

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**PRODUCTION AND ASSESSMENT OF HAZARD
MAPS FOR DAM-BREAK CONNECTED FLOODS
IN THE CASE OF EMBANKMENT DAMS
THROUGH GIS AND MATHEMATICAL
MODELS**

by
Mohamed NAJAR

June, 2021
İZMİR

**PRODUCTION AND ASSESSMENT OF HAZARD
MAPS FOR DAM-BREAK CONNECTED FLOODS
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THROUGH GIS AND MATHEMATICAL
MODELS**

**A Thesis Submitted to the
Graduate School of Natural and Applied Sciences of Dokuz Eylül University
In Partial Fulfillment of the Requirements for the Degree of Doctor of
Philosophy in Geographical Information Systems**

**by
Mohamed NAJAR**

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Ph.D. THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “**PRODUCTION AND ASSESSMENT OF HAZARD MAPS FOR DAM-BREAK CONNECTED FLOODS IN THE CASE OF EMBANKMENT DAMS THROUGH GIS AND MATHEMATICAL MODELS**” completed by **MOHAMED NAJAR** under supervision of **ASSOC. PROF. DR. ALİ GÜL** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Doctor of Philosophy.

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PRODUCTION AND ASSESSMENT OF HAZARD MAPS FOR DAM-BREAK CONNECTED FLOODS IN THE CASE OF EMBANKMENT DAMS THROUGH GIS AND MATHEMATICAL MODELS

ABSTRACT

Dam hazard analysis is a key component of dam failure mitigation and disaster post-management. Many dams have either no hydro-meteorological data available or very little associated only with the hydraulic structure's planning phase. These data inadequacy causes major difficulties in performing hazard analyses. In the presented study, a combined hydrologic and hydraulic modeling approach is developed in a novel analytical framework for evaluating dam break issues in cases of ungauged basins. The study is aimed to provide an operational framework that allows engineers and specialists to understand, simulate and then manage the phenomenon of dam failure. The framework consolidates the application of synthetic unit hydrograph concept, hypothetical hyetograph estimation, distributed hydrological modeling, and hydraulic dam break simulation approaches. The study benefits from a dynamically downscaled climate model of 1km resolution to perform extreme precipitation frequency analysis, resulting in grid-based Intensity-Duration-Frequency (IDF) curve featured in spatially-distributed design hyetograph generation. The framework also adopted the climate change effects to decrease the uncertainties in both the design and risk assessment phases. An application of the two-dimensional hydrodynamic model is utilized to model the flood wave evolution resulting from varying dam-break events. In this respect, an alternative scenario-based approach is adopted in order to quantify differences in flood depth, velocity and arrival time of flood waves under various dam-break conditions. The simulation shows that both downstream regions are subject to significant potential risks against dam-break with the figure of more than 80% of the inundation area classified as a high-risk zone.

Keywords: Dam break, ungauged basins, instantaneous intensity method, geomorphological instantaneous unit hydrograph, hydrologic/hydraulic modelling, climate change, hazard mapping

DOLGU BARAJLAR ÖRNEĞİNDE BARAJ YIKILMASINA BAĞLI TAŞKIN TEHLİKE HARİTALARININ CBS VE MATEMATİK MODELLER ÜZERİNDEN OLUŞTURULMASI VE DEĞERLENDİRİLMESİ

ÖZ

Baraj tehlike analizleri, baraj yıkılması kaynaklı etkilerin azaltılması ve olası felaketlerde afet yönetiminin temelini oluşturması bakımından temel bir güvenlik bileşeni olmaktadır. Ancak birçok baraj örneğinde, bu türden analizleri mümkün kılacak hidro-meteorolojik veriler çoğunlukla hiç mevcut olmamakta veya sadece barajın planlama aşamasına yönelik toplanan verilerden ibaret şekilde kısıtlı süreli olmaktadır. Bu veri yetersizlikleri, baraj tehdit analizlerinin önündeki temel güçlük olmaktadır. Sunulan çalışmada, hidrometrik ölçümü bulunmayan/yetersiz havzalarda baraj yıkılması risklerinin değerlendirilmesi amacıyla bütünleşik hidrolojik ve hidrolik modelleme yaklaşımlarına dayalı bilimsel yenilikçi bir çerçeve ortaya koyulmaktadır. Çalışma, konuyla ilgili mühendisler ve uzmanlar tarafından özellikle ölçümü yetersiz havzalarda baraj yıkılması olgularının irdelenmesi, modellenmesi ve gerektiğinde kontrol edilmesini sağlayacak bir fonksiyonel yaklaşım çerçevesi oluşturmayı amaçlamaktadır. Bu yöntem bütünü; birim hidrograf kavramı, kurgusal hiyetograf eldesi, yayılı hidrolojik modelleme ve baraj yıkılması hidrolik hesap simülasyonlarını birarada değerlendirmektedir. Çalışma ayrıca, mekansal yayılı bir tasarım hiyetografi oluşturma amaçlı yine yayılı (grid tabanlı) Yağış Şiddeti-Süre-Frekans ilişkilerini ortaya çıkaran ve ekstrem yağışlara yönelik frekans analizleri için dinamik olarak ölçek indirgenmiş 1 km çözünürlüklü bir iklim modelini esas almaktadır. Yöntemsel yaklaşımda aynı zamanda hem tasarım hem de risk değerlendirme aşamalarındaki belirsizlikleri azaltmak için iklim değişikliği etkilerinin dikkate alınması prensibi benimsenmiştir. İki boyutlu hidrodinamik modelin bir uygulaması, farklı baraj yıkılması durumlarından kaynaklanan taşkın dalgası gelişiminin modellenmesi amacıyla ortaya koyulmaktadır. Yıkılma senaryoları simülasyonları, her iki barajın mansap bölgeleri genelinde %80 gibi bir oranı aşan alanların yüksek riskli bölge olarak sınıflandırılan su baskını alanlarına dönüşebileceği bulgusuyla, bu alanların baraj yıkılmalarına karşı önemli potansiyel risklere maruz kaldığını göstermektedir.

Anahtar Kelimeler: Baraj yıkılması, ölçümü olmayan havzalar, ani şiddet yöntemi, jeomorfolojik anlık birim hidrograf, hidrolojik/hidrolik modelleme, iklim değişikliği, taşkın tehlike haritalaması



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CHAPTER 1

INTRODUCTION

1.1 Motivation, Problem Statement and Study Objectives

Dams are hydraulic structures that regulate rivers' flows and are as old as human civilization, Sadd el-Kafara, which means in Arabic "Dam of the Pagans," the first reported constructed large dam built around 2700-2600 BC being around 11 m tall, 105 m wide at the crest, and 81 m at the bottom (Chen, 2015; Everard, 2014) . The main motivation of constructing dams was, and still is, to collect and control the water stored in their reservoirs for several purposes (e.g., hydropower generation, water supply, irrigation, etc.) as well as for protection against flooding. During humankind's history, hundreds of thousands of dams of different types and dikes have been built globally (Zagonjoli, 2007). Dams have been designed with extreme caution to prevent any failures. In recent decades, the safety level has been elevated even further by closely monitoring dams to detect irregularity in dam infrastructure and the surrounding environment (Peter, 2017). In this respect, experts' extensive and cumulated experiences in dam safety yields to the development of several instructions describing the state of art guidelines of constructing and successfully monitoring dam safeties and treating any arisen issues (FEMA, 2015). However, the history of water structures coexists with the history of their failure resulting in a catastrophic consequence. In 1975, one of the most tragedies associated with the collapse of the dams occurred. An entire city was removed due to the failure of the Banqiao Dam in Henan Province - china, killing an estimated 171,000 people (Lumbroso et al., 2021; Zhong et al., 2018). This catastrophic accident is equivalent to the Chernobyl tragedy (McCully, 1996). Most recently, at the beginning of 2019, the Córrego do Feijão dam break at an iron ore mining complex in Brumadinho Municipality in Brazil killed at least 250 people and release over 12M cubic meter of water into Paraopeba River and the surrounding area (Thompson et al., 2020).

Anticipating what could happen if a dam fails is essential to take precautionary measures such as installing emergency warning systems and designing evacuation

plans. Dam break analysis (wildly known as Dam Breach) is a term used to describe researches that deliver information to this end. Generally, Dam break analysis consist of three steps i) dam-break modeling, ii) the flood wave propagation, and iii) hazard/risk assessments. The dam-break modeling component can answer the questions related to failure mechanisms, the possible cause of failure, and estimate both the volume of water that will be released and the time scale of this release. The flood wave propagation component unfolds information about routed water's propagation behavior downstream, including water depth, velocity, and the extent of the inundation. By knowing the flood inundation extent (compromised areas), the hazard/risk analysis component play a role in supporting the decision-making process by quantifying the impact of the flood on the society, evaluating the potential economic losses, and estimating the number of individuals at risk.

On the other hand, climate change-related factors (e.g., fluctuations in the frequency of heavy precipitation events or dramatic variation in daily temperature) pose new challenges while also increasing risks and uncertainties (Appenzeller et al., 2014; Field et al., 2012; Walsh et al., 2014). These risks and uncertainties must be adequately handled in a process that is continuous and updated (Fluixá-Sanmartín et al., 2018). To quantify the impact of climate change on the dam break events, there is a necessity to strongly understand and accurately simulate the hydrological process in the upstream part of dam basin which normally requires computational studies and simulation efforts that depend on continuous observations and data about catchment meteorology, hydrology, topography, land uses, etc. However, in many dam sites with comparably limited hydrologic operations and moderate economic priorities, there is either no hydro-meteorological data available or is very little associated only to the planning phase of the hydraulic structure. These data inadequacies bring in the major difficulties in performing failure analyses of the dam structures, due to the incapability of data-based simulations for catchment hydrology and despite the rather proficient estimates on climate change driven meteorological aspects. This thesis attempts to develop an operational framework that enables engineers and water specialist to investigate the dam break in ungauged/poorly gauged watersheds. To this end, two main research objectives were identified for this thesis:

- To understand, and accordingly simulate, the hydrological response in the dam upstream, which would eventually help estimate spatio-temporal aspects of flood inundation and hazard maps on the downstream area.
- To quantify the impact of climate change of dam break analysis and investigate whether the existing water detained structures adequately designed to this end.

With the intention to accomplish these research objectives, a combined hydrologic and hydraulic modeling approach is presented in a novel analytical framework for assessing dam break issues in the case of ungauged basins.

1.2 Approach

The objectives of this thesis are accomplished by employing the hydrological synthetic methods to enable studying and then extracting required information for ungauged catchments. The aforementioned framework combines design hyetograph generation, synthetic unit hydrograph-based flood hydrograph estimation, distributed hydrologic modelling and scenario hydraulic simulations for dam break failure cases. The proposed approach is demonstrated in two sample basins with a notable difference in sizes, in order to conduct a comparative analysis and test the validated use of the generic framework. The approach also benefited from additional supplementary considerations including, for example, bias correction to adjust the metrological variables, geo-statistical frequency analysis for meteorological data, exploratory spatial and time-series data analysis. In addition, the framework employed an alternative scenario-based comparative approach (i) to cover the impact of both overtopping and piping modes of failure, (ii) to unfold varying flood behaviour with respect to the timing to initiate the breach as early-stage or delayed-stage (defined through the hydrograph times to reach the first inflection and peak point on the flood hydrograph), and (iii) to examine varying influences of the reservoir water availability at different levels on dam-break outflow as well as the corresponding flood-related components in considerably built-up downstream. Besides, the impact of climate

change on the flood hydrograph and consequently the associated hazard analysis in the downstream area would be investigated for the near future time period of (2025 – 2050) under RCP8.5 scenario.

1.3 Thesis Organization

This thesis is organized into five chapters. An introduction pointing out the problem statements, the study motivation, objectives and the followed approach to accomplish the stated objectives are presented in Chapter 1. In Chapter 2, a comprehensive literature review covering dam-break previous studies, the mechanism of embankment failure and the associated progressions involved during the embankment's breaching, the hydraulics concept of dam beak models, and demonstration of the available empirical models used for predicting breach geometry and parameters of embankments and water retention structures. Furthermore, the chapter covers the impacts of climate change on the dam break analysis. In Chapter 3, after general information about the study area, the proposed framework is illustrated and how the framework main-/sub-components are connected. The chapter also demonstrates the land use prediction process. In Chapter 4, the results obtained by applying the proposed methodology are discussed. The results of both climate change and land-use change are additionally presented. The chapter also covers the results of the hazard assessment for all studied scenarios. Finally, the conclusions of the research are addressed in the last chapter.

CHAPTER 2

LITERATURE REVIEW

In this Chapter, a review works of literature including recent scientific journals, books, and guidelines related to Dam-Break modeling, breach parameters estimation methods, climate change impacts on the safety of dam failures and the approaches used to study ungauged basins are comprehensively studied according to the recent various federal agencies that handling dam breach safety cases, also, based on scientific studies of other researchers in this study area.

2.1 Literature Review of Dam Break Modeling

In general, dam break analysis is carried out by taking into account the case study's specifications, the type and adequacy of available data, the study area's geographical and metrological features, and the case-specific assumptions. Both physical and numerical models are developed to study dam break phenomenon. One-, two-, combined one-two-, and three-dimensional models can be utilized to simulated the flood propagation resulting from dam break. However, the limitation related to computational time and costs considering the feasibility of hydraulic modeling may affect the selection of the proper model to use. Two-dimensional and combined one-two dimensional model are the most common used model because of the accuracy of their calculations, their ability to cover all the study area, and the ability to adjust the calculation resolution of as per the project need.

In the recent decade, several researchers extensively studied both the dam breaks and the flood-wave propagation in both one-dimensional and two-dimensional models. Li et al. (1991) examined six reservoirs - located north of Italy, to evaluate the flood damages to the downstream regions. The HCH/DIGHE model was used to measure the hydrograph generated from a dam-break flood event in a way combined with MIKE11 (1D) model to simulate the flood-wave in the downstream river bed. The study covered also, some issue connected to dam-breach, the impact of the dam-breach parameters, and calculation of the hydraulic resistance factors. Two early studies by

Bozkuş & Kasap (1998) and later by Bozkuş & Güner (2001) employed numerical models for simulating dam break flood events against an experimental physical dam break model in a comparative approach and concluded that there is a notable variation for the peak discharge occurrence time (T_p) between the physical and numerical results indicating that the time to peak characteristic is sensitive to the channel's bottom surface friction. Brufau et al. (2002) modelled the flood wave propagation in both one and two dimensions using the shallow water flow equations for unsteady flow. Their model attempted to override the issue that arises when flow develops over different dry bed slopes. Their system showed a satisfying performance in handling complex flow domains. Yanmaz & Beşer (2005) investigated the safety of gravity dams by employing a probabilistic evaluation approach. A probabilistic strategy through random loading and resistance aspects were used for the safety analysis. One of the earliest two-dimensional models in the field of dam-breaks performed by Vásquez & Leal (2006). A finite element method was employed to discretize the computational domain by using a triangular mesh. Both dry (zero) and wet (non-zero) initial water depths were adopted as downstream boundary conditions to run the model. The model successfully simulated both the hydraulic jump that was produced from a wet downstream condition and the surge that was moving over an initially dry bed. Alcrudo & Mulet (2007) studied the Tous Dam, which was experienced with a major dam-break event in 1982, to develop and validate a flood wave model. Due to the absence of a validated topography model of the area prior to dam-break occurring time, they run the flood propagation model on two different topography estimates to understand the uncertainties associated with the terrain variety. Palumbo et al., (2008) simulated the dam-break flow by taking into consideration the turbulent stresses that may appear from the re-circulating flows in a limited extent. Macchione (2008) developed a model to present the main aspects influencing both the formation of the hydrograph peak discharge and the breach development. He described the breach occurrence as a function of the shear stress generated by the flow. The geometry of the embankment, the shape of the breach and the planimetric shape of the reservoir had been taken into consideration. The result after the model calibration indicated a high accuracy of the physical layout for both overtopping and piping failure modes. In the following study by Macchione and Rino (2008), sensitivity analysis was carried out in order to quantify

the influence of the side slope parameter on the outputs. The sensitivity analysis showed that dam height, reservoir volume, and water mass in the reservoir are significant factors that influence the flood hydrograph. In a different effort that expands assessments through statistical analyses toward the estimation of expected breach parameters relationships, Froehlich (2008) used the data compiled from 74 embankment dam failure cases and developed a set of empirical models for breach cases that form in the shape of a trapezoid. From the study, which also employed Monte Carlo simulation techniques to estimate the uncertainty degree of the predicted peak flows and water depths in the downstream, an evidence that breach geometry has a non-vertical trapezoidal shape was concluded. Yochum et al. (2008), on the other hand, modelled an actual dam break event using the HEC-RAS model. Because the study adapted the actual breach geometry parameters, the obtained results have a good agreement with the depths collected as post-flood dataset. Ying et al. (2009) developed a dam-break model based on the finite volume method using an unstructured triangular mesh. To simplify the computation and reduce the numerical imbalance between source and flux terms, the model considered the effects of pressure and gravity in the shallow water equations. The developed model showed capacity to simulate dam-break connected flows which may take place over complicated terrain with different flow type (e.g., subcritical flows, supercritical flows, or trans-critical flows). Singh et al. (2011) developed a two-dimensional numerical model for simulation of dam-break flow propagation. Experiment with a frictionless horizontal bottom was established to validate the 2D model. As a positive agreement between the simulation and the experimental results was observed, the developed model was found suitable for simulating dam-break flows. Marco (2011) studied the failure of Gleno Dam - located in the Central Italian Alps, caused by structural deficiencies. The study attempts to set-up a dam-break model valid for mountainous areas. Integration of the one-dimensional modelling of dam wave propagation with a first-order finite volume numerical scheme was used to present the main result of the study. Bozkuş & Bağ (2011) simulated a fictitious dam break that takes place under a set of pre-defined conditions. Their analysis focused on the post-failure consequences and attempted to estimate the inundation depth with respect to time in the downstream valley. FLDWAV software was used to forecast the flood characteristic. Based on the analysis outputs, a set of

recommendations were suggested to the local administrators in charge of public safety. Honghai & Altinakar (2012) integrated the GIS and remote sensing technologies to develop a decision system that basically support the managements of for dam-break flood formed on two-dimensional flood simulations. They used HEC-RAS & HEC-FDA to validate their system. The results indicate that the decision support system provides a reliable setting for estimating different flood damage. Mahdizadeh et al. (2012) followed up on their previously introduced one-dimensional (1D) shallow-water model for simulating free-surface interaction with the flows issuing vertically through finite gaps. They enhanced the model by using a modified wave propagation algorithm (e.g., extends the shallow-water scheme to two dimensions) to understand the interaction between the free-surface flow and large underground pipe networks used for sewage and storm drainage that can be caused by flood propagation over urban areas. Navier-Stokes equations were employed to validate the algorithm. Bosa & Petti (2013) applied a two-dimensional numerical model (e.g., 2DH model) to understand the impacts of the overtopping flood wave in the Piave Valley - A region in Italy witnessed a catastrophe dam-break in 1900, about 1700 individual passed away in the valley region alone. They verified if the simplifications assumed by the two-dimensional model properly simulated the growth of the wave. Tsakiris & Spiliotis (2013) employed a semi-analytical approach to simulate the breach formation as well as to estimate the outflow hydrograph resulting from a hypothetical overtopping type dam break event. The study accepted the assumptions of a constant vertical erosion rate for the breach generation and a parabolic shape for the breach. The study presented two sets of solution based on the reservoir capacity. Moramarco et al. (2014) aimed to exemplify the collected reservoir levels data of the studied dam break event (i.e., partial sudden collapse of the spillway of the Montedoglio dam in December 2010), this achieved by using a discharge hydrograph gaged at several downstream river sites. The work employed a one-dimensional model to simulate the breach development and the flood wave propagation in the downstream valley. In this study, the breach development time was obtained by using an optimization method. In a recent study by Issakhov & Zhandaulet (2020) the dam break flow was simulated using a method developed based on the Navier-stokes equation. Both Newtonian and non-Newtonian fluid models were used to model the free surface movement of water and mud impurity

movement, respectively. the governing equations were solved using the numerical Pressure-Implicit with Splitting of Operators algorithm. Miliani et al. (2021) employed the lattice Boltzmann method as a three-dimensional tool to model the water-free-surface behaviour. The study claims an impressive performance of the model against classical numerical methods. The study also provides guidelines for the correct modeling of free-surface flows by distinguishing the use of no-slip or free-slip boundary conditions. Urzică et al. (2021) used the 2D HEC-RAS model to develop a small-scale conceptual method. They used a very high spatial resolution topographical data set (LIDAR data of 0.5 m) to test the flood control capacity of a multi-reservoir system based on piping failure- mode mechanism. The results showed that the multi-reservoir system of the river plays a significant role in flood mitigation.

Several studies were conducted to best reflect the effect of densely populated area in the propagation of the dam break generated flood waves. For the models using coarse grid sizes, Chen et al. (2012a) employed two parameters, building coverage ratio (BCR) and conveyance reduction factor (CRF), to simulate flooding in an urban region by two-dimensional model. They found out that the model enhanced the accuracy of the modeling and slightly increased the computational efficiency. In addition, Chen et al. (2012b) utilized a multi-layer approach to model dam break flood plain by taking into considerations two additional factors, elevation and roughness, over those of their previous study. The multilayer approach notably increased the accuracy of the spatially distributed simulation specially nearby built-up areas and marginally facilitated the computation. Bellos & Tsakiris (2015) simulated a flood event in a built-up area using a fully dynamic numerical model (FLOW-R2D). They solved the 2D Shallow Water Equations using the Finite Difference Method and the McCormack numerical system. The buildings resistance such as the reflection boundary, the local elevation rise, and the local increase of the Manning roughness coefficient were employed to examine the performance of the model. the study concluded the best way to represent building is by employing the reflection boundary method in the Flow-R2D model. Elçi et al. (2017) investigated the impact of buildings and other obstacles defined through defining Area Reduction Factor (ARF) and Width Reduction Factor (WRF), on the propagation of the flood waves from dam break

events in the examples of two separate dams by combining one-dimensional HEC-RAS and two-dimensional FLO-2D models, and they assessed the associated impacts on flood velocity and depth by comparing three scenarios ranked with varying breach characteristics (Haltas, Elçi, et al., 2016). Ye et al. (2020) employed the mesh-free method to simulate dam-break wave propagation under different bed configurations. The study also investigated the combined effect of dry and wet bed for the dam-break wave.

Number of previous studies were carried out to simulate the flood-wave in the downstream area of a dam located in Izmir city. Güney et al. (2014) carried out an experimental model to simulate the flood-wave propagation by construction a distorted physical model of Ürkmez Dam with vertical and horizontal scales of 1/30 and 1/150, respectively. The model consists of a dam upstream reservoir, dam body, and topographical representation of downstream region. As the model represents the topography and the building in the downstream area, the effect of the agriculture existence on the flood wave was not covered in the study. Haltas et al. (2016) utilized the one-dimensional hydraulic routing HEC-RAS model to estimate the flood hydrograph generated from a partial failure of the dam. Then, a two-dimensional routing model FLOW-2D was employed to simulate the spreading of the dam-break flood after the flood wave exits in the valley. A very recent study - following the work of Güney et al., (2014)– was carried out by Oguzhan & Aksoy (2020). The study aimed to understand the vegetation effects on the flood wave propagation resulting from a dam-break. It was proven that the presence of vegetation makes a significant decrease in water depths as the flood wave propagates and considerably reduces flooding impact on the downstream settlements.

2.2 Dam Breach Characteristics

2.2.1 Breach development

To understand the breach development, the progress of the breach in time and space should be investigated (Vaskinn et al., 2004). Breach initiation time is the time that starting from the first observable leak (e.g., overtopping, piping, and so forth) and

ending when the so-called breach formation phase began (Wahl, 1998; Dhiman & Patra, 2018). Embankment dams share the mode of failure development, whereas the geometry of the breach is gradually developed in time, resulting in complex-regime flow and high amount of sediment transport (ASCE/EWRI, 2011). According to Morris et al. (2008), the dam breaching process consists of three phases 1) breach triggering, 2) breach formation, and 3) breach evolution. Within the breach triggering/initiation phase, the flow amount is relatively small; and in case the water leak kept under control, the dam might not collapse. The phase of breach formation initiates at the moment when the dam failure is about to start and continues until when the breach reaches its maximum shape. The size of the dam reservoir plays a significant role. For a small capacity reservoir, the hydrograph peak from an embankment breach may take place before the breach geometry cross-section forms its maximum shape, simply because of a rapid decline in reservoir water level during the breach progress. On the contrary, the peak outflow in the case of dams with a relatively large storage pool may occur when the breach fully develops. An initial assumption here is that a channel formed on the embankment body is often taken into consideration in all breach models. The initial formation of the channel determines the breach starting condition. This means, if the initial channel does not form, then the progress of the breach process will not happen. As the initial channel formation controlled by on several factors (e.g., flow characteristics, structure coverage, and improper/insufficient compaction), predicting the exact location of starting breach position is difficult. Two common trigger events may initiate the breaching process: i) overtopping and ii) piping failure modes.

2.2.1.1 Overtopping Failure Mode

Different factors can initiate the failure of the earth-fill dam, e.g., piping, overtopping, seepage, and foundation defect. One of the most basic collapse modes for embankments is overtopping. Basically, overtopping failure (Figure 2.1a) occurs when the inflows become higher than the designed inflow, malfunctioning reasons, lake of spillway operation systems, inadequate capacity of spillways, or as a consequence of landslides into the reservoir (Brunnera, 2014). In cases of inefficiency of the spillway,

which causes the water stored in the dam lake to rise to the limit that allows it to run over the crest of embankment for adequate time, which eventually leads to the collapse of the embankment body. Once an initial breach mechanism started and the high upstream storage water levels continue, the breaching will persist in developing, and any effort made to stop it will be useless (Zagonjoli, 2007). Overtopping failure mode may not lead to structural collapse, but still presents a significant flood hazard. Similarly, the rapid release of upstream stored water in order to drop the water level to safe limits could conclude to not commendable consequences for downstream areas (Pugh & Harris, 1982). Figure 2.1a illustrates the failure mechanism progress for the overtopping failure mode.

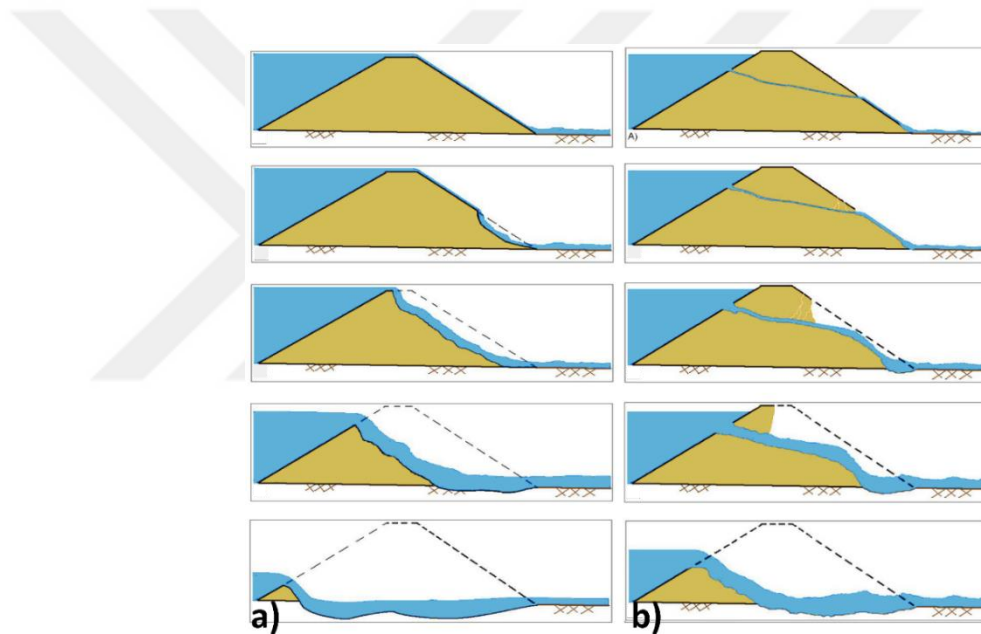


Figure 2.1 Dam breach development of a) overtopping, and b) piping mode of failure (following the (Brunner, 2014) representation)

2.2.1.2 Piping Failure Mode

The penetration of the water through the embankments body or its foundation may gradually weaken embankment structure or its foundation, leading ultimately to the embankment to collapse. To this end, the piping failure mode (Brunner, 2014) can be defined as a failure induced by water seeping through the embankment's body, bearing within its small dam material particles, continuously enlarging the channel

(Figure 2.1b). Observations of real embankment failures supported with laboratory experiments results reveal that piping (erosion) is dominant for the water structures with a non-cohesive material without a cohesive core (Zagonjoli, 2007). In case of the ability to detect the initial piping before it reaches severe conditions, a repair (restoration) might be successful. Here, in order to prevent the progress of piping enlargement adequately, proper instrumentation is required to compute the infiltration rate through the dam body.

Foundation failure, also known as seepage failure, likely take place in case of the foundation material saturated, resulting to 1) eroding of the foundation material itself or 2) sliding failure due to a weakening of the rock base. A graphical representation of how the water flows through a dam's pervious (penetrable) foundation is illustrated in Figure 2.2.

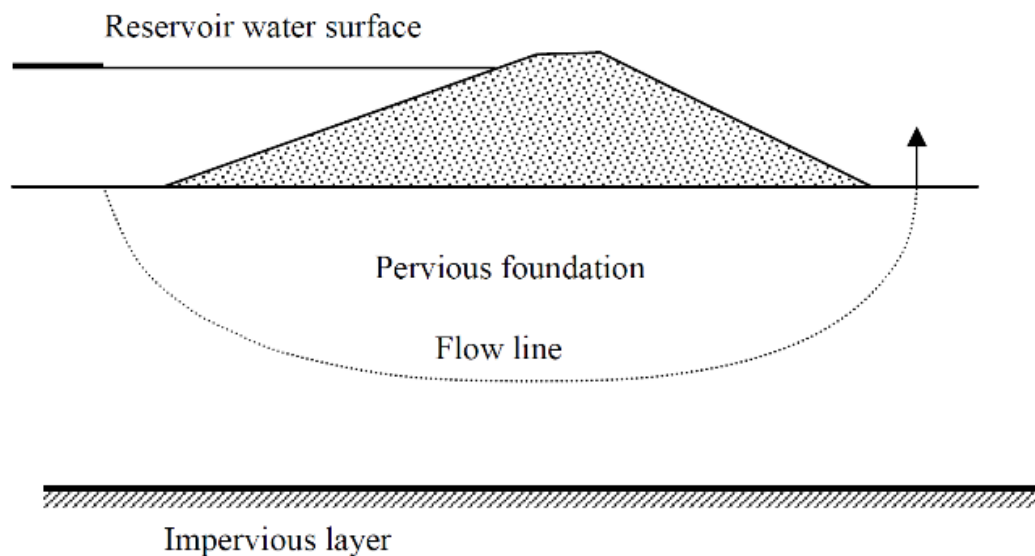


Figure 2.2 Foundation failure schematic diagram (following (Zagonjoli, 2007))

According to the United States Geological Survey (USGS) extensive report about the documented dam failure events, overtopping mode of failure has a rate of approximately 34% of occurrence, while piping and foundation defects failure modes share the rates of nearly 30% (Costa & Schuster, 1988).

2.3 Impact of Climate Change on Dam Break

Climate change can bring destructive consequences to the hydrological and ecological systems in several ways. e.g., Changes in the large-scale hydrological cycle, such as: i) increased atmospheric water vapor content, ii) changing precipitation both pattern and severity, and iii) soil moisture and runoff changes, all have been related to the observed warming over many decades (Bates et al., 2008). The increase in precipitation intensity and variability are forecasted to increase both the risks and uncertainties of flooding in many areas worldwide (Bates et al., 2008; Lenderink & Van Meijgaard, 2008; Wasko & Sharma, 2015; Westra et al., 2013; Winarta et al., 2019). In other words, the effect of climate change on dam safety is becoming more pronounced. Most dams, which were designed decades ago, may be unsuited to a warmer climate with much stronger precipitation events. Therefore, various climate change impacts must be evaluated by integrating collectively added effects on each operational factor, taking into account their interdependencies, rather than by simply adding up individual impacts. In this respect, several studies were conducted to quantify the impact of the climate change on dam break phenomenon. (Fluixá-Sanmartín et al., 2018) conducted a comprehensive and multidisciplinary review research to study climate change impacts that could affect dam safety. The study tabulated a summary of the main impacts recognized for each dam risk component (Table 2.1), while some recommendations were presented as well.

Table 2.1 A summary of the effects of climate change on the various dam safety components, as well as proposed techniques for evaluating them (Fluixá-Sanmartín et al., 2018)

Risk component	Climate change impacts	Assessment methods
Flood	Changes in regional floods are anticipated due to variations in: • extreme rainfall event patterns. • the processes of both snowmelt and snow cover. • The percentage of vegetation or soil moisture.	• Application of climate projections, downscaling techniques, and hydro-meteorological modelling. • Understanding the uncertainties caused by climate and hydrological projections and expected variations in the watershed model.
Reservoir water levels	Variations in water budget are due to: • rainfall variability, evapotranspiration, or decreased snow and ice storage. • Adaptations of land use and variation in water demand.	• Application of climate projections, downscaling methods, and water resource management systems. • Highlighting the non-climatic components such as land-use change and adjusting the exploitation rules of a reservoir.
Gate performance	• Variation of abrasion processes because of increases in the sediment content of the water. • Blockage of the gates as a result of suspended substance. • Fluctuations in temperature causing stresses and resulting deformations.	• Perform reliabilities assessments of the consequences of new climatic conditions and stressors. • Application of fault trees.
Flood routing strategy	Operation rules are likely to adapt under certain climate conditions (e.g., changes in heavy rainfalls inducing variations in the flood hydrographs' concentration time)	• Re-assessment of flood routing operation.
Failure modes	New failure mode may occur; for example, glacier phenomenon associated flood may directly impact the dam structure.	• Develop tools and guidelines to identify, describe, and simulated the may arise failure mode.

Table 2.1 continues

Risk component	Climate change impacts	Assessment methods
Probability of failure	• Temperature variations may provoke stresses in the dam body, especially for concrete dams. • Decrease water content in soil may increase the internal erosion (piping) process in embankment dams.	• Expert judgment can be employed to elicit probability in several guidelines.
Outflow hydrographs	The routing of outflow discharge is influenced by: • surface roughness • viscosity of water caused by sediment concentration	• Inundation flood models can be practiced to determine how sensitive the outflow hydrographs are to certain conditions.
Socio-economic consequences	Direct consequences might be evaluated from • Changes caused by population growth. • Upgrade and re-evaluate the assets' economic value. while indirect ends • The fluctuation in both irrigation water or hydropower generation value implies adjustments/increases in services cost	Direct ends might be estimated by: • application of demographic projections. • Performing population growth models, land-use changes, and flood severity through a socio-economic aspect Indirect consequences include: • Estimation expressed as a set proportion of direct expenses: • Complex economic system modeling and cost evaluation of provided services.

Fluixá-Sanmartín et al. (2019) provide a detailed study about the effects of climate change on the Spanish dam's safety under different hydrological scenarios. The study integrated differently projected effects that may play a role in each segment of the driven risks, from the input hydrology to the outflow hydrograph's consequences. Their study employed different projection models (i.e., RCP2.6, RCP4.5, and RCP8.5) to assess the risk evolution of the dam up to the year 2100. For the most of the scenarios considered, the findings suggest a gradual worsening of the dam failure risk. Winarta et al. (2019) utilized the HEC-RAS hydraulic platform to assesses the impact of climate change on a single flood component (i.e., depth of inundation) resulting from dam breaks during an extreme rainfall event. The analysis points out that the depth and flood level due to dam break creates a significant impact and risk to the downstream

city by inundating transportation networks and various public facilities. Mallakpour et al. (2019) assessed the climate change impacts on California's major dam by computing both historical and projection-based flood return periods as proxy for potential changes in the risk of hydrological failure of dams due to a warming climate. The conclusions confirm that the probability of hydrological failure is likely to increase for most California dams by the end of twenty-one century. Lin et al. (2021) studied the potential risk to various hydrological infrastructures over 50 years between the period of 1950-2099 for both the maximum rainfall and peak water stage associated to 100-year recurrence interval. The findings reveal that the predicted annual maximum precipitation is most likely to intensify under the RCP8.5 climate scenario, and the frequency of the annual precipitation maxima that exceed the threshold of 100 mm/day would increase in the future. Their results indicate that evaluating the risk of dam breaks due to extreme weather conditions or potential environmental concerns under climate change scenarios is crucial.

2.4 Hydrology of Dam Break Simulation in Ungauged/Poorly-Gauged Basins

Floods acting in urban areas are extremely difficult to manage because of the nature of urban flood generation processes themselves as well as the associated complex combination of climatic, topographic, hydrologic and hydraulic characteristics in these areas. This complex behavior normally requires computational studies and simulation efforts that depend on continuous observations and data about catchment meteorology, hydrology, topography, land uses, etc. However, in many dam sites with comparably limited hydrologic operations and moderate economic priorities, there is either no hydro-meteorological data available or is very little associated only to the planning phase of the hydraulic structure. These data inadequacies bring in the major difficulties in performing failure analyses of the dam structures, due to the incapability of data-based simulations for catchment hydrology and despite the rather proficient estimates on climate change driven meteorological aspects.

Recently, several attempts have been made to model flood inundation behavior in ungauged basins. Papaioannou et al. (2018) developed a framework for flood risk

evaluation particular for ungauged basins that mainly consist of urban areas to implement the EU Floods Directive in Greece. They provided validation of their finding against an actual flood event and concluded that the commonly used flood analysis approach is subject to major uncertainties in some flood-related parameters. Sarchani et al. (2020) studied flood inundation in ungauged basins by examining parameter uncertainties through employing a combined 1D/2D hydraulic modeling approach. Petroselli et al. (2019) studied flood mapping in small and ungauged basins by comparing outputs resulting from the combination of two hydrological and two hydraulic models. Akay & Baduna Koçyiğit (2020) prioritized the potential of flash flood occurrence in a sub-basin scale by employing traditional multi-criteria decision-making methods. The geomorphological instantaneous unit hydrograph method was adopted and coupled with the Monte Carlo sensitivity analysis to predict yield hydrographs for sub-basins. Bertini et al. (2020) highlighted the weaknesses and potentials of using satellite data to study ungauged basins from a hydrological aspect. The "CMORPH—Climate Prediction Center MORPHing technique satellite precipitation estimates" was employed to produce the intensity–duration–area–frequency curves and generate the design peak discharges for the ungauged catchment. The results indicate that CMORPH performed well in estimating both low-duration and short return period storm events, but further researches are required for large return period storms.

2.5 Hydraulics of Flow in case of Dam Break

The breach flow hydrograph plays a significant role in the evaluation of the flood wave characteristics in the downstream regions. At the initial formation phase of piping failure, the flow through the breach channel can be simulated using orifice and weir equations or by the one-dimensional Navier-Stokes equation referred to as Saint–Venant equations.

The weir equation (Eq.(2.1)) that used to compute the discharge for the free flow condition is expressed as:

$$Q = CLH^{3/2} \quad (2.1)$$

where, Q ($\text{m}^3 \cdot \text{s}^{-1}$) is the discharge, C ($\text{m}^{1/2} \cdot \text{s}^{-1}$) is weir coefficient, L (m) is the weir crest length, and H (m) is energy head over the weir crest. In extreme cases, if tailwater arises up to the embankment crest, Eq. (2.1) becomes Eq. (2.2),

$$Q = C_s LH^{3/2} \quad (2.2)$$

where C_s is a coefficient for the submergence effect.

Attention should be given to the difference between the weir coefficient and the discharge coefficient. These two might be used interchangeably; however, they are different. The weir coefficient is an aggregate parameter that includes the discharge coefficient, the gravitational constant, and constants based on geometric properties, Eq. (2.3) shows the relationship between weir coefficient and the discharge coefficient.

$$C = \frac{2}{3} C_d \sqrt{2g} \quad (2.3)$$

where C_d again is the dimensionless discharge coefficient (Powledge et al., 1989; Goodell, 2016).

2.6 Empirical Approaches to Dam-Break Analysis

A group of common experimental approaches for forecasting dam breach components (i.e., geometry and formation time) were applied to evaluate outflow flood hydrographs for a dam-break event. The approaches are MacDonald & Langridge-Monopolis (1984), Bureau of Reclamation (USBR, 1988), Von Thun & Gillette (1990), Froehlich (2008); Xu & Zhang (2009), and Singh & Snorrason (1984). These empirical methods were formulated through a statistical analysis of the failure records. Figure 2.3 defines the breach geometry parameters.

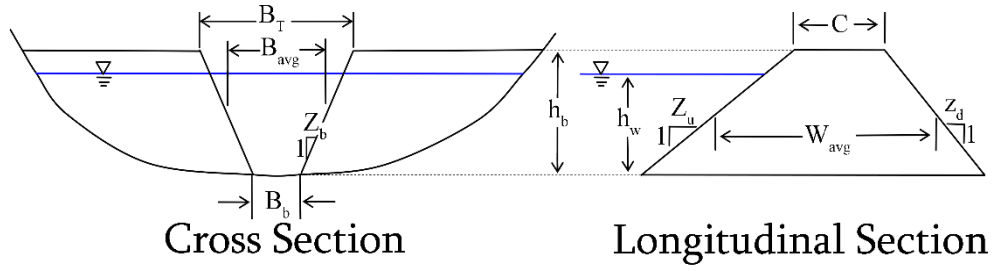


Figure 2.3 Dam breach variable definition sketch

where, B_b is the breach bottom width (meter), B_{avg} is the average breach width (meter), h_b is the breach height (meter), h_w is maximum depth of water stored behind the breach (meter), W_{avg} is the average width of dam in direction of flow (meter), C is the width of the dam crest (meter), Z_d and Z_u are the slopes of the downstream and upstream faces of the embankment, respectively, and Z_b is the side slope of breach.

2.6.1 Singh and Snorrason

One of the earlier researches to estimate the width of the dam breach was implemented by Singh & Snorrason (1984). Their empirical formula was calculated in the approach based on twenty documented dam failure cases. The breach width is a function of only one variable, dam height, ranging between twice and five times of the height variable (Eq.(2.4)). Also, according to the study, they expressed that the dam failure time (t_f) varied from a quarter-hour to one hour (Eq. (2.5)).

$$2h_d \leq B_{avg} \leq 5h_d \quad (2.4)$$

$$0.25 \leq t_f \leq 1.00 \quad (2.5)$$

2.6.2 MacDonald and Langridge-Monopolis

The study by MacDonald and Langridge-Monopolis (1984), on the other hand, provides two sets of equations which differentiate between the types of dams (e.g. earth-fill dam or other type dams). In their study, data set of fourty-two dam failure events were assessed to develop the regression equation that predicts the breach

factors. Eqs. (2.6) and (2.7) that describe the volume of eroded material (V_{er} , m³) are functions of the water volume passing through the breach cross section (V_{out} , m³) and depth of reservoir water (h_w , m) at the time of failure, for earth-fill dam type and other types, respectively. Based on the referred study, both equations were extracted from the regression analysis performed over the dam cases with the height ranging between 4.27 m and 92.96 m, and the available water volume between 0.0037 and 660.0 (10⁶ m³). The resulting time of failure (t_f , hr) is then computed following the Eq. (2.8) based on the calculated volume of eroded material.

$$V_{er} = 0.0261(V_{out} * h_w)^{0.769} \quad (\text{for earthfill dams}) \quad (2.6)$$

$$V_{er} = 0.00348(V_{out} * h_w)^{0.852} \quad (\text{for earth-fill dams with a clay core or rockfill dams}) \quad (2.7)$$

$$t_f = 0.0179(V_{er})^{0.364} \quad (2.8)$$

2.6.3 Bureau of Reclamation

U.S. Department of the Interior, Bureau of Reclamation (USBR, 1988) provided a third approach to obtain the width of rectangular breach with respect to depth of the water in the upstream reservoir (Eq. (2.9)). This formula can be used as a rule of thumb for choosing the average breach width that can be mostly used in classifying potential hazards. This approach suggests that the time for breach to develop is around 1% of the breach width (Eq. (2.10)).

$$B = 3 h_w \quad (2.9)$$

$$t_f = 0.011 B \quad (2.10)$$

2.6.4 Von Thun and Gillette

The fourth method put forward by Von Thun & Gillette (1990) used the data obtained from fifty-seven historically recorded dam failure cases that were previously studied by both Froehlich (1987) and MacDonald & Langridge-Monopolis (1984) to

estimate the relationships for predicting average breach width. Here, the computed width (Eq. (2.11)) is a function of reservoir water depth (h_w , m) and the coefficient (C_b , m), where the V_{out} (10^6 m^3) and height of dams (h_d , m) at breach time range between 0.027 and 660, and 3.66 and 92.96, respectively. Besides, Von Thun & Gillette (1990) stated that the majority of dams (89 %) have h_d less than 30 m and V_{out} less than 25 (10^6 m^3). The C_b values were directly related to the reservoir size and varied from 6.1 m for relatively small reservoirs to 54.9 m for large sized reservoirs.

$$B_{avg} = 2.5 h_w + C_b \quad (2.11)$$

In addition to the developed relationship, it is recommended in the approach to use 1H:1V breach slope side except for the dams with cohesive cores, where a ratio of 1H:2V or even 1H:3V look more appropriate for other cases. Regarding to the breach formation time, the approach provides two set of equations based on the dam fill material as it is shown in the Eqs. (2.12) to (2.15).

$$t_f = 0.02 h_w + 0.25 \quad (\text{erosion resistive material}) \quad (2.12)$$

$$t_f = 0.015 h_w \quad (\text{easily erodible}) \quad (2.13)$$

Breach development time with respect to h_w and B_{avg} :

$$t_f = \frac{B_{avg}}{4h_w} \quad (\text{erosion resistive material}) \quad (2.14)$$

$$t_f = \frac{B_{avg}}{4h_w + 61.0} \quad (\text{highly erodible}) \quad (2.15)$$

2.6.5 Froehlich

The next approach investigated in the current study was originally developed by Froehlich (2008) and exemplifies an improved study about dam breach phenomenon. The study up to some extent can be considered as a further enhancement of one of his previous studies (Froehlich, 1987), where an increased number of documented dam

breach cases were covered. The study states that in the case of overtopping type failure, the side slope ratio is 1H:1V, while for the piping or seepage failure the side slope ratio differs to 0.7H:1V.

Froehlich's relationship calculates the average width of the breach (B_{avg} , m) as it is shown in Eq. (2.16).

$$B_{avg} = 0.27 K_o V_w^{0.32} h_b^{0.04} \quad (2.16)$$

where, $K_o = 1.3$ for overtopping, and $K_o = 1.0$ for other failure mode. The equation below considers that the breach formation time is directly proportional to reservoir volume (V_w , m^3) and inversely proportional to the breach height (h_b , m) (Eq. (2.17)).

$$t_f = 63.2 \sqrt{\frac{V_w}{g h_b^2}} \quad (2.17)$$

2.6.6 Xu and Zhang

In the last approach published in a study by Xu & Zhang (2009), analyses were performed using a wide range of historically documented dam failure cases – 182 earth and rockfill dams from both U.S and China. However, due to data limitations their final formulas were developed using a relatively smaller subset (75 dams) of those dam break cases. The ranges of h_d and V_{out} used in the study are very similar to the ranges that were used by Von Thun and Gillette (1990) (i.e., $3.2 \text{ m} \leq h_d \leq 92 \text{ m}$; and $0.105 \times 10^6 \text{ m}^3 \leq V_{out} \leq 660 \times 10^6 \text{ m}^3$). However, in this study the majority of dams (around 80 %) have heights less than 30 m and V_{out} less than $25 \times 10^6 \text{ m}^3$. They also, developed an equation (Eq. (2.18)) to calculate the average breach width (B_{avg} , m).

$$\frac{B_{avg}}{h_b} = 0.787 \left(\frac{h_d}{h_r} \right)^{0.133} \left(\frac{V_w^{1/3}}{h_w} \right)^{0.652} e^{B_3} \quad (2.18)$$

Where, h_r is 15 meters, which is the reference height used to distinguish between small and large dams, B_3 is the coefficient that control dam properties and equal to the summation of b_3 , b_4 , and b_5 . B_3 is -0.226 for homogenous/zoned-fill, b_4 is equal to -0.389 and 0.149 for piping and overtopping, respectively; and b_5 controls the erodibility of the dam.

Unlike the previous methods, Xu & Zhang (2009) method does not estimate the side slopes of the breach. Alternatively, the method calculates the top width of the breach (B_T , m) through Eq. (2.19) that governs the relationship between the breach top width and the height of the final breach.

$$\frac{B_T}{h_b} = 1.062 \left(\frac{h_d}{h_r} \right)^{0.092} \left(\frac{V_w^{1/3}}{h_w} \right)^{0.508} e^{(b_3+b_4+b_5)} \quad (2.19)$$

Above, b_3 is -0.089 for homogenous/zoned-fill, b_4 is equal to -0.239 and 0.299 for piping and overtopping, respectively; and b_5 is -0.062 for medium erodible dam.

All above equations help estimate the breach geometry parameters (see section 3.9.1) suggested by the different approaches of earlier studies which then serve as input for hydraulic modelling.

CHAPTER 3

ANALYSIS FRAMEWORK AND METHODOLOGY

3.1 Case Study, Data and Software Resources/Tools

The study was implemented in two case-study basins, namely, Tahtalı Basin and Ürkmez Basin, which are located in the vicinity of the metropolitan city of Izmir in the western part of Turkey (Figure 3.1). The basins were selected for two main reasons arising from (1) the apparent disparity in basin areas (i.e., the size ratio of Tahtalı catchment to Ürkmez is 17.8:1), and (2) the fact that both basins are located in a touristic region with a dense residential population right on the dams downstream, where any dam failure would carry catastrophic impacts in both cases. Table 3.1 shows the main characteristics for Tahtalı and Ürkmez dams. Both dams supply water to the metropolitan city Izmir located at the Aegean transect where severe climate impacts have already begun to appear in a greater frequency. The city and the neighborhood have recently been more remembered with flash droughts alarming the reservoir levels which is then followed by flash flood damages requiring remedial actions against life/property losses in inundated areas as well as the immediately appearing full reservoir capacities with very short notice. A significant event, when this thesis was under final revision for submission, was recorded on the February 2nd, 2021, with the 109.6 mm of 4-hr accumulated rainfall observed at the Izmir station that corresponds to the recurrence interval of above 200 years (against the previously observed 4-hr maximum of 92.6 mm accumulated rainfall in the year 2006).

In the presented study, a Digital Elevation Model (DEM) with 10 m spatial resolution was generated using a 5-meter interval contour lines in combination with the point elevation data available from the map sheets in 1:5000 scale. The generated DEM was further processed to delineate the watershed boundaries and extract each sub-basin's geomorphological and physical characteristics. The Corine land cover (CLC 1990 & predicted 2035) datasets were used to depict the land cover and assign Manning roughness values for each land use category based on the study by Papaioannou et al (2018) (see APPENDIX-1). Following the Soil Conservation

Service (SCS, 1972) method -currently Natural Resources Conservation Service (NRCS)- and the instructions given in the Technical Release 55 (TR55), runoff curve numbers (CN) were determined by assigning hydrologic soil groups based on the land use definitions and the soil texture information identified from the NRCS soil texture triangle based on the soil data compiled from the SoilGridsTM database (Cronshey et al., 1985).

Table 3.1 Basic Dam Characteristics (DSI 2019)

Characteristic	Dam Name	
	Tahtalı	Ürkmez
Type of dam	Clay core + rock fill with cutting ditch	Zoned-fill
Height of dam (m)	54.5	32.0
Crest elevation (m)	62.5	48.0
Crest length (m)	475.0	428.0
Crest width (m)	10.0	8.0
Reservoir volume for minimum level (m ³)	19600.0×10 ³	375.0×10 ³
Reservoir volume for normal level (m ³)	306650.0×10 ³	7950.0×10 ³
Reservoir volume for active level (m ³)	287050.0×10 ³	7575.0×10 ³
Spillway Capacity (m ³ /s)	1825.0	347.00
Spillway Type	Controlled	Uncontrolled

As part of the climate dataset, meteorological data (precipitation, temperature, solar radiation, etc.) in a dynamically downscaled spatial resolution of 1 km and in the sub-daily temporal resolution of 3-hr were received from the regional climate model outputs, which were provided by the Center for Climate Change and Policy Studies of Boğaziçi University, and have been utilized to derive the annual maximum rainfall

data series for the estimation of the IDF relationships for different return periods through the geostatistical analysis techniques implemented in the study.



Figure 3.1 Case-study dam sites and catchments

3.2 Overview of Flood Modelling Approach

Within the integrated hydrologic modeling and dam failure simulation framework developed in the case of ungauged basins of the presented study, a distributed grid-based approach (in 1-km processing resolution) was adopted to improve both the hydrological model's accuracy and the design hyetographs estimation for ungauged watersheds. The suggested framework consists of four modeling pillars: i. generating synthetic design storm hyetographs with spatially varied/distributed characteristics, ii. estimating synthetic direct runoff hydrograph at the dam site (to aid in the

hydrologic calibration effort), iii. distributed hydrologic modeling at the level of sub-catchments differentiated in dam upstream, and iv. hydraulic modeling toward dam-break driven flood hazard identification. The framework required a pre-analysis bias correction to adjust the precipitation and temperature data. The framework outcomes are: (1) the flood hydrographs for the return periods of 100, 500, 1000 years and well as the Probable Maximum Flood (PMF), and (2) flood related components distributions (flood velocity depth, etc.), flood inundation maps and spatio-temporal flood hazard maps. This analytical framework also yielded secondary assessments regarding the functional capacities of the investigated dams taking into the consideration the climate change impacts, for example, by exploring whether the dams' spillway capacities are adequate in accommodating extreme flood cases especially under the overtopping scenario considerations. Eventually within the multipart framework, dam-break induced flood propagation is investigated for piping failure modes structured for different breach scenarios.

The flowchart of the framework approach is illustrated in Figure 3.2. In addition, The developed framework can be employed also to study and quantify the impact of climate change on the dam break analysis by using one of projections scenarios (e.g., RCP4.5 & RCP 8.5, etc.) as main input. The framework is explained in greater details with computational steps in the following section

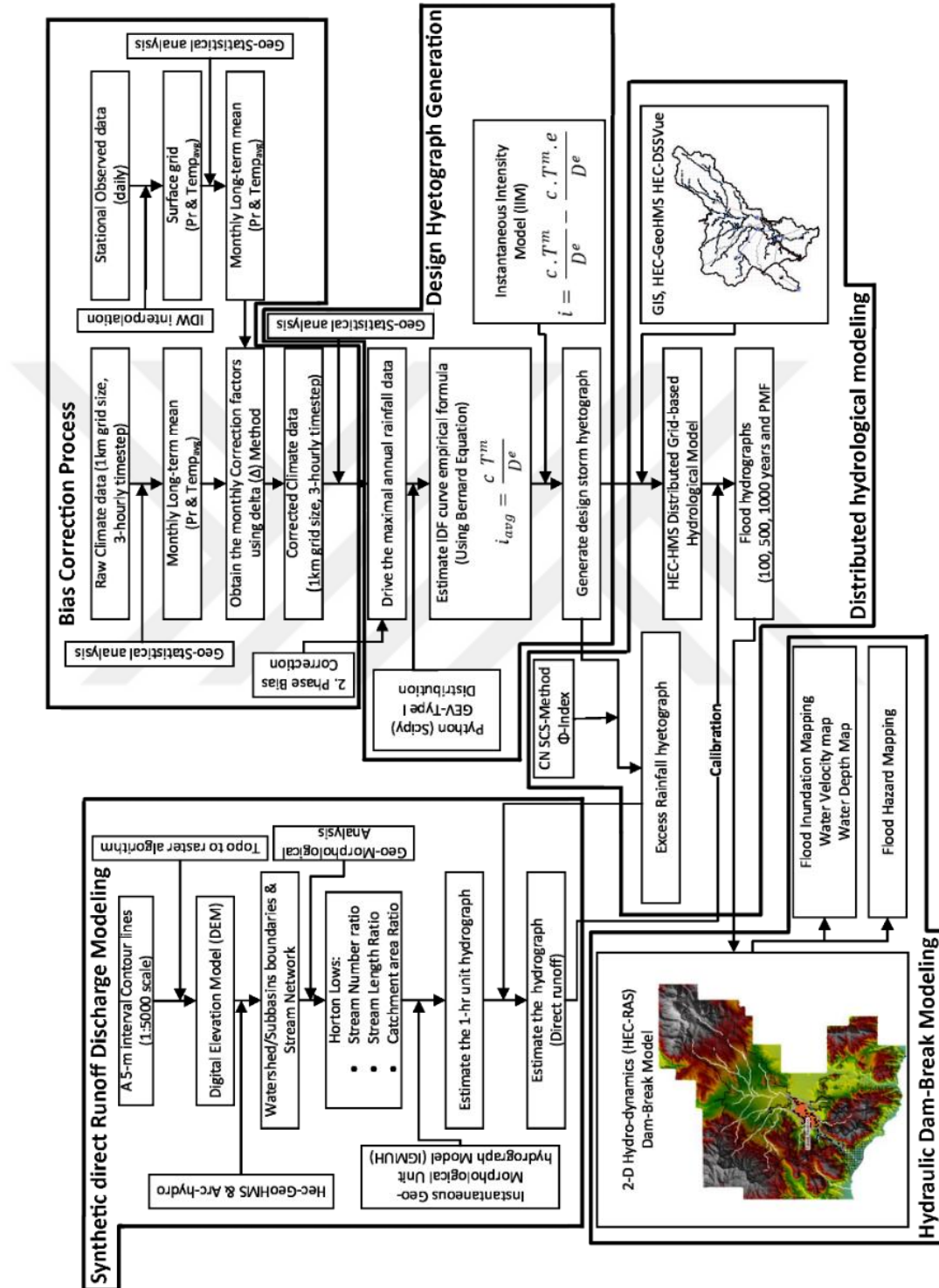


Figure 3.2 Flowchart of the proposed framework

3.3 Pre-analysis Bias Correction

A prior bias correction is essential for the climate data (e.g., precipitation and temperature) before employing it to derive meaningful and realistic outputs from the hydrological models (Feddersen & Andersen, 2005; Hansen et al., 2006; Piani et al., 2010; Sharma et al., 2007; Teutschbein & Seibert, 2010). The main objective is to identify any probable bias between observed and modelled climate variables in order to compute the so-called bias correction factor ($\Delta\mu$) to apply on the model data. Several bias correction techniques are used to correct the raw climate data. These include (1) linear scaling, (2) power transformation, (3) local intensity scaling, (4) delta change factor, (5) variance scaling, and (6) distribution transfer (Teutschbein & Seibert, 2012) approaches. A comparison between the advantages and disadvantages of various bias methods are summarized in Watanabe et al. (2012) and Teutschbein & Seibert (2012).

In the current study, a two-phase bias correction process is applied over the precipitation time series in a grid-based manner (again in 1-km processing resolution): (1) applying the correction for all timestamps in the time-series, and subsequently (2) applying a secondary correction only for the extreme values through comparisons with the maximum values of standard duration precipitation conditions that are computed from the frequency analyses over the recorded annual maxima. The delta-change bias correction method, in the form of bias addition/subtraction, was selected to maintain the variance estimate for both precipitation and temperature time series from the climate modelling. For precipitation data, the delta factor was applied for the rainy days only. $\Delta\mu$ is determined by computing the differences between the long-term observed monthly mean (μ_o) and the modeled (μ_m) (Eq. (3.1)). Similarly, the monthly long-term means for temperature data can be obtained based on the same rationale (Eq. (3.2)) and adapted to both the precipitation and temperature model outputs ($\chi_{m,i}$) by simple addition/subtraction (Eq. (3.3)).

$$\Delta\mu_{p,i} = \frac{\sum_1^n \sum_1^k p_{o,i} - \sum_1^n \sum_1^k p_{m,i}}{n} \quad (3.1)$$

where $\Delta\mu_{p,i}$ is the rainfall long-term mean for i^{th} month, $p_{o,i}$ is the observation i^{th} timestamp (within the i^{th} month), $p_{m,i}$ is the model i^{th} timestamp (within the i^{th} month), k is the number of timestamps within the i^{th} month, and n is the total number of the years;

$$\Delta\mu_{t,i} = \mu_{ot,i} - \mu_{mt,i} \quad (3.2)$$

where $\Delta\mu_{t,i}$ is the temperature long-term mean for i^{th} month, $\mu_{ot,i}$ & $\mu_{mt,i}$ are the temperature mean value of the observed and the modeled data for i^{th} month, respectively; and

$$\chi_{cor,i} = \chi_{m,i} + \Delta\mu_i \quad (3.3)$$

where $\chi_{cor,i}$ is the corrected model variable at the i^{th} timestamp.

A general underestimate in the modeled precipitation intensity associated with frequency overestimation was noticed when comparing the observed (i.e., Izmir station) and the modeled extreme values. This result agrees with the finding of previous studies. (Dai & Trenberth, 2004; Stephens et al., 2010; Sun et al., 2006). Therefore, a phase-two bias correction becomes necessary for the climate model maximal annual rainfall data to maintain each standard duration's mean intensity value for both modeled and observed data within the same range. Accordingly, a spatial cluster analysis is utilized to quantify geographic distribution patterns.

3.4 Synthetic Unit Hydrograph Generation

As the Unit Hydrograph (UH) concept requires rainfall-runoff observation at the basin outlet to generate a direct runoff hydrograph, synthetic UHs may provide a compromise solution in cases of unavailable simultaneous data or poor data quality problems. Here, the term "synthetic" indicates the UH is basically generated from watershed physical characteristics rather than the real rainfall-runoff data. Singh et al.

(2014) reviewed the synthetic UH models in cases of ungauged basins. They concluded that the geomorphological-based synthetic UHs could be considered the most useful and appealing prediction means in approaching to the hydrology conditions of ungauged basins.

One of the advantages of geomorphological models, commonly known as Geomorphological Instantaneous Unit Hydrograph (GIUH) models, is their capability of coupling hydrological systems' principles with quantitative geomorphology to estimate synthetic UH of a given basin. Due to the recent advancement in geospatial data analysis capacities for harvesting detailed information about the catchment's physical characteristics, the hydrology community greatly found it more convenient to couple the distribution function-based approaches with the classical GIUH approach for the development of synthetic UHs (Chen et al., 2019; Kumar, 2014; Singh et al., 2014).

Nash (1957) developed a mathematical model (Eq. (3.4)) based on a cascade of "n" equal-area linear reservoirs with equal storage coefficient "k" to derive the IUH for a basin. Within the model designed, the farthest (n^{th}) reservoir is placed where the instantaneous unit depth of effective rainfall is formulated. In Nash model construction, the closest reservoir to the outlet takes the 1st place in the reservoir series, while the farthest in the series becoming the n^{th} reservoir. Then, the linear reservoir series formulates the instantaneous unit depth of effective rainfall routed through the fluvial system. Each reservoir's discharge serves as the inflow into the next connected downstream reservoir in the series, as the flow moves toward the basin's main outlet. The mathematical form of Nash model is:

$$u(t) = \left[\frac{t}{k} \right]^{n-1} \frac{e^{-\frac{t}{k}}}{k \Gamma(n)} \quad (3.4)$$

where, $u(t)$ is the depth of runoff per unit time per unit effective rainfall (giving UH ordinates), index n is the shape parameter that reflects the number of conceptual linear reservoir series, and k is the scale parameter, which is the storage constant of

the reservoir that describes the average watershed flow time of concentration. It is noteworthy here that the parameter n is dimensionless, and k has the unit of time, and $\Gamma(n)$ is the standard gamma function. The area under the curve defined by the Nash model is unity. Thus, the excess-rainfall and direct surface runoff depths are equal to unity. The shape parameter n depends on Horton (1945) and Strahler's (1952) (H-S) definitions for the geomorphologic characteristics of area ratio (R_A), bifurcation ratio (R_B), and length ratio (R_L) of the basin, while the parameter k depends on both basin geomorphology and the flow velocity (Chavan and Srinivas, 2015 ; Chen et al., 2019).

3.4.1 Derivation of the Synthetic UH Based on The Nash Approach

The total shape of the GIUH can be achieved by matching both peak (q_p) and time-to-peak (t_p) equations (Eqs. (3.5) & (3.6)) derived by Rodríguez-Iturbe et al. (1979) in order to generate relationships to calculate n and k parameters of the Nash model. Kumar (2014) brought out the following procedure for obtaining the derived equations of k and n computations (Eqs. (3.15) & (3.16)).

$$q_p = 1.31 R_L^{0.43} \frac{v}{L_\Omega} \quad (3.5)$$

$$t_p = 0.44 \left(\frac{L_\Omega}{v} \right) \left(\frac{R_B}{R_L} \right)^{0.55} (R_L)^{-0.38} \quad (3.6)$$

R_B , R_L , R_A (Eqs. (3.7) to (3.9)) are the bifurcation, length and area ratios, respectively, L_Ω is the length (in kilometers) of the highest order stream, and v is a dynamic velocity parameter (m.s^{-1}).

$$R_B = \frac{N_i}{N_{i+1}} \quad (3.7)$$

$$R_L = \frac{\bar{L}_i}{\bar{L}_{i-1}} \quad (3.8)$$

$$R_A = \frac{\bar{A}_i}{\bar{A}_{i-1}} \quad (3.9)$$

where, N_w is the count of streams of the i^{th} order, $\bar{L}_w$ is the mean length of the i^{th} order stream, and $\bar{A}_w$ is the mean area of the subbasins with the i^{th} order.

A non-dimensional term can be produced by multiplying q_p & t_p . This term is not dependent on the v and thereby not dependent on the storm characteristics, and hence, it is a function (Eq. (3.10)) of only the geomorphological characteristics of the catchment.

$$q_p \cdot t_p = 0.5764 \left(\frac{R_B}{R_A} \right) \cdot (R_L)^{0.05} \quad (3.10)$$

t_p could be estimated by equating the first derivative (respect to t) of Eq. (3.6) to zero. Thus, taking the natural logarithm of both sides of Eq. (3.6), differentiating with respect to t , as Eq (3.11) shows:

$$\frac{\partial}{\partial t} \ln[u(t)] = \left[\frac{1}{k} + \frac{(n-1)}{t} \right] \quad (3.11)$$

By equating Eq. (3.11) to zero and simplify Eq. (3.12) show up:

$$t = t_p = k(n-1) \quad (3.12)$$

Substitution t_p in the original Nash model (Eq. (3.4)),

$$q_p = \frac{1}{k \Gamma(n)} e^{-(n-1)} (n-1)^{n-1} \quad (3.13)$$

By multiplying q_p & t_p (Eqs. (3.12 & (3.13):

$$q_p \cdot t_p = \frac{(n-1)}{\Gamma(n)} e^{-(n-1)} (n-1)^{n-1} \quad (3.14)$$

By equating Eq. (3.10) and Eq. (3.24)

$$\frac{(n-1)}{\Gamma(n)} e^{-(n-1)} (n-1)^{n-1} = 0.5764 \left(\frac{R_B}{R_A} \right)^{0.55} \cdot (R_L)^{0.05} \quad (3.15)$$

All the right-hand side's terms on the Eq. (3.24) are known with the only remaining unknown, Nash model shape parameter n , which can be calculated by solving one of the optimization methods (e.g., Newton Raphson method of non-linear optimization). The Nash model parameter k for the given v can be obtained by solving Eqs. (3.6 and (3.12, as Eq. (3.16) shows:

$$k = 0.44 \left(\frac{L_\Omega}{v} \right) \left(\frac{R_B}{R_L} \right)^{0.55} \cdot \frac{(R_L)^{-0.38}}{(n-1)} \quad (3.16)$$

On the other hand, (Croley, 1979) applied a numerical solution to formulate an empirical equation to calculate n (Eq. (3.17)) & k (Eq. (3.18)) against R_A , R_B , and R_L .

$$n = 3.29 \left(\frac{R_B}{R_A} \right)^{0.78} R_L^{0.07} \quad (3.17)$$

$$k = 0.70 \left[\frac{R_A}{R_B R_L} \right]^{0.48} \frac{L_\Omega}{v} \quad (3.18)$$

In this study's scope, both the derivative (Eqs. (3.15) & (3.16)) and numerical (Eqs. (3.17) & (3.18)) solutions are used to estimate the shape (n) and scale (k) indices for the ungauged basins.

Several previous research attempts were made to determine the numeric relationship between velocity and catchment geomorphology (e.g., Shen et al., 2016 ; Yao et al., 2014). Recently, Chen et al. (2019) estimated the optimal flow velocity with acceptable accuracy by analyzing 30 historical flood events. They developed a relationship (Eq. (3.19)) to obtain the optimal flow velocity (for use of the Nash model scale (k) and shape (n) parameters) against two physical parameters - derivable factors: area of watershed (A), and the highest-order river length (L_Ω).

$$v = EXP \left\{ 0.755 \left[\frac{A}{L_{\Omega}^2} \right]^{-0.139} \right\} \quad (3.19)$$

The obtained values of shape (n) and scale (k) parameters are used to generate the complete shape of the GIUH based Nash model using Eq. (3.4). Then, the UHs of certain time (UH_t) can be obtained by shifting the IUH by “t” and then averaging the two hydrographs (Onuşluoğlu et al., 2010). Subsequently, the direct runoff hydrograph can be generated by convoluting the excess-rainfall hyetograph for each return period with the UH_t obtained above by employing the lagging (discrete convolution) method (Chow et al., 1988; Usul, 2001).

3.5 Generating Distributed Design Hyetographs

A design hyetograph is the temporal representation of the rainfall pattern with a duration (D) and associated with a return period (R). R is usually determined by conducting statistical IDF analysis on the observed precipitation records. The storm characteristics, namely the maximum intensity, total precipitation depth and duration, can be estimated in a grid-based manner, as in the given study (Figure 3.3), through geo-statistical analyses followed by applying curve-fitting technique for assigning the best fit probability distribution. Empirical IDF relationships can then be used to develop the Instantaneous Intensity Hyetograph model parameters.

3.5.1 The Annual Maximum Precipitation Data

Extreme precipitation is usually defined with the annual maxima so that the number of the extreme values for each standard duration would be equal to the total number of evaluated years. This traditional process is known as the block-maxima method (Feng et al., 2007; Gumbel, 1958).

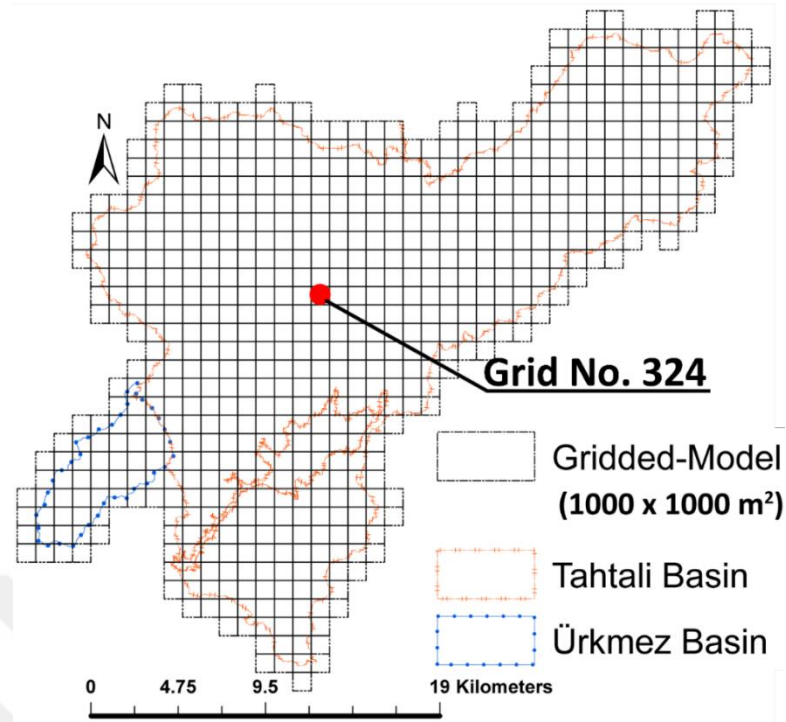


Figure 3.3 The distributed grid model of the study area showing the spatial resolution of the analysis

The annual maximum (AM) series data were aggregated from the dynamically downscaled climate data in the period 1970-2004 as the reference and the period 2025-2050 for the RCP8.5 scenario consideration. The data had been subjected to a quality validation and adjustment process (previously explained in the bias correction section) to assure the reliable use in further steps. The annual precipitation maxima were extracted from 3-, 6-, 12-, 18- and 24-hours by accounting moving sums over the rainy block periods at each grid location to construct the AM time series of standard durations (denoted in the study as AM3, AM6, AM12, AM18, and AM24, respectively). Relevant literature (e.g., (Singh, 1998)) generally suggests applying the General Extreme Value (GEV) distribution in extreme hydrological studies because of its commonly agreed suitability for frequency analysis of extreme values of meteorological or climatic and hydrologic variables, such as floods, rainfall, droughts, etc.. The extreme value theory, which presents the statistical structure to infer the probability of extreme events, has been used to develop methods explaining outliers' behavior. The GEV provides great descriptive and predictive abilities to capture the properties AM data (e.g., skewness and kurtosis) (Porto de Carvalho et al., 2014). Three kinds of distributions of extreme values may be represented in a single GEV

parametric form (Jenkinson, 1955). Eqs. (3.20) and (3.21) show the density and cumulative distributions formula for GEV, respectively.

$$f(AM) = \left(\frac{1}{\sigma}\right) e^{\left\{-[1+\zeta Z]^{-\left(\frac{1}{\zeta}\right)}\right\}} \cdot (1 + \zeta Z)^{\left(-1 - \frac{1}{\zeta}\right)} \quad (3.20)$$

$$F(AM) = e^{\left\{-[1+\zeta Z]^{-\left(\frac{1}{\zeta}\right)}\right\}} \quad (3.21)$$

Where, $Z = (AM - \mu)$. The ξ , μ , σ and are the parameters of form, position and scale, respectively. The form parameter ξ is used to define the type of the distribution. In the cases of $\xi < 0$ corresponds to the Weibull distribution, the $\xi > 0$ corresponds to the distribution of Fréchet, when $\xi = 0$ have the Gumbel distribution. The goodness of fit would be evaluated using Kolmogorov-Smirnov (K-S) and Anderson-Darling (A-D) goodness-of-fit tests to confirm the fitting of the distribution used.

3.5.2 The Instantaneous Intensity Hyetograph Model

The Instantaneous Intensity is a method to estimate design hyetograph when IDF equation is defined. The method considers a continuous precipitation intensity throughout the storm duration (Chow et al., 1988). It consists of a pair of curves that are characterized by using the instantaneous intensity functions (Figure 3.4), $f(ta)$ and $f(tb)$, representing the rainfall intensities at the peak instantaneous intensity. In this method, the value extracted from the IDF curve is equal rainfall depth for a given storm period 'D' where the storm event reaches its peak. The cumulative rainfall amount within the same period 'D' can be calculated as the area under the paired curves $f(ta)$ and $f(tb)$. Therefore, by knowing the IDF curve formula, a hyetograph of a pre-defined design storm period equation defining the variation in intensity with time can be derived. Meanwhile, with the aim of increasing the accuracy of generated hyetograph, there is a need to select the smallest (as possible) computation interval, Δt_i (Chow et al., 1988; Onuşluel Gül et al., 2010).

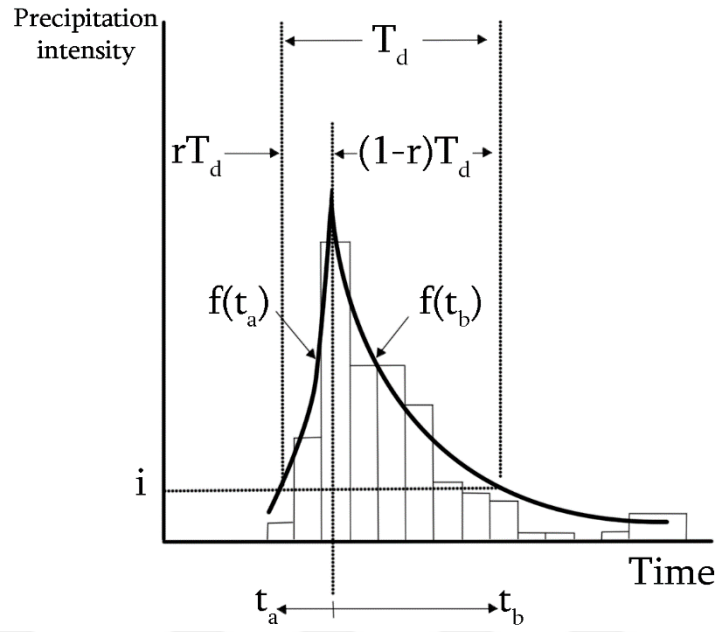


Figure 3.4 Instantaneous intensity curves (Chow et al., 1988)

In the presented study, Bernard (1932) equation (Eq. (3.22)) was used to develop the instantaneous intensity hyetograph (Eq. (3.23)) using the storm advancement coefficient (r) equal to 0.5 so that the peak intensity would occur in the middle of storm period:

$$i_{avg} = \frac{a \cdot R^b}{D^c} \quad (3.22)$$

Where, i_{avg} : the average rainfall intensity, D is the storm duration, R is the return period, and a , b , and c are the coefficients altering with location and return period.

$$i = \frac{a \cdot R^b}{D^c} - \frac{a \cdot c \cdot R^b}{D^c} = \frac{a \cdot R^b (1 - c)}{D^c} \quad (3.23)$$

Within the scope of the presented study, a storm duration of 24-hour that has a peak value occurring at the 12th hour from the beginning of the storm was selected. A smaller time interval was defined around the peak to increase the model's accuracy while 1-hour time intervals were considered for representing the hyetograph in general.

The design hyetographs were developed for 100-, 500-, and 1000-year return periods, as well as for Probable Maximum Precipitation (PMP) event in a gridded-based manner (ca. 700 grid points in total (see Figure 3.3). The excess rainfall was calculated from the hyetograph by subtracting hydrological deficits (e.g., infiltration, evapotranspiration, and canopy absorption). To this end, the SCS-CN approach was utilized where two parameters are essential, the maximum potential retention (S) and the initial abstraction (h_{a0}), in order to measure effective rainfall development. Here, the h_{a0} value equal to $0.2 S$ is accepted as the threshold for surface runoff generation. Thus, one final parameter needs to be estimated (i.e., maximum potential retention) for each grid point location. According to the SCS-CN approach, S is a function of Curve Number (CN) that can be generated based on the hydrologic soil group and land cover attributes combination, following the NRCS classification. An assumption of type II antecedent soil moisture conditions (AMC II) in the soil profile before the event started was accepted in the presented study to reflect average conditions.

3.6 Land Use Prediction

The Corine land use maps of 2006, 2012, and 2018 years were used in the land-use-land-change (LULC) prediction analysis. Land Change Modeler in TerrSet developed by Clark labs was used to predict the land use of 2035 year. As the model limit the number of classes to be modeled in each run to a maximum eight classes, an aggregation process was applied, considering the hydrological aspects of the targeted analysis. Table 3.2 shows the aggregated classes from Corine land cover definitions that apply to the study area.

Table 3.2 Matching CORINE land classes with aggregate land classes

CORINE Code	CORINE Land Use Definitions	Aggregate / New code	New code definitions
111	Continuous urban fabric	1	Built-up areas
112	Discontinuous urban fabric	1	Built-up areas
121	Industrial or commercial units	1	Built-up areas
133	Construction sites	1	Built-up areas
142	Sport and leisure facilities	1	Built-up areas

Table 3.2 continues

CORINE Code	CORINE Land Use Definitions	Aggregate / New code	New code definitions
122	Road and rail networks and associated land	2	Transportation
124	Airports	2	Transportation
212	Permanently irrigated land	3	Agriculture / AGRR_W
211	Non-irrigated arable land	4	Agriculture / AGRR_non_W
242	Complex cultivation patterns	4	Agriculture / AGRR_non_W
243	Land principally occupied by agriculture, with significant areas of natural vegetation	4	Agriculture / AGRR_non_W
221	Vineyards	5	Trees and fruits
222	Fruit trees and berry plantations	5	Trees and fruits
223	Olive groves	5	Trees and fruits
311	Broad-leaved forest	6	Forest
312	Coniferous forest	6	Forest
313	Mixed forest	6	Forest
231	Pastures	7	Pastures
321	Natural grassland	7	Pastures
323	Sclerophyllous vegetation	7	Pastures
324	Transitional woodland/shrub	7	Pastures
331	Beaches, dunes, sands	8	Bare Soil
332	Bare rock	8	Bare Soil
333	Sparsely vegetated areas	8	Bare Soil
334	Burnt areas	8	Bare Soil
131	Mineral extraction sites	8	Bare Soil
132	Dump sites	8	Bare Soil
411	Inland marshes	9	Water bodies
512	Water bodies	9	Water bodies
523	Sea and ocean	9	Water bodies

Since there is a need to examine irrigated agricultural and non-irrigated agriculture areas separately, the AGRR class is divided into AGRR_nonW (non-irrigated agriculture) and AGRR_W (irrigated agriculture). Accordingly, 211, 242, and 243 CORINE classes classified as AGRR_nonW, and 212 class is nominated as AGRR_W.

With the scope of the study, it was found appropriate to exclude the protected areas from the land use obtained from the CORINE database. It has been accepted that these areas will not be affected by future changes. In the production stage of GIS data for protected areas, the 1/100.000 scaled İzmir-Manisa Landscaping Plan (MEU, 2020) published by the Ministry of Environment and Urbanization was used (Figure 3.5). As can be seen from the visual, the protected areas are;

- Natural protected area,
- 2nd and 3rd degree archaeological site,
- Natural and archaeological site and
- Culture and tourism areas are in the form of a protection and development zone.

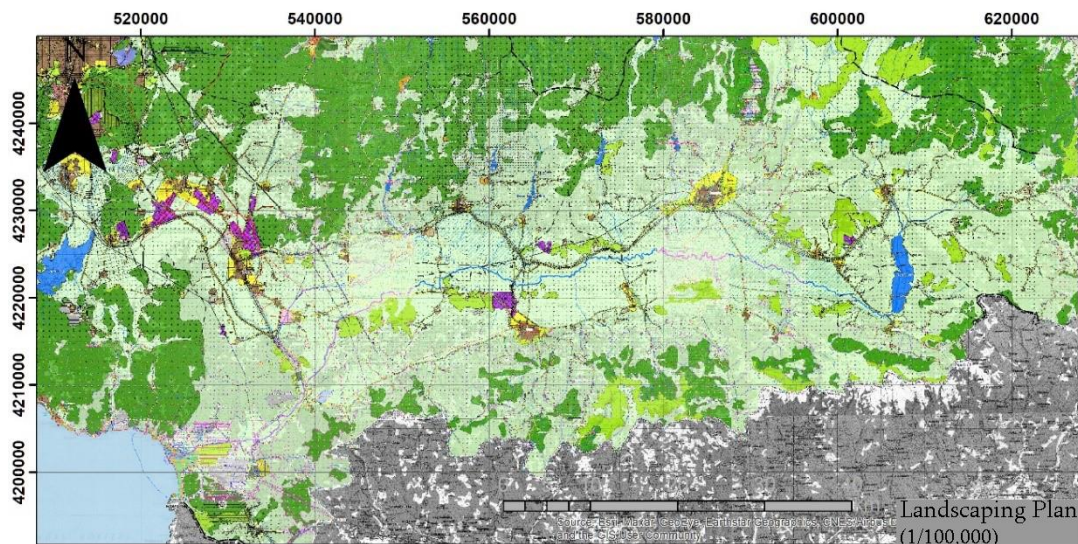


Figure 3.5 The layout containing the working area of the 1/100.000 scale Landscaping Plan obtained from the Ministry of Environment and Urbanization

In Figure 3.5, the map provided from the ministry's official website and the related areas of interest (protected areas) are georeferenced then digitized in a GIS-supported environment. The protected area, which represented as a vector data layer, was removed from the land use classes of 2006, 2012, and 2018. Land use vector layers were obtained by extracting protected area data then convert them into raster format

in the GIS-supported environment and reclassify them according to the aggregated codes in provides Table 3.2.

3.6.1 Creating the Land Use Change Model

In order to model the land-use change, a verification step was created based on the current (2012) and (2006) previous years. The validation step is an important sub-step for the years that the model will predict. The steps to be performed for land-use prediction models are given in order:

- Determining the land-use transition rates for the current (2012) and previous (2006) reference years (see APPENDIX-2).
- Identifying factors (driver variables) that control the land-use change.
- Creation of LULC sub-model (LULC-sub).
- Determining the weights of driver variables and creating transition potentials for each land-use class with LULC-sub. And then determination of accuracy rates of these transition potentials by Multi Layer Perception (MLP) method.
- Running LULC for the current third year (2018).
- Verification analysis.
- LULC's forecast for the coming years (2035).

As shown in APPENDIX-2, 65 land-use class transitions had been identified for 8 land use classes in the LULC environment. Within these land-use transitions, a maximum of 8 land-use transitions can be defined to LULC-sub. This is the maximum limit of the MLP tool allowed in TerrSet software and used in the sub-model.

3.6.2 Determining the Factors (Drive Variables)

At this stage, the factors that strengthen the land-use change or driver variables were determined, and the data belonging to these variables were prepared.

Land-use change is driven by different factors and their combinations, leading to pressure on using resources, changing opportunities created by markets, policy

intervention, loss of adaptation capacity, and changes in social organization and attitudes (Lambin et al., 2003). The drivers or driver variables of land-use change vary by the geographic scale and certainly do in terms of their relative role (Mustard et al., 2012). On a global scale, deforestation, land degradation, overgrazing, rapid population growth, urbanization and industrialization are associated, among other things, with land-use changes. Land-use changes also determine the vulnerability of places and people to climatic, economic, or socio-political disruption (Kasperson et al., 1995). Globally, land use/cover changes significantly affect central aspects of the world system's functioning. The drivers of land-use change differ over time and from country to country and regions of a particular country (Paul & Rashid, 2017).

To this end, the driver variables were determined by literature studies and expert opinions. These variables are; (1) The distance to rivers, (2) The distance to roads, (3) Elevation (DEM), (4) Slope, (5) The distance from 2006 residential areas, (6) Transition/phenomenon possibility from irrigated agriculture class to all land use classes, (7) Probability of transition/phenomenon from all land use classes to settlement class, (8) distance to places that pass to all land use classes (irrigated agriculture disturbance potential), (9) Distance from all land use classes to settlement class (potential transition to settlement) between 2006-2012, and (10) The distance from all land use classes to the water structures class between 2006 and 2012 (the potential for transition to water bodies).

The reference years were chosen as the years 2006 and 2012. One reason for this is that the land-use changes between these two reference years are more pronounced. During this period, changes were also made in driver variables. More systematic and less uncertain variables were used to reduce the stochastic structure of the model. The driver variables dealt with in this period are as follows;

- Elevation / Digital elevation model (DEM) (Figure 3.6),
- The distance to the urban areas in 2006 (Figure 3.7a),
- The distance to the rivers (Figure 3.7b),
- Distance to roads (Figure 3.7c),
- Distance to Urban Development Areas (UDA) (Figure 3.7d),

- The distance to the 2012 water bodies (Figure 3.7e), and
- Slope (Figure 3.7f).

The distance to 2006 urban areas were obtained from the CORINE-2006 land-use by measuring the Euclidean distance. River stream network were extracted using GIS-based hydrological analysis. The road data was downloaded from the World Vmap0 database developed by food and Agriculture Organizations (FAO), which is an independent GIS laboratory database in vector data format (GeoNetwork-FAO, 2021).

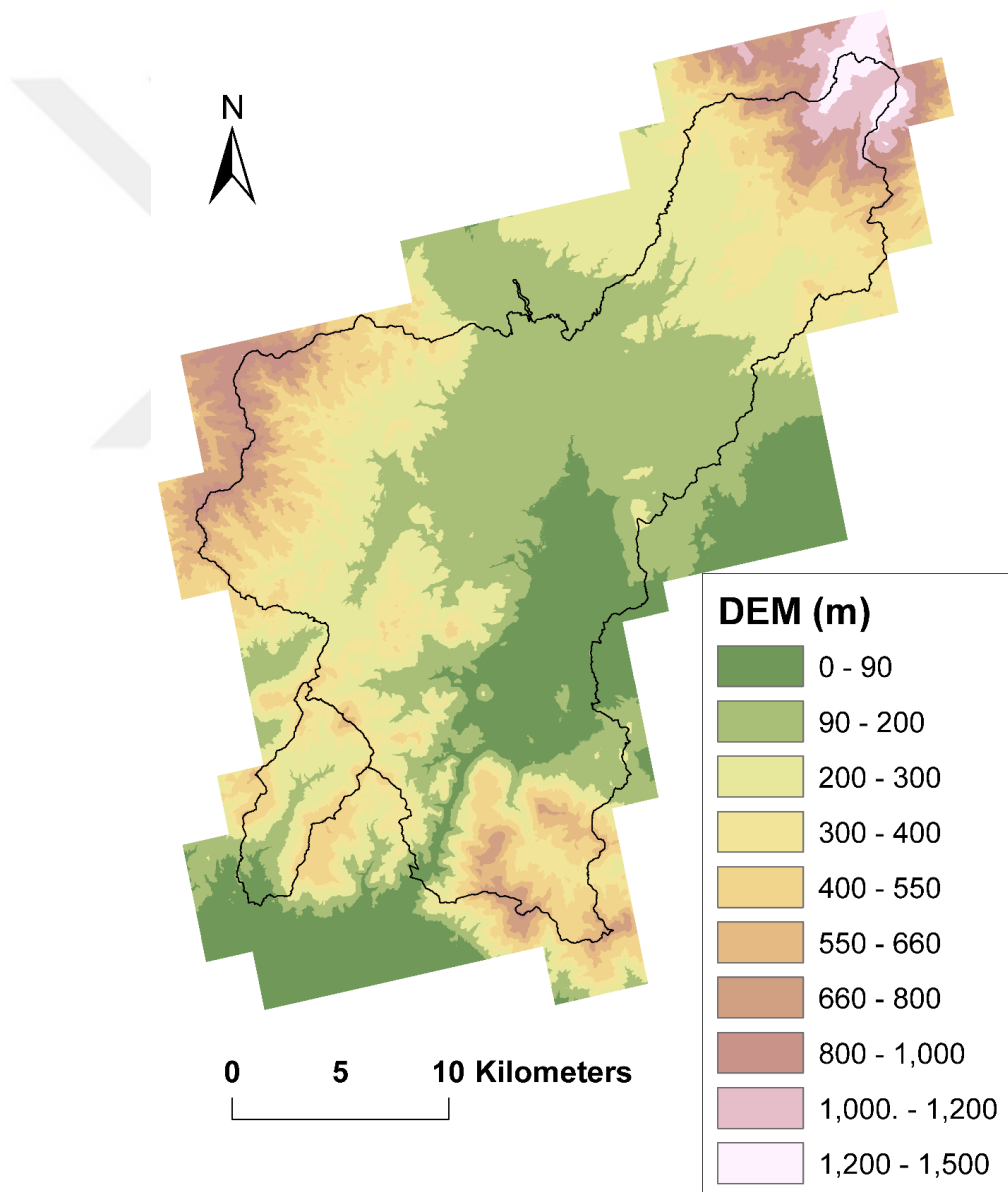


Figure 3.6 Digital Elevation Model

The distance layer data to rivers and roads were produced in the same way as the distance layer data to the urban areas in 2006 was generated. In order to create the distance layer to the UDA, the 2014 İzmir-Manisa 1/100.000 scaled Environmental Plan was used. The distance layer data to the water bodies of 2012 was created by applying the Euclidean Distance tool to the water bodies data obtained from the 2012 land-use class-map.

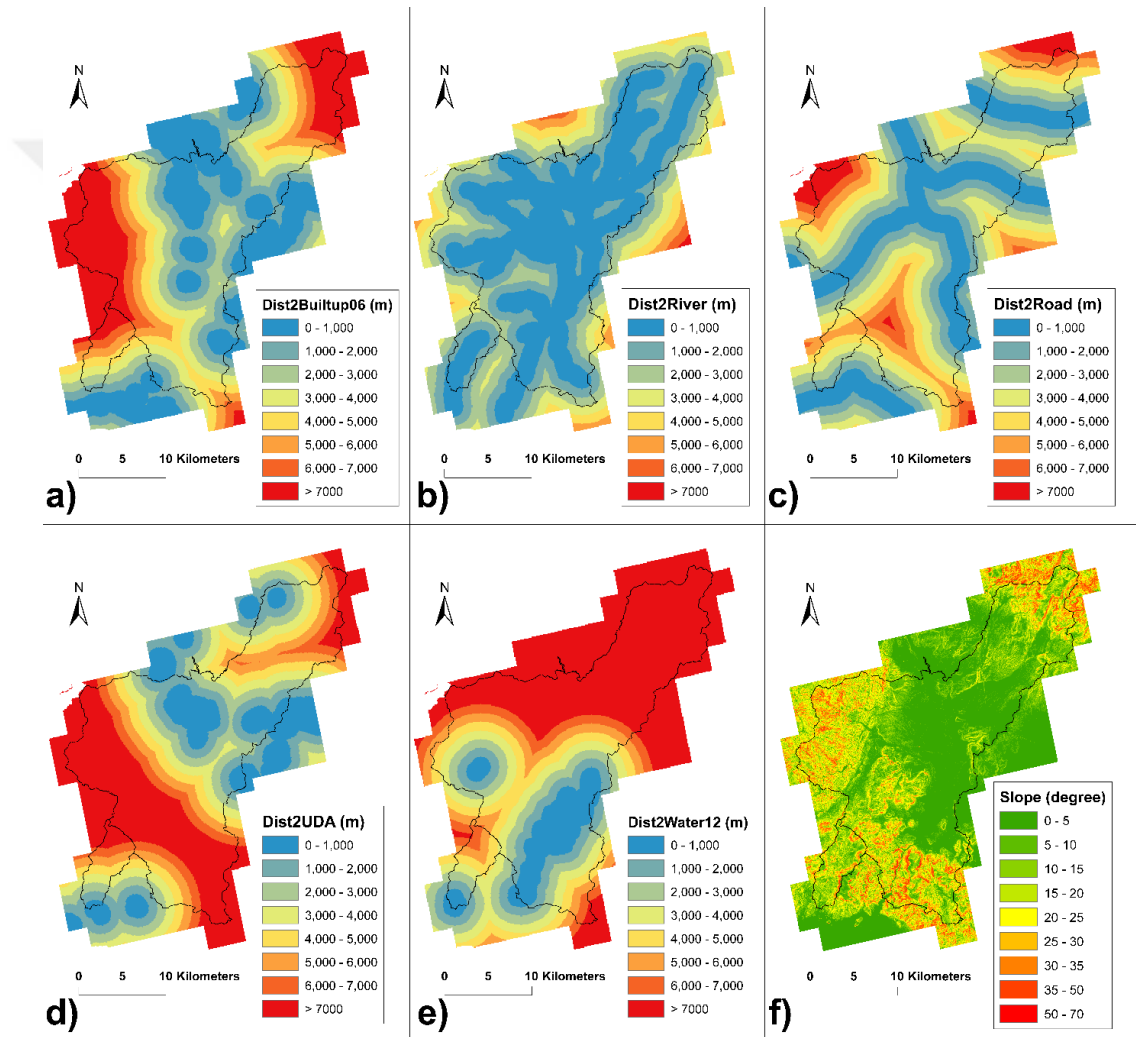


Figure 3.7 Distance to a) urban areas in LU-2006, b) stream network, c) roads network, d) urban development areas, e) water bodies in LU-2012 layers and f) slope layout

Some land cover classes are suitable for multiple uses. For example, agricultural areas are under the pressure of land-use change due to population growth. This pressure not only forces change in land cover but, also intensifies existing land use

(Messerli et al., 2000). It is, therefore, necessary to understand the main drivers of land-use changes in the basin. The above driver variables contribute to a certain degree of land-use changes. Some of the variables are natural variables such as elevation and slope, while others appear as anthropogenic variables. For example, examining the distance to residential areas is considered as an indicator of how much these areas can expand in the future. In LULC model, all variables are tested by looking at Cramer V values in the land-use change sub-model expressed as LULC-sub. The Cramer V scale allows for a quick test of the potential generating capacity of a driver variable. In this way, both quantitative and qualitative variables can be tested (Eastman, 2015b, 2015a).

The test procedure in LULC-Sub is based on a situation table analysis. For the qualitative variables, it uses the local categories of the variable to correlate the land cover distribution in the later land cover map. To perform this test, quantitative variables are divided into 256 categories. This is a quick but imprecise potential determination tool. The quantitative association measure in the LULC-sub test procedure is Cramer's V. A high Cramer's V value indicates that the potential explanatory value of the variable is good. However, if Cramer's V value is too low, it is a good indicator that a variable can be eliminated. The P-value refers to the probability that Cramer's V-value is significantly different from 0. This shows that all pixels are sampled independently, and there is no spatial dependence on their values. Therefore, a high value of p is a sure sign that the variable can be rejected. In general, it is known that variables with Cramer's V value of 0.15 or higher are useful, while those with values of 0.4 or higher are most useful (Eastman, 2015b, 2015a). Within the study's scope, all variable candidates mentioned above were subjected to the Cramer V scale test.

LULC-sub is a sub-model of LULC that is created in a very effort and time-intensive way before making verification and future predictions. In this sub-model, transition potentials of the transitions shown in APPENDIX-2 are produced under the influence of driver variables. In the sub-model (LULC-sub), there is a Multi-Layer Sensor (MLP) tool that enables the creation of transition potentials and can perform multi-layer analysis.

MLP has an interface where training parameters can be set, and the number of iterations can be determined. At the same time, calculating transition potentials with MLP, the samples used for the training process are taken from the pixels that have undergone and have passed through the modeled transition. MLP tracks and changes the starting and ending learning rates of a dynamic learning procedure. The dynamic learning procedure starts with an initial learning speed and gradually decreases the repetitions until the final learning rate is reached when the maximum number of repetitions is reached. If significant oscillations in the Root Mean Square error are detected after the first 100 iterations, the learning rates (start and finish) are reduced by half, and the process is restarted (Eastman, 2015b, 2015a).

For MLP, one or more of the transitions in APPENDIX-2 are selected first. Although MLP gives transition potentials for all 65 transitions shown in APPENDIX-2, the Change Prediction tool of LULC allows a maximum of 8 transitions to be used for future prediction. Therefore, the transitions in APPENDIX-2 have been discussed before running LULC-sub within the scope of the study. After the transitions were determined, the Cramer V values of the driver variables were determined for each transition. Cramer V values of driver variables are given in Table 3.3. The transitions entered in LULC-sub are given in Table 3.4 together with the accuracy values obtained by running the MLP tool.

Table 3.3 Cramer V values of driver variables

Potential Driver Variables	Cramer V's*
DEM	0.3864
SLOPE	0.2717
DISTANCE TO Built-up areas (2006)	0.2663
DISTANCE TO RIVERS	0.1781
DISTANCE TO ROADS	0.1963
DISTANCE TO Urban Development Areas (UDA)	0.3007
DISTANCE TO WATER (2012)	0.2331

* Note: Cramer V's > 0.15 to evaluate the driver variables

Table 3.4 Selected transitions and MLP values for LULC-sub

Transition No.	Evaluated Transitions	MLP's*	Transition Area (m ²)
13	AGRR_nonW to Built-up	0.91	1472986.51
47	AGRR_nonW to Trees and Fruits	0.97	45948.08
15	AGRR_nonW to Forest	0.73	1300341.11
25	AGRR_W to Transportation	0.95	465331.80
14	Forest to AGRR_nonW	0.78	1454484.18
7	Forest to Pastures	0.65	20618540.29
19	Pastures to AGRR_nonW	0.79	979241.72

* MLP > 0.50 to include the transition into the model

3.6.3 Verification Analysis

The Change Prediction step in LULC provides a control for the dynamic land-cover change estimation process. Once a due date has been specified, the amount of change at each transition can be modeled. Markov Chain analysis is used to model these transitions. There are two basic variation models here: a hard prediction model and a soft prediction model. Generally, soft prediction model results are preferred for habitat and biodiversity assessment. While hard prediction provides only one realization, soft prediction is a comprehensive assessment of change potential. In a study where many land-use transitions are evaluated together, the soft forecast option is generally used (Eastman, 2015b, 2015a). The steps of the verification phase of the predictive analysis performed within the scope of the study are given in Figure 3.8. After the LULC calibration and verification are completed, predictions can be made for the upcoming years.

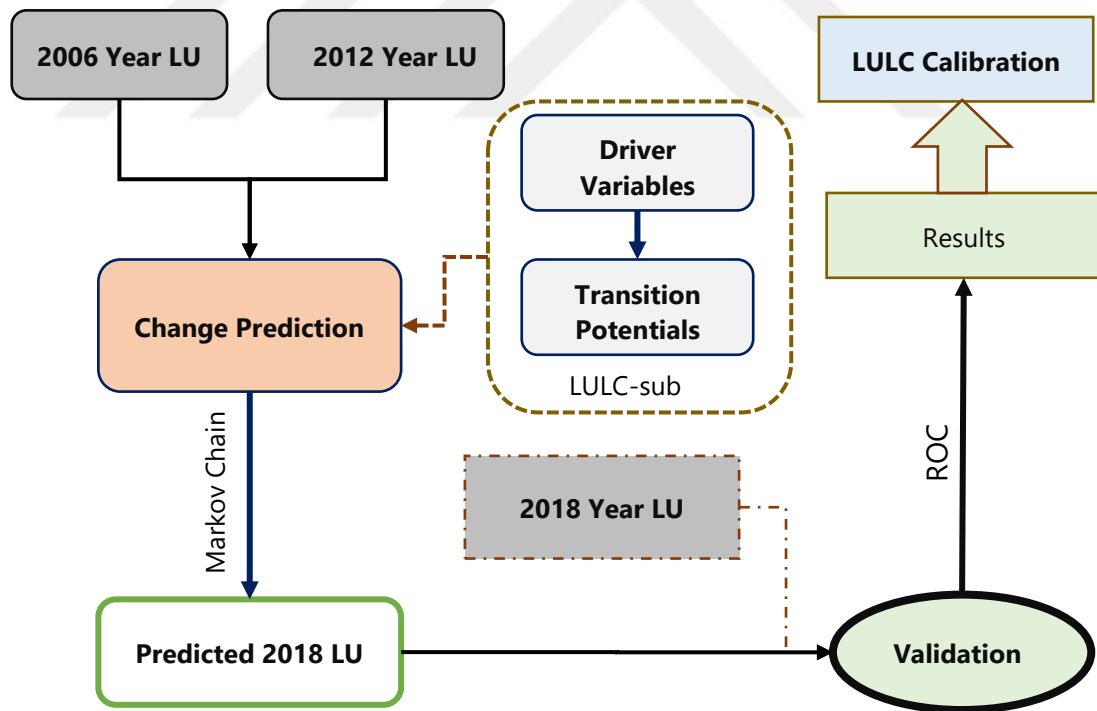


Figure 3.8 Flow charts of the verification analysis of LULC

Accuracy analysis in LULC is based on the use of the current two years known and used in the model as well as another current year for verification purposes. The year

2018 was taken as a basis for the accuracy analysis within the scope of the project. Markovian matrix was created with Markov chain analysis to estimate the year 2018. This matrix shows the transition possibilities between land uses. Receiver Operating Characteristic (ROC) analysis was performed to compare the predictive layer data obtained in the LULC model with the known layer data of 2018. When the distinction threshold in binary classifier systems varies, the ROC curve is calculated using the sensitivity to accuracy ratio. It can be expressed clearly as the proportion of true positives to false positives. Expressing the success of the system with a single value is expressed by the area under the ROC curve (Area Under Curve - AUC). The higher this field value is, the higher the reliability value of the system. In addition to the AUC value, two other parameters need to be examined in the ROC. One of them is T(p) true-positive, and the other one is F(p) false-positive parameters. If T(p) is higher than F(p) in the analyzed samples, the results are "successful". As the ROC curve approaches the 'false positive rate' axis, the level of success decreases (NetCAD, 2021).

The ROC analysis results within the scope of this study are given in Table 3.5 for each land-use class.

Table 3.5 AUC values for each land use class of the ROC analysis made as a result of the predictive analysis.

LANDUSES	ROC (Validation Values) AUC's
Built-up areas	0.92
Transportation	0.937
Agriculture / AGRR_non_W	0.916727
Agriculture / AGRR_W	0.965502
Trees and Fruits	0.912706
Forest	0.888727
Pastures	0.889147
Bare Soil	0.937497
Water bodies	0.9381

3.6.4 LULC's Forecasting Model for the Coming Years

Following the accuracy analysis, the LULC model was started to be practiced for predicting the upcoming years. The reference year to be considered is 2035. Therefore, the year of 2035 should be entered in the Markov Chain analysis. In this way, Markovian analysis under existing driver variables and transition potentials will be provided. Figure 3.10a and Figure 3.10b show the land use layers for the reference year of 1990 and predicted 2035, respectively. In Figure 3.9, the dynamic of change of land-use class over 45 years since 1990 is shown.

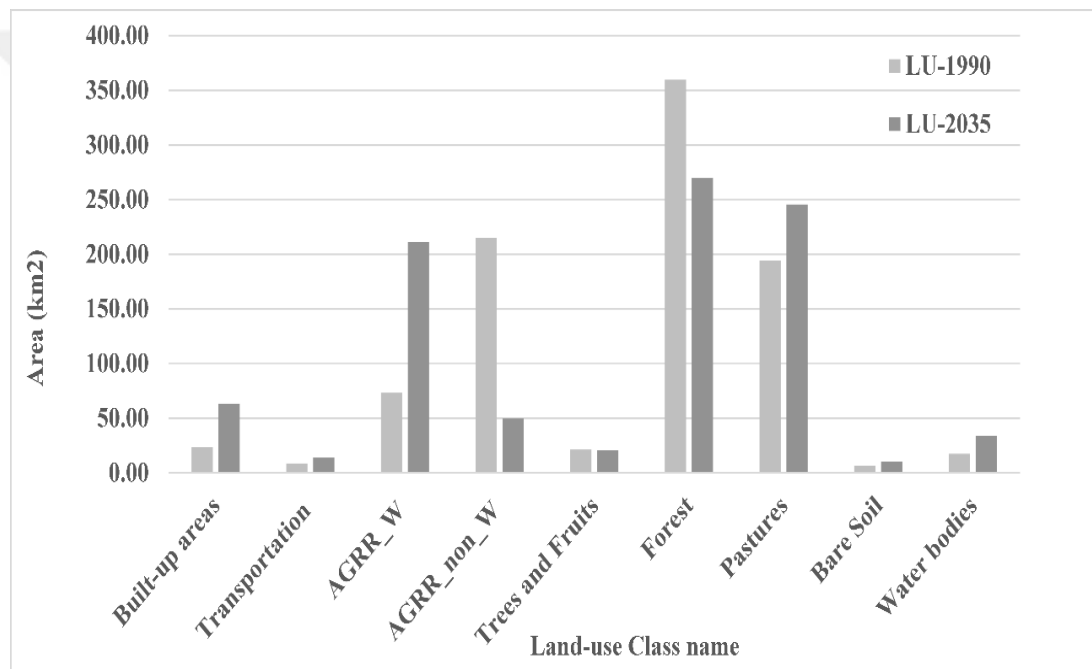


Figure 3.9 The distribution of land use classes between 1990 and 2035 years

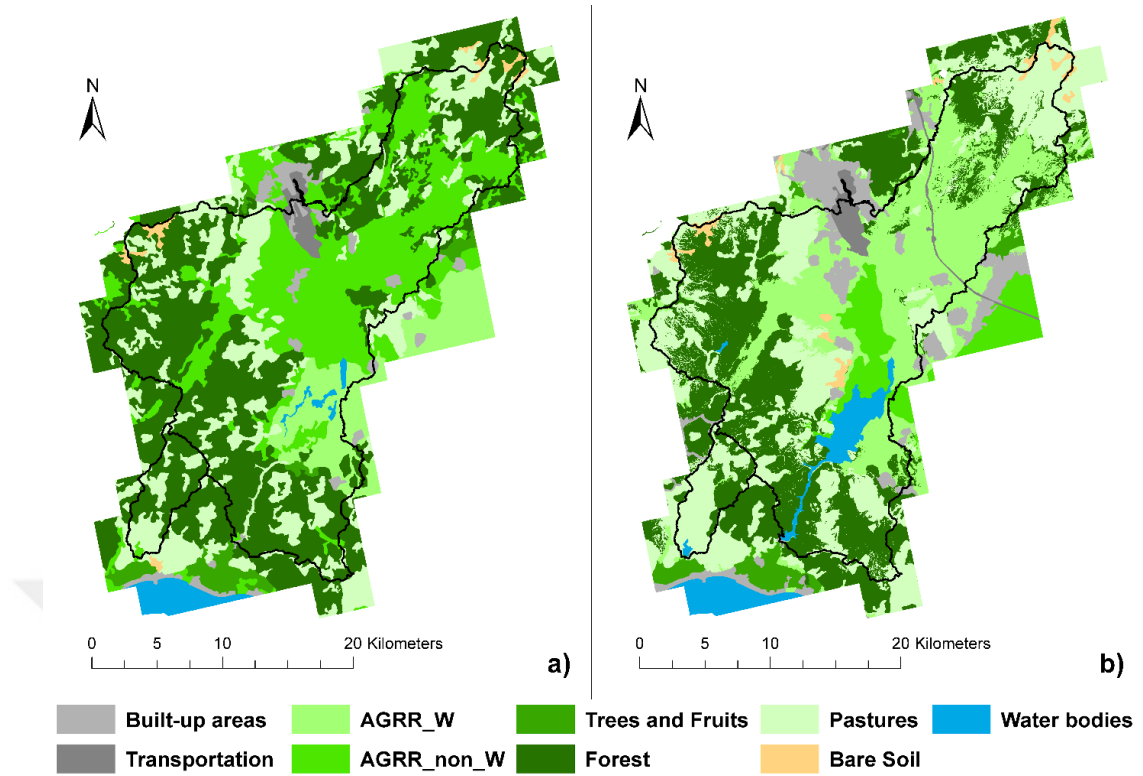


Figure 3.10 Land-use maps for a) 1990 reference, and b) 2035 predicted years

3.7 Curve Number (CN) Generation

The potential maximum retention S could be estimated based on CN. CN is a dimensionless number that ranged between 0 and 100 defined by SCS. For impervious and water surface $CN = 0$, for remain natural surface $CN < 100$. Eq. (3.24) shows the formula governs the relationship between the potential maximum retention S and CN.

$$S = \frac{25400}{CN} - 254 \quad (3.24)$$

Table 3.6 shows the CN values for different land-use, Hydrologic Soil groups and Impervious ratio

3.7.1.1 Soil Group Type

Soils are classified by the SCS into four Hydrologic Soil Groups (HSG) based on the soil's potential runoff. The HSG's are A, B, C and D. Where A's generally have the smallest runoff potential and D's the greatest. (NRCS, 1986), Technical Release–55 provides a details of soil classification. It can be sum-up as the listed in Table 3.7.

Table 3.6 CN values for different land-use, Hydrologic Soil groups and Impervious ratio (Chow et al., 1988)

Land-Use Description	Corine 3 rd level	Hydrologic Soil Group			
	Legend	A	B	C	D
Cultivated Land: without conservation Treatment	211, 242	72	81	88	91
with conservation Treatment	212, 222	62	72	78	81
Pasture or range land: poor condition	243	68	79	86	89
good condition	231	39	61	74	80
Meadow: good condition	234	30	58	71	78
Wood or forest land: thin stand,					
poor cover, no mulch	323, 311, 313	45	66	77	83
good cover	312, 223, 221	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more than area	321, 142, 141 333, 331, 324,	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	332, 334	49	69	79	84
Commercial and business area (85% impervious)	121, 124	89	92	94	95
Industrial districts (72% impervious)	133, 132	81	88	91	93
Residential	111, 112	51	68	79	84
Streets and roads:					
Paved with curbs and storm sewers	122, 123	98	98	98	98
gravel	131	76	85	89	91

Table 3.7 Hydrologic Soil Groups (HSG) properties (Cronshey et al., 1985)

HSG	Soil Type	Characteristics
Group A	sand, loamy sand or sandy loam	low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission
Group B	silt loam or loam	moderate infiltration rate when thoroughly wetted and consists chiefly or moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
Group C	soils are sandy clay loam	low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure.
Group D	soils are clay loam, silty clay loam, sandy clay, silty clay or clay	This HSG has the highest runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high-water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.

3.7.1.2 Soil Texture Triangle

In order to be able to classify soil type (e.g., sand, loam, clay, etc.) to represent it as HSG classification, the Soil Texture Triangle (Figure 3.11) employed. The Soil Texture Triangle developed by the U.S. Department of Agriculture for defining textures based on proportions of sand, silt and clay. The particle size distribution is characterized by the soil texture, which is determined by the proportions by weight of clay, silt and sand. Clay is defined as particles with a diameter less than 0.002 mm. Silt has a particle diameter range from 0.002 mm to 0.05 mm and sand has a particle diameter range from 0.05 to 2 mm (NRCS, 2020).

To apply the concept of The Soil Texture Triangle classification, the soil content (clay, silt, and sand) with a 5 cm depth downloaded from SoilGrids™ project, it is a system for global digital soil mapping that uses state-of-the-art machine learning methods to map the spatial distribution of soil properties across the globe. This system provides a wide ranges of soil properties, Driven properties (e.g., Organic carbon density in g/dm^3 , and Soil organic carbon stock in t/ha), Physical Soil Properties (e.g., Bulk density in cg/cm^3 , Clay content in g/kg , Coarse fragments in cm^3/dm^3 , Sand in g/kg , and Silt in g/kg), and Chemical Soil Properties (e.g., Cation exchange capacity (at ph 7), in mmol(c)/kg , Nitrogen in cg/kg , Soil organic carbon in dg/kg , and pH water). Beside it is provide global soil classification map based on World Reference Base (2006) Soil Groups.

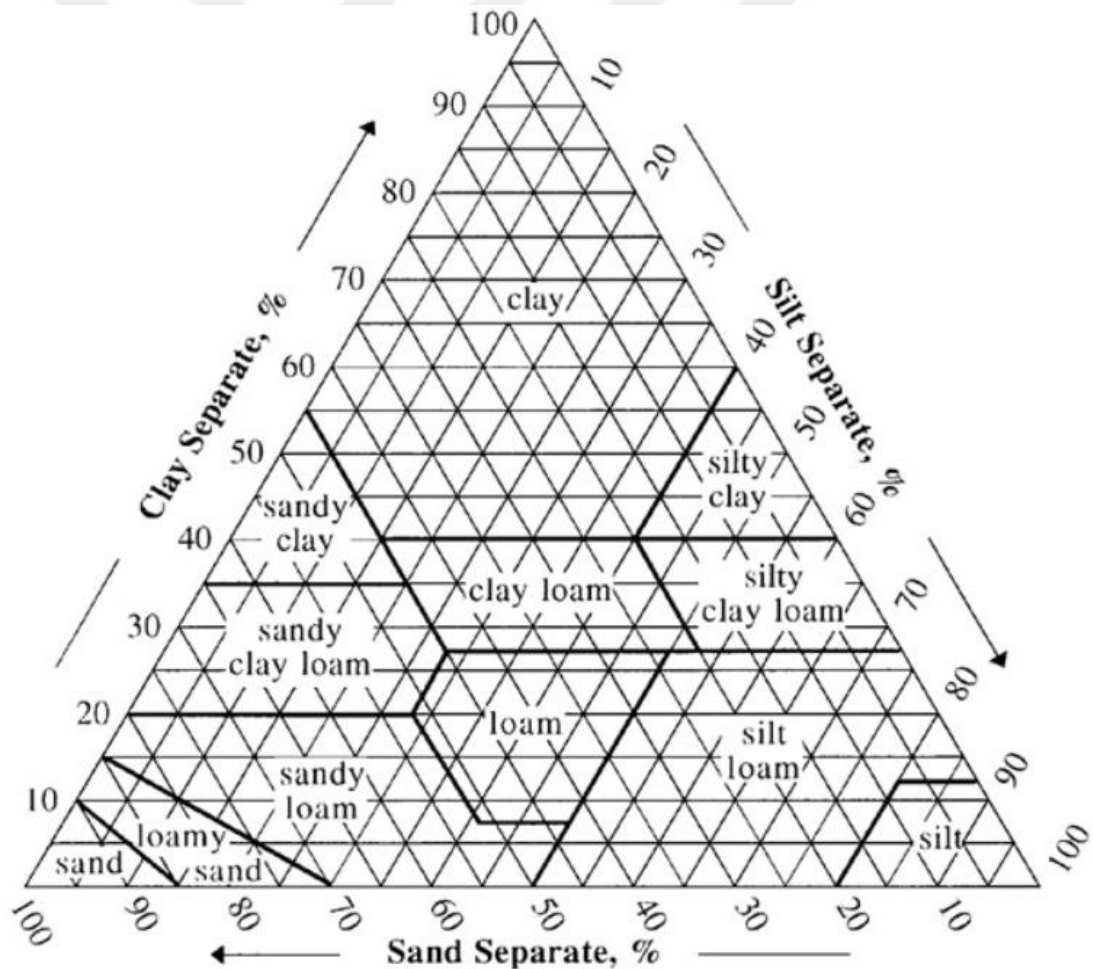


Figure 3.11 Soil Texture Triangle (graphical presentation) (NRCS, 2020)

Table 3.8, Shows the command line used within the GIS environment (Raster Calculator) to classify soil based on Clay, Silt, and sand content using NRCS Soil Texture Triangle

Table 3.8 The command line used to classify soil based on Clay, Silt, and sand content (Cronshey et al., 1985)

Soil Category	Raster Calculator Script
SAND	((silt + 1.5*clay) < 15)
LOAMY SAND	((silt + 1.5*clay >= 15) && (silt + 2*clay < 30))
SANDY LOAM	((clay >= 7 && clay < 20) && (sand > 52) && ((silt + 2*clay) >= 30) (clay < 7 && silt < 50 && (silt+2*clay)>=30))
LOAM	((clay >= 7 && clay < 27) && (silt >= 28 && silt < 50) && (sand <= 52))
SILT LOAM	((silt >= 50 && (clay >= 12 && clay < 27)) ((silt >= 50 && silt < 80) && clay < 12))
SILT	(silt >= 80 && clay < 12)
SANDY CLAY LOAM	((clay >= 20 && clay < 35) && (silt < 28) && (sand > 45))
CLAY LOAM	((clay >= 27 && clay < 40) && (sand > 20 && sand <= 45))
SILTY CLAY LOAM	((clay >= 27 && clay < 40) && (sand <= 20))
SANDY CLAY	(clay >= 35 && sand > 45)
SILTY CLAY	(clay >= 40 && silt >= 40)
CLAY	(clay >= 40 && sand <= 45 && silt < 40)

The analysis shows that Tahtalı and Ürkmez Basins consist of two main soil types (Figure 3.12): Clay loam (SHG D) and loam (SHG B).

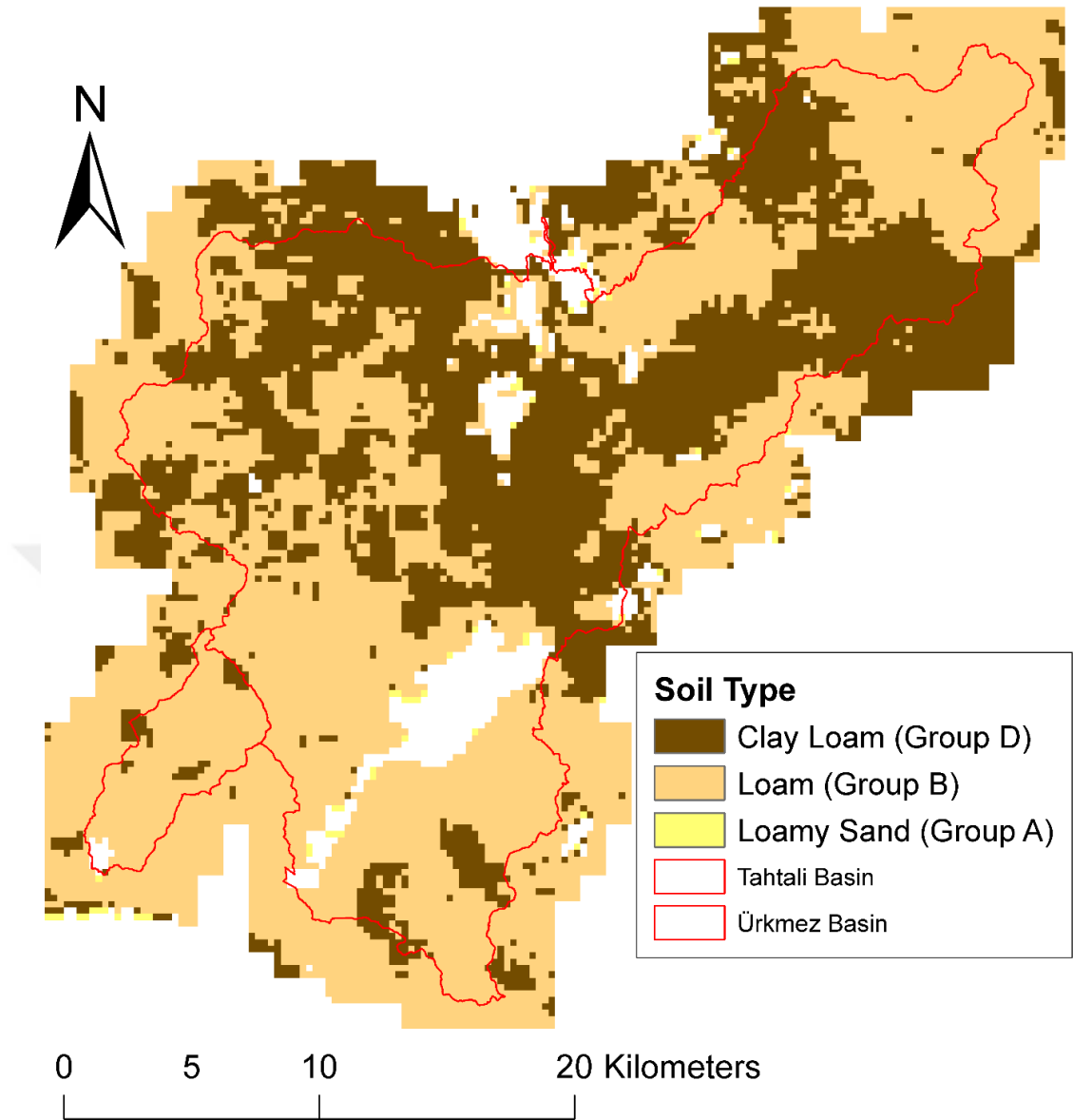


Figure 3.12 Hydrologic Soil Groups map

Here, the HGS (Figure 3.12), and land-use maps (CLC 1990 & 2035 (Figure 3.10a & b) are produced, Curve number distribution maps for the study area can be calculated. Using the GIS combine tool, it is possible to produce a unique combination of HGS & Land-use. CN number (Figure 3.13) assigned to the produced unique combination referring to Table 3.6.

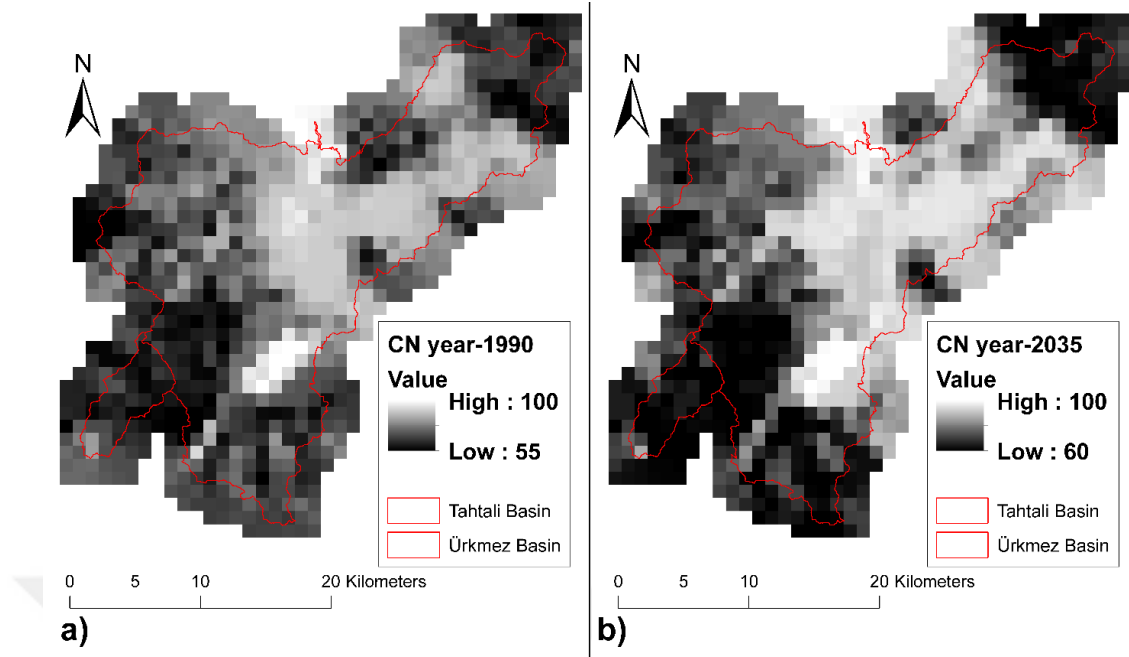


Figure 3.13 Curve Number maps generated for a) 1990-, b) 2035 years

3.8 Hydrologic Modeling

The HEC-HMS model by the United States Army Corps of Engineers (USACE), which is basically structured by a basin model, a meteorological model and control specifications as main model components (Onuşluel Gül et al., 2010; USACE, 2021), is the main hydrological modeling platform employed in the current study. The runoff hydrograph generated from the GIUH model was employed for model parameters' calibration. In order to increase the accuracy in hydrologic outputs, Tahtalı basin was divided into six sub-basins (Figure 3.14). As each sub-basin outlet feeds the reservoir lake, the hydrographs were routed through the reservoir to generate the superposition flood hydrograph.

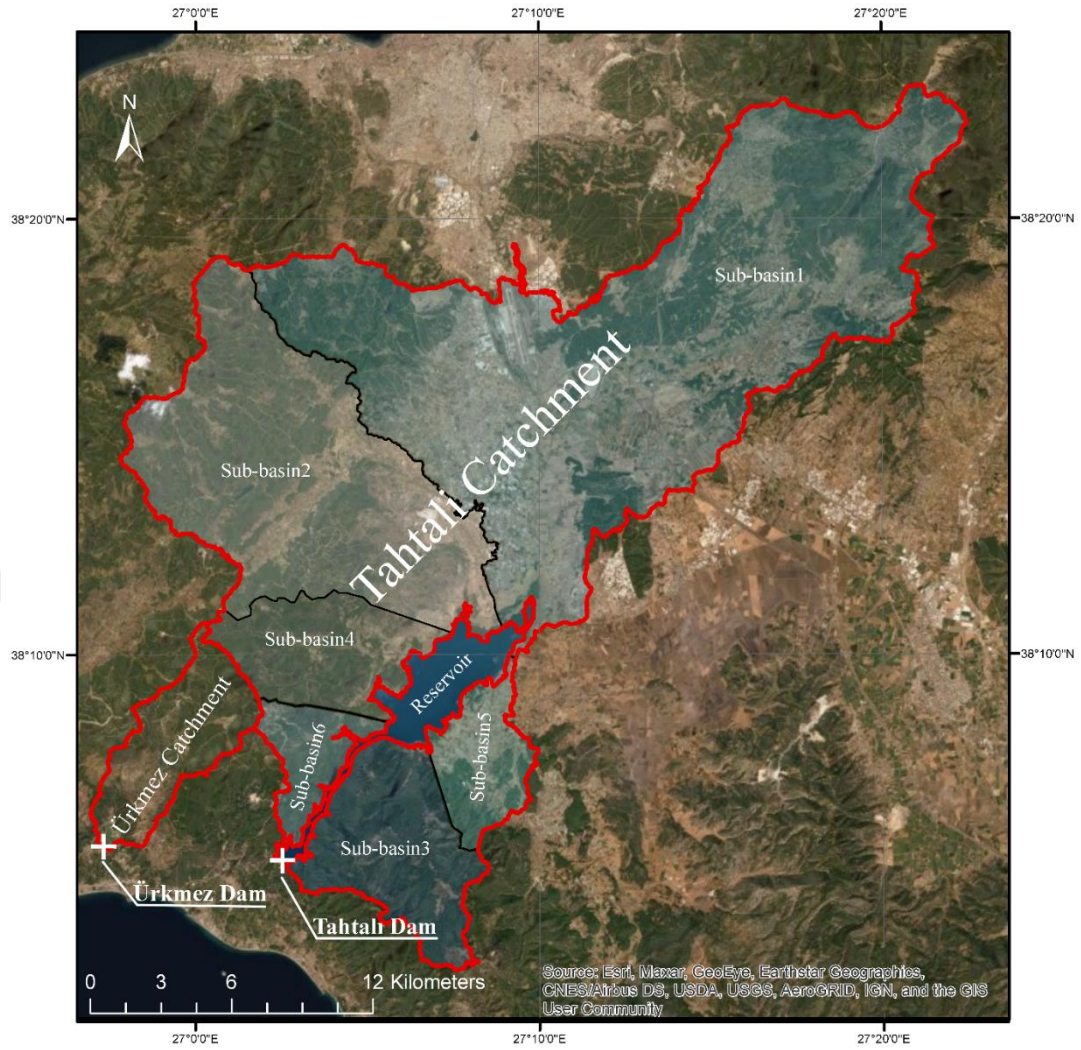


Figure 3.14 General segmentation of the study area

Here, the SCS-CN abstraction method for computing the rainfall loss rates and the Muskingum-Cunge method for routing surface flow in stream reaches were utilized. As HEC-HMS was set up in the study to allow distributed simulations, the HEC-DSSVue package was handled to construct a temporally gridded precipitation dataset representing the depths generated from the instantaneous intensity method. Figure 3.15 shows, as an example, the Tahtalı subbasin-1 schematic of the HEC model. Figure 3.16a and Figure 3.16b show the depressionless DEM and flow direction maps of Tahtalı subbasins-1.

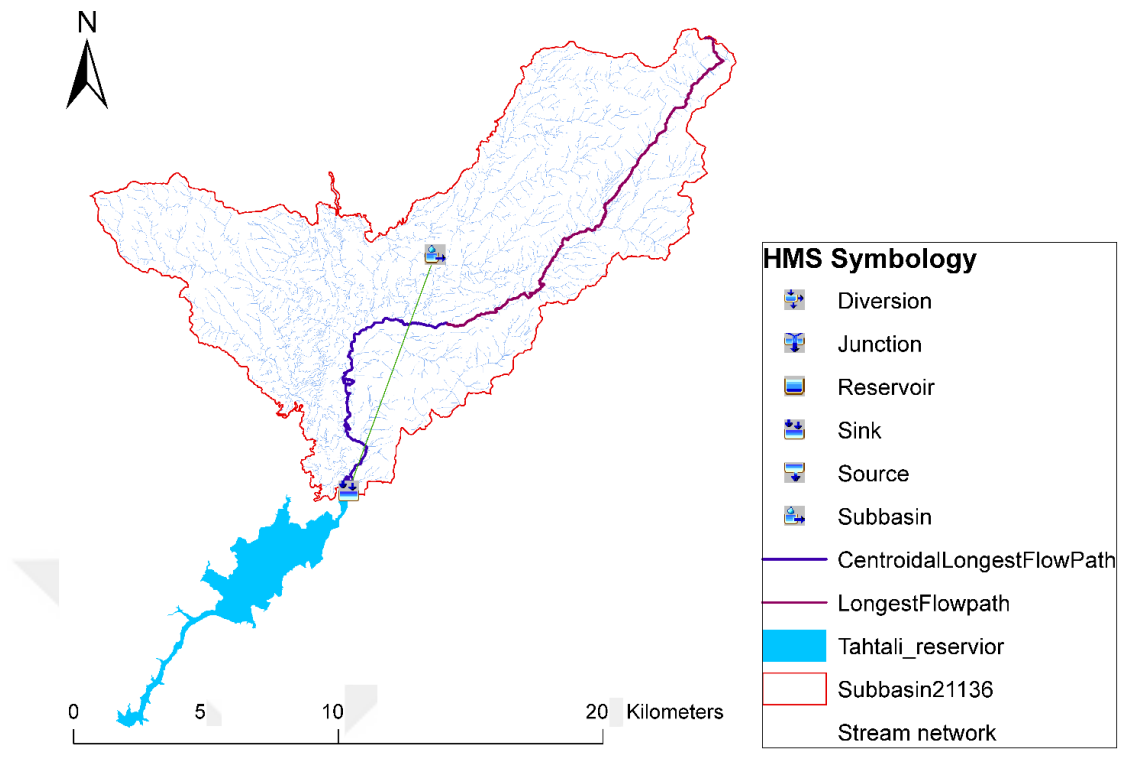


Figure 3.15 Basin schematic generated with HEC-GeoHMS

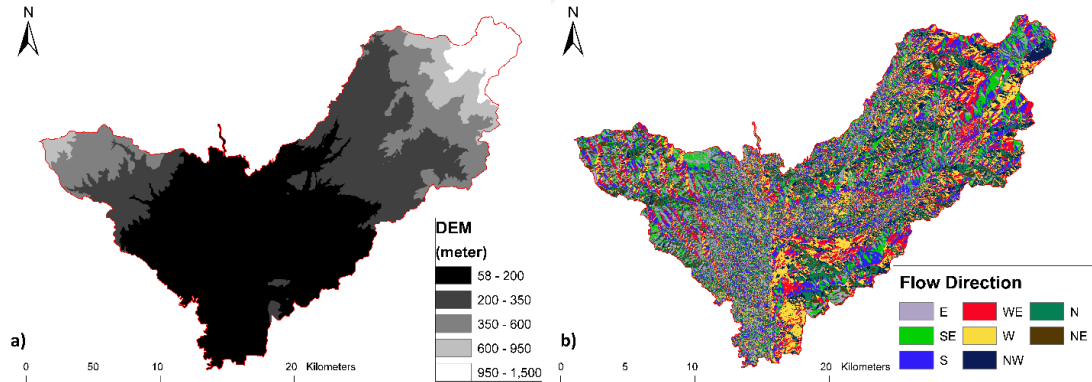


Figure 3.16 The maps of a) depressionless DEM, b) flow directions of the Tahtalı subbasin-1

In considering the two other meteorological components, i.e., average temperature and solar radiation, a percentile analysis associated to the 90th % probability of exceedance was performed for each input time series in order for identifying the representative month of the year in which extreme events were mostly recorded (see Figure 3.17). As the modified Clark (ModClark) method takes count of the spatial variability of processes and characteristics (USACE, 2021; Yang et al., 1999), it was selected for rainfall-runoff transformation. A standard hydrologic grid mesh of 1 km

was defined for the entire region to control the distributed model processing. After selecting a suitable simulation period and a computational time interval of 10 minutes, all required modeling components were prepared accordingly. The model was operated for 100-, 500- and 1000-year recurrence intervals as well as for the extreme consideration of PMP case.

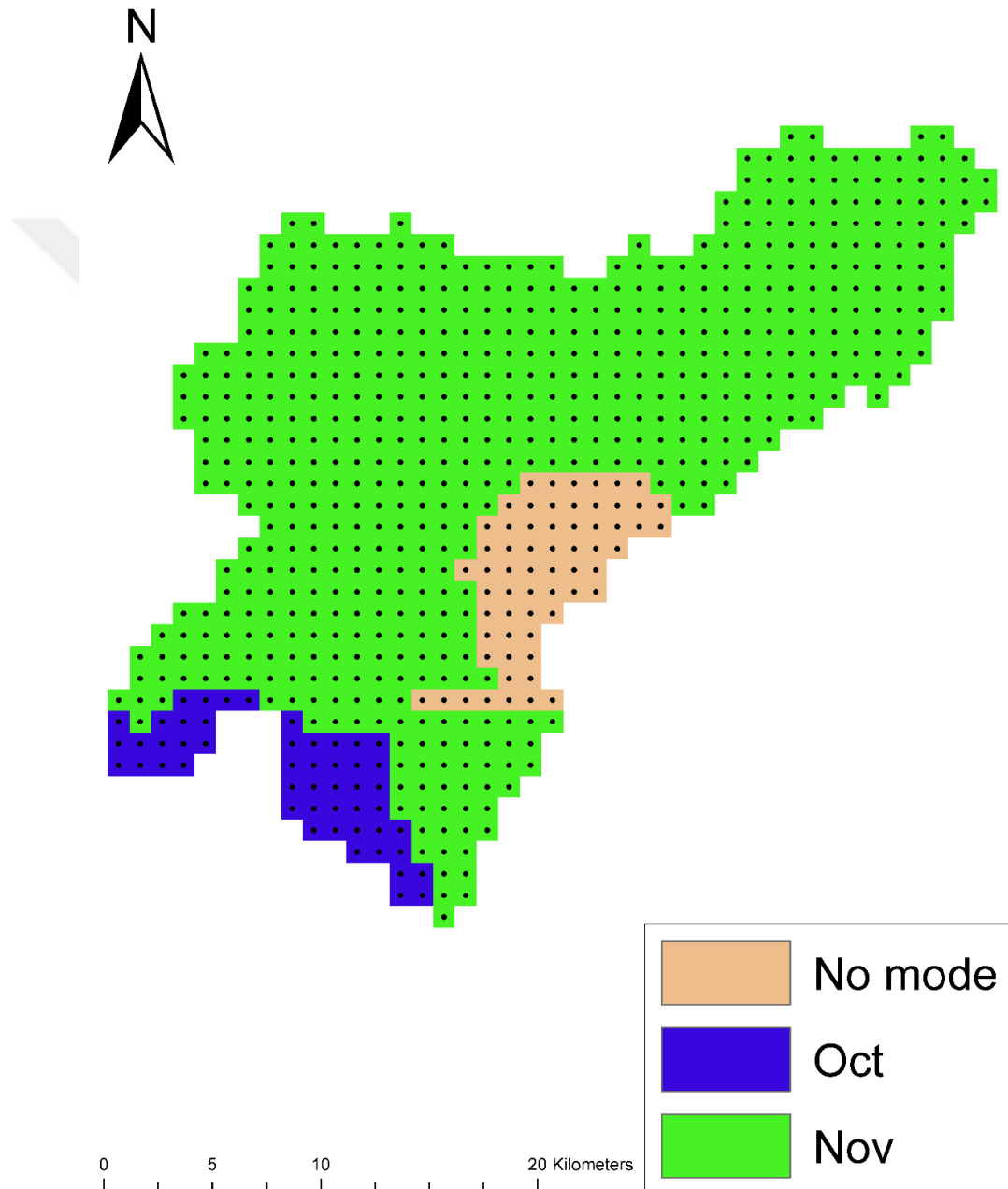


Figure 3.17 90th percentile of the occurrence precipitation extreme events for the period of 1970-2004

3.9 Hydraulic-Hydrodynamic Simulations for Dam Failure Analyses

The two-dimensional (2D) HEC-RAS model by USACE (2020) has been widely implemented in several flood inundation studies (e.g., Papaioannou et al. 2016; Quirogaa et al. 2016; Vozinaki et al. 2017; Patel et al. 2017; Afshari et al. 2018; Shustikova et al. 2019; Costabile et al. 2020; Garcia et al. 2020). It has been expressed with remarkable evidence that the performance of the 2D-RAS model has already proven its adequacy when compared to some other models based on a benchmark analysis conducted by the developers (Brunner, 2018). For this reason, the two-dimensional HEC-RAS hydrodynamics model has been chosen in the current work to simulate the dam-break connected flood inundation mapping.

The computed flood hydrograph generated from the hydrologic modelling were utilized as input to the hydraulic model. Considering the accurately modelled upstream reservoir bathymetry, the level pool routing method was used conveniently. An inline structure was employed to represent both dam body and spillway dimension in the study presented. A storage area was used to model the upstream reservoir, and a 2-dimensional mesh with a 20-meter resolution to simulate the downstream area, while on the other hand, a 1-meter mesh resolution was applied in representing the river and spillway centerlines (Figure 3.18).



Figure 3.18 HEC-RAS schematic representation including upstream storage area, inline structure, downstream mesh and location of inflow hydrographs for a) Tahtalı, and b) Ürkmez dams.

3.9.1 Dam Breach Geometry Estimation

Five of the most popular experimental approaches (see section 2.6) for predicting dam breach characteristics (e.g., the size of dam breach and formation time) were employed to predict parameters for breach definition. The study site of Ürkmez Dam was selected to perform the analysis. The breach modelling techniques were formulated from the statistical study of data derived from the recorded dam break events of different dam sizes. These approaches were used to predict Ürkmez Dam breach characteristics subjected to numerous scenarios of failure mode type (e.g., piping and overtopping) with several initial upstream reservoir water levels. A decreasing interval of 2 m - from the maximum upstream reservoir storage level (48 m a.s.l.), were investigated for these methods. The centerline of the dam was accepted to be the dam breach location for both piping and overtopping collapse types. As the weir and piping coefficients for the earthen sand and gravel type of dams (similar to the Ürkmez Dam case) are advised to range between 2.6 and 3.0 in case of overtopping mode, and 0.5 to 0.6 in piping failure mode (Brunner, 2014), these coefficients were set to be in average as 2.8 and 0.55, for overtopping and piping failure modes, respectively.” for cases similar to Ürkmez Dam. In the case of piping failure mode, the breach start level though piping was considered to be at the level equal to the half of the water height in the reservoir ($h_w/2$).

The Probable Maximum Precipitation (PMP) was calculated using (Hershfield, 1965) statistical method based on the annual maximum precipitation observations for 67 years in the period 1938-2015. The Probable Maximum Flood (PMF) hydrograph generated from the PMP was used as input hydrograph feeding the HEC-RAS 5.0.7 hydraulic model. Since the topography of upstream reservoir was modeled accurately, the level pool routing method was used conveniently. The main river bed was assumed to be the breach final bottom elevation. The Saint-Venant full-momentum set of equations were practiced to compute the outflow flood hydrograph generated by a dam breach for each simulation definition.

3.9.1.1 Quantity and Timing of the Maximum Discharge

The resulting flood hydrograph associated with dam-break in the case of Ürkmez Dam was calculated by using the Saint-Venant full momentum set of equations in HEC-RAS 5.0.7 with a different initial reservoir level for five different empirical approaches for both overtopping and piping failure modes as shown in **Error! Reference source not found.** The flood hydrographs were obtained at a point immediately after the downstream face of the dam. Here, slight increases are observed in the outflow hydrograph peak quantities in a pattern inversely proportional with the initial reservoir levels. The rationale behind this is that the dam becomes subject to different rates of inflow (observed from the PMF hydrograph) as the time passes from the initial reservoir water elevation. Indeed, higher rates of flow will have been accessed at the breach formation time in the case of lower initial reservoir elevations so that longer time will be required to allow the breach process to initiate (also considering the additional time for the full development of breach geometry on top of the breach start through the gradual development of breach geometry). As shown in Figure 3.19, the overtopping failure type has also a tendency to provide higher peak discharge value than the piping failure type. It was observed also that, Von Thun & Gillette approach (Eq.(2.14)), which estimates the breach development time with respect to the parameters h_w and B_{avg} , provides time to peak longer than the one obtained by Eq. (2.12) (i.e., time to peak as function of h_w). Besides, the peak discharge obtained using (t_f) from Eq. (2.14) is always less than the peak quantity obtained through using (t_f) from Eq. (2.12) .

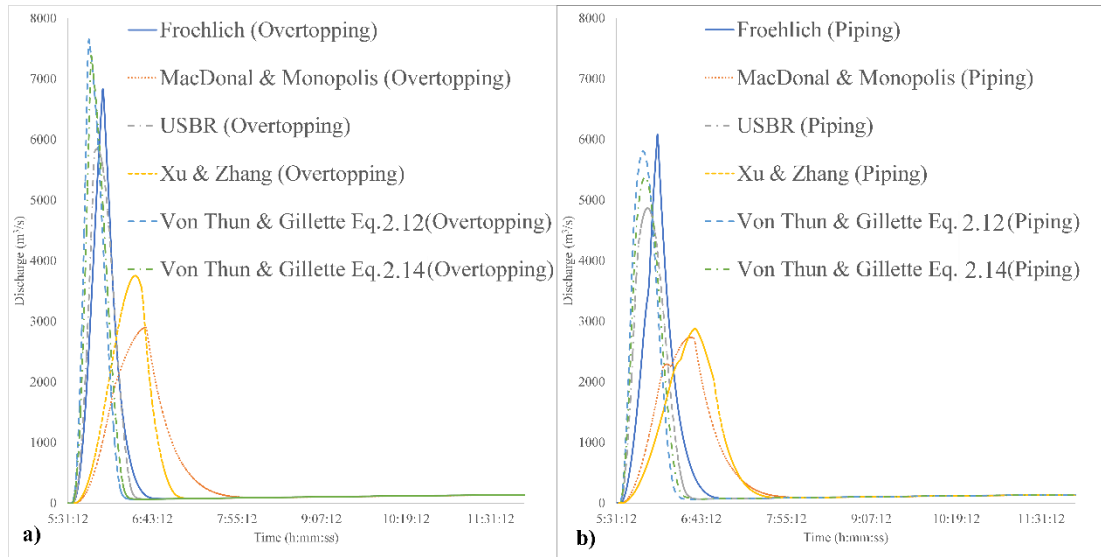


Figure 3.19 Flood hydrographs for reservoir elevation at 48 m a.s.l. for a) overtopping, and b) piping failure mode

Table 3.9 Maximum discharge & Time to peak discharge for all scenarios

Approach	Variable	Unit	Failure Type	Initial Reservoir Level (m)				
				48.0	46.0	44.0	42.0	40.0
USBR	Q _p	(10 ³ m ³ /s)	Overtop.	5.85	5.89	5.91	5.92	5.93
	T _p	min.		23.05	22.92	22.88	22.90	22.95
	Q _p	(10 ³ m ³ /s)	Piping	4.87	4.92	4.95	4.97	4.98
	T _p	min.		25.05	24.82	24.93	27.88	25.02
Von Thun & Gillette	Eq. (10)	Q _p	Overtopping	7.64	7.69	7.71	7.72	7.72
		T _p		16.05	15.83	15.88	15.92	16.02
	Eq. (12)	Q _p		7.37	7.41	7.44	7.44	7.45
		T _p		18.00	17.87	17.88	17.93	17.59

Table 3.9 continue

Approach	Variable	Unit	Failure Type	Initial Reservoir Level (m)				
				48.0	46.0	44.0	42.0	40.0
MacDonald & Monopolis	Q _p	(103 m ³ /s)	Overtop.	2.90	2.98	3.00	3.02	3.03
	T _p	min.		64.00	64.82	64.88	63.90	63.95
	Q _p	(10 ³ m ³ /s)	Piping	2.73	2.78	2.81	2.85	2.85
	T _p	min.		64.02	61.82	61.88	61.90	61.98
Xu & Zhang	Q _p	(10 ³ m ³ /s)	Overtop.	3.75	3.82	3.87	3.89	3.91
	T _p	min.		56.12	55.83	54.88	55.90	54.97
	Q _p	(10 ³ m ³ /s)	Piping	2.88	2.94	2.98	3.00	3.02
	T _p	min.		65.08	64.83	64.88	64.93	64.95

3.9.1.2 Failure Mode Impact on Flood Hydrographs

With the aim of examining the effect of failure type on the resulting hydrograph. Firstly, the geometry of the breach for all the approaches were predicted for the maximum reservoir level for both overtopping and piping failure mode (Figure 3.19). Figure 3.19a & Figure 3.19b clearly reveal that, for the same reservoir level (i.e., 48 m), there are two differentiated sets of hydrograph patterns that are associated with the different methods applied. The first set of hydrographs has the highest amplitudes with higher discharge peaks that resulted in less times to peaks as well as the lower base times. The hydrographs of this first pattern were calculated by using Von Thun & Gillette, USBR, and Froehlich approaches. Xu & Zhang and MacDonald & Monopolis prediction approaches established the second set of hydrographs which are characterized by relatively longer times to peak discharges (and longer base times) and approximately halved peak values with respect to the former group values. This pattern

remains the same for both overtopping and piping failure types. Besides, while Xu & Zhang approach provides a higher peak value (e.g., +22.5 %) in the overtopping failure mode than MacDonald & Monopolis method, its peak attenuates to become nearly at the same level with MacDonald & Monopolis method for the piping mode. Secondly, the effect of failure mode was also investigated by varying the initial water level for the upstream reservoir (e.g., 40 m, 42 m, 44 m, 46 m, and 48 m).

Figure 3.20a to Figure 3.20c show that a very minimal change was observed when varying the initial water level for the overtopping failure mode, where, the breach trigger at elevation 48.0 m. Figure 3.20d to Figure 3.20f illustrate the flood hydrographs that were generated using piping failure modes (for piping to start at the half of the water height in the reservoir above the reservoir bottom elevation (Z_b) of 18 m; i.e., $Z_b + h_w/2$) against the initial water level (I.W.L.) assumption of 38.0 m. (e.g., the piping breach elevation of 33.0 m is computed for the breach trigger consideration at the reservoir water level of 48.0 m that corresponds to 30.0 m reservoir water depth (h_w) above Z_b). It is also apparent from the graphs of the piping mode that the time quantity to peak evolved gradually for the Frohlich hydrographs for varying initial water levels associated with corresponding piping start levels, while in the latter two, the changing input variables for inducing the piping did not make significant effect on the peak arrival time.

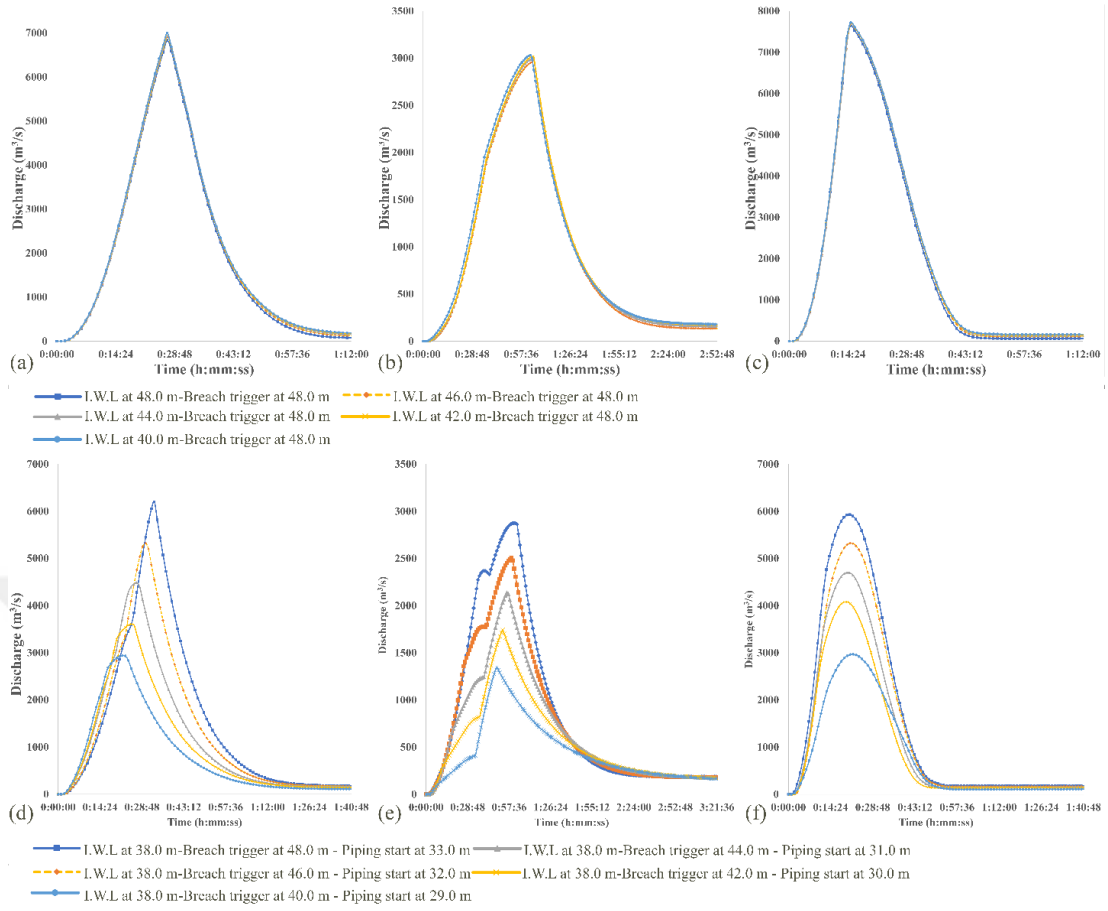


Figure 3.20 Flood hydrographs for different initial water levels (I.W.L.) using (a) Froehlich approach, (b) MacDonald & Monopolis approach, (c) Von Thun & Gillette (Eq. 12) approach for overtopping failure modes, and (d) Froehlich approach, (e) MacDonald & Monopolis

3.9.1.3 Sensitivity Analysis

With the aim to investigate which parameter of dam breach has more influence on the result, a sensitivity analysis was applied by using the (Froehlich, 2008) approach with the overtopping failure type and dam maximum operating water elevation at 48 m a.s.l. To determine the controlling parameter, the breach formation time (t_f), the average breach width (B_{avg}), and side slope (Z_b) were increased by a constant steps of 25 %, 50 %, 75 % and 100 %, respectively, and then reduced by the same scale. The resulting flood hydrographs directly at the downstream side of the dam were estimated for each individual case. The sensitivity analysis for B_{avg} clearly shows that an increase of breach width by a constant percentage tends to increase the maximum discharge value and at the same time decrease T_p and vice versa. As Figure 3.21 reveals, doubling

the value of B_{avg} leads to an increase in the maximum discharge value by 34 % while dropping the time to peak up to 50 %.

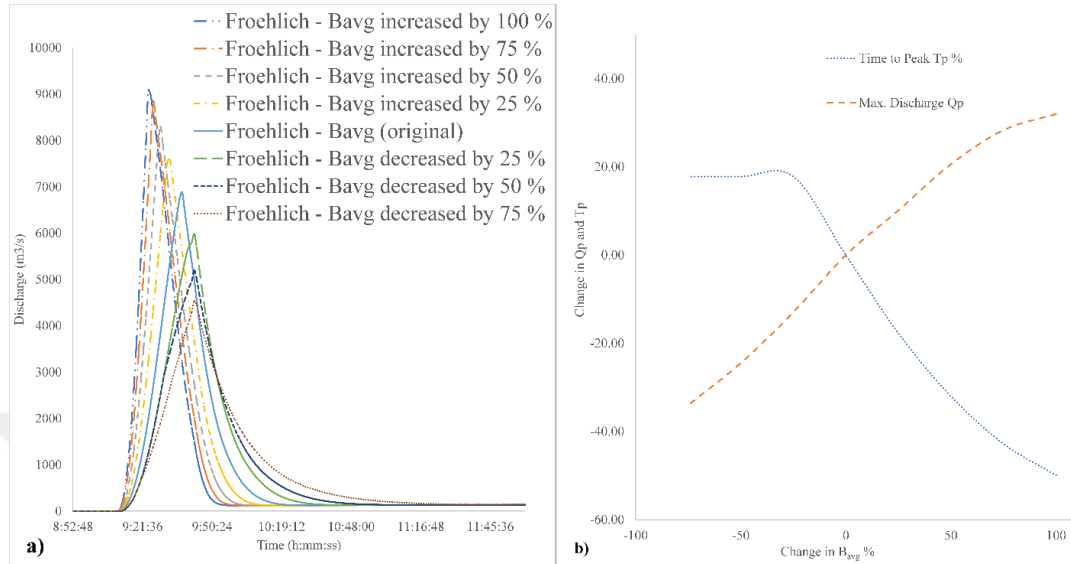


Figure 3.21 Flood hydrographs for varying B_{avg} values at the dam site, and b) percent change in Q_p and T_p with B_{avg}

Apparently, no remarkable variation was distinguished in time to peak and a minor difference in peak discharge value with respect to the corresponding changes in slope of breach side. Indeed, decreasing the breach side slope by 75 % led to change by 0.0 % and -15 % in time to peak and peak discharge, respectively (Figure 3.22).

The results obtained by varying the time of breach formation - with standardization of all other parameters (e.g., average breach width (B_{avg}) and side slope (Z_b) of the breach cross-section) - suggest that maximum discharge value, Q_p and the time of peak discharge, T_p are significantly sensitive to the change in time for the breach to fully develop.

As

Table 3.10, and Figure 3.23 suggest, increasing the breach formation time by 50 %, from 0.55 hr to 0.83 hr, led to a decrease in Q_p by 23.18 % and an increase in T_p by 46.44 %. Nevertheless, declining breach formation time by 50 %, from 0.55 hr to 0.14 hr, led to an increase in Q_p by 32 % while T_p was halved.

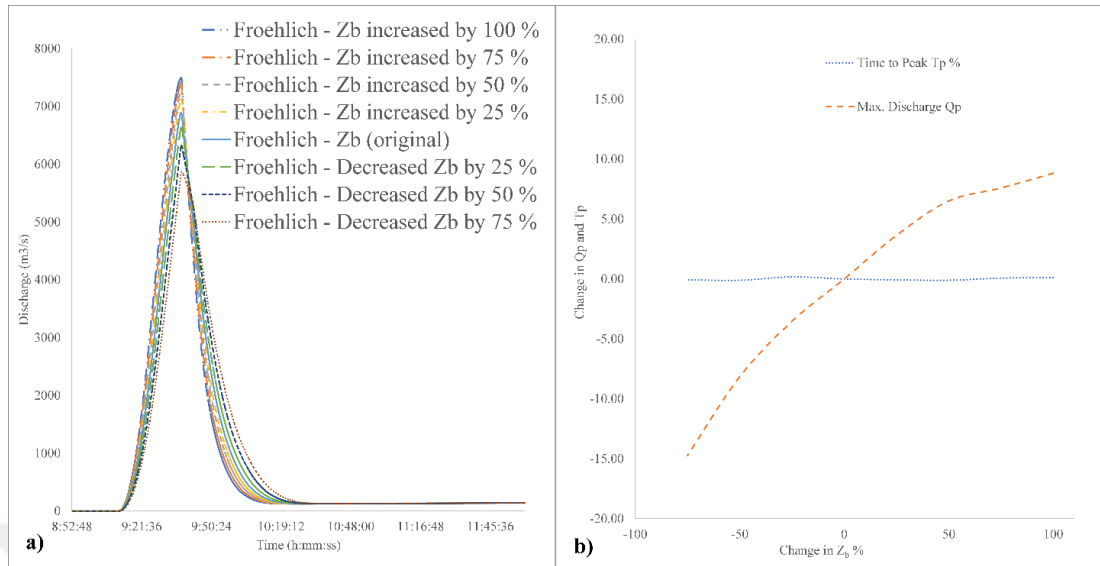


Figure 3.22 a) Flood hydrographs for varying Z_b values at the dam site, and b) percent change in Q_p and T_p against varying Z_b .

Table 3.10 The maximum discharge and time to the maximum values against varying breach formation times

t_f (hr)	t_f dif. (%)	Q_p ($10^3 m^3/s$)	Q_p dif. (%)	T_p (min)	T_p dif (%)	Note
1.10	$t_f + 100\%$	4.17	-39.33	53.07	88.86	Increased t_f
0.96	$t_f + 75\%$	4.66	-32.24	47.15	67.79	
0.83	$t_f + 50\%$	5.29	-23.18	41.15	46.44	
0.69	$t_f + 25\%$	6.04	-12.25	35.08	24.84	
0.55	$t_f + 0\%$	6.88	0.00	28.10	0.00	Original t_f
0.41	$t_f - 25\%$	7.87	14.32	21.13	-24.80	Decreased t_f
0.28	$t_f - 50\%$	9.08	31.97	14.08	-49.89	
0.14	$t_f - 75\%$	10.77	56.57	8.07	-71.28	

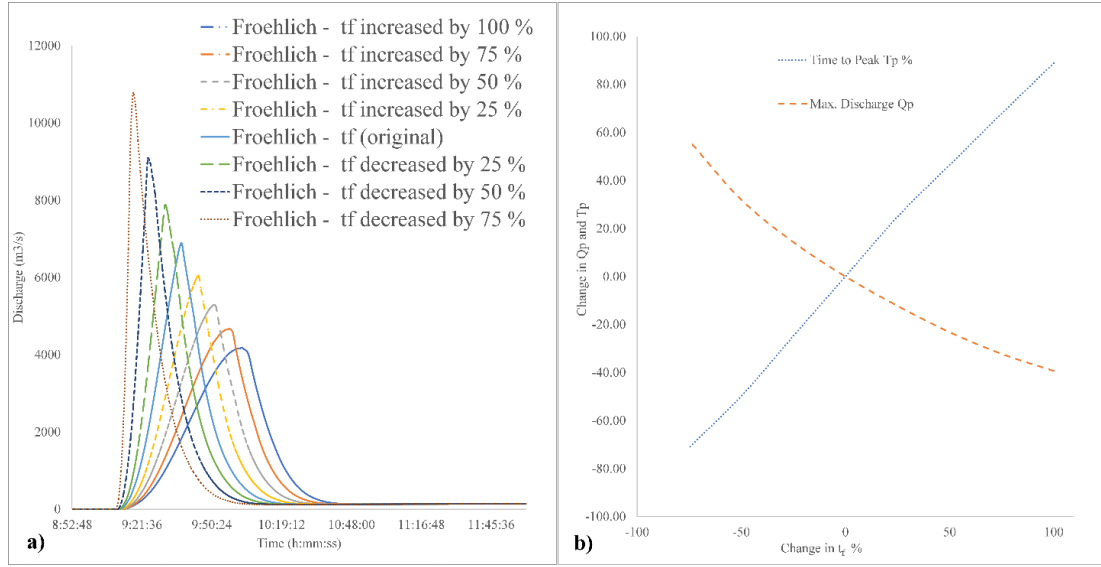


Figure 3.23 a) Flood hydrographs for varying t_f at the dam site, and b) percent change in Q_p and T_p with respect to t_f .

The results from the sensitivity analysis in the current study help reveal that the breach side slope has no notable impact on neither the peak discharge arrival time nor the peak discharge value itself. Nevertheless, the outflow hydrograph is under the significant influence of the breach formation width and is extremely sensitive to the breach formation time. These findings that resulted from the disconnected study on a site with varied dam characteristics and topography features as such proved to be in a reconfirmed agreement with the previous studies on the same subject in support of the sensitivity aspects of the aforementioned breach parameters (e.g., (Basheer et al., 2017; Bozkuş, 2004; Bozkuş & Güner, 2001; Petrascheck & Sydler, 1984; Wahl, 1998)).

Based on comparisons with an observational judgment among all the methods employed in the study, the USBR and Von Thun & Gillette approaches estimated the highest peaks and the lowest times to corresponding peaks, Froehlich (2008) method provided moderate outputs for the both quantities (peak discharge and peak time) while the lowest peaks against delayed peak arrival times were estimated through the MacDonald & Langridge-Monopolis and Xu & Zhang equations. In general, the piping failure modes that were simulated in all empirical approaches tend to give lower peak flood discharges in comparison to the overtopping failure mode as expected.

Therefore, Froehlich (2008) set of empirical equations were used to estimate the breach parameters, as breach geometry based on what was explained above provides reasonable outputs for both quantities (peak discharge and peak time) with regards to the other empirical methods (Najar & Gül, 2021). The breach parameters of Tahtalı dam were estimated as 137 m, 2.04 h and 0.7 for B_b , t_b , and Z_p , whereas, in Ürkmez case, the same set of geometries were set equal to 30 m, 0.57 h and 0.7, respectively (Figure 3.24). A radial spillway gate was defined in the hydraulic model for the Tahtalı dam. The geometry of the gate was obtained from the source of the spillway gate producer (Rona, 1998). Figure 3.25 shows the visualization and HEC-RAS definition of the spillway.

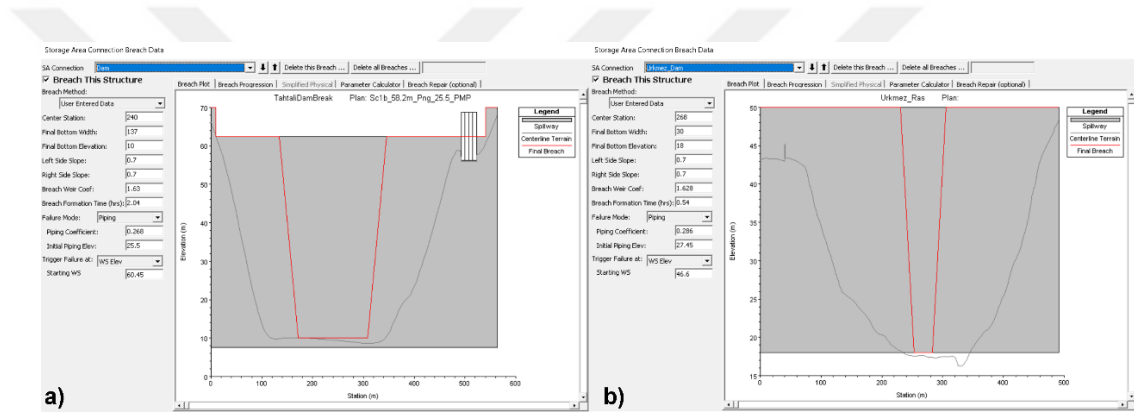


Figure 3.24 Breach geometry definitions for a) Tahtalı, and b) Ürkmez dams.

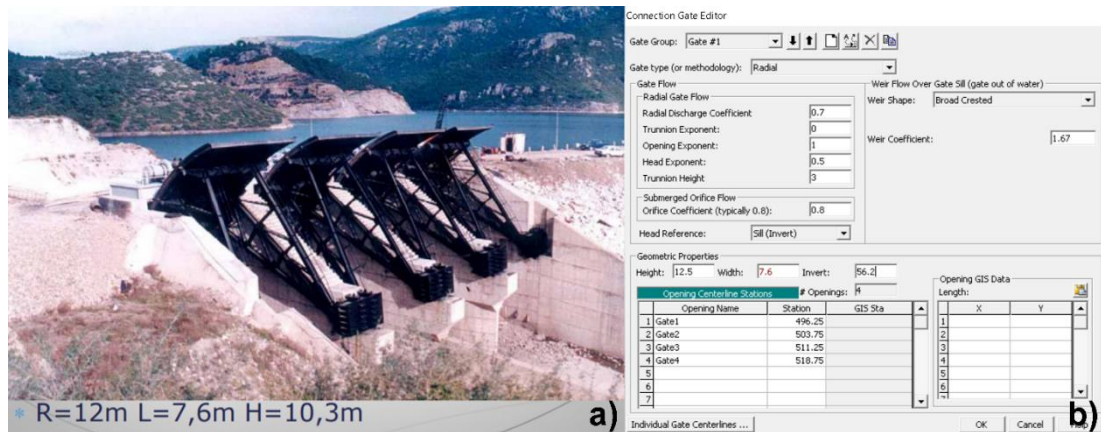


Figure 3.25 Tahtalı Spillway gate a) visualization (Rona, 1998) and b) HEC-RAS definition

3.9.2 Dam Break Failure Scenarios

The Saint-Venant full-momentum set of equations were basically employed in generating flood hydrograph resulting from dam breach simulation under each scenario definition. The built-up area representation within the 2D hydrodynamic model considers one of the most critical factors in flood inundation modeling in built-up areas (Bellos & Tsakiris, 2015; Haltas, Elçi, et al., 2016; Papaioannou et al., 2018). In this respect, the topography was mapped based on the precise and updated Digital Terrain Model (DTM) that accurately represents the dam site downstream by adapting the building volumes and other cadastral features onto available DEM (Figure 3.26).

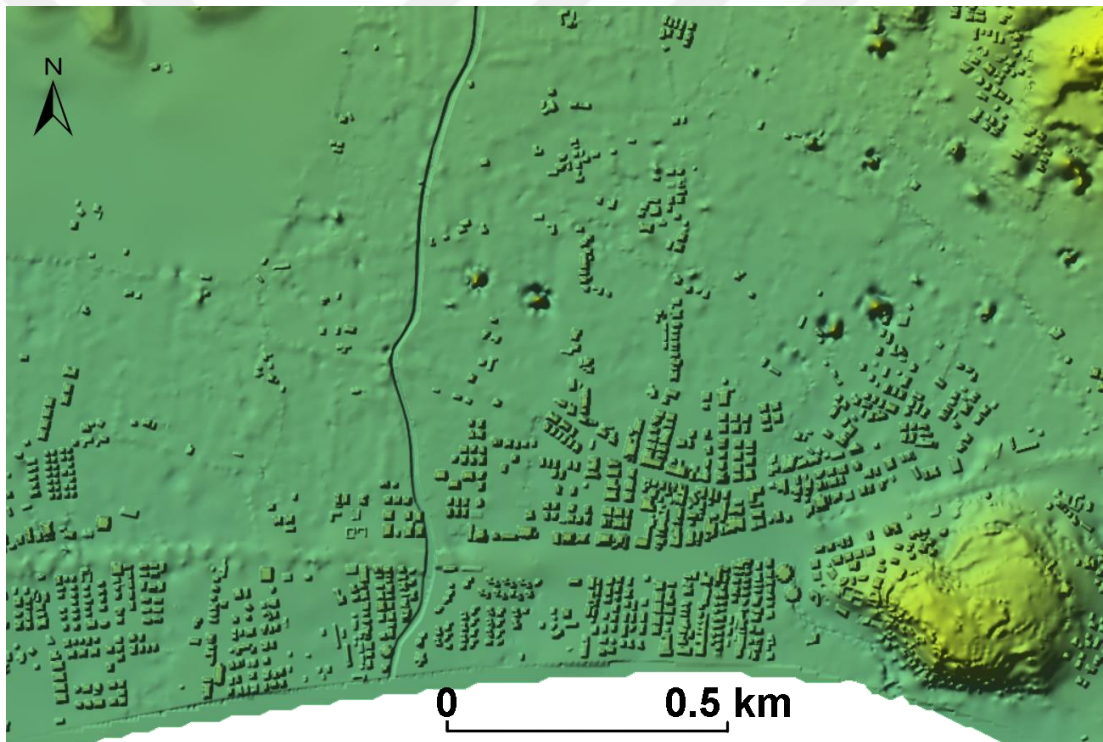


Figure 3.26 Digital Surface Model

An overtopping failure mode of dam break model was run for each return period for both Tahtalı and Ürkmez dams to investigate the adequacy of spillway capacity for both reference and projected climate periods. Moreover, five scenarios - using different failure-start water levels (the normal, minimum water levels for both dams and the mid-level of the active volume), and corresponding piping start elevations were set with assumptions based on different pressure distributions on the dam body) - were

chosen to simulate the evolution of a piping-mode dam-break event. Three out of five (i.e., Sc1, Sc2, and Sc3) mainly consider the evolution of dam-break driven flood wave propagation by altering the reservoir's initial water level at the time of breach initiation. Scenarios Sc1, Sc1a and Sc1b, on the other hand, were designed in such a way that the PMF impact of the downstream flood wave behavior would be investigated.

Table 3.11 tabulates the definition of the employed breach scenarios and their associated initial water and breach starting levels. The table also includes peak flood discharge, and total inundation area, which were calculated in the 2D HEC-RAS model based on each given set of parameters.



Table 3.11 Dam Breach Scenario definitions used for Tahtalı and Ürkmez cases

Dam Name	Scenario	Reservoir water-level (m)	Piping Breach Elevation (m)	Finding		Entering Flood Wave	Explanation
				Total Inundation area (km ²)	Peak outflow Discharge (10 ³ m ³ /s)		
Tahtalı	Sc1	60.5	25.5	8.40	579.66	--	Maximum water Level
	Sc 1a	60.5	25.5	8.40	579.10	PMF	Maximum water level and the PMF at Peak value
	Sc 1b	60.5	25.5	8.42	587.89	PMF	Maximum water level and the PMF at inflection point
	Sc 2	55.5	23.85	8.00	379.24	--	Normal water Level
	Sc 3	43.25	19.35	7.32	154.52	--	Active volume mid-level
Ürkmez	Sc 1	46.63	27.45	3.24	2.93	--	Maximum water Level
	Sc 1a	46.63	27.45	3.24	3.65	PMF	Maximum water level and the PMF at Peak value
	Sc 1b	46.63	27.45	3.26	3.65	PMF	Maximum water level and the PMF at inflection point
	Sc 2	43.5	25.15	3.08	2.36	--	Normal water Level
	Sc 3	33.45	21.8	2.35	0.6	--	Active volume mid-level

CHAPTER 4

RESULTS AND DISCUSSIONS

This chapter presents the results obtained from each component of the proposed framework. With the intention to understand the climate change impact on both hydrological and hydraulic models, the findings of the reference (1970-2004) and the near-future projected (2025-2050) periods are presented in order.

4.1 Bias Treatment in Meteorological Inputs

As previously mentioned in the methodology description, a two-phase bias correction was implemented on meteorological data series for resolving the bias issue. This entire process was applied in a grid-based manner. The first component helped eliminate the bias quantities in time series values in a computational effort by comparing the monthly long-term means of the observed and modeled time series and then applying the computed delta change factors to the sub-daily values. Figure 4.1 shows the monthly total precipitation over Ürkmez and Tahtalı basins for the actual observed precipitations, row precipitation time series from climate model (pre-bias correction) and treated rainfall data (post-bias correction (phase-one)). Table 4.1 and Table 4.2 illustrate the model data's statistics pre-and post-applying the bias correction for reference-based rainfall and temperature data.

Table 4.1 Model data's statistics pre-and post-applying the bias correction for rainfall variable for the reference period

Reference Period 1970 - 2004	Observed-rainfall data (mm/month)	Model-Rainfall Pre-Bias Correction (mm/month)	Model-Rainfall Post-Bias Correction (mm/month)	Rainfall Monthly Correction factors (mm/3hr) ($\Delta\mu_{p,i}$)
Jan	114.68	73.16	110.82	0.228
Feb	94.64	46.86	92.13	0.306

Table 4.1 continues

Reference Period 1970 - 2004	Observed- rainfall data (mm/month)	Model-Rainfall Pre- Bias Correction (mm/month)	Model-Rainfall Post- Bias Correction (mm/month)	Rainfall Monthly Correction factors (mm/3hr) ($\Delta\mu_{p,i}$)
Mar	77.66	42.05	75.58	0.219
Apr	48.98	29.68	49.39	0.135
May	26.19	25.52	27.09	0.006
Jun	6.94	14.91	12.32	-0.098
Jul	2.32	5.26	4.41	-0.066
Aug	2.17	6.88	5.52	-0.094
Sep	13.40	15.58	15.12	-0.031
Oct	45.67	32.64	43.75	0.110
Nov	99.66	67.09	97.70	0.203
Dec	139.24	80.11	131.21	0.319

Table 4.1 clearly shows that summer months' modeled rainfall data (i.e., June, July, August, and September) overestimated the precipitation compared to observations. At the same time, the remaining months experienced an underestimation in the same regard. February and December are the most biased months with a bias correction factor of 0.318 and 0.298 mm/3hr, respectively. Contrarily, May and September are the most accurate with a correction factor of 0.011 and -0.026 mm/3hr, respectively. On the other hand, all months in the modeled temperature data showed an underestimation, particularly from May to September, which encountered the highest bias correction factors with an average nearly between 3 and 5 degrees as Table 4.2 and Figure 4.2b help indicate.

Table 4.2 Model data's statistics pre-and post-applying the bias correction for temperature variable for the reference period

Reference Period 1970 - 2004	Observed- Temperature data (Degree-C)	Model- Temperature Pre-Bias Correction (Degree-C)	Model- Temperature Post Bias Correction (Degree-C)	Temperature Monthly Correction factors (Degree-C) ($\Delta\mu_{t,i}$)
Jan	8.18	6.93	8.10	1.25
Feb	8.63	7.35	8.89	1.29
Mar	11.02	9.41	11.02	1.62
Apr	14.97	13.22	14.95	1.77
May	19.85	16.26	19.69	3.62
Jun	24.68	19.91	24.98	4.8
Jul	27.21	22.52	26.90	4.72
Aug	26.76	22.85	26.35	3.94
Sep	22.93	20.04	22.59	2.92
Oct	18.17	15.74	18.09	2.45
Nov	13.08	10.83	13.12	2.26
Dec	9.57	8.15	9.69	1.43

Figure 4.2 displays pre- and post-bias corrected precipitation variable onto which the adjustment factors were applied on a sub-daily resolution for the reference and projected periods.

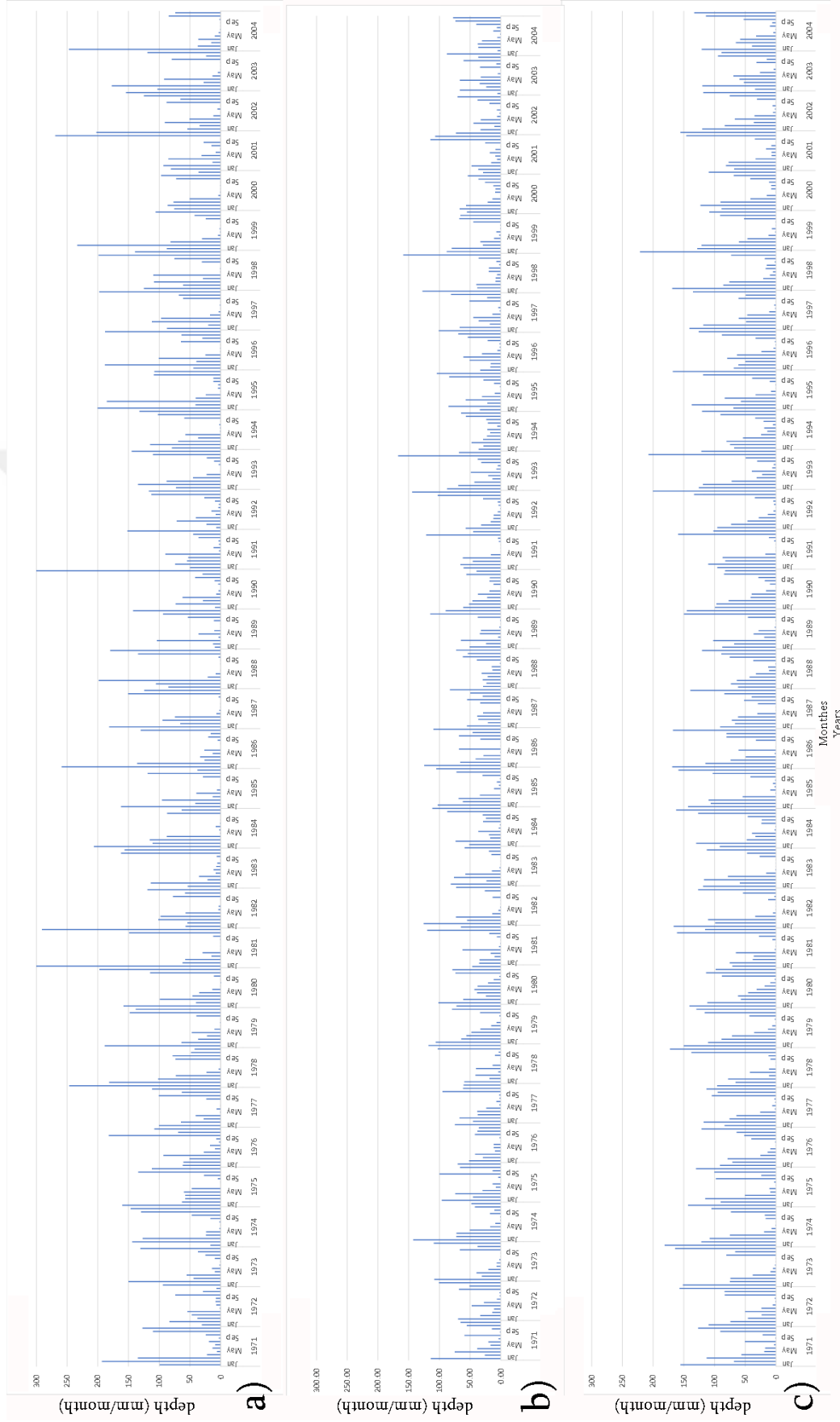


Figure 4.1: Monthly Total precipitation of a) observed, b) pre-bias correction, c) post-bias correctio (phase one) time series

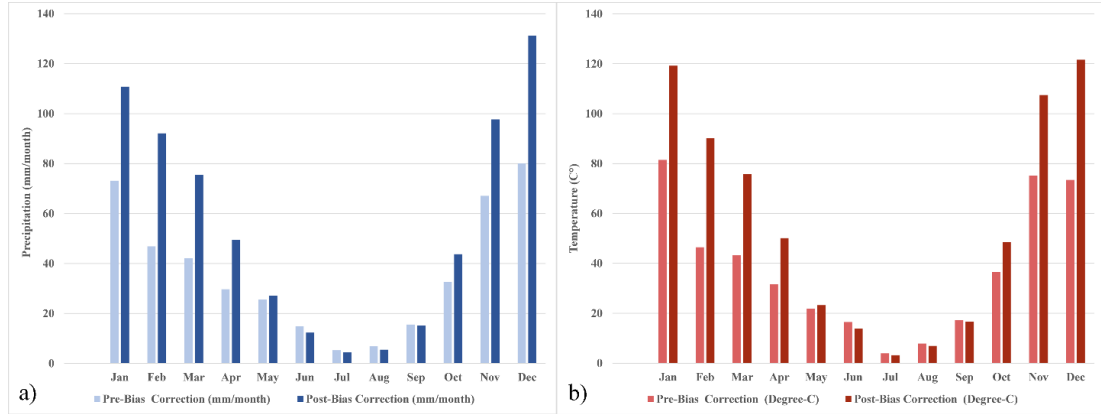


Figure 4.2 Rainfall model data statistics pre-and post-bias correction for a) reference (1970-2005) and b) projected (2025-2050) periods

The secondary bias correction was applied on only the precipitation data after extracting the annual maxima of the considered standard durations. This additional bias treatment was implemented by taking count of the clustered patterns of the average differences for the grid data points in the entire study-area and individually for all the standard durations considered. Figure 4.3 shows the spatial distribution of the secondary bias correction factors displayed in clustered variation patterns.

The figure indicates for the short standard periods (AM3), and the (AM6) middle portion of the study area always underestimates the extreme annual events. Simultaneously, the longer standard durations (i.e., AM12, AM18, and AM24) encountered an overestimation for the same part - the middle part - of the studied area. The analysis also informs that the upper part of the Tahtalı basin constantly exposes the biggest bias up to 20 mm as an underestimation.

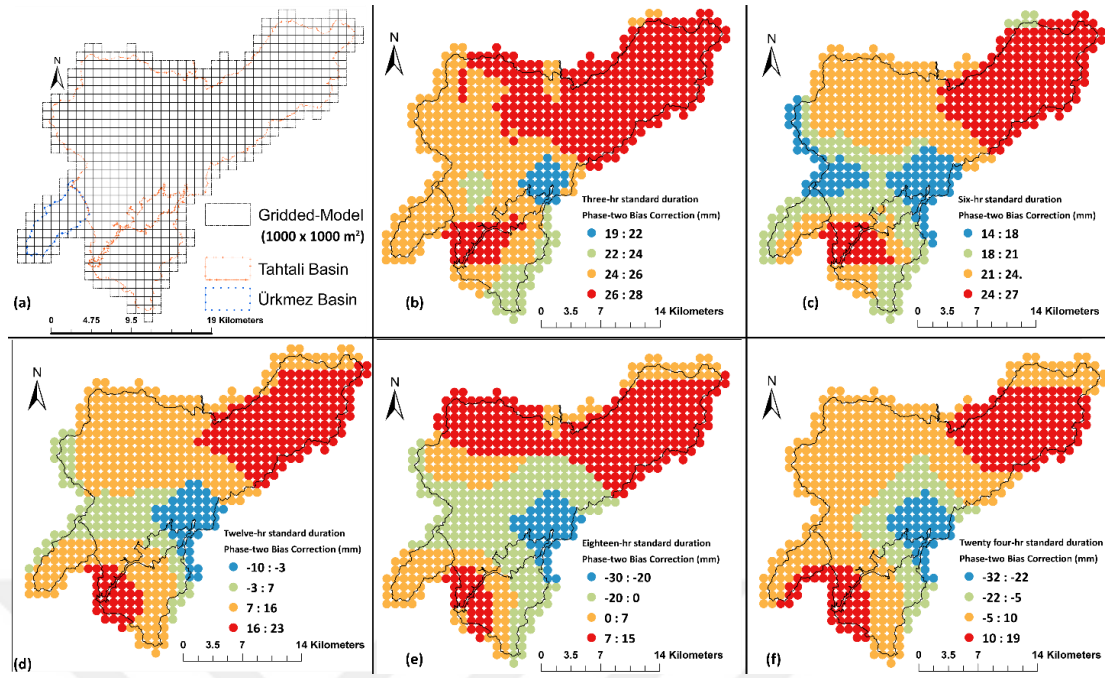


Figure 4.3 a) The distributed grid model of the study area showing the geographic patterns of average differences between observed and modeled extreme precipitation values for (b) 3-hr, (c) 6-hr, (d) 12-hr, (e) 18-hr, and (f) 24-hr standard durations.

4.2 Developing Geomorphological Instantaneous Unit Hydrograph

4.2.1 Geomorphological Characteristics of the Catchment

As part of the efforts for developing the synthetic hydrographs, a set of geomorphological basin characteristics were computed from the topography models for both dam catchments to the final end of obtaining the GIUHs for the modelling units. Here, due to the varying sizes of the two case-studies, Tahtalı catchment was considered in six subbasins (see Figure 3.14) flowing into the dam reservoir while in the case of Ürkmez the comparably smaller dam catchment was taken as a single part in deriving the necessary morphological catchment characteristics. Figure 4.4 shows the stream orders required for the computation of the RB, RL, and RA parameters for each subbasin. Table 4.3 summarizes all necessary parameters that were collectively used in generating GIUHs in all sub-/catchment

Table 4.3 The watersheds' morphological characteristics

Name	A (km ²)	L _Ω (km)	v (m/s)	R _A	R _B	R _L	n*	k (h)*	n**	k (h)**	Time of concentration (h)
subbasin-1	272.95	15.27	1.66	0.90	2.10	1.09	6.42	1.22	6.48	1.14	7.75
subbasin-2	120.37	13.87	1.75	0.89	4.68	1.04	12.08	0.48	14.19	0.65	4.60
subbasin-3	27.62	7.90	1.89	0.93	3.56	1.22	9.49	0.30	10.21	0.39	1.75
subbasin-4	25.65	5.96	1.85	0.82	1.85	0.92	6.14	0.52	6.03	0.45	1.70
subbasin-5	11.90	4.02	1.91	0.87	2.15	1.31	6.78	0.22	6.83	0.23	1.85
subbasin-6	7.09	2.79	1.94	1.07	2.47	1.30	6.42	0.16	6.48	0.17	1.25
Ürkmez basin	30.55	7.34	1.86	0.94	2.23	1.09	6.51	0.51	6.60	0.48	2.15

* Using Eqs. (3.17 & (3.18 (Croley, 1979), ** Using Newton-Raphson optimization method (Eqs. (3.16 & (3.17)

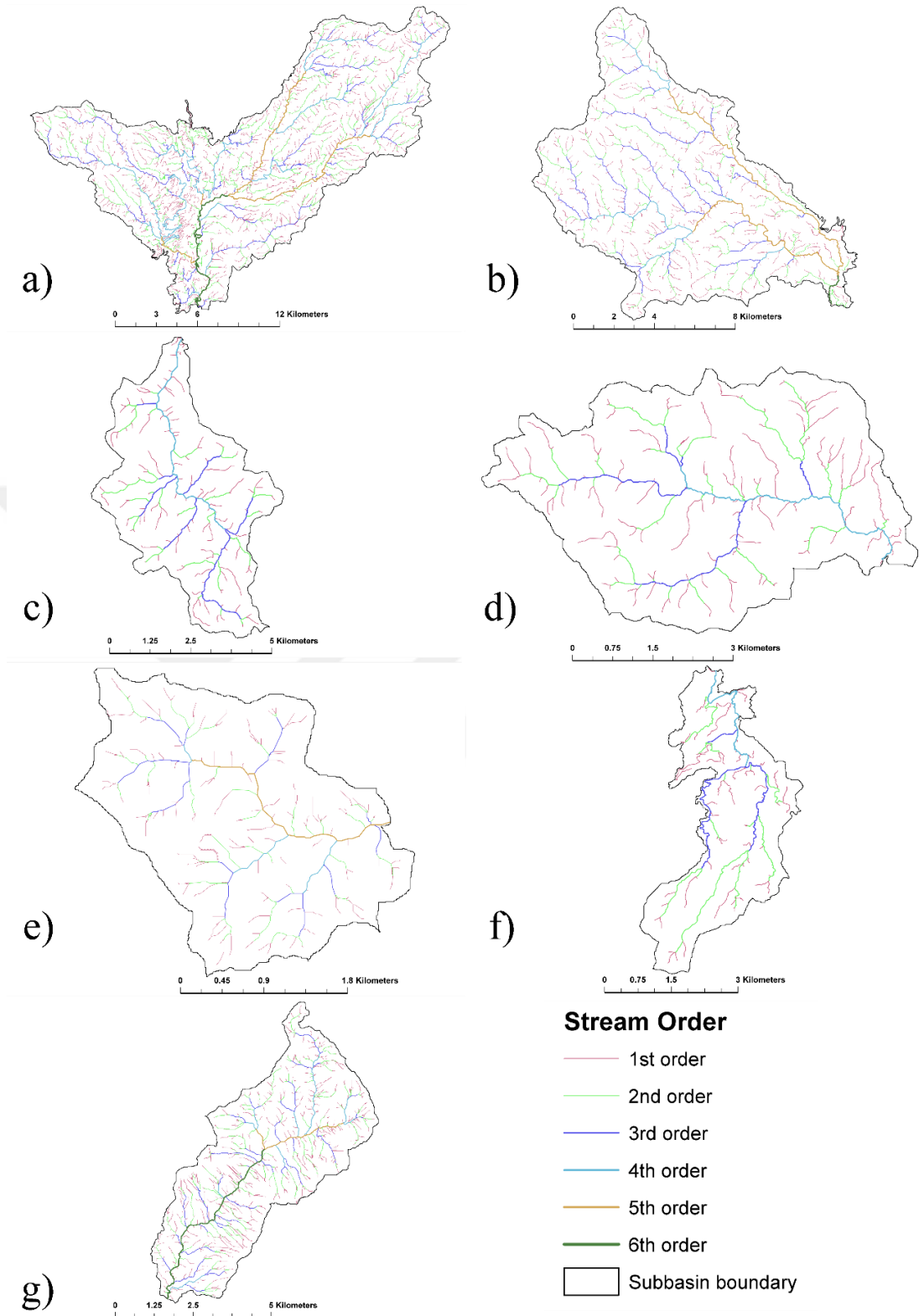


Figure 4.4 Stream order definitions for Tahtalı b) subbasin-1, c) subbasin-2, d) subbasin-3, e) subbasin-4, f) subbasin-5, g) subbasin-6, and h) the Ürkmez basin as a whole

The ordinates of the synthetic instantaneous unit hydrographs (IUH), which represent the direct runoff at the catchment outlet resulting from unit (1 mm) rainfall falling homogeneously over the considered catchment in theoretically zero duration, can be generated by utilizing the Nash model upon computations of the shape and scale factors to complete the IUH shape. The final 1-hour UHs were obtained for each subbasin by shifting the IUH by 1-hour and then averaging the two hydrographs (Gül et al. 2010). Figure 4.5 illustrates the 1-hr unit hydrographs for Tahtalı subbasins. Figure 4.6 shows the 1-hr unit hydrographs for Ürkmez basin.

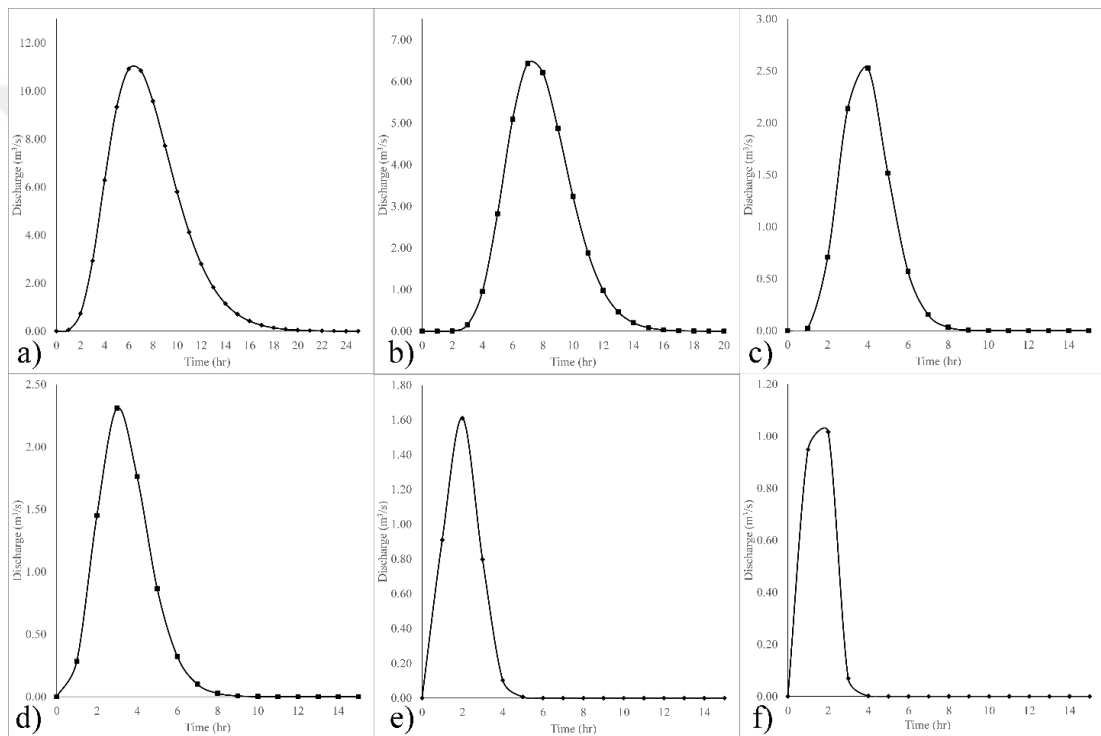


Figure 4.5 One-hour unit hydrograph for Tahtalı a) subbasin-1, b) subbasin-2, c) subbasin-3, d) subbasin-4, e) subbasin-5, and f) subbasin-6

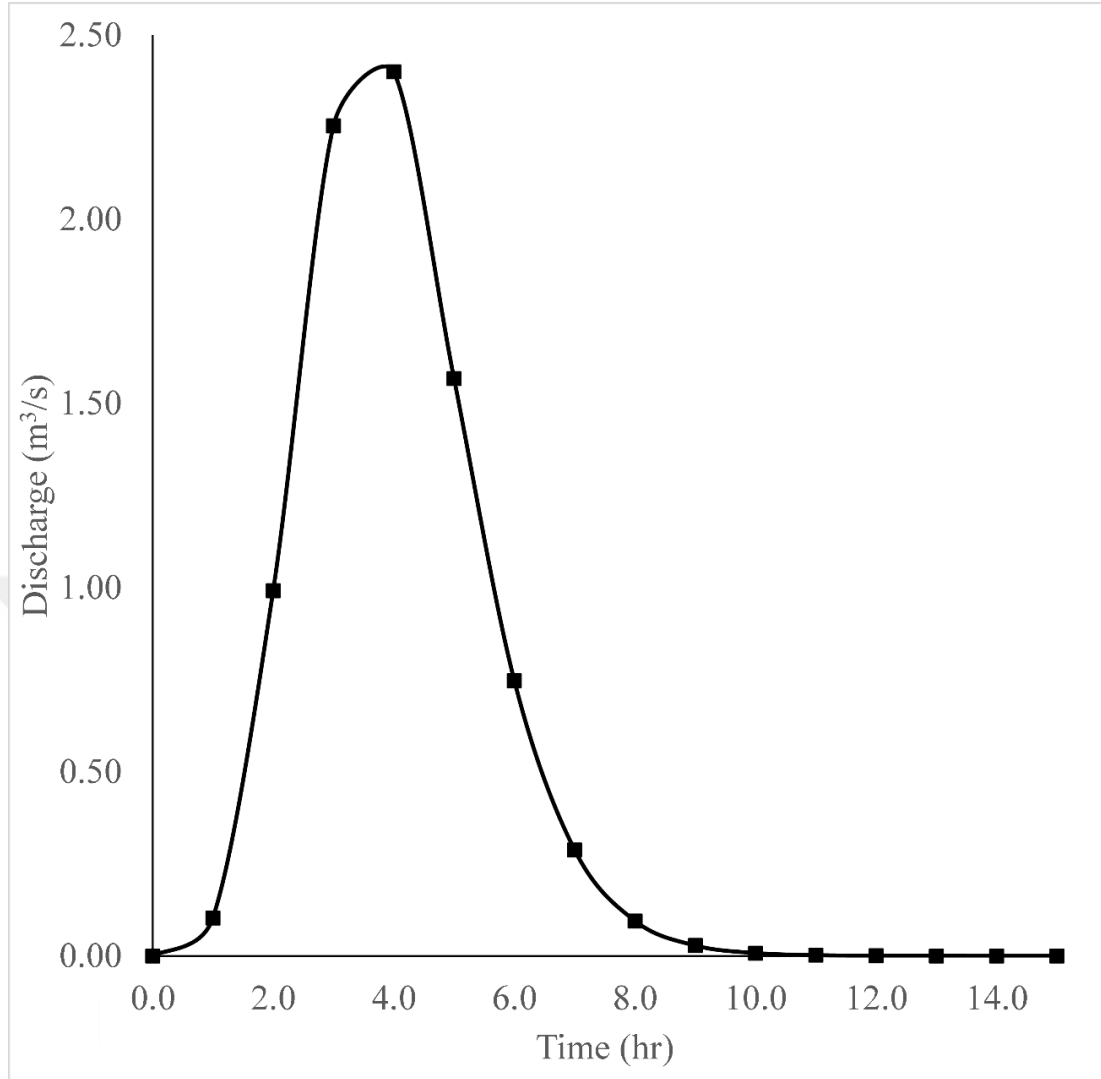


Figure 4.6 One-hour unit hydrograph for Ürkmez basin

4.3 Developing Design Hyetographs

As described in the methodology section, design hyetographs generation task that would be implemented using the instantaneous intensity method required the determination of IDF relationships on a spatial basis.

4.3.1 Estimation of IDF Equation Parameters on a Grid Base

The establishment of IDF relationships require the identification of precipitation depths with durations shorter than 24 hours. In the presented study, AM3, AM6, AM12, AM18, and AM24 data values were aggregated from the bias-corrected time

series through geostatistical analysis over the grid base (see Figure 3.3). Type I - General Extreme Value (GEV) probability distribution, which appeared at the 95 % confidence level with the good fit signs based both on the non-parametric Anderson-Darling and Kolmogorov-Smirnov tests, was fit to the standard duration maximum precipitation series to compute the rainfall intensities that associate to the IDF curve. Figure 4.7 displays the degree of fit between the empirical distribution function (marked with dots) and the cumulative distribution function of the selected GEV distribution (indicated with lines) for different precipitation durations of the annual maximum series for a sample grid cell (see Figure 3.3 for the Grid # 324) arbitrarily selected around the basin center of the gridded representation. Figure 4.8 shows the reference-IDF relationships drawn for 2-year to 500-year recurrence intervals for the sample grid location (Grid # 324) for graphical display purposes, while Figure 4.9 displays the derived IDF curve for the same exact Grid point under the RCP8.5 climate change scenario for the period of 2025-2050.

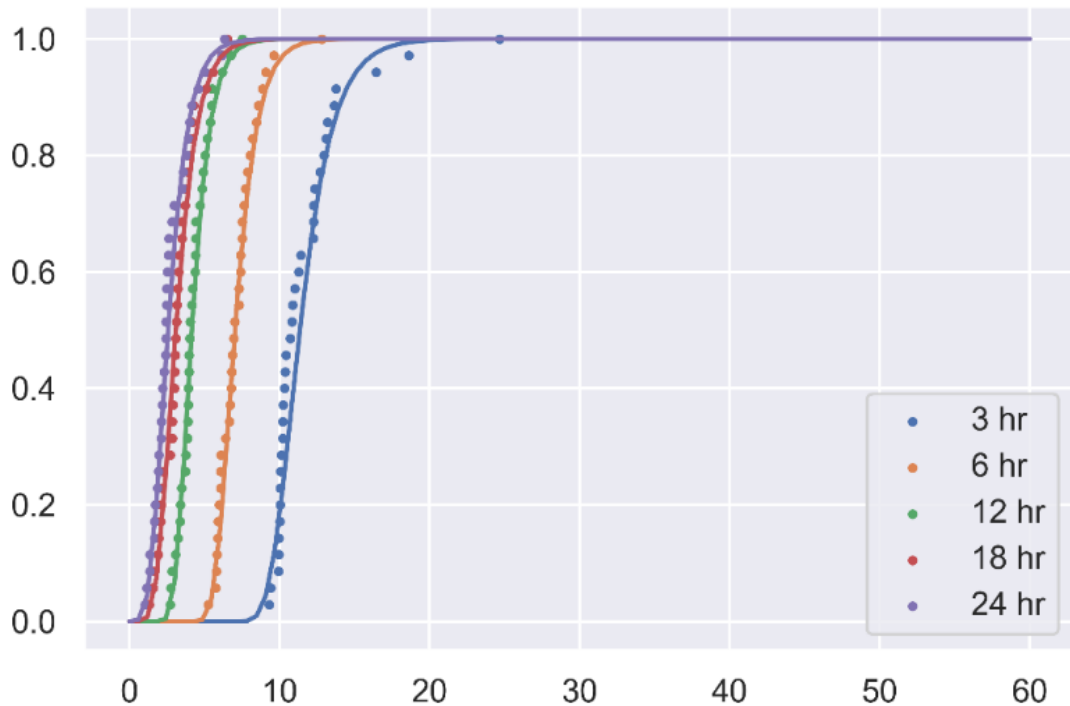


Figure 4.7 Empirical vs. Type I - General Extreme Value cumulative distribution function

In Figure 4.10 and Figure 4.11, the spatial distribution of the reference-, and projected-IDF empirical equation parameters based on the Bernard Equation are

displayed for both Tahtalı and Ürkmez Basins on the structured reference grid of 1 km resolution (see Figure 3.3). These parameter constants were generated by employing an optimization method to all possible standard duration, return period and intensity combinations, allowing the extraction of spatially-distributed design hyetographs across the study catchments.

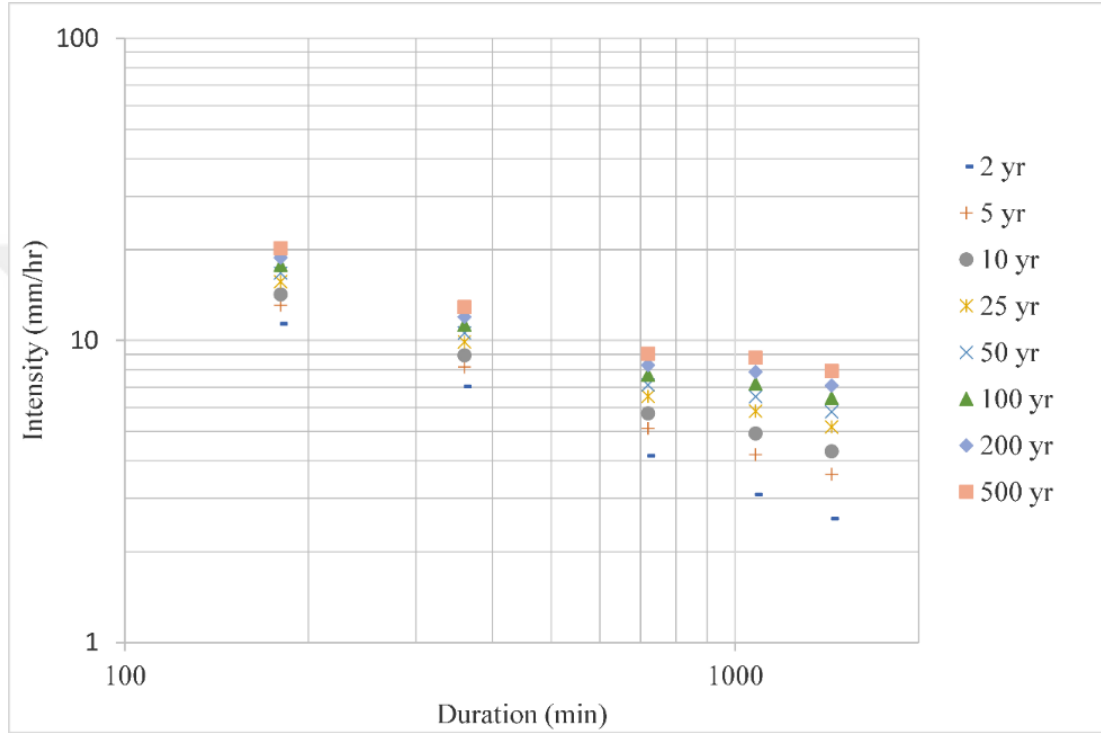


Figure 4.8 IDF relationships obtained for the annual maximum series of the sample grid #324 for the reference (1970-2005) period.

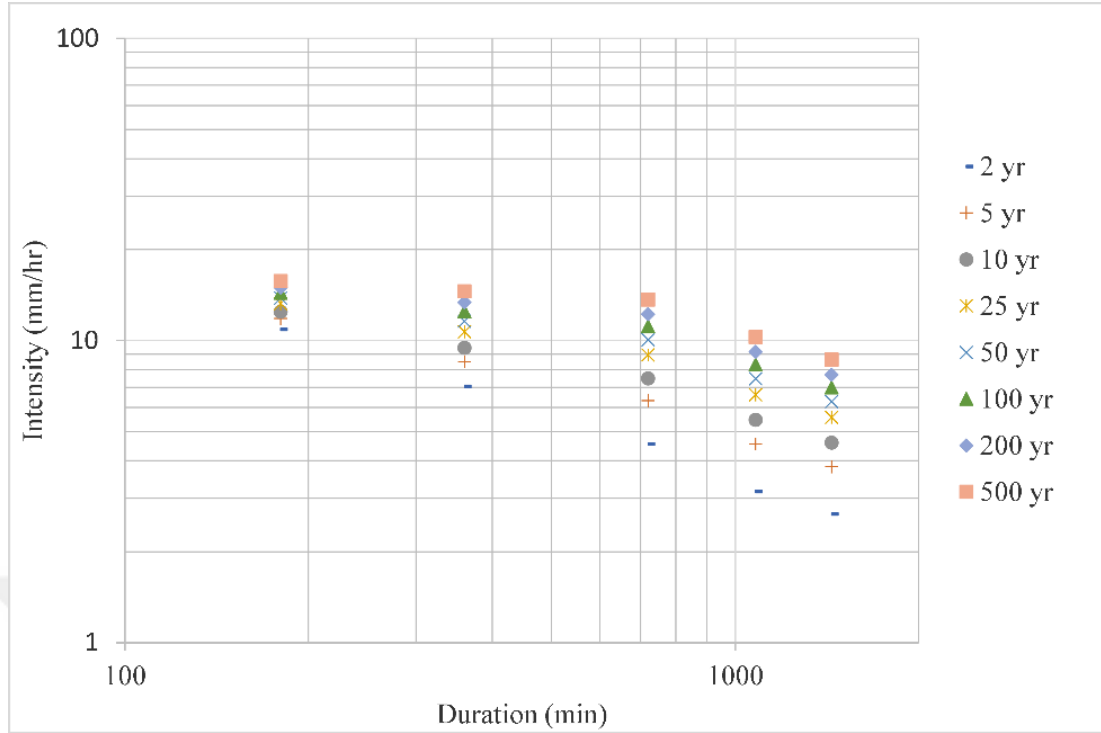


Figure 4.9 IDF relationships obtained for the annual maximum series of the sample grid #324 for the projected (2025-2050) period.

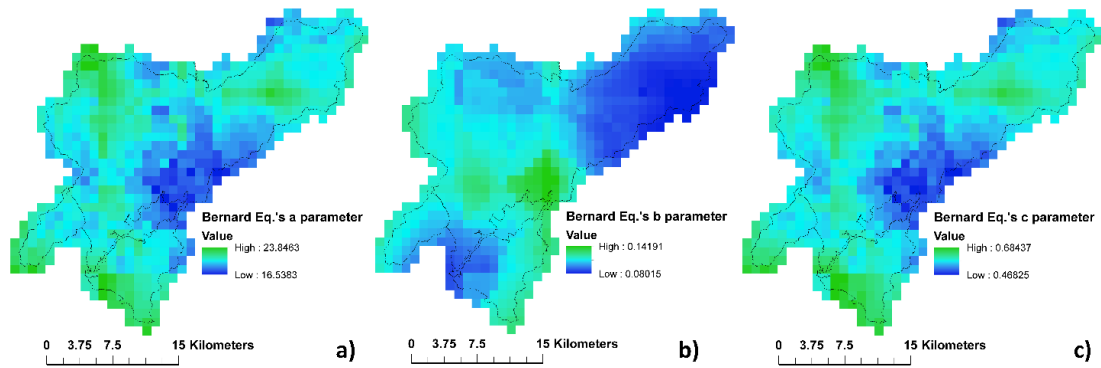


Figure 4.10 Maps of spatially distributed parameters of empirical IDF equation (for reference period) over the Tahtalı & Ürkmez basins: (a) “a” parameter; (b) “b” parameter, and (c) “c” parameter distributions

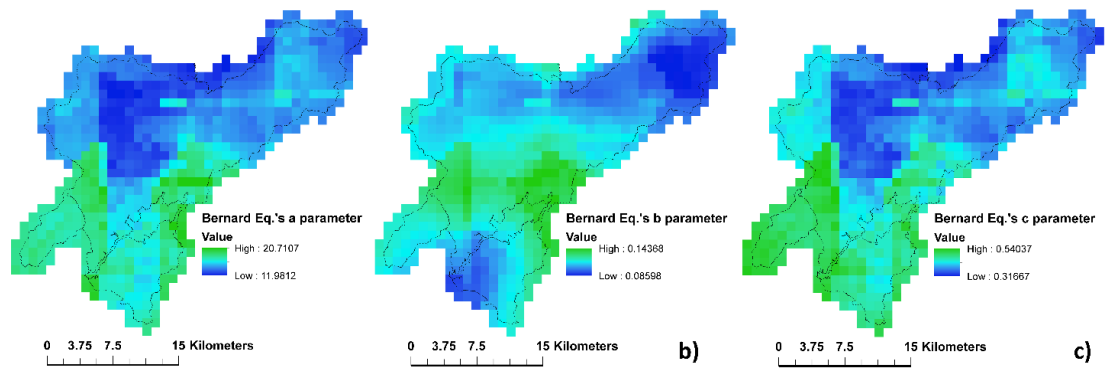


Figure 4.11 Maps of spatially distributed parameters of empirical IDF equation (for projected period) over the Tahtalı & Ürkmez basins: (a) “a” parameter; (b) “b” parameter, and (c) “c” parameter distributions

Since the IDF curve parameters necessary for estimating design storms using instantaneous intensity method have been estimated based on the gridded climate data, the values are necessarily valid for each individual cell location in the entire grid. Moving from the definition of IDF equation parameters on individual grid locations, the 24-h duration design storm hyetographs were formulated by utilizing the previously explained concept of the instantaneous intensity pair curves for $T = 100$ -, 500-, 1000-year intervals, as well as the worst-case scenario for PMP. Figure 4.12 and Figure 4.13 illustrate (for the sample grid) the design hydrographs and the corresponding infiltration index (phi-index) for each return period extracted from climate data for both reference and projected periods.

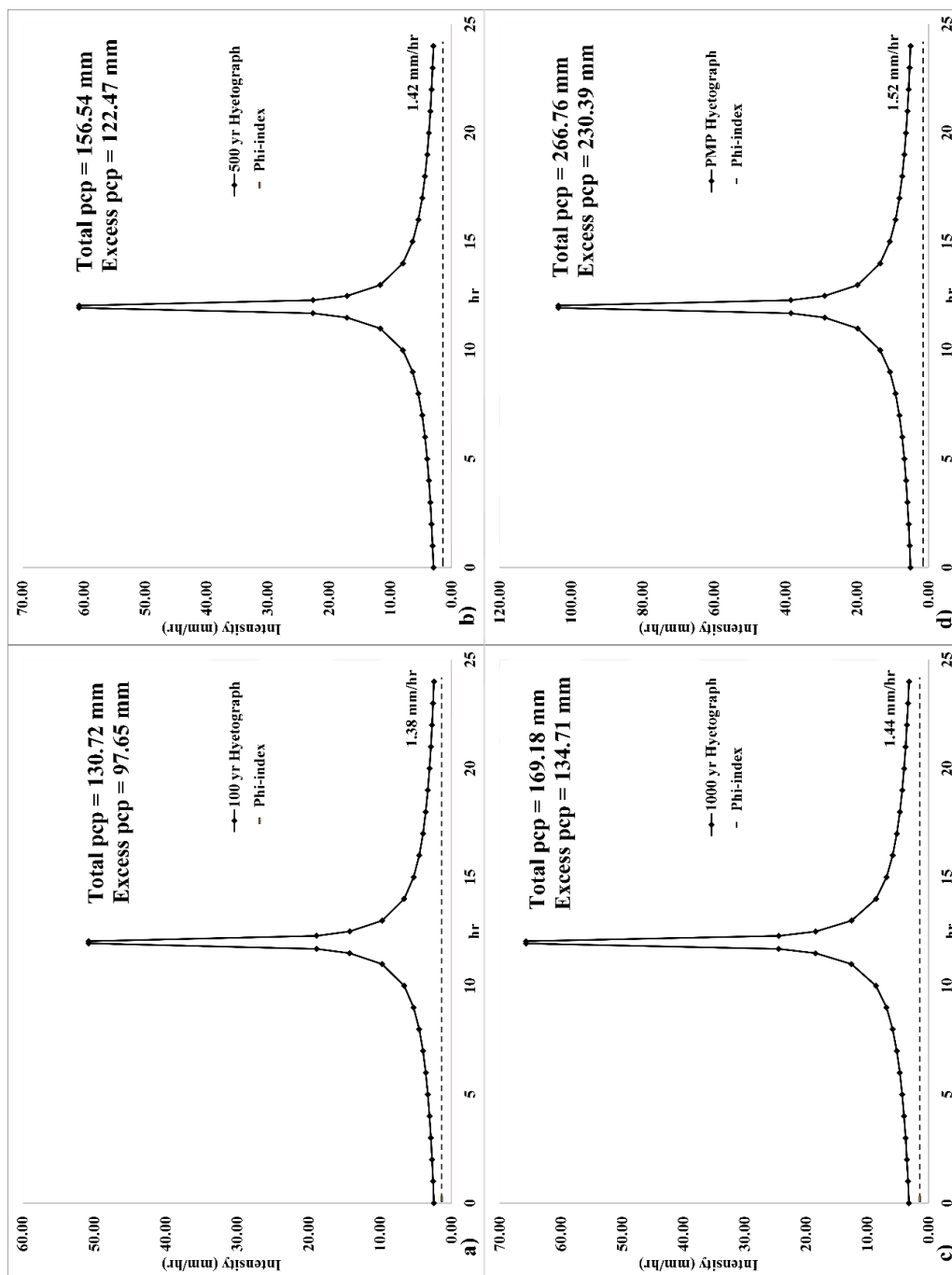


Figure 4.12 Reference period-based 24-hr duration design hyetographs generated using the inst. Intensity method for a) 100-, b) 500-, c) 1000-year return period and d) for PMP for the sample grid #324

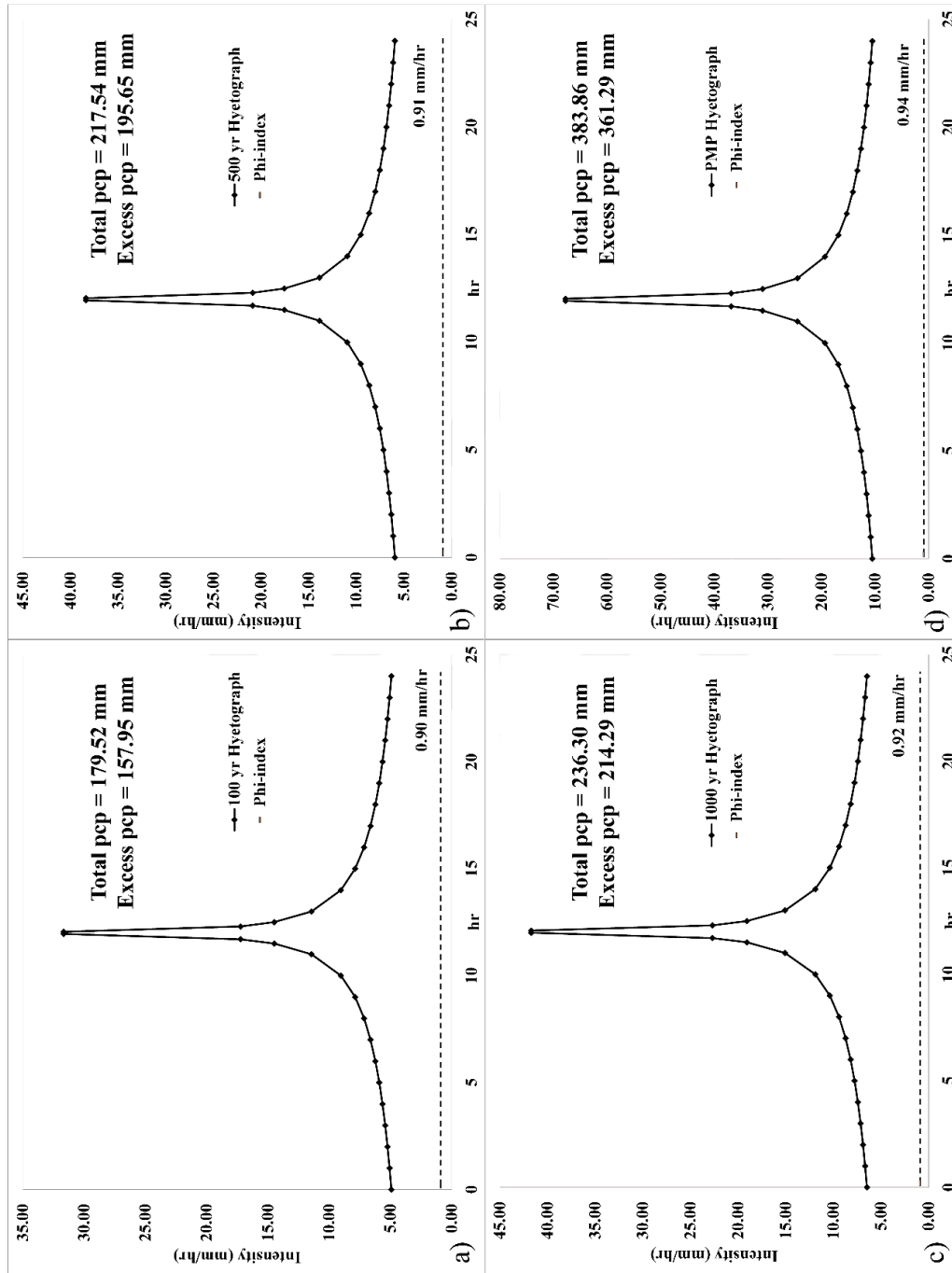


Figure 4.13 Projected period-based 24-hr duration design hyetographs generated using the inst. Intensity method for a) 100-, b) 500-, c) 1000-year return period and d) for PMP for the sample grid #324

4.4 Hydrological Modelling

The hydrological modelling basically investigating the upstream portion of the river catchment to provide inputs to the dam break model. The application of the spatially-distributed hydrologic simulating scheme in the six subbasins of Tahtalı and the entire single catchment of Ürkmez was oriented to the evaluation of flood hydrographs at the outlets used in the downstream part of hydraulic simulation for all investigated occurrence interval. Here, the hydrological model was mainly applied for the biggest three subbasins of Tahtalı while the GIUH outputs were considered for direct use for the remaining three.

For calibrating the hydrologic model set-ups generated for the three main subbasins in Tahtalı and the single basin of Ürkmez, GIUH-based synthetic hydrographs were basically used. One may easily question the exact need for implementing hydrologic modelling when there is always the possibility of estimating the GIUH-based hydrograph at the basins' outlets. The rationale behind is the need toward gaining capacity for distributed hydrologic modelling under changing conditions of the distributed climate scenario inputs (i.e., examining impacts of varied extreme precipitation cases associated to different recurrence periods).

Both the velocity approach concept (Tr-55) (NRCS, 2010) and the harmonic slope method (Kirpich 1940) were used comparably to estimate the time of concentration for each subbasin. Figure 4.14 to Figure 4.17 show the flood hydrographs for Tahtalı subbasin-1, -2, -3, and Ürkmez basin for different recurrence intervals for the reference (1970-2005) and the near-future projected (2025-2050) periods. Figure 4.18 shows the superposition (combined) inflow hydrograph generated from Tahtalı six subbasins after routing in the upstream reservoir. As Figure 4.14 to Figure 4.18 clearly show, climate change (in the near future, based on the dynamically downscaled MPI RCP 8.5 climate model) significantly influences the hydrological cycle.

Table 4.4 illustrates the climate change impacts by quantifying the variations induced to the flood hydrographs between the two studied periods in terms of flood

peak and floodwater volume quantities. Table 4.5 tabulated the calibration parameters of the hydrological models as well as fitting accuracy in the term of Nash–Sutcliffe value.



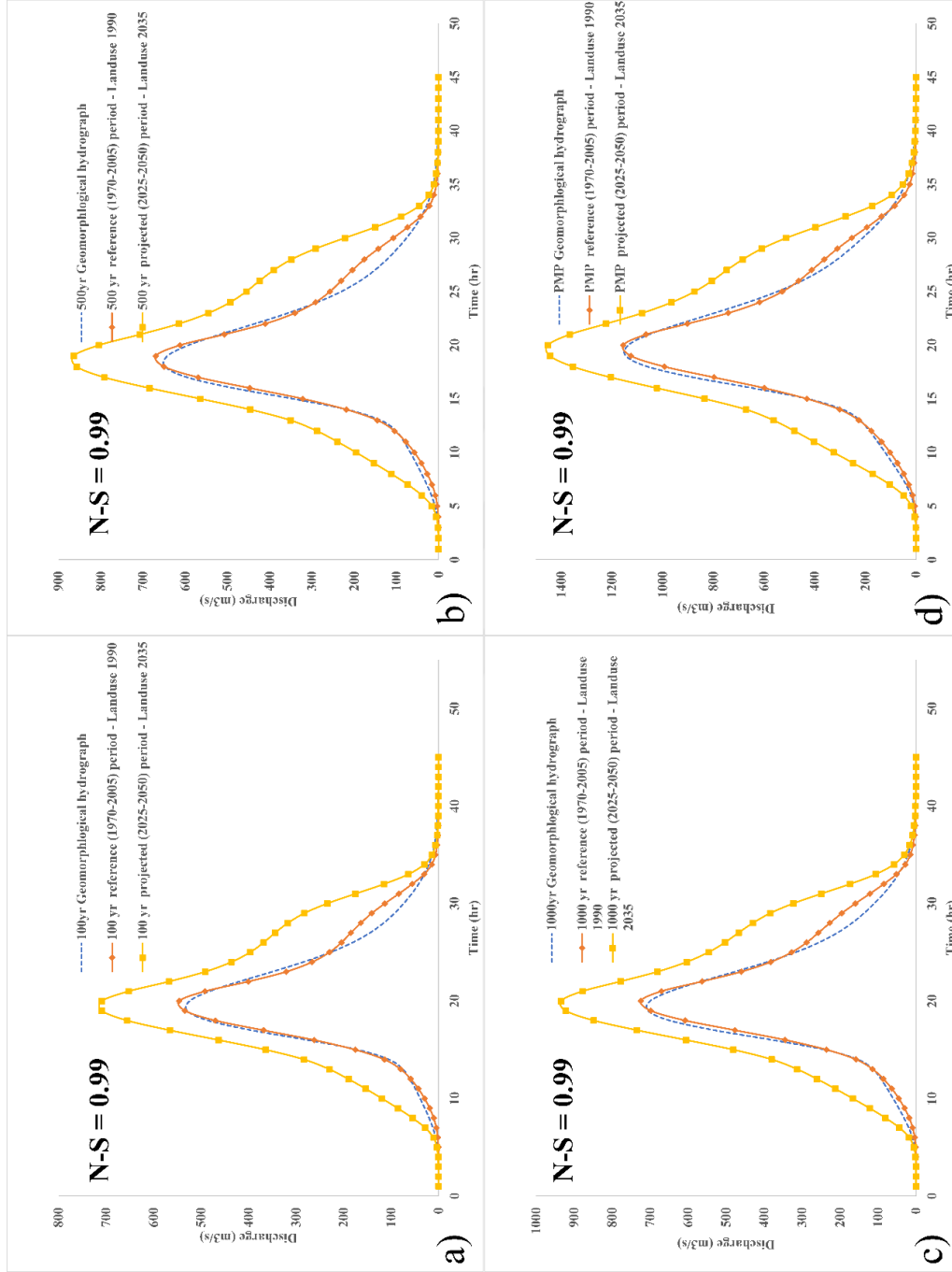


Figure 4.14 The distributed hydrological model resulting hydrograph vs. the GMUH model hydrographs for Tahtalı subbasin-1 for a) 100-, b) 500-, and c) 1000-year return period, and d) PMF

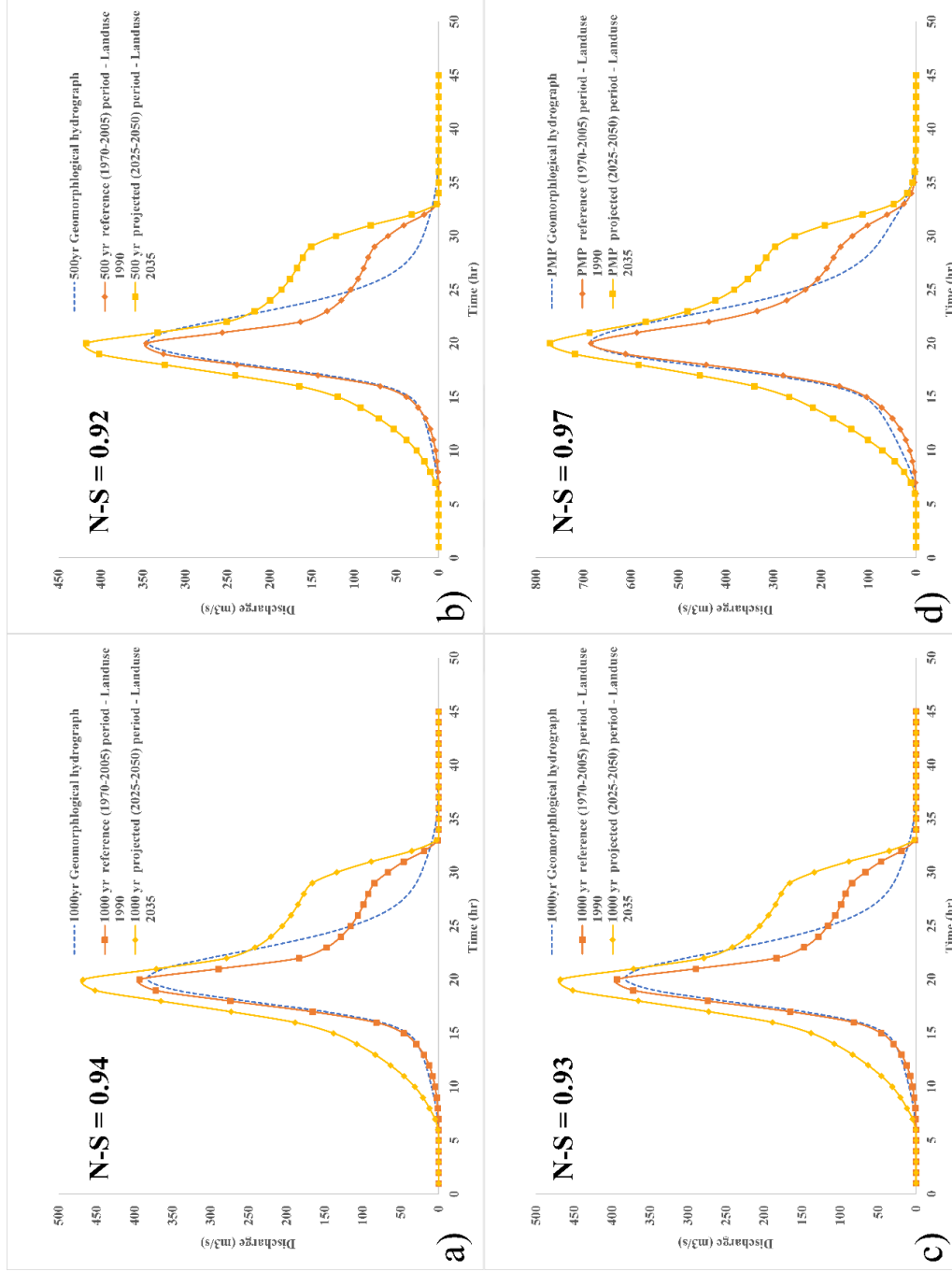


Figure 4.15 The distributed hydrological model resulting hydrograph vs. the GMUH model hydrographs for Tahtalı subbasin-2 for a) 100-, b)500-, and c) 1000-year return period, and d) PMF

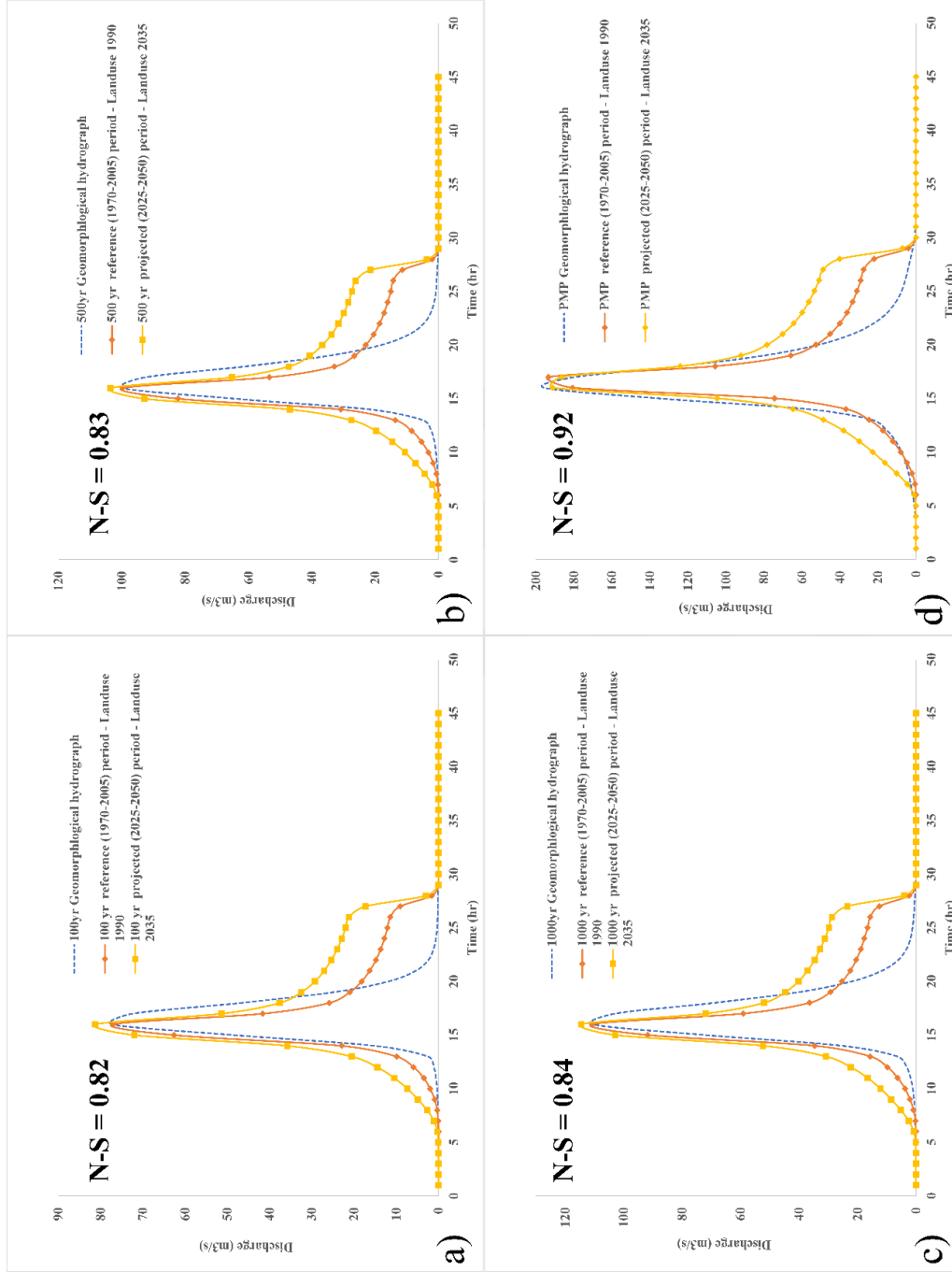


Figure 4.16 The distributed hydrological model resulting hydrograph vs. the GMUH model hydrographs for Tahtalı subbasin-3 for a) 100-, b) 500-, and c) 1000-year return period, and d) PMF

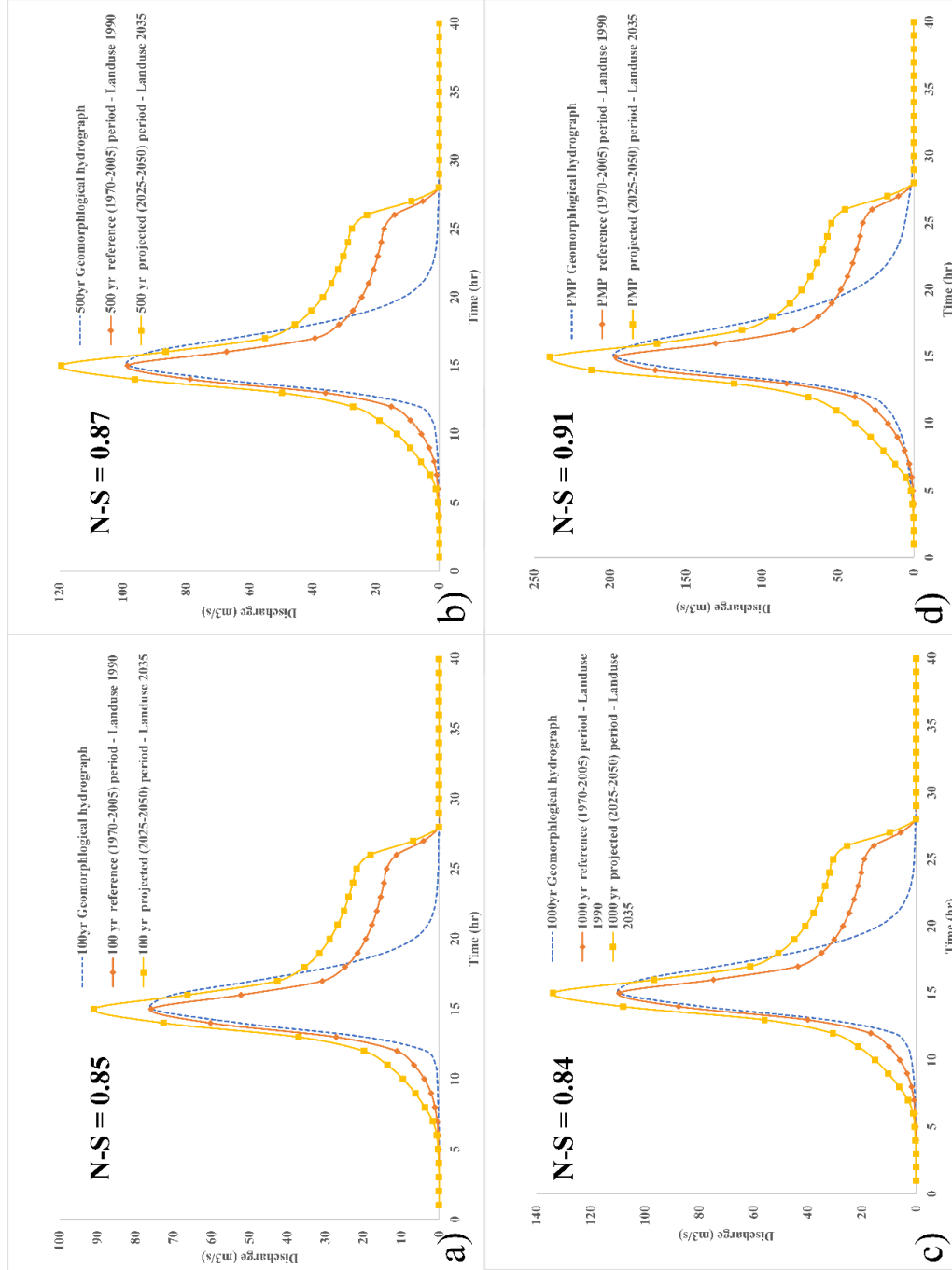


Figure 4.17 The distributed hydrological model resulting hydrograph vs. the GiMUH model hydrographs for Ürkmez basin for a) 100-, b) 500-, and c) 1000-year return period, and d) PMF

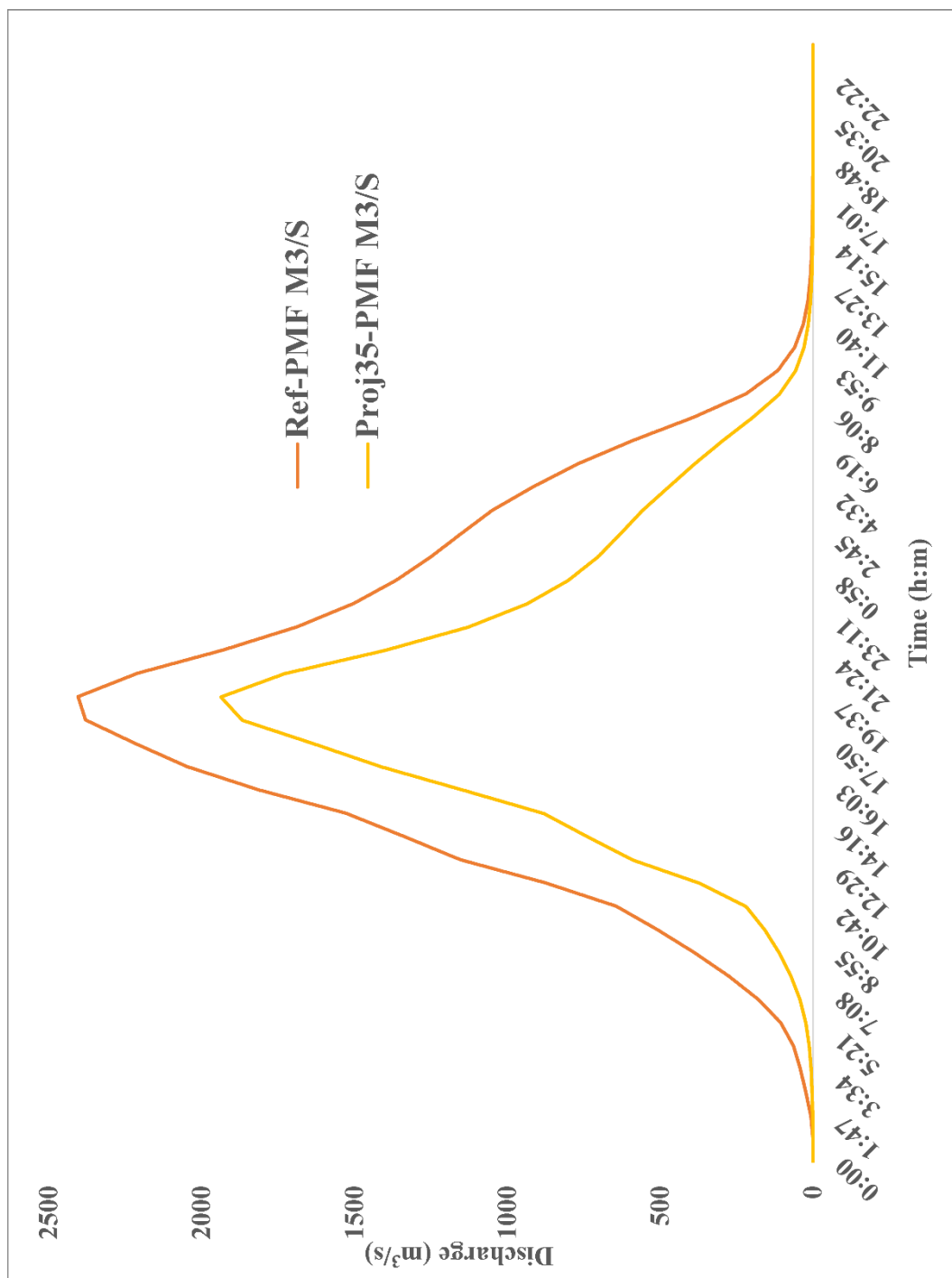


Figure 4.18 Superposition inflow PMF hydrograph generated for Tahtalı subbasins under a) reference and b) projected climate conditions

Table 4.4 The percentage change of flood peak and floodwater volume caused by climate change

Basin	Segmentation	Peak Quantity (%)*			Volume Quantity (%)*			
		100-year return period	500-year return period	1000-year return period	PMF	100-year return period	500-year return period	1000-year return period
	Subbasin-1	29.82	28.96	28.96	25.70	68.19	66.42	65.77
	Subbasin-2	22.83	20.60	18.91	12.59	66.63	65.20	63.22
	Subbasin-3	5.45	3.91	3.34	-1.09	47.25	45.34	44.60
Tahtalı								
Ürkmez Basin		22.28	19.92	21.32	22.28	22.13	40.67	42.54

*The percentages were calculated based on the reference (1970-2005) flood hydrograph quantities.

Table 4.5 Distributed hydrological model (HEC-HMS) calibration parameters

Return Period		100-year			500-year		
Basin		Tahtalı	Ürkmez		Tahtalı	Ürkmez	
Segmentation		Subbasin-1	Subbasin-2	Subbasin-3	Basin	Subbasin-1	Subbasin-2 Subbasin-3 Basin
Ratio		0.2	0.2	0.1	0.1	0.2	0.1 0.11
Factor		1	0.9	0.92	0.875	1	1 0.965 0.9
Time of Concentration (Hr)		7.75	4.6	1.75	2.15	7.75	4.6 1.75 2.15
Storage Coefficient (Hr)		1.325	0.75	0.5	0.1	1.5	0.5 0.1
Nash-Sutcliffe		0.99	0.936	0.818	0.854	0.991	0.923 0.831 0.869
Return Period		1000-year			PMF		
Basin		Tahtalı	Ürkmez		Tahtalı	Ürkmez	
Segmentation		Subbasin-1	Subbasin-2	Subbasin-3	Basin	Subbasin-1	Subbasin-2 Subbasin-3 Basin
Ratio		0.2	0.1	0.1	0.125	0.2	0.1 0.15
Factor		0.1	1	0.99	0.9	1	1 1 0.9
Time of Concentration (Hr)		7.75	4.6	1.75	2.15	7.75	4.6 1.75 2.15
Storage Coefficient (Hr)		1.65	0.5	0.5	0.1	1.7	1.15 0.1 0.1
Nash-Sutcliffe		0.99	0.925	0.837	0.876	0.995	0.97 0.915 0.905

4.5 Hydraulic-Hydrodynamic Modelling

For the thesis's hydraulic modeling part, the lower parts of the Tahtalı and Ürkmez basins, which mainly consist of densely urbanized areas, were simulated to evaluate and generate hazard maps in a potential case of a dam break failure event. The flood hydrographs generated from the HEC-HMS hydrological models were utilized to feed the HEC-RAS hydraulic model as inputs.

The maximum flow depths, times elapsed to reach maximum flow depths as well as the maximum flow velocities maps are presented for both dams based on the Sc1b failure scenario setting, which is mainly considered as the critical case since it provided the highest hydrograph peak (see Table 3.11). In the case of dam break event under reference (1970-2005) period climate conditions, the critical scenario for Tahtalı and Ürkmez Dams has failure formation times of 2.04 and 0.61 hours respectively. This particular case resulted the peak flood discharge to be measured as 587.89 and 3.65 $10^3 \text{ m}^3/\text{s}$ for Tahtalı and Ürkmez Dams, respectively. The maximum flow depths were plotted in Figure 4.19a and Figure 4.19b and the maximum flow velocities layout are presented in Figure 4.20a and Figure 4.20b for Tahtalı and Ürkmez dam-break events, respectively. For the Tahtalı Dam's downstream area, the maximum flow depth reaches above 10 m associated with a maximum flow velocity ranging from 3 to more than 5 m/s at near the populated built-up areas. In total the inundated area compromises 8.4 km^2 in total. For Ürkmez downstream, the total flooded area comprises 3.26 km^2 with the figures of the maximum flow depth reaching roughly 5 m near the eastern part of the residential area and the maximum flow velocity reaching 5 m/s. In the central and western parts of the residential area, the maximum flow depth ranged between 1 and 3 m, with the maximum flow velocities ranging between 1 and 3 m/s in the center and much less than 1 m/s in the west (see Figure 4.19b and Figure 4.20b). In simulations for the critical dam-break scenario, flood waves appeared to arrive at the main longshore highways in the residential downstream parts in 25 mins and 50 mins for Ürkmez and Tahtalı dam-break cases, respectively. Simulation shows that the maximum flow depth flood waves reach the residential area boundary in

approximately 40 and 105 min for Ürkmez and Tahtalı dam cases, respectively, as Figure 4.21a and Figure 4.21b help indicate.

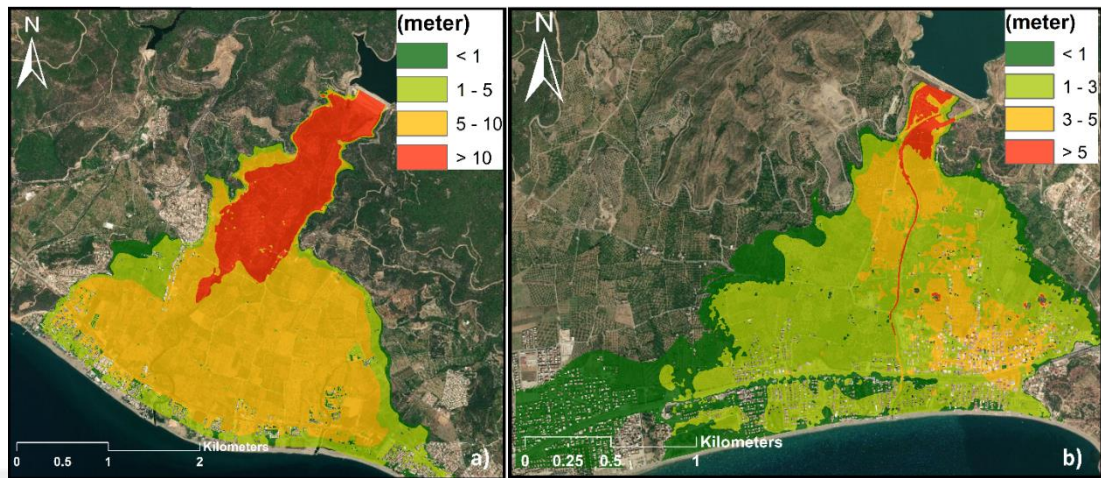


Figure 4.19 Maximum flow depth of a) Tahtalı Dam and b) Ürkmez Dam.

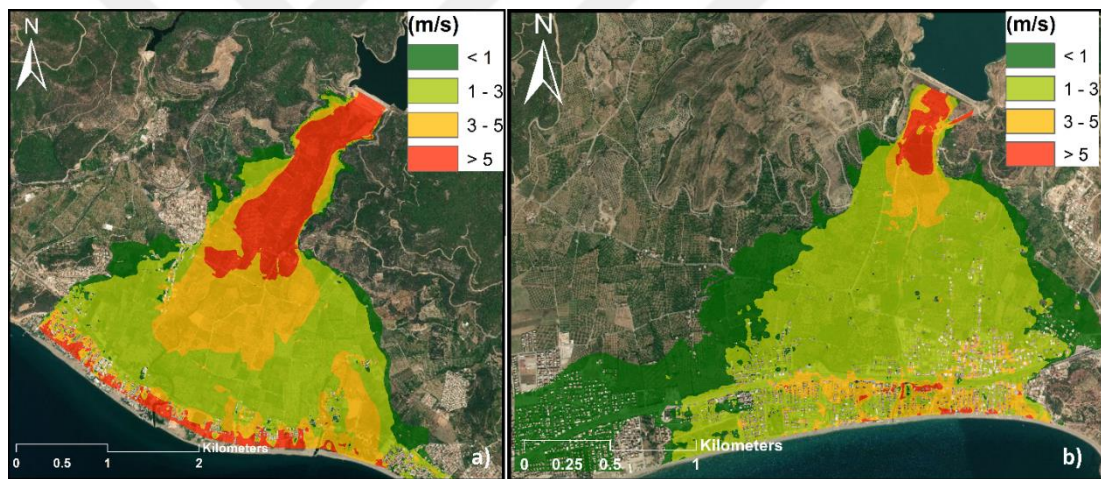


Figure 4.20 Maximum flow velocity of a) Tahtalı Dam and b) Ürkmez Dam

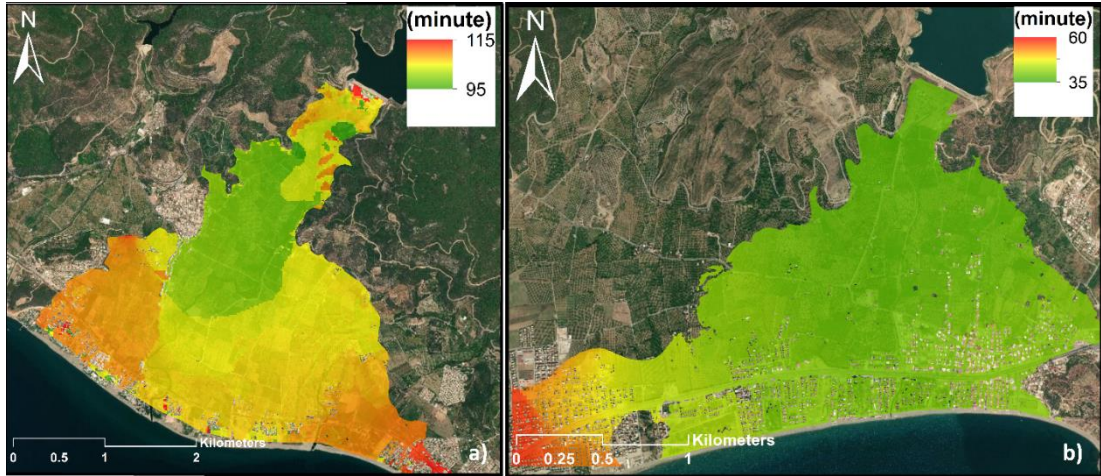


Figure 4.21 Time to maximum flow depth of a) Tahtalı Dam and b) Ürkmez Dam.

4.5.1 Sensitivity Analysis

With the aim to understand the impact of the inflow PMF hydrograph and initial water reservoir level on the outflow flood hydrograph as well as the flood-related components in the downstream (i.e., the maximum flow depth, maximum flow velocity, and time to reach the maximum depth), a two-phase sensitivity analysis was performed. Phase one is devoted to studying and thus understanding the effect of the presence/absence of the inflow flood hydrograph in cases of dam break in the aspect of hazard assessment. Phase two is devoted to reviewing the impact of climate change on dam break studies.

Therefore, to accomplish phase one, five scenarios were used in the analyses over each dam case. In the reference (1970 -2005) period, Sc1, Sc1a, and Sc1b scenarios were comparably used to study the impact of the inflow PMF, while another scenario combination covering Sc1, Sc2, and Sc3 in a comparable manner were employed to unfold the differences in the above-mentioned flood components. The details under differing scenario approaches are presented in Table 3.11.

For both dam cases, the Sc1a scenario assumes that the dam breach initiates as soon as PMF reaches its hydrograph peak, while Sc1b takes into account that the dam breach occurs when PMF discharge reaches the inflection point of the rising limb in the hydrograph. Here, these two specific scenarios were structured in such a way that the

water level in the reservoir will reach the maximum design level (as initial condition to initiate dam breach) while at the same time the PMF discharge arrives at the aforementioned hydrograph points (i.e., the peak and inflection points); thus, upon considering a trial approach to estimate the initial water level definitions separately in both scenarios to allow the above conditions occur simultaneously. On the other hand, Sc1, Sc2, and Sc3 representing the breach scenarios of the maximum operation level, normal operation level and mid-level of active volume of the reservoir, respectively, while at the same time assuming that dams' breach takes place from piping without any consideration of the PMF inflow received in reservoirs.

For Tahtalı Dam, the Sc1, Sc1a, and Sc1b comparisons reveal a slight increase in the resulting peak hydrograph by 1.4 % between Sc1 and Sc1a, while a negligible decrease of around 0.1 % is observed between Sc1 and Sc1b. On the other hand, Ürkmez Dam shows slightly different behavior as a similar increase (at around 24.5 %) in the peak of the resulting discharge in both Sc1a and Sc1b scenarios compared to Sc1. A significant decrease in the peak value was normally detected while comparing among the scenarios Sc1, Sc2, and Sc3 for both dams, subject to the assumed differences in reservoir water levels to cause the dam breach (see Table 3.11).

The maximum flow depth and velocity values resulting from the Sc1, Sc1a, and Sc1b scenarios reveal minor variations in the context of quartile statistics computed (Figure 4.22a and Figure 4.22b), despite the slightly differing spatio-temporal development pattern of the flood progress observed temporally in both dams downstream as Figure 4.22 to Figure 4.23 represent. One may normally expect to have clear divergence between the scenarios in terms of the maximum depth levels and the maximum flood velocities modeled in the area under the diversifying considerations of (1) no PMF inflow or (2) partial PMF contribution (following the dam break initiation around the peak and at the first inflection point of hydrograph curve), but the similar appearance of the flood inundation components at least in terms of the 1st, 2nd (median) and 3rd quartiles as well as the extremes strongly help indicate that the additional impact of inflow (even considered with PMF) is negligible when compared to the impact of the evacuating water volume within the available storage of the

reservoirs. However, the resulting maximum flow depths decrease for both Sc2 and Sc3 in both dam cases, as normally expected due to the assumed difference in the reservoir water level at the time of breach (see Figure 4.25a and Figure 4.25b). The same applies to the maximum velocities summarized under both scenarios (see Figure 4.26a and Figure 4.26b).

When the resulting times to local maximum flow depths are examined against five studied scenarios, a proportional change between the scenarios that consider differently defined water levels of initiating the dam failure is clearly detected in the case of Tahtalı, indicating the significance of the water volume, which evacuates from the reservoir, on the time of receiving the detrimental flood impact in the floodplain as well as activating emergency plans and rescue operations. Yet, in the case of Ürkmez, the most significant variation in this behavior is observed in the case of much less water to evacuate from the reservoir such that the times to elapse until being subjected to the maximum flood depths locally change in a wider range of time values (see Figure 4.27a and Figure 4.27b).

With regard to the Maximum inundation areas affected from any potential dam-break failure, it is observed that the modeled flood inundation areas are almost identical for the Sc1, Sc1a, and Sc1b cases for both dams as Figure 4.28 illustrates.

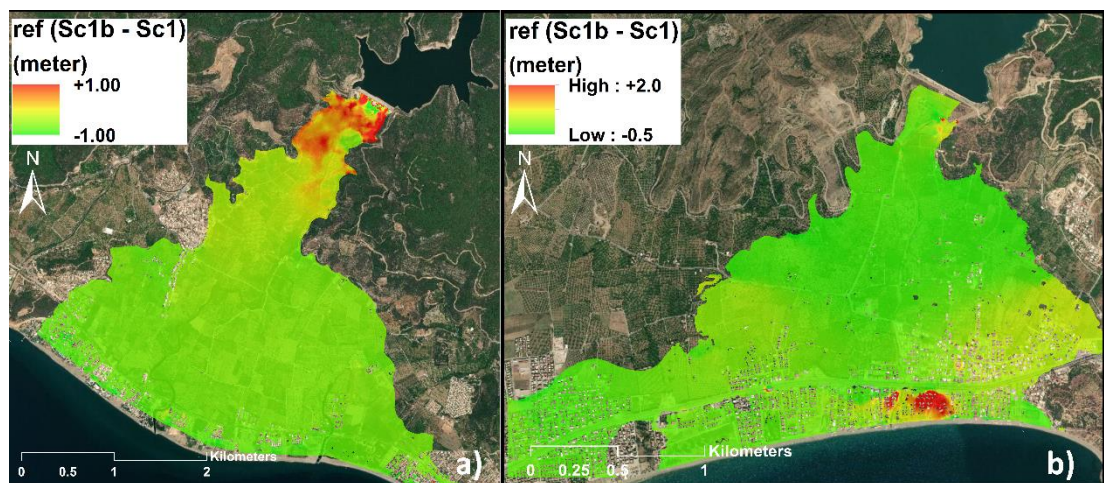


Figure 4.22 Post failure observed maximum flow depths of a) Tahtalı, and b) Ürkmez dams under different scenarios (ref_Sc1b – ref_Sc1)

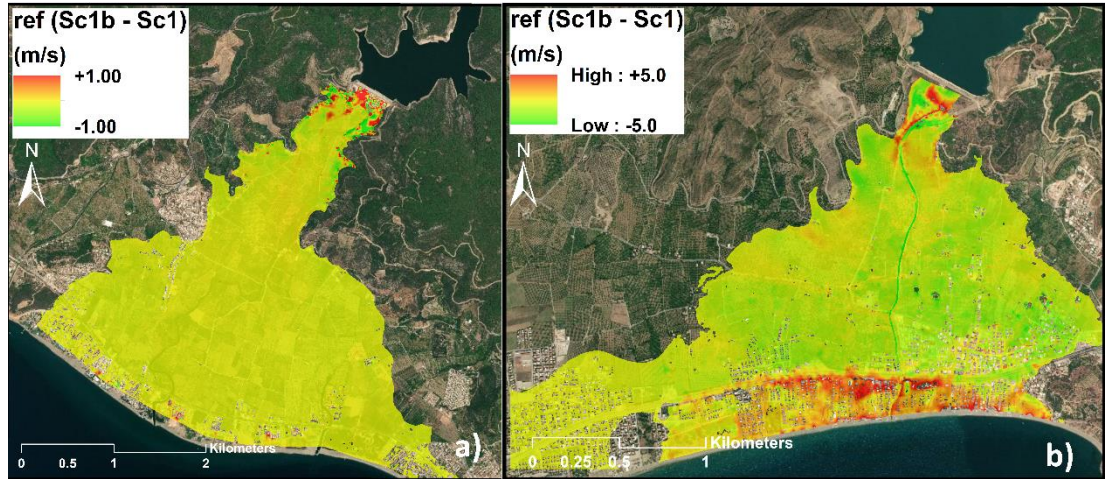


Figure 4.23 Post failure observed maximum flow velocities of a) Tahtalı, and b) Ürkmez dams under different scenarios (ref_Sc1b – ref_Sc1)

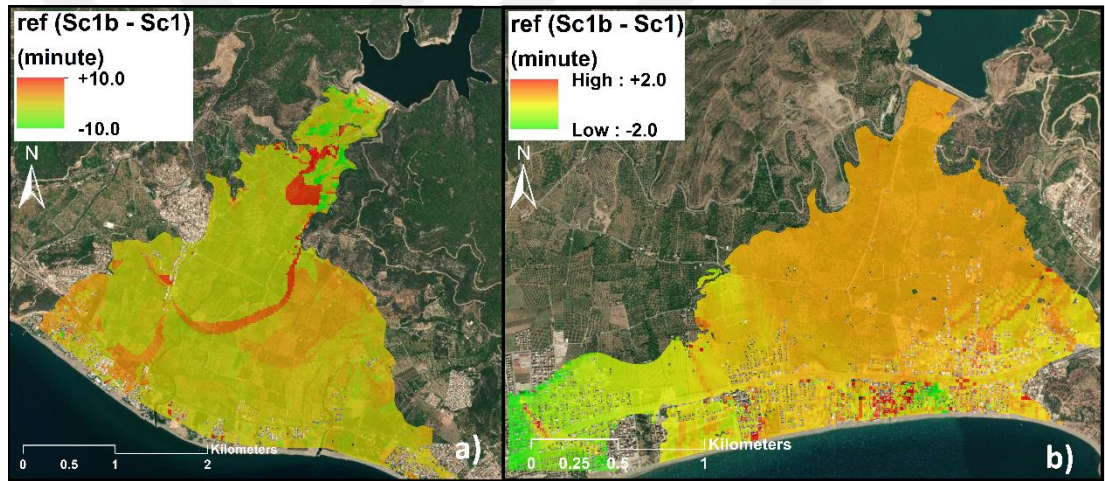


Figure 4.24 Post failure observed of the time to maximum flow depths of a) Tahtalı, and b) Ürkmez dams under different scenarios (ref_Sc1b – ref_Sc1)

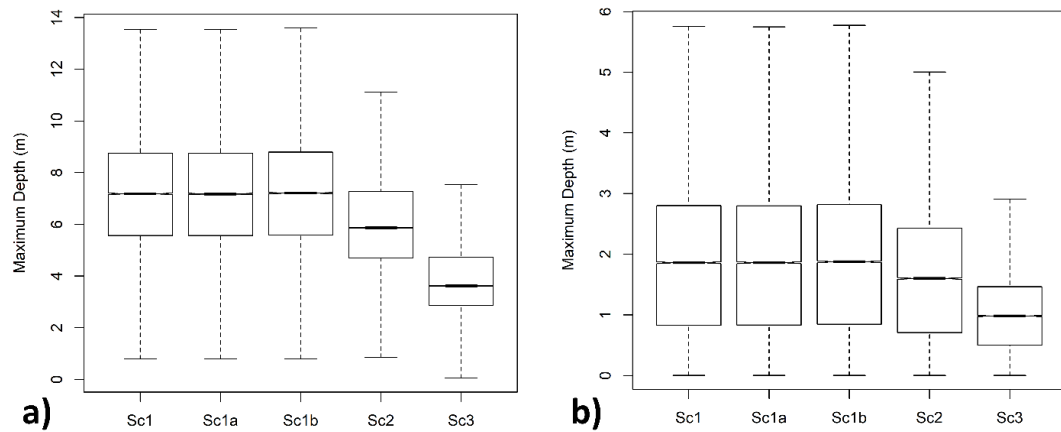


Figure 4.25 Statistics of Maximum flow depth map for a) Tahtalı Dam, b) Ürkmez Dam

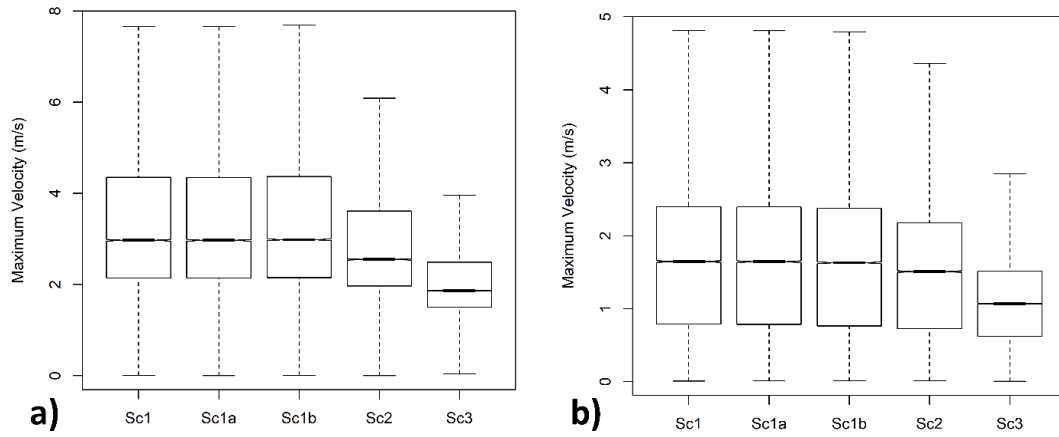


Figure 4.26 Statistics of Maximum flow velocity map for a) Tahtalı Dam, b) Ürkmez Dam

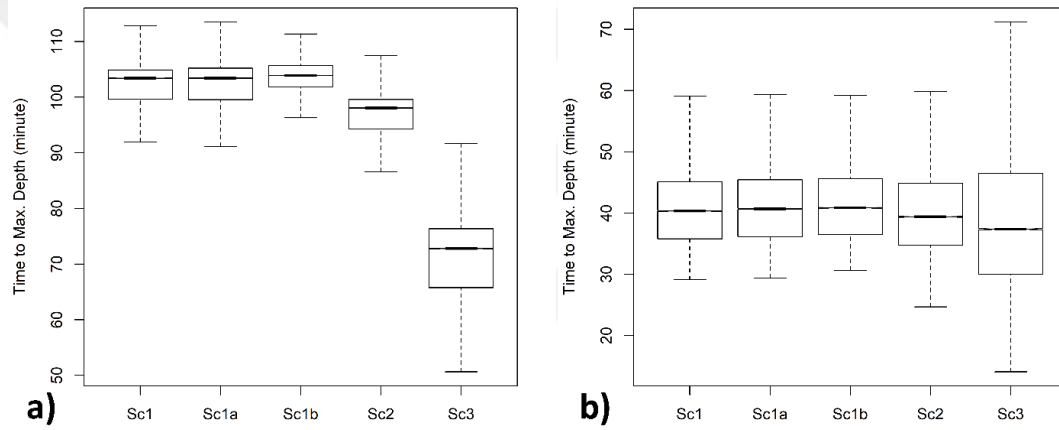


Figure 4.27 Statistics of time to Maximum flow depth map for a) Tahtalı Dam, b) Ürkmez Dam

Based on the comparison among Sc1, Sc2 and Sc3, a positively proportional change behavior is observed between the reservoir water level at the time of dam-break and flood inundation area (see Table 3.11) such that, in the case of Tahtalı dam the inundation area apparently decreases by 4.2 % and 10.9 % if the reservoir level that initiates the failure occurs at the corresponding level (55.5 m) of normal reservoir volume or the mid-level (43.25 m) of the active operation volume instead of the maximum level of 60.5 m, respectively. In the case of Ürkmez Dam, the same decrease rates are observed in 3.2 % and 23.2 %, respectively, with a clearer reduction in the inundated area when the reservoir water is much less at the time of any potential break (not necessarily due to meteorological factors).

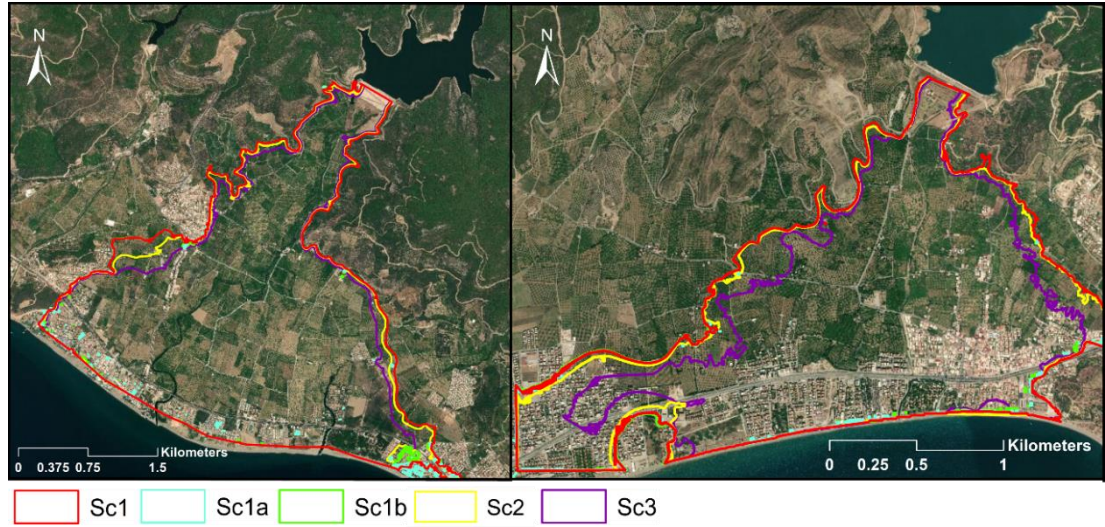


Figure 4.28 Maximum inundation mapping for a) Tahtalı dam and b) Ürkmez dam

In order to accomplish the phase-two of the sensitivity analysis, the dam break simulations corresponding to the flood hydrographs derived from both projection and reference periods were reviewed for the critical scenarios (Sc1b). The climate projection-based Sc1b dam break scenario was compared to the reference-based Sc1b dam break. Figure 4.29 to Figure 4.32 represents the comparison of the related components (i.e., the maximum flow depth, maximum flow velocity, and time to reach the maximum depth) observed after Tahtalı and Ürkmez dams' failure under the critical scenario.

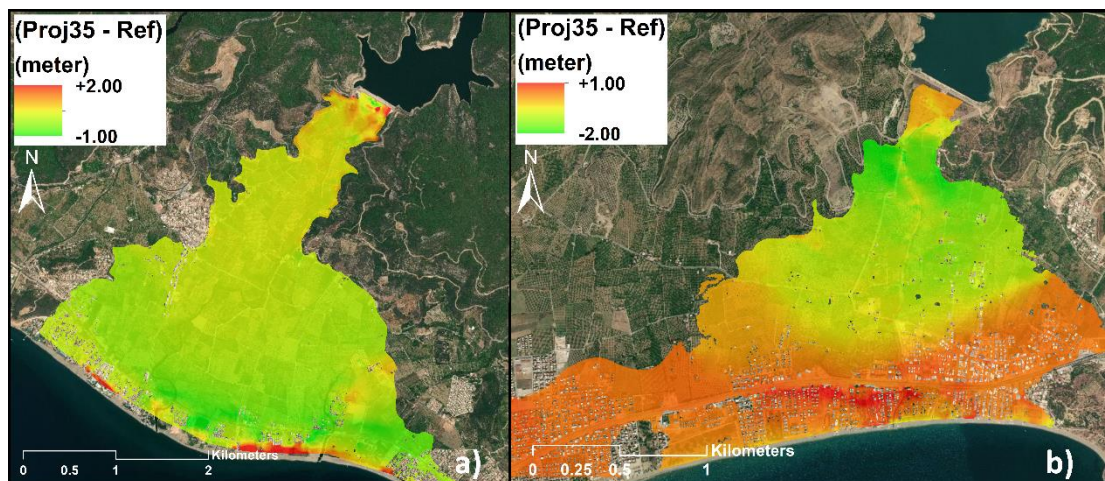


Figure 4.29 Comparison of maximum flow depths observed after the failure of a) Tahtalı, and b) Ürkmez dams under different scenarios (Proj_Sc1b – ref_Sc1b)

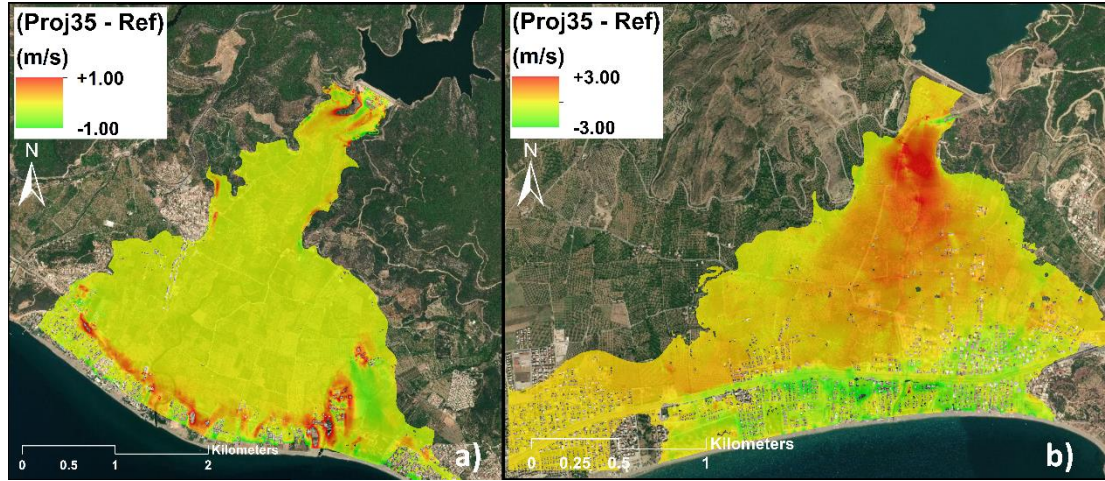


Figure 4.30 Comparison of maximum flow velocities observed after the failure of a) Tahtalı, and b) Ürkmez dams under different scenarios (Proj_Sc1b – ref_Sc1b)

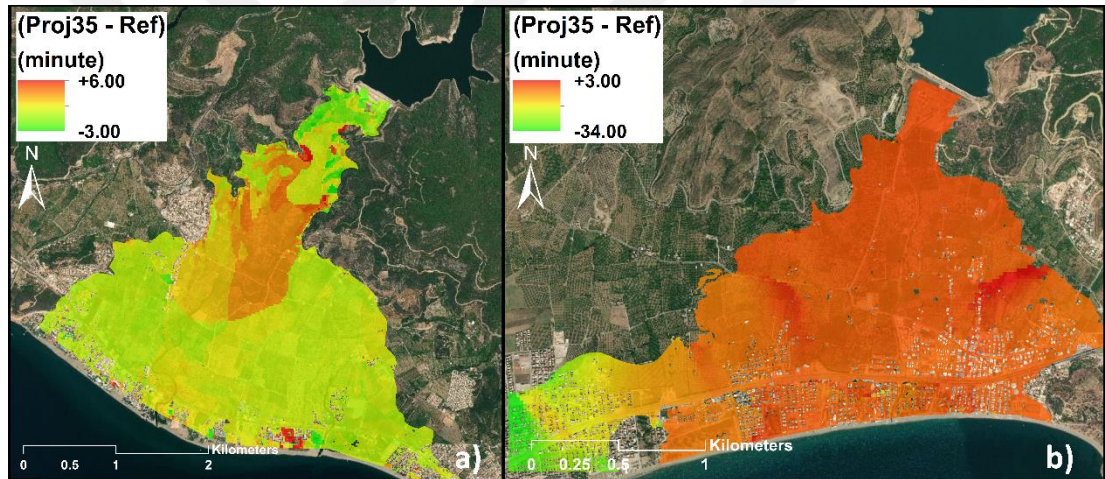


Figure 4.31 Comparison of the time to maximum flow depths observed after the failure of a) Tahtalı, and b) Ürkmez dams under different scenarios (Proj_Sc1b – ref_Sc1b)

The comparison of the simulation of the maximum flow depth that associated with the critical scenarios reveals that the maximum flow depth difference for the projected climate-based and referenced based climate PMF hydrographs ranged from -1.00 to -3.00 meter. However, the majority of the downstream area recorded maximum flow difference around zero meter, as Figure 4.29a helps indicate. Similar behavior was also recorded by the Ürkmez dam, whereas the difference ranged between -2.00 and +1.00 meters. Yet, the far downstream part of Ürkmez dam (the densely populated area)

showed an increase in the maximum depth of flood by around 2.00 meters (Figure 4.29b).

Similarly, when comparing the simulated maximum flow velocities of the same dam (e.g., Tahtalı), a neglectable difference was observed as all inundated downstream areas subject to the velocity difference ranged between -1.00 and +1.00 meter per second (Figure 4.30a). Here, Ürkmez Dam showed different behavior. Whereas, in the middle part of the downstream area of Ürkmez dam, a velocity increase of around 3.00 meters per second was observed. This difference is decreased to the range of -3.00 and -1.00 meters per second in the built-up area, as Figure 4.30b illustrates.

Regarding the arrival time to maximum depth, a negligible diversion of timing was observed for both dams. Whereas the difference in the time to the maximum flow depth obtained in the downstream for the Tahtalı dam range from -3.00 to +6.00 minutes, while in Ürkmez dam from zero to +3.00 minutes for the majority of the downstream area. However, the far western-south part of the downstream area showed decreased timing from +20.00 to 30.00 minutes, as Figure 4.32 suggests.

4.5.2 Hazard Mapping

Flood hazard is defined from the aspects of a variety of factors that include life losses, personal injuries, and property damages caused by any potential flood event (Ball et al. 2019). Flood hazard evaluation is a key input to understanding risk. The main objective of flood hazard mapping is to identify areas at risk of flooding and contribute to flooding risk management and post-disaster recovery planning. The threat to individual safety, vehicle, structural and infrastructure damage caused by floods depends largely upon several factors which include velocity of floodwaters, depth of floodwaters, the combination of the two, isolation during a flood, effective warning time and rate of rising for floodwater (Ball et al., 2019; Smith et al., 2014). There is a positive correlation between the emergence of these factors and their associated danger to people and properties (Derdous et al., 2015). The quantification of flood hazards and classification of hazard levels utilizing hazard thresholds is a

critical issue, whereas flood hazard varies considerably for people, vehicles, and building stability. In the presented study, the Australian Rainfall and Runoff guidelines (ARR) (SCARM, 2000 ; McLuckie et al., 2018) were followed to classify the hazard status in inundated areas with the classification consideration based on the depth-velocity relationship. As

Table 4.6 explains, hazard categories include H1 level being generally safe for vehicles, people, and buildings, H2 level that is unsafe for small vehicles, H3 level as unsafe for children, the elderly, and vehicles, H4 level that is unsafe for people and vehicles, and H5 & H6 as the highest risk zones unsafe for people, vehicles and all buildings vulnerable to structural damage. In this study, H1, H2, and H3 classes are considered to represent low and moderate risk-zones, while H4, H5, and H6 classes represent high-risk zones for people, vehicles, and buildings. Figure 4.32 graphically illustrates the combined hazard categories defined by the ARR.

Table 4.6 ARR Combined hazard curves – vulnerability thresholds classification limits (Smith et al., 2014)

Hazard Vulnerability Classification	Description	Classification Limit ($D \times V$)	Limiting Still Water Depth (D)	Limiting Velocity (V)
H1	Generally safe for vehicles, people and buildings.	$D \times V \leq 0.3$	0.3	2.0
H2	Unsafe for small vehicles	$D \times V \leq 0.6$	0.5	2.0
H3	Unsafe for vehicles, children and the elderly.	$D \times V \leq 0.6$	1.2	2.0
H4	Unsafe for vehicles and people.	$D \times V \leq 1.0$	2.0	2.0
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure.	$D \times V \leq 4.0$	4.0	4.0
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.	$D \times V \leq 4.0$	--	--

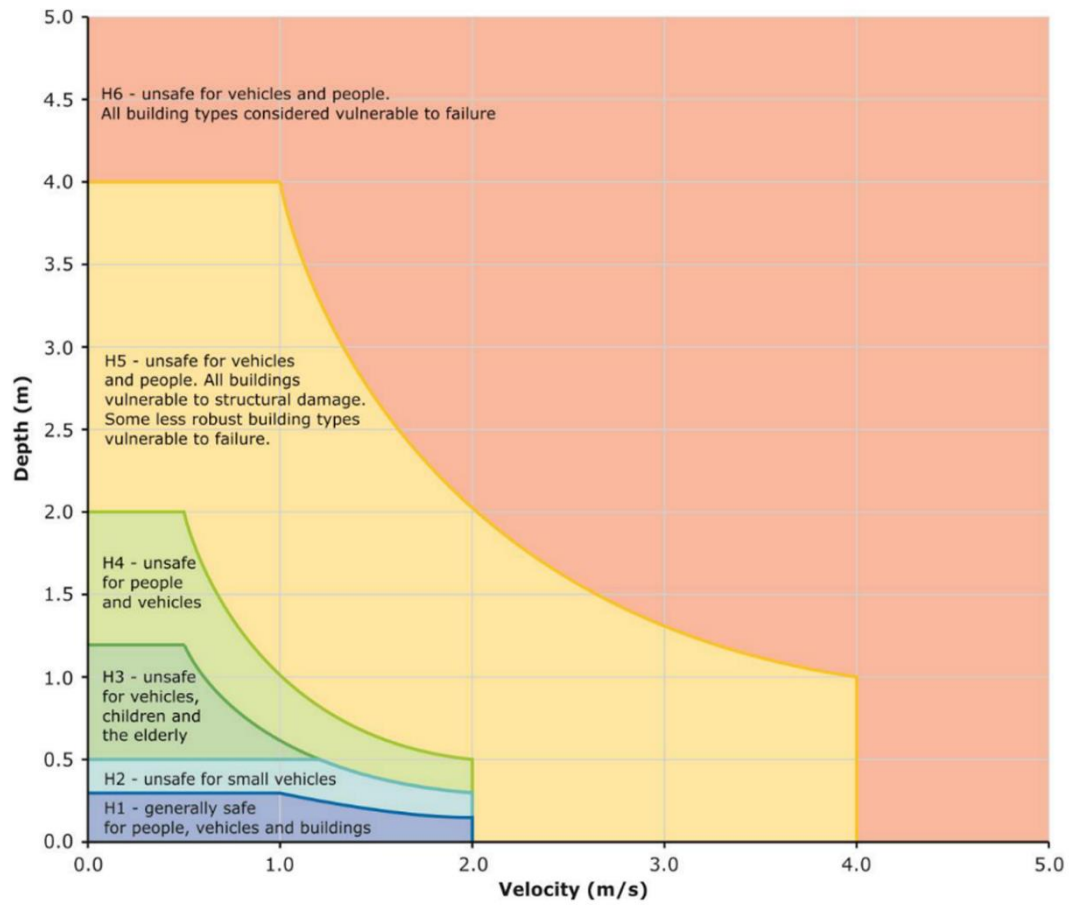


Figure 4.32 Combined flood hazard curves (Smith et al., 2014)

In the context of the presented study, the inundation area and the associated high-risk zone ratios from the temporal series of hazard inundation maps were developed at time steps of 30 mins and 15 mins for Tahtalı and Ürkmez dams, respectively, for each examined scenario for the both reference (197.-2005) and projection (2025-2050) periods. In the case of hazard map series generated under the reference period climate conditions, the hazard classification maps for the Tahtalı downstream with respect to the critical scenario (Sc1b) represented in Figure 4.33a to Figure 4.33d. Similarly, Figure 4.34a to Figure 4.34a display the hazard map series for the Ürkmez case under the same scenario consideration. Table 4.7 tabulated the flood-related statistics of spatio-temporal hazard maps series based on the abovementioned scenarios (see Table 3.11).

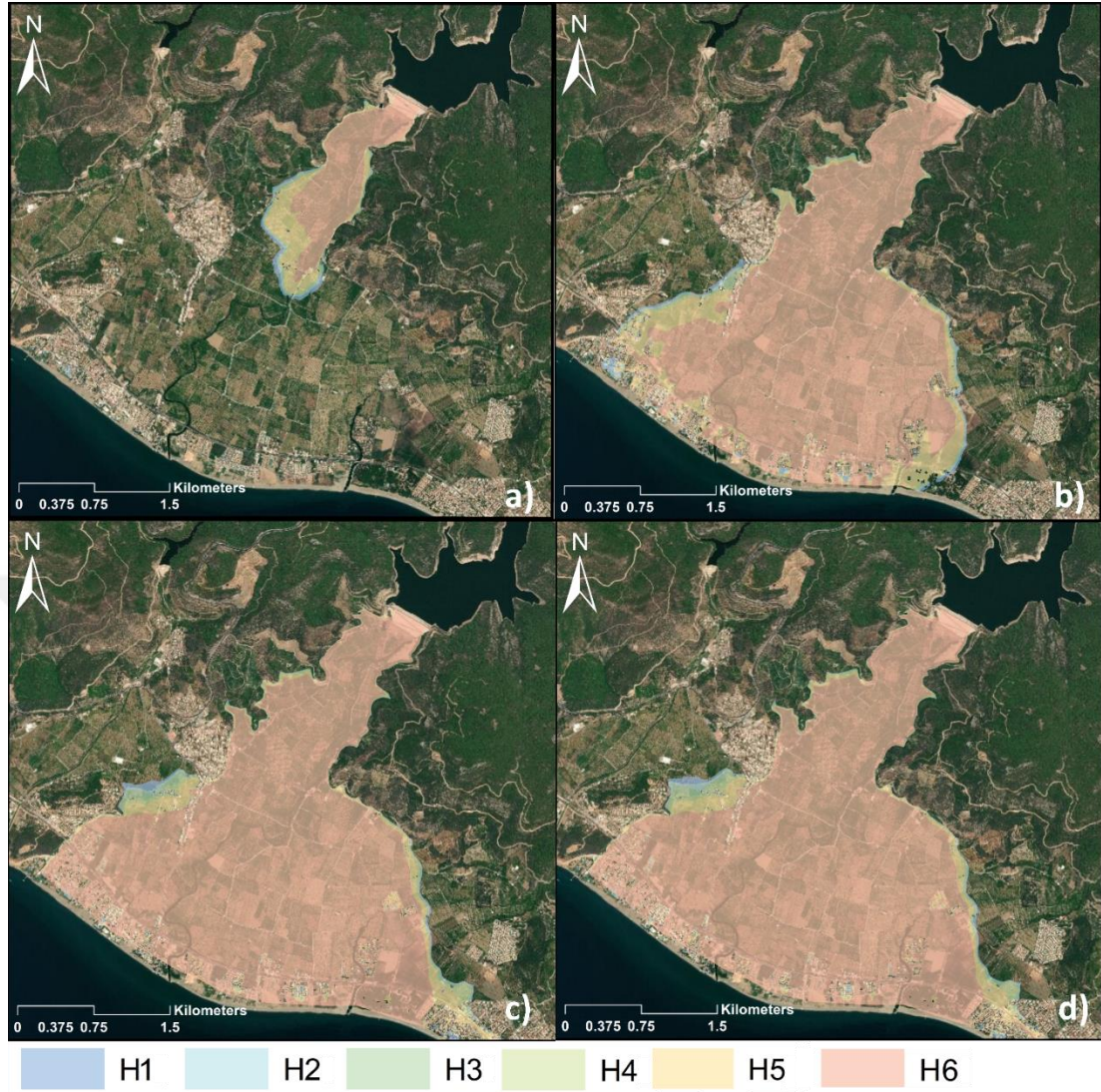


Figure 4.33 The spatio-temporal change in water extent and hazard classification of Tahtalı dam after:
a) 30 min, b) 60 min, c) 90 min, d) 120 min for the reference period

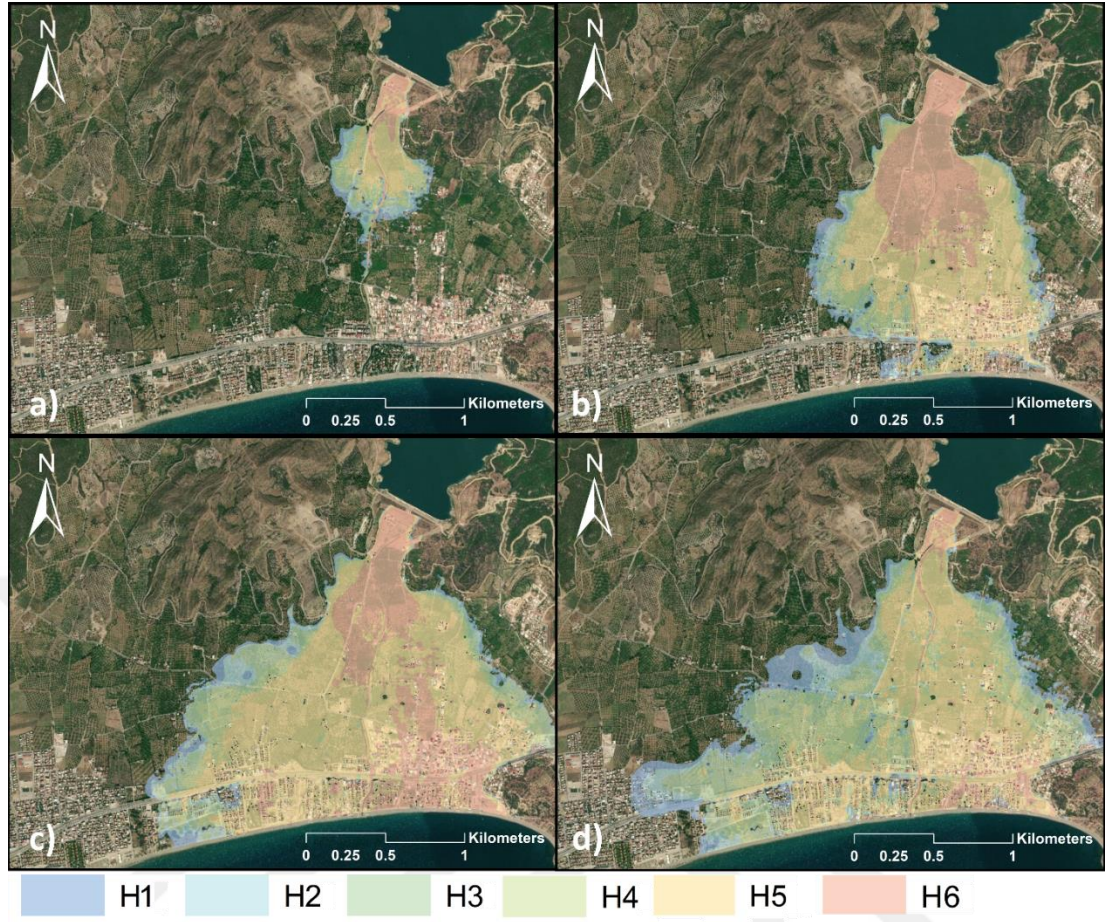


Figure 4.34 The spatio-temporal change in water extent and hazard classification of Ürkmez dam after: a) 15 min, b) 30 min, c) 45 min, d) 60 min for the reference period

Table 4.7 The statistics of spatio-temporal hazard maps series

Dam name	Scenario	High Risk Zone (HR) Statistics (H6 + H5 + H4)							
		Time Step-1		Time Step-2		Time Step-3		Time Step-4	
		inundation Area (km ²)	HR ratio (%)	inundation Area (km ²)	HR ratio (%)	inundation Area (km ²)	HR ratio (%)	inundation Area (km ²)	HR ratio (%)
Tahtalı	Sc1	1.30	90.86	7.27	96.80	8.27	97.20	8.32	97.08
	Sc1a	1.29	91.10	7.26	96.81	8.27	97.21	8.32	97.09
	Sc1b	1.25	91.01	7.22	96.72	8.27	97.22	8.33	97.14
	Sc2	1.24	90.86	7.32	96.93	7.93	96.69	7.77	96.06
	Sc3	1.08	90.04	7.05	31.98	7.17	95.83	6.75	90.44
Ürkmez	Sc1	0.26	66.30	1.60	84.45	2.65	82.57	2.62	62.30
	Sc1a	0.24	66.06	1.56	83.45	2.65	82.98	2.63	63.76
	Sc1b	0.37	66.58	1.69	83.45	2.66	82.96	2.65	64.96
	Sc2	0.26	66.44	1.64	86.26	2.55	78.76	2.48	49.86
	Sc3	0.22	61.73	1.46	73.72	1.97	49.68	1.92	16.53

In the case of hazard map series generated under the future climate conditions, the hazard classification maps for the Tahtalı downstream with respect to the critical scenario (Sc1b) represented in Figure 4.35a to Figure 4.35d. Similarly, Figure 4.36a to Figure 4.36a display the hazard map series for the Ürkmez case under the same scenario consideration.

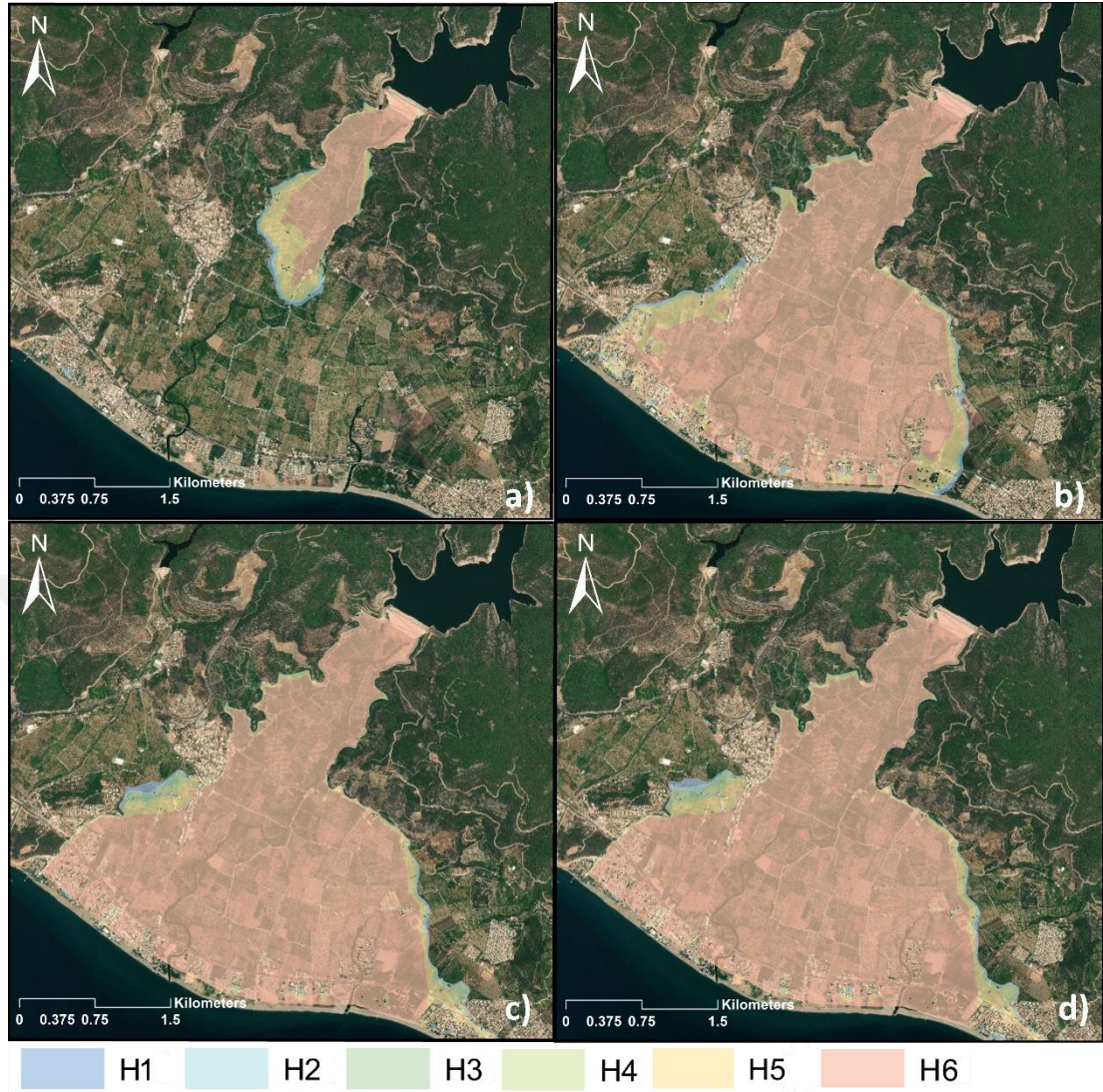


Figure 4.35 The spatio-temporal change in water extent and hazard classification of Tahtalı dam after:
a) 30 min, b) 60 min, c) 90 min, d) 120 min for the projected period

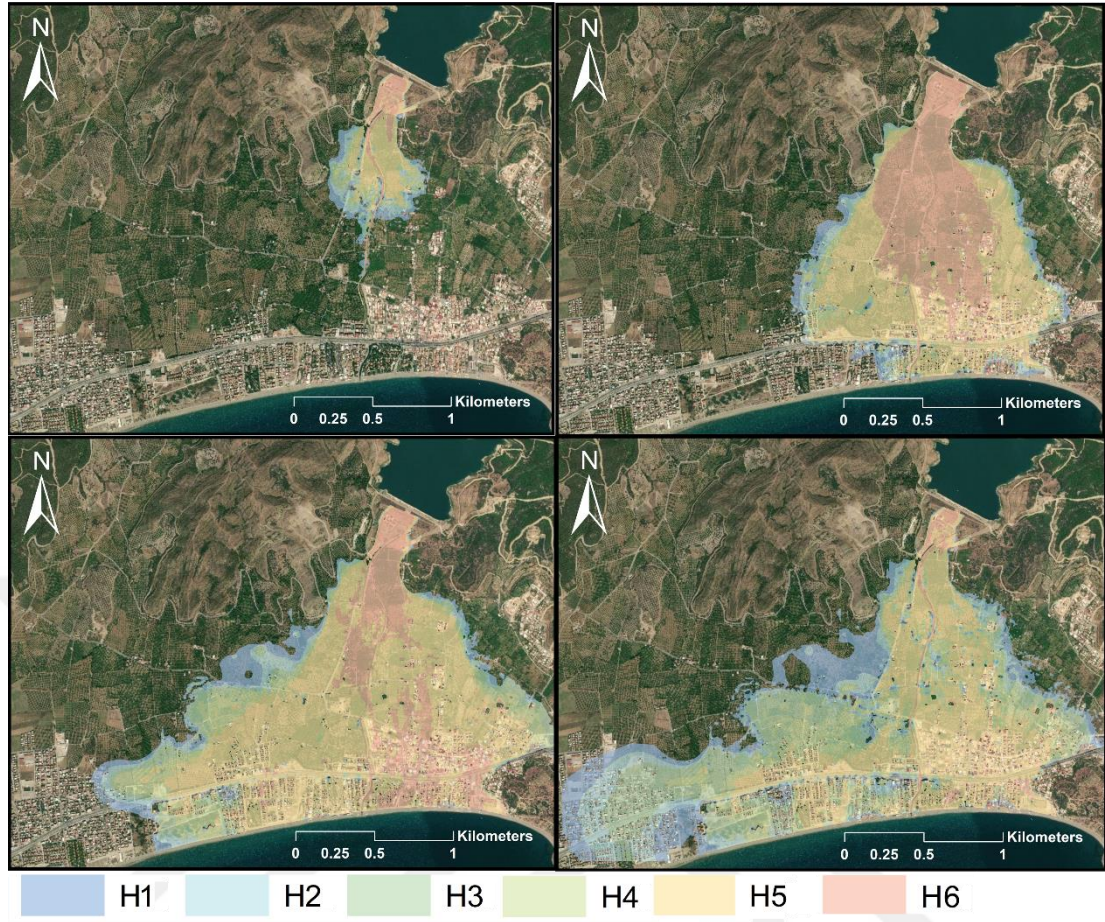


Figure 4.36 The spatio-temporal change in water extent and hazard classification of Ürkmez dam after: a) 15 min, b) 30 min, c) 45 min, d) 60 min for the projected period

For Tahtalı Dam break, the first two-time steps (i.e., 30 and 60 minutes) shows an increase in the inundated by 5% and 1.5% respectively, while for the remain two-time steps (i.e., 90 and 120 minutes) a neglectable decrease was recorded in the inundated areas. For Ürkmez dam, an insignificant increase was observed in the inundated area for all the time steps (i.e., 15, 30, 45, and 60 minutes).

CHAPTER 5

CONCLUSIONS

Moving basically from the need for complete and employable historical data in performing hazard assessment based on hydrologic, hydraulic, or combined analyses in small or mesoscale dam sites, which are mostly remembered with the handicap of unavailable or short-duration data, the presented work sets forth a holistic view for investigating dam break-driven flood influence by compiling a series of distributed analysis/modeling components in a framework approach. The study puts together the analytical components of (1) handling distributed modeling outputs of meteorological variables (precipitation and temperature) and bias elimination on data in a two-stage process with cross-comparisons to the meteorology records in the region and frequency distributions of extremes, (2) generating rainfall hyetographs through the application of instantaneous intensity method in a grid-distributed way, (3) estimating flood hydrographs specifically for hydrologic calibration purposes through the use of a synthetic unit hydrograph (GIUH) and excess rainfall hyetograph lumped for the catchment part of interest, (4) setting-up and calibrating the hydrologic model (on HEC-HMS) to allow distributed hydrologic simulations involving the greater detail of basin hydrologic parameters, (5) evaluating and thus estimating the breaching parameters use of method-specific breach parameter estimations, (6) performing hydraulic modeling (on 2D HEC-RAS) for simulating dam-break failure cases under both overtopping and piping conditions, (7) analyzing scenario-based variations (diversified with respects to dam-break development under different hydrology and reservoir conditions) of flood inundation aspects on vulnerable downstream settlements, (8) classifying the flood hazard in populated areas based on the flood hazard curves approach, and (9) investigating the combined impact of near future climate change and Land use/change on the dam break cases. Geospatial/geostatistical analyses of modeled input (mostly meteorological time-series) and output (e.g., inundation depths) data greatly take the central role in the study framework due to the fully distributed character of the analyses in various parts of the study, ranging from the elevation model generation from precise topography data to the exploratory

analysis of flood components (e.g., maximum flow depths, flow velocities, times to reach local maximum depths, etc.) in the inundation areas.

The approach that is introduced through compiled analytical components helps prove the tailored capacity of the synthetic methods, such as the instantaneous intensity method or geomorphologic instantaneous unit hydrograph estimation, in dam break failure analyses, even in basin cases of ungauged flows, with distributed analysis structure. The minimum conditions here mainly require the availability of meteorological data representative for the dam study region as well as a precise and accurate topography representation for extracting geomorphological basin characteristics. The meteorological extremes estimated with recurrence intervals of 100 years and more (e.g., 500-, 1000-years and PMF) provide a basis for bringing diversified scenario definitions while examining dam break risks and associated emergency conditions in the impact areas.

The proposed framework also, integrated the climate and land use change predictions into such a hazard assessment. This is very important as the loss of stationarity assumption would normally make changes in hazard and risk definitions. This way of thinking is quite vital in counter check of reservoir cases against failure risks with respect to the hydraulic components (mostly spill/sluceways), as the design considerations and safety factors might be greatly weakened or even expired in a greatly uncertain future.

As part of the breach geometry estimation analysis, the study investigated flooding impacts based on the flood hydrographs resulting from dam break phenomenon. Here, a series of breach prediction methods that commonly appeared in relevant scientific literature were employed in the case study. Based on comparisons with an observational judgment among all the methods employed in the study, the USBR and Von Thun & Gillette approaches estimated the highest peaks and the lowest times to corresponding peaks, Froehlich (2008) method provided moderate outputs for the both quantities (peak discharge and peak time) while the lowest peaks against delayed peak arrival times were estimated through the MacDonald & Langridge-Monopolis and Xu

& Zhang equations. In general, the piping failure modes that were simulated in all empirical approaches tend to give lower peak flood discharges in comparison to the overtopping failure mode as expected. The breach geometry sensitivity analysis pointed out that both the peak discharge and the peak arrival time (i.e., time to peak) show great sensitivity to the breach full development time (t_f) parameter (even following an almost identity line relationship with T_p) and again a considerable sensitivity to the breach width (B_{avg}) parameter. Besides, the results provided strong indication to infer that the breach side slope (Z_b) does not affect the time to peak discharge at all, even though its negligible impact on the evolution of the peak discharge may still be observed.

In particular to the case-study dams locations, the presented work helped reveal that the spillway capacities for both Tahtalı and Ürkmez dams are adequate for evacuating the flood wave from the reservoir, even under near future climate change, without any potential risk of overtopping mode of failure even within the consideration of the probable maximum flood quantities estimated separately in both cases. The remaining problem in terms of the two dam cases could still be the risks of dam failure arising from internal erosion (piping) impacts on dam bodies due to hydrodynamic or external (e.g., earthquake) forces associated with structural defects. In both cases, these piping-sourced failure types were investigated through different scenario definitions that depend on estimated arguments. Concerning the study areas under the hypothetical piping failure conditions, the achieved results show that both Tahtalı and Ürkmez reservoirs lead to significant threats to their downstream. With reference to the predictive hazard maps, considerable losses to human lives and property damages are expected to occur in more than 80% of the affected area appearing under the definition of high-risk zone regardless the simulated scenario. Besides, the climate change analysis reveals that these risks will slightly increase in term of the high-risk zones and total inundation areas.

The obtained results provide decision makers, urban & emergency planners and vulnerable communities in the final end, to help formulate evacuation procedures and to consider an adapted site planning in the areas especially associated with high and

intermediate (transition) hazard levels. The presented work is targeted to provide a contribution to the field of dam break studies and risk assessments in a tailored context by employing the synthetic hydrological models to override the data inadequacy specially in ungauged/poor gauged basins and benefit from the availability of meteorological/climate data.



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APPENDICES

APPENDIX-1

Table A.1 Average values of Manning's roughness coefficient based on CORINE land cover data (Papaioannou et al., 2018)

LABEL1	LABEL2	LABEL3	manning's (n)
1 Artificial surfaces	1.1 Urban fabric	1.1.1 Continuous urban fabric	0.013
		1.1.2 Discontinuous urban fabric	
	1.2 Industrial, commercial and transport units	1.2.1 Industrial or commercial units	0.013
		1.2.2 Road and rail networks and associated land	
		1.2.3 Port areas	
		1.2.4 Airports	
	1.3 Mine, dump and construction sites	1.3.1 Mineral extraction sites	0.013
		1.3.2 Dump sites	
		1.3.3 Construction sites	
	1.4 Artificial, non-agricultural vegetated areas	1.4.1 Green urban areas	0.025
		1.4.2 Sport and leisure facilities	
2 Agricultural areas	2.1 Arable land	2.1.1 Non-irrigated arable land	0.03
		2.1.2 Permanently irrigated land	
		2.1.3 Rice fields	
	2.2 Permanent crops	2.2.1 Vineyards	0.08
		2.2.2 Fruit trees and berry plantations	
		2.2.3 Olive groves	
	2.3 Pastures	2.3.1 Pastures	0.035

Table A.1 continues

LABEL1	LABEL2	LABEL3	manning's (n)
3 Forest and semi natural areas	2.4 Heterogeneous agricultural areas	2.4.1 Annual crops associated with permanent crops	0.04
		2.4.2 Complex cultivation patterns	
		2.4.3 Land principally occupied by agriculture, with significant areas of natural vegetation	0.05
		2.4.4 Agro-forestry areas	0.06
	3.1 Forests	3.1.1 Broad-leaved forest	0.1
		3.1.2 Coniferous forest	
		3.1.3 Mixed forest	
	3.2 Scrub and/or herbaceous vegetation associations	3.2.1 Natural grasslands	0.04
		3.2.2 Moors and heathland	0.05
		3.2.3 Sclerophyllous vegetation	
		3.2.4 Transitional woodland- shrub	0.06
	3.3 Open spaces with little or no vegetation	3.3.1 Beaches, dunes, sands	0.025
		3.3.2 Bare rocks	0.035
		3.3.3 Sparsely vegetated areas	0.027
		3.3.4 Burnt areas	0.025
		3.3.5 Glaciers and perpetual snow	0.01
4 Wetlands	4.1 Inland wetlands	4.1.1 Inland marshes	0.04
		4.1.2 Peat bogs	
	4.2 Maritime wetlands	4.2.1 Salt marshes	
		4.2.2 Salines	
		4.2.3 Intertidal flats	

Table A.1 continues

LABEL1	LABEL2	LABEL3	manning's (n)
5 Water bodies	5.1 Inland waters	5.1.1 Water courses	0.05
		5.1.2 Water bodies	
	5.2 Marine waters	5.2.1 Coastal lagoons	
		5.2.2 Estuaries	
		5.2.3 Sea and ocean	

APPENDIX-2

Table A.1 Land-use transition rates for the current (2012) and previous (2006) reference years

Serial	Land Use Class Transition	Area (km ²)
1	Forest to Forest	331.32
2	Agriculture/AGRR_non_W to Agriculture/AGRR_non_W	202.86
3	Pastures to Pastures	164.31
4	Agriculture/AGRR_W to Agriculture/AGRR_W	47.53
5	Built-up areas to Built-up areas	43.33
6	Water bodies to Water bodies	32.08
7	Forest to Pastures	20.62
8	Trees and fruits to Trees and fruits	18.28
9	Pastures to Forest	17.61
10	Transportation to Transportation	12.92
11	Bare Soil to Bare Soil	9.56
12	Agriculture/AGRR_non_W to Agriculture/AGRR_W	2.61
13	Agriculture/AGRR_non_W to Built-up areas	1.47
14	Forest to Agriculture/AGRR_non_W	1.45
15	Agriculture/AGRR_non_W to Forest	1.30
16	Pastures to Built-up areas	1.30
17	Forest to Built-up areas	1.21
18	Agriculture/AGRR_non_W to Pastures	1.09
19	Pastures to Agriculture/AGRR_non_W	0.98
20	Agriculture/AGRR_W to Forest	0.80
21	Bare Soil to Agriculture/AGRR_non_W	0.58
22	Agriculture/AGRR_non_W to Transportation	0.51
23	Bare Soil to Pastures	0.51
24	Agriculture/AGRR_W to Agriculture/AGRR_non_W	0.48
25	Agriculture/AGRR_W to Built-up areas	0.47
26	Built-up areas to Agriculture/AGRR_non_W	0.36
27	Forest to Transportation	0.22

Table A.1 continues

Serial	Land Use Class Transition	Area (km²)
28	Trees and fruits to Built-up areas	0.21
29	Agriculture/AGRR_non_W to Bare Soil	0.13
30	Forest to Bare Soil	0.12
31	Bare Soil to Forest	0.11
32	Water bodies to Forest	0.11
33	Transportation to Agriculture/AGRR_non_W	0.11
34	Forest to Water bodies	0.10
35	Transportation to Built-up areas	0.10
36	Pastures to Bare Soil	0.09
37	Built-up areas to Forest	0.08
38	Agriculture/AGRR_non_W to Water bodies	0.08
39	Water bodies to Agriculture/AGRR_W	0.07
40	Built-up areas to Pastures	0.07
41	Pastures to Trees and fruits	0.06
42	Transportation to Forest	0.06
43	Trees and fruits to Pastures	0.06
44	Built-up areas to Trees and fruits	0.05
45	Built-up areas to Transportation	0.05
46	Trees and fruits to Agriculture/AGRR_non_W	0.05
47	Agriculture/AGRR_non_W to Trees and fruits	0.05
48	Pastures to Transportation	0.04
49	Trees and fruits to Forest	0.04
50	Built-up areas to Water bodies	0.04
51	Built-up areas to Agriculture/AGRR_W	0.04
52	Agriculture/AGRR_W to Water bodies	0.03
53	Water bodies to Agriculture/AGRR_non_W	0.03
54	Forest to Trees and fruits	0.03
55	Transportation to Pastures	0.03
56	Agriculture/AGRR_W to Transportation	0.03

Table A.1 continues

Serial	Land Use Class Transition	Area (km²)
57	Water bodies to Pastures	0.03
58	Transportation to Agriculture/AGRR_W	0.02
59	Pastures to Water bodies	0.01
60	Water bodies to Built-up areas	0.01
61	Bare Soil to Built-up areas	0.01
62	Agriculture/AGRR_W to Bare Soil	0.00
63	Agriculture/AGRR_W to Pastures	0.00
64	Forest to Agriculture/AGRR_W	0.00
65	Built-up areas to Bare Soil	0.00