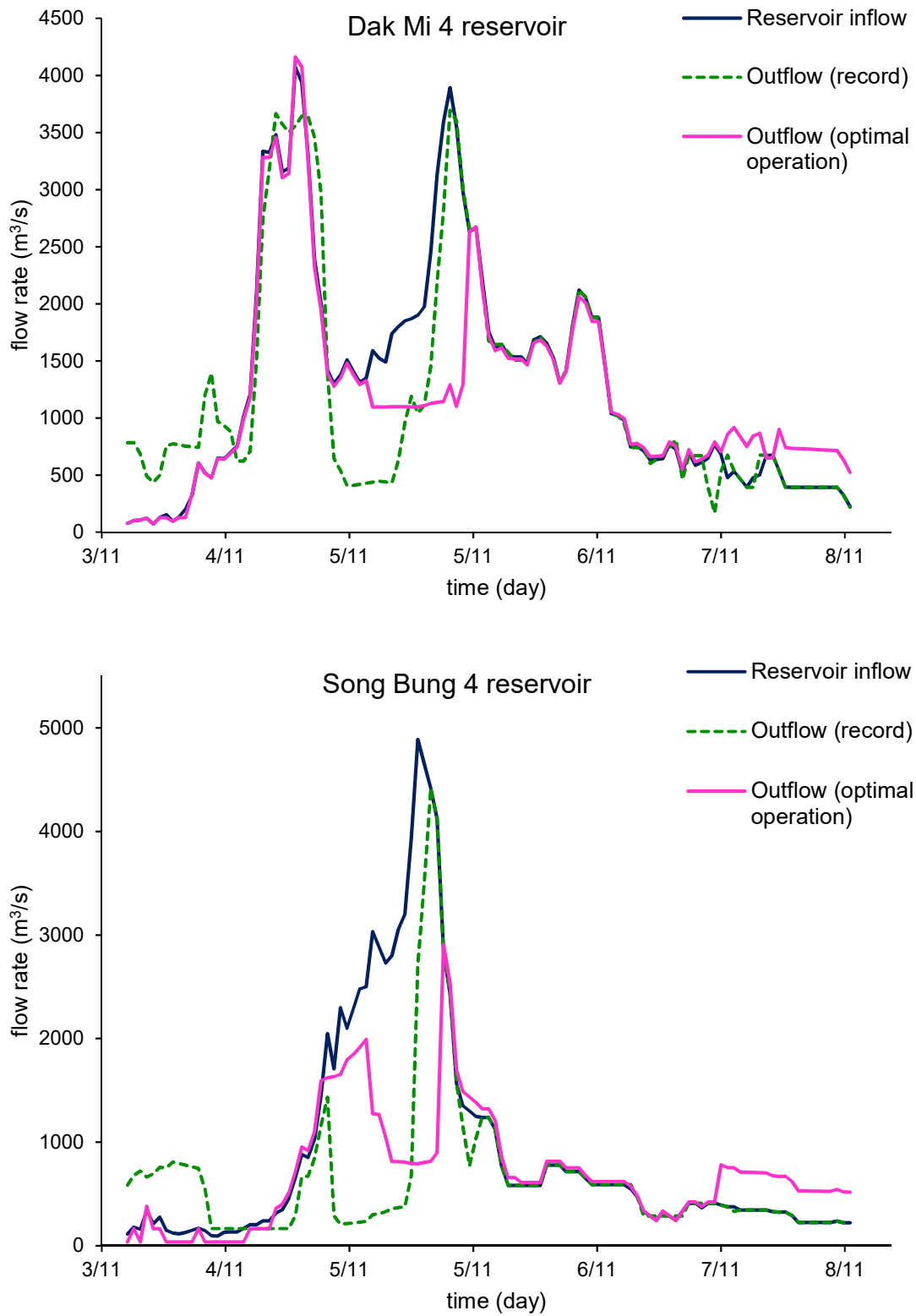


Figure 5.19. Hydrographs of Dak Mi 4 and Song Bung 4 reservoirs in flood event 2017.

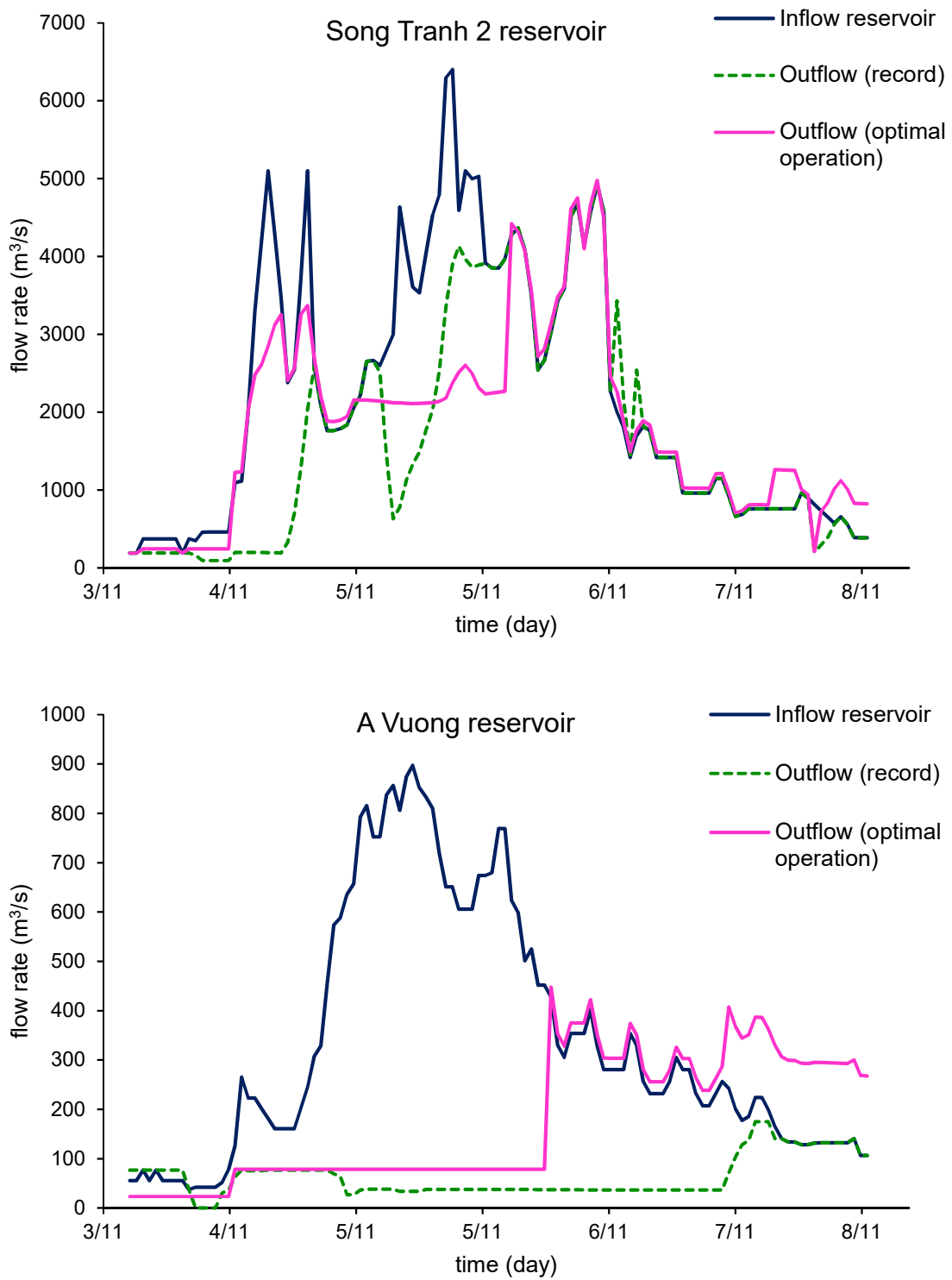


Figure 5.20. Hydrographs of Song Tranh 2 and A Vuong reservoirs in flood event 2017.

Magnitude and duration of flooding in downstream critical control points obtained from the simulation of two scenarios that are: the historical operation rules and the application of the SCE algorithms for operating are compared to highlight the differences. Tab. 5.7 illustrates that the peaks

runoff and maximum water level at downstream derived from the simulation-based optimization model (proposed model) are lesser than from current rules.

Table 5.7. Comparison of the flood operating results among operators and the proposed model

Control point	Scenario	Downstream control-point			
		Maximum water level (m)	HR (%)	Peak runoff (m ³ /s)	QR (%)
Ai Nghia	Operator	9.44	7.68	2 621	19.11
	Optimal operation	9.06	11.37	2 232	31.11
Giao Thuy	Operator	8.82	7.48	16 561	19.89
	Optimal operation	8.56	10.22	15 208	26.43

Results of the downstream peak runoff reduction rate (QR) and maximum level reduction rate (HR) during flood events were also calculated and listed in Tab. 5.7. The results of historical operation and proposed model are: (1) at Ai Nghia station for criterion QR by 19.11 and 31.11%, respectively; for criterion HR by 7.68 and 11.37%, respectively; (2) at Giao Thuy station for criterion QR by 19.89 and 26.43%, respectively; for criterion HR by 7.48 and 10.22%, respectively. From Tab. 5.7, once we can find that the two downstream control points peak runoff reduction rate (QR) and maximum level reduction rate (HR) derived from the proposed model are more significant than historical operation.

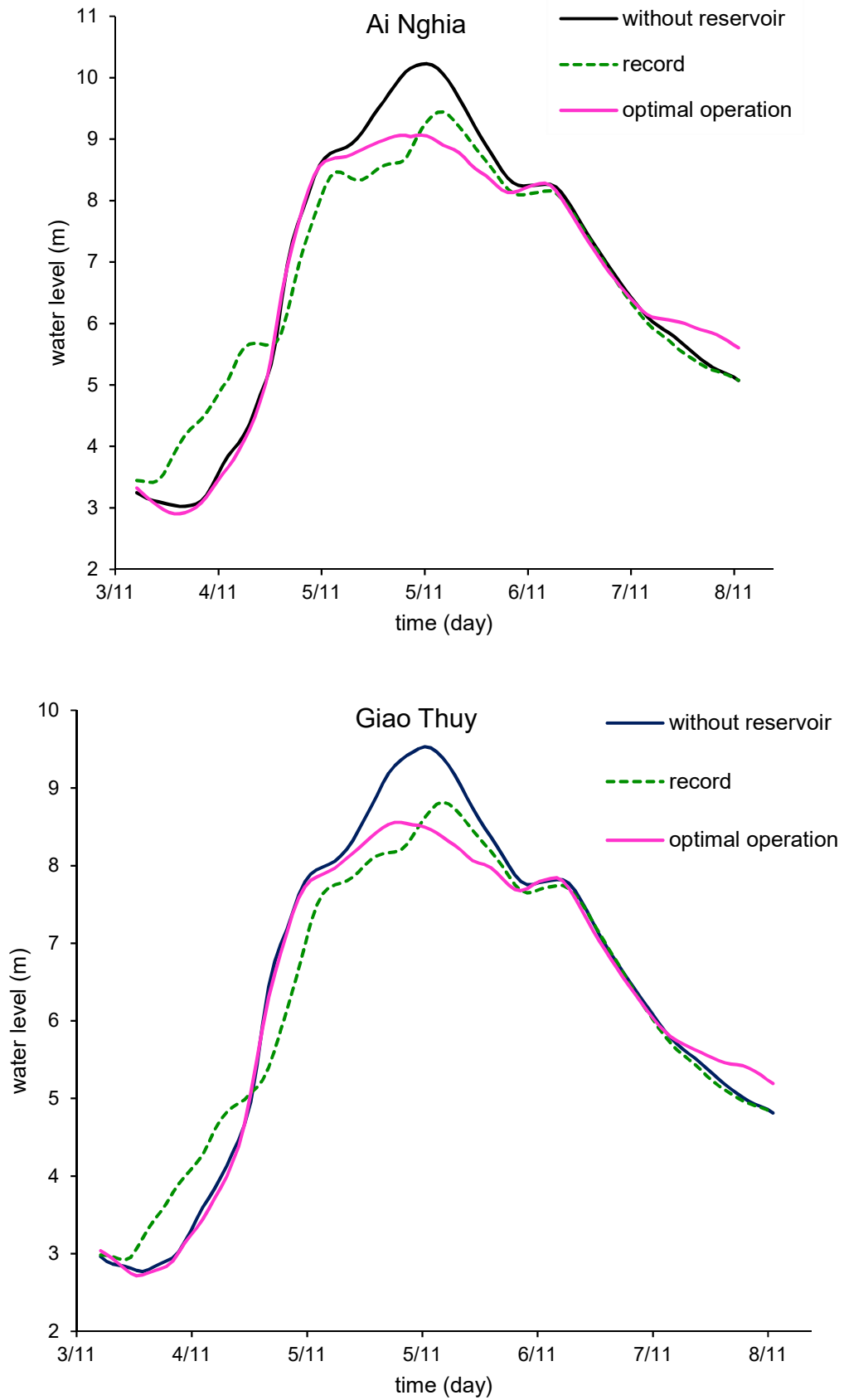


Figure 5.21. Water level hydrograph of three scenarios at the control points in 2017 flood event.

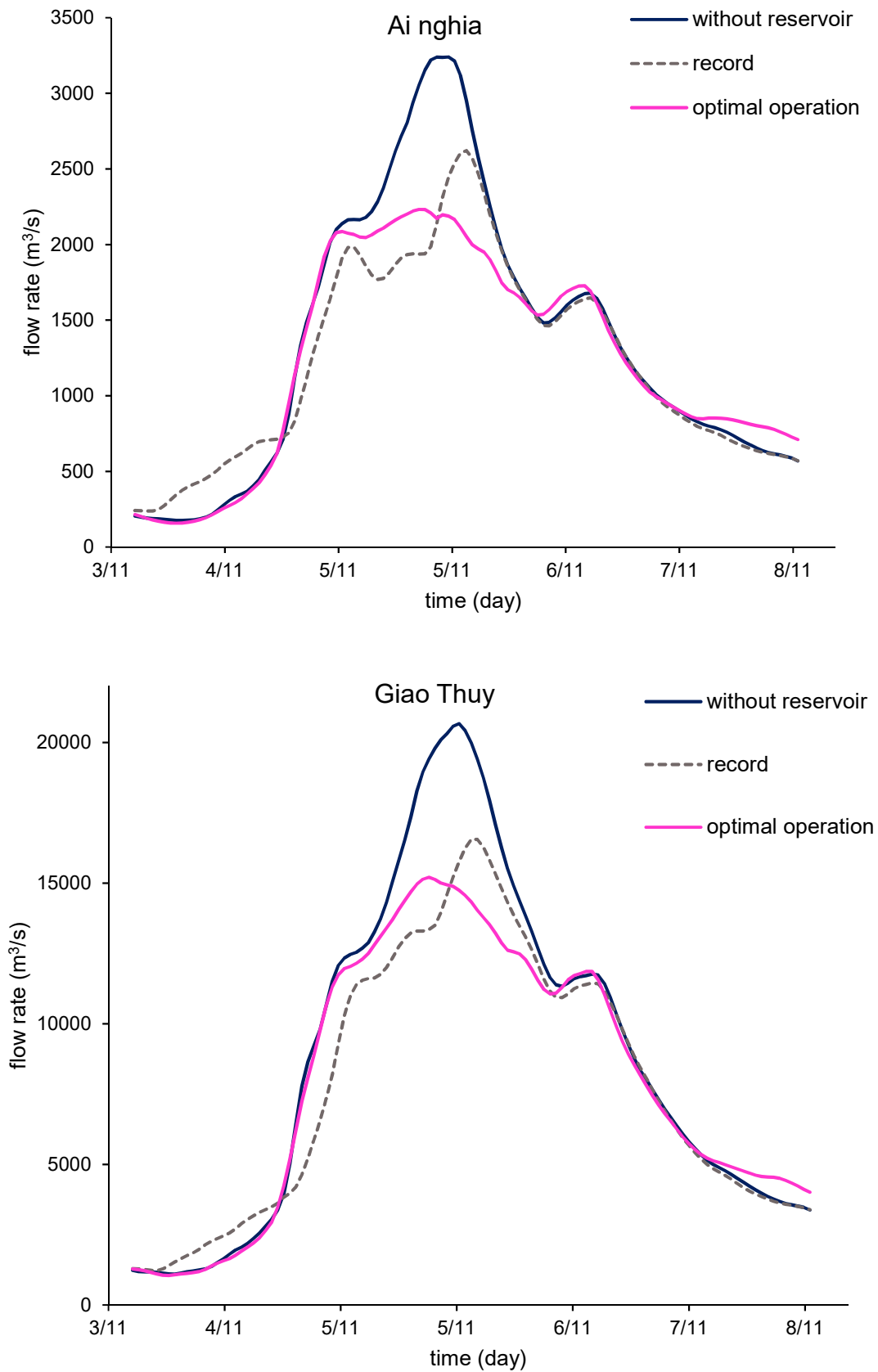


Figure 5.22. Runoff hydrographs of flood events 2017 at the control points.

5.6. Conclusion

The primary purpose of flood control is to avoid downstream flood damages and to protect the reservoir itself. The main goal of the current research was to determine the optimal release during flood events for the multi-reservoir system in the Vu Gia Thu Bon catchment. To attain these goals, the simulation-optimization framework is adopted for determining the spillway gates operation strategy. The Structure Operation (SO) module in the hydrodynamic model Mike 11 is adopted for simulation of the operation multi-reservoir while the Shuffled Complex Evolution (SCE) algorithm is applied to determine the best set of spillway gates stages.

The method has been successfully implemented for the multi-reservoir system in the Vu Gia Thu Bon catchment. Three flood events in 2007, 2009 and 2017 were selected for demonstration. In order to assess performance of the approach and for comparison purpose, three developed scenarios that are representing operations the reservoir system in the historical, the current rules and the proposed model have been used. The results indicate that the proposed model provides much better performance for all scenarios in terms of reducing the peak flow as well as reducing the maximum water levels at selected downstream control points compared to the rest scenarios. Our experiments obtained results find the agreement are in line with previous studies' results (Hsu & Wei, 2007; Wei & Hsu, 2009). Consequently, the SCE algorithm demonstrates its effectiveness for optimizing of complex reservoir systems.

In respect to the obtained results, there are still several limitations which should be handled in future work. Specifically, estimating the flood damages in terms of economic indicators has not been proposed. Consequently, the exploration of the destruction concerning the floods to different regions, i.e., agricultural, residential and industrial areas, should be lighted up in detailed.

Chapter 6

Flood limit water level methodology

The multi-reservoir systems in the Vu Gia Thu Bon catchment is dedicated to both flood control and integrated water resources development. The balance between flood control and water conservation is achieved with the Flood Limit Water Level (FLWL) methodology.

This chapter aims to optimize the FLWL for a parallel reservoir system with multi-objectives and by applying a combination of simulation and optimization models.

6.1. Introduction

The primary purpose of flood control is to avoid downstream flood damages and the protection of the reservoir itself (Zhou et al., 2018). Often one reservoir's sufficient storage capacity isn't enough to secure downstream flood control points. In this situation of a multi-reservoir system and in order to provide a sufficient protection level, the different reservoirs are required to work together to protect these flood control points from flooding and inundation. The main variables controlling the operation of the flood control multi-reservoir system are the storage capacity available and the estimated amount of inflow from an incoming flood (Liu et al., 2017).

The flood limit water level (FLWL) is an effective method to balance flood control and water conservation during the flood season (Ouyang et al., 2015; Xie et al., 2018). It is the maximum allowed water level required for flood control and also the maximum water level reserved for water conservation such as water supply, hydropower generation during the flood season (Hua et al., 2012; Peng et al., 2017). In the flood season, the reservoir water level must be maintained below the FLWL in order to leave enough room for flood storage. Once the flood peak passes and starts to recede, the reservoir stage must be reduced to the FLWL as soon as possible to provide adequate storage for the next flood events. This value is the most significant parameter of a tradeoff between the activities of flood control and conservation (Liu et al., 2015; Yun & Singh, 2008). The water level of reservoirs should not be too high during the flood seasons due to the likelihood that significant floods can occur, while the reservoir water level should not be too low due to water storage demands (Liu et al., 2019).

For the design of FLWL in a reservoir, some applications on multi-reservoir flood control operations use two objective functions, including maximization of the water supply benefits and minimization of potential flood damage costs (Chang et al., 2017; Opan, 2018; Xie et al., 2018; Yun & Singh, 2008). A possibility for determining FLWL is to use annual design storms or annual design floods (i.e., a design frequency or return period is chosen depending on the importance of the reservoir) through reservoir regulation (Chen et al., 2013).

To improve the water storage of a reservoir while preserving its flood control protection, the development of FLWL management can be grouped into three main approaches: (i) using a fixed FLWL value for the entire flood season, (ii)

using different FLWLs for different sub-periods of the flood season, and (iii) using a dynamic control of FLWL.

The first approach is known as static control of FLWL that is fairly simple to implement. In this approach, FLWL is set as a fixed threshold in which the reservoir water levels are not allowed to exceed during the entire flood seasons. Because of this, the reservoirs are often not to be refilled to the target water level by the end of a flood season (Peng et al., 2017). To avoid this problem, the second approach divides the flood season into several sub seasons and using different FLWLs for different sub seasons. It increases to some degree utilization of water resources but still belongs to the system of static control without using the hydrological forecast data (Tan et al., 2017).

The last approach applies a dynamic control of FLWL that allows the reservoir stages to fluctuate between a given range of lower and upper limits. This method is thus more efficient as it allows better use of flood control storage and increases the production potential of water resources without lowering the flood protection standard (Liu et al., 2019). However, the main difficulty with this approach is the determination of the upper bound and lower bound of FLWL is main research issue in dynamic control of FLWL research (Gong et al., 2019; Li et al., 2010; Ouyang et al., 2015; Tan et al., 2017; Xie et al., 2018; Yun & Singh, 2008; Zhou et al., 2014).

In the current research, an optimal design model for the FLWL boundary of a multi-reservoir is proposed to simultaneously optimize the flood control risk and hydropower generation potential of the reservoir system in the flood season. The popular Shuffled Complex Evolution (SCE) global optimization method that is used the AutoCal software, is coupled with the hydrodynamic Mike 11 model for optimizing the FLWL boundary. The SCE algorithm is one of the techniques that are robust optimization techniques to find the global optimum solution of complex problems with many functions. The proposed model is applied to the four large multi-purpose reservoirs in the Vu Gia Thu Bon catchment using the hourly inflow data series for representative hydrological years.

6.2. Methodology

6.2.1. Multi-objective optimization framework

Multi-objective optimization of a multi-purpose multi-reservoir system refers to a problem that involves several objectives to be optimized simultaneously, such

as flood control, water supply, hydropower generation. However, the objectives are often in conflict with each other and are calculated by different units (Ngo et al., 2007). Thus, when there are two or more performance measures, one of the most critical components of multi-objective problem solving is how to evaluate the parameter sets. The techniques to solve multi-objective optimization can be classified into two main groups: (i) aggregation approaches, and (ii) Pareto domination approaches.

An aggregate objective function method transforms a multi-objective optimization problem into a scalar optimization problem (Dellino et al., 2008). Usually, these aggregate functions use weighted sum, distance function, and utility function (Soon & Madsen, 2005). In contrast to the aggregation approach, there is a set of trade-off solutions, generally known as Pareto optimal solutions (also known as non-dominated). Such solutions are optimal in the sense that no other solutions are better than them in the creative potential, or can dominate them when considering all the objectives (Xin, 2013).

In the current approach, the aggregation approach has been chosen and is applied. The multi-objective optimization problem explores the entire Pareto front between the objective functions by performing several optimization runs using different weights. The weight allocated to the objective function in the combination of the various objective functions to be transformed into one aggregate calculation. Depending on the specific model application being considered, the assigned weights should reflect the relative priorities given to the different objectives. The defined objective functions are aggregated into one measure as follows:

$$f(X) = w_1 f_1(X_1, X_2, \dots, X_k) + w_2 f_2(X_1, X_2, \dots, X_k) + \dots + w_q f_q(X_1, X_2, \dots, X_k) \quad (6.1)$$

where:

f_1, f_2, \dots, f_q , are the individual objective functions;

w_1, w_2, \dots, w_q , are weighting factors ($0 < w_i < 1$) and $\sum_{i=1}^q w_i = 1$;

X_1, X_2, \dots, X_k , are the parameter sets.

Transformation functions are used to account for variations in the magnitudes of the different units, so the weighted objective function can be changed, as shown below:

$$f(X) = w_1 g_1 f_1(X_1, X_2, \dots, X_k) + w_2 g_2 f_2(X_1, X_2, \dots, X_k) + \dots + w_q g_q f_q(X_1, X_2, \dots, X_k) \quad (6.2)$$

where:

g_i is the transformation functions assigned to each objective function.

$$g_i(F_i) = \frac{F_i}{\sigma_i} + \varepsilon_i \quad (6.3)$$

σ_i is the standard deviation of the i^{th} objective function of the initial population used in the optimization algorithm;

ε_i is a transformation constant given by:

$$\varepsilon_i = \max \left\{ \min \left\{ \frac{F_j}{\sigma_j} \right\}, j = 1, 2, \dots, N \right\} - \min \left\{ \frac{F_i}{\sigma_i} \right\} \quad (6.4)$$

6.2.2. Optimization algorithm selection

The above objective functions are used to build a simulation-based optimization model with decision variables of FLWL. Fig. 6.1 shows the framework of the simulation model coupled with the optimization model. The optimization of FLWL, a multi-reservoir system, can be formulated as a combination of a simulation model and an optimization algorithm.

In this method, the SO module of the hydrodynamic model Mike 11 is adopted for the simulation of the operation multi-reservoir considering the physical constraints of the system as well as operation policies. The SCE algorithm is applied to determine the best set of decision variables, such as FLWL. In this study, the SCE algorithm, as implemented in the AutoCal (DHI, 2017) software, is adopted for optimizing FLWL of the multi-reservoir system in the case study.

In the first step, the SCE algorithm generates an initial population that meets all the constraints. Once the sets of FLWL are determined, the SO module is run to simulate the operation of a multi-reservoir and to determine the releases from all reservoirs. This hydraulic model also computes the flow discharges and water levels in the river network. Then, the optimization model evaluates the objective function based on the selected results from the simulation model. If one of the criteria for termination is satisfied, then stop the program; otherwise, return to execute the simulation model with a new set of FLWL generated by the SCE algorithm.

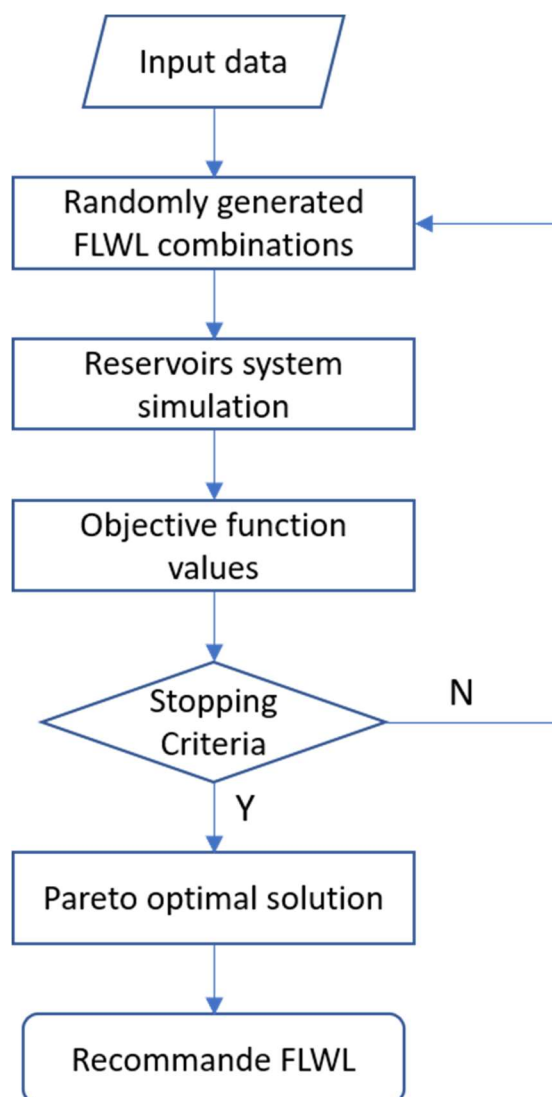


Figure 6.1. Framework of the optimal scheduling model of FLWL.

6.3. Application

6.3.1. Conventional operating rules

Flood control and hydropower generation that may be equally crucial in the operation of a reservoir system, correspond to two different water levels in the reservoir, FLWL and normal water level, respectively (see Fig. 6.2). The FLWL should not be surpassed by the reservoir water level during the flood season to maintain adequate storage for flood prevention. The normal water level is the highest water level under regular reservoir operation. Note that the storage volume defined between FLWL and the normal water level is called flood control storage, while the storage volume defined between the normal water level and the dead water level is the conservation storage (or active storage) and is used for hydropower generation (Fig. 6.3).

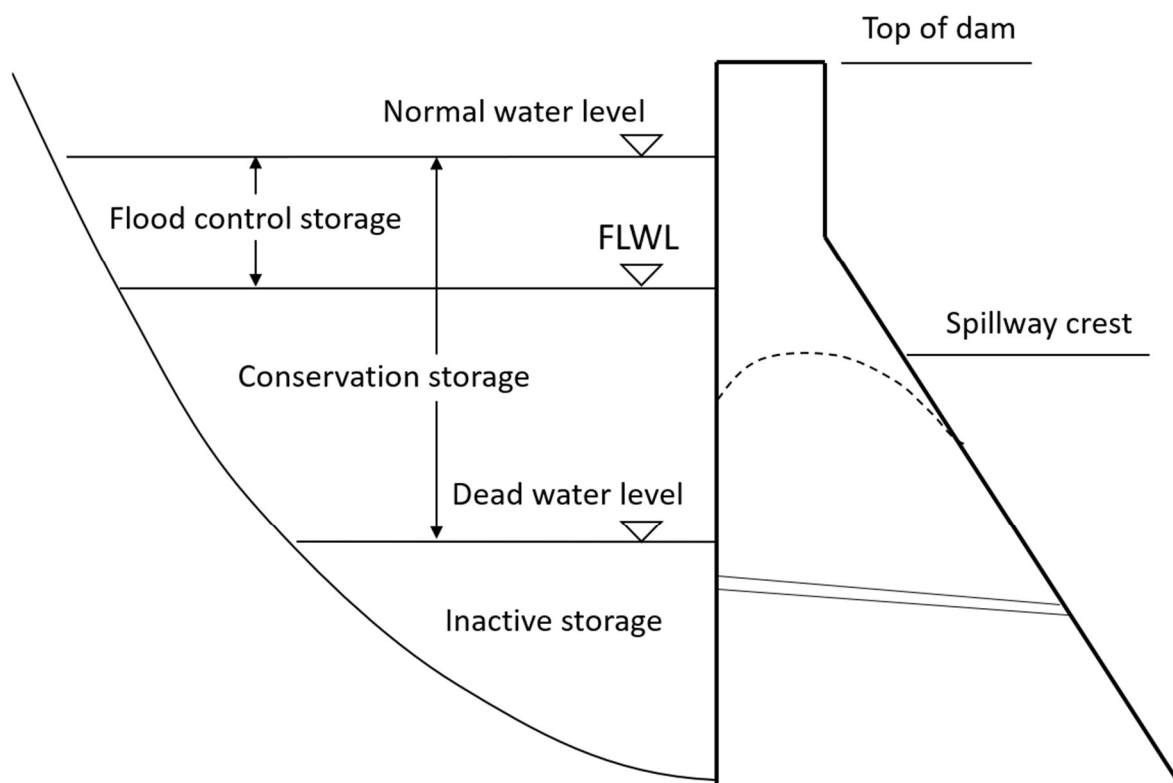


Figure 6.2. Sketch of index water levels and storage zones of the reservoir.

The four major reservoirs in the Vu Gia Thu Bon catchment include A Vuong, Dak Mi 4, song Tranh 2, and song Bung 4, which have been put into operations since 2009, 2011, 2012 and 2014, respectively. Following the operational regulation, the flood season is normally from 1st September to 15th December of every year. During the flood season, the multi-reservoir system is operated in the following order of priority:

- Strictly ensuring the safety of the dams;
- Taking part in reducing downstream floods;
- Ensuring efficiency in hydropower generation.

The conventional operating rules of the four reservoirs in the Vu Gia Thu Bon catchment during flood season are as follows: the FLWLs have fixed values from the 1st September to 15th November. When the reservoir inflows exceed the downstream safety discharge, retaining excess floodwater in flood storage reduces the flood peaks. Once the flood has subsided, the reservoir stages should return to FLWL to keep adequate storage for other potential flood events. The reservoir is refilled to the normal water level from 15th November.

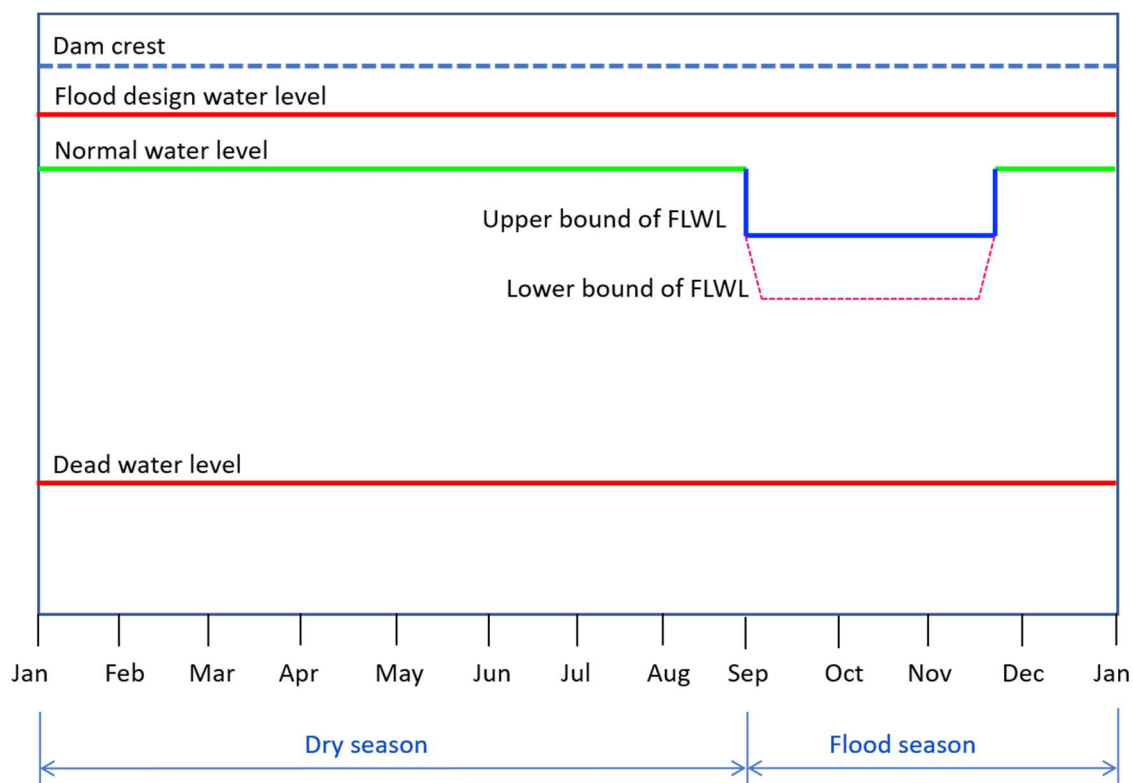


Figure 6.3. Flood limit water level (Hypothesis: flood season from 1st September to 15th December).

According to the description above, the FLWL should not be kept high during the flood season to offer adequate storage for flood prevention. In the dry season, with hydropower generation, it is difficult to avoid possible shortages of water simultaneously. The problem is that precipitations are concentrated during the flood season, from September to December, and bring over 70% of annual rainfall volumes. However, the large amount of inflows generated during this season by these intense precipitations are usually released through spillway because the reservoirs do not have enough flood control capacities. Moreover, during flood season, the reservoir stage of the four reservoirs must be lowered to the upper bound of FLWL to secure additional storage for preventing possible flooding. As a result, the decision-makers should carefully select the most appropriate FLWL for the four reservoirs by considering potential shortages downstream and available water resources for the next year.

6.3.2. Dynamic control of FLWL

Most flood control reservoirs are components of basin wide multiple-reservoir systems. Two or more reservoirs located in the same river basin will have common control points. A reservoir may have one or more control points that are influenced only by that reservoir and several other control points which are

affected by other reservoirs as well. Multiple reservoirs release decisions may be based on maintaining some specified relative balance between the percentages of flood control storage (corresponding to the FLWL) capacity utilized in each reservoir. The flood storage capacity in reservoirs system is evenly distributed or allocated by watershed area, oncoming flood volume, oncoming flood peak, reservoirs capacity and other factors.

To describe the relationship of floodwater reduction and FLWL, flood control storage is introduced as shown in Fig. 6.4. The increment in maximum release decreases with additional flood control storage ($V_4 > V_3 > V_2 > V_1$), and the floodwater reduction is an increasing concave function of flood control storage. For a typical hydrograph with the same operation rules, accumulated floodwater reduction increases at a decreasing rate with increase FLWL.

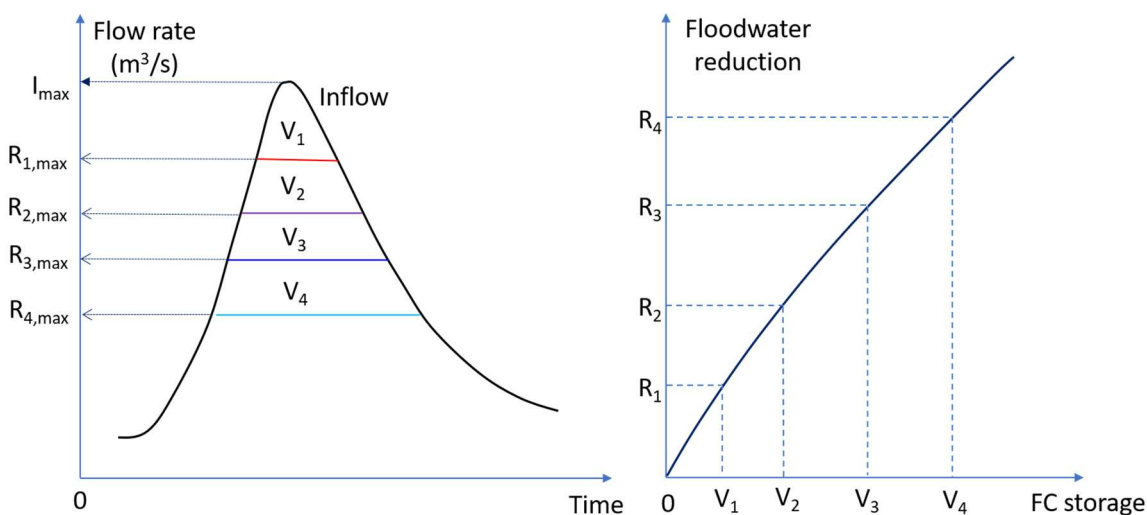


Figure 6.4. Increase of floodwater reduction with increasing flood control storage.

According to the operational regulation of the multiple reservoirs system in the Vu Gia Thu Bon catchment, the reservoir levels can fluctuate between the upper limit and the lower limit of the dynamic control of FLWL (pre-release procedure). Different storage levels and flood control characteristics of four major reservoirs in the Vu Gia Thu Bon catchment are listed in Tab. 6.1.

Operational regulation of multiple reservoirs in the Vu Gia Thu Bon catchment defines a pre-release procedure with conditions as follows (Government of Vietnam, 2015):

Before the flood occurs, the water level of the reservoir must decrease to the lower limit of the FLWL in effective lead-time in order to accommodate the impending flood with the condition water level at the control points are lower

warning level 2 (see chapter 3). However, in many cases, the flood occurs downstream before in upstream; hence the pre-strategy strategy can not operate. In this situation, the upper limit FLWL influences flood mitigation downstream.

Table 6.1. FLWL in reservoirs system (Government of Vietnam, 2015)

Reservoir	Normal water level (m)	Upper FLWL (m)	Lower FLWL (m)	Flood storage capacity (10 ⁶ m ³)
A Vuong	380	376	370	35.14
Dak Mi 4	258	255	251	31.07
Song Bung 4	222.5	217.5	214.3	75.3
Song Tranh 2	175	172	165	61.45

The objective is then to define the approach for selecting a flood storage capacity corresponding to upper limit FLWL that drives optimal flood reduction for the four parallel reservoirs in the Vu Gia Thu Bon catchment.

6.3.3. Simulation method

Mike 11 is suitable for simulation of the flow in the river system, including the reservoir operations. In the current approach, a simulation model that represents the releases from the four reservoirs, through the operational structures spillway gates, specified in Mike 11 as a control structure (gate type is radial). The gate operations are determined from a control strategy. The control strategy describes how the gate opened level depends on the value of the control point, such as the reservoir stage, the downstream water level and the time of the year. For a specific gate, it is possible to choose between an arbitrary number of control strategies by using a list of ‘if’ statements.

After the combinations of the FLWL are determined, reservoir operation is simulated according to a specific procedure. The procedure divided into three sub-procedures: normal operation, flood control and post-flood procedures. Each sub-procedure is described in detail as below:

(a) Normal operation procedure

If the inflow is high but not sufficient to be considered as a flood, the normal operating procedure should be followed to allow the proper use of high inflows. There are two situations:

- A flood occurs, but the reservoir stage is lower than the FLWL. In this situation, the reservoir still has storage for water conservation;
- The reservoir stage is in the FLWL. The release discharge equals to inflow to maintain the current stage at these reservoirs.

(b) Flood control procedure

The reservoirs have to store water to reduce the releases to downstream by the following conditions (Fig. 6.5): reservoir inflows are higher than the discharge value defined by Tab. 6.2:

Table 6.2. Defined discharge for the procedure of flood control

	A Vuong	Dak Mi 4	Song Bung 4	Song Tranh 2
Discharge (m ³ /s)	600	700	700	1500

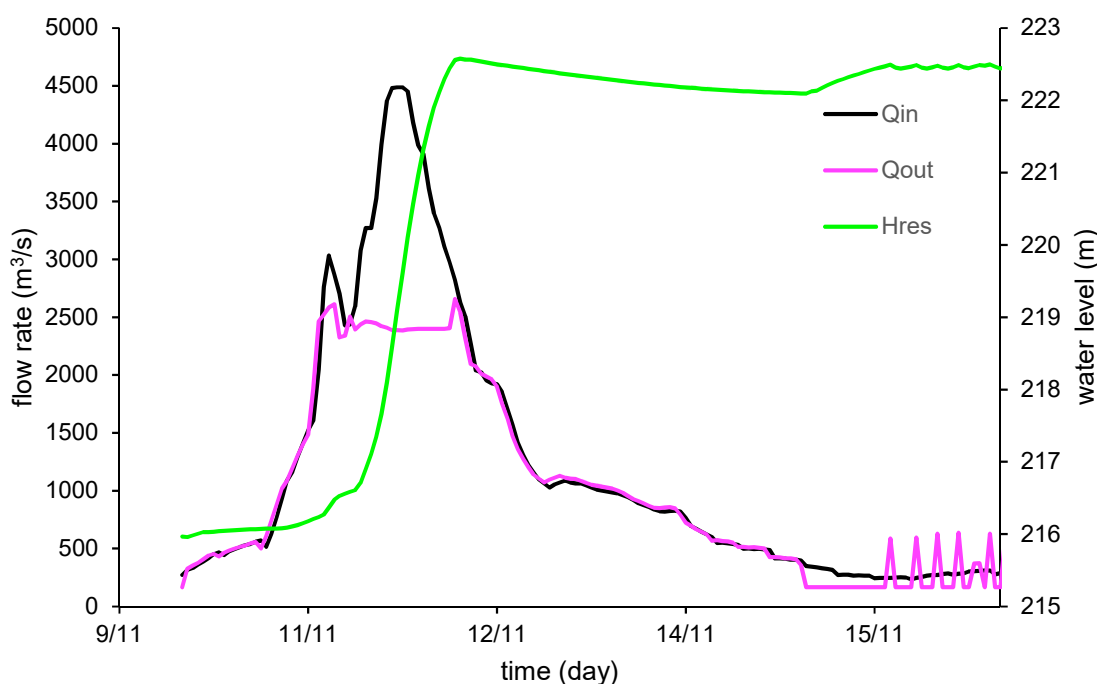


Figure 6.5. Flood control procedure.

(c) Post-flood procedure

During the flood recession period, the post-flood procedure is the method for ensuring the reservoir stage is descends to the upper bound of FLWL. When the water levels at the control points are below the alarm level 1, the total outflow must be greater than the reservoir inflow during the period from 24 to 72 hours.

6.3.4. Optimization method

The primary aim of this section is to deal with the trade-off between flood damages and hydropower generation of the reservoir systems in the Vu Gia Thu Bon basin. Two objectives that are to minimize the downstream flood peak and to maximize the hydropower potential, are introduced to reconcile these two conflicting aspects of reservoir systems operation.

(a) Objective function

After the determination of the dynamic control bounds, the simulation-based optimization model is used to find out a series of optimal combinations of the upper limit of FLWL in the multi-reservoir that can yield a good trade-off between the economic benefits of potential hydropower generation (i.e., maximizing the hydroelectricity) and risk rate of flood control (i.e., minimizing the flood damages).

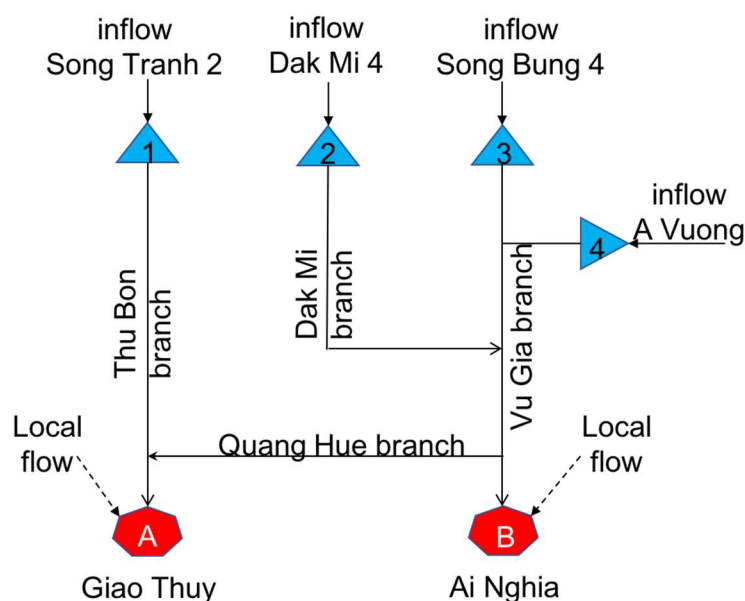


Figure 6.6. The Vu Gia Thu Bon river-reservoir system.

* *Minimizing the flood damages*

$$\min F_1 = \min \sum_{k=1}^n H_k^2 \quad (6.5)$$

where:

H_k is the maximum water level at the k^{th} control point. In the Vu Gia Thu Bon catchment, the control points are selected at Ai Nghia and Giao Thuy stations (Fig. 6.6);

n is the number of control points.

*** Maximizing the hydropower generation:**

$$\max F = \max \sum_{t=1}^T \sum_{j=1}^m K_j Q_j^t H_j^t \quad (6.6)$$

where:

K_j is the hydropower generation efficiency of the j^{th} reservoir;

Q_j is release discharge for hydropower generation of the j^{th} reservoir in period t ;

H_j is average hydropower head of the j^{th} reservoir in period t ;

T is the total number of time steps;

m is the number of the reservoir; $m = 4$.

The hydropower head depends essential to the reservoir water level during the flood season (i.e., FLWL); therefore, maximizing potential hydropower generation can express as follows:

$$\min F_2 = \left(\sum_{j=1}^m \frac{1}{T} \sum_{t=1}^T (S_t^j - S_{max}^j)^2 \right) \quad (6.7)$$

where:

S_t^j is the reservoir stage of the j^{th} reservoir in period t ;

S_{max}^j is the maximum reservoir stage of j^{th} reservoir.

*** Multi-objective function**

The two single-objective functions can be then integrated into a multi-objective function using different weights as follows:

$$\text{Minimize } F = w_1 g_1 \left(\sum_{k=1}^m H_k^2 \right) + w_2 g_2 \left(\sum_{j=1}^m \frac{1}{T} \sum_{t=1}^T (S_t^j - S_{max}^j)^2 \right) \quad (6.8)$$

where:

w_i is the weight assigned to the i^{th} objective; $0 \leq w_i \leq 1$ and $\sum w_i = 1$.

The first term on the right-hand side in Equation (6.8) defines the optimal value for flood peak at downstream control points (F_1 is the minimization of the max water level). Whereas, the second term indicates the optimal value for the potential hydropower generation during the flood season (F_2 is the minimization of deviations of reservoir levels from the normal water levels of four reservoirs).

(b) Constraints

Constraints of the reservoir operation are as follows:

* *Water volume balance*

$$V_{(t+1)} = V_t + (\Delta Q_t \times \Delta t) \quad (6.9)$$

$$\Delta Q_t = I_t - R_t \quad (6.10)$$

where:

$V_{(t+1)}$ and V_t are the reservoir storages at the time $(t+1)$ and t , respectively;

I_t is the reservoir inflow at time t ;

R_t is the release to the downstream through hydropower plant and spillway gates during period t .

* *Reservoir stage constraints*

$$S_t^{min} \leq S_t \leq S_t^{max} \quad (6.11)$$

where:

S_t is the reservoir stage at time t ;

S_t^{min} and S_t^{max} are the minimum and maximum reservoir stages during period t .

* *Reservoir discharge constraints*

$$R_t^{min} \leq R_t \leq R_t^{max} \quad (6.12)$$

where:

R_t^{min} and R_t^{max} are the minimum and maximum release at time t .

*** Feasible parameters space of FLWL**

Table 6.3. Feasible parameters space of upper bound of the FLWL

Reservoir	Variable	Present value (m)	Lower bound (m)	Upper bound (m)
A Vuong	FLWL _{AV}	376	370	376
Dak Mi 4	FLWL _{DM4}	255	249	255
Song Bung 4	FLWL _{SB4}	217.5	214	219
Song Tranh 2	FLWL _{ST2}	172	167	172

(c) Site data

The flood season in the Vu Gia Thu Bon catchment is mainly from September to December. Therefore, in the current approach, the hourly runoff data series from 1st September to 15th December are employed to find the optimal operation (Fig. 6.7).

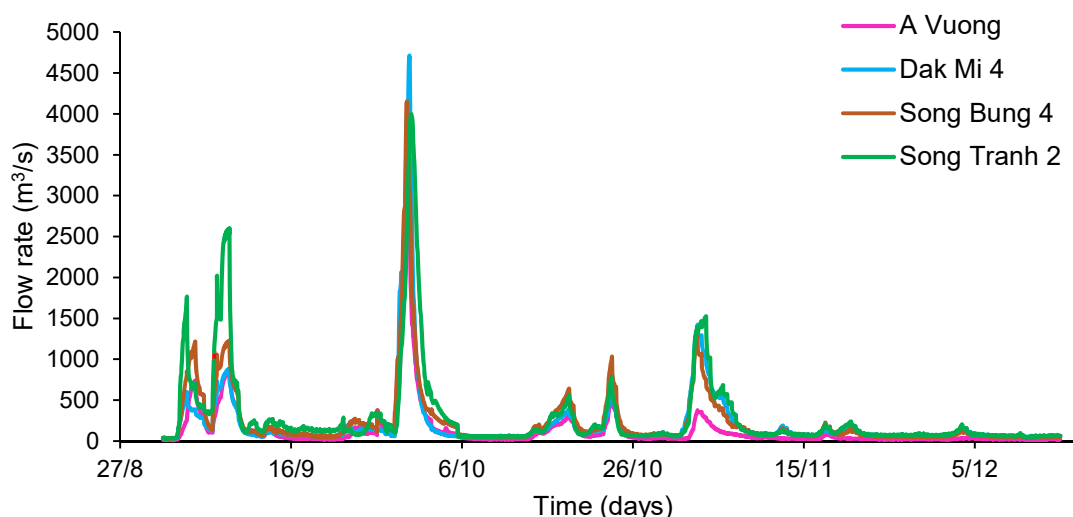


Figure 6.7. Hydrographs of the four reservoirs in the 2009 flood season (Vo & Gourbesville, 2016).

Based on the historical records, massive floods occurred in the vast majority reservoir basin of the Vu Gia Thu Bon catchment in 1999, 2007, and 2009, and their recurrence periods are about approximately 20 years (Tab. 6.4).

Table 6.4. Maximum reservoir inflow corresponding the 20-years return period.

Reservoir	A Vuong	Dak Mi 4	Song Tranh 2	Song Bung 4
Q _{max 5%} (m³/s)	3600	5370	6400	5705

However, the flood peaks of four reservoirs are different. To further explore the influence of flood processes of different magnitudes on the competition of objective functions, these floods are used to define the objective function.

6.4. Results

Preliminary optimization tests showed that after around 500 model evaluations, the entire population converged around the global optimum. The following SCE parameters were selected: the maximum number of model evaluations was 500; the number of iteration loops was 5; the minimum relative change in the objective function was 0.001.

Seven scenarios run with different weight combinations (corresponding to a total of 3500 model evaluations) were carried out to analyze the trade-off between the two objectives and shows the Pareto front as Fig. 6.8.

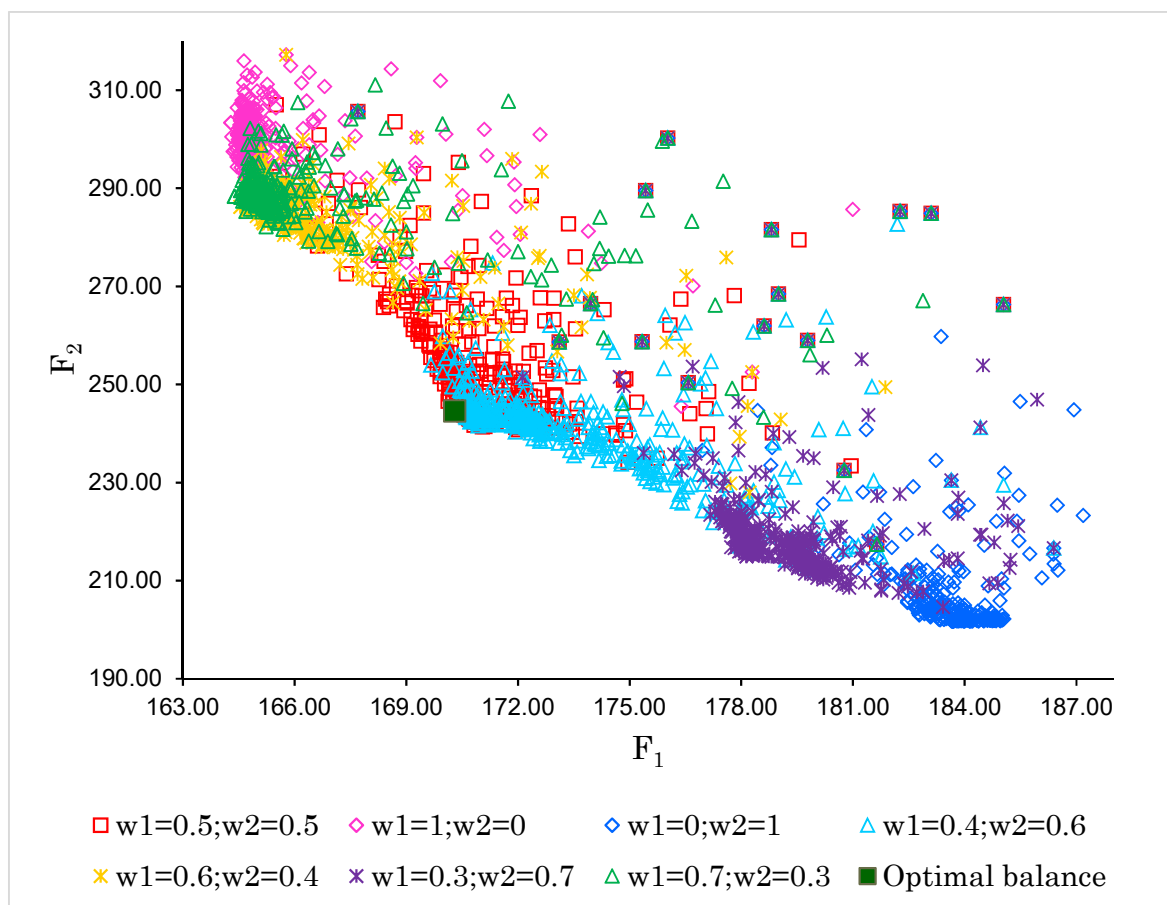


Figure 6.8. Objective function values of evaluated decision variables sets.

Two scenarios were used to estimate the tails of the Pareto front. The solutions display in the objective function of only flood control ($w_1=1$; $w_2=0$) with points purple while objective function hydropower potential generation ($w_1=0$; $w_2=1$)

with blue points. The red points are the objective function using the same weight ($w_1=0.5$; $w_2=0.5$) to define balance optimum. The last four scenarios are applied to estimate the intermediary parts.

Table 6.5. The objective functions values of optimal the upper bound of FLWL using the SCE algorithm for different values of w_1 and w_2

Objective function	Weights						
	$w_1=1.0$	$w_1=0.7$	$w_1=0.6$	$w_1=0.5$	$w_1=0.4$	$w_1=0.3$	$w_1=0.0$
	$w_2=0.0$	$w_2=0.3$	$w_2=0.4$	$w_2=0.5$	$w_2=0.6$	$w_2=0.7$	$w_2=1.0$
F_1	164.29	164.39	164.55	170.3	170.73	178.14	183.69
F_2	303.42	288.42	286.26	244.65	242.45	214.97	201.73

Table. 6.5 shows the value of objective functions with different combinations of weights. It is indicated that the objective of flood control is monotonously increased with the w_1 , and the objective of potential hydropower generation is monotonously increased with the w_2 . The values of the upper bound of FLWL with different weights are shown in Table 6.6.

Table 6.6. The optimal values of the upper bound of FLWL for different combinations of weights

Upper FLWL value	Weights						
	$w_1=1.0$	$w_1=0.7$	$w_1=0.6$	$w_1=0.5$	$w_1=0.4$	$w_1=0.3$	$w_1=0.0$
	$w_2=0.0$	$w_2=0.3$	$w_2=0.4$	$w_2=0.5$	$w_2=0.6$	$w_2=0.7$	$w_2=1.0$
$FLWL_{AV}$	370.2	371.0	371.1	375.8	376.0	376.0	376.0
$FLWL_{DM4}$	250.7	251.3	251.4	251.7	252.7	252.9	255.0
$FLWL_{SB4}$	214.1	214.4	214.4	215.6	214.7	218.9	219.0
$FLWL_{ST2}$	167.2	168.6	168.8	170.1	171.4	171.7	172.0

In the Vu Gia Thu Bon catchment, flood control is the priority of the multi-reservoir system during the flood season. Therefore, the points mostly assemble on the top left side of Fig. 6.8 where a lower value of maximum water level is more favorable. The operator can decide a single solution among objectives according to other criteria. The determination of the FLWL was an optimization issue and was subject to balance both risk and benefits constraints. In this case, the most appropriate solution could be a balanced optimum (Fig. 6.8). The final optimization results of the upper FLWL boundaries for A Vuong, Dak Mi 4, Song

Bung 4, and Song Tranh 2 reservoirs were 175.8 m, 251.7 m, 215.6 m, and 170.1 m, respectively (Tab. 6.7).

Table 6.7. The optimal values of the upper bound of FLWL for the balanced solution

Reservoir	Upper bound of FLWL			
	A Vuong	Dak Mi 4	Song Tranh 2	Song Bung 4
Present value (m)	376	255	172	217.5
Balanced solution (m)	375.8	251.7	170.1	215.6

Chapter 7

Conclusions and perspectives

7.1. Summary and conclusions

The Vu Gia Thu Bon catchment plays an important role in terms of social and economic points of view for the central region of Vietnam. Hydropower reservoirs assist a large population regarding energy supply and flood mitigation. However, reservoir operation during flood season is challenging the provision of service due to pressure of storage and flood control at downstream areas.

In this catchment, there are four hydropower reservoirs - A Vuong, Dak Mi 4, Song Bung 4 and Song Tranh 2 - that have been operated since 2009, 2011, 2012 and 2014, respectively. These hydropower plants supply 746 MW of energy and provide a flood control to local populations.

The main objective of the current research work was the development of a coupled simulation-based optimization model for optimal operation of multi-reservoir system for flood control.

1. Analysis of the existing reservoir system operation rules in the Vu Gia Thu Bon catchment using modeling and simulation technique.

The operation rules of the multi-reservoir system in the Vu Gia Thu Bon catchment have been analyzed by applying the Mike 11 river modeling tool. The model set up includes the main streams and tributaries of the Vu Gia Thu Bon basin and a logical decision tree defining the reservoir regulation. These strategies define the reservoir release as a function of the actual reservoir stage, the water level at downstream control points, and the time of the year. A data set of seven years of flood season data was used to evaluate the control strategies with respect to flood control. The most important issues of the reservoir system operation in flood season are to reduce downstream flood peak stage and store floodwaters for future uses. The obtained results show that the multi-reservoir system operation using the complete control system can reduce maximum water levels and peak discharge at downstream area. For large flood events, the maximum water levels could decrease 0.96 m and 0.92 m on average at Ai Nghia and Giao Thuy gauging stations, respectively. For the medium flood events, the multi-reservoir system operated according to current rules can reduced the maximum water level but the effectiveness of the reservoir system on flood reduction was not significant. Proposing strategies for reservoir operation of the Vu Gia Thu Bon catchment will be discussed in the next section.

2. Optimization the multi-reservoir operation for flood control using simulation-based optimization technique and applying the approach to historical floods recorded within the study area.

The primary purpose of flood control is to avoid downstream flood damages and to protect the reservoir itself. Often one reservoir's sufficient storage capacity isn't enough to secure downstream flood control points. In the situation of a multi-reservoir system, these reservoirs are required to work together to protect these flood control points from flooding. To attain these goals in this research, an algorithm was presented based on a simulation-based optimization approach.

The simulation-optimization framework is adopted for determining the spillway gates operation strategy of the multi-reservoir system in the Vu Gia Thu Bon catchment during flood events. In the proposed model, the Structure Operation (SO) module of the hydrodynamic model Mike 11 is adopted for simulation of the operation multi-reservoir considering the physical constraints of the system as well as operation policies. The Shuffled Complex Evolution (SCE) algorithm is applied to determine the best set of decision variables such as spillway gates stages and water levels at downstream control points. In the current research, the total number of parameters for optimizing is 42. The application of the proposed model to the four major reservoirs in flood events 2007, 2009 and 2017 showed optimal operation obtained by the model could efficiently reduce the flood peaks in a wide range from 10 to 26% in downstream control points compared with the current rules. The results show that the SCE algorithm is an efficient tool for optimizing complex systems.

3. Optimal design flood limit water level using combination of simulation and optimization models.

The reservoir systems in the Vu Gia Thu Bon catchment performs both flood control and hydropower generation. The fact that the steep slope of the mountainous topography significantly limits the storage capacity of the reservoirs in this catchment area. The Flood Limit Water Level (FLWL) is a key parameter to balance flood control and water conservation during flood seasons. As a consequence, the optimization of the FLWL in flood season plays a significant role in flooding control in this catchment.

The current research was also aiming to optimize the FLWL for parallel reservoir system with multi-objective by applying a combination of simulation and optimization models. An optimization algorithm was introduced and the

maximum water level of the downstream control points and hydropower potential generation in the flood season are used as objective functions. The popular Shuffled Complex Evolution (SCE) global optimization method that is implemented in the AutoCal software, was used to couple with the hydrodynamic Mike 11 model for optimizing the FLWL boundary. However, the objectives are often in conflict with each other and are calculated with variable characterized by different units. Thus, when there are two or more performance measures, one of the most critical components of multi-objective problem solving is to define how to evaluate the parameter sets. In the current research, the aggregation approach is applied. The multi-objective optimization problem explores the entire Pareto front between the objective functions by performing several optimizations runs using different weights. The results demonstrate that an optimized FLWL can be found that compared to the current rules in reduce downstream flood peaks. In summary, experimental results show that the multi-objective optimal scheduling model established in this study can provide decision-makers with a set of alternative feasible optimized scheduling schemes by considering the two objectives of minimizing flood damages and maximizing hydropower generation.

4. Strategic proposal for reservoir operation of the Vu Gia Thu Bon catchment.

The current operation rules of the multi-reservoir system in the Vu Gia Thu Bon catchment could be improved by applying the following regulations:

- The storage levels in the reservoirs should be kept below the upper bound of flood limit water level (FLWL) that are defined in chapter 6, which could provide enough storage for flood mitigation. These optimal FLWL values are expected to contribute in reducing downstream flood peaks compared to the current rules.
- The objective of pre-release procedure is to estimate a proportion of the usable volume of reservoirs before flooding occurs and allows the reservoirs to reduce the water levels. In this case, 24-48h forecasts of both reservoir inflows and water levels at the control points are used in the formulation of the reservoir operations.
- In the second flood stage - flood control procedure, the released amounts are less than the reservoir inflows. The reservoirs need to store water to reduce the flows to downstream according to the water levels at the control points. Which means that the reservoirs start the optimal

operations when the water levels are 8.7 m at Ai Nghia station for A Vuong, Dak Mi 4, Song Bung 4 reservoirs and 3.6 m at Cau Lau station for Song Tranh 2 reservoir.

7.2. Perspectives

From the experience gained with coupled simulation-based optimization models, several adjustments were made to better fulfill the demands of users and population. With its tributaries and numerous reservoirs such as in the Vu Gia Thu Bon catchment, the entire river system forms a complex network for flow storage, control and transportation. In such system, reservoirs operations are important for energy production and for flood control for the safety of people and properties. For real-time operations, the proposed strategies are the following:

1. Development of an inflow forecasting model using a distributed hydrological model.

The forecasting model can solve a major issue in real-time management of reservoir systems for more reasonable and reliable hydrologic forecasts at least 24 or 48 h ahead.

2. Development of a decision support system (DSS) for flood management decision.

A Decision support system (DSS) is a computerized program used to help courses of action, determinations and judgments in a business and an organization (Gourbesville et al., 2016; Yuliantini et al., 2019). A DSS collects and analyzes large quantities of data, compiling detailed information that can be used to solve problems and in decision-making (Gourbesville et al., 2018). A growing area of DSS application for water resources engineering, such as real-time water management (Gourbesville et al., 2018), optimizing reservoir operations (Alemu et al., 2010).

The DSS is expected to integrate real-time procedures depending on data, potential optimization method and recent government documentations to make decisions for multi-reservoir operation during flood season. The DSS could provide a series of recommendations for the quantity and timing of reservoir releases to optimize electrical energy produced while balancing requirements concerns related to flooding control. The DSS could be described as it incorporates two integrated models of system operation: a simulation model and

an optimization model. The DSS for real-time multi-reservoir operation can be organized according to the following four procedures:

Normal procedure:

The normal procedure could cover two situations: (i) there is no flood forecasted in the effective lead-time, (ii) a flood is forecasted during the effective lead-time, but the reservoir stage is lower than the lower limit of the FLWL.

- In the first situation, the reservoir operates for hydropower generation according to the rules.
- In the second situation, the reservoir storage water, but the reservoir stage cannot surpass the upper limit of the FLWL. When the reservoir stage reaches the upper limit of the FLWL, the release equal to inflow.

Pre-release procedure:

Before occurring of flooding, the reservoir stage must decrease to the lower limit of the FLWL in effective lead-time. The hypothesis is the reservoir stage before the flood occurs in the bound of FLWL. When a flood is forecasted to happen during the effective lead-time, giving right decisions for pre-release procedure based on the current reservoir stage and regarding the total inflow can make significant contribution for the next stages.

Flood-control procedure:

The objective of the flood control procedure is expected to reduce the maximum water levels at downstream areas. The development of real-time operating rules can be based on Machine Learning algorithms. Machine learning technic is potentially useful to obtain experience and rules from the historical data of inflow and release. These algorithms can be used for real-time reservoir operation. The coupled simulation-based optimization model can be used as an off-line mode for optimization of multi-reservoir system operation using combination inflows.

Machine learning models can be trained by using the results of the multi-reservoir operation optimization model, including data of inflow and release historical data. For practical uses, it is possible for combining the reservoir operation with inflow prediction by a distributed hydrological model.

Post-flood procedure:

During the flood recession period, this procedure cover two situations: (i) the reservoir stage has been wrongly lowered under the lower limit of the FLWL, (ii) the reservoir stage is higher than the upper limit of the FLWL.

- In the first situation, if incorrectly forecasted of inflow happened, this stage would propose decision for ensuring water storage in target reservoirs to the upper limit of the FLWL.
- In the second situation, the release volume can be estimated as an amount that is larger than inflow to decrease the reservoir stage to the upper limit of the FLWL.

3. Development flood inundation and damage estimation model.

The current research has been focused on the maximum water levels monitored at downstream control points. The development of depth-damage functions for flood damage estimation can help operators in developing mitigation actions and projects. The flood damage estimation model can be coupled with reservoir operation model for calculating flood damages in downstream areas and can be fine-tuned for minimizing impact of flood events.

Finally, the coupled simulation-based optimization model could provide a framework for reservoir operation in other river-reservoir systems in Vietnam. Although there are specific differences between distinguish river systems in Vietnam such as flood season period and characteristic reservoir systems, the majority reservoir systems are more likely face the same challenges. In terms of future climate changes, a comprehensive study considering the hydrological, socio-economical is necessary. The results will contribute to enhance the properly evaluation of the trade-off between flooding risks and economic benefits, and also to promote the feasibility of this significant operational change.

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Appendix

The inflows used in the modeling approach are extracted from the Mike SHE model (Vo, 2015) that is established over 10.350 km² of the Vu Gia Thu Bon catchment. This appendix describes the model setup, calibration, validation and main results.

A. Mike SHE descriptions

The Mike SHE model is a deterministic distributed hydrological model, developed and extended by DHI Water & Environment since the last decades of the 20th century (DHI 2012). Mike SHE covers the major processes in the hydrologic cycle and includes process models for evapotranspiration, overland flow, unsaturated flow, groundwater flow, channel flow, and their interactions. Each of these processes can be represented at different levels of spatial distribution and complexity, according to the goals of the modeling study, the availability of field data, and the modeler's choices (Butts et al.).

Due to its performances, Mike SHE has been used in a broad range of applications. It is being used operationally in many countries around the world by organizations ranging from universities and research centers to consulting engineering companies. Mike SHE has been used for the analysis, planning, and management of a wide range of water resources and environmental and ecological problems related to surface water and groundwater, such as river basin management and planning, water supply design, management and optimization, irrigation and drainage, soil and water management, groundwater management, interactions between water surface and groundwater, ecological evaluations flood plain studies, impact of land use and climate change.

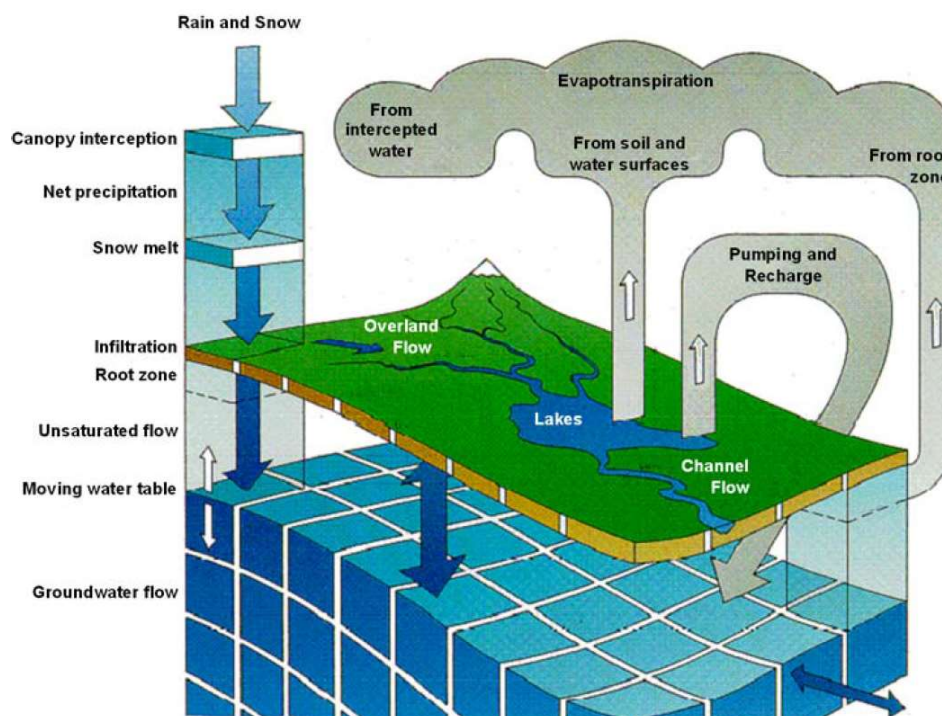


Figure A1. Schematic of Mike SHE model (DHI, 2012).

One of the advantages of fully deterministic distributed models is the possibility of overcoming the weakness and the lack of systematic data. Regarding the specific issues in the Vu Gia Thu Bon catchment, Mike SHE represents a reasonable alternative for hydrological modeling in a data scarcity situation. Mike SHE is a deterministic modeling system based on physical laws. Most of the used parameters are physical variables that evolve in a range of values that can be defined according to physical processes. In such context, realistic assumptions could be made on such variables and then allow the development of an efficient hydrological model over the large Vu Gia Thu Bon catchment.

B. Model setup for Vu Gia Thu Bon catchment

1. Topography

The elevation data used in the model is taken from SRTM DEM with the horizontal resolution 90m from NASA (<http://www.cgiar-csi.org>).

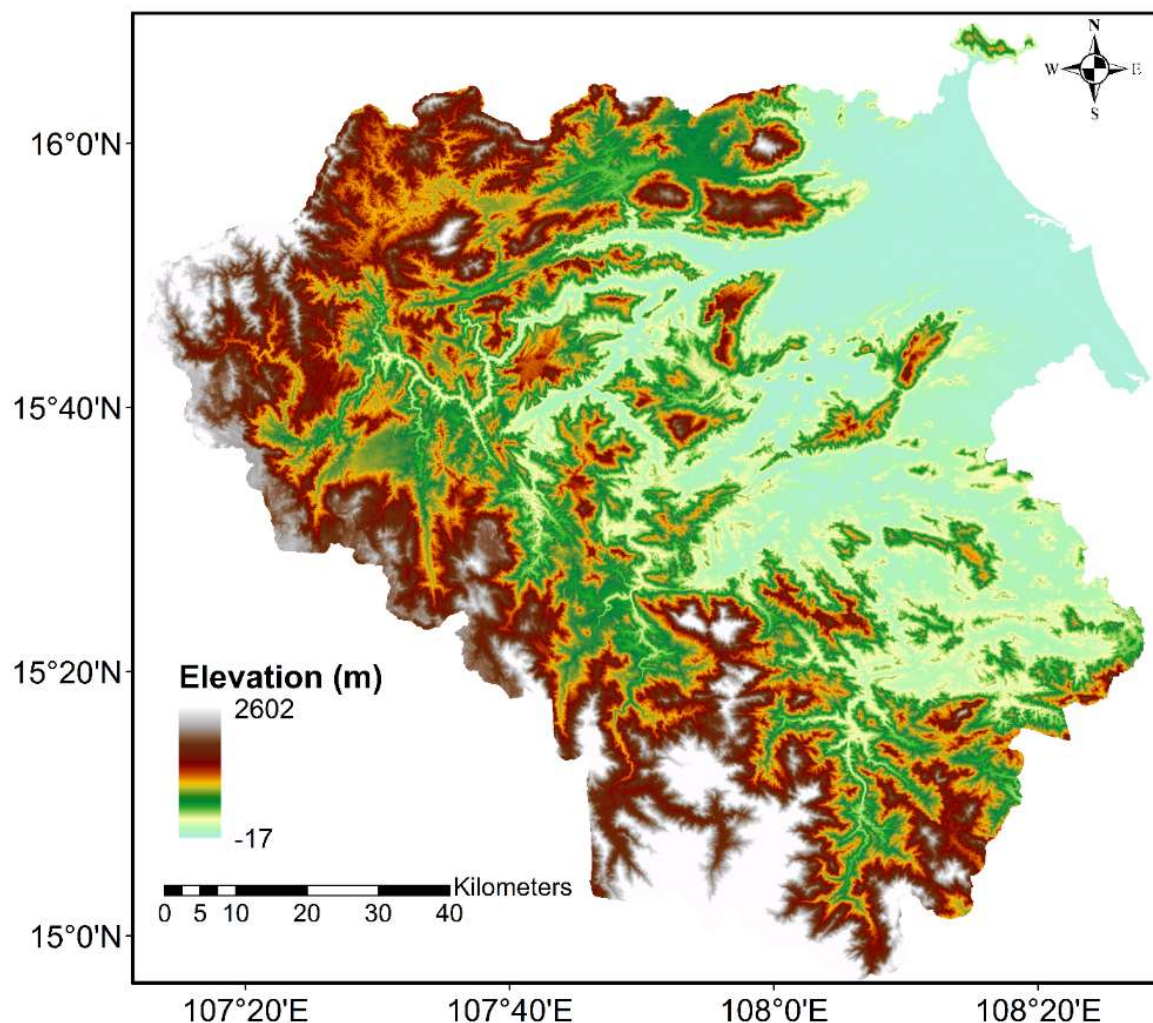


Figure A2. Topography of the Vu Gia Thu Bon catchment.

2. Land use and soil maps

The land use and soil data are simplified from the data of project Land Use and Climate Change Interaction in Central Vietnam (LUCCI) and project Impacts of Climate Change in Mid-Central Vietnam (P1-08 VIE). The input data are defined with five types of soil and nine types of land use.

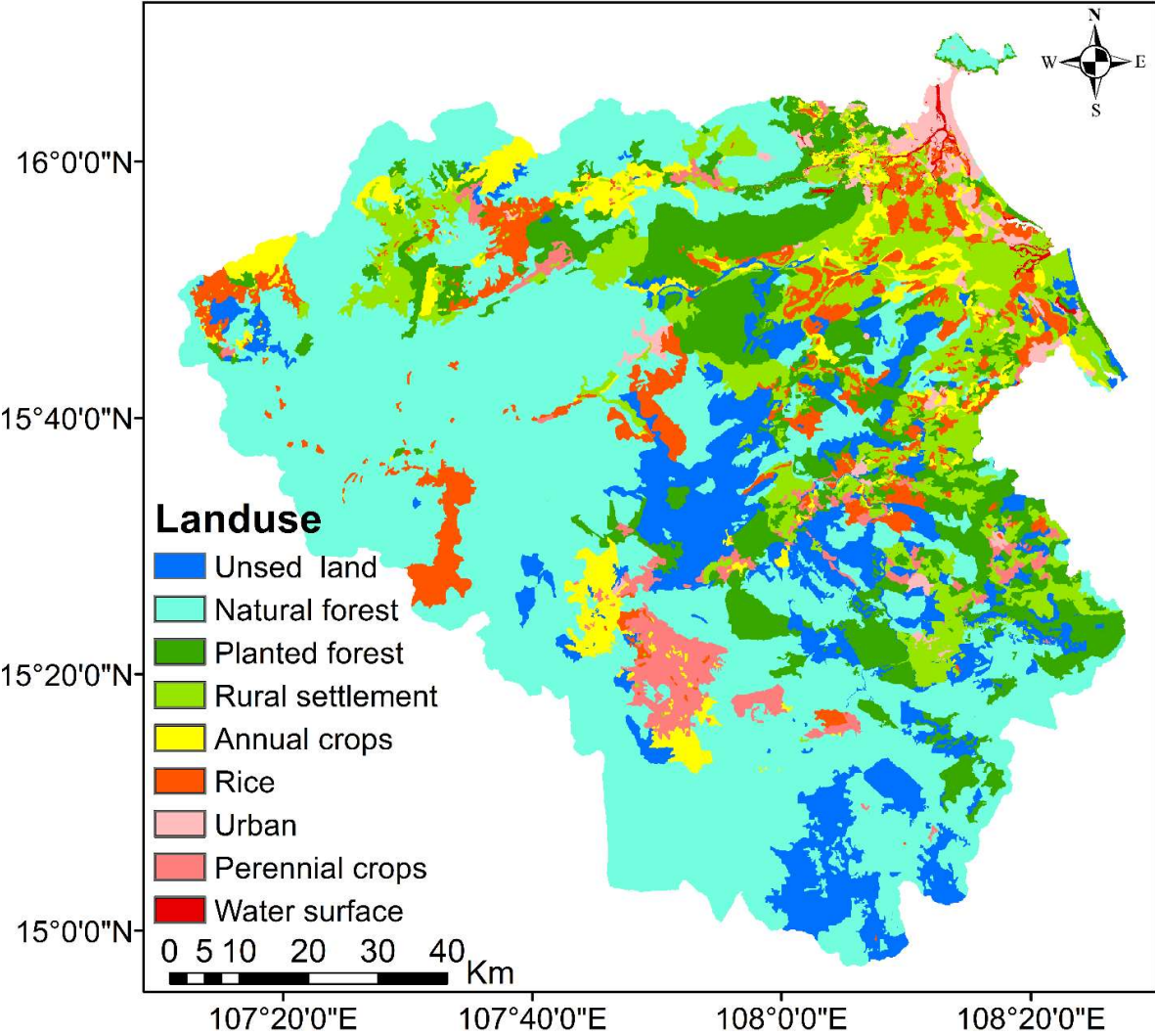


Figure A3. Land use map at Vu Gia Thu Bon catchment.

The soil map is supplied by Vie 08-P1 project that describes 44 types of soil of catchment. This map is resampled to 5 principal soils types as Figure A4. In these components, clay and silt loam are more than 80 % of total. Hence, they are judged to be two factors deciding on the infiltration and base flow in this catchment.

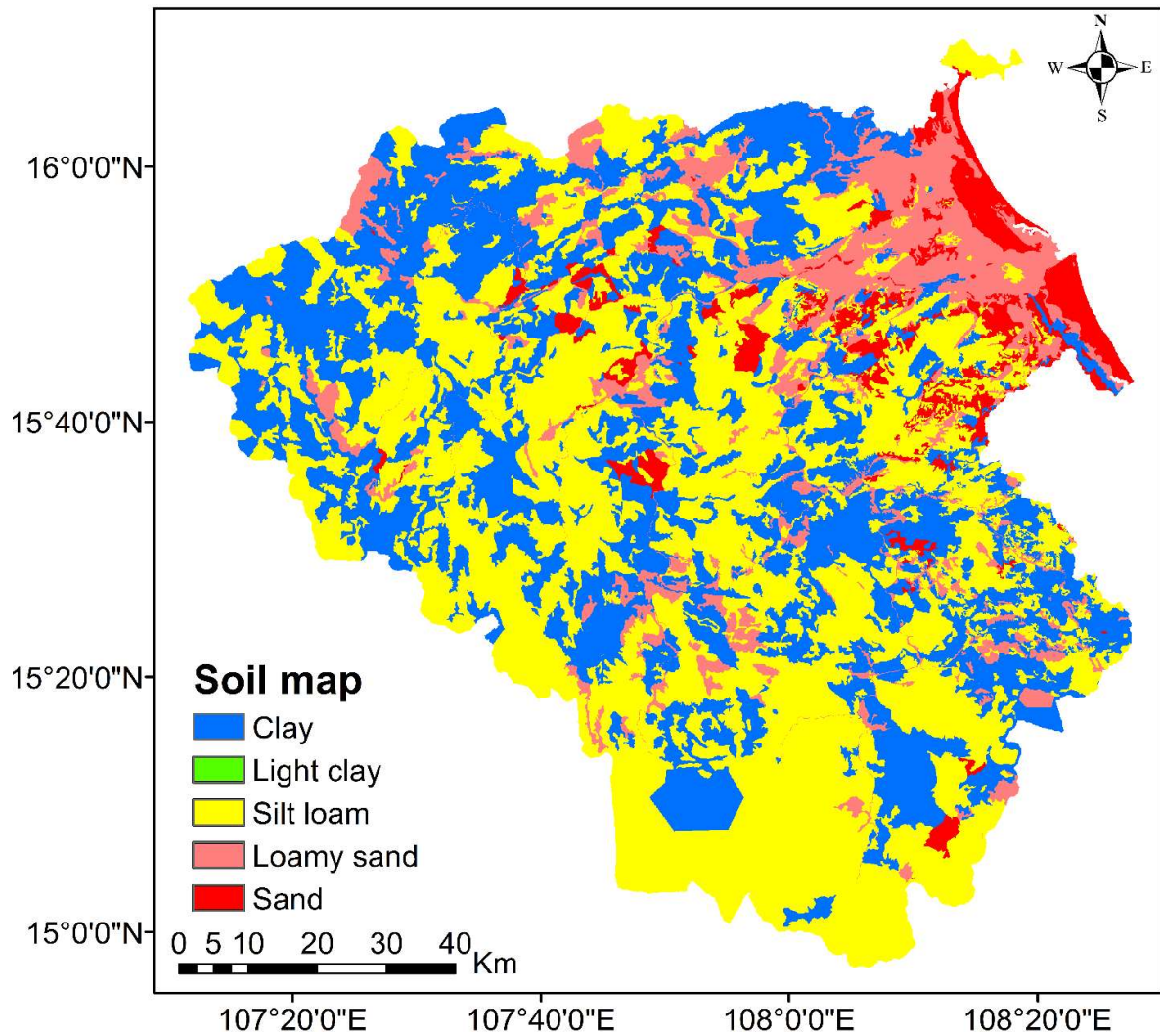


Figure A4. Soil map at Vu Gia Thu Bon catchment.

3. Vegetation

The harvest schedule is set up for main plants such as forest, homestead, rice, sugarcane, and grass. Each kind of crop is specified by vegetation property. The vegetation property in this simulation is from DHI results (DHI 2012a).

4. Evapotranspiration

Data are inherited from the study of (Vu et al.). These authors calculated the potential evapotranspiration in Nong Son basin by using the Penman–Monteith equation. A monthly mean potential evapotranspiration for each vegetation type and average over the catchment were constructed.

5. Precipitations

the simulations use the rainfall data that are re-distributed spatially based on daily rainfall data from 15 rain gauge stations with the Kriging method.

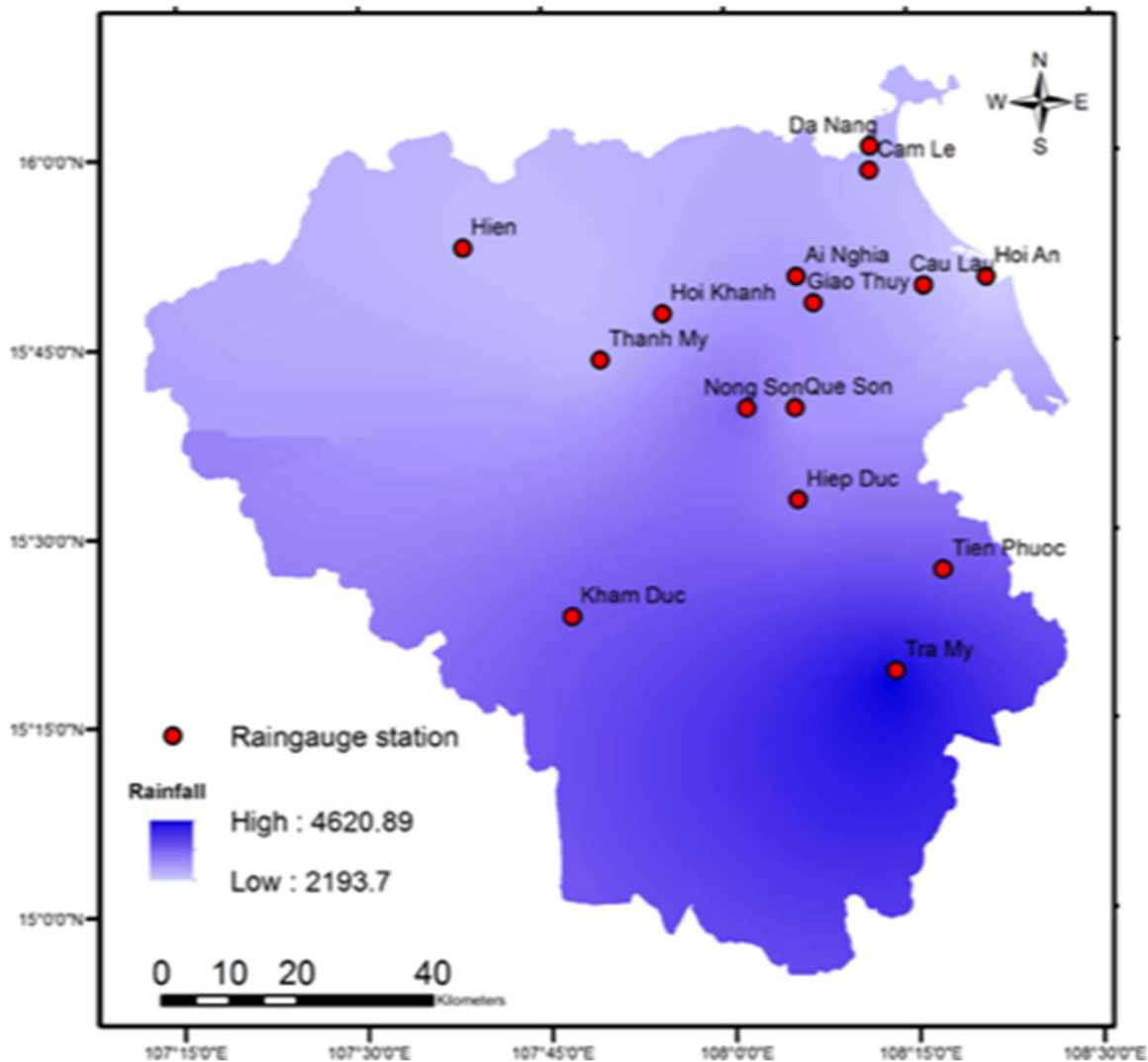


Figure A5. The Annual rainfall interpolation result at 15 rain gauge station correspondent with Kriging method.

6. River and lakes

In order to simulate the river flows better, the MIKE SHE model is coupled with a hydrodynamic MIKE 11 model (1D model). The model is developed over 44 major branches with a length varying from 20 to 202 km. The geometry of each river branch is specified via cross-sections. The cross-sections applied in this model are from two sources: a few of them at downstream are taken from the measurements and the remaining ones are extracted from the DEM. The initial bed resistance is set up with Stickler roughness coefficient (M) varying from 15 to 25 $m^{1/3}/s$ for upstream tributaries, and the value changing from 30 to 50 $m^{1/3}/s$ for downstream branches.

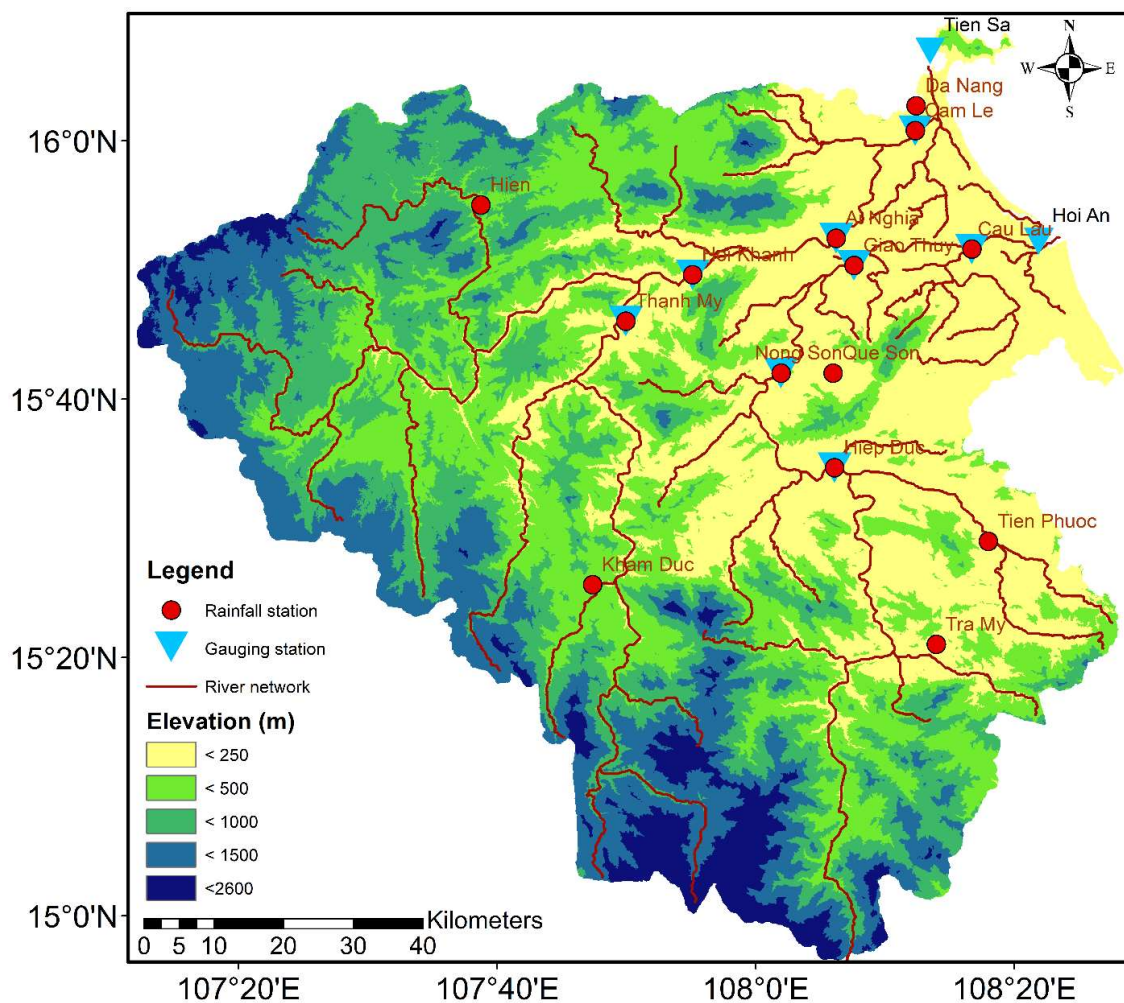


Figure A6. River network and hydro-meteorological station at Vu Gia Thu Bon catchment.

7. Overland flow

The overland flow appears after the net rainfall rate exceeds the infiltration capacity of the soil, water is then ponded on the ground surface. The main parameter to calculate this flow is Stickler roughness coefficient (M). For Vu Gia Thu Bon, this parameter is determined depending on the land use map and at 2-90 m^{1/3}/s.

8. Unsaturated zone

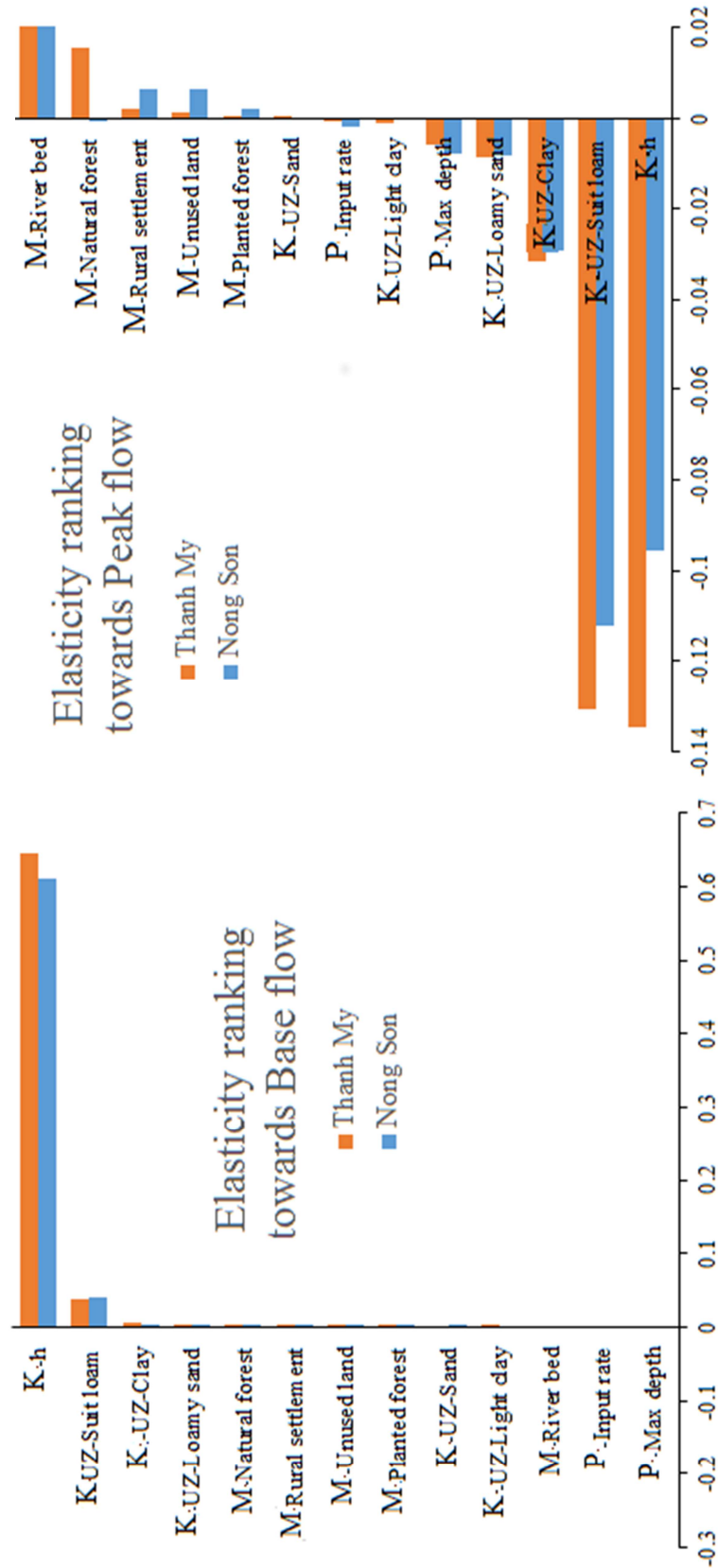
DHI suggested three methods for describing the flow in this zone: Richards' equation, gravity flow and two-layer UZ. However, the application demonstrates that the various approaches do not provide very different results. For the current application, the simple two-layer water balance method is chosen to reduce the computational time. The physical property of each soil type is presented via the water content at saturation, water content at field capacity, water content at wilting point, and saturated hydraulic conductivity.

9. Saturated zone

The groundwater is supplied by Central Vietnam Division of Water Resources Planning and Investigation (<http://www.ceviwrpi.gov.vn>). The characteristic of the aquifer is mainly presented by horizontal hydraulic and vertical hydraulic conductivities.

C. Results

1. Elasticity ranking of peak and base flow due to the input parameter changes.



2. Calibrated parameter values of MIKE SHE model.

Key parameter	Unit	Optimal value
* River bed resistance - Strickler Coefficient		
- Tributary and upstream of Vu Gia	m ^(1/3) /s	18
- Tributary and upstream of Thu Bon	m ^(1/3) /s	25
- Linking branch	m ^(1/3) /s	30
- Downstream	m ^(1/3) /s	40
* Overland flow - Strickler Coefficient		
- Planted forest	m ^(1/3) /s	5
- Rural settlement	m ^(1/3) /s	8
- Rice	m ^(1/3) /s	16
- Annual crops	m ^(1/3) /s	8
- Perennial crops	m ^(1/3) /s	8
- Unsed land	m ^(1/3) /s	5
- Natural forest	m ^(1/3) /s	2
- Urban	m ^(1/3) /s	90
- Water surface	m ^(1/3) /s	33
* Unsaturated flow - soil property		
- K _{uz} -Clay	m/s	1.2 10 ⁻⁸
- K _{uz} -Suit loam	m/s	2.45 10 ⁻⁶
- K _{uz} -Loamy Sand	m/s	8.5 10 ⁻⁶
- K _{uz} -Light clay	m/s	2.085 10 ⁻⁴
- K _{uz} -Sand	m/s	2.89 10 ⁻⁴
* Saturated zone		
- K _h - Horizontal hydraulic conductivity	m/s	6.7 10 ⁻⁵

3. Calibrated and validated hydrographs of discharge

Hydrographs in Fig. A7-A8 demonstrate that the model simulates relatively accurately the runoff in Vu Gia Thu Bon catchment. Simulated base flows at the two stations Nong Son and Thanh My are similar to the measurements. However, it seems that the peak of sub-main flood is not presented well. The quality of observation data may cause this limitation. In dry season, the data in these two stations are only captured once or twice per day, so could not present precisely the time of the sub-main flood appearance. It is really difficult to overcome the problem concerning missing data, so the simulated base flow might be acceptable. Following the hydrographs, peak floods are almost the same as observation data.

The efficiency of the MIKE SHE model is also shown through the statistical coefficients in Tab. A1. Observed and simulated daily and monthly discharges are compared. These numbers demonstrate the accuracy of the model and its efficiency for describing the hydrological processes of Vu Gia Thu Bon catchment. The R and E coefficients at Nong Son and Thanh My for the calibration period are 0.92, 0.89 and 0.82, 0.78, respectively. In the validation period, these factors slightly decrease. R and E coefficients at Nong Son station are 0.91 and 0.82 and at Thanh My, 0.90 and 0.69. The RMSE coefficients at Nong Son and Thanh My in both periods are relatively small.

From the previous results and regarding the specific issues in the Vu Gia Thu Bon catchment, the model efficiency is likewise confirmed by the capacity to predict extreme peak flow and baseflow. Therefore, reservoir inflow series are extracted from the hydrological Mike SHE model. The figures A9-A15 shows time series of reservoir inflow in the flood season from 2003-2009. Once the reservoir hydrographs are obtained, they are entered into the SO module as inputs.

Table A1. Statistical indices of Mike SHE model in Vu Gia Thu Bon catchment.

	Station	Daily			Monthly		
		RMSE	R	E	RMSE	R	E
Calibration (1991-2000)	Thanh My	132.3	0.89	0.78	58.06	0.96	0.89
	Nong Son	288.7	0.92	0.82	160.4	0.97	0.86
Validation (2001-2010)	Thanh My	123.2	0.9	0.69	47.03	0.96	0.87
	Nong Son	250.5	0.91	0.82	131.0	0.97	0.87

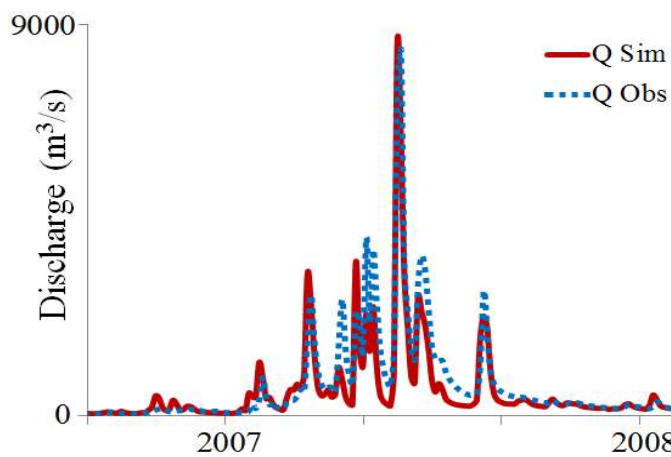
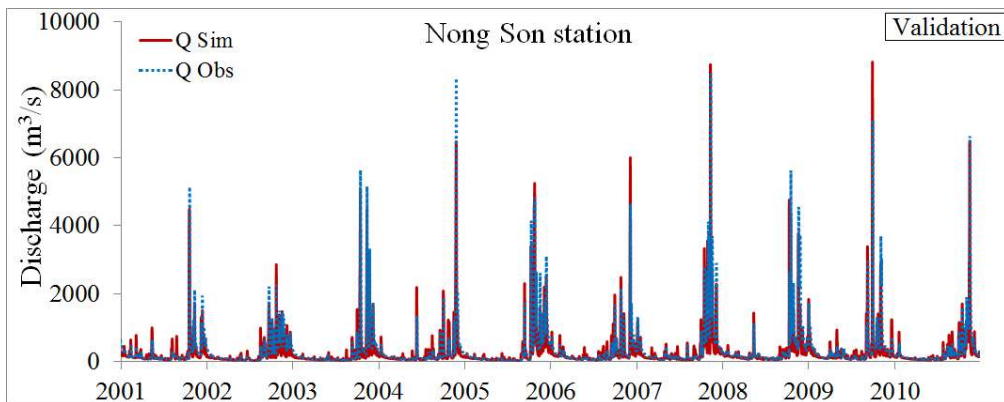
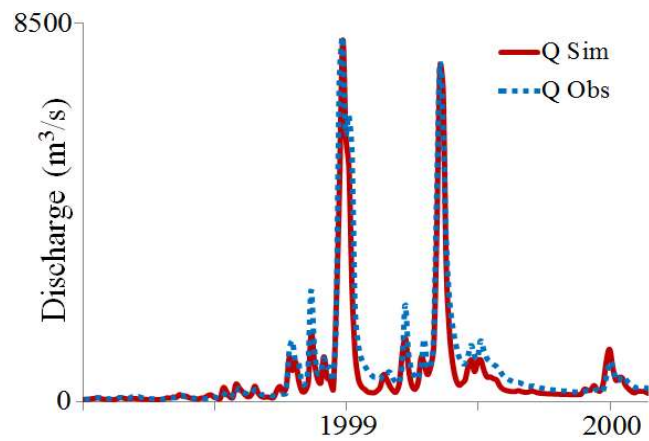
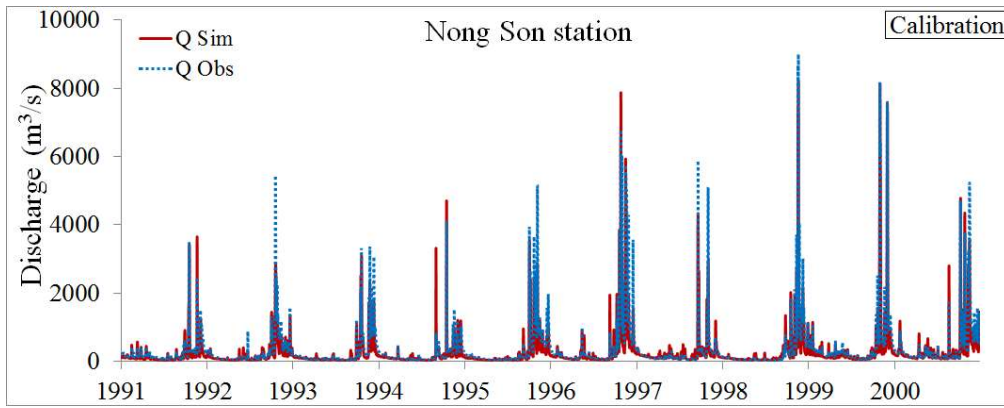


Figure A7. Calibrated and validated hydrographs of discharge at Nong Son station.

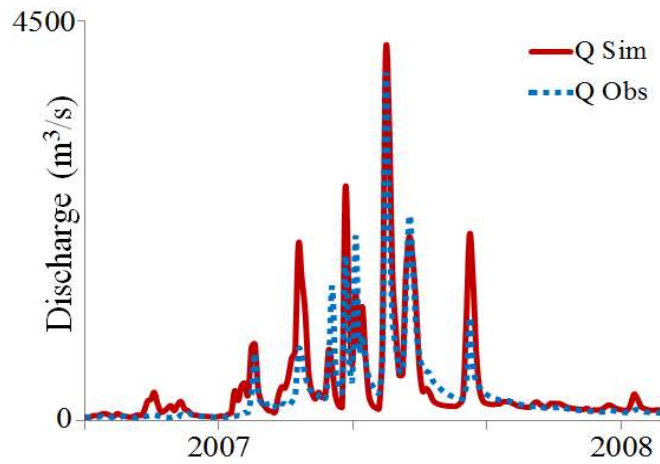
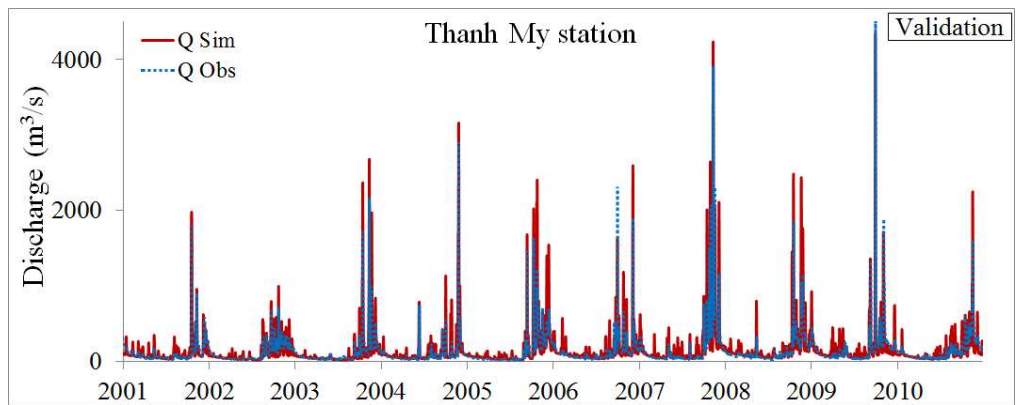
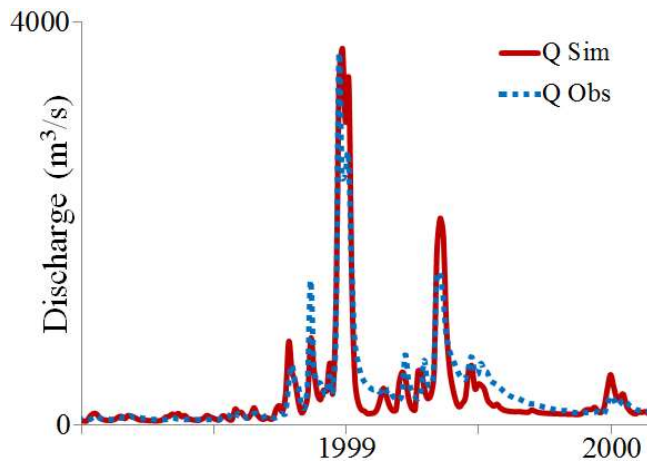
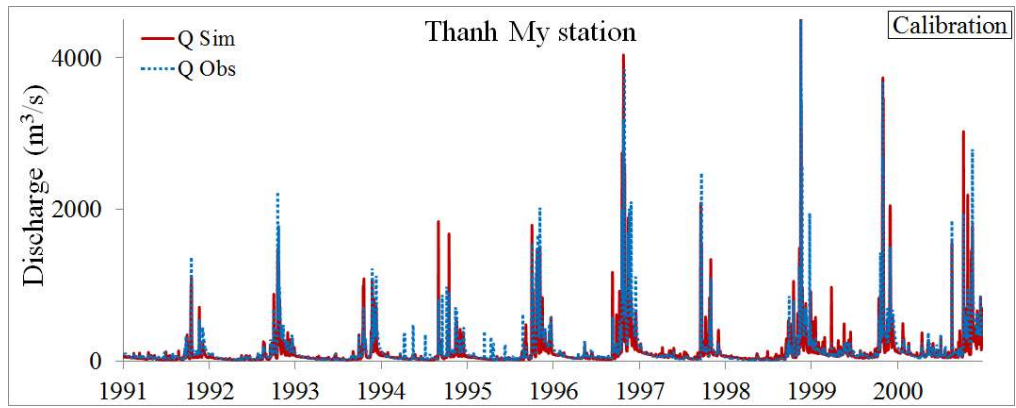


Figure A8. Calibrated and validated hydrographs of discharge at Thanh My station.

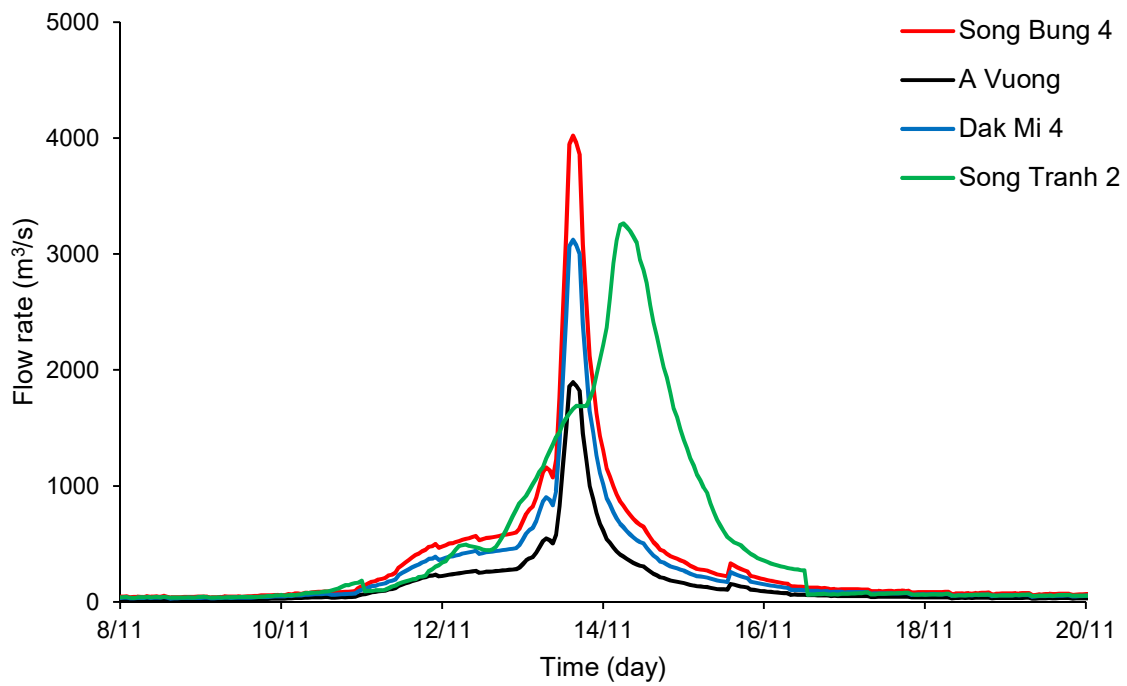
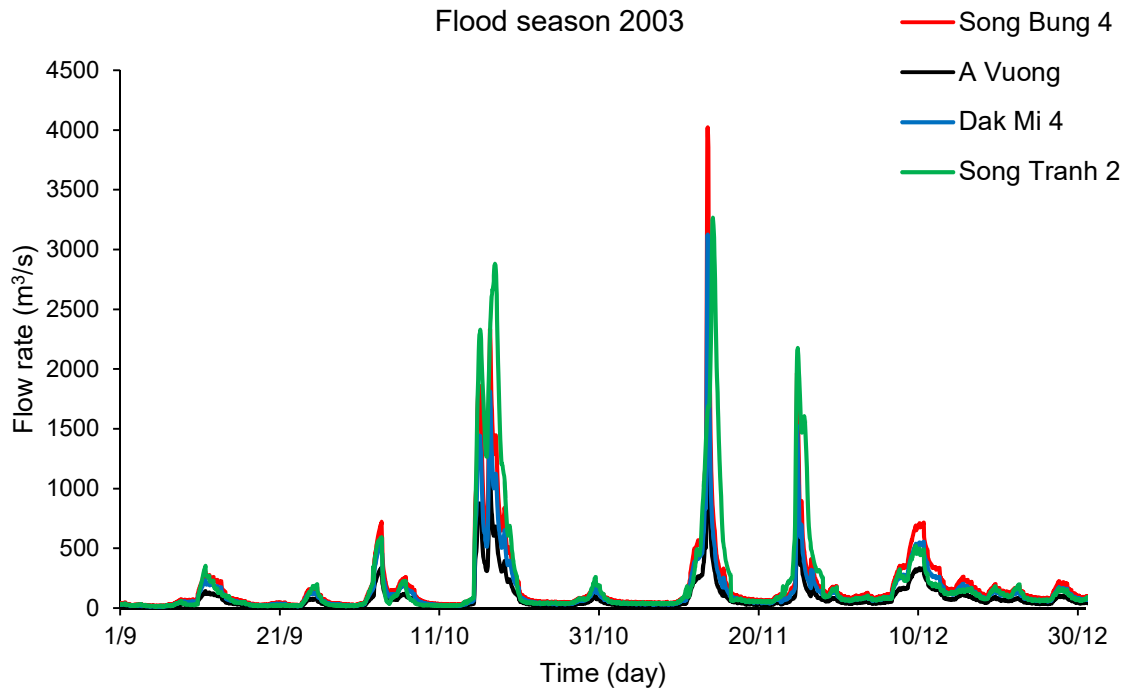


Figure A9. Inflow reservoir hydrographs in the 2003 flood season.

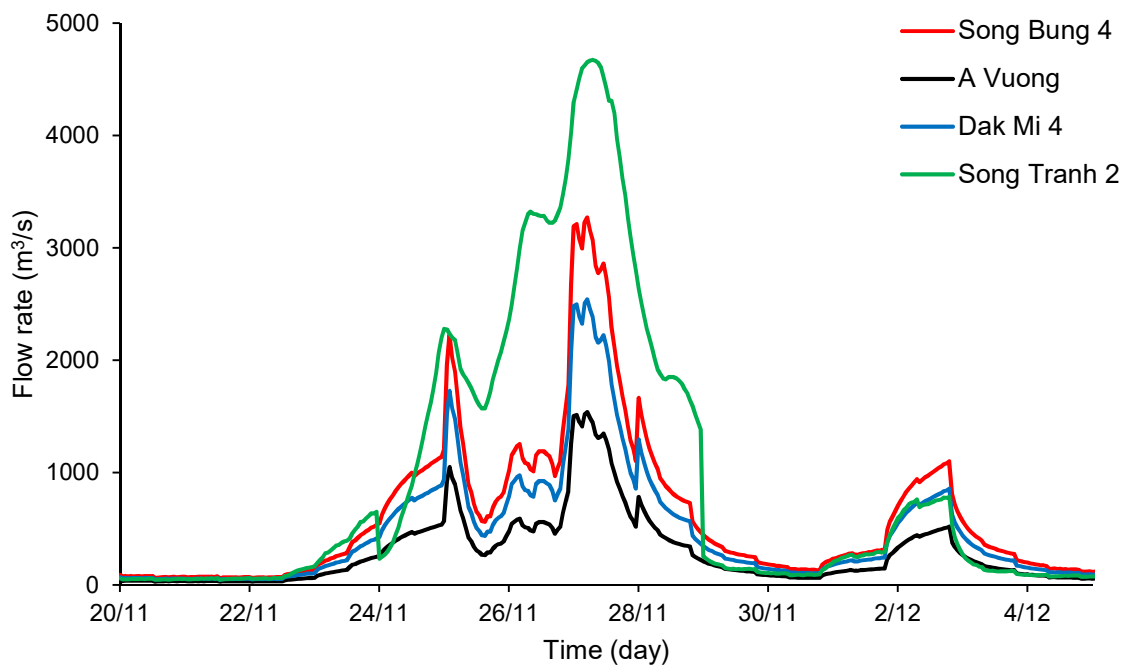
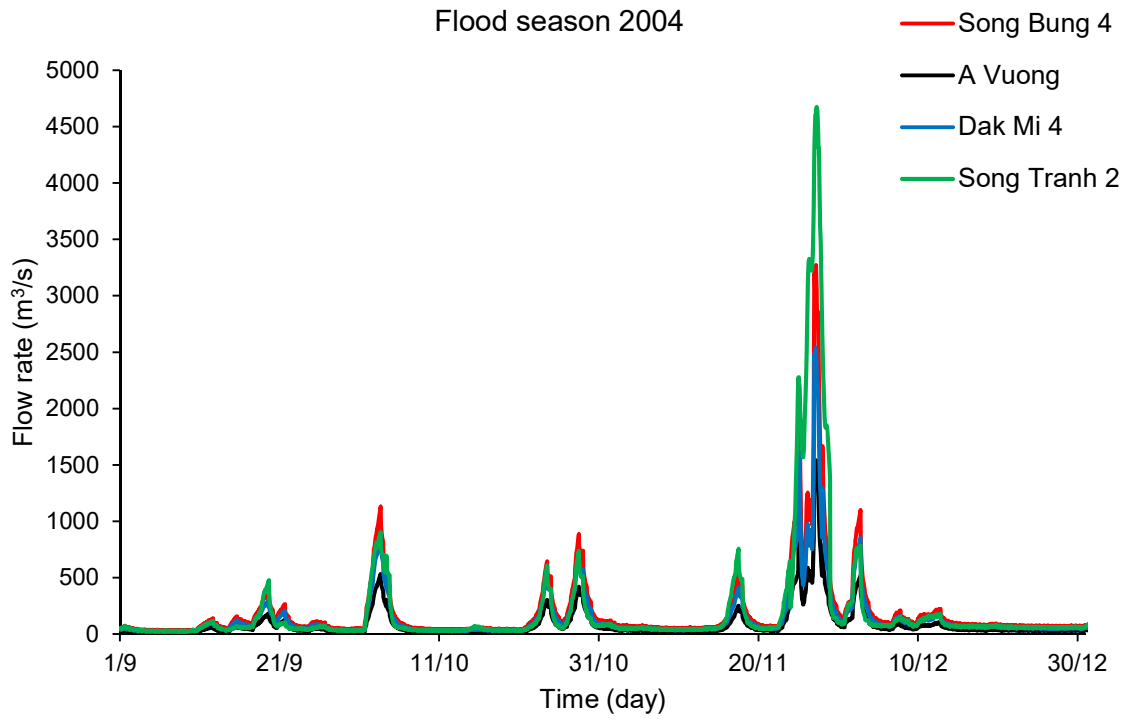


Figure A10. Inflow reservoir hydrographs in the 2004 flood season.

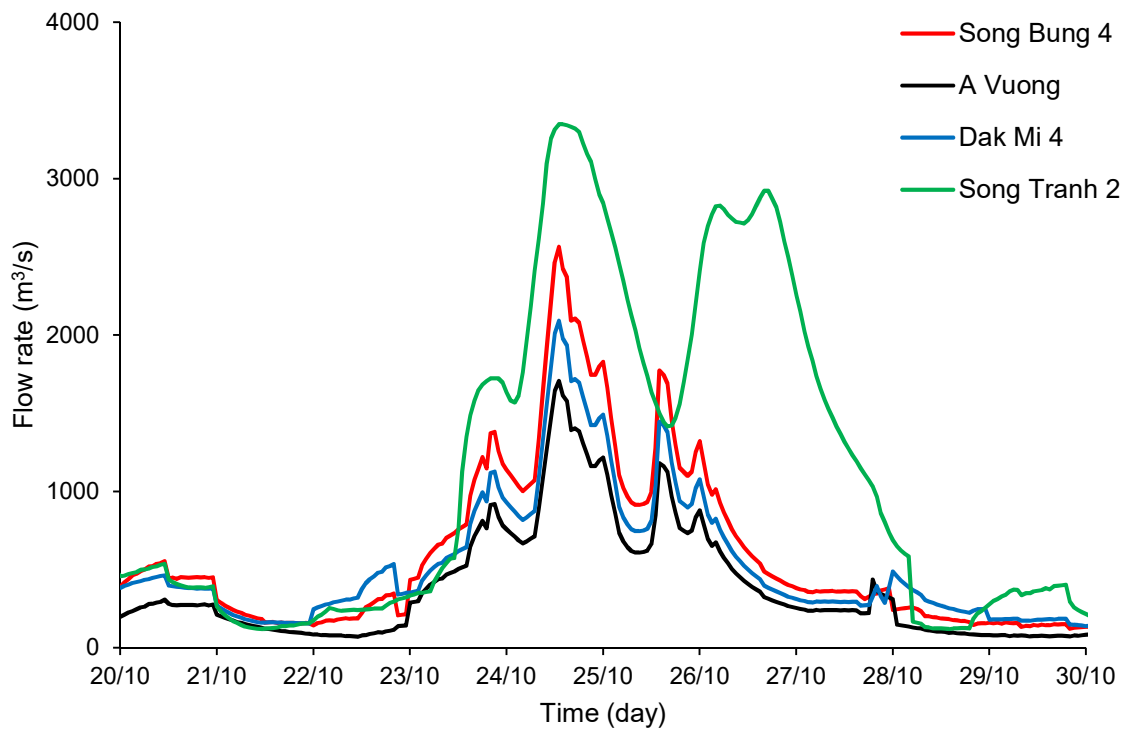
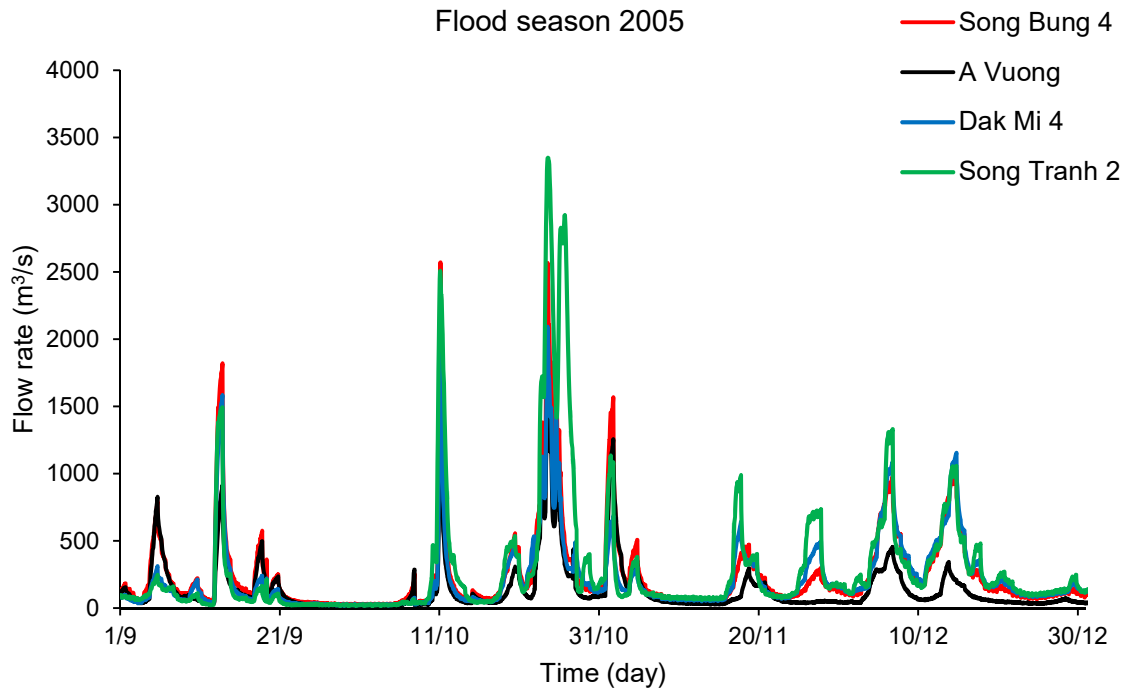


Figure A11. Inflow reservoir hydrographs in the 2005 flood season.

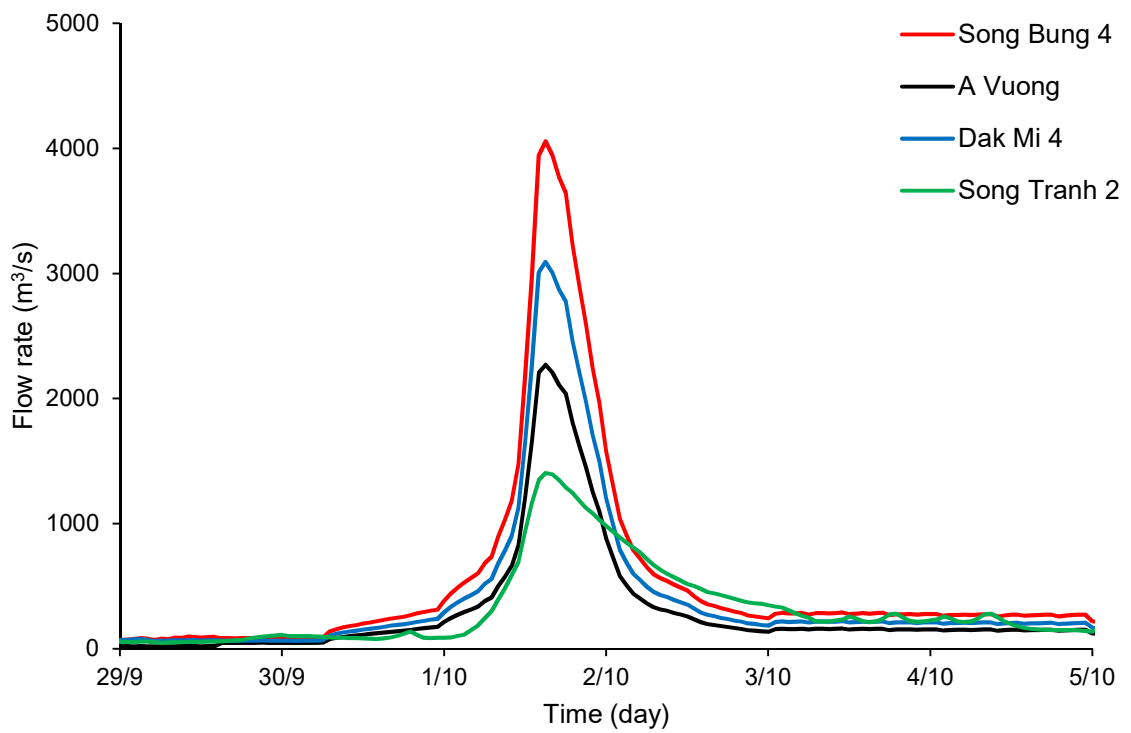
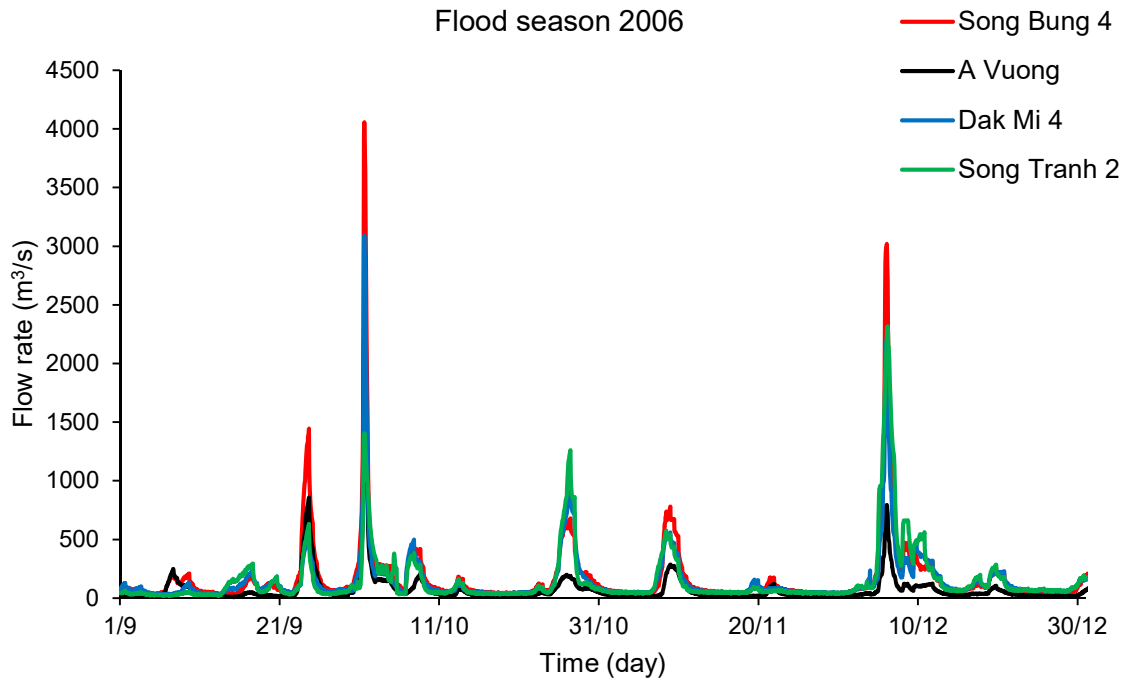


Figure A12. Inflow reservoir hydrographs in the 2006 flood season.

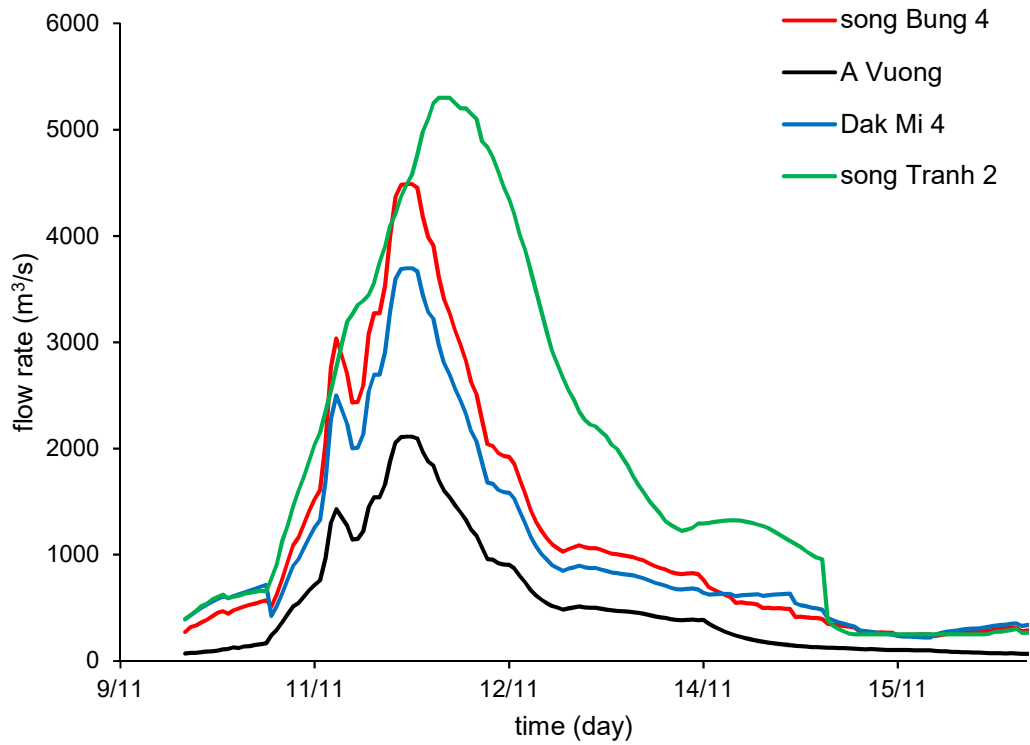
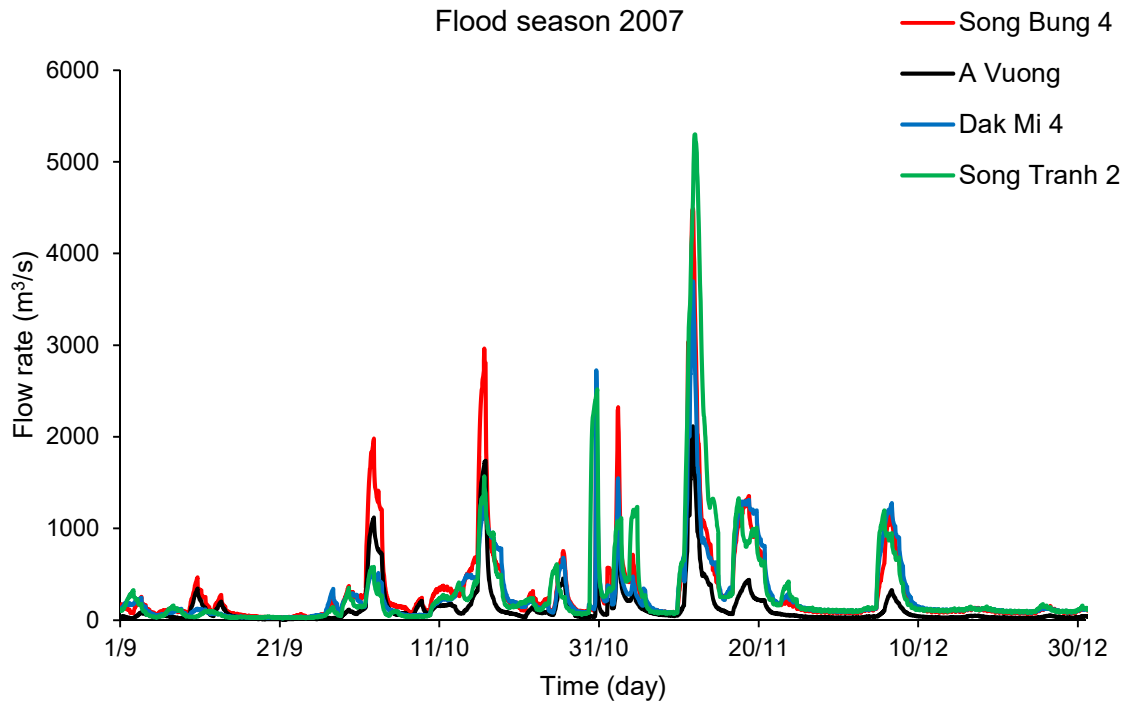


Figure A13. Inflow reservoir hydrographs in the 2007 flood season.

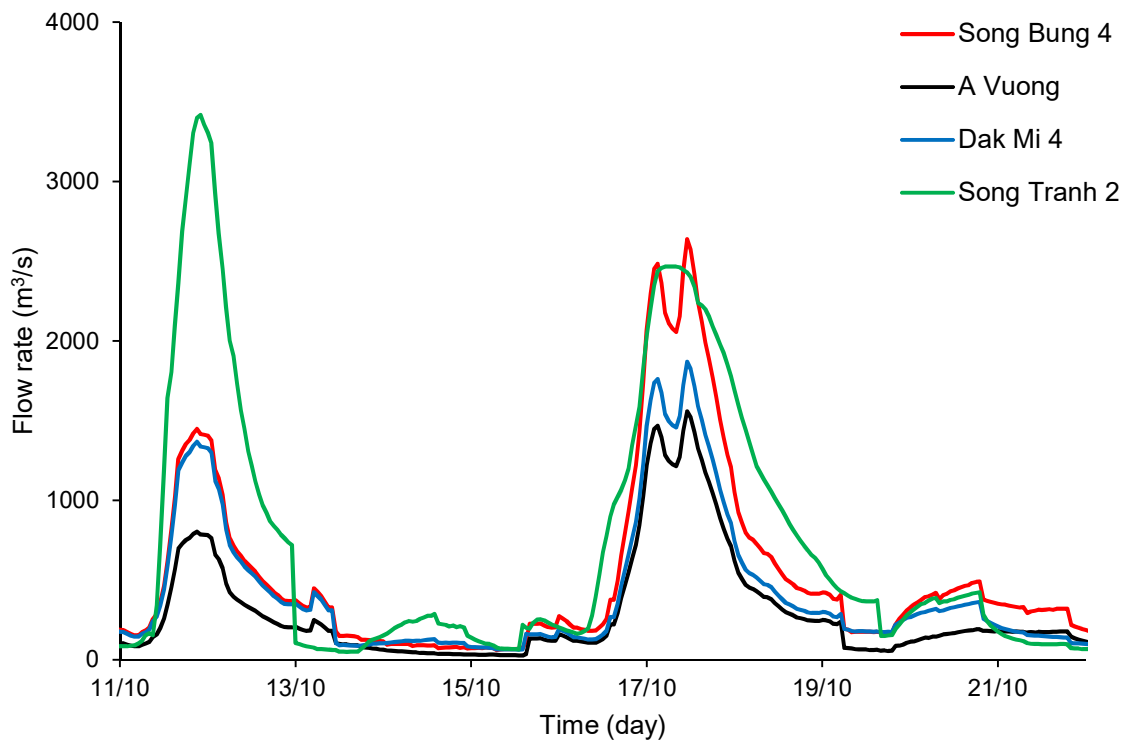
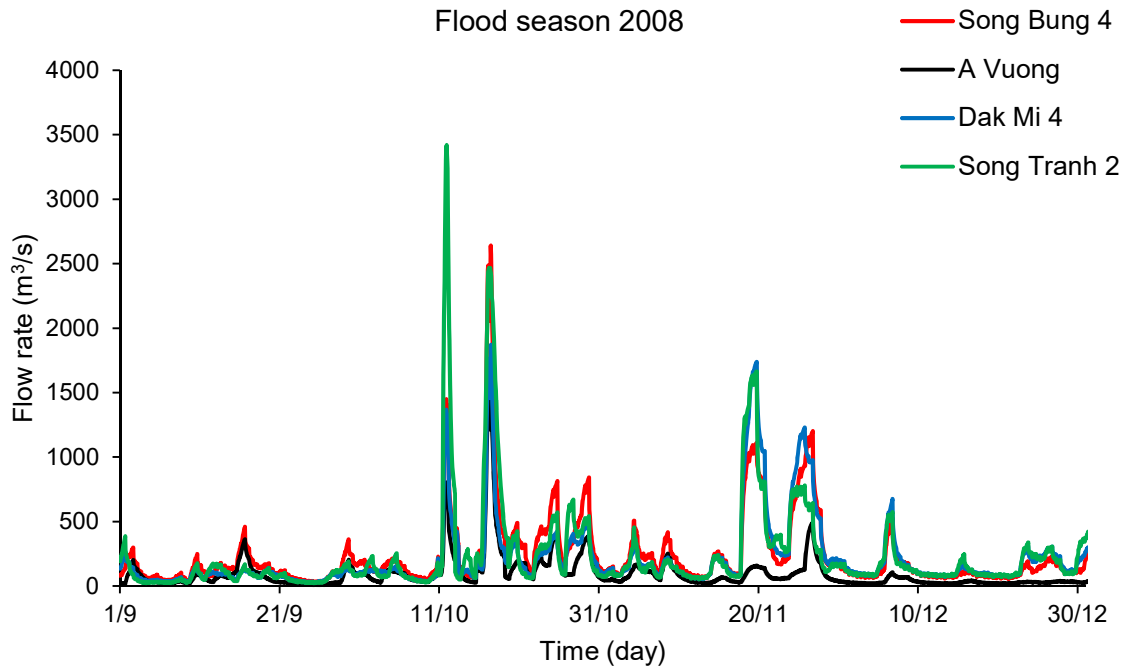


Figure A14. Inflow reservoir hydrographs in the 2008 flood season.

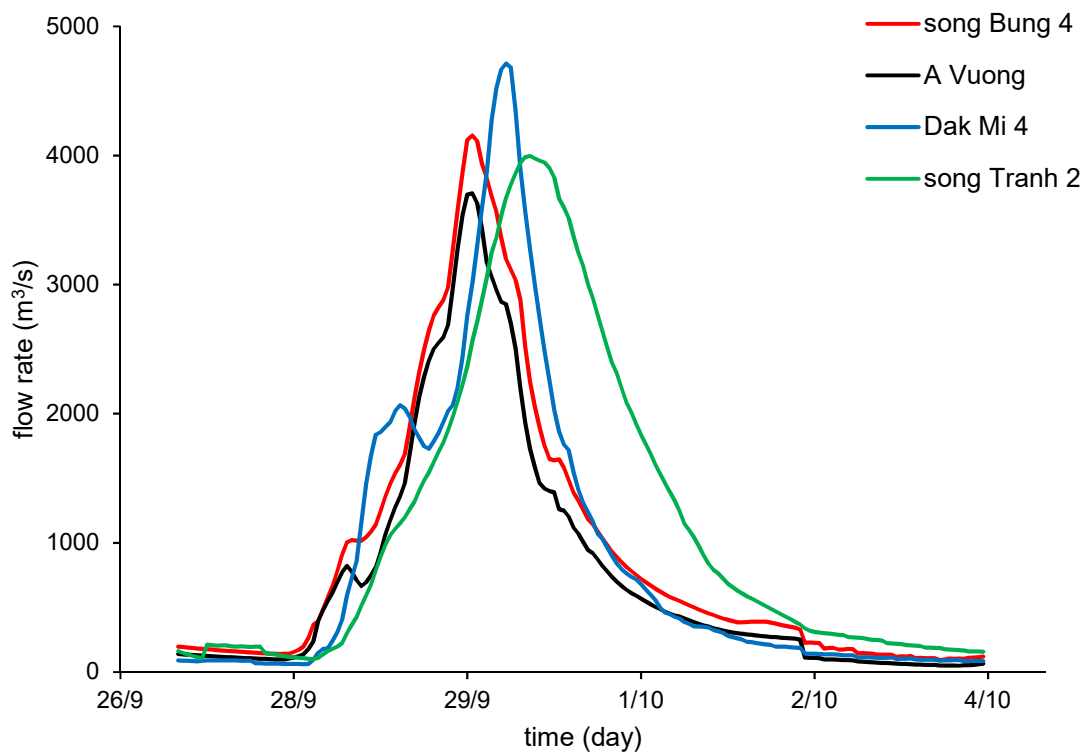
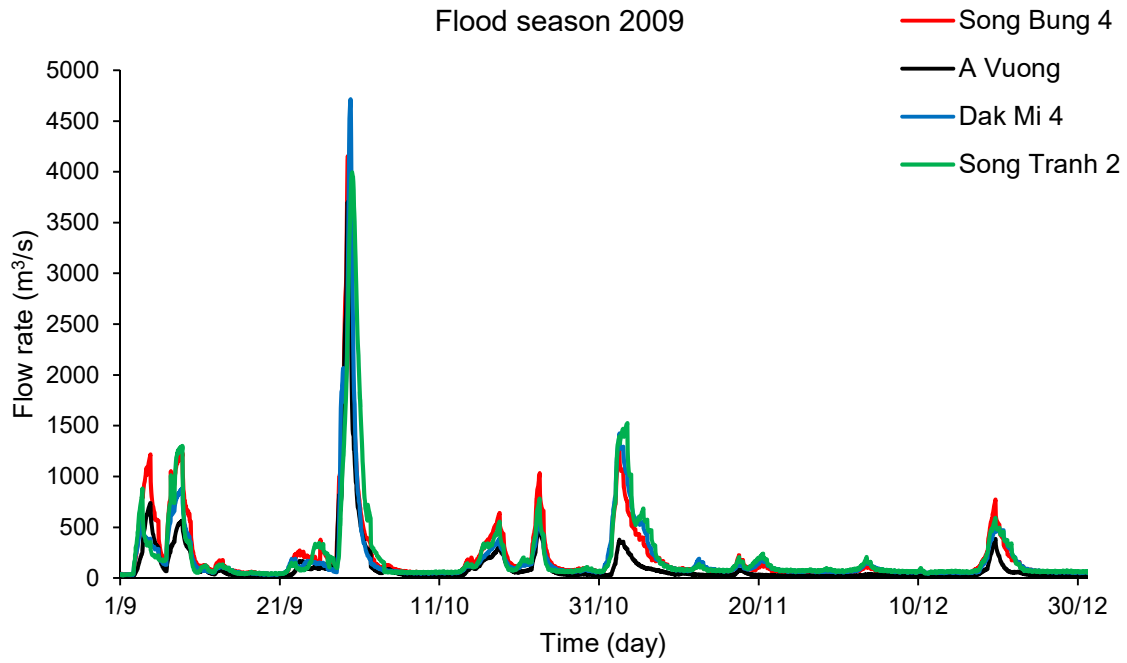


Figure A15. Inflow reservoir hydrographs in the 2009 flood season.