# 1-D AND 2-D FLOOD MODELING STUDIES AND UPSTREAM STRUCTURAL MEASURES FOR SAMSUN CITY TERME DISTRICT

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**1-D AND 2-D FLOOD MODELING STUDIES AND UPSTREAM STRUCTURAL MEASURES FOR SAMSUN CITY TERME DISTRICT**

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# ABSTRACT

1-D AND 2-D FLOOD MODELING STUDIES AND UPSTREAM STRUCTURAL MEASURES FOR SAMSUN CITY TERME DISTRICT

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In this study, Samsun City Terme District flood problem is examined with 1-D and 2-D flood modeling approach. In July 2012 Terme City Centre was exposed to a flood event. Approximately 510 $m^3/s$ flood discharge passed through the city. The river water level reached top of the levees and some parts were overflowed. The area is exposed to flooding as if the other urban areas located in Black Sea Region. The possible causes and effects of the flood problem on the Terme District are examined and some upstream structural measures are presented. MIKE by DHI (Danish Hydraulic Institute) software is used for the computer based flood modeling. MIKE 11 is selected for one-dimensional hydraulic modeling and MIKE 21 is selected for two-dimensional flood modeling. The flood models are studied from City of Terme entrance (Terme Bridge) to the upstream Salıpazarı region of the river (Salıpazarı Bridge). The studies show that the meanders on the upstream part of the Terme River help in routing and attenuating the discharge especially in flood events.

The capacity of the stream channel cannot be increased by increasing the width of the stream channel, because of the urbanization problem, therefore upstream structural measures are studied on scenario basis. Four sub catchments of Terme River are considered, in each scenario as contributing the downstream flooding. Model studies with various discharge hydrographs are carried on for different scenarios including both existing situation and possible projected situations.

Keywords: Flood Modeling, MIKE 11, MIKE 21, Meanders, Samsun, Terme

# ÖZ

SAMSUN İLİ TERME İLÇESİ 1 BOYUTLU VE 2 BOYUTLU TAŞKIN MODELLEMESİ VE YUKARI HAVZA YAPISAL ÇÖZÜM ÖNERİLERİ

Bozoğlu, Başar

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Bu çalışmada, Samsun İli Terme İlçesi taşkın problemi 1 Boyutlu ve 2 Boyutlu taşkın modelleme yaklaşımı ile incelenmiştir. Terme şehir merkezi 2012 yılı Ağustos ayında küçük çapta bir taşkına maruz kalmıştır. Yaklaşık olarak 510 $m^3/s$ taşkın debisi şehir merkezindeki dere yatağından geçmiştir. Yatak su seviyesi şev üstü kotlarına kadar yükselmiş, bazı bölümlerde ise su yatağı terk ederek yayılım göstermiştir. Çalışma alanı Karadeniz Bölgesinde yer alan tüm kentsel alanlar gibi taşkın riskine maruz kalmaktadır. Terme İlçesi'nin taşkın probleminin olası sebepleri ve etkileri incelenerek çözüm önerilerinde bulunulmuştur. DHI (Danish Hydraulic Institute) MIKE taşkın model yazılımı olarak çalışmalarda kullanılmıştır. Bir boyutlu hidrolik modelleme çalışmalarında MIKE 11 yazılımı kullanılmıştır. İki boyutlu taşkın modelleme çalışmalarında ise MIKE 21 kullanılmıştır. Taşkın modelleme çalışmaları dere memba kısmı için gerçekleştirilmiştir. Model sonuçlarına bakıldığında membada bulunan menderes oluşumlarının taşkın geciktirme görevi yaparak şehre gelen debiyi özellikle taşkın durumlarında düşürdüğü gözlenmiştir.

Şehir merkezinde dere yatağı genişliğinin arttırılarak kapasitesinin artırılması mevcut yapılaşma sebebiyle mümkün değildir. Bu sebepten dolayı memba kısmında yapısal çözümler farklı senaryolar için çalışılmıştır. Terme nehrini besleyen dört adet alt havza değerlendirilmeye alınarak her bir senaryo için mansap taşkın durumu değerlendirilmiştir. Model çalışmaları çeşitli taşkın debileri için gerçekleştirilmiş ve değişik senaryolar dikkate alınmıştır.

Anahtar Kelimeler: Taşkın Modeli, MIKE 11, MIKE 21, Menderes, Samsun, Terme

***To My Family…***

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| | |
|---|---|
| A : | flow area, $m^2$ |
| *a* : | momentum distribution coefficient |
| C : | Chezy resistance coefficient, $m^{1/2}s^{-1}$ |
| Fu, Fv : | horizontal diffusion terms |
| g : | acceleration due to gravity ($m/s^2$) |
| h : | depth above datum, *m* |
| $p_a(x,y,t)$ : | atmospheric pressure ($kg/m/s^2$) |
| Q : | lateral flow, $m^2s^{-1}$ |
| q(x,y,t) : | flux densities in x- and y- directions |
| R : | hydraulic radius, *m* |
| S : | magnitude of discharge due to point source |
| SI : | Sinuosity |
| t : | time, (sec) |
| u,v,w : | flow velocity components (m/s) |
| x,y,z : | Cartesian coordinates (m) |
| $\eta(x,y)$ : | surface elevation, (m) |
| $\rho_0(x,y,t)$ : | reference density of water ($kg/m/s^2$) |

# CHAPTER 1

# INTRODUCTION

## 1.1 Problem Statement

In many countries and regions of the world, flood is the one of the disasters that affects both lives and properties. Flood dangers and impacts can be mitigated or alleviated but they can never be completely eliminated or avoided.

The source of the flood event and the environment of the hazard specify the type of flood such as, fluvial floods (flash floods or plain floods), coastal floods (due to waves and surges), floods due to failures of hydraulic structures like dams, pluvial floods (local drainage system failure).

The basic requirement towards minimization of flood effect includes the identification of problem and the characteristics of the study area. Study on floods and floodplains require the analysis of hydrologic, hydraulic, topographic, and other related components. Most of the floodplain calculation methods are traditional manual applications and they require a significant amount of time and effort. Recently, computer-based mathematical models are highly popular and they provide effective tools for decision-making and management of flood control measures. These models reduce the computation time while improving the accuracy of determination of flood boundaries.

Flood mitigation studies depend on accuracy of flood area modeling. From engineering point of view, well-calibrated flood models are crucial to study on possibilities of structural measures. Following the above considerations, in this study, it is intended to investigate flooding problem of an urbanized area and the possible upstream solutions with the use of a computer-based mathematical model, MIKE by DHI (MIKE 11 and MIKE 21) are presented.

## 1.2 Study Area

### 1.2.1 Location of the Study Area

In this study, Samsun Terme City is selected because of the data availability and previous flood studies performed on this area.

The Terme District of the Samsun City is located at the Middle Black Sea Region of Turkey at about 40°32´-40º41´ North and 29º29´-30º08´ East. Terme district is 58 km away from Samsun. The Terme River passes through the city center and separates city into two parts. The project area begins from the Black Sea and extends through 32 km upstream of Terme. Six kilometer beginning from the Black Sea Region of the study area is the settlement area of the city. The study area of the Terme River from Terme Bridge to Salıpazarı Bridge (26 km) can be seen in Figure 1.1.


**Figure 1.1:** Terme River with branches

### 1.2.2 Description and Overview of the Study Area

The project area is composed of the Terme River and its upstream part with four branches. In July 2012, Terme City Centre was exposed to a flood event. Approximately 510 $m^3/s$ peak flood discharge passed through the city. The river water level reached top of the levees and some parts were over tapped in the city. The hydrological report (11.07.2012) of the DSI 7$^{th}$ Regional Directory states that this 510 $m^3/s$ discharge almost equals to 6-year return period of flood discharge. Flood disaster could be a bigger future problem with higher return periods for the city. The DSI 7$^{th}$ Regional Directorate initiated a tender about “Samsun Terme District, Terme River Flood Hazard Map Designation” issue. The tender included hydraulic model studies for the Terme City and the project results showed that almost entire city was flooded with 500-years return period of discharge. This study aims to improve the previous completed project (DSI, 2013). It is obvious that flooding is a big problem in this area and the details of the problem with proposed upstream structural measures were studied in this thesis. The upstream part of the Terme River was investigated to understand the flow characteristics of the sub-basins. In addition, the dam project on one of the branches of Terme River (Salıpazarı Dam) was included in the study. The model studies and hydrological works mainly focused on the problem definition and solutions.

The rehabilitation of the Terme River upstream part was designed and some parts were constructed by DSI 7$^{th}$ Regional Directory. The meanders at the upstream part of the Terme were rearranged as straight line at project site. The effects of the meanders and the ongoing project were other issues for the flood problem. This issue was also considered in this study.

Various scenarios for the identification of the problem and possible solutions of the flood problem were studied; besides, the existing situation several possibilities were considered.

## 1.3 Objectives and scope of the study

The objectives of the presented study include two components: a research component and an application component. These components can be summarized as follows:

As the research component, the basic objective is studying the effects of meanders to the flood peak discharge at the downstream of the meanders. While the meanders occur on flat areas with low river slopes, the flood plain at meanders reduce the peak discharge approaching to the downstream (Terme City Centre). The aim is to specify the flood attenuation effects of the meanders from upstream to downstream with the use of computer based hydraulic models.

As the application component, the hydraulic modeling was applied to the urbanized area and its upstream. The hydraulic model MIKE 11 (one-dimensional hydraulic model) and MIKE 21 (two-dimensional hydraulic model) were applied to the Terme River for unsteady flow simulations. Some of the flood peaks and hydrographs were obtained from the previous DSI project, the other ones were obtained from existing data. These hydrological data were defined as model inputs. The aim of the application step is to define the Terme River flood problem and to propose applicable solutions.

The thesis is composed of six chapters. This chapter introduces the study and identifies the objectives of the research. Chapter 2 investigates literature related to floods. Chapter 3 focuses on methods of hydraulic modeling technique. Chapter 4 describes the model inputs and Chapter 5 gives details of the results obtained from the study. Finally, conclusions and future recommendations are given in Chapter 6.

## 1.4 Data and software used in the study

The base point of the study is recently completed by DSI “Samsun Terme District Terme River Flood Hazard Map Designation” project. Some of the data were obtained from that project for research studies. In addition, DSI 7$^{th}$ Regional

Directorate provided “Salıpazarı Dam” preliminary design project report. Project location is at one of the branches of the Terme River. The existing condition and the projected condition with the dam of the basin were studied with the given data.

The present model studies were made with 1/5000 scaled orthophotos. The point elevation values and the aerial photos were obtained from the General Directorate of Land Registry and Cadaster. The point elevation data is 1/5000 scaled and the resolution of the aerial photo is 30 cm. The grid sizes of the elevation points are 5 m. In addition to that, the 1/1000 scaled point elevation data were obtained from DSI 7$^{th}$ Regional Directory field studies. The river bathymetry measurements and approximately 50 m left and right bank side measurements data were obtained for some parts of the river.

In this study, hydraulic modeling was conducted with Danish Hydraulic Institute (DHI) MIKE11 (one-dimensional) and MIKE21 (two-dimensional) software. ArcGIS software of Environmental Systems Research Institute (ESRI) was used as the main GIS software. Drawing works of the project was studied with AutoCAD software of Autodesk and Civil 3D was used for DEM studies.

# CHAPTER 2

# LITERATURE SURVEY

In this century, severe flood events have been observed in all over the world. Most of them were highly destructive. The most destructive deluge occurred in August 2002, in Czech Republic, Germany, Austria, Hungary, and Romania. The number of flood fatalities reached 55 and the material damage soared to USD 20 billion (Genovese, 2006).

Turkey also has flood problems. Various flood mitigation facilities were constructed and some flood management strategies were established in Turkey following the severe floods. Some of the floods are August 25-26, 1982 (Ankara), June 18-20, 1990 (Trabzon), May 16-17, 1991 (Eastern Anatolia), November 4, 1995 (İzmir), May 21, 1998 (Western Black Sea), May 28, 1998 (Hatay), November 2, 2006 (Batman), and October 9, 2011 (Antalya) (Şahin et al., 2013).

The low capacity of the hydraulic structures, urbanization without a proper city planning, reducing the stream capacity to have more settlement area, insufficient capacity of sewers for flash floods can be some of the reasons of the general flood problems in Turkey. One of the floods on İluh River caused a severe flood on 2 November 2006 in Batman, which is located in Southeastern Anatolia Region. In this flood, 10 people died and the total damages of flood costs were in the order of millions of Turkish Liras. The capacity of İluh River was decreased drastically because of buildings of various types in the main channel and floodplains. As remedial measures, cleaning of river bed, demolishing of all types of facilities on the waterway, and increasing the flow carrying capacities of the hydraulic structures were recommended (Sunkar and Tonbul, 2010).

The flood problem is a recent issue neither for Turkey nor for other countries. Therefore, the need for the flood protection and flood management are not recent too. There are many studies about flood management around the world. Recent researches suggest a risk-based approach in flood management (Hooijer et al., 2004; Petrow et al., 2006; van Alphen and van Beek, 2006). The necessity to move towards a risk based approach has also been recognized by the European Parliament (de Moel et al., 2009), which adopted a new Flood Directive (2007/60/EC) on 23 October 2007. According to the EU Flood Directive, the member states must prepare the flood hazard and risk maps for their territory and then these maps will be used for flood risk management plans. The EU Flood Directive points out the preparation of preliminary flood risk assessments due by 2011, flood hazard and risk maps need to be created by 2013 and finally, the aim is to get a flood management plan by 2015. In addition to that, flood maps are needed to be revised for every 6 years.

Flood hazard maps and the flood risk maps are two general types of flood maps. Flood hazard maps contain information about the probability and/or magnitude of an event, whereas the flood risk maps contain additional information about the consequences (e.g. economic damage, number of people affected) (de Moel et al., 2009).

The calculation of the flood hazard can be done by using methods of varying complexity (Buchele et al., 2006), depending on the amount of data, resources, and available time. The most accepted and therefore commonly used approach is computer-based flood mapping. Advanced deterministic approaches generally consist of construction of a physically based fully 2-D hydraulic model (e.g. TELEMAC-2D, Galland et al., 1991; Hervouet and Van Haren, 1996). The common properties of the 2-D hydraulic models are historical flood data used for calibration and the flood-hazard maps created in a GIS (geographical information system) environment from model results.

Various types of hydraulic models are being used in flood studies all over the world. The capacities of the software and the accuracy of the results differ. Over the past decade, a number of studies have documented about the application of 2-D hydraulic models to complex urban problems, including numerical solutions of the full 2-D shallow-water equations, and grid-based geomorphological routing models (Hunter et al., 2008).

The applications of MIKE 11 and MIKE 21 are common in flood modeling in all over the world. Mišík et al., (2013) carried out a project with MIKE 21 in city Prague. The city was affected from flood events a lot of time, the last one was in June 2013. Important part of flood protection measures in city Prague is mobile flood barriers along banks of river Vltava. The mobile flood barriers would fail eventually and they could cause flooding. It was decided that contingency planning and crisis management for such situations would be prepared based on numerical simulation of flood protection failure scenarios. Critical places of flood protection hypothetical failures were assessed and unfavorable discharge and water level scenarios were prepared. Flooding of most vulnerable urban areas was simulated by 2-D hydrodynamic unsteady flow modeling. Selected localities were defined with scenarios of flooding through sewer system manholes. Results of simulations were presented in form of maps showing flooding extent, inundation depth, water surface elevations, flow velocity magnitudes and directions, as well as by text description of flooding situation, all in selected time steps (Figure 2.1). Video animations showing flooding evolution in space and time were created. All results were elaborated in the form of interactive graphical application, which helps planners and crisis managers at city level.

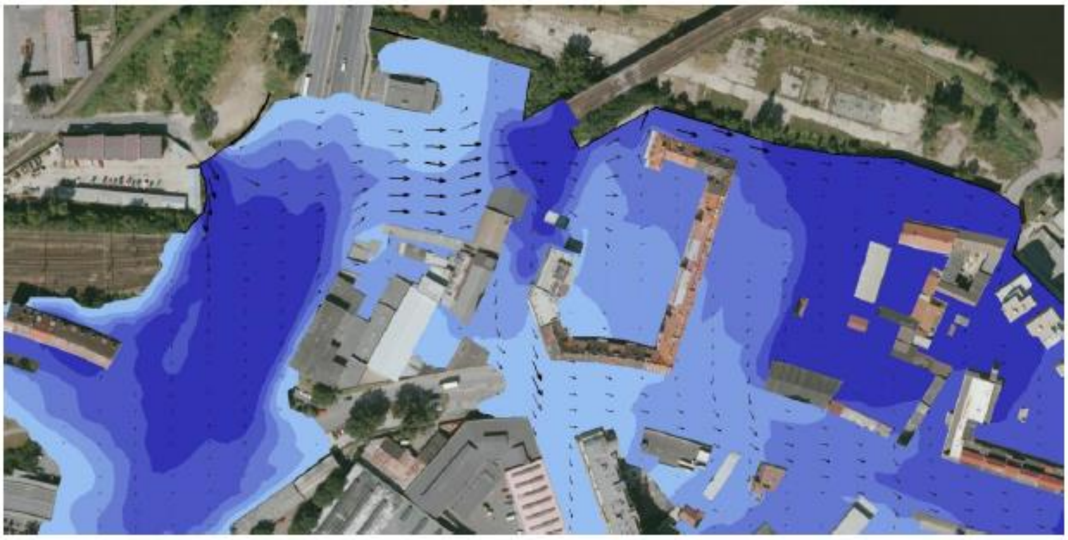

**Figure 2.1:** Simulated depths of flooding and flow directions (Mišík et al., 2013)

Ayamama River (Istanbul, Turkey) flood event occurred on 9th September 2009. The reasons of the flood were high intensity rainfall and dam-breaching of Ata Pond. The studies on this event were carried out using both 1-D and 2-D flood models. One and two-dimensional flood modeling of the Ata Pond breaching was studied using HEC-RAS and LISFLOOD-Roe models and comparison of the model results using the real flood extent were presented (Özdemir et al., 2013). The HEC-RAS model solves the full 1-D Saint Venant equations for unsteady flow, whereas LISFLOOD-Roe is the 2-D shallow water model using Saint Venant formulation. The simulated flooding in the both models were compared with the real flood extent that gathered from photos taken after the flood event. The results show that LISFLOOD-Roe hydraulic model gives more than 80% fit to the extent of real flood event. This study reveals that modeling of the probable flooding in urban areas is necessary and very important in urban planning.

Numerical simulation of flood wave propagation due to dam break studies was carried out with different studies in Turkey. One of them is the Ürkmez Dam break floodplain modeling and mapping study (Haltaş et al., 2013). This study includes numerical models Hec-RAS and FLO-2D. Dam break of the model was applied with HEC-RAS and flood wave propagation modeled with FLO-2D. The physical model of the study area was also constructed. The comparison of the physical and the numerical models results were studied. Twenty-five meter grid size numeric model and the physical model results were close to each other. However, physical model cannot represent the study area topography with sufficient resolution. DEM of the numerical models with 25 m grid size represents the model area more successfully. As a result combined physical and numerical model were recommended (Haltaş et al., 2013).

## CHAPTER 3

## MODEL STUDY APPROACH

### 3.1 Introduction

Developments in fully dynamic, unsteady models have provided engineers with highly accurate hydraulic modeling techniques that result in two and three-dimensional graphical visualizations for flood analysis. The key to graphical visualizations in dynamic modeling is the inclusion of time-series data within a spatial interface (Snead, 2000).

The modeling studies can be grouped related to their general properties. Mainly there are three flood-modeling approaches (Onuşluel, 2005):

• Engineering experience – In flood modeling studies, engineer's experience and judgment, with minimal consideration of hydraulic computations, can be explained as the base of the modeling studies. The first approaches for the model studies do not need any data except observations about 100 years ago. In levee design or determination of roadway embankment heights, historical flood level records were used as inputs since there were not any other data collected. Today, engineering experiences are fed by hydrological data and they are used for the preliminary studies of the projects.

• Physical modeling – Most of the physical modeling studies were implemented in the late 1800s and early 1900s before the computers were common. Especially in the hydraulic laboratories of universities, it was studied on conceptual and real data.

Nowadays, physical models are only constructed at large hydraulic laboratories for special problem solutions and they are simply used with numerical models for comparison. Physical models are expensive and not applicable for large scales. The model needs to be constructed and to be operated for each simulation. It also requires special engineering expertise. Scale is still a problem for simulations.

• Numerical modeling – While, in the initial studies, analytical procedures were carried out through manual computations, afterwards, computer programs have replaced them efficiently. Today, computer programs with GIS integration are the most appropriate techniques used in flood modeling studies. Numerical models have too many options. Each model also has different tools to define the problems accurately. Nowadays, numerical models are the main problem solution technique for floodplains.

Selecting numerical model for studies gives advantages in improving and changing the solutions with time. Since flood model components (settlement, watershed, channel, etc.,) change with time, model must also vary in time. A flood model represents the present situation of the river and the study area. Therefore, changes on the model conditions, such as river bed rehabilitation or structural adjustments, affect model result reliability directly. Flood maps should be updated by using new data available over time.

Model details depend on the purpose. The needed data and preparation of the data may change with purpose. Some flood maps are prepared for only showing general flood areas and others are prepared for a flood control project. These maps need to be more accurate and detailed. The used model software also changes due to the study needs and the result types.

Nowadays, computer modeling techniques have been widely used in water related studies. Engineers may determine occurred area and results of the floods by using these modeling tools. Computer modeling techniques have assisted engineers in determining where and when flooding may occur more accurately (Snead, 2000).

Determination of the water surface profiles for different flow conditions can be made by the computer based numerical models. Automated floodplain mapping supplies significant advantages compared to traditional floodplain mapping, such as saving both time and resources, providing more speed and efficiency, and developing flood depths in addition to flood extents (Noman,2001; Snead, 2000).

There are four steps in flood area determination based on computer models (Onuşluel, 2005):

**1)** Pre-processing: Preparation of the data as model input, such as Digital Elevation Model (DEM) of the area, hydrological inputs, such as precipitation, discharge measurement and roughness of the river bed.

**2)** Hydrologic Studies: Calculation of flood hydrographs or flood peak discharges depending on the scope and the available data for studies.

**3)** Hydraulic Modeling: A hydraulic model to determine water surface profiles at study area.

**4)** Post processing: Floodplain mapping and visualization of the results.

Figure 3.1 shows an algorithm for flood area determination procedure by using computer model.

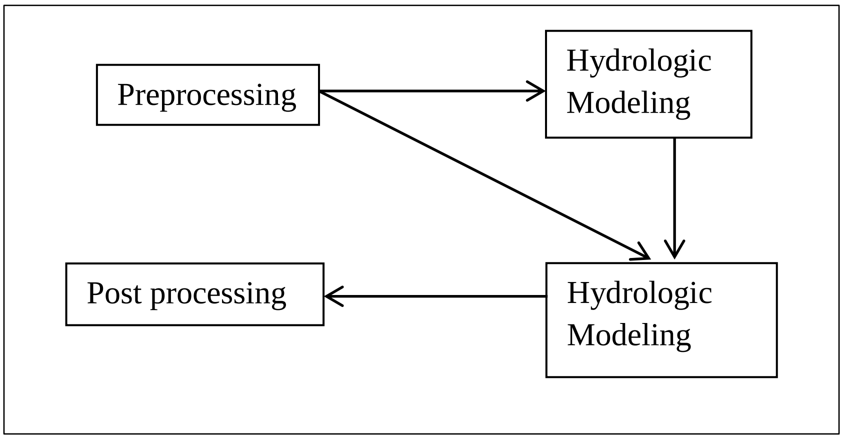


**Figure 3.1:** Flood area determination based on computer models (Onuşluel, 2005)

Since flows in river beds are naturally random and unsteady, steady state methods often do not accurately show water surface profiles. Model solutions with unsteady methods are more accurate and realistic. Developments in fully dynamic, unsteady models have provided engineers with highly accurate hydraulic modeling methods that result in two and three-dimensional graphs (Snead, 2000).

The application of a standard numerical model, such as MIKE by DHI or HEC-RAS, enables the engineer to simulate the hydraulics of the floodplain, to evaluate existing conditions, to determine proper design of hydraulic structures, and to assess the effects of these structures. Performed by a competent engineer, floodplain modeling is an objective and defensible method to determine river hydraulic information (Klotz et al., 2003).

The use of a hydraulic model in simulation of river hydraulics offers many advantages (Onuşluel, 2005):

**1)** Hydraulic models are preferred because they have proven scientific analytical tools.

**2)** Hypothetical flood events can also be simulated by using a hydraulic model. A simulation can perform a flood event before the real one occurs. The optimal solutions can be attained with different hydraulic conditions.

**3)** The project feasibility of the hydraulic structures needed for cost analysis and the design of the structures must be done for realistic cases. The hydraulic model simulations give advantages to the engineers flood frequency analyses to calculate the feasible structural design.

**4)** A hydraulic model allows quick modification of key variables, such as Manning's, to develop scenarios for the determination of the most appropriate solutions on flood problems (Klotz et al., 2003).

There are various types of computer models for computations of water surface profile. Each program has specific interface for mapping the results. A collective tool for the result maps is needed for easy comparison and common use. The interaction for such software in numerical models has great advantage. Such as MIKE by DHI used in this study has a Geographic Information Systems (GIS) interaction, which eases the interpretation of the results.

Geographic Information Systems (GIS) tool offering the ideal environment for this type of work is widely used in floodplain delineation studies. GIS offers engineers powerful capability to analyze and to express visually flood measures.

GIS is an excellent tool for the management of results and calculations. However, it cannot be used for flood modeling. GIS can be used with a flood simulation model to delineate flood areas. GIS has several advantages for flood model studies such as; it is possible to integrate data from different sources and the display, data organizing capabilities of GIS are powerful.

## 3.2 Methods of modeling

Floodplain modeling studies should follow 10 steps given below for a complete analysis (Figure 3.2):

1) Setting project and study objectives
2) Study phases
3) Field study
4) Determination of the hydrologic and hydraulic simulation types needed
5) Determination of data needs
6) Defining hydrologic modeling procedures
7) Preparing input data and calibration
8) Performing production runs for base conditions
9) Performing project evaluations
10) Preparing the report (Klotz et al., 2003)

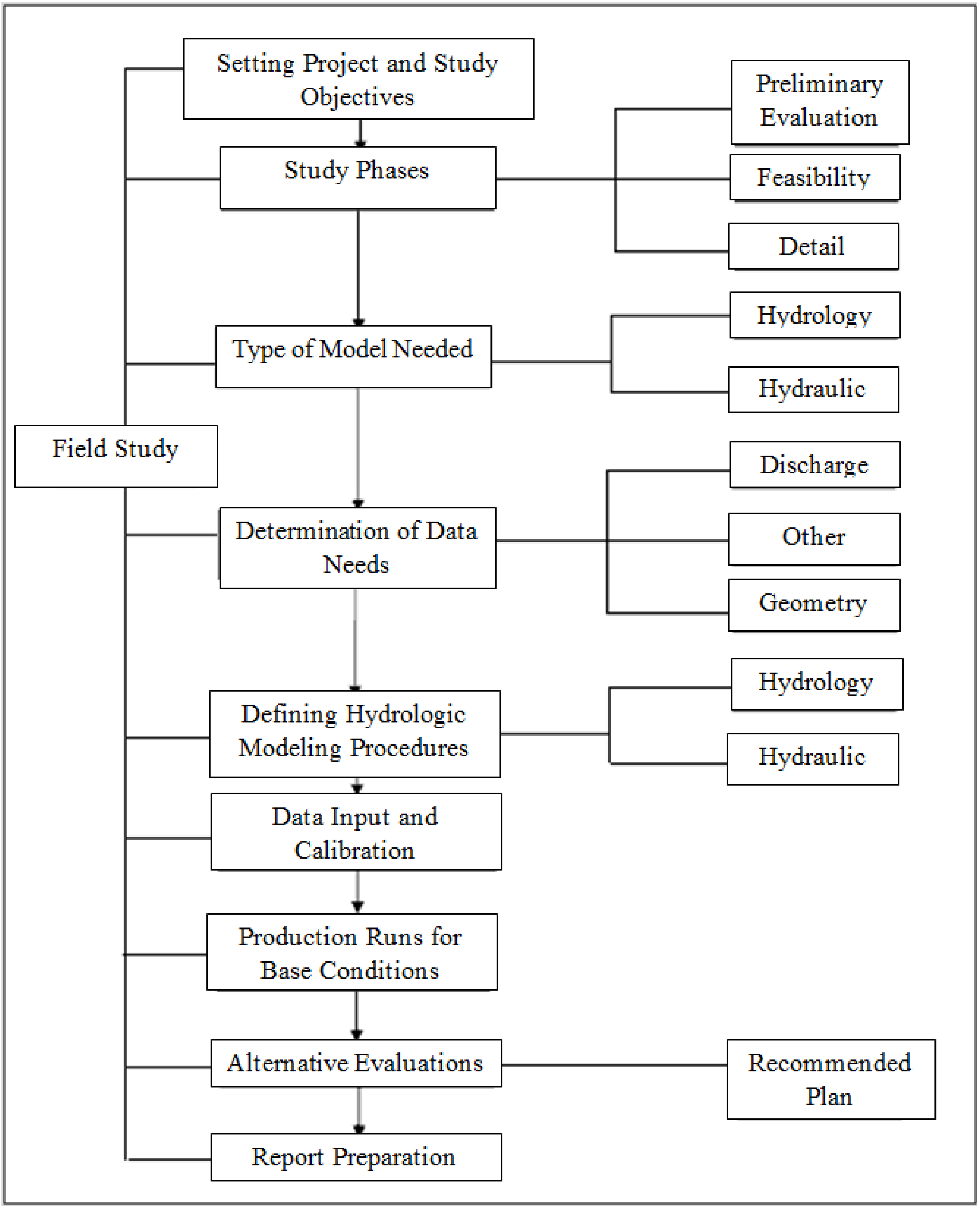


**Figure 3.2:** Steps of floodplain modeling studies (Klotz et al. 2003)

**1) Setting project and study objectives:** The main objective of flood modeling studies is the analysis of flood damage reduction. The other objectives considered may be the analysis of hydraulic structures with flood aspect.

**2) Study phases:** The preliminary evaluations involve the project area examination if such a model study is needed or not. In addition, the type of model and the required result should be defined clearly.

In the feasibility phase, the scope and magnitude of the project are specified. Flood hydrology and hydraulic analyses are performed for base conditions. For this purpose, hypothetical frequency flood profiles are determined, and then final hydrologic and hydraulic studies are performed to determine potential flood damages along the analyzed stream (Klotz et al., 2003).

**3) Field Study**: Field studies are needed for all modeling studies. The hydraulic engineer should take photographs of representative reaches of the river, bridges and culvert crossings of the main channel, and the adjacent floodplains. The channel bed material should also be surveyed at different locations. Bed material grain size is an important factor in estimating Manning's for the channel. The river banks should also be surveyed as well. In addition, the deposition on river bed and the channel cross section changes may need to be included in the floodplain modeling process.

High-water marks can be obtained from interviews with local residents to be used as calibration data.

**4) Determination of hydrologic and hydraulic simulation types needed:** This is another important step in flood modeling. The flood modeling type changes with the project area and the needs. Mostly one-dimensional model is used for river modeling; however, urban floods should be studied with two-dimensional model. Moreover, coupled one-dimensional and two-dimensional models are common in use if necessary (Klotz et al., 2003).

**5) Determination of data needs**: Flow and geometry related data (Surface elevation map and channel geometry) are the main requirements for flood modeling. If the study reach is short and not complicated, only a peak discharge value may be necessary. On the other hand, a full hydrograph is required if the reach is long with tributaries. Channel cross-sectional data and the surface elevation data must be collected at a sufficient number of points (Klotz et al., 2003).

Flow data should be collected from related organizations as discharge measurements, precipitations, evaporation and infiltration, rainfall-runoff relation and snow rates. The flood peak discharges or flood hydrographs must be calculated in hydrological analysis.

**6) Defining hydrologic and hydraulic modeling procedures:** This step is an important part of the planning process. If a hydrologic model such as rainfall-runoff model is used in the modeling process, it should be the acceptable and accurate one for the project area. This accuracy can be obtained by the model calibration with the hydrologic data for the project area (Klotz et al., 2003).

Hydraulic modeling procedures are defined with respect to project area properties and project requirements.

**7) Preparing input data and calibration:** Preparation of input data and model calibration require significant amounts of time and effort. Point elevation values should be controlled for fallacious data. The structural details should also be controlled. After triangular surface rendering, river bed elevations must be checked for incorrect values.

Calibration Data: If watermarks produced by a flood are known and stream gage data are available, discharge and water level relation can be used as calibration data. Moreover, areal rainfall maps must be prepared, using the Theissen or Isohyetal techniques both for gaged and non-gaged basins. If stream gages are available, the recorded flood events should be obtained from the agency in charge or from a reliable web site. If several actual storm-flood events are available, all events should be used in the calibration and verification process (Klotz et al., 2003).

**8) Performing production runs for base conditions:** Since the models aim to represent real situations, model calibration and verification processes are the most important steps in modeling studies. Actual representation of the floodplain depends on calibration, but data availability is generally the most problematic issue for modelers. Further adjustment to model parameters during the model runs may be required based on available local data and engineering experience (Klotz et al., 2003).

The model studies should be made for calibration process. The available calibration data such as high-water marks versus discharge relation can be used for model preparation. The roughness value of the river must be changed until the water level for known discharge is obtained. Sometimes this process needs many simulation runs.

**9) Performing project evaluations:** When modeling process is completed, the engineer has several water surface profiles, indicating the flood levels from actual or hypothetical floods at any location in the study area. These water surface profiles and inundated areas are obtained by examining a number of scenarios, which deal with flood mitigation studies or changes in the basin. For example, probability of construction a reservoir on a branch can be worked as a scenario. Since both the hydrologic and the hydraulic model will be changed in these cases, they should be run for the expected changes; and, thus, new profiles are computed. Such additional runs are recommended in order to show possible future developments that will affect flood plains. Although all possible alternatives are examined in initial planning activities, only adequate solutions with respect to effectiveness and costs should be analyzed in detail. Both for economic and practical terms, model simulations can be considered (Klotz et al., 2003).

**10) Preparing the report:** The report should be brief, clean and well-written. Technical works done and the data used for the project can be specified in this report (Klotz et al., 2003).

## 3.3 Modeling Software

### 3.3.1 One-dimensional Model Software

Water movement in one-direction can be modeled by MIKE 11. The application areas of the MIKE 11 software are; flood prediction, sediment transport, water quality, dam break analyses and flood modeling studies. The effective use as a flood model is for valley floods and the flood movement inside the river bed.

**MIKE 11**

MIKE 11 is applicable for simulating rivers and other open surface water bodies, which can be approximated as 1-D flow (DHI, 2009).

The MIKE 11 solution of the continuity and momentum equations is based on an implicit finite difference scheme developed by Abbott and Ionescu (1967). The scheme is setup to solve the Saint Venant equations – i.e. simplified hydraulic calculations. The water level and flow velocity are calculated at each time step by solving the continuity equation and the momentum equation centered on Q-points. By default, the equations are solved with two iterations. The first iteration starts from the results of the previous time step and the second uses the centered values from the first iteration. The number of iterations is user specified.

Cross sections are easily specified on the user interface. The water level (h points) is calculated at each cross section and at model; interpolated interior points that are located evenly and specified by the user- entered maximum distance. The flow Q is then calculated at point's midway between neighboring h-points and at structures.

The hydraulic resistance is based on the friction slope from the empirical equation, Manning's or Chezy, with several ways modifying the roughness to account for variations throughout the cross-sectional area.

The following Equation 3.1 is the continuity equation and the Equation 3.2 is the momentum equation for 1-D flood model.

$$\frac{\partial q}{\partial x}+\frac{\partial A_{fl}}{\partial t}=q_{in} \tag{3.1}$$

$$\frac{\partial q}{\partial t}+\frac{\partial\left(\alpha\frac{q^2}{A_{fl}}\right)}{\partial x}+gA_{fl}\frac{\partial h}{\partial x}+\frac{g\mathrm{q}|q|}{C^2A_{fl}R}=0 \tag{3.2}$$

A : flow area, $m^2$

q : lateral flow, $m^2s^{-1}$

h : depth above datum, *m*

C : Chezy resistance coefficient, $m^{1/2}s^{-1}$

R : hydraulic radius, *m*

*a* : momentum distribution coefficient

x : Cartesian coordinates

g : acceleration due to gravity ($m/s^2$)

**Model Technique**

MIKE 11 interface consists of some components. All of these components are parts of the model studies. They interact with each other and creating a model together. Figure 3.3 shows model components interaction.

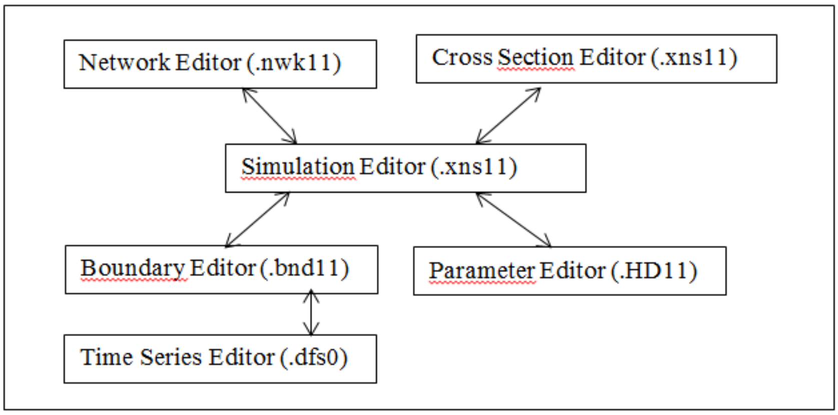


**Figure 3.3:** MIKE 11 Flood model scheme (DHI, 2009)

Simulation editor can be described as the control panel of the MIKE 11 hydraulic modeling. The sub divisions of the model are;

1- Network editor (River network defined)
2- Cross Section Editor (Cross sections on the river network defined)
3- Boundary Editor (Upstream and downstream conditions defined)
4- Parameter Editor (Manning values defined)

Simulation editor links four components of the model. The data editing would be done from related sub-component.

### 3.3.2 Two-dimensional Model Software

MIKE 21 Flow Model is a modeling system for 2-D free-surface flows used for this study.

The hydrodynamic (HD) module is the basic module in the MIKE 21, which provides the hydrodynamic basis for the computations performed. The hydrodynamic module simulates water level variations and flows in response to a variety of forcing functions.

### MIKE 21 Single Grid

MIKE 21 single grid model is a general numerical modeling system for the simulation of water levels and flows. It simulates unsteady two-dimensional flows in one layer (vertically homogeneous) fluids and it is applied in a large number of studies (DHI, 2010).

MIKE 21 makes use of a so-called Alternating Direction Implicit (ADI) technique to integrate the equations for mass and momentum conservation in the space-time domain. A Double Sweep (DS) algorithm resolves the equation matrices that result for each direction and each individual grid line.

MIKE 21 has the following properties (DHI, 2010):

- Zero numerical mass, momentum and negligible numerical energy falsification over the range of practical applications, though centering of all difference terms and dominant coefficients, achieved without resort to iteration.

- Second to third order accurate convective momentum terms, i.e. "second- and third-order" respectively in terms of the discretization error in a Taylor series expansion.

- Well-conditioned algorithm solution that is providing accurate, reliable and fast operation.

### MIKE 21 Flexible Mesh

The FM (Flexible Mesh) Series meets the increasing demand for realistic representations of nature. The modeling system has been developed for complex applications within oceanographic, coastal and estuarine environments. However, being a general modeling system for 2-D and 3D free-surface flows it may also be applied for studies of inland surface waters, e.g. overland flooding and lakes or reservoirs (DHI, 2010).

The Hydrodynamic Module provides the basis for computations performed in many other modules, but can also be used alone. It simulates the water level variations and flows in response to a variety of forcing functions on flood plains, in lakes, estuaries and coastal areas.

The Hydrodynamic Module included in MIKE 21 Flow Model FM simulates unsteady flow taking into account density variations, bathymetry and external forcing.

**Model Equations**

The modeling system bases on the numerical solution of the two/three-dimensional incompressible Reynolds averaged Navier-Stokes equations subject to the assumptions of Boussinesq and of hydrostatic pressure. Thus, the model consists of continuity, momentum, temperature, salinity and density equations and it is closed by a turbulent closure scheme. The density does not depend on the pressure, but only on the temperature and the salinity (DHI, 2010).

Unstructured mesh technique gives the maximum degree of flexibility, for example:

**1)** Control of node distribution allows for optimal usage of nodes

**2)** Adoption of mesh resolution to the relevant physical scales

**3)** Depth-adaptive and boundary-fitted mesh. The governing equations are presented using Cartesian coordinates. The local continuity equation is written as follows;

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=S \qquad (\mathbf{3.3})$$

Two horizontal momentum equations for the x and y component, respectively

$$\frac{\partial u}{\partial t}+\frac{\partial u^2}{\partial x}+\frac{\partial vu}{\partial y}+\frac{\partial wu}{\partial z} \qquad (\mathbf{3.4})$$

$$=\int v-g\frac{\partial \eta}{\partial x}-\frac{1}{\rho_0}\frac{\partial p_a}{\partial x}-\frac{g}{\rho_0}\int_z^\eta \frac{\partial \rho}{\partial x}dz+F_u+\frac{\partial}{\partial z}\left(v_t\frac{\partial u}{\partial z}\right)+u_sS$$

$$\frac{\partial v}{\partial t}+\frac{\partial v^2}{\partial y}+\frac{\partial uv}{\partial x}+\frac{\partial wv}{\partial z} \tag{3.5}$$
$$= -fu - g\frac{\partial \eta}{\partial y} - \frac{1}{\rho_0}\frac{\partial p_a}{\partial y} - \frac{g}{\rho_0}\int_z^{\eta}\frac{\partial \rho}{\partial y}dz + F_v + \frac{\partial}{\partial z}\left(v_t\frac{\partial v}{\partial z}\right) + v_s S$$

$$F_u = \frac{\partial}{\partial x}\left(2A\frac{\partial u}{\partial x}\right) + \frac{\partial}{\partial y}\left(A\left(\frac{\partial v}{\partial x}+\frac{\partial v}{\partial x}\right)\right) \tag{3.6}$$

$$F_v = \frac{\partial}{\partial x}\left(A\left(\frac{\partial u}{\partial y}+\frac{\partial v}{\partial x}\right)\right) + \frac{\partial}{\partial y}\left(2A\frac{\partial v}{\partial x}\right) \tag{3.7}$$

| | |
|---|---|
| t : | time, (sec) |
| x,y,z : | Cartesian coordinates (m) |
| u,v,w : | flow velocity components (m/s) |
| S : | magnitude of discharge due to point source |
| Fu, Fv : | horizontal diffusion terms |
| h : | depth above datum, (m) |
| η(x,y) : | surface elevation, (m) |
| g : | acceleration due to gravity (m/s$^2$) |
| $p_a$(x,y,t) : | atmospheric pressure (kg/m/s$^2$) |
| $\rho_0$(x,y,t) : | reference density of water (kg/m/s$^2$) |

**Solution Technique**

The spatial discretization of the primitive equations is performed using a cell-centered finite volume method. The spatial domain is discretized by subdivision of the continuum into non-overlapping elements/cells. In the 2-D model, the elements can be triangles or quadrilateral elements.

**Model Input**

Input data can be divided into the following groups (DHI, 2010):

• Domain and time parameters:

- Computational mesh (the coordinate type is defined in the computational mesh file) and bathymetry

- Simulation length and overall time step

• Calibration factors

- Bed resistance
- Momentum dispersion coefficients
- Wind friction factors

• Initial conditions

- Water surface level
- Velocity components

• Boundary conditions

- Closed boundary
- Water level
- Discharge

• Other driving forces

- Wind speed and direction
- Tide
- Source/sink discharge

- Wave radiation stresses

Providing MIKE 21 Flow Model FM with a suitable mesh is essential for obtaining reliable results from the models. Setting up the mesh includes the appropriate selection of the area to be modeled, adequate resolution of the bathymetry, flow, wind and wave fields under consideration and definition of codes for defining boundaries.

**Model Output**

Computed output results at each mesh element and for each time step consist of (DHI, 2010):

• Basic variables

- Water depth and surface elevation
- Flux densities in main directions
- Velocities in main directions
- Densities, temperatures and salinities

• Additional variables

- Current speed and direction
- Wind velocities
- Air pressure
- Drag coefficient
- Precipitation/evaporation
- Courant/CFL number
- Eddy viscosity
- Element area/volume

The output results can be saved in defined points, lines and areas. Output from MIKE 21 Flow Model FM is typically post-processed using the Data Viewer available in the common MIKE Zero shell. The Data Viewer is a tool for analysis and visualization of unstructured data, e.g. to view meshes, spectra, bathymetries, result files in different format with graphical extraction of time series and line series from plan view and import of graphical overlays.

# CHAPTER 4

# FLOOD MODEL INPUTS

## 4.1 Mapping

The most important step of the hydraulic modeling procedure is the mapping. The required DEM was created from the elevation data obtained the field studies at the project site. Project area data includes very long river branch and side measurements. The study area has 26 km river line from Terme Bridge to upstream part of the Salıpazarı Bridge and 6 km part from the Black Sea coastline to Terme City Center (Terme Bridge). Digital elevation data for such a wide area was obtained from existing available data from related governmental organization.

Two types of data were obtained for model studies. The first elevation data could be defined as a base data, which represents the general modeling area. The 1/5000 scaled data was used for this purpose. Since the model area is mostly agricultural and the map elevation details (bigger grid size) for representing the flood areas do not have so much effect on study results, 1/5000 scaled data was found sufficient for model studies. However, river bed needs more detailed elevation values for modeling. The 1/1000 scaled field measurements of the Terme River bed bathymetry was obtained from DSI 7$^{th}$ Regional Directory. The bathymetry and bank level measurements were done for the most of the project line. Some parts were missing; however, rest was adequate for the study.

These two elevation values were overlapped and corrected for the final map of the model studies, which has 1/5000 scale outside the river and 1/1000 scale for river bed.

The 1/5000 scaled model data from Terme Bridge (Terme City entrance) to the Salıpazarı Bridge region includes land and agricultural flat areas. The elevation data was obtained from General Directorate of Land Registry and Cadaster. The source of the elevation data is aerial photography. The orthophotos of the area were obtained by General Directorate of Land Registry and Cadaster in 1/5000 scale. The spatial resolution of the aerial photos is 30 cm and the grid size of the elevation point data is 5 m. Both elevation values and the aerial photos of the project area were used in data preparation. The aerial photos were used for DEM correction. The obvious elevation differences especially at bank levels were corrected by the comparison of the aerial photos and the elevation data.

The indices and locations of 1/5000 scaled map are given in Figure 4.1 and Figure 4.2.

The 1/1000 scaled elevation data was obtained from DSI $7^{th}$ Regional Directory. The river bathymetry measurements and approximately 50 m left and right bank side measurements were obtained. The measurements represent the river bed as much as possible. The meandering parts of the river have more values than the straight parts of the river. The site measurements begin from Terme Bridge and it continues to the Salıpazarı Bridge. Some parts of the elevation values were missing, however, remaining were used for model studies. Figure 4.3 shows the extent of 1/1000 scaled data and missing part of the data.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | F37C_04_B | F37C_05_A | F37C_05_B | F38D_01_A | F38D_01_B |
| | | | | | F37C_04_C | F37C_05_D | F37C_05_C | F38D_01_D | F38D_01_C |
| | | | | F37C_09_A | F37C_09_B | F37C_10_A | F37C_10_B | F38D_06_A | F38D_06_B |
| | | F37C_08_D | F37C_08_C | F37C_09_D | F37C_09_C | | | | |
| | F37C_12_B | F37C_13_A | F37C_13_B | | | | | | |
| | F37C_12_C | F37C_13_D | | | | | | | |
| F37C_17_A | F37C_17_B | F37C_18_A | | | | | | | |
| F37C_17_D | F37C_17_C | | | | | | | | |

**Figure 4.1:** The indices and locations of the 1/5000 scaled maps

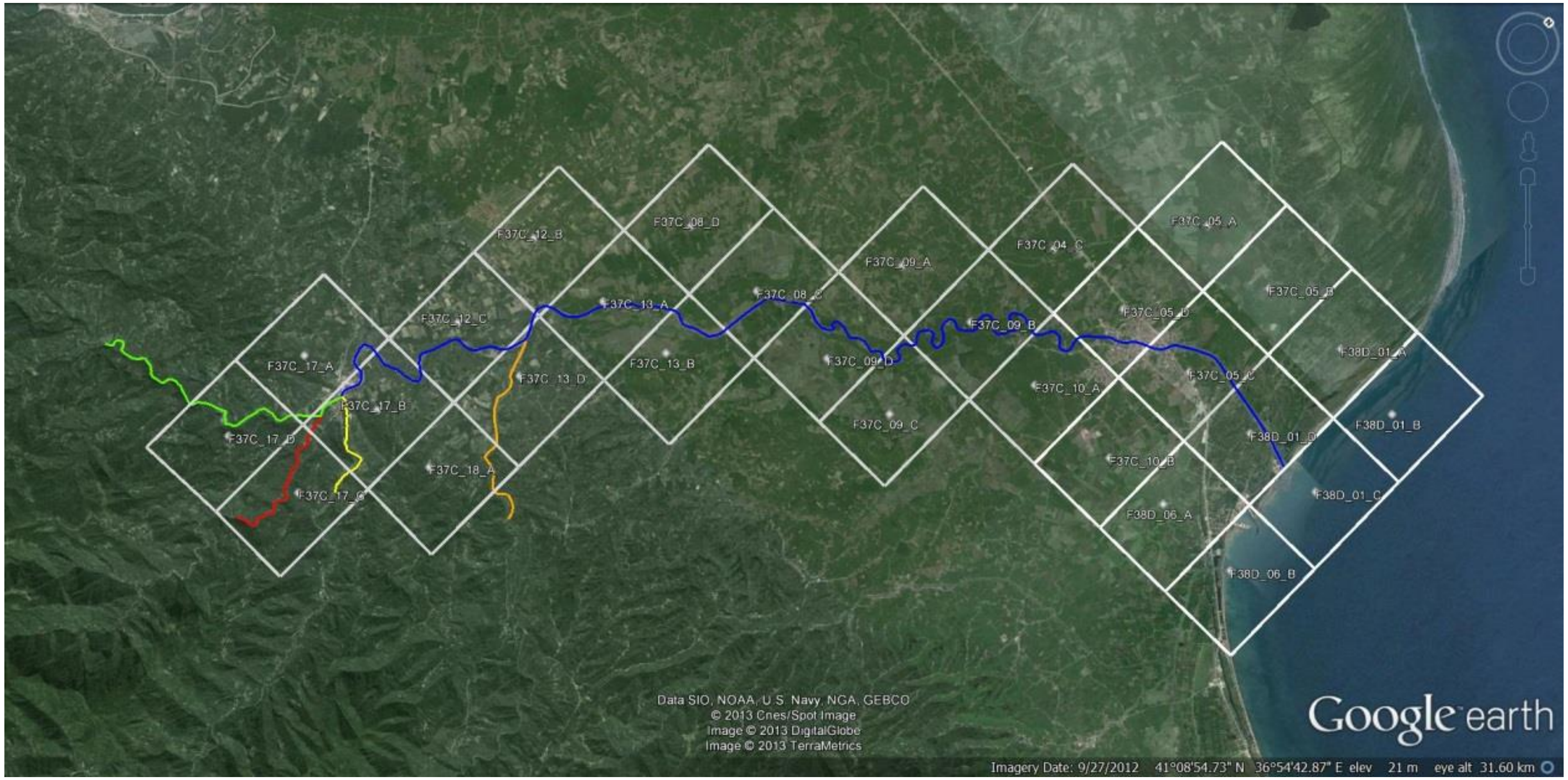


**Figure 4.2:** Representation of the project area with 1/5000 scaled maps


**Figure 4.3:** Terme River 1/1000 scaled mapping data

The obtained x-y-z point values from DSI and General Directorate of Land Registry were used to create a combined DEM. The CAD program was used for creating a triangulated irregular network data (TIN). TIN is an efficient way for representing continuous surfaces as a series of linked triangles. Two separate TINs were created for two separate data (1/1000 and 1/5000) and these data were overlapped for final model map. The final map was corrected and the connection parts were smoothened. This triangular elevation model was used as the base of the digital elevation model (DEM).

### 4.1.1 Mapping Procedure

Digital Elevation Model (DEM) is a type of raster GIS layer. In a DEM, each cell of raster GIS layer has a value corresponding to its elevation (z-values at regularly spaced intervals). DEM data files contain the elevation of the terrain over a specified area, usually at a fixed grid interval over the “Bare Earth”. The intervals between each of the grid points will be always referenced to some geographical coordinate system (latitude and longitude or projected coordinate UTM (Universal Transverse Mercator).

Digital elevation model can be created in many ways. One of the approaches for creating DEM is using survey data. The points all over the project area are read manually with GPS. The datum and x, y, z coordinates were stored in digital documents. Then these digital point values can be visualized directly on CAD program. The edge values for creating triangular surface model become ready with these data. The next step is creating a surface from the known x, y and z values. CIVIL 3D and GIS tools can be used to create a surface from point values.

Digital elevation model of the project site was created with the following steps;

- Digital point values were visualized using Auto-Cad. The layers including point elevation values were selected and a single layer for elevations was created and isolated.

- These points must include x, y and x values. Missing data were eliminated.

- Both CIVIL 3D and GIS tools were used for creating triangulated irregular network (TIN). CIVIL 3D was used for the first map (Terme River Bathymetry) and GIS was used for the second entire map since the area of the second map is much bigger than the first one. These TIN data include continuous surface values.

- TIN was exported to DEM as the next step. At this point continuous surface became a grid surface. Each cell of the created DEM has a value corresponding to its elevation. The resolution of the DEM can be changed through TIN to DEM conversion.

The model studies were carried out using single DEM. This is important for the accuracy of the model and compression of the results. However, DEM should be converted to MIKE elevation map format to be included in the model. The term “Bathymetry” is used for the DEM as model map of MIKE. Two different types of the bathymetry can be created for the two different types of model. MIKE 21 is single grid model and it uses grid base bathymetry for model calculations. MIKE 21 FM is flexible mesh model and it uses triangular mesh for model calculations. Since MIKE 21 FM was used in this study, the DEM was converted to the flexible mesh with MIKE ZERO Mesh Generator.

### 4.1.2 Model Map Generation

The model input map was generated from the combined 1/1000 and 1/5000 scaled DEM. The elevation values of the DEM were used as point elevation values of the MIKE ZERO Mesh Generator. Triangular mesh for the model was created from these elevation values. One of the advantages of the flexible mesh is creating different size of elements for different parts of the maps. These different sizes of the elements give advantages for modeling. The river bed was created with small area size triangles compared to the remaining parts (Figure 4.4). This means details of the river bed were represented better than the remaining parts. The flood plain part of the map does not require so many details since the studies do not focus on these parts. This element size differences also give advantage for the model calculation time. Model calculation time is directly related to the number of calculation nodes in the model. Each element represents one calculation node. Since the total calculation nodes are reduced with bigger size of elements at some parts, calculation time is reduced too. Figure 4.5 shows the model input map (bathymetry) for the studies.

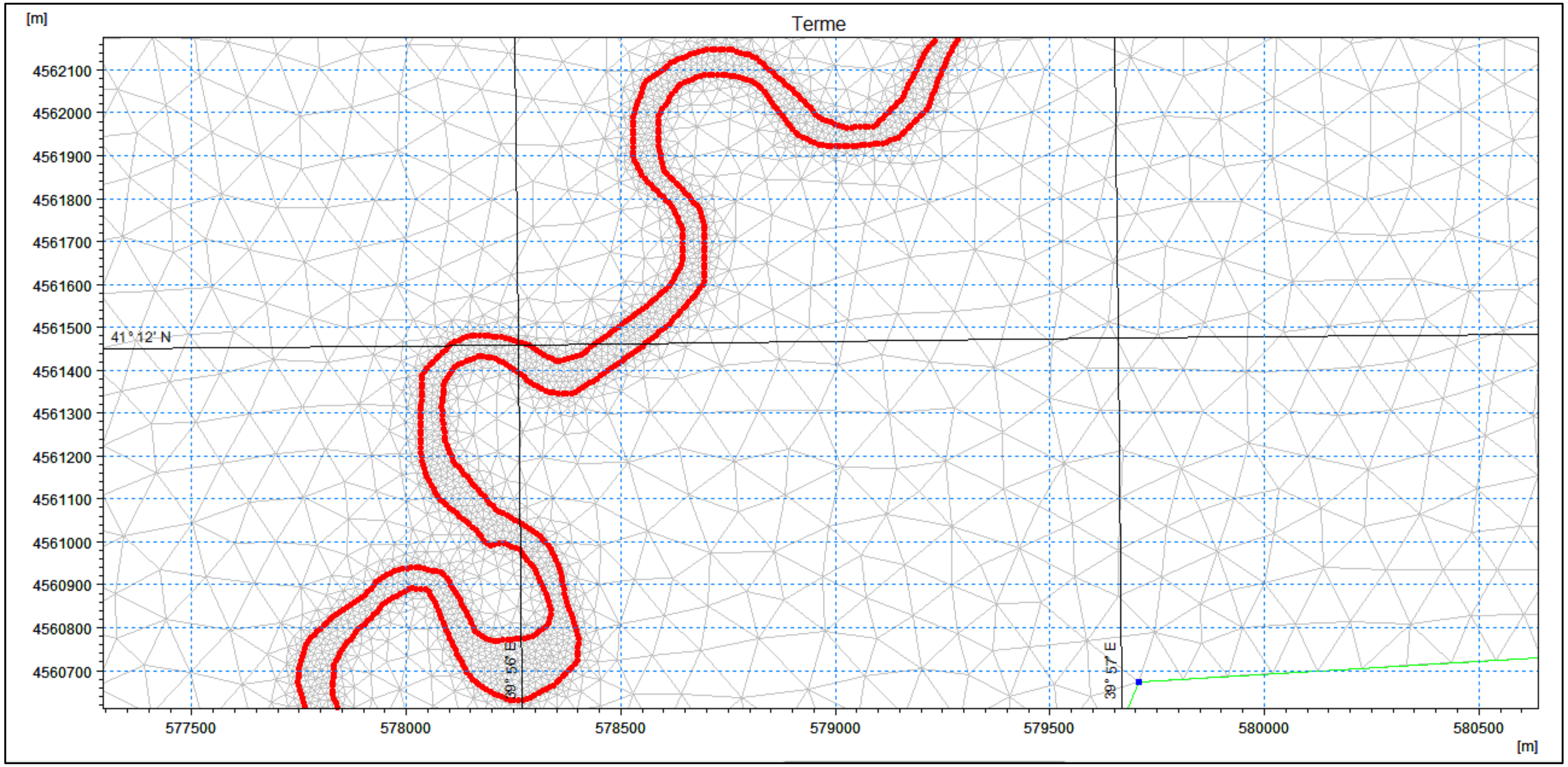


**Figure 4.4:** Flexiable mesh of the representative area

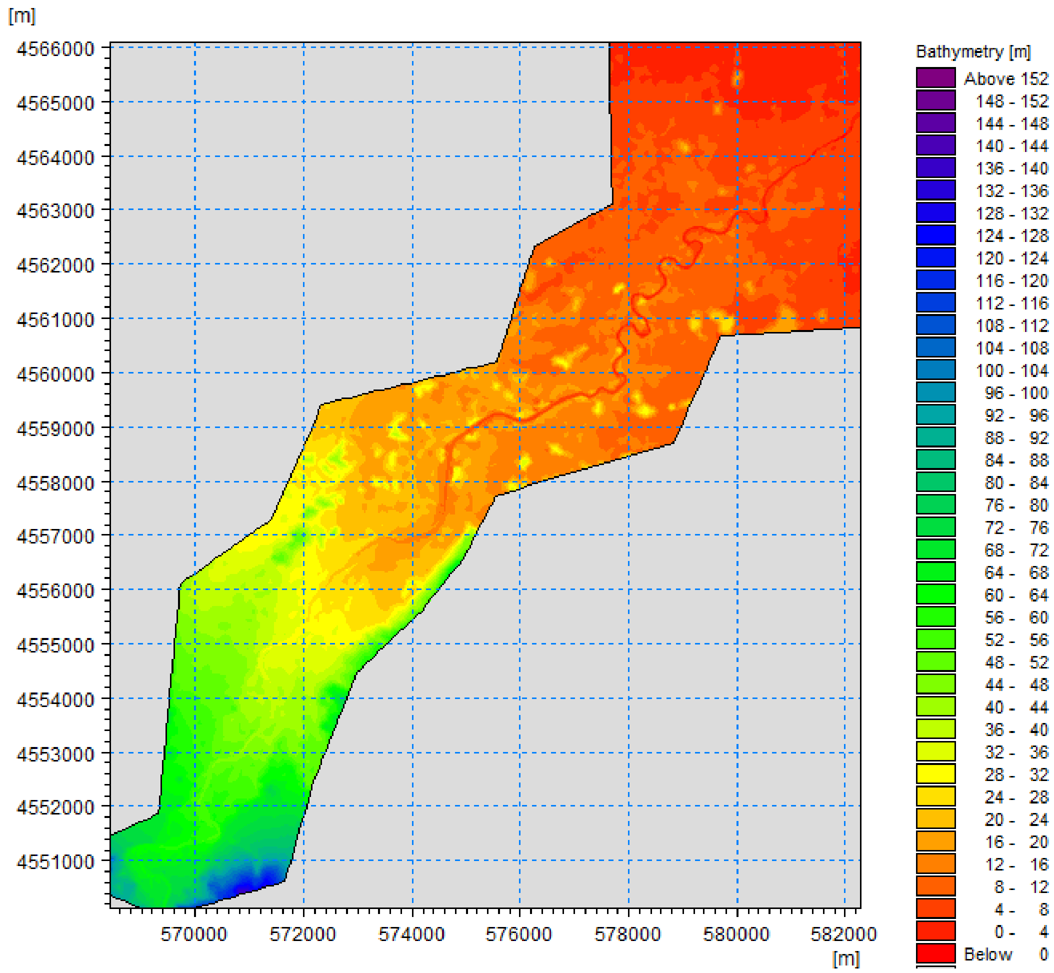


**Figure 4.5:** Bathymetry of the study area

## 4.2 Hydrological Data

The project area has seven meteorological stations and three stream gaging stations.

The meteorological stations in the basin are Düzdağ DSI, Terme DSI, Hasanuğurlu DSI, Çarşamba DMI, Kızılot DSI, Tekkiraz DMI and Akkuş DMI as shown in Figure 4.6.

The stream gaging stations in the basin are 2245 Gökçeali AGI, 22-02 Terme Bridge AGI and 22-105 Salıpazarı AGI stations and they are shown in Figure 4.7.


**Figure 4.6:** The location of meteorological stations

These stations were used in rainfall – runoff calculations by using DSI Synthetic (Direct Mockus) method. Representative thiessen polygon percentages and the operation years of the stations are given in Table 4.1.

**Table 4.1**: List of meteorological stations and area representative percentages

| **Station Name** | **Operation Years** | **Collected Data** | **Areal Representative Percentages** |
|---|---|---|---|
| **Düzdağ DSİ** | 1973-1998 | 26 years | 81.4% |
| **Terme DSİ** | 1963-1996 | 34 years | 12.5% |
| **Akkuş DMİ** | 1965-1991 | 27 years | 4.3% |
| **Hasanuğurlu DSİ** | 1974-1998 | 24 years | 0.8% |
| **Tekkiraz DMİ** | 1968-1978 | 10 years | 0.7% |
| **Çarşamba DMİ** | 1935-1991 | 57 years | 0.3% |

The DSI Synthetic method was used for the flood discharge calculations. However, the comparison between the peak discharges having different return periods obtained by flood frequency analyses and Synthetic method at 22-45 stream gaging station were not close to each other even if CN number was selected as 100 (Table 4.2). The table is obtained from the "Samsun Terme District, Terme River Flood Hazard Map Designation" report (DSI, 2013).

**Table 4.2:** Project area peak discharges compression (DSI, 2013)

| Project area peak discharges ($m^3/s$) | 2 Years | 5 Years | 10 Years | 25 Years | 50 Years | 100 Years | 500 Years |
|---|---|---|---|---|---|---|---|
| (Synthetic method) | 324.49 | 483.09 | 588.87 | 724.36 | 825.42 | **926.67** | **1163.17** |
| Point Flood Frequency method | 320.33 | 510.91 | 651.32 | 843.08 | 995.53 | **1155.29** | **1518.21** |

The reason for that can be the rainfall characteristics of the area. Mountainous area has regional precipitation patterns and may trigger the flash floods. In addition, the basin does not have enough meteorological stations to represent the basin precipitation and discharge characteristics. The observed discharge values from stream gaging stations were used to calculate the flood peak discharges for different return periods.

The stream gaging stations in the basin are 22-45 Gökçeali AGI, 22-02 Terme Bridge AGI and 22-105 Salıpazarı AGI. The locations of the stream gaging stations are shown in Figure 4.7.

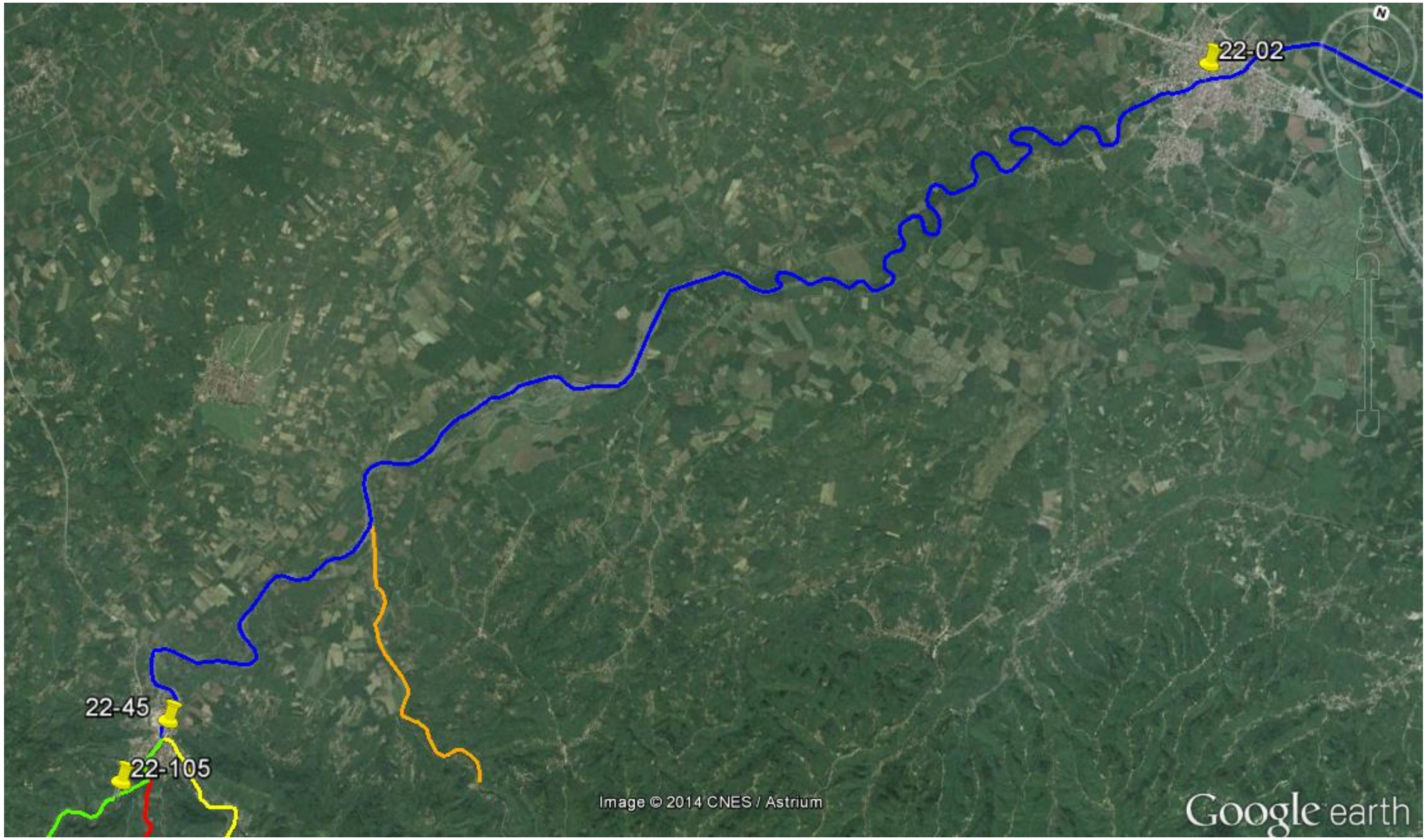


**Figure 4.7:** Discharge measurement stations

Gökçeali AGI has the longest measurement period (43 years) and the location is suitable for representing the project area. DSI 7$^{th}$ Regional Directorate stated that 22-02 Terme Bridge gaging station's discharge values are not suitable for representing the basin due to the meanders between 22-45 AGI and the 22-02 AGI. Total discharges at 22-02 are reduced if water leaves the channel before the gauge. The difference between 22-45 and the 22-02 from measurements are shown in Table 4.3.

**Table 4.3:** Discharge difference between 22-45 AGI and the 22-02 AGI

| | **2245 Gökçeli** | | **22-02 Terme Bridge** | | **Peak Discharge** |
|---|---|---|---|---|---|
| | | **Peak** | | **Peak** | **Flood Attenuation Ratio** |
| **No** | **Years** | **Discharges** | **Years** | **Discharges** | |
| | | **$m^3/s$** | | **$m^3/s$** | **%** |
| 1 | 1969 | 95,1 | 1969 | 91,0 | -4,3 |
| 2 | 1970 | 117,0 | 1970 | 130,0 | 11,1 |
| 3 | 1971 | 354,0 | 1971 | 250,0 | -29,4 |
| 4 | 1972 | 598,0 | 1972 | 250,0 | -58,2 |
| 5 | 1973 | 269,0 | 1973 | 340,0 | 26,4 |
| 6 | 1974 | 313,0 | 1974 | 220,0 | -29,7 |
| 7 | 1975 | 152,0 | 1975 | 165,0 | 8,6 |
| 8 | 1976 | 170,0 | 1976 | 145,0 | -14,7 |
| 9 | 1977 | 580,0 | 1977 | 290,0 | -50,0 |
| 10 | 1978 | 183,0 | 1978 | 150,0 | -18,0 |
| 11 | 1979 | 183,0 | 1979 | 200,0 | 9,3 |
| 12 | 1980 | 229,0 | 1980 | 200,0 | -12,7 |
| 13 | 1981 | 435,0 | 1981 | 260,0 | -40,2 |
| 14 | 1982 | 548,0 | 1982 | 330,0 | -39,8 |
| 15 | 1983 | 318,0 | 1983 | -- | |
| 16 | 1984 | 371,0 | 1984 | 270,0 | -27,2 |
| 17 | 1985 | 69,3 | 1985 | 89,0 | 28,4 |
| 18 | 1986 | 390,0 | 1986 | 270,0 | -30,8 |
| 19 | 1987 | 223,0 | 1987 | 170,0 | -23,8 |
| 20 | 1988 | 306,0 | 1988 | 240,0 | -21,6 |
| 21 | 1989 | 215,0 | 1989 | 190,0 | -11,6 |
| 22 | 1990 | 314,0 | 1990 | -- | |

**Table 4.3 (Cont.):** Discharge difference between 22-45 AGI and the 22-02 AGI

| | | | | | |
|---|---|---|---|---|---|
| 23 | 1991 | 155,0 | 1991 | 145,0 | -6,5 |
| 24 | 1992 | 290,0 | | | |
| 25 | 1993 | 140,0 | | | |
| 26 | 1994 | 96,2 | | | |
| 27 | 1995 | 387,0 | | | |
| 28 | 1996 | 124,0 | | | |
| 29 | 1997 | 143,0 | | | |
| 30 | 1998 | 234,0 | | | |
| 31 | 1999 | 154,0 | | | |
| 32 | 2000 | 311,0 | | | |
| 33 | 2001 | 88,2 | | | |
| 34 | 2002 | 273,0 | | | |
| 35 | 2003 | 132,0 | | | |
| 36 | 2004 | 200,0 | | | |
| 37 | 2005 | 94,2 | | | |
| 38 | 2006 | 465,0 | | | |
| 39 | 2007 | 193,0 | | | |
| 40 | 2008 | 138,0 | | | |
| 41 | 2009 | 148,0 | | | |
| 42 | 2010 | 507,0 | | | |
| 43 | 2011 | 221,0 | | | |

All these data showed that 22-45 Gökçeali AGI total monthly discharge values are sufficient for flood hydrograph calculations.

The studies were made for 2,5,10, 25, 50, 100 and 500-years flood discharges. The hydrographs for each peak discharges can be seen in Appendix A. Peak discharges for the 2245 Gökçeali AGI are shown in Table 4.4.

**Table 4.4**: Flood peak discharges 22-45 AGI

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **Q 2245 $m^3/s$** | 219,71 | 350,43 | 446,74 | 578,27 | 682,83 | 792,41 | 1041,34 |

The other flood discharges for sub-basins were calculated with the area ratio method. For example, 22-02 Terme Bridge peak flood discharges were calculated with the following formula.

$$Q_{Terme} = \left(\frac{436{,}4}{232{,}8}\right)^{0{,}6} \times Q_{2245} \qquad \mathbf{(4.1)}$$

Results are shown in Table 4.5.

**Table 4.5:** Area ratio between 22-45 AGI and Terme Bridge

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **2245 ($m^3/s$)** | 219,71 | 350,43 | 446,74 | 578,27 | 682,83 | 792,41 | 1041,34 |
| **Terme Bridge ($m^3/s$)** | 320,33 | 510,91 | 651,32 | 843,08 | 995,53 | 1155,29 | 1518,21 |

**Sub-basins**

There are four river branches creating the main stream at the project area. These four river branches' flows affect the main stream and the flow passing through Terme City region. Figure 4.8 shows the branches and the related sub-basins of the project area.


**Figure 4.8:** Terme River Sub-basins

The sub-basin characteristics were studied to see the hydrological similarities between the basins. GIS software was used for the analyses. After the sub-basin boundaries were created; area, perimeter, minimum elevation, maximum elevation, mean elevation and mean slope of the terrain values were calculated. The related values are given in Table 4.6.

**Table 4.6:** Sub-basin characteristics

| Basin | Mean Slope | Area (km²) | Min El. (m) | Max El.(m) | Mean El. (m) | Perimeter (m) |
|---|---|---|---|---|---|---|
| **1** | 45,77 | 75,14 | 70,00 | 1260,00 | 609,33 | 62175,49 |
| **2** | 45,38 | 46,66 | 70,00 | 1205,00 | 706,27 | 51298,73 |
| **3** | 46,86 | 109,96 | 70,00 | 1315,00 | 695,27 | 82210,36 |
| **4** | 40,47 | 134,88 | 35,00 | 1540,00 | 625,72 | 108244,26 |

Three of the sub-basins are represented with the stream gaging station. The Sub-basin 1 (B1) discharge is measured with the 22-105 AGI that is the downstream of the river branch. After Basin 1 and Basin 2 connection, Basin 3 joins to them. All these three basins are measured with the 22-45 Gökçeali AGI.

Table 4.6 shows that the sub-basins have similar properties. They have also common borders, so that the area ratio method can be used in calculating the flood discharges for the sub-basins of the project area.

**Snow cover areas**

Snowmelt due to melting of the snow is an important contribution to surface runoff. Water due to snowmelt has a significant role on annual water cycle.

The similarity of the sub-basins in the project area was analyzed from snow cover area distribution in the sub-basins. Since snow cover area contributes directly to hydrograph as snow melts, the sub-basins' snow cover area was determined using GIS analysis on annual basis. The snow cover area information was obtained from the NASA MODIS/Terra Snow Cover Daily L3 Global 500m Grid snow cover products (Hall et al., 2006). 82 Satellite images of the sub-basins were obtained for the study area. Daily snow cover area values were obtained from these images and annual snow covered areas were calculated. The analysis was performed for 13 years, between 2000-2013. The distribution of the data was not a normal distribution so medians of the data are also checked. Table 4.7 shows the snow cover area distribution for sub-basins.

**Table 4.7:** Snow covered area distribution of sub-basins for the years 2000-2013

| | **Basin 1** | | **Basin 2** | | **Basin 3** | | **Basin 4** | |
|---|---|---|---|---|---|---|---|---|
| | **Mean** | **Median** | **Mean** | **Median** | **Mean** | **Median** | **Mean** | **Median** |
| **2000** | 8,95 | 10,00 | 11,60 | 13,00 | 9,24 | 10,00 | 12,61 | 13,00 |
| **2001** | 6,09 | 6,00 | 7,54 | 8,00 | 5,36 | 5,00 | 5,65 | 5,00 |
| **2002** | 19,48 | 20,00 | 24,75 | 26,00 | 21,31 | 21,00 | 20,72 | 19,00 |
| **2003** | 8,55 | 9,00 | 11,72 | 13,00 | 8,35 | 9,00 | 7,71 | 7,00 |
| **2004** | 19,96 | 19,00 | 23,57 | 25,00 | 21,63 | 21,00 | 19,79 | 19,00 |
| **2005** | 10,92 | 11,00 | 15,16 | 16,00 | 12,37 | 13,00 | 11,15 | 11,00 |
| **2006** | 15,56 | 16,00 | 19,48 | 21,00 | 16,95 | 17,00 | 15,74 | 14,00 |
| **2007** | 14,52 | 14,00 | 21,26 | 24,00 | 15,51 | 16,00 | 14,93 | 14,00 |
| **2008** | 23,22 | 24,00 | 28,84 | 33,00 | 27,85 | 29,00 | 26,26 | 26,00 |
| **2009** | 10,96 | 12,00 | 14,07 | 16,00 | 11,97 | 12,00 | 10,96 | 11,00 |
| **2010** | 4,49 | 5,00 | 5,68 | 6,00 | 5,55 | 6,00 | 4,89 | 5,00 |
| **2011** | 10,76 | 11,00 | 18,24 | 21,00 | 12,61 | 13,00 | 11,71 | 10,00 |
| **2012** | 22,57 | 23,00 | 25,02 | 26,00 | 21,10 | 21,00 | 19,35 | 19,00 |
| **2013** | 11,00 | 11,00 | 12,99 | 13,00 | 12,83 | 13,00 | 13,29 | 13,00 |

The differences in the snow covered area amounts for the sub-basins are not significant. Therefore, we can say that four sub-basins have the same snow storage capacity.

**Defining the sub-basins flood peak discharges**

The basin characteristics of the sub-basins are considered as hydrologically similar. In addition to that, snow cover areas of the sub-basins are almost similar. Therefore, four sub-basins can be defined as hydrologically similar basins.

The stream gaging station 22-45 Gökçeali AGI covers the Basin 1, Basin 2 and Basin 3. Branch of Basin 4 connection to the main stream is just downstream of the Gökçeali AGI. Since all sub-basins have the similar characteristics and they are close to each other, area ratio method was used for calculating the flood discharges for the sub-basins.

The following Table 4.8, Table 4.9, Table 4.10 and Table 4.11 show the area ratio results of Basin 1, Basin 2, Basin 3 and Basin 4, respectively.

- $$Q_{Basin1} = \left(\frac{75{,}14}{232{,}8}\right) \times Q_{2245} \qquad (\mathbf{4.2})$$

**Table 4.8:** Flood peak discharges for Basin 1 for different return periods

| Basin 1 (area ratio results) (m³/s) | | | | | | |
|---|---|---|---|---|---|---|
| $Q_2$ | $Q_5$ | $Q_{10}$ | $Q_{25}$ | $Q_{50}$ | $Q_{100}$ | $Q_{500}$ |
| 70,92 | 113,11 | 144,19 | 186,64 | 220,39 | 255,76 | 336,11 |

- $$Q_{Basin2} = \left(\frac{46{,}66}{232{,}8}\right) \times Q_{2245} \qquad (\mathbf{4.3})$$

**Table 4.9:** Flood peak discharges for Basin 2 for different return periods

| Basin 2 (area ratio results) (m³/s) | | | | | | |
|---|---|---|---|---|---|---|
| $Q_2$ | $Q_5$ | $Q_{10}$ | $Q_{25}$ | $Q_{50}$ | $Q_{100}$ | $Q_{500}$ |
| 44,04 | 70,24 | 89,54 | 115,90 | 136,86 | 158,82 | 208,71 |

- $Q_{\mathrm{Basin3}} = \left(\frac{109{,}96}{232{,}8}\right) \times Q_{2245}$ **(4.4)**

**Table 4.10:** Flood peak discharges for Basin 3 for different return periods

| **Basin 3 (area ratio results) (m³/s)** | | | | | | |
|---|---|---|---|---|---|---|
| $Q_2$ | $Q_5$ | $Q_{10}$ | $Q_{25}$ | $Q_{50}$ | $Q_{100}$ | $Q_{500}$ |
| 103,78 | 165,52 | 211,01 | 273,14 | 322,53 | 374,28 | 491,86 |

- $Q_{Basin4} = \left(\frac{134{,}88}{232{,}8}\right) \times Q_{2245}$ **(4.5)**

**Table 4.11:** Flood peak discharges for Basin 4 for different return periods

| **Basin 4 (area ratio results) (m³/s)** | | | | | | |
|---|---|---|---|---|---|---|
| $Q_2$ | $Q_5$ | $Q_{10}$ | $Q_{25}$ | $Q_{50}$ | $Q_{100}$ | $Q_{500}$ |
| 127,30 | 203,03 | 258,83 | 335,04 | 395,62 | 459,11 | 603,33 |

### 4.2.1 Salıpazarı Dam Hydrology

Salıpazarı Dam project is planned to be located at the downstream part of Basin 1. The project is at the design stage and it is going to be constructed. DSI 7$^{th}$ Regional Directory provided the preliminary report for the thesis studies. The model studies were carried on both for existing situation (without dam construction) and projected situation (after dam construction). Figure 4.9 shows the dam water levels for different purposes.

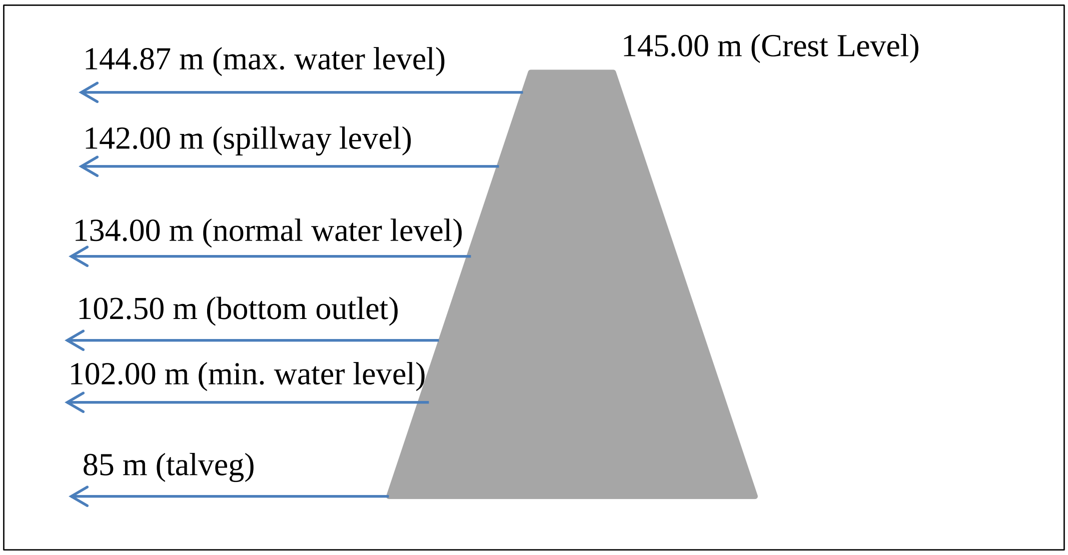


**Figure 4.9:** Salıpazarı Dam sketch

*The Given Dam Characteristics from DSI*

The purpose of the dam is irrigation, municipal water supply and flood protection. Minimum reservoir water level is 102,00 meter, maximum reservoir water level and crest level are 144,87 meter and 145,00 meter respectively. The dam type is roller compacted concrete (RCC). The spillway is at 142 m level, it is uncontrolled and width of the spillway is 40 m. The bottom outlet level is 102,5 m. Water intakes for irrigation and water supply are at the levels of 126,50 m, 118,50 m, 110,50 m and 102,50 m. The last intake (102,50 m) has the purpose of the reservoir level reduction. These technical details were obtained from DSI 7$^{th}$ Regional Directory.

The Stage-Area-Volume relation of the dam is given in Table 4.12.

**Table 4.12:** Elevation-Area-Volume relation of the Salıpazarı Dam

| Stage (m) | Area ($km^2$) | Volume ($hm^3$) |
|---|---|---|
| 85 | 0 | |
| 90 | 0,02 | 0,06 |
| 95 | 0,06 | 0,026 |
| 100 | 0,13 | 0,73 |
| 105 | 0,19 | 1,53 |
| 110 | 0,28 | 2,71 |
| 115 | 0,4 | 4,4 |
| 120 | 0,54 | 6,74 |
| 125 | 0,63 | 9,66 |
| 130 | 0,72 | 13,04 |
| 135 | 0,83 | 16,91 |
| 140 | 0,94 | 21,31 |
| 145 | 1,06 | 26,3 |
| 150 | 1,19 | 31,92 |
| 155 | 1,36 | 38,31 |

The reservoir volume at the normal operating level (134,00 m) is 15,90 $hm^3$ and the volume at the maximum water level (142,00 m) is 23,31 $hm^3$. The storage capacity of the reservoir between normal water level and the maximum water level is 7,41 $hm^3$. The $Q_{500}$ flood discharge brings 8,89 $hm^3$ water to the peak discharge (14,5 hours) and 19,01 $hm^3$ into the reservoir area at the first 24 hours of the hydrograph. Total flood hydrograph includes 29 $hm^3$ water.

The model studies for the $Q_{500}$ discharge were carried out with the reservoir volume consideration. It is assumed that when the water level of the reservoir is 134.00 m at the time of the $Q_{500}$ flood, both bottom outlet and the spillway operates. The calculation of the constant discharge for bottom outlet at flood event is given.

The bottom outlet was designed specially to reduce the reservoir volume (level of 134.00 m) in case of the flood. Table 4.13 gives the bottom outlet discharge with respect to reservoir level.

**Table 4.13:** Bottom outlet discharge vs. reservoir level

| Bottom Outlet Discharge | |
|---|---|
| Q ($m^3/s$) | Reservoir Water Level |
| 35 | 102.43 |
| 37.5 | 104.94 |
| 40 | 107.63 |
| 42.5 | 110.49 |
| 45 | 113.52 |
| 47.5 | 116.73 |
| 50 | 120.11 |
| 52.5 | 123.66 |
| 55 | 127.39 |
| 57.5 | 131.29 |
| 60 | 135.36 |
| 62.5 | 139.61 |
| 63 | 140.48 |
| 63.5 | 141.36 |
| 64 | 142.24 |
| 64.5 | 143.13 |
| 65 | 144.03 |
| 65.5 | 144.93 |

When the water level is above the 134,00 meter, and the bottom outlet operates, it discharges between 64 $m^3/s$ (approximate value at level 142 m) and 59,5 $m^3/s$ (approximate value at level 134,00 m).

Bottom outlet discharge was selected for the model studies with respect to Table 4.13. The possible situation is assumed for the model calculations and bottom outlet operates between 64 $m^3/s$ and 59,5 $m^3/s$ for this scenario. The constant 62 $m^3/s$ bottom outlet discharge was selected as the average downstream discharge for model studies.

The calculations for the reservoir volume reduction were obtained from the DSI 7$^{th}$ Regional Directory. Table 4.14 gives the level and bottom outlet discharges with remaining reservoir level.

**Table 4.14**: Reservoir flush

| T (hour) | Inflow Discharge ( $m^3/s$) | Reservoir Water Level | Outflow Discharge ( $m^3/s$) | Reservoir Volume ($hm^3$) |
|---|---|---|---|---|
| 0.00 | 4.1 | 142.00 | 64.00 | 23.31 |
| 24.00 | 4.1 | 139.40 | 62.37 | 20.79 |
| 48.00 | 4.1 | 133.68 | 58.97 | 15.89 |
| 72.00 | 4.1 | 127.43 | 55.02 | 11.31 |
| 96.00 | 4.1 | 120.61 | 50.36 | 7.10 |
| 120.00 | 4.1 | 111.99 | 43.75 | 3.38 |
| 138.71 | 4.1 | 102.50 | 35.06 | 1.00 |

Maximum water level (142,00 m) is reduced to the normal water level (134,00 m) in 48 hours.

## 4.3 Model Input Hydrographs

### Hydrograph 1

The model hydrograph was obtained for 2245 Gökçeali AGI for 2, 25, 50, 100 and 500-years return periods. Basin 1, Basin 2 and Basin 3 discharges cumulatively are represented with this hydrograph. As the model input, this hydrograph was applied from the Salıpazarı City Center.  Peak discharges for this hydrograph can be seen in Table 4.15.

**Table 4.15**: Model 2245 AGI flood discharges

| Years | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
|---|---|---|---|---|---|---|---|
| 2245 ($m^3/s$) | 219,71 | 350,43 | 446,74 | 578,27 | 682,83 | 792,41 | 1041,34 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 2**

This model hydrograph was obtained for Basin 1 (which is obtained from 2245 Gökçeali AGI with area ratio method) for 2, 5, 10, 25, 50, 100 and 500-years return periods. As a model input this hydrograph covers the Salıpazarı Dam river branch. Peak discharges for this hydrograph can be seen in Table 4.16.

**Table 4.16**: Basin 1 peak discharges

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **Basin 1 ($m^3/s$)** | 70,92 | 113,11 | 144,19 | 186,64 | 220,39 | 255,76 | 336,11 |

The hydrograph reaches peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 3**

This model hydrograph was obtained for Basin 2 (which is obtained from 2245 Gökçeali AGI with point area ratio method) for 2, 5, 10, 25, 50, 100 and 500-years return periods. Peak discharges for this hydrograph can be seen in Table 4.17.

**Table 4.17**: Basin 2 peak discharges

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **Basin 2 ($m^3/s$)** | 44,04 | 70,24 | 89,54 | 115,90 | 136,86 | 158,82 | 208,71 |

The hydrograph reaches its peak discharge in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 4**

This model hydrograph was obtained for Basin 3 (which is obtained from 2245 Gökçeali AGI with area ratio method) for 2, 5, 10, 25, 50, 100 and 500-years return periods. Peak discharges for this hydrograph can be seen in Table 4.18.

**Table 4.18**: Basin 3 peak discharges

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **Basin 3 ($m^3/s$)** | 103.78 | 165.52 | 211.01 | 273.14 | 322.53 | 374.28 | 491.86 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 5**

This model hydrograph was obtained for Basin 4 (which is obtained from 2245 Gökçeali AGI with point area ratio method) for 2, 5, 10, 25, 50, 100 and 500-years return periods. Peak discharges for this hydrograph can be seen in Table 4.19.

**Table 4.19**: Basin 4 peak discharges

| **Years** | **2** | **5** | **10** | **25** | **50** | **100** | **500** |
|---|---|---|---|---|---|---|---|
| **Basin 4 ($m^3/s$)** | 127,30 | 203,03 | 258,83 | 335,04 | 395,62 | 459,11 | 603,33 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 6**

This model hydrograph is the summation of the $Q_{500}$ discharges of the Basin 2 and Basin 3. In addition, Basin 1 was included in the summation hydrograph with constant 62 $m^3/s$ discharge and $Q_{500}$ spillway discharge. The hydrograph is prepared for the point of the Salıpazarı Bridge (2245 AGI). The aim of the hydrograph for model studies is simulating the situation when $Q_{500}$ flood discharge affecting the Basins and controlling Basin 1 by the Salıpazarı Dam through its bottom outlet and spillway. Peak discharge value for this hydrograph can be seen in Table 4.20.

**Table 4.20**: Hydrograph 6 peak discharge value

| **Years** | **500** |
|---|---|
| **Q ($m^3/s$)** | 790,23 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Hydrograph 7**

This model hydrograph is the summation of the $Q_{500}$ discharges of the Basin 2 and constant 62 $m^3/s$ discharges of Basin 3. In addition, Basin 1 was included in the hydrograph with constant 62 $m^3/s$ discharge and $Q_{500}$ spillway discharge. The hydrograph was prepared for the point of the Salıpazarı Bridge (2245 AGI). The aim of the hydrograph for model studies is simulating the situation when $Q_{500}$ flood discharge affecting the basins and Basin 2 is uncontrolled and remaining two basins have structures. Peak discharge value for this hydrograph can be seen in Table 4.21.

**Table 4.21**: Hydrograph 7 peak discharge value

| **Years** | **500** |
|---|---|
| **Q ($m^3/s$)** | 360,5 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0,05 hours (3 minutes).

**Hydrograph 8**

This model hydrograph is the summation of the $Q_{500}$ discharges of the Basin 3 and constant 62 $m^3/s$ discharges of Basin 2. In addition, Basin 1 was included to the hydrograph with constant 62 $m^3/s$ discharge and $Q_{500}$ spillway discharge. The hydrograph is prepared for the point of the Salıpazarı Bridge (2245 AGI). The aim of the hydrograph for model studies is simulating the situation when $Q_{500}$ flood discharge affecting the basins and Basin 3 is uncontrolled and remaining two basins have structures. Peak discharges for this hydrograph can be seen in Table 4.22.

**Table 4.22**: Hydrograph 8 peak discharge value

| **Years** | **500** |
|---|---|
| **Q ($m^3/s$)** | 643,7 |

The hydrograph reaches its peak discharges in 14,50 hours. The total base time of the hydrograph is 75 hours. Time step interval for Q-points is 0.05 hours (3 minutes).

**Salıpazarı Dam Bottom Outlet Constant Discharge**

The previous studies of Salıpazarı Dam Design project gives the bottom outlet discharge between 59,5 $m^3/s$ and 64 $m^3/s$ for the normal operating level and maximum water level respectively. The bottom outlet discharge is selected as constant 62 m³/s since the reservoir flood water level will change between 134 m and 142 m.

**Salıpazarı Dam $Q_{500}$ Flood Routing**

The previous studies of Salıpazarı Dam Design project gives the $Q_{500}$ flood discharge routing values. Since the spillway was designed for the catastrophic flood discharge, $Q_{500}$ flood routing downstream values are relatively small. Peak discharge of the $Q_{500}$ after flood routing is Q=27.18 (m³/s). The flood routing calculations (given by the DSI 7th Regional Directory) include the bottom release of the water (operating the bottom outlet of the dam). Since that Basin 1 discharge controlling by the dam, model study includes both $Q_{500}$ flood routing spillway hydrograph and bottom outlet discharge superposition.

Summary table of the model hydrographs can be seen in Table 4.23. This table indicates which hydrographs include which discharges of the basins as described before. Figure 4.10 also shows the locations of the basins on a sketch.

**Table 4.23**: Summary table of hydrographs

| Hydrograph No | Basin 1 | Basin 2 | Basin 3 | Basin 4 |
|---|---|---|---|---|
| **1** | ✓ | ✓ | ✓ | - |
| **2** | ✓ | - | - | - |
| **3** | - | ✓ | - | - |
| **4** | - | - | ✓ | - |
| **5** | - | - | - | ✓ |
| **6** | 62 m³/s + Spillway | ✓($Q_{500}$) | ✓($Q_{500}$) | - |
| **7** | 62 m³/s + Spillway | ✓($Q_{500}$) | 62 m³/s | - |
| **8** | 62 m³/s + Spillway | 62 m³/s | ✓($Q_{500}$) | - |

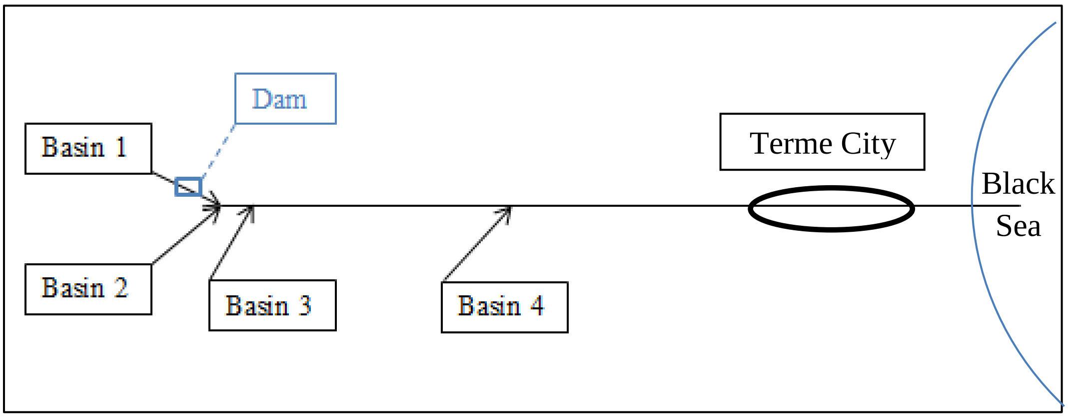


**Figure 4.10:** Sketch of study area

# CHAPTER 5

# MODEL STUDIES AND THE RESULTS

## 5.1 Model Studies

### 5.1.1 One-Dimensional Model Studies

In this study, 1-D modeling computations are applicable for the case where the water does not leave the main channel boundary. However, the study area bank level shows local up and downs which causes the water dissipation from some parts of the bed. Since the present study is focused on the $Q_{500}$ flood design discharge (causes the floods at some parts), 1-D modeling cannot represent the whole study area, 2-D modeling can be used for two-dimensional behavior areal distribution of the water in flood events.

One-dimensional modeling studies simulate the case where water does not leave the channel. The model area for this study was selected as the Terme Bridge part of the river (Figure 5.1). The aim was creating bank full flow inside the river bed. The previous reports and the studies stated that the capacity of the river at city center is approximately 510 $m^3/s$. This study aims to confirm that information.

The sinuosity of the river is important for the selection of the 1-D study area. Since the water over tapped at meandering part of the river, 1-D study area is selected from the straight part of the river. Equation 13 gives the sinuosity calculation. The channel length is 1950 m and the down valley length is 1850 m. The sinuosity value of the study area is 1,05 which means river path is almost straight.

$$SI = \frac{channel\ length}{downvalley\ length} \tag{5.1}$$

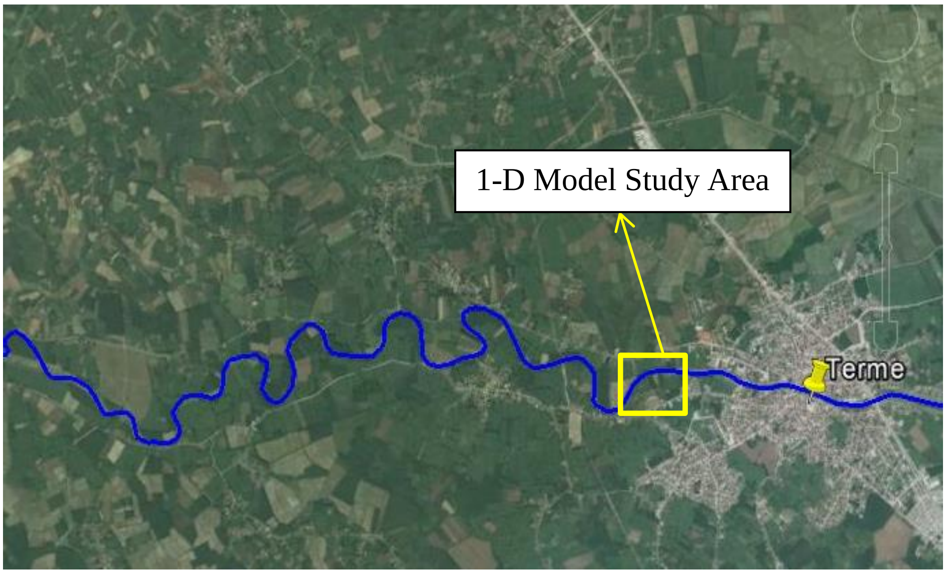


**Figure 5.1:** One-Dimensional model study area

The inputs for the 1-D modeling studies are river network and cross sections. Figure 5.2 shows the MIKE 11 river network and Figure 5.3 shows the cross sections along the river. Input boundary conditions are specified as different discharges and output boundary condition is specified as time dependent water level.

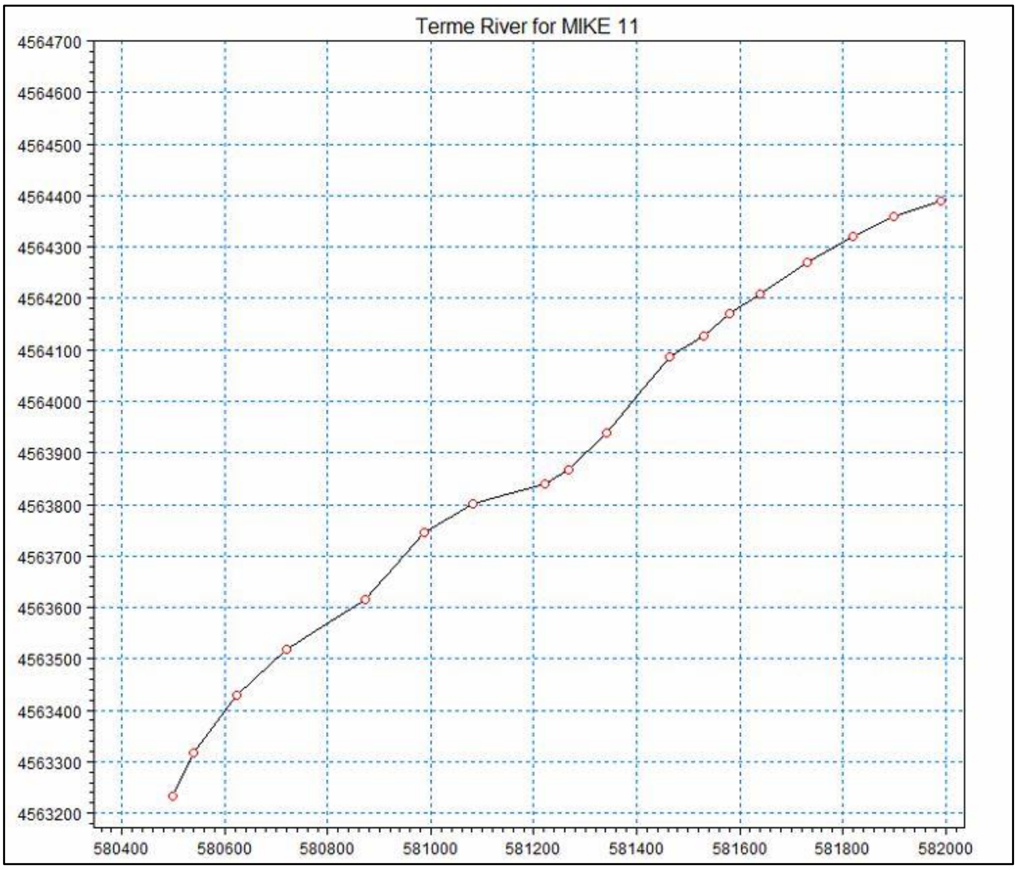


**Figure 5.2:** MIKE 11 Terme River network

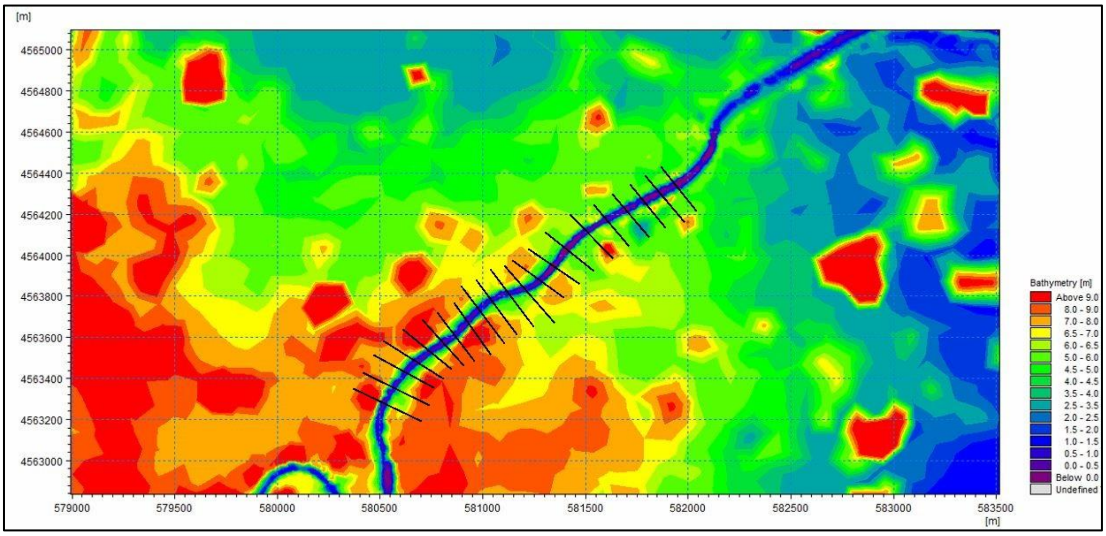


**Figure 5.34:** Terme River cross-section locations

For 1950 m river network, 21 cross sections are generated. The Manning value is used as n=0.025 and n=0.035 for model studies and the results show that discharge values are Q=480 $m^3/s$ and Q=530 $m^3/s$ respectively for bank full condition.

### 5.1.2 Two-Dimensional Model Studies

The model study area includes the Terme River from Salıpazarı City (2245 AGI Bridge) to Terme City (Terme Bridge). The model studies aim to compare model outflows at Terme City center. The outflow hydrograph calculation points were selected as lines. These lines were placed at the Terme River city center entrance and after Basin 4 connection. The following Figure 5.4 shows the outflow sections of the model study.

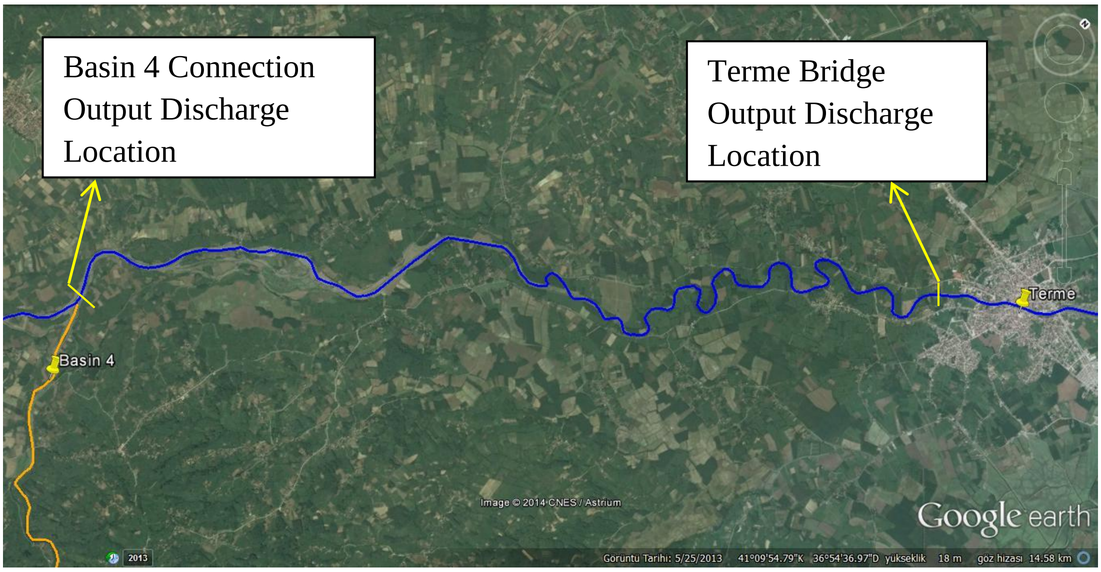


**Figure 5.4:** Model output discharges locations

Since the aim of the project was the comparison of the outflows at Terme Bridge; hydrographs were kept as variables depending on the scenarios but the bed resistance was kept constant. The constant model inputs were; input discharge points, output discharge points, bathymetry of the area, Manning's value, model boundary conditions and the other model calculation effects (viscosity, flood and dry, wind force, evaporation etc.). The scenario based main variable of the model was the input discharges. Input discharges were selected from the studied hydrographs and some constant discharges. The location of the input hydrographs to represent the Basin 1, Basin 2 and Basin 3 summation was selected as the Salıpazarı Bridge (2245 AGI). Basin 4 input hydrograph point was selected as Basin 4 and Terme River connection. The model output measure points were selected as shown in Figure 5.4.

The Manning's n value is taken from the "Samsun Terme District, Terme River Flood Hazard Map Designation" report (DSI, 2013). The value of n had been computed by Cowan's method with analyzing several primary factors affecting the roughness coefficient. Four material samples had been taken from the river bathymetry and representative Manning's n value had been calculated. One of the samples had been taken under the Terme Bridge and the n value had been calculated as n=0,029 with Cowan's method for that point. The other samples had been taken upstream part of the river and the average n value was calculated as 0,045.

The calibration for the model studies for Manning's value is also done for thesis studies. Bank full condition of the Terme City center and known discharge for that water level were used for calibration.

The inflow boundary conditions for the whole model studies are selected as scenario based hydrographs. The same input points are selected for different scenarios. Only input hydrographs are changed. The outflow boundary condition is selected as a free flow with assumption of a huge pond after outflow point. That means sea water level could not affect the stream flow. Also back watering did not affect the outflow stream condition. The study area is selected from Salıpazarı to Terme City entrance and the outflow hydrographs are calculated at Terme Bridge.

The model scenarios are created for three different situations. The first one existing situation includes the current condition of the study area. The second one is about application of Salıpazarı Dam Project, which is under final planning stage. The last one is hypothetical structures, which are aimed to present possible solutions for the study area. All the model result hydrographs are given in Appendix B.

### Scenario 1

This scenario includes four different model studies for flood hydrographs having different return periods namely 25, 50, 100 and 500 years. The aim of the studies is to see the input hydrograph peak discharge and output hydrograph peak discharge differences due to the meanders effect. The input point is selected as the Salıpazarı (22-45 Gökçeali AGI) and output point is selected as the Terme City center (Terme Bridge). The DSI report at the date of 11/07/2012 says; the flood event at the day of 09/07/2012 was measured as 990 $m^3/s$ (22-45 Gökçeali AGI) and discharge was measured as 510 $m^3/s$ at city center.

This scenario represents the existing situation of the river and the basins. Since the comparison of discharge measurements between Salıpazarı Bridge and the Terme Bridge is the main subject, only the basins, which are at the upstream part of the Salıpazarı Bridge, are included (Basin 1, Basin 2 and Basin 3). The sub-basin between Salıpazarı Bridge and Terme Bridge (Basin 4) is not included in this scenario. Figure 5.5 shows the sketch of the network for the scenario base.

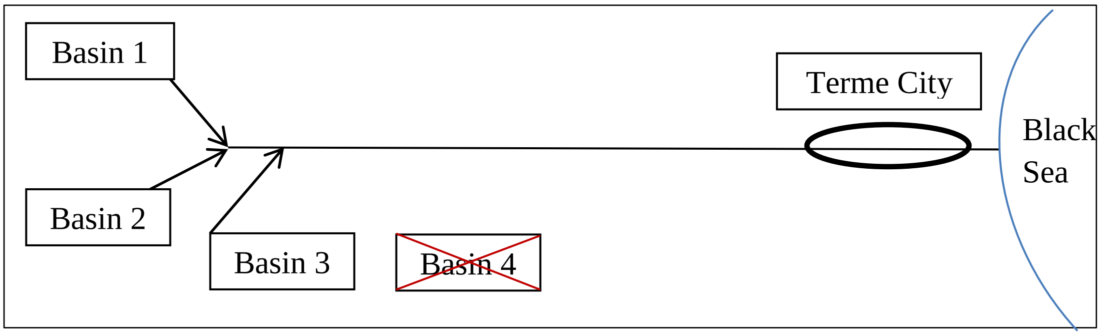


**Figure 5.5:** Sketch of Scenario 1

The following Table 5.1 and Table 5.2 show model input information and the model output results respectively. In addition, Figure 5.6 shows the related input and output hydrographs of the model. Figure 5.7 shows flood water depth as one of the model results.

**Table 5.1:** Scenario 1 model information

| | |
|---|---|
| Input hydrograph location | 22-45 AGI |
| Output hydrograph location | Terme Bridge |
| Bed resistance (1/n) | 32 |
| Input Hydrograph | Hydrograph 1 |

**Table 5.2:** Scenario 1 model results

| Return Period | Input Hydrograph Peak Discharges ($m^3/s$) **(1)** | Output Hydrograph Peak Discharges ($m^3/s$) **(2)** | Attenuations ($m^3/s$) **(3)=(1)-(2)** | Percentage of Difference **(4)=(3) / (1)** |
|---|---|---|---|---|
| $Q_{25}$ | 578,27 | 472,60 | 105,67 | % 18 |
| $Q_{50}$ | 682,83 | 535,40 | 147,83 | % 22 |
| $Q_{100}$ | 792,41 | 573,40 | 219,01 | % 28 |
| $Q_{500}$ | 1041,34 | 619,00 | 422,34 | % 41 |

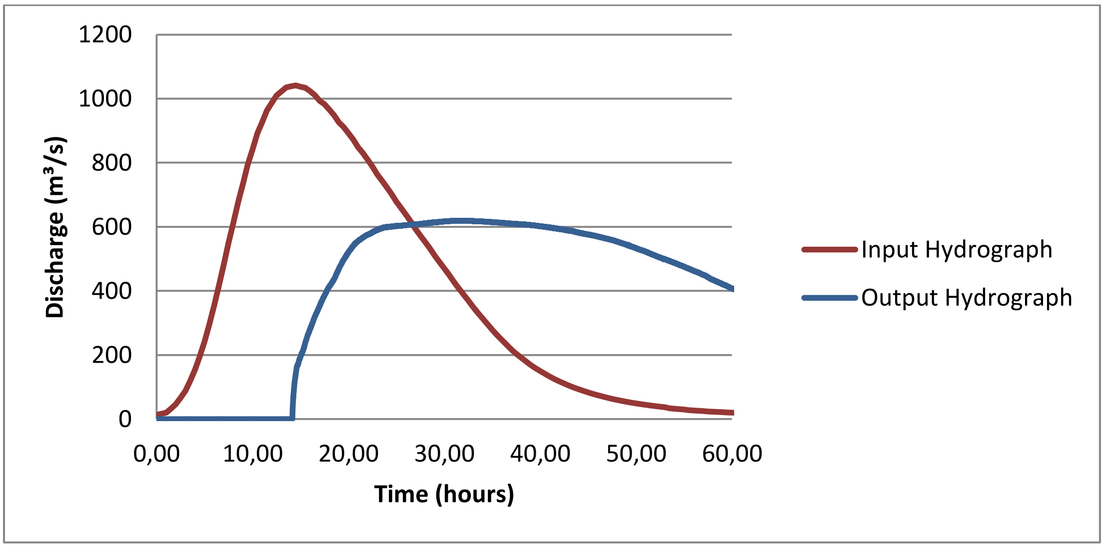


**Figure 5.6:** Scenario 1 Input – Output hydrographs of the model

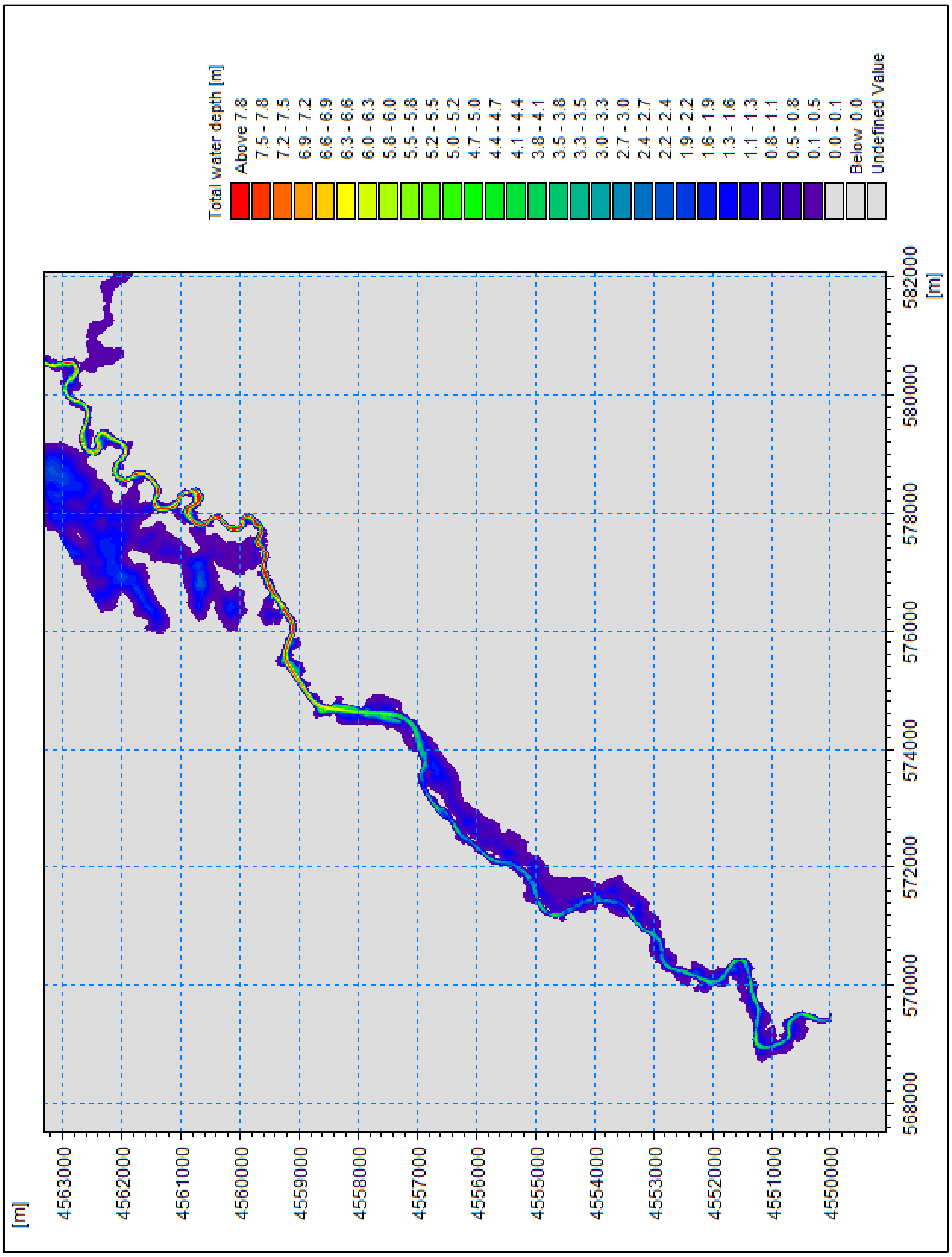


**Figure 5.7:** $Q_{500}$ water depth for Scenario 1

The model results state that at the downstream part of the stream between Salıpazarı City and the Terme City, the meanders affect the discharge values. The routing capacity of the meanders at the study area can be seen from the peak discharge differences. Figure 5.7 shows that at some parts of the river, water leaves the river bank and spreads over the open field. This causes the discharge reduction at the Terme City. The channel capacity in the Terme City center is approximately 500 $m^3/s$. Results show that if $Q_{500}$ passes through Salıpazarı Bridge and Basin 4 does not contribute the Terme River with any flood discharge, then channel capacity will be approximately sufficient in Terme City.

**Scenario 2**

This scenario consists of one model. The aim of the study is to include sub-basin discharge (Basin 4) between Terme Bridge and Salıpazarı Bridge. The model studies are carried out only for $Q_{500}$ flood discharge, which is used as the design discharge at the project studies.

This scenario represents the existing situation. It includes whole basins in the study area. Hydrograph 1 is used as input hydrograph to represent the Basin 1, Basin 2 and Basin 3. Addition to that, Hydrograph 5 is used as input to represent the Basin 4. Both of the Hydrographs reach the peak discharges at the same time individually. However, since the hydrograph input points are not the same, peak discharges do not overlap. Otherwise, it is needed to apply individual modeling for the Basin 4 to calculate peak discharge time. Figure 5.8 shows the sketch of the network for the scenario base.

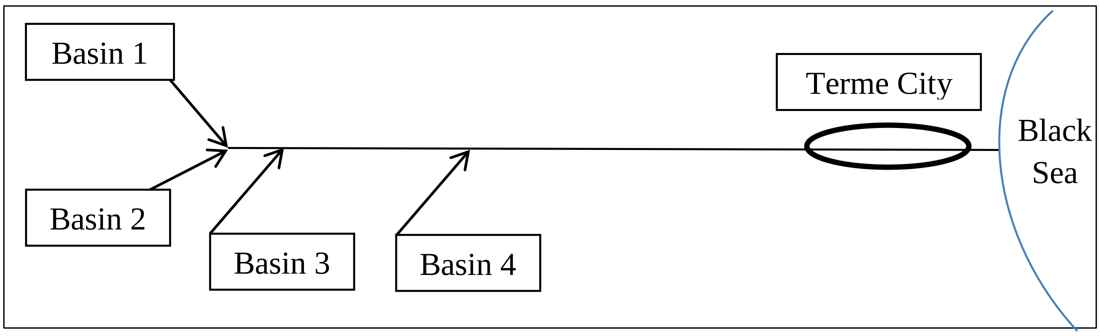


**Figure 5.8:** Sketch of Scenario 2

Table 5.3 and Table 5.4 show model input information and the model output results respectively. Figure 5.9 also shows the model flood water depth results. Figure 5.10 shows the related input and output hydrographs of the model.

**Table 5.3:** Scenario 2 model information

| | | |
|---|---|---|
| Input hydrograph location(s) | 22-45 AGI | Basin 4 Connection |
| Output hydrograph location | Terme Bridge | After Basin 4 |
| Bed resistance (1/n) | 32 | - |
| Input Hydrograph(s) | Hydrograph 1 | Hydrograph 5 |

**Table 5.4:** Scenario 2 model results

| Return Period | Input Hydrograph Peak Discharge ($m^3/s$) | | Output Hydrograph Peak Discharge ($m^3/s$) | | Attenuations ($m^3/s$) | Percentage of Difference |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | **(5)=(3)-(4)** | **(6)=(5)/(3)** |
| $Q_{500}$ | 1041,34 | 603,33 | 1340,00 | 710,80 | 629,20 | %47 |

The model results show that Basin 4 participation to the Terme River has a major effect on the Terme City flood. Even if the peak discharges are not overlapping, Basin 4 has the highest $Q_{500}$ value compared to the other three basins and it has important effect on the results. Since the meandering is effective after Basin 4 connection and peak discharges are not overlapping, peak discharge difference is calculated between Basin 4 connection and the Terme City. Figure 5.9 shows the massive water spreading out of the river bank after Basin 4 connection.

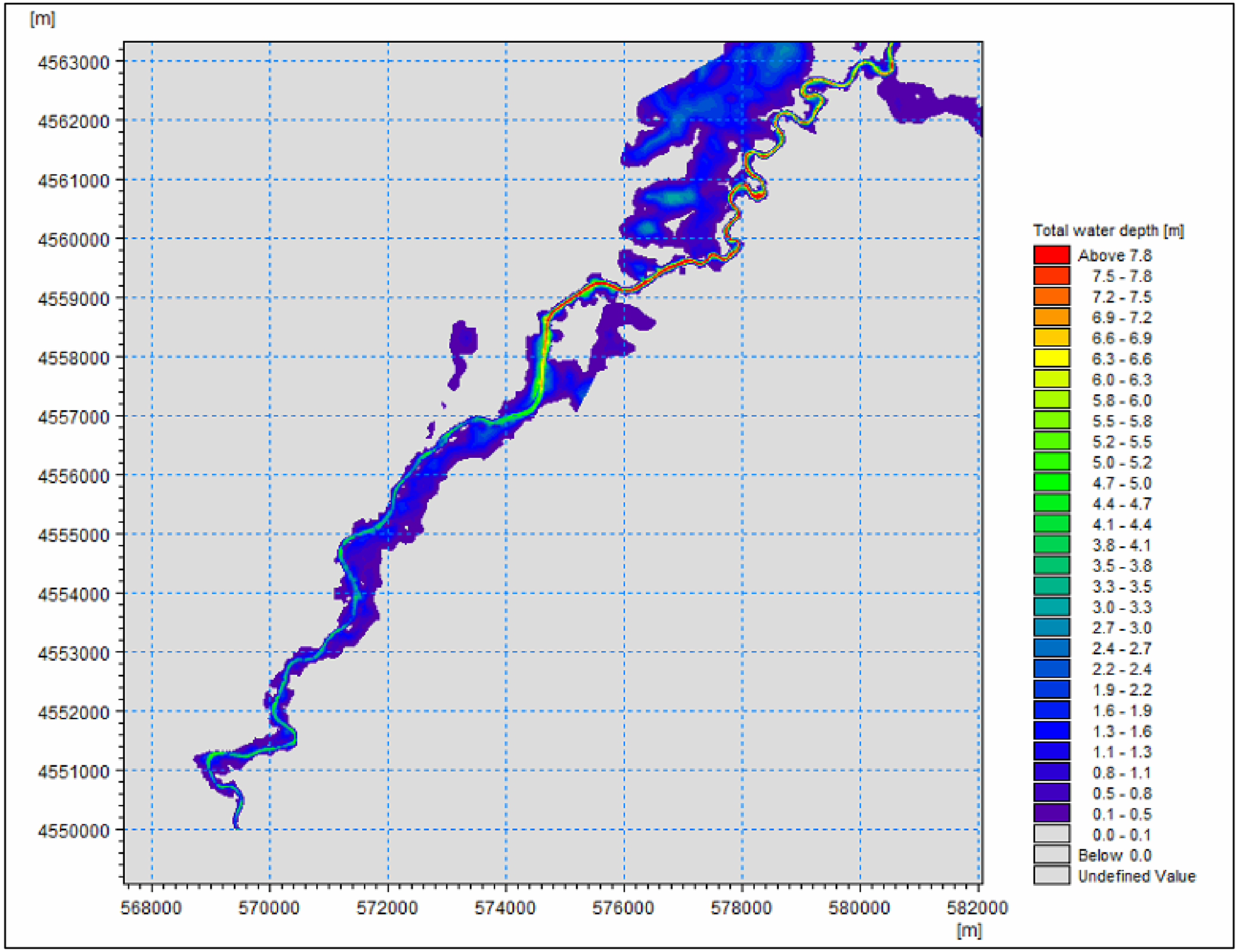


**Figure 5.9:** $Q_{500}$ water depth for Scenario 2

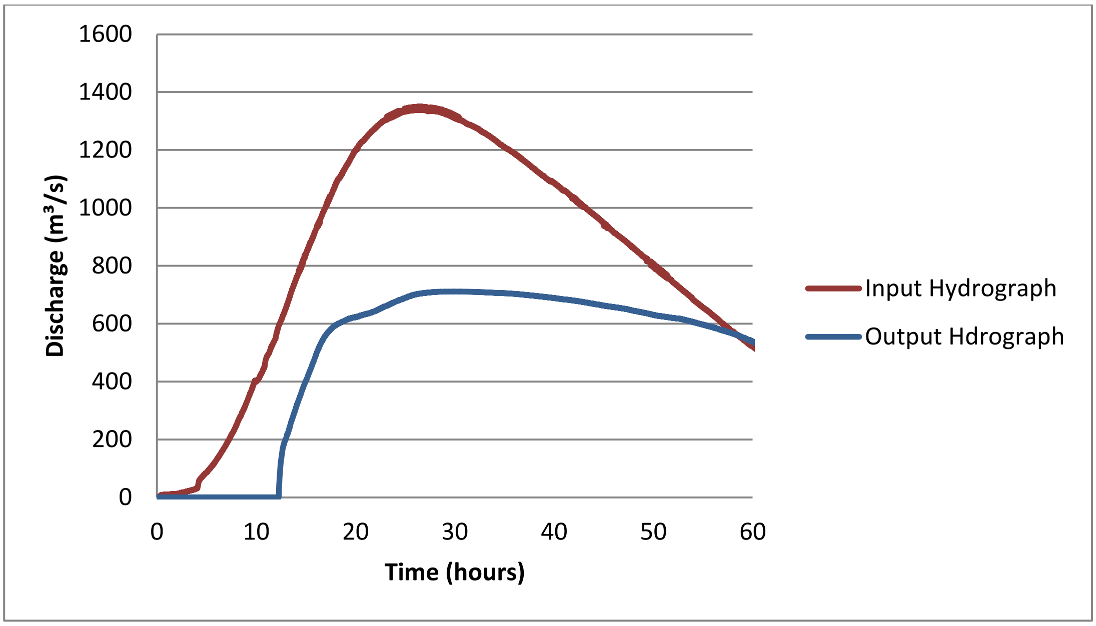


**Figure 5.10:** Scenario 2 Input – Output hydrographs of the model

**Scenario 3**

This scenario represents the Salıpazarı Dam project constructed situation. Salıpazarı Dam project includes flood capacity so it has effect on the Basin 1 output discharge. The model studies were based on the Salıpazarı Dam Design Project preliminary report information that was given by the DSI 7$^{th}$ Regional Directory for thesis studies.

The aim of the study is to include the Salıpazarı Dam flood capacity for model calculations. Since the studies are based on the preliminary project of the dam, these models can be called as projected situation.

This scenario represents the situation after the Salıpazarı Dam construction. Since the location of Salıpazarı Dam is at the downstream part of the Basin 1, hydrological studies were changed only for Basin 1. The other basins' hydrographs were kept the same with the existing situation. Figure 5.11 shows the sketch of the network for the scenario base.

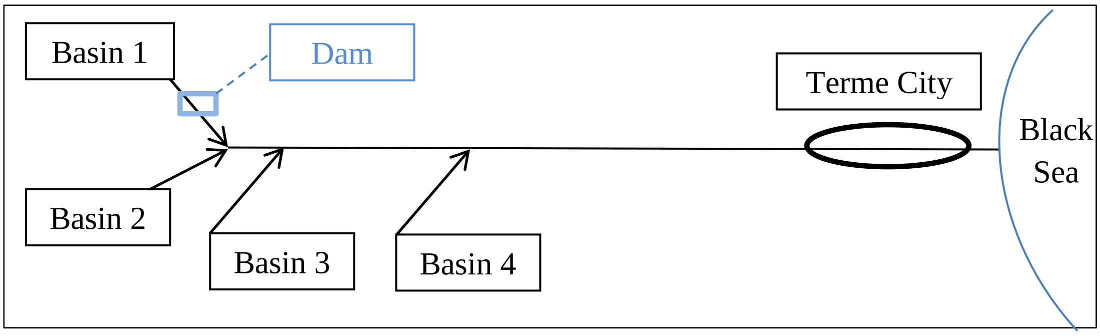


**Figure 5.11:** Sketch of Scenario 3

The model studies were carried out for $Q_{500}$ flood discharge situation. Since the other basins keep existing situations, only the hydrograph of Basin 1 is changed. The new hydrograph includes the value of 62 $m^3/s$ constant bottom outlet flow and the spillway design discharge for $Q_{500}$

Hydrograph 6 is used as input hydrograph to represent Basin 1, Basin 2 and Basin 3. In addition to that, hydrograph 5 is used as input to represent Basin 4. Both of the hydrographs reach the peak discharges at the same time individually. However, since the hydrograph input points are not the same, peak discharges do not overlap.

Table 5.5 and Table 5.6 shows model input information and the model output results respectively. Figure 5.12 shows the related input and output hydrographs of the model and Figure 5.13 shows the model flood water depth result.

**Table 5.5:** Scenario 3 model information

| | | |
|---|---|---|
| Input hydrograph location(s) | 22-45 AGI | Basin 4 Connection |
| Output hydrograph location | Terme Bridge | After Basin 4 |
| Bed resistance (1/n) | 32 | - |
| Input Hydrograph(s) | Hydrograph 6 | Hydrograph 5 |

**Table 5.6:** Scenario 3 model results

| Return Period | Input Hydrograph Peak Discharge (m³/s) | | Output Hydrograph Peak Discharge (m³/s) | | Attenuations (m³/s) | Percentage of Difference |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | **(5)=(3)-(4)** | **(6)=(5)/(3)** |
| $Q_{500}$ | 790,23 | 603,33 | 1153,00 | 681,80 | 471,20 | 41 % |

The model result showed that even if the Salıpazarı Dam is constructed with the planned flood capacity, it is not sufficient to protect Terme City against flooding for the condition of the whole basins are affected from the flood at the same time.

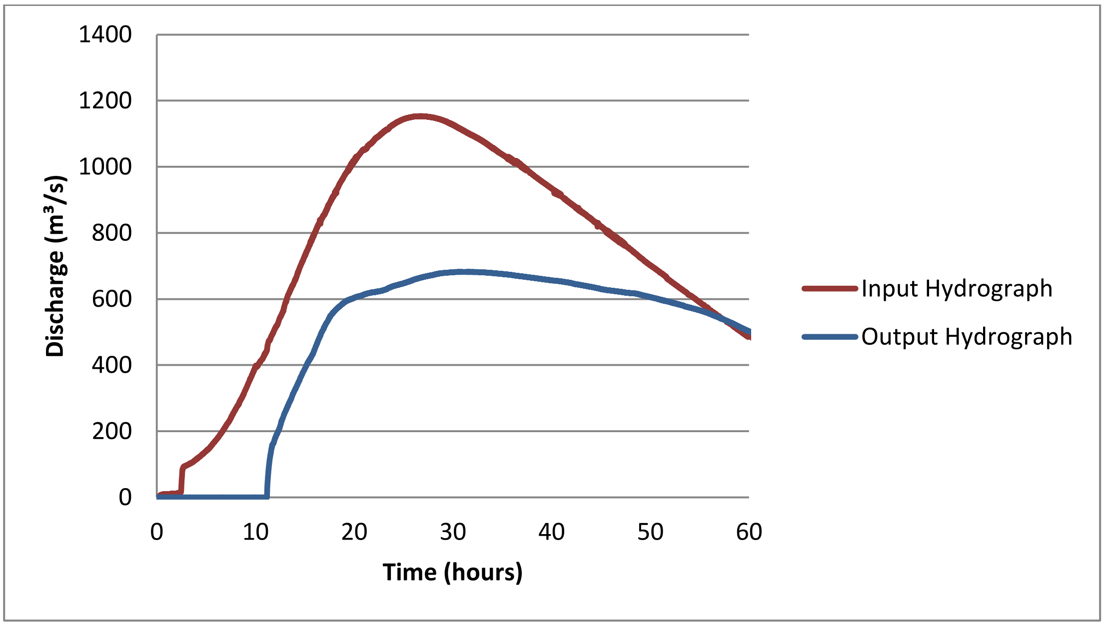


**Figure 5.12:** Scenario 3 Input – Output hydrographs of the model

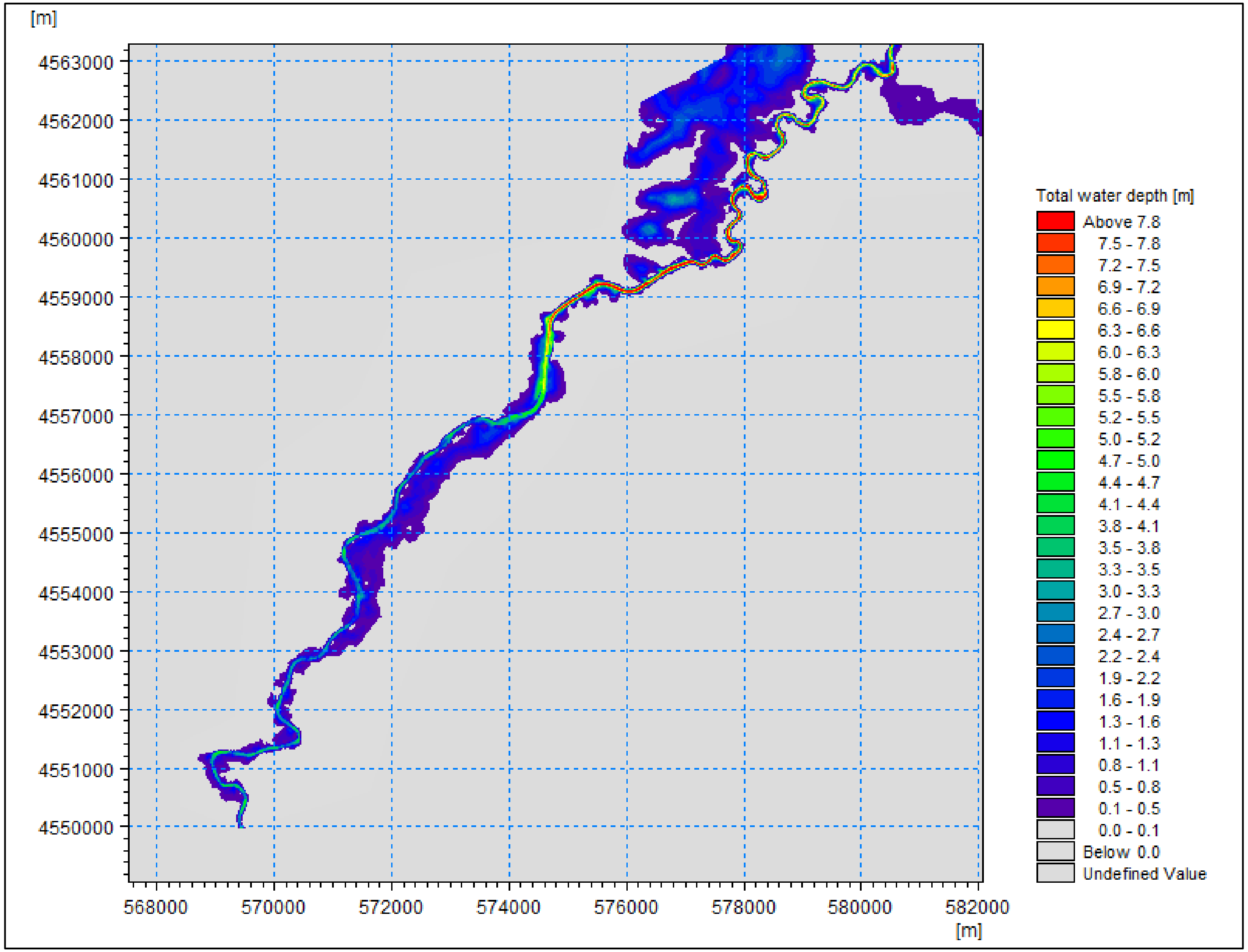


**Figure 5.13:** $Q_{500}$ water depth for Scenario 3

**Scenario 4**

This scenario represents the Salıpazarı Dam design project constructed situation and possible future projects for remaining sub-basins.

Since the Scenario 3 showed that the Salıpazarı Dam flood capacity is not sufficient at the time of the other three sub-basins, which are also affected from the flood, the aim of this scenario is controlling the whole flood discharges of the Basin 2 and Basin 3 in addition to Salıpazarı Dam flood capacity for model calculations.

This scenario represents the possible solutions at the basins for the upstream part of the Salıpazarı City. Basin 1 could be controlled with Salıpazarı Dam and only bottom outlet discharge (62 $m^3/s$) is included in the model. The assumption of this scenario is controlling the whole $Q_{500}$ flood capacity of the Basin 2 and Basin 3 with upstream hypothetical control structures that means Basin 2 and Basin 3 have no effect with their discharges. Controlled discharges from these basins are considered, and the Basin 4 contribution remains the same with the existing situation.

The model studies were carried out for $Q_{500}$ flood discharge situation. Since the Basin 4 contribution remains the same with the existing situation, the hydrograph of Basin 4 is used directly. The remaining basins are represented with a constant bottom outlet discharge of 62 $m^3/s$ in the model. Figure 5.14 shows the sketch of the network for the scenario base.

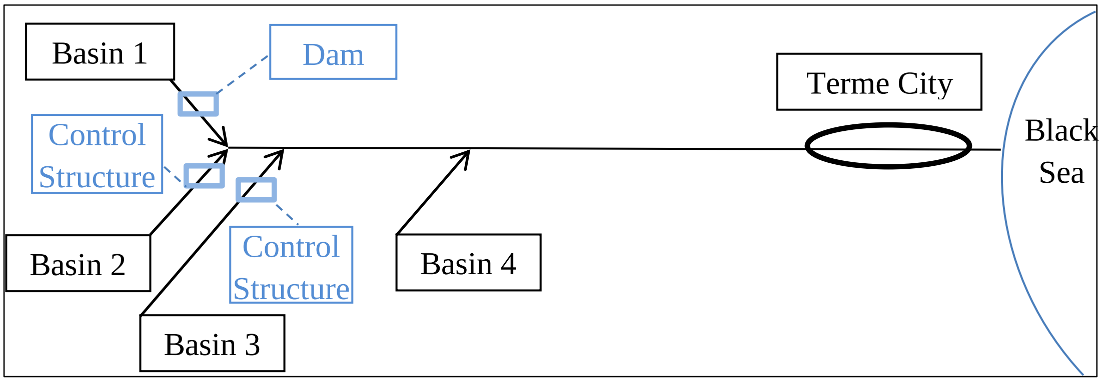


**Figure 5.14:** Sketch of Scenario 4

Table 5.7 and Table 5.8 shows model input information and the model output results respectively. Figure 5.15 also shows the model flood water depth result. Figure 5.16 shows the related input and output hydrographs of the model.

**Table 5.7:** Scenario 4 model information

| | | |
|---|---|---|
| Input hydrograph location(s) | 22-45 AGI | Basin 4 Connection |
| Output hydrograph location | Terme Bridge | After Basin 4 |
| Bed resistance (1/n) | 32 | - |
| Input Hydrograph(s) | Constant 62 $m^3/s$ | Hydrograph 5 |

**Table 5.8:** Scenario 4 model results

| Return Period | Input Hydrograph Peak Discharge ($m^3/s$) | | Output Hydrograph Peak Discharge ($m^3/s$) | | Attenuation ($m^3/s$) | Percentage of Difference |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | **(5)=(3)-(4)** | **(6)=(5)/(3)** |
| $Q_{500}$ | 62 | 603,33 | 658,23 | 527,4 | 130,83 | %20 |

The model results show that if upstream precautions are applied before the Salıpazarı Bridge, the $Q_{500}$ flood discharge from Basin 4 can pass from the Terme city center safely.

The inputs of the scenario are Basin 4 $Q_{500}$ flood discharge and the rest of the basins contribute 62 $m^3/s$ discharge from Salıpazarı Bridge. So that this scenario can also be interpreted as flood is due to the Basin 4 hydrograph and the rest of the basins contributing flooding with small amount of discharges. For such a situation Basin 4 individually creates bank full flow at Terme City center. This also shows the importance of the Basin 4 in flood studies.

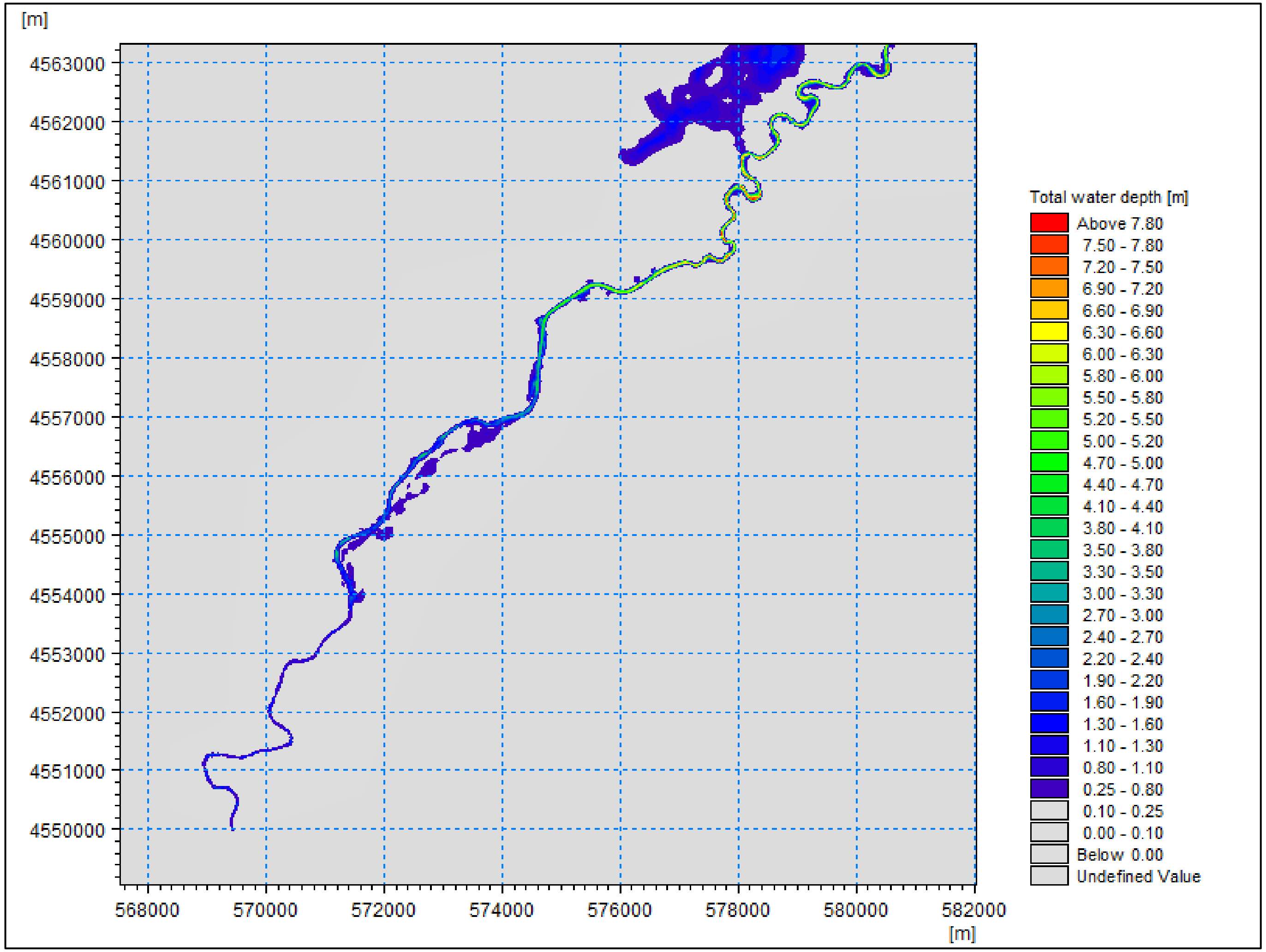


**Figure 5.15:** $Q_{500}$ water depth for Scenario 4

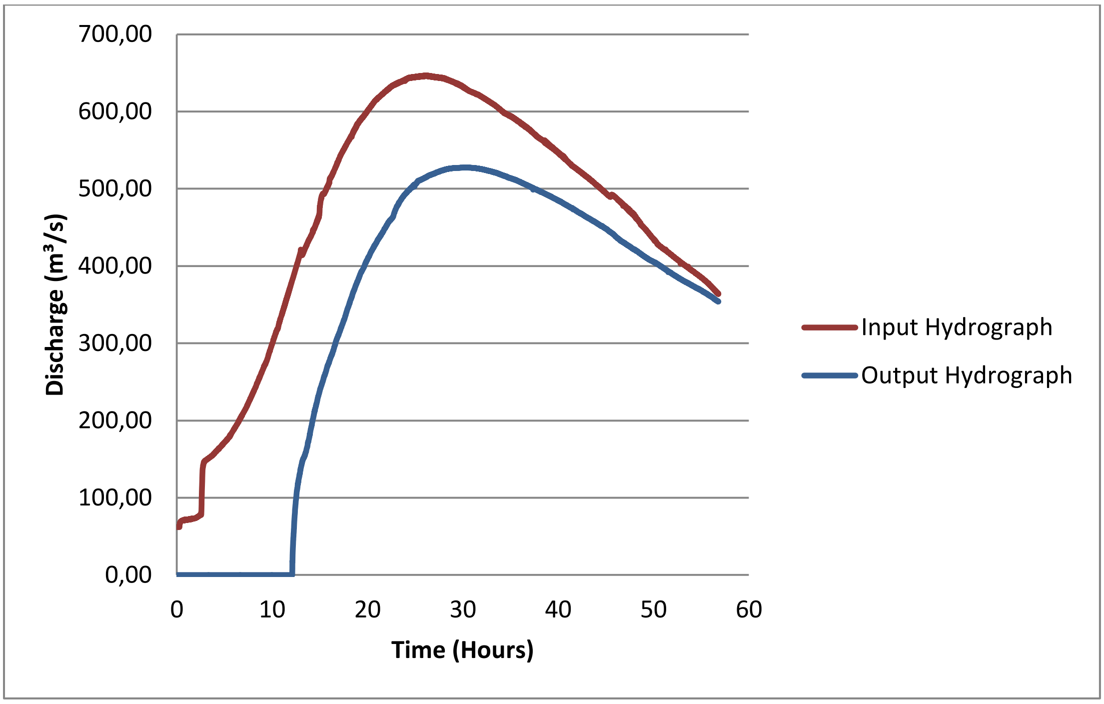


**Figure 5.16:** Scenario 4 Input – Output hydrographs of the model

### Scenario 5

This scenario represents the Salıpazarı Dam Design project constructed situation and possible future projects for remaining basins. The Scenario 4 model results show that $Q_{500}$ flood discharge of the Basin 4, individually create bank full flow at the Terme City. Structural solutions for two basins are considered in this scenario.

The aim of the study is to include the Salıpazarı Dam flood capacity for model calculations and controlling the two of the three remaining basins. The selection of control structures for Basin 2, Basin 3 and Basin 4 includes complex design procedures. Therefore, simple assumption is made for this scenario. It is assumed that all basins have similar characteristics and the control structures' properties are the same as Salıpazarı Dam. Therefore, Salıpazarı Dam behavior at flood situation was applied to the other three basins for flood control purposes. This means two of the three basins are controlled by the same structural design of the Salıpazarı Dam. The reason of this assumption is complexity of a new design and limited information for the remaining area. In addition, this approach keeps the studies at the safe side. This scenario assumes each basin brings 62 $m^3/s$ at the time of flood event. Possible future designs for flood protection on Basin 2, Basin 3 and Basin 4 will have flood capacity for $Q_{500}$ discharge.

This scenario represents the possible upstream solutions for Basin 2, Basin 3 and Basin 4. The base hydrological input for this scenario is Basin 1 controlled by Salıpazarı Dam. In addition to that, two of the three basins are controlled by the same way for flood studies and only one basin remains uncontrolled. The study also aims to show which basin has important role for flood situation.

The model studies are carried out for $Q_{500}$ flood discharge situation. Three different models are studied for three individually uncontrolled basins. Table 5.9 shows model input information.

**Table 5.9:** Scenario 5 model information

| | | |
|---|---|---|
| Input hydrograph location(s) | 22-45 AGI | Basin 4 Connection |
| Output hydrograph location | Terme Bridge | After Basin 4 |
| Bed resistance (1/n) | 32 | - |
| Input Hydrograph(s) | Hydrograph 7 and 8 | Hydrograph 5 |

- ***Uncontrolled Basin 2 model results***

Table 5.10 shows model output results. Figure 5.17 shows the sketch of the network for the scenario base. In addition, Figure 5.18 shows the related input and output hydrographs of the model and Figure 5.19 shows the flood water depth as one of the model results.

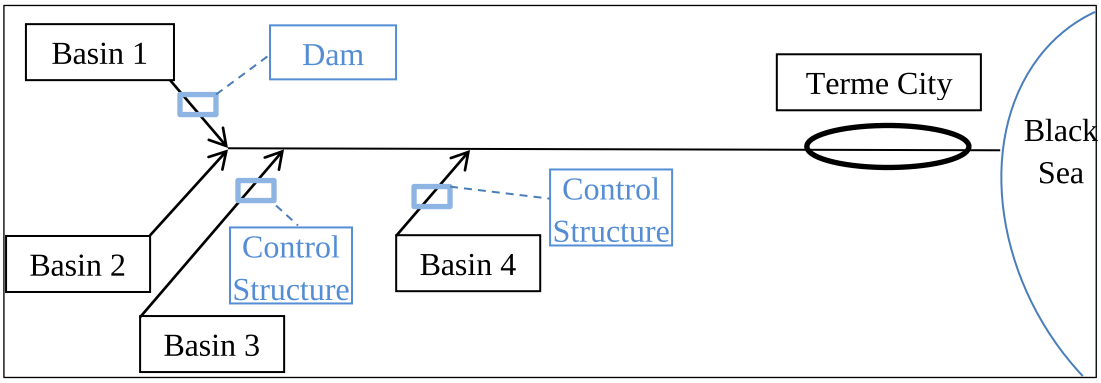


**Figure 5.17:** Sketch of Scenario 5 model_1

**Table 5.10:** Scenario 5 model_1 results

| Return Period | Input Hydrograph Peak Discharge (m³/s) | | Output Hydrograph Peak Discharge (m³/s) | | Attenuations (m³/s) **(5)=(3)-(4)** | Percentage of Difference **(6)=(5)/(3)** |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | | |
| $Q_{500}$ | 360,5 | 62 | 422,5 | 385,22 | 37,28 | % 9 |

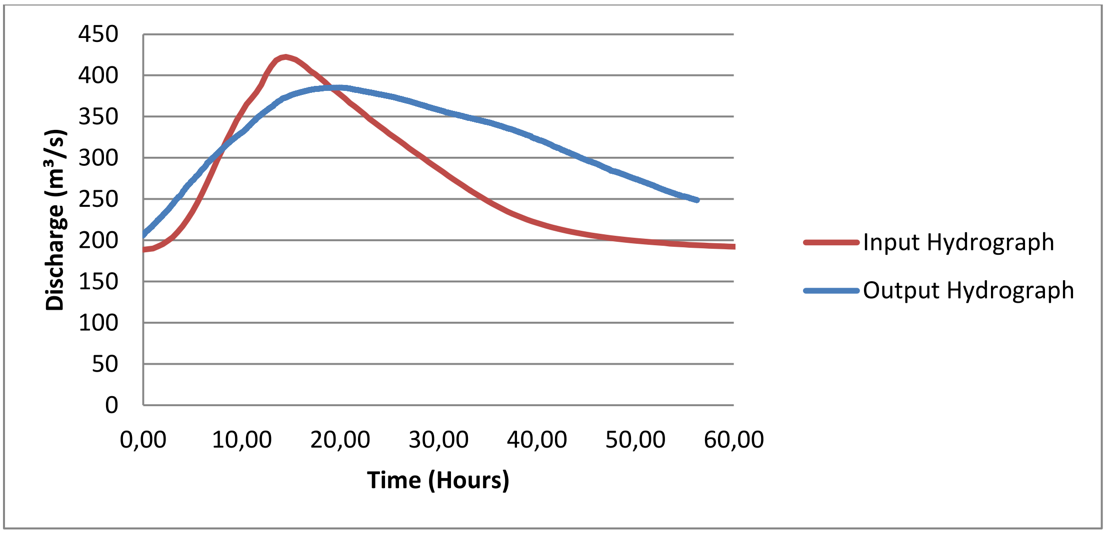


**Figure 5.18:** Scenerio 5 model_1 Input – Output hydrographs of the model

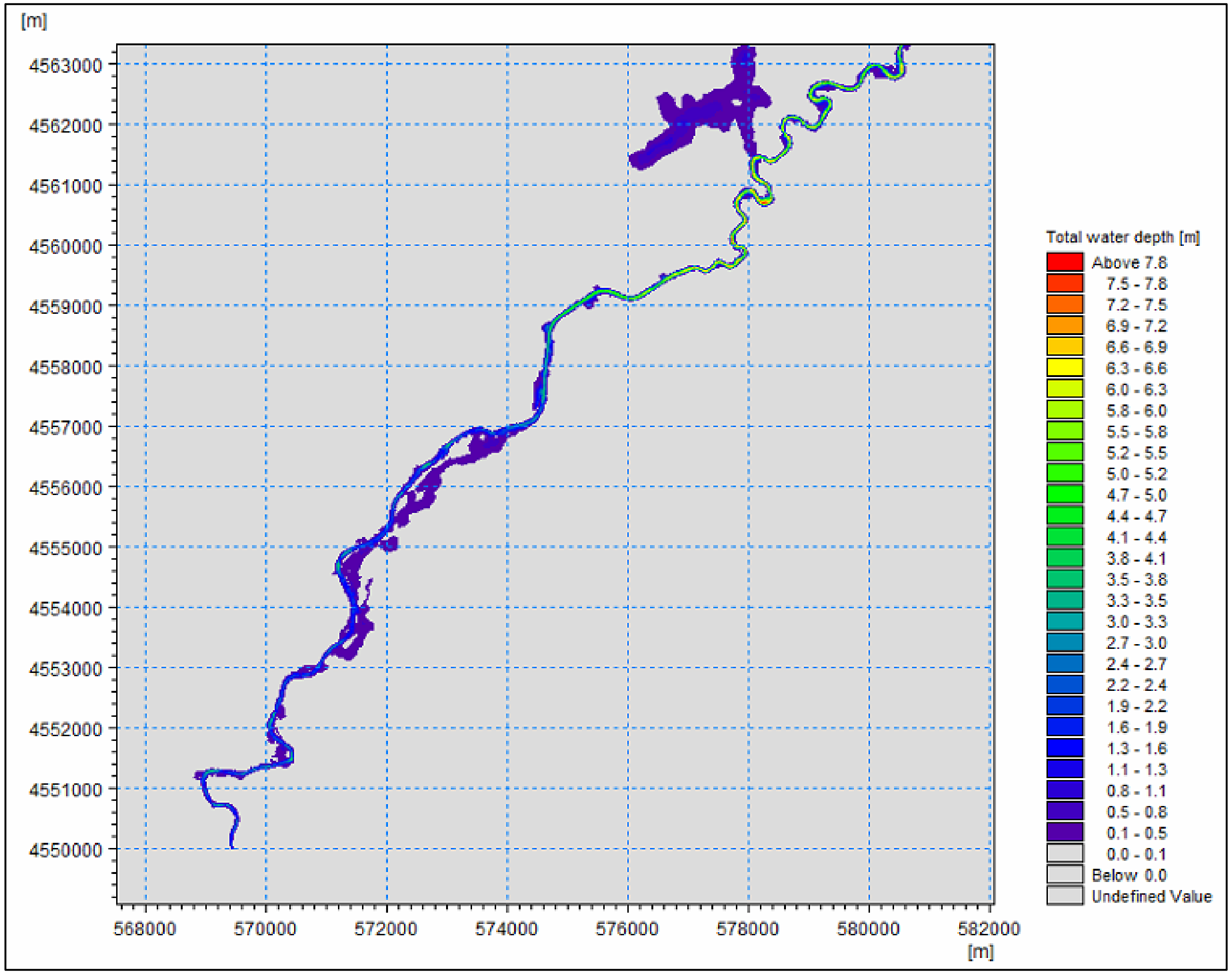


**Figure 5.19:** $Q_{500}$ water depth for Scenario 5 model_1

- ***Uncontrolled Basin 3 model results***

Table 5.11 shows model output results. Figure 5.20 shows the sketch of the network for the scenario base. Figure 5.21 also shows the flood water depth as one of the model results. Figure 5.22 shows the related input and output hydrographs of the model.

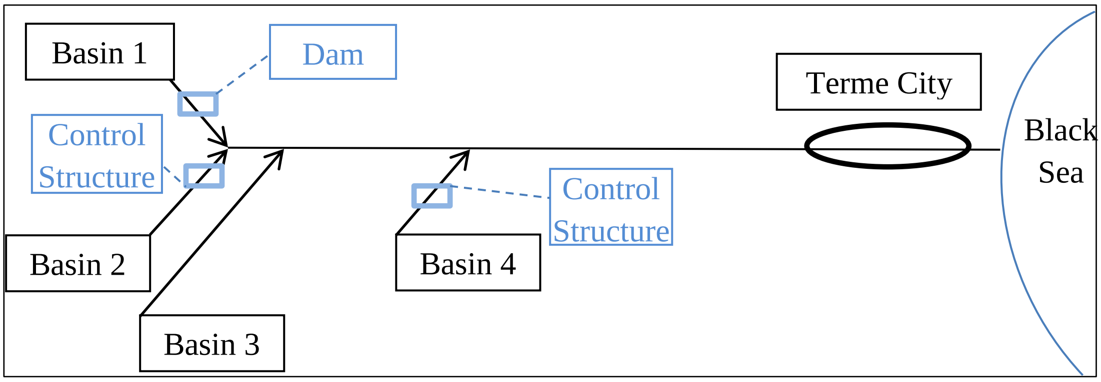


**Figure 5.20:** Sketch of Scenario 5 model_2

**Table 5.11:** Scenario 5 model_2 results

| Return Period | Input Hydrograph Peak Discharge ($m^3/s$) | | Output Hydrograph Peak Discharge ($m^3/s$) | | Attenuations ($m^3/s$) **(5)=(3)-(4)** | Percentage of Difference **(6)=(5)/(3)** |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | | |
| $Q_{500}$ | 643,7 | 62 | 705,70 | 545 | 162,31 | %23 |

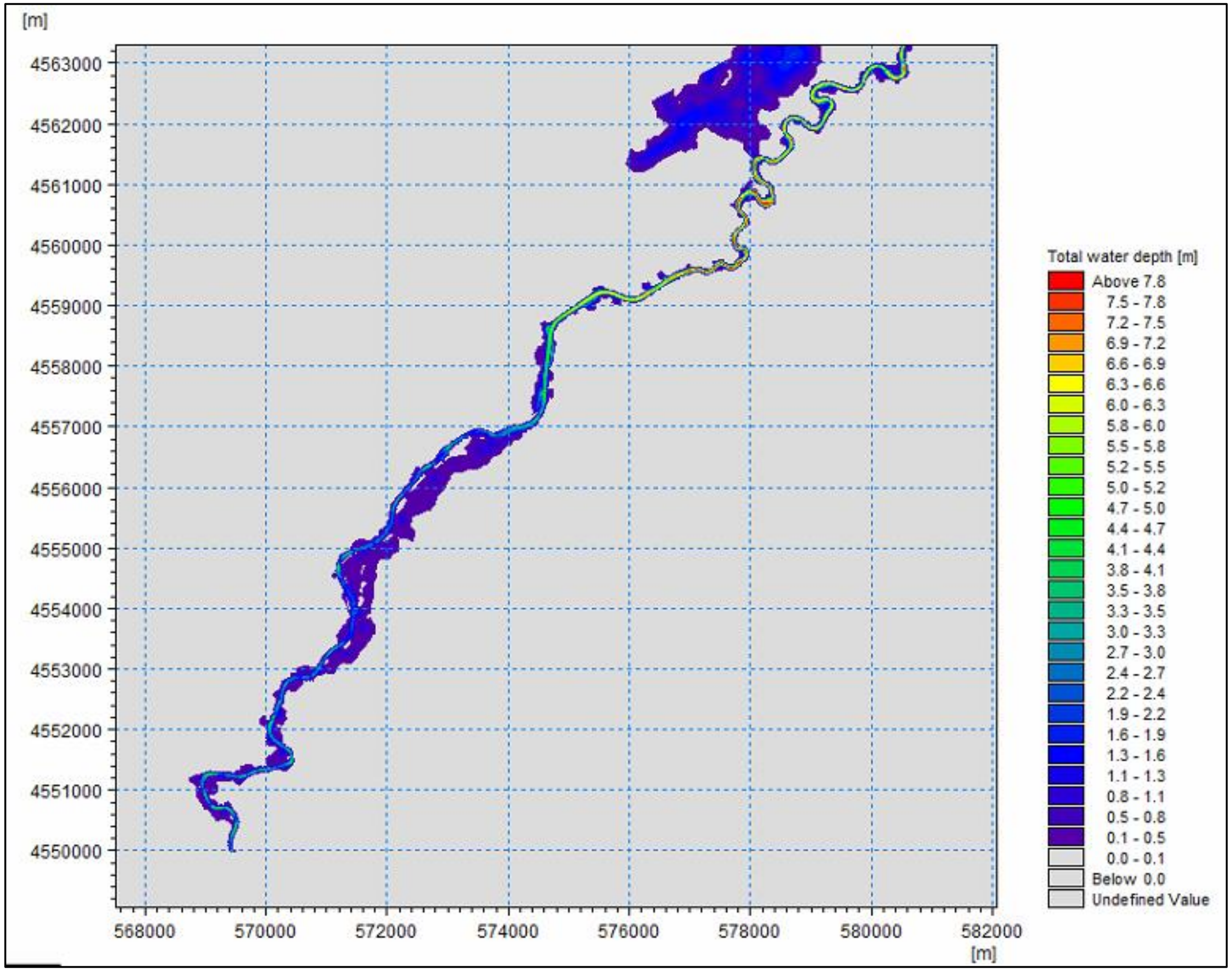


**Figure 5.21:** $Q_{500}$ water depth for Scenario 5 model_2

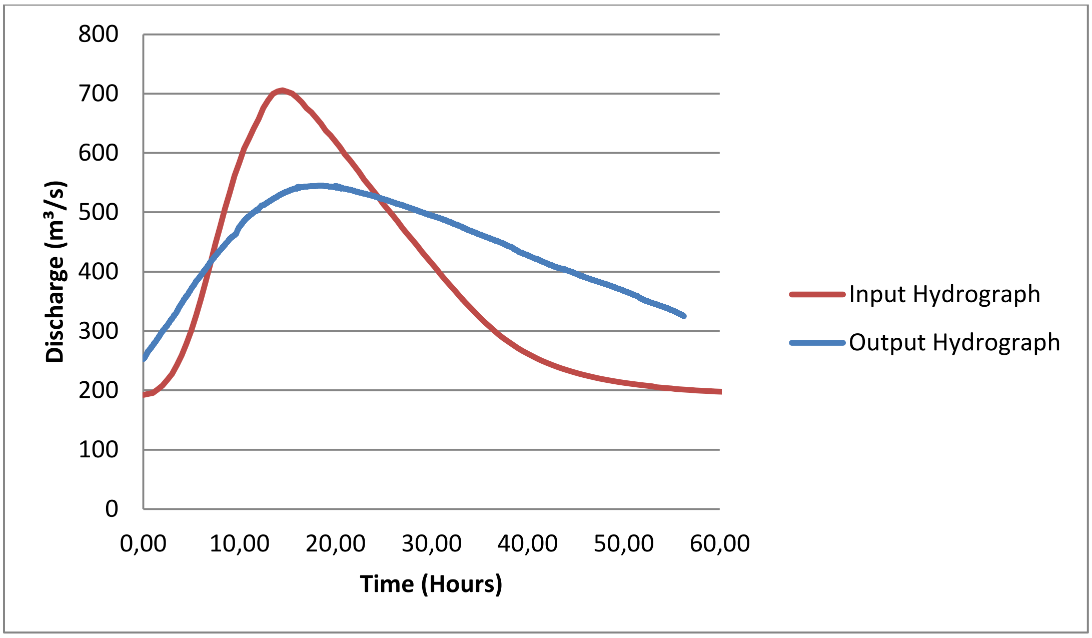


**Figure 5.22:** Scenario 5 model_2 Input – Output hydrographs of the model

- ***Uncontrolled Basin 4 model results***

The following Table 5.12 shows model output results. Figure 5.23 shows the sketch of the network for the scenario base. Figure 5.24 shows the related input and output hydrographs of the model and Figure 5.25 shows the flood water depth as one of the model results.

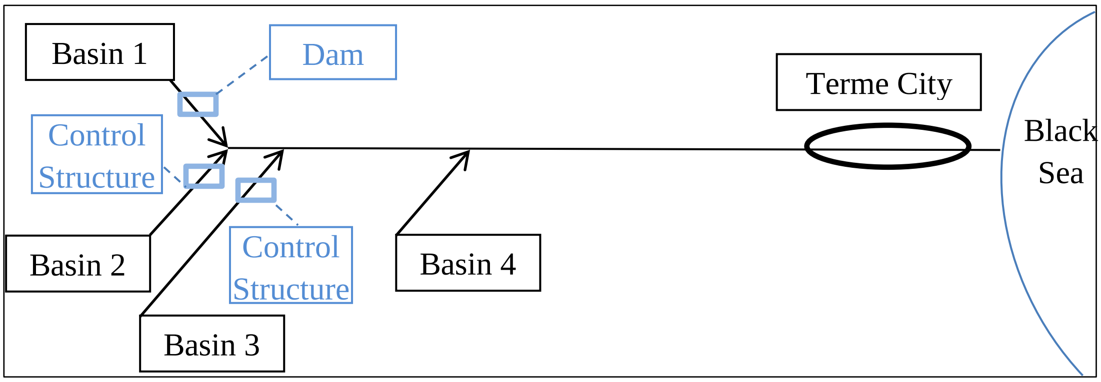


**Figure 5.23:** Sketch of Scenario 5 model_3

**Table 5.12:** Scenario 5 model_3 results

| Return Period | Input Hydrograph Peak Discharge ($m^3/s$) | | Output Hydrograph Peak Discharge ($m^3/s$) | | Attenuations ($m^3/s$) **(5)=(3)-(4)** | Percentage of Difference **(6)=(5)/(3)** |
|---|---|---|---|---|---|---|
| | (Terme B) **(1)** | (Basin 4) **(2)** | (Basin 4) **(3)** | (City C) **(4)** | | |
| $Q_{500}$ | 213,12 | 603,33 | 718,44 | 578,59 | 139,85 | %19 |

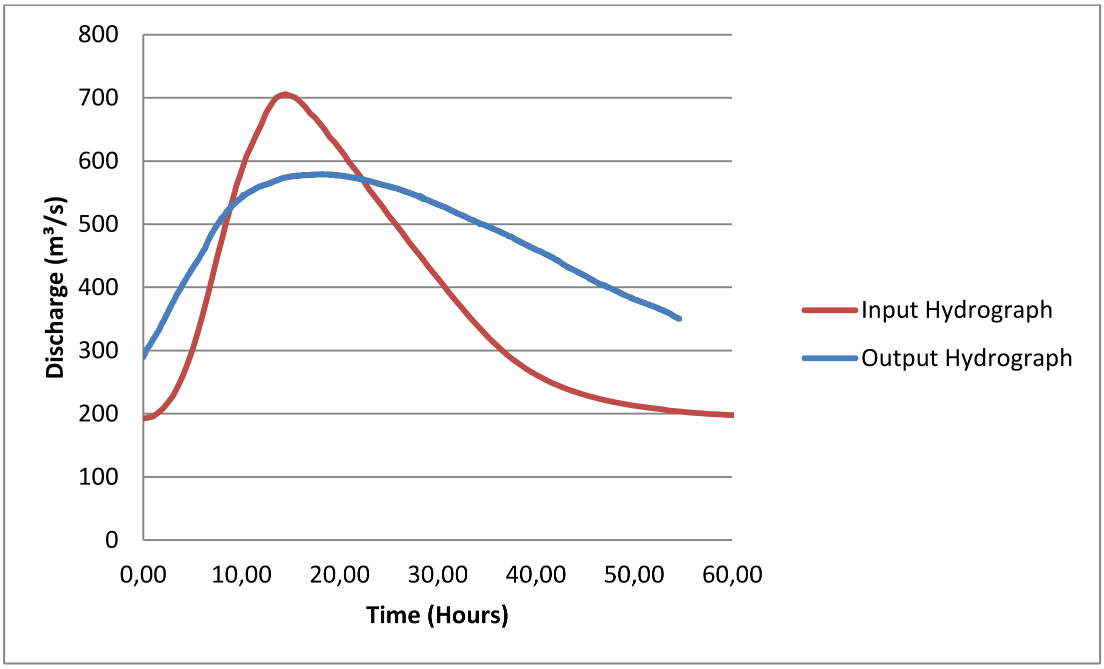


**Figure 5.24:** Scenario 5 model_3 Input – Output hydrographs of the model

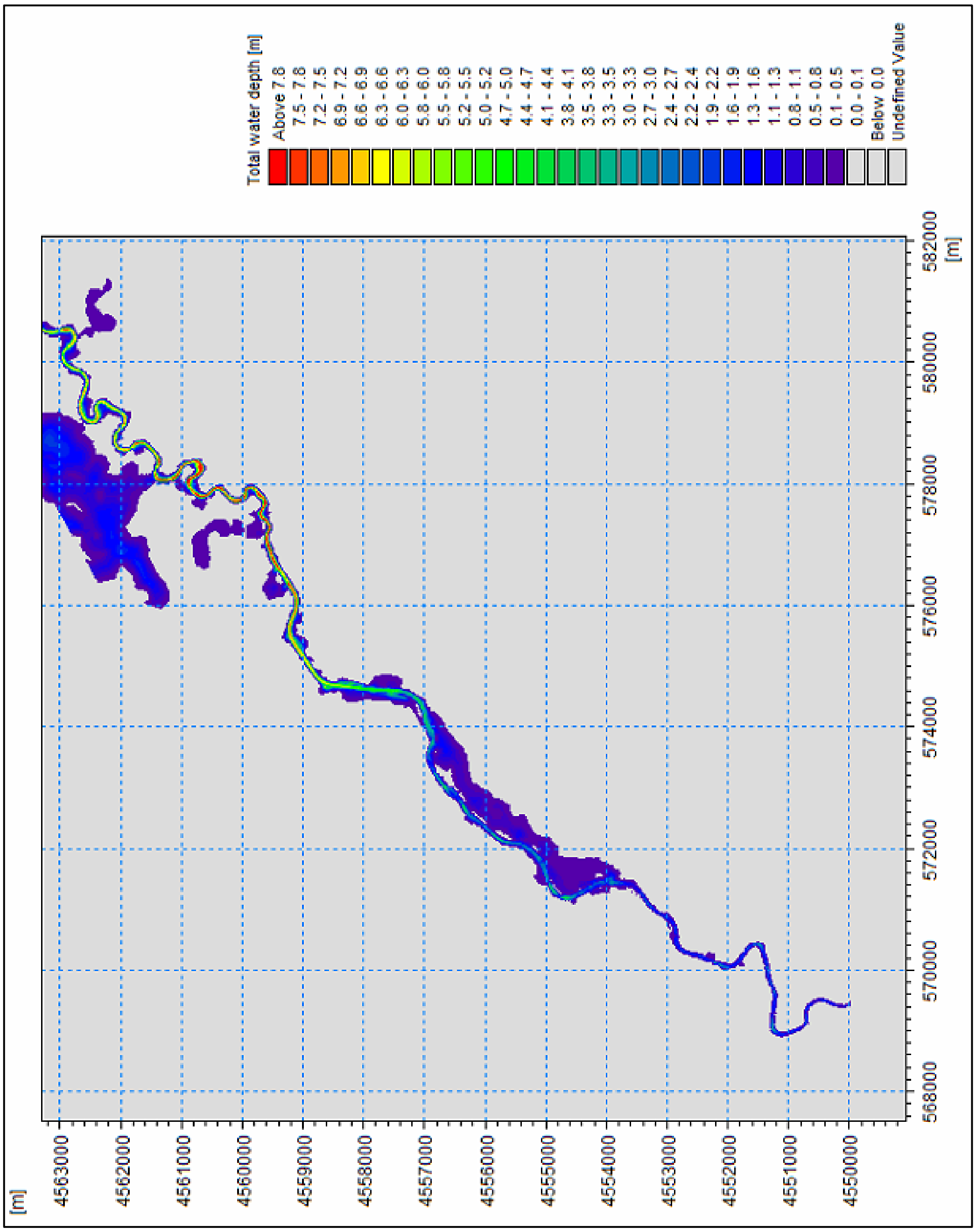


**Figure 5.25:** $Q_{500}$ water depth for Scenario 5 model_3

The model results can be evaluated individually. The results can also be interpreted as the $Q_{500}$ flood discharge is coming from only one basin. The other basins have their existing situations and they are participating to the Terme River with small amount of discharges. This aspect can also be used in evaluating the results.

The results of the models state that uncontrolled Basin 2 assumption has safe discharge at Terme City center. The uncontrolled Basin 3 has 545 $m^3/s$ discharge at city center so it can be shown as limit value with approximately ±5 % model error. However, uncontrolled Basin 4 situation is the risky one with respect to others.

**Scenario 6**

This scenario represents the projected situation of the Terme River. The meanders affecting the flood discharge are studied with detail. This scenario includes the Terme River hypothetical rehabilitation. The rehabilitation of the Terme River upstream part is projected and some parts are constructed by the DSI 7th Regional Directory.

This scenario includes the rearranging the Terme River bed downstream parts. The width of the river bed was modeled with respect to original distance. In addition, river bed level was changed keeping the existing level mostly. The bed resistance of the river was changed. Bed resistance value was used as 40 (n=0.025). Figure 5.26 shows the sketch of the network for the scenario base.

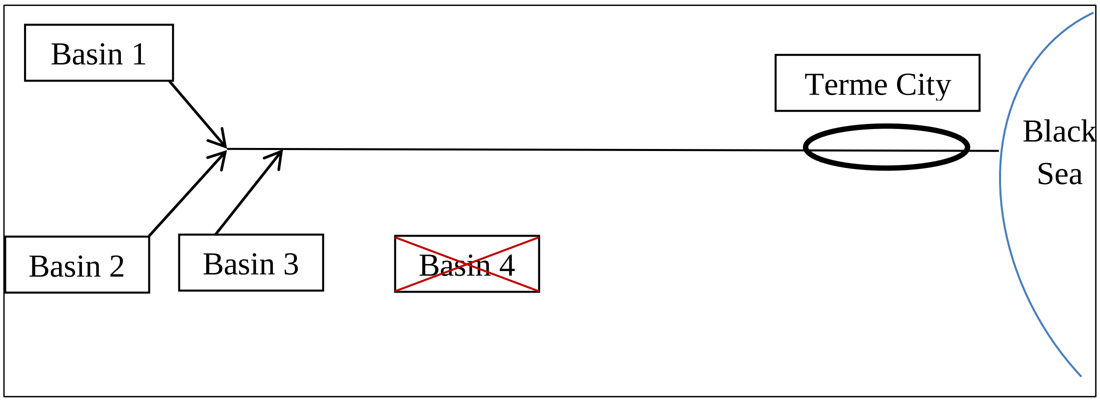


**Figure 5.26:** Sketch of Scenario 6

The flood discharge value was used as $Q_{100}$ and it is compared with the existing situation for $Q_{100}$ value. Table 5.13 and Table 5.14 show model input information and the model output results respectively. Figure 5.27 shows the related input and output hydrographs of the model and Figure 5.28 shows the flood water depth as one of the model results.

**Table 5.13:** Scenario 6 model information

| | |
|---|---|
| Input hydrograph location(s) | 22-45 AGI |
| Output hydrograph location | Terme Bridge |
| Bed resistance (1/n) | 40 |
| Input Hydrograph(s) | Hydrograph 1 |

**Table 5.14:** Scenario 6 model results

| Return Period | Input Hydrograph Peak Discharge ($m^3/s$) | Output Hydrograph Peak Discharge ($m^3/s$) | Attenuations ($m^3/s$) | Percentage of Difference |
|---|---|---|---|---|
| | (Terme B) **(1)** | (City C) **(2)** | **(3)=(1)-(2)** | **(4)=(3)/(2)** |
| $Q_{100}$ existing | 792,41 | 573,40 | 219,01 | 28 % |
| $Q_{100}$ rehab. | 792,41 | 672.30 | 120,11 | 15 % |

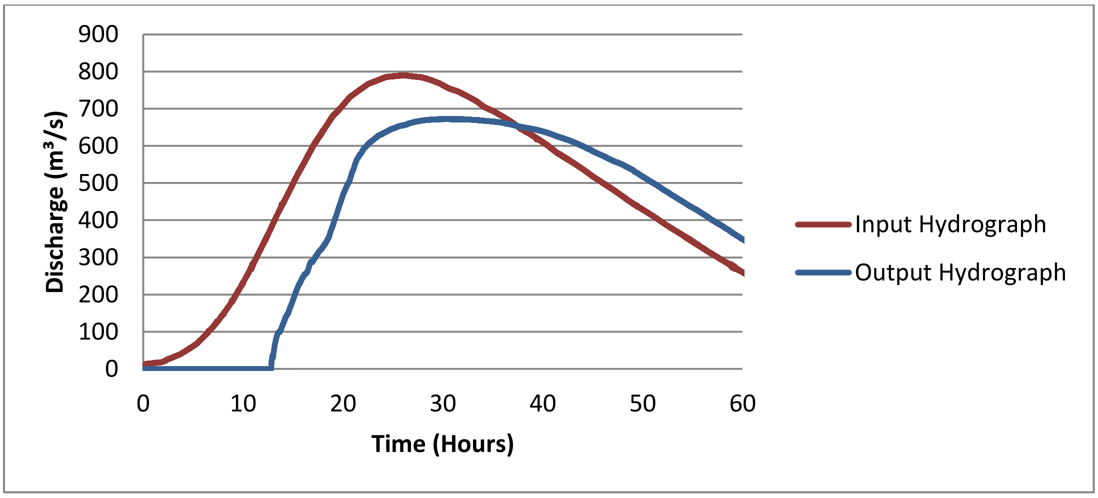


**Figure 5.27:** Scenario 6 Input – Output hydrographs of the model

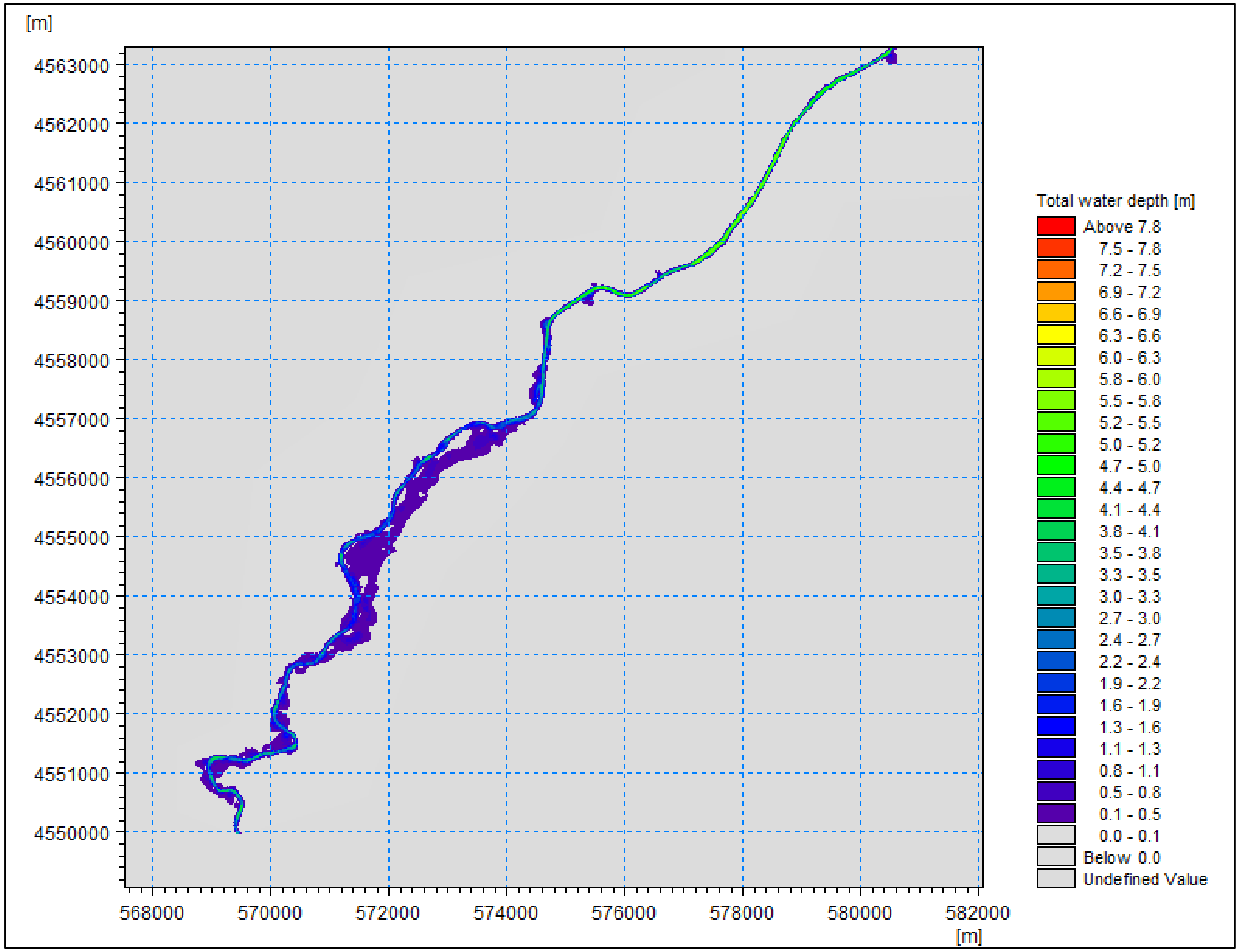


**Figure 5.28:** $Q_{500}$ water depth for Scenario 6

# CHAPTER 6

# CONCLUSION AND FUTURE RECOMMENDATION

In the present study, Samsun City Terme District flood problem was presented with model studies for different scenarios. Model studies were conducted with 1-D and 2-D flood model software. Aim of the studies is to understand the reasons of the flood problem at Terme District and to point out possible solutions for this problem.

The study area has four sub-basins and this means different scenarios are needed for representing the possible existing situations. The model studies were carried out with different discharges for different scenarios. These scenarios can be grouped into three.

- Existing situations of the basins,
- After Salıpazarı Dam constructed on Basin 1,
- Hypothetical structures for problem solution.

In total, six different scenarios were studied and modeled for different discharges of the four sub-basins. The existing conditions of the Terme River between Salıpazarı Bridge and the Terme Bridge were firstly determined, then model studies were conducted. Considering present research, following conclusions can be derived:

- The existing circumstance of the Terme River states that the meanders of the river have a major effect on the flood situation. The discharge measurements between Salıpazarı Bridge and the Terme Bridge show approximately 35% reduction in the peak discharge.

- The model studies with and without Basin 4 state that, Basin 4 has the important role on the Terme City flood. The flood discharge of Basin 4 is higher than the other three basins. Since Basin 4 connection is closer to the city, risk factor is increasing.

- Salıpazarı Dam flood capacity is not sufficient individually to protect Terme City against flooding. Since other basins do not have any flood protection structure, this means if the flood event affects Basin 1 and any of other basins simultaneously, Terme city flood risk still exists.

- After Salıpazarı Dam is constructed, additional control structures would also be needed for other sub-basins.

- All comparisons of the results were made for the existing situations of the river bed. If the river bed characteristics change then the conclusions of the study and future recommendations also change.

Considering the model results and conclusions, following recommendations can shed light on the Terme City flood problem,

- Terme River existing condition was used for model studies. It is known that meanders of the river reduce the peak discharge of the flood. This existing situation can be kept since the upstream flood affects the agricultural areas by the meandering part of the river. In addition, river bed rehabilitation and increasing the stream capacity cannot be a solution as long as the Terme City River capacity remains the same. On the contrary, these kinds of constructions can increase the flood problem at the urbanized downstream areas.

- The Salıpazarı Dam project can be supported with other control structures for remaining basins. The flood capacity of the dam is not sufficient itself. In addition, model studies state that control structures for other basins can be considered for two of the three sub-basins. In other words three of the four sub-basins must be controlled for the solution of the Terme City flood problem.

- The model studies are focused on input discharges and the output discharges of the study area. Input discharges are defined at Salıpazarı Bridge. Output discharges are measured after Basin 4 connection and at Terme city center. The structural solutions for the study area must be based on reducing the discharges at the measurement points with upstream solutions.

- All model studies were based on the assumption of the peak discharges overlapping at basins. The structural solutions can focus on flood routing with multiple non-storage systems that can change the peak discharge times of the basins so the cumulative hydrographs of the basins would have smaller peak discharges. The studies can be expanded with that respect.

- Early warning systems for the sub-basins can be a solution for flood mitigation. The flood peak discharge reaches from Salıpazarı Bridge to the Terme City approximately in 4 hours.

- Hydrological modeling has an important role on flood modeling studies. Well calibrated hydrological models are needed for the calculations of the model input discharges with rainfall-runoff relation. Classical methods used to estimate hydrographs/peak discharges have limitations and may lead to unrealistic results

- Hydraulic models need calibrations. If water marks produced by a flood are known and stream gage data are available, discharge and water level relation can be used as calibration data. Arial photos or the satellite images can be also used to estimate the flooded areas after the flood events. The roughness value of the river can be calibrated using the water level for known discharge. Sometimes this process needs many simulation runs.

- Flexible mesh has advantages on model studies. One of them is reducing the calculation time of the model. Grid based models for the same area need almost four times greater computation time compared to the time needed in modelling with flexible mesh. The other advantage is the creating different size of the mesh elements which helps to get a well-detailed map. Flat areas like agricultural areas can be represented with large element size meshes and the urbanized areas can be represented with small element size meshes in a single model. On the contrary, single grid size models do not have such an advantage since the grid size is constant in a single model.

- As a future work recommendation GPU integrated modeling can be given. The traditional computer based models work with CPU (Central Processing Unit). The run time of the model limited on CPU power depends on core numbers. GPU based models work with Graphical Processing Unit, which has not traditional cores. Since that GPU based models have less run times.

The model studies and the scenarios can be expanded since the variables of the study area are too much to consider. The present study creates a base scenario and gives information about the Terme City flood problem reality and possible solutions.

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# APPENDIX A

## MODEL INPUT HYDROGRAPHS

The model input hydrographs are designed for the needs of the scenarios. Explanation and details of the hydrographs are given in Chapter 4. Each of the hydrographs given in Figure A.1 to Figure A.8 are used for model studies. The following given figures are directly taken from MIKE software input pages.

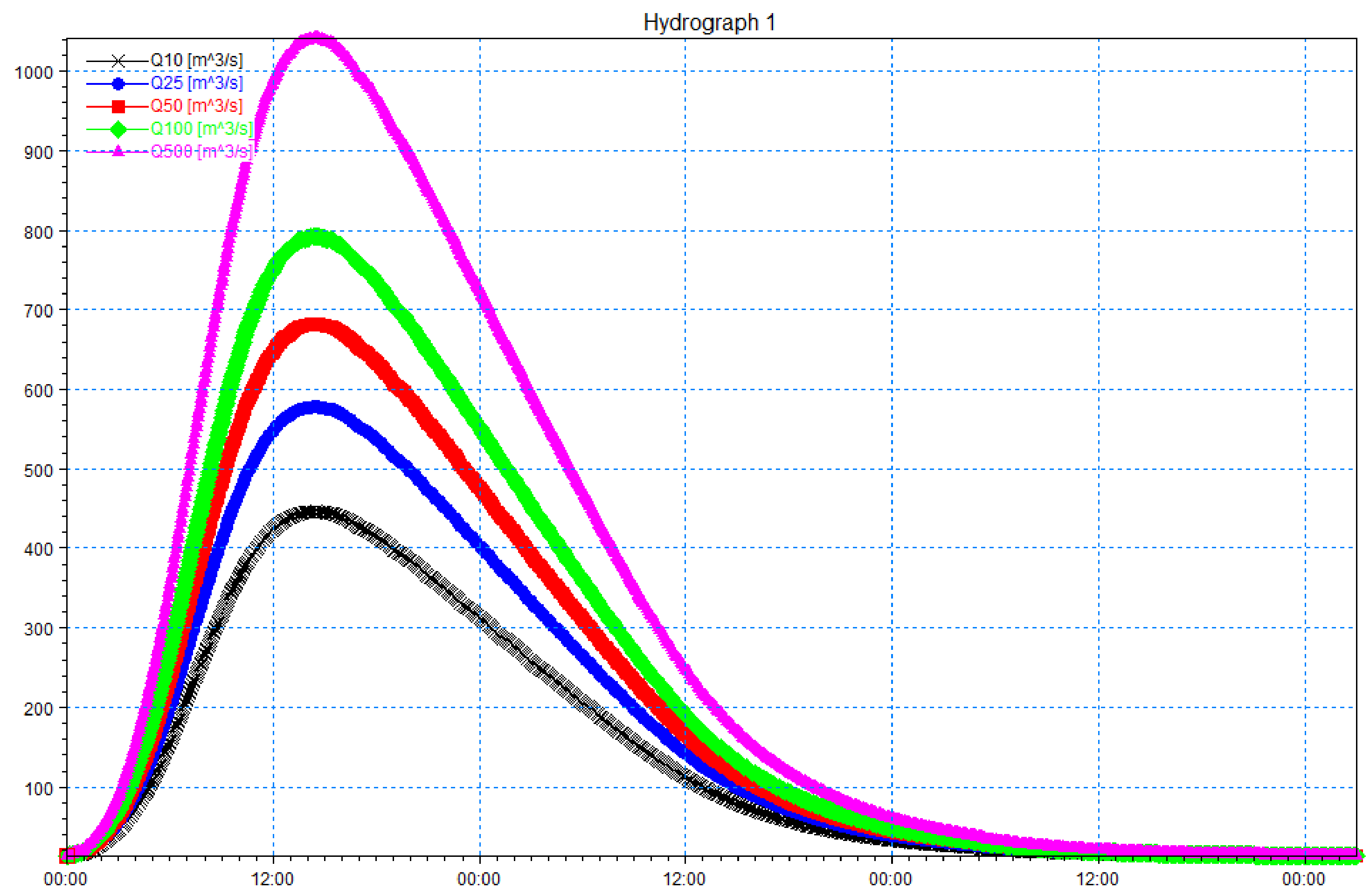


**Figure A.1:** Hydrograph 1

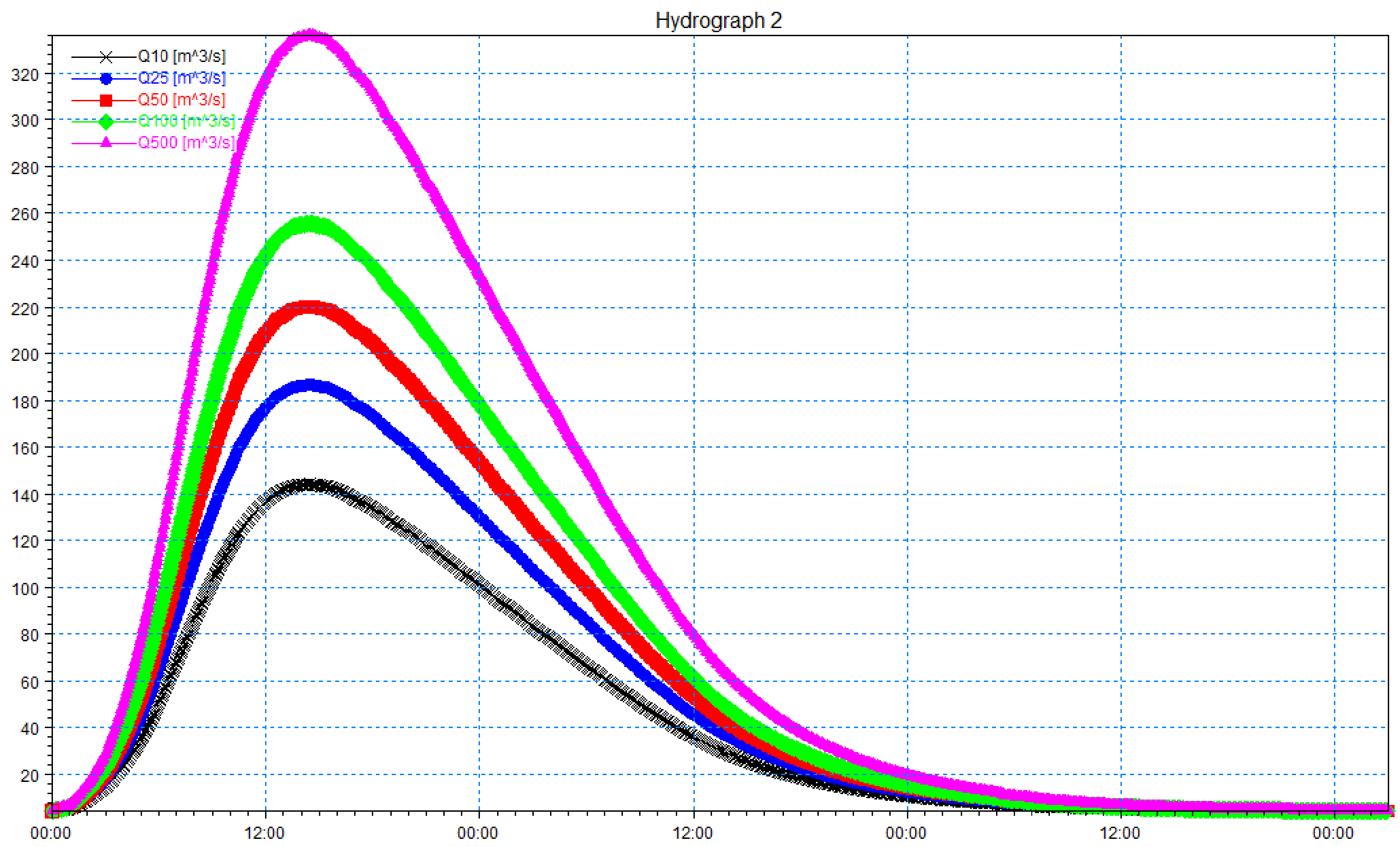


**Figure A.2:** Hydrograph 2

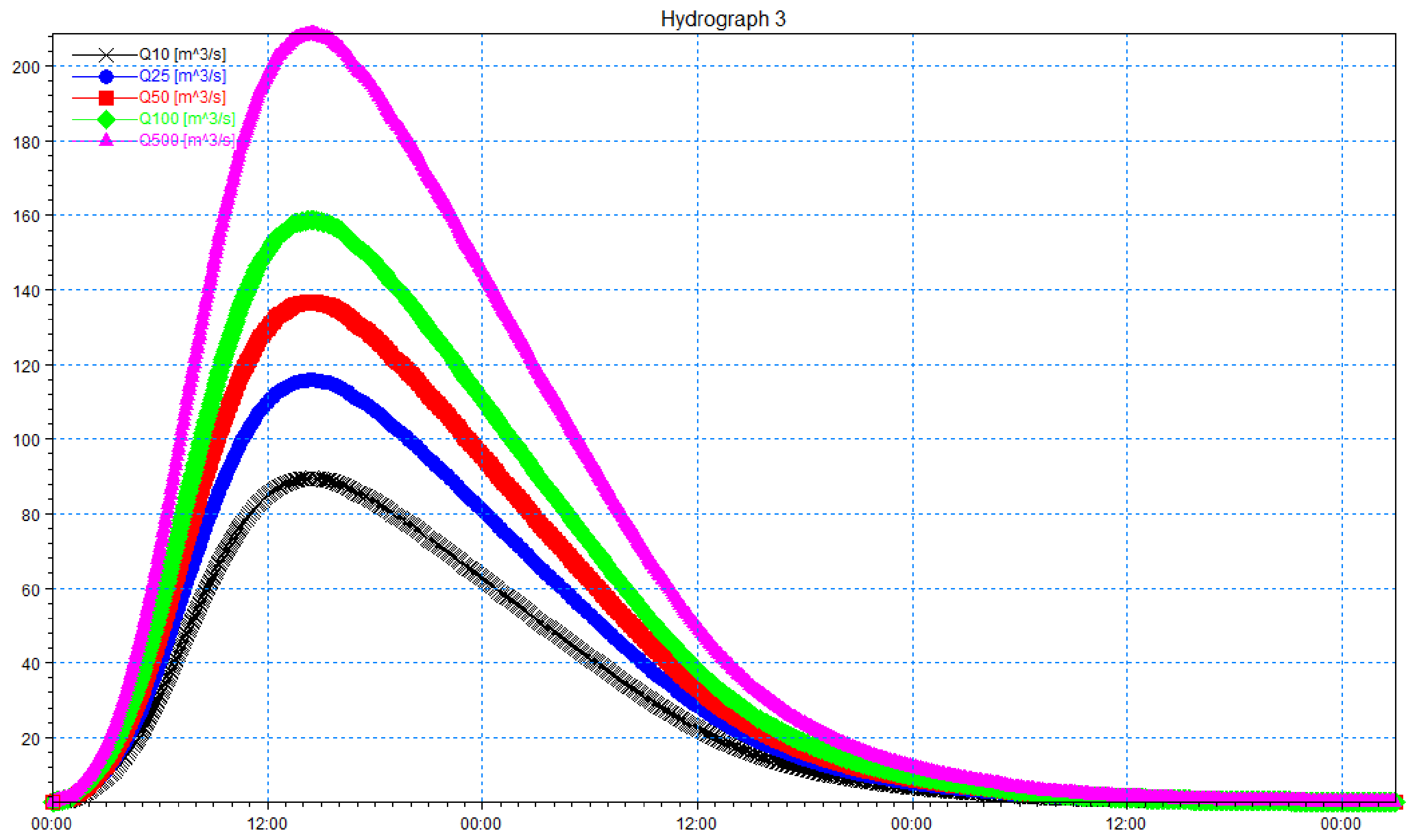


**Figure A.3:** Hydrograph 3

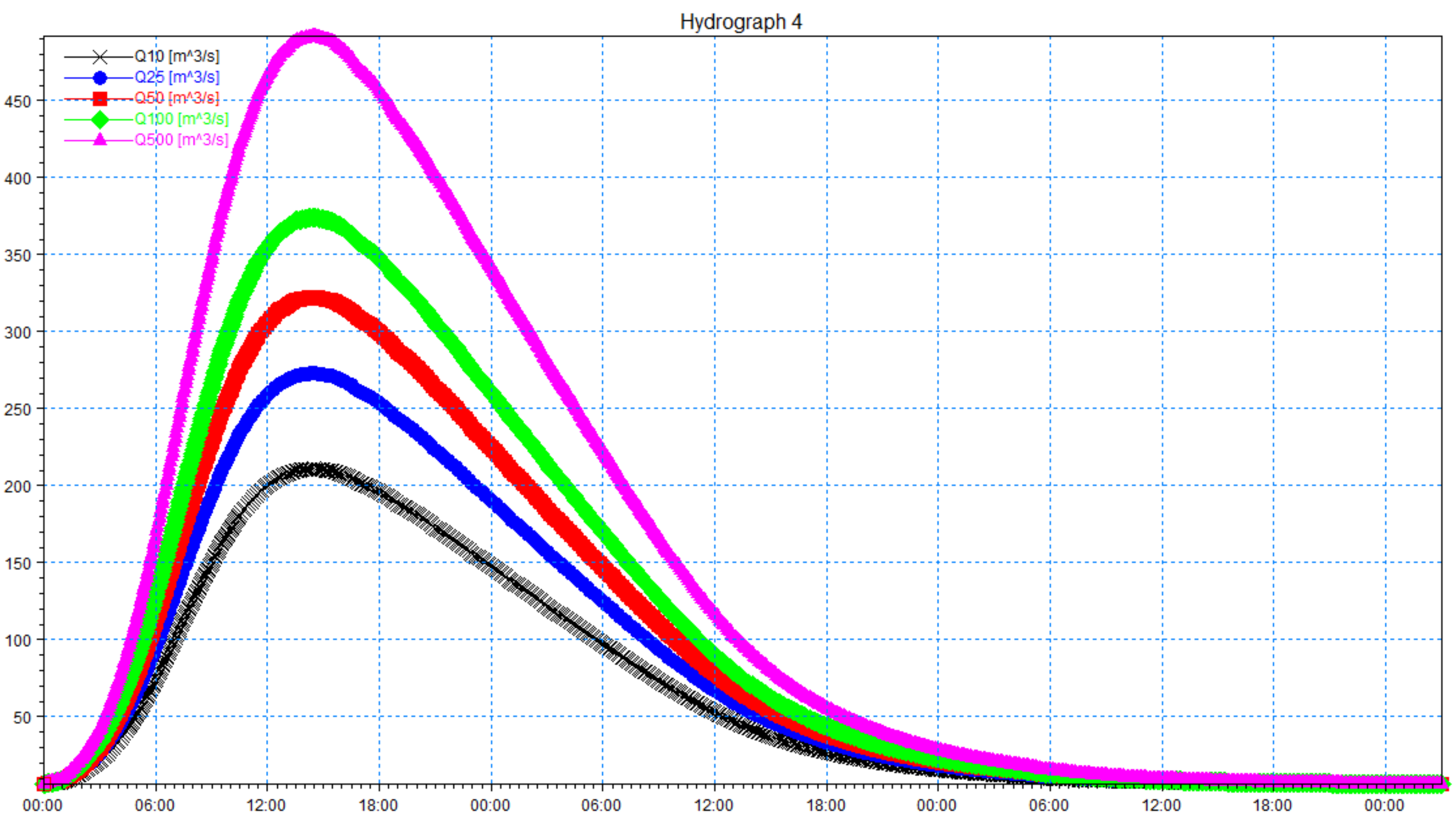


**Figure A.4:** Hydrograph 4

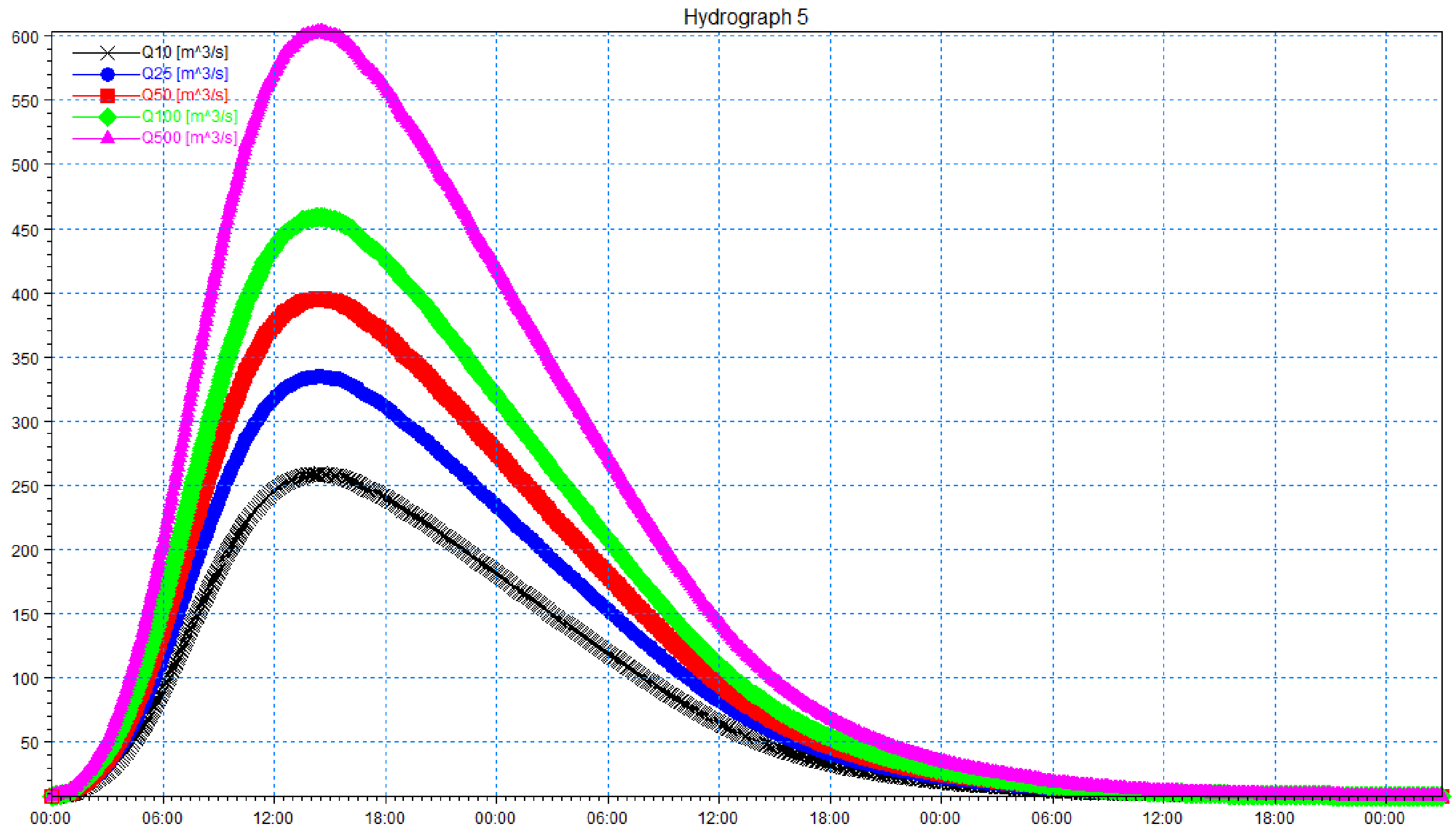


**Figure A.5:** Hydrograph 5

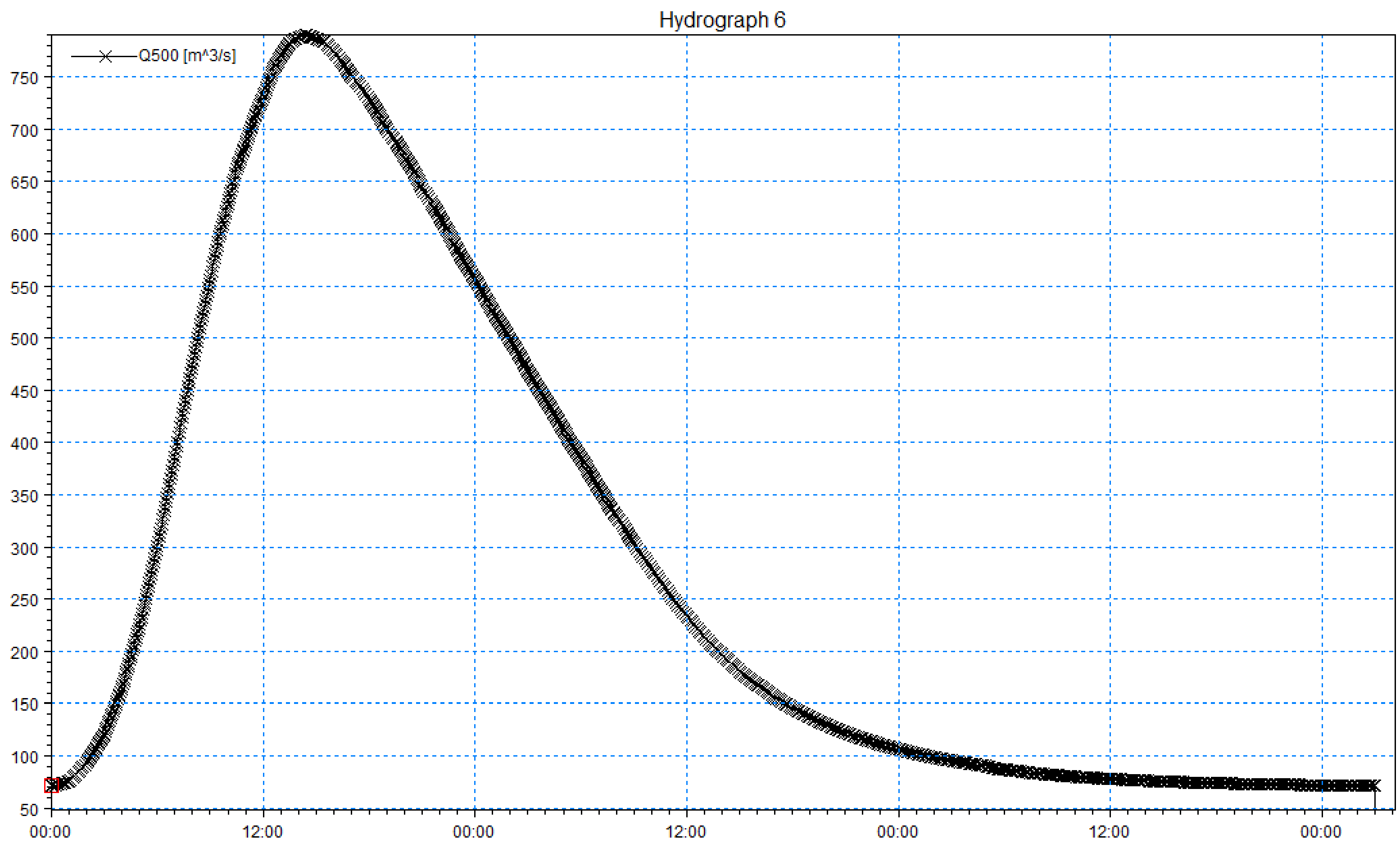


**Figure A.6:** Hydrograph 6

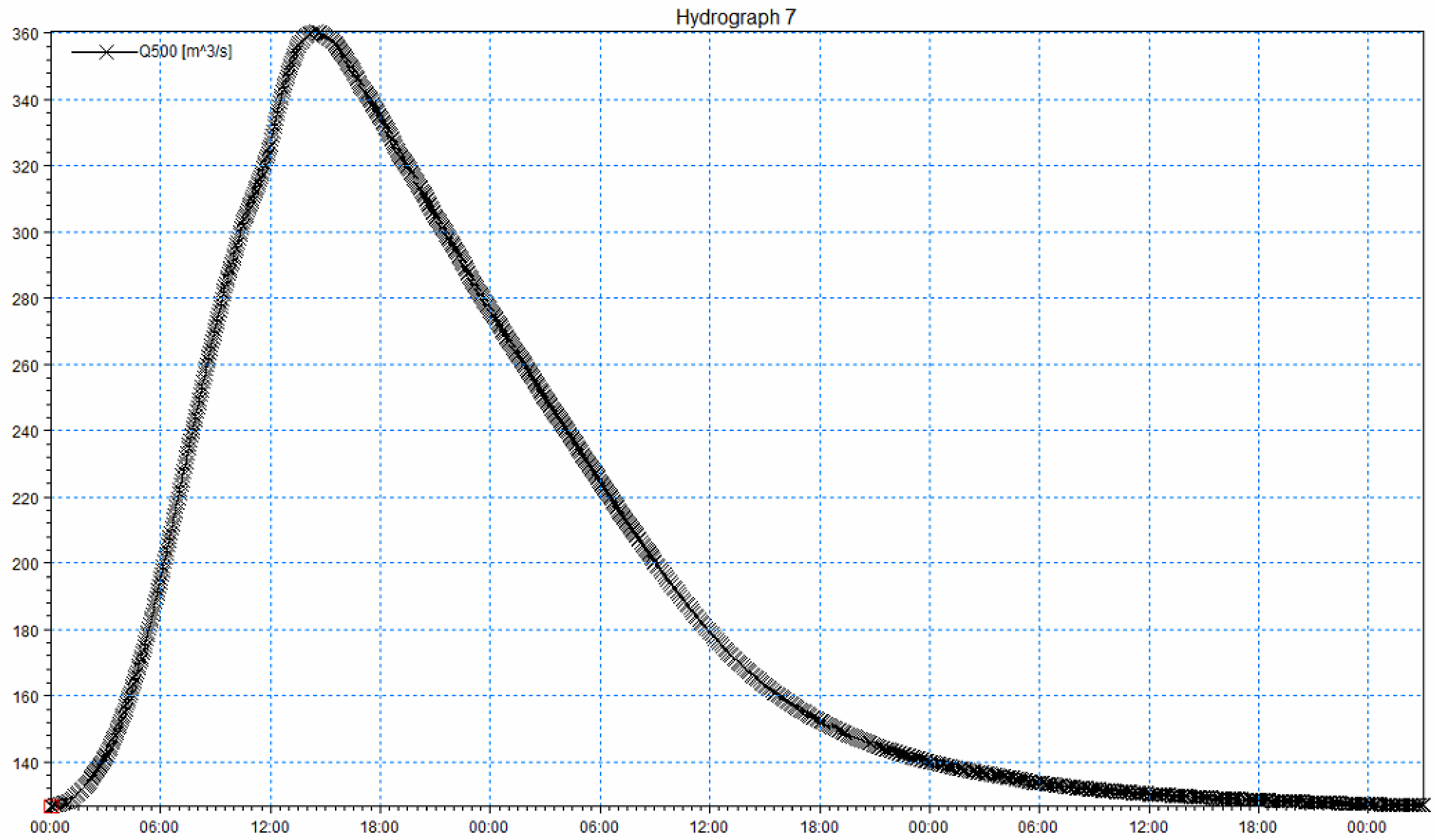


**Figure A.7:** Hydrograph 7

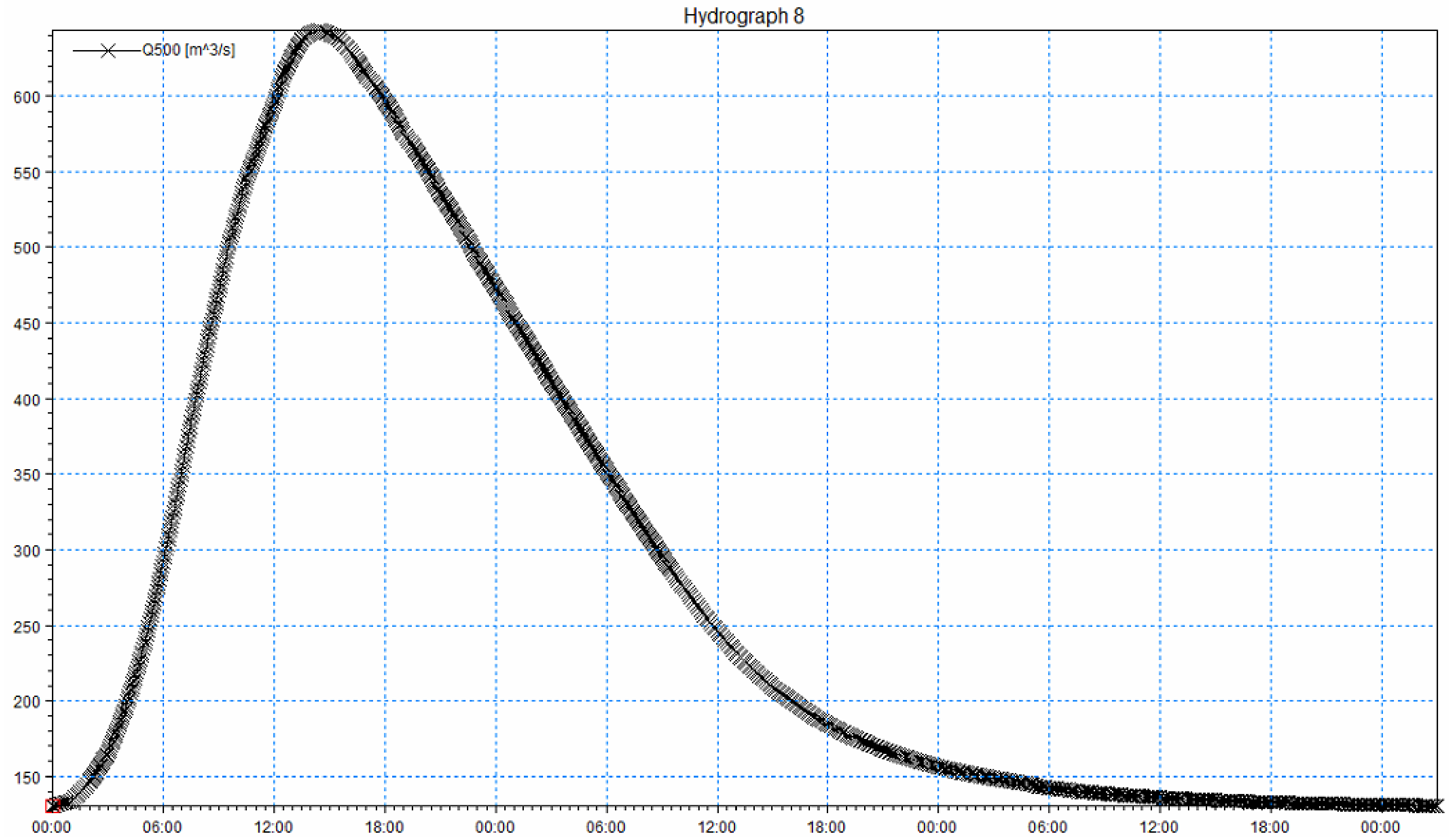


**Figure A.8:** Hydrograph 8

# APPENDIX B

## MODEL RESULT HYDROGRAPHS

The model result hydrographs are calculated for the Terme City for different scenarios. Explanation of the models and details of the hydrographs are given in Chapter 5. Each of the result hydrographs given in Figure B.1 to Figure B.8 are obtained from model studies. The following given figures are directly taken from MIKE software output pages.

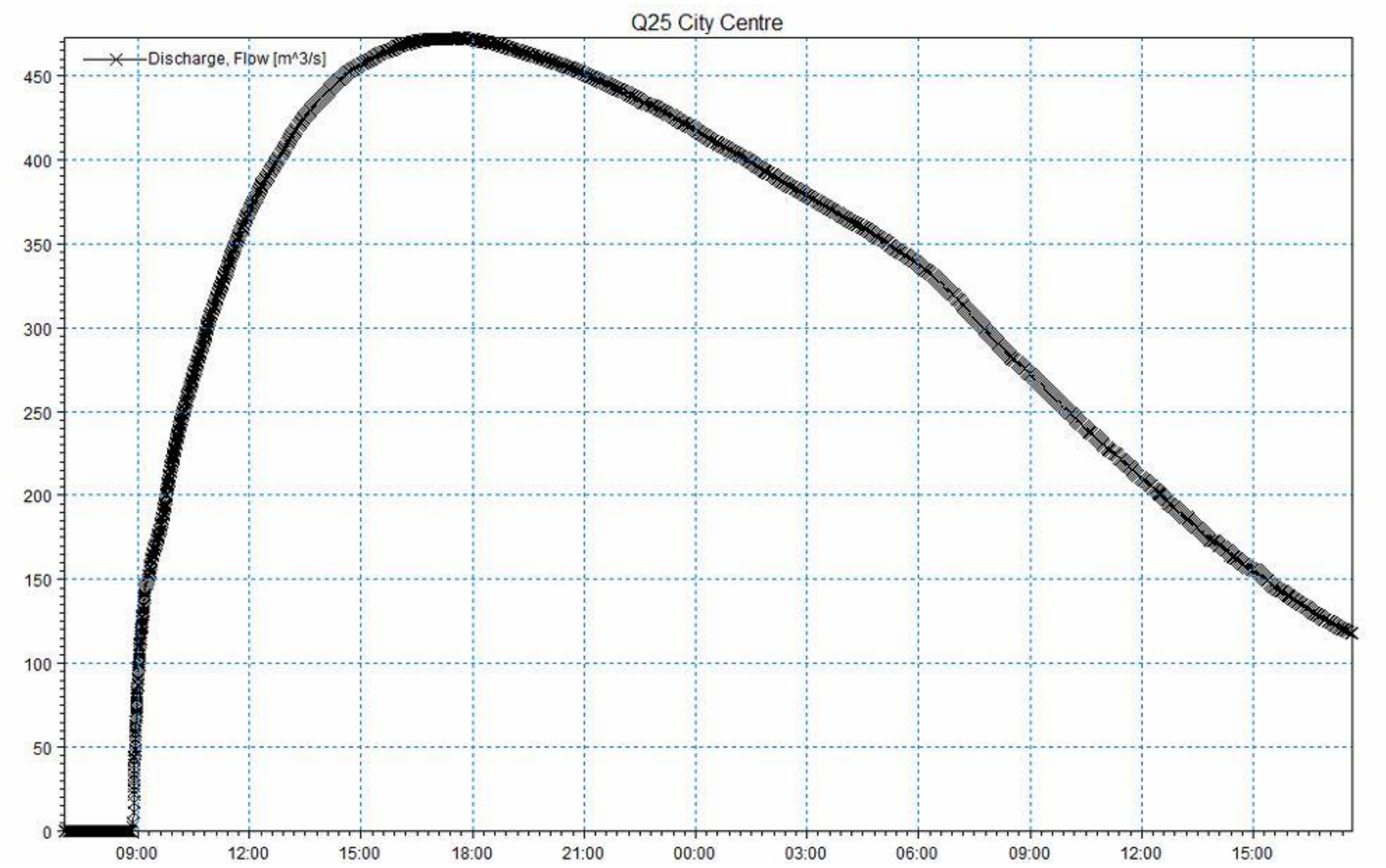


**Figure B.1:** Scenario 1 Res. 1

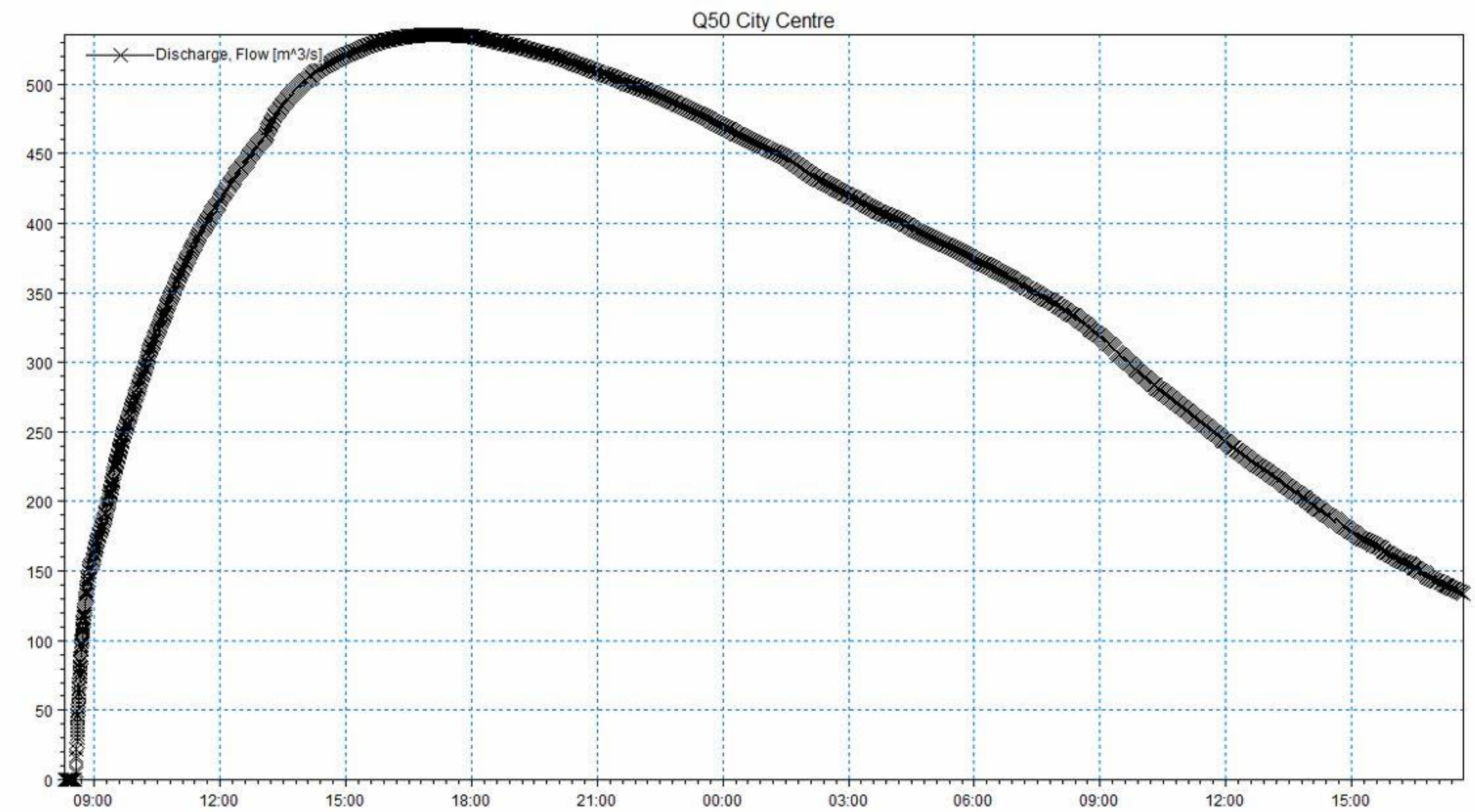


**Figure B.2:** Scenario 1 Res. 2

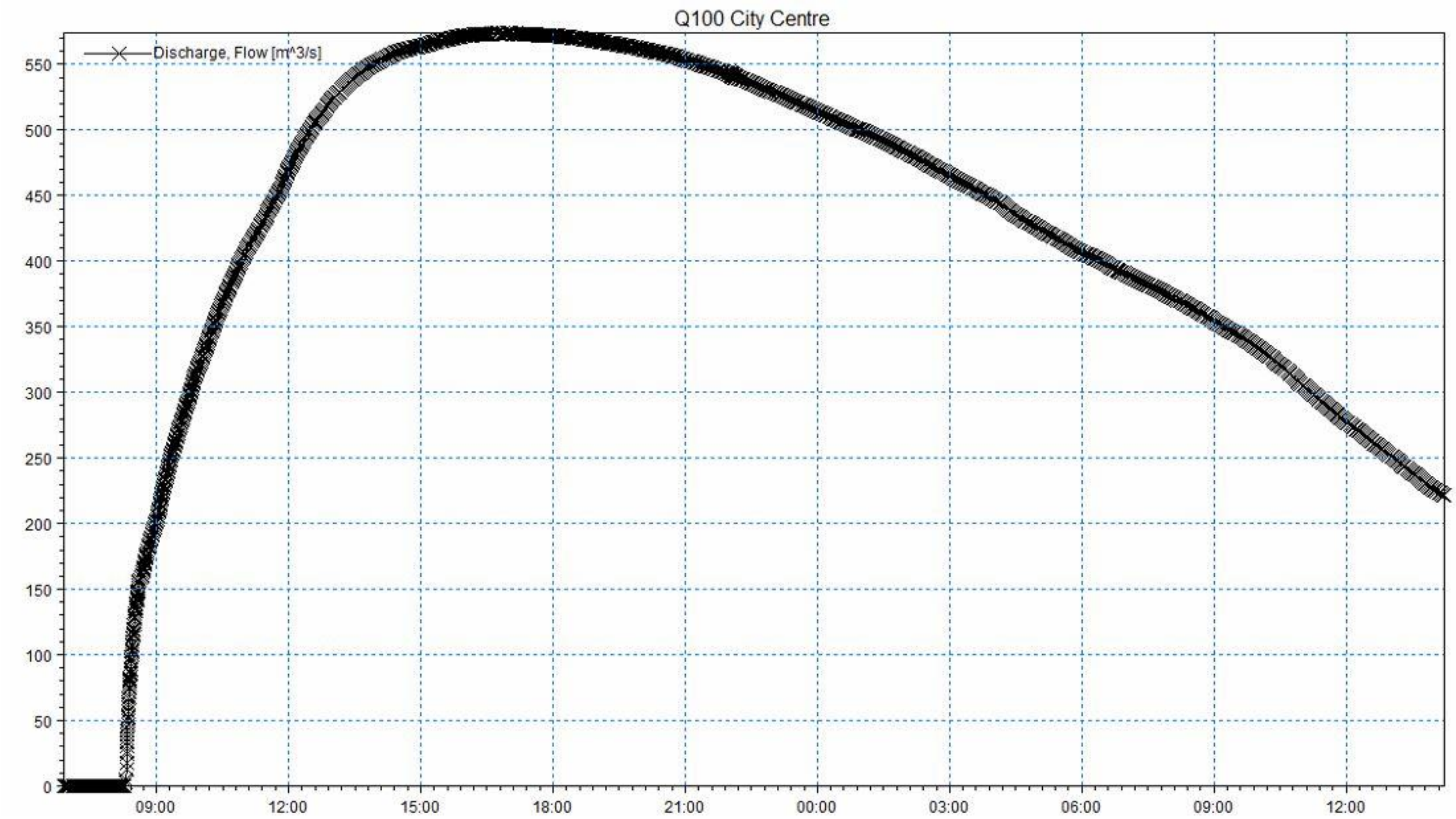


**Figure B.3:** Scenario 1 Res. 3

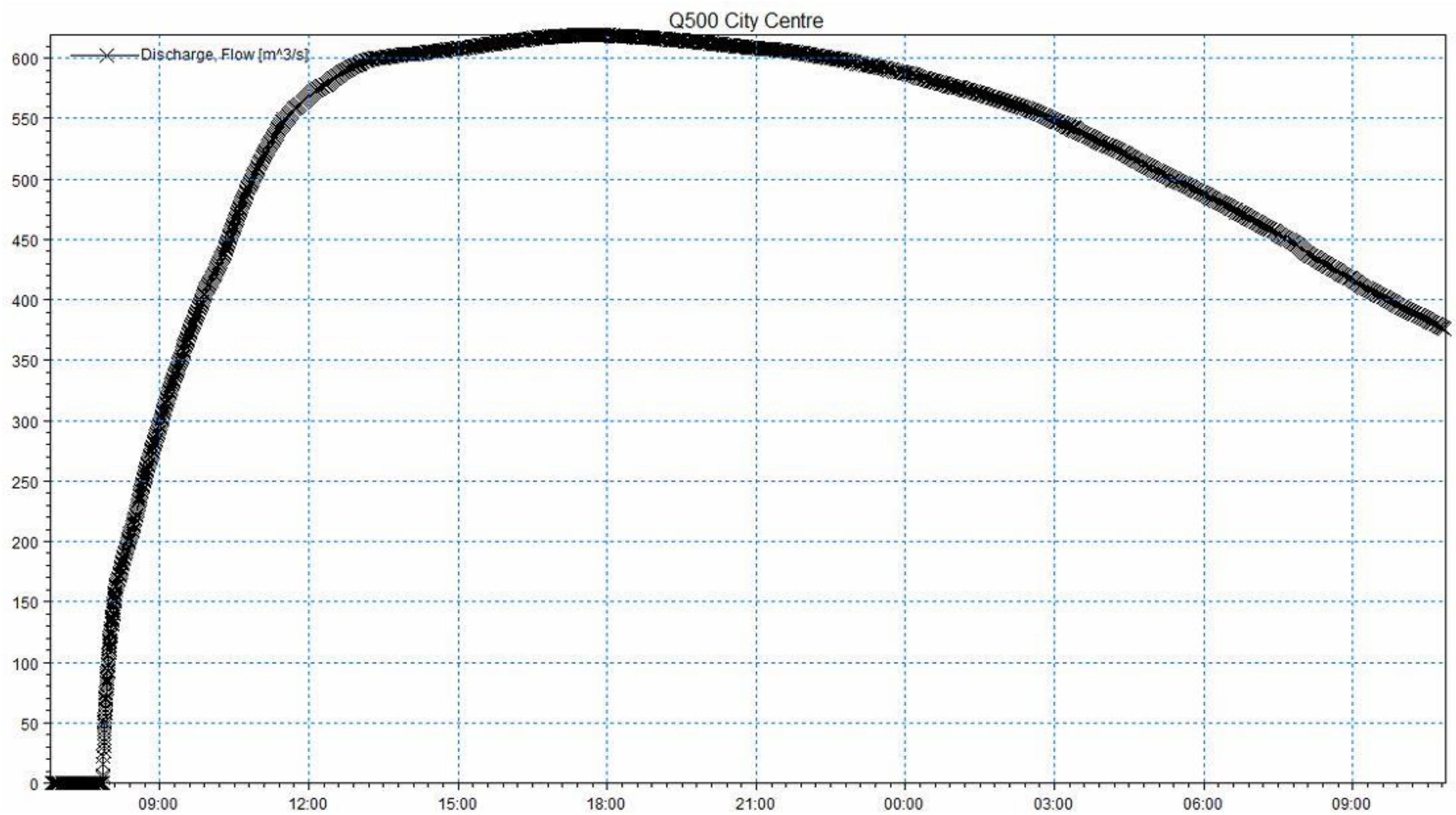


**Figure B.4:** Scenario 1 Res. 4

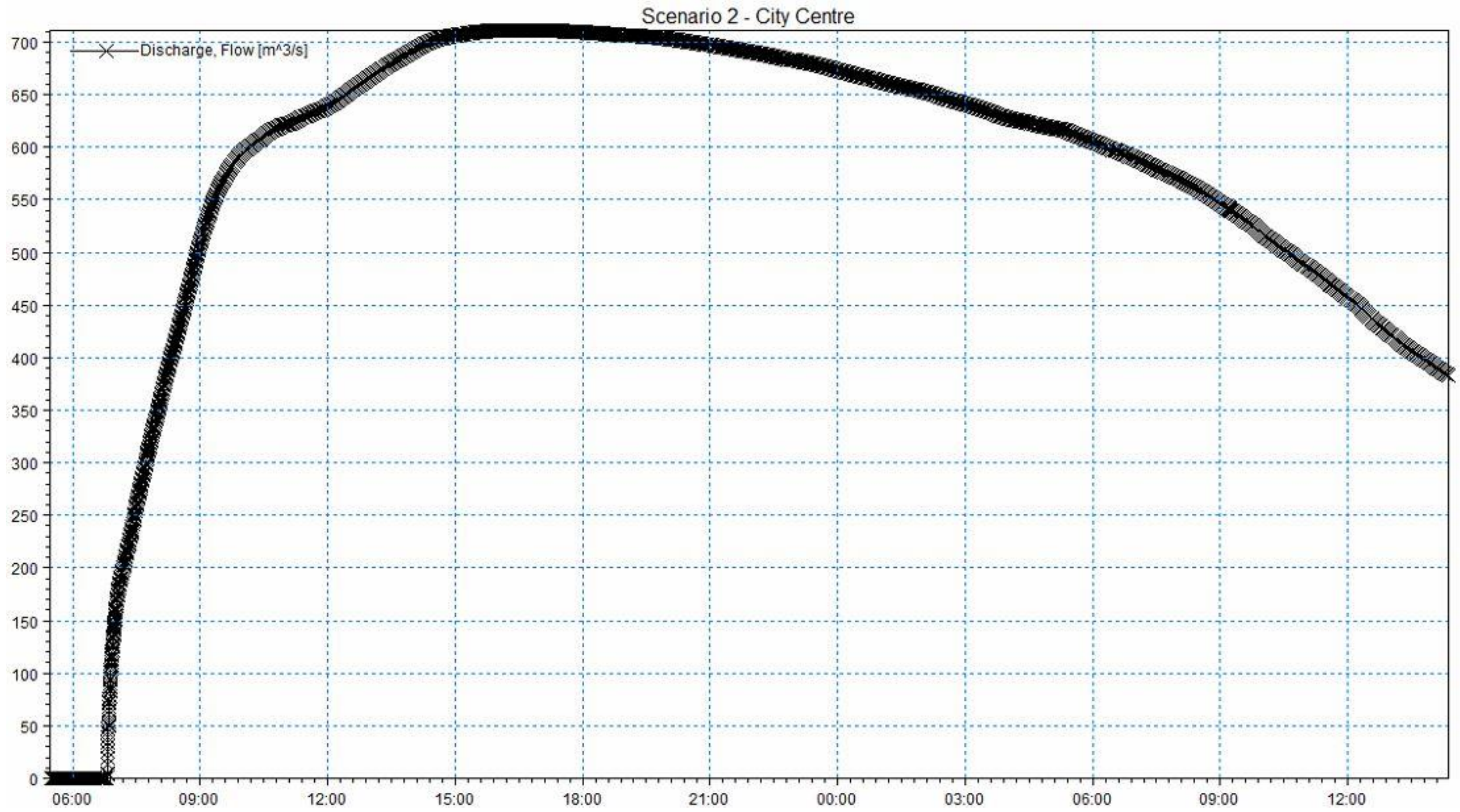


**Figure B.5:** Scenario 2 Result

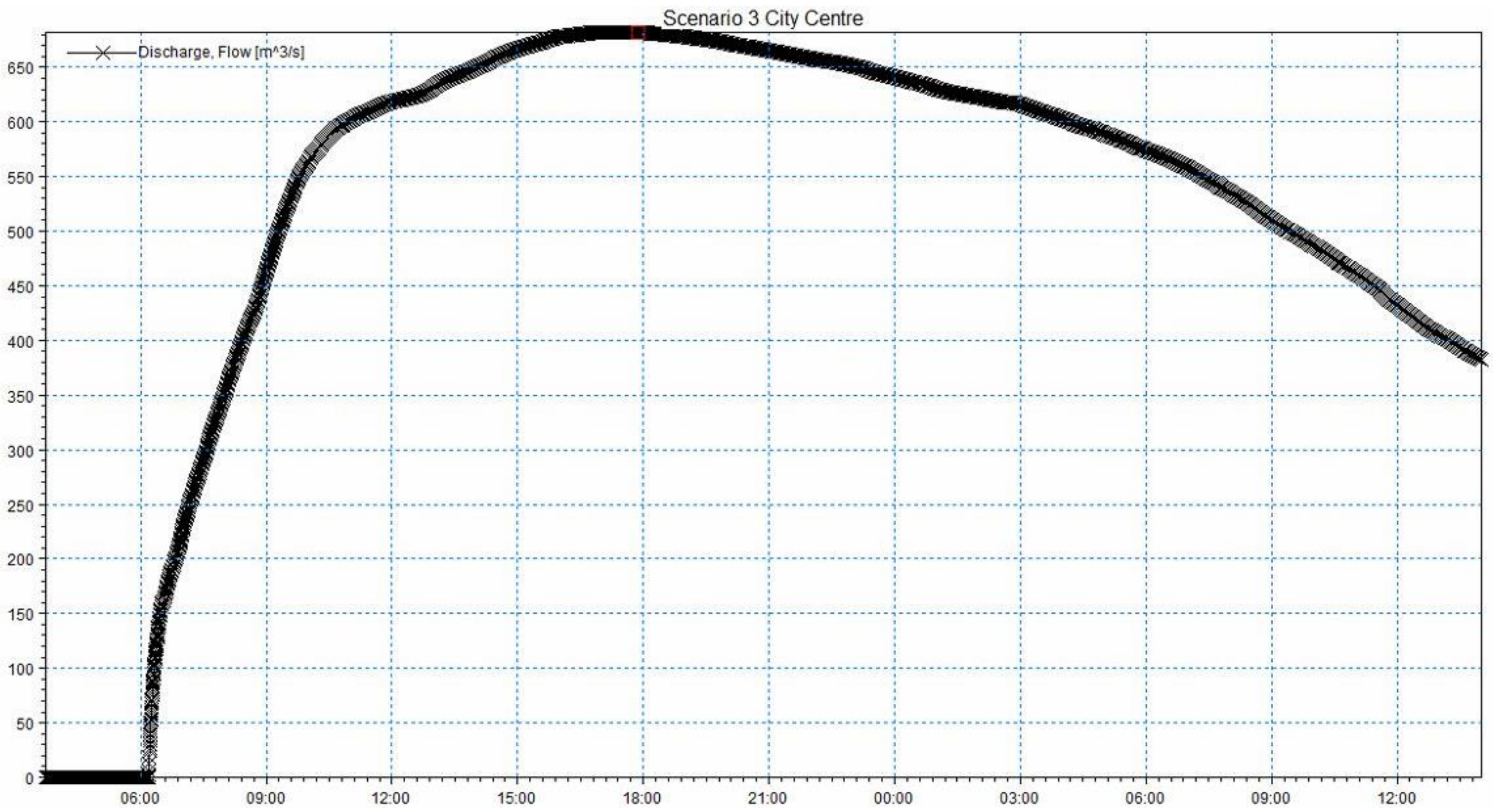


**Figure B.6:** Scenario 3 Result

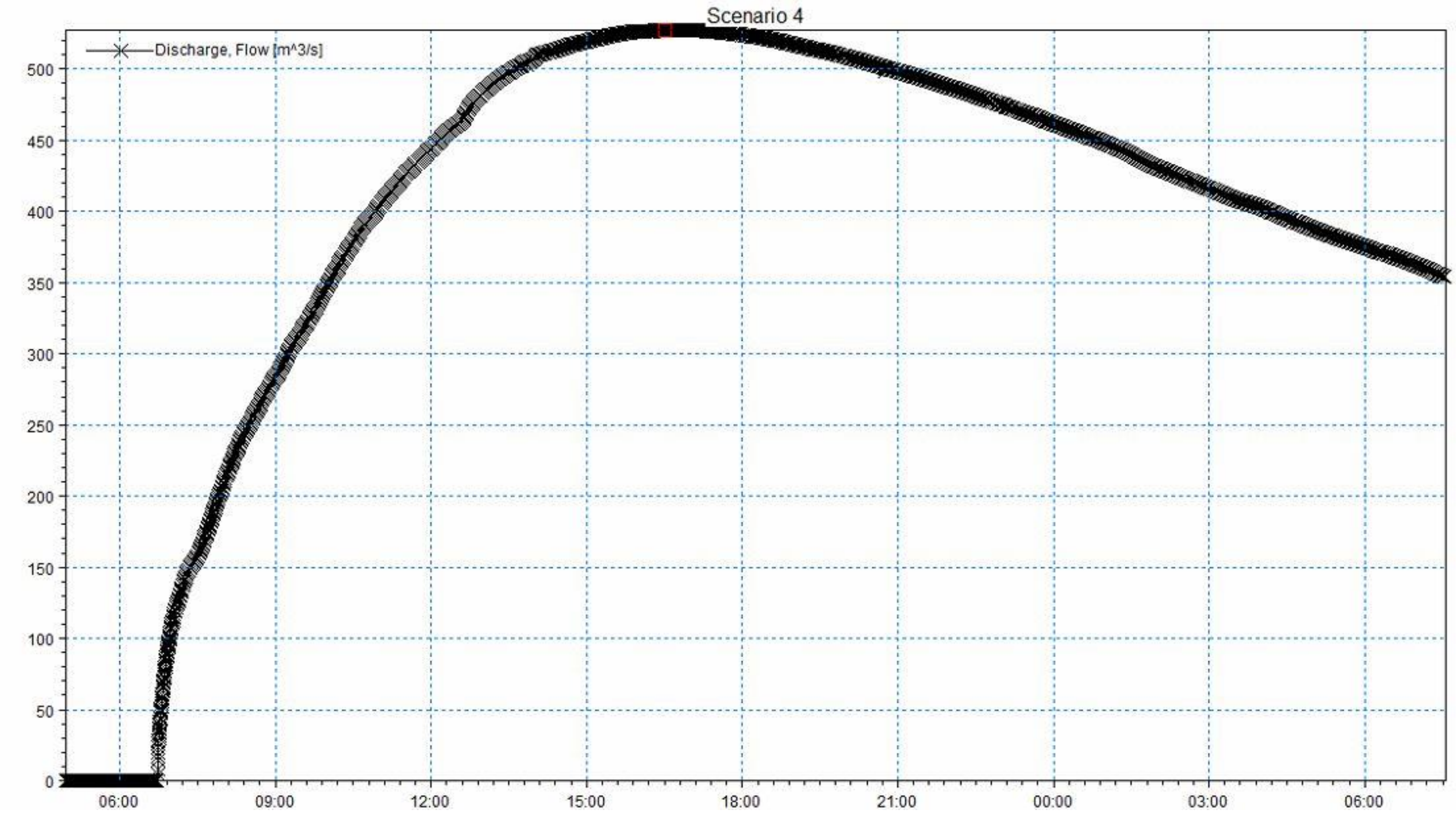


**Figure B.7:** Scenario 4 Result

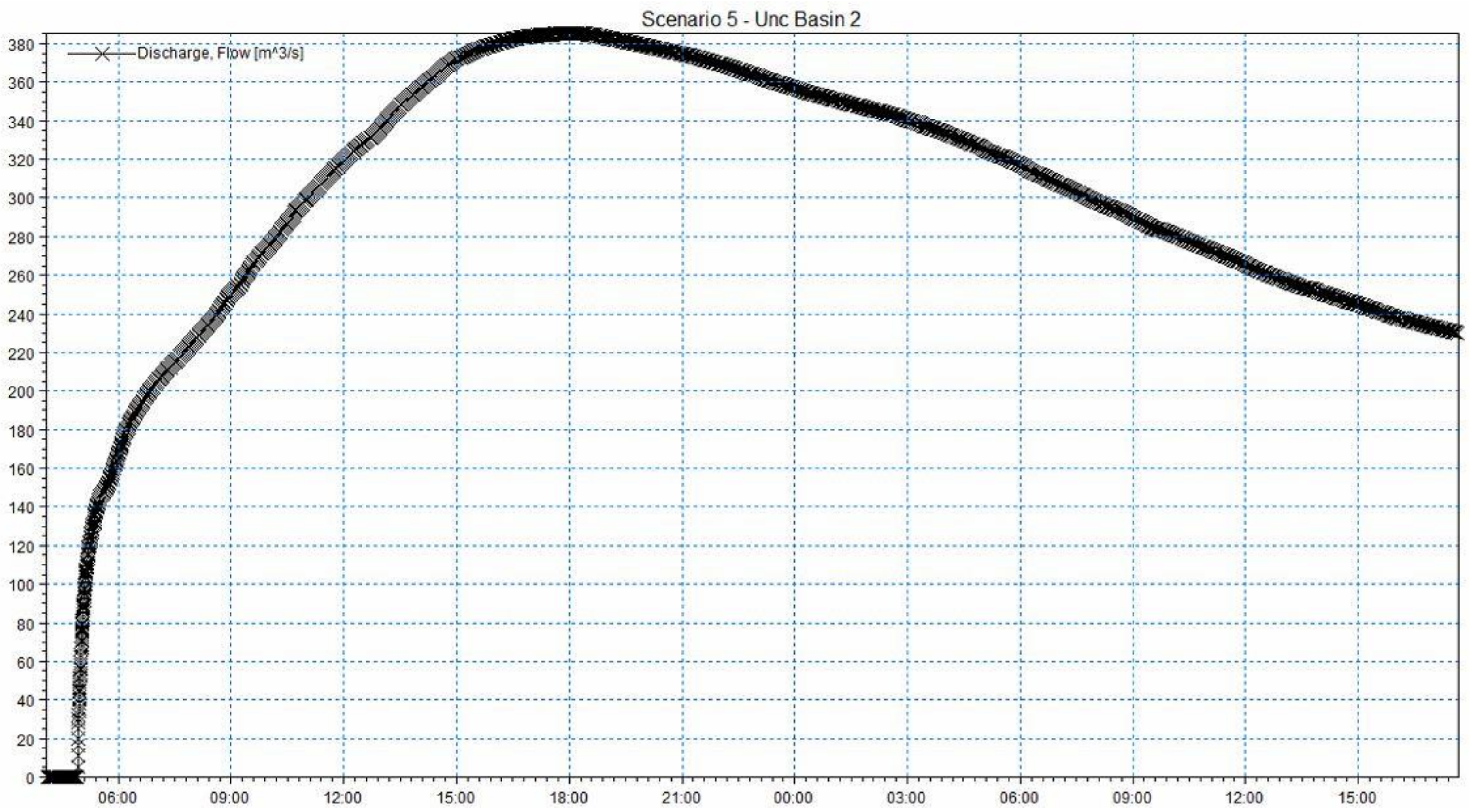


**Figure B.8:** Scenario 5 Res. 1

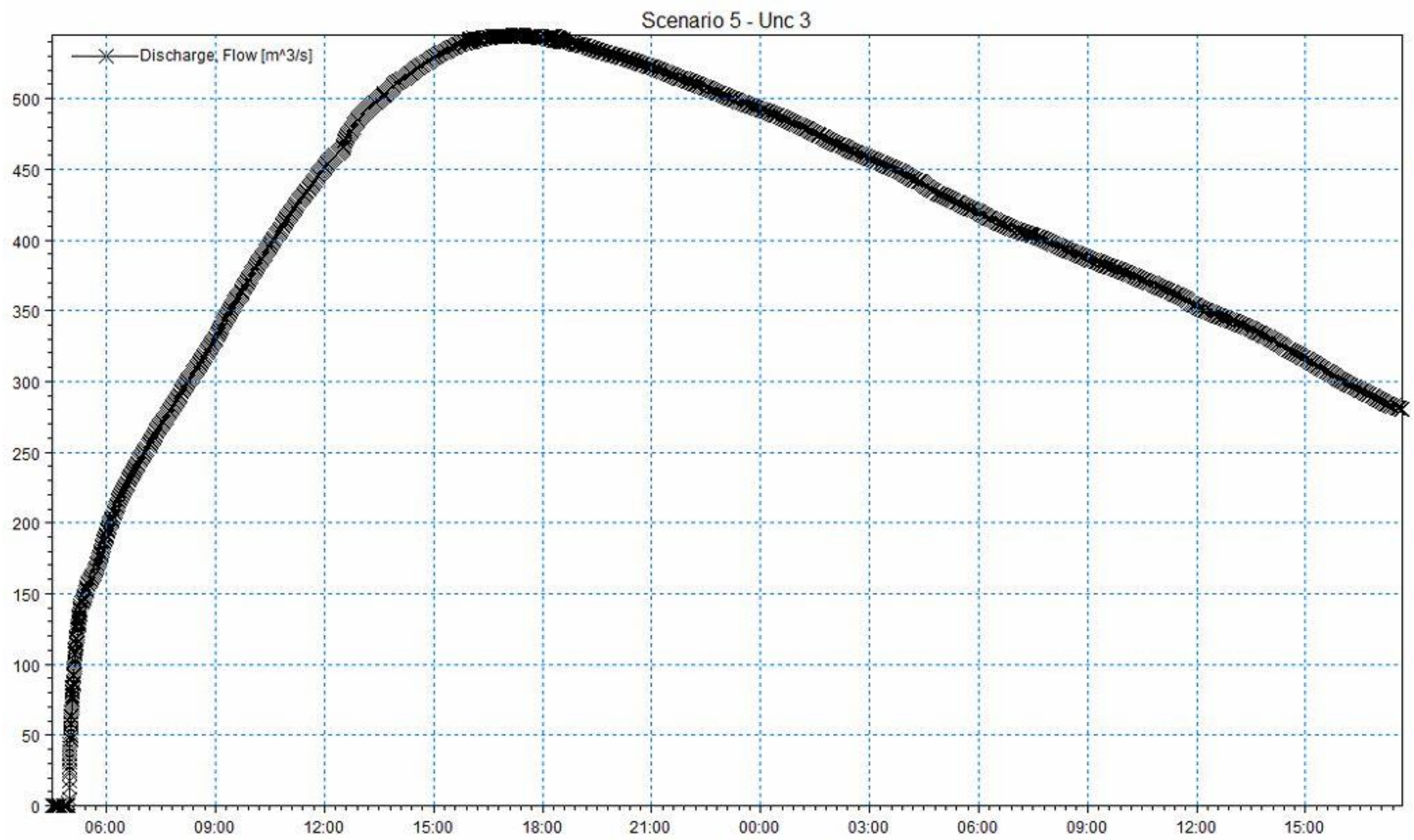


**Figure B.9:** Scenario 5 Res. 2

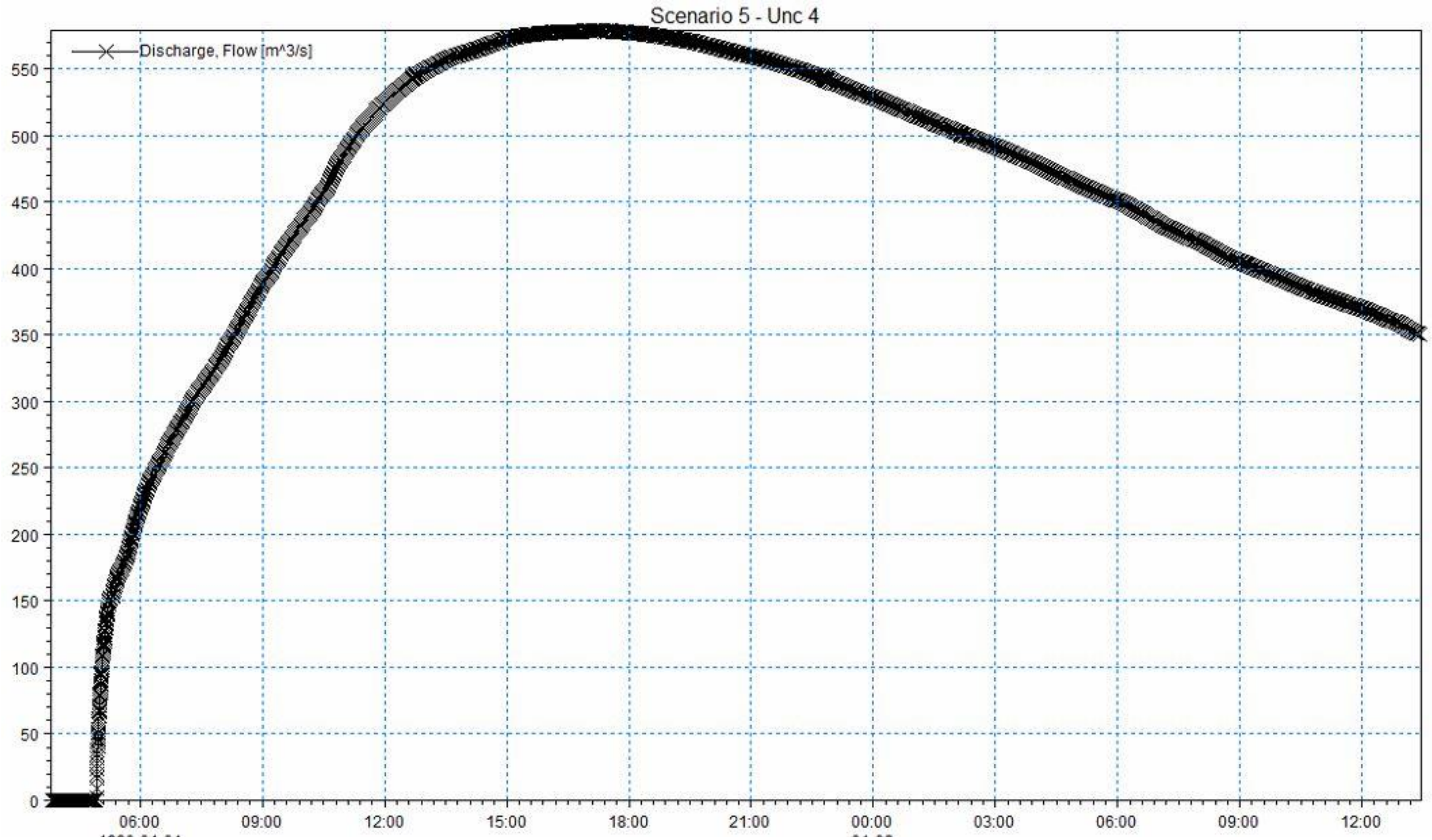


**Figure B.10:** Scenario 5 Res. 3

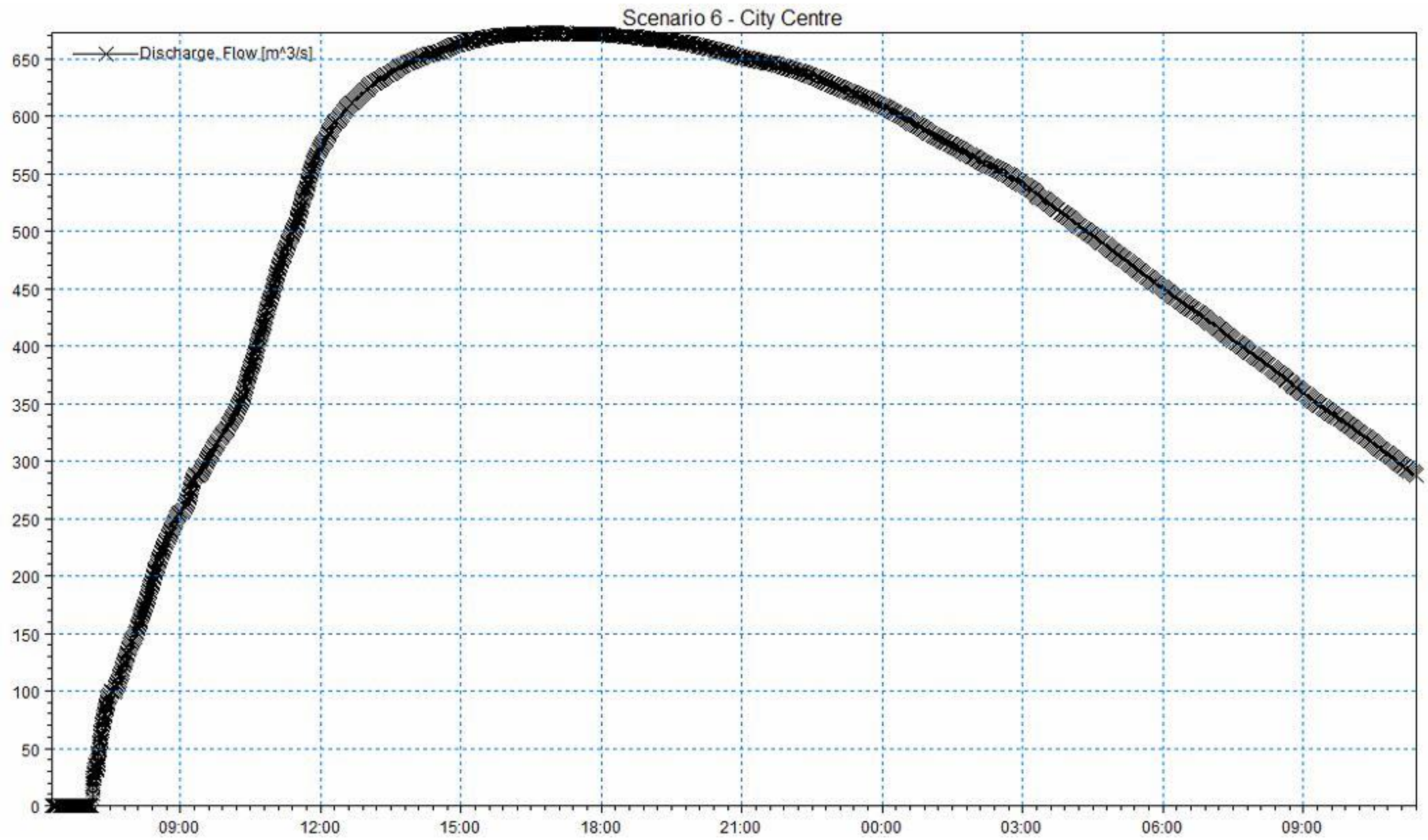


**Figure B.11:** Scenario 6 Result