Design flood estimation for the Quang Tri Province in Vietnam

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Summary

Estimating design floods for catchment areas is an important activity in hydrology. The sizing of bridges, culverts and other facilities; the design capacities of levees, spillways and other control structures; and reservoir operation or management depend upon the estimated magnitude of various design flood values. Design floods are also important for political decisions in hydrologic discussions.

The research goal of this study was to find the design floods for all the sub-catchment areas of the Ben Hai River, the Thach Han River and the O Lau River in the Quang Tri Province using Flood Frequency Analysis and the rainfall-runoff model MIKE-NAM. Quang Tri is located in the middle of Vietnam.

Flood- and rainfall frequency analysis has been done with FFA Spreadsheet 2.0. This was the first part of the research. Because discharge data were available for only one sub-catchment area, a rainfall runoff model called MIKE-NAM had been used to produce flood hydrographs for the other subareas.

This model has been calibrated with flood data from 1998 and 1999. It was easier to find a set of parameters of catchment area characteristics for the 1999-flood than for the 1998. The final set of parameters has a value for R^2 = 0.761 for the 1998-flood and R^2 = 0.925 for the 1999-flood. This R^2 describes how the modeled graph fits to the observed graph and has a minimum of 0 (no fit) and a maximum of 1.0 (perfect fit).

Because the output of this method wasn't as accurate as planned two additional methods had been added. The 'rational method' uses characteristic times to estimate design floods. The three-day rainfall method uses an increase factor and a historical rainfall to get an approach of the design floods. However, this last method is especially important for the total discharge volume.

The most important conclusion of the research is that the limited available data is a limitation for the accuracy of the research. Hourly rainfall data during storms will improve the accuracy of the results. Discharge data for more than one gauging station in the subarea also will improve the accuracy of the research.

Another important conclusion is that MIKE-NAM isn't suitable for design flood estimations. An more advanced model including techniques to translate design rainfall in design floods is recommended for further research.

Preface

During my stay in Vietnam I was watching national television. The channel showed the destroying typhoon Ketsana in the middle of Vietnam. The 28th and 29th of September 2009 were black days for many citizens in this area. In Vietnam only, 41 people didn't survive this heavy storm. Although this storm was almost thousand kilometers away from my room in Hanoi, I realized I was working on a *hot* and important subject.

Vietnam is a very interesting country for water research. Vietnam has a coastal line with a length of more than 3000km. Two big river deltas are located in the country. During the monsoon season extreme rainfalls can occur in parts of the country. These subjects could be input for many water studies.

I am happy I could do such a water research in this country. That is why I want to thank Mr. Giang for giving me this opportunity and for his assistance. I also want to thank Mr. Booij for his support. The accurate feedback he gave me had really helped me. He also helped me with the search for an assignment.

I also want to thank everyone who showed me some of the Vietnamese culture, especially Mrs. Lieu and Mr. Tuyet. They made my stay in Hanoi much easier and I have really appreciated their care and hospitality.

I also want to thank my best Vietnamese friend Duc. He introduced me to a lot of Vietnamese and Western people.

At last I want to thank everyone in Holland who supported me during my stay in Vietnam. I liked their mails and their reactions on my stories.

Hanoi, November 2009
Michiel

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1 Introduction

Estimating design floods for catchment areas is an important activity in hydrology. The sizing of bridges, culverts and other facilities, the design capacities of levees, spillways and other control structures, and reservoir operation or management depend upon the estimated magnitude of various design flood values (Fang et al. 2007). Design floods are also important for political decisions in hydrologic discussions. The design floods are the 'chance' in risk calculation and are necessary for cost-benefit analysis.

A tool for design flood estimation is *flood frequency analysis.* This tool is a mathematical approach of design flood estimation. This tool uses historical discharge data and probability functions to estimate design floods (Ramachandra Rao & Hamed, 2000).

The behavior of rainfall-runoff is also an essential part of flood estimation. Precipitation influences the water level and the discharge in rivers. Rainfall runoff modeling is a method to translate rainfall data into discharge data. The influence of environmental factors, as land occupation; difference in height and level of groundwater, on the rainfall runoff can be modeled. It is essential to calculate flood hydrographs for all required locations on the catchment and floodplain for input to future hydraulic models(Hydrologic report, 2003).

The research goal of this study is to estimate the design floods for all the sub-catchment areas in the Quang Tri Province using Flood Frequency Analysis and the rainfall-runoff model MIKE-NAM.

A design flood is a flood with a specified annual exceedence probability. For example, an annual exceedence probability of one in hundred years gives a *design flood* of [Z]m^3/s in River R.

Quang Tri has a monsoon climate. Because province has a wet period and a dry period in a year, the variation in the discharge of the rivers is huge. In the dry period the discharge is sometimes not more than 10m^3/s, but in the wet period the maximum value can be more than 1000m^3/s. In this study these maximum discharge values will be investigated.

There have been done more studies on this subject for tropical areas like Quang Tri. For example a study has been done for the Quang Ngai Province in the middle of Vietnam (Hydrologic report, 2003). The Hydrologic Report (2003) used the rainfall-runoff model RORB instead of MIKE-NAM.

In the first two parts of this report a mathematical approach of design flood and design rainfall estimation is given. Using the Log-Pearson III distribution and data sets an approach of design floods and design rainfalls has been made.

The next part involves the MIKE-NAM model to translate the design rainfalls into matching discharges. MIKE-NAM is a hydrological model to simulate runoff from the rainfall and is developed by DHI Water & Environment. Working procedure of NAM is to calculate runoff in the certain (sub)-catchment areas from design rainfall after deducting water loss including storing on the leaf tree and infiltration. Runoff is created and runs through number of reservoirs including surface, sub-surface and two reservoirs under the surface. Catchment is considered as number of sub-catchment and water will transport from surface to the springs and finally to the river (Vu & Bui, 2007).

This report continues with the study area and the available research data in chapter 2. Chapter 3 describes the statistical methods of design flood estimation. In the first part of chapter 3 the tool 'flood frequency analysis' will be explained. An equal frequency analysis has been done in the second part of this chapter, but this time for rainfall data. Chapter 4 describe the rainfall-runoff modeling with MIKE-NAM, including the model calibration and model application. Chapter 5 gives an additional method for design flood estimation. The results and discussion can be found in chapter 6. Chapter 7 finishes the report with a conclusion and recommendations.

2 Study area and research data

This chapter will give relevant information about the study area. Especially about the big rivers, the climate and the several sub-catchment areas of the province. The second part of this chapter will present the available data collected by the Hanoi University of Science.

2.1 Study area

The study area of this research is the Quang Tri Province in Vietnam. Quang Tri Province is located in the central part of this country (see figure 1). Quang Tri has very special geographic characteristics, with mountains in the west and a very flat east side. The east side has a coastal zone which has two main rivers, the Ben Hai River and the Thach Han River. These two rivers have many tributaries and finish in the East Sea (Krutwagen, 2007). Because the eastern part of the province is relatively low, the sea plays an important role in the hydrological regime of the estuaries.

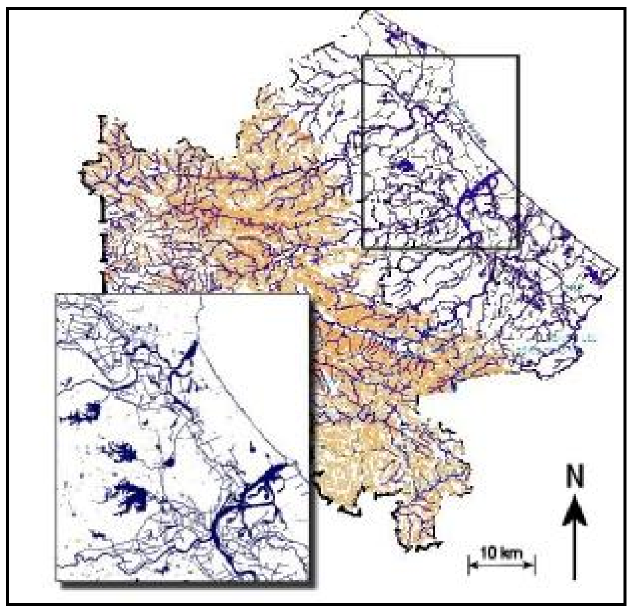


Figure 1 (left): Location of Quang Tri Province
Figure 2 (right): River network in Quang Tri Province with detail of the estuaries

2.1.1Rivers

The northern river (see figure 2) is the Ben Hai River. The Ben Hai River basin has a total surface area of 809 km^2. The most important tributary of Ben Hai River is the Sa Lung River. The southern river (see figure 2) is the Thach Han River. The catchment area of the Thach Han River is 2660 km^2. The two most important tributaries of Thach Han River are Thach Han and Cam Lo River. The average discharge of the Ben Hai River is 43.17 m^3/s and the Thach Han River has an average discharge of 128.25 m^3/s. Although not located in the Quang Tri Province, a third river is important for the hydrological regime in this area: O Lau River. This river is located for the biggest part in the Hue Province, south of Quang Tri Province. However, downstream it has its influence on the flood inundation in the Quang Tri Province. O Lau River has a catchment area of 885 km^2.

2.1.2 Climate

Quang Tri Province is located in the tropical zone. This means the annual average temperature is above 18°C. The province has a dry and a wet season. The average rainfall is presented in figure 3 for seven rainfall stations in and near the Quang Tri Province. Monsoons are common in the wet season. These characteristic heavy rainfalls occur mainly in September, October and November and come from the eastern sea side.

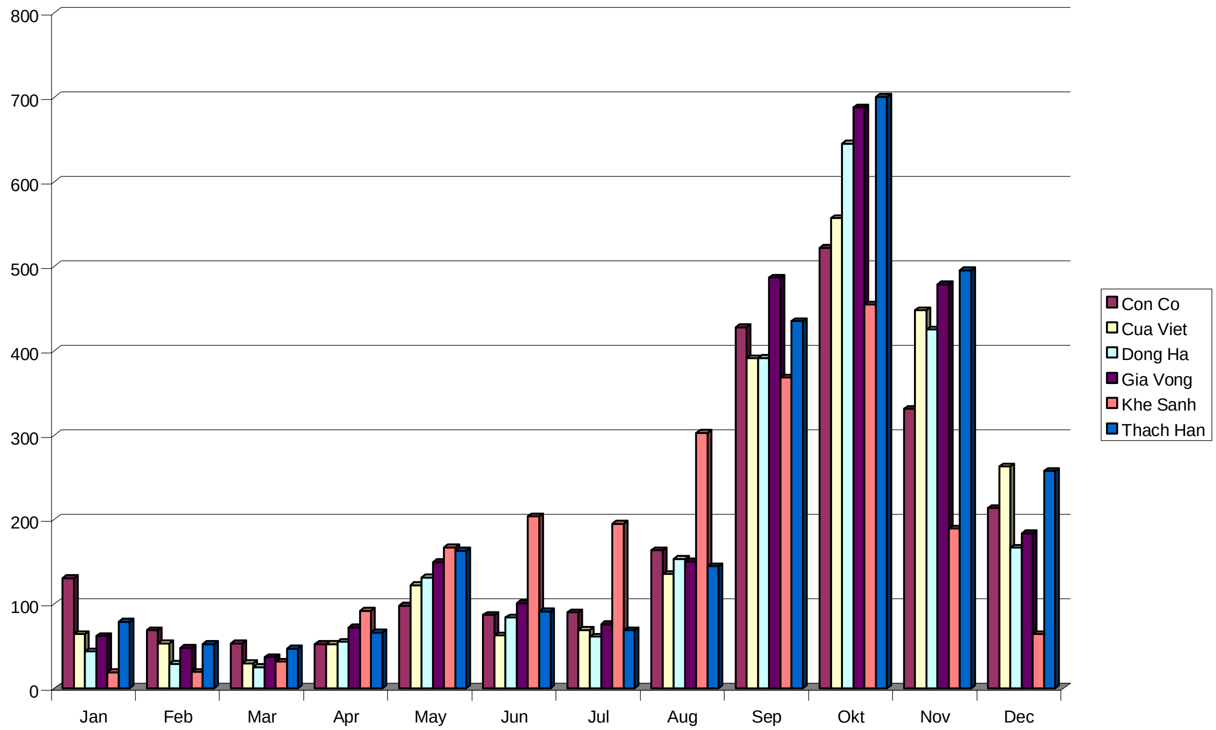


Figure 3: Average monthly rainfall [mm] in the Quang Tri Province over the period 1976-2000

2.1.3 Sub-catchment areas

For this study the Quang Tri Province and a part of the Hue Province is divided in smaller areas. With a GIS-program the study area is divided in catchment areas and subareas. The highest points in the study area will form the boundaries of the subareas. The partition is based on tributaries of the rivers. The study area is divided in 19 subareas and a coastal basin. These sub-catchments are important for the next steps in this study, because they will be used as input in MIKE-NAM model. In figure 4 the partition of the study area is presented. The big basin in the eastern part is the coastal basin. Because of the low land, this basin will experience the biggest trouble during a big flood. In this basin, the Thach Han River and the Ben Hai River have a connection. The coastal basin has a surface area of 871.5 km^2. This basin won't return in MIKE-NAM model. The surfaces of the other sub-catchments are shown in table 1. The first two letters are representing the river, so SL is Sa Lung; BH is Ben Hai; CL is Cam Lo; TH is Thach Han and OL is O Lau. The total surface areas given in table 1 are the surface areas given in paragraph 2.1.1 minus the part of each catchment area located in the coastal basin.

Table 1: Surface area of the sub-catchment areas in the Quang Tri Province

subarea	Area [km^2]		subarea	Area [km^2]		subarea	Area [km^2]
SL1	80.5		CL1	334.2		OL1	238.4
SL2	225.1		CL2	65.4		OL2	161.9
SL3	26.3		CL3	73.1		OL3	93.6
BH1	275.2		TH1	1067.0		OL4	44.8
BH2	72.8		TH2	192.2		OL5	75.5
BH3	15.6		TH3	95.2			
			TH4	43.8			
			TH5	312.0			
Total	**695.5**			**2182.8**			**614.1**

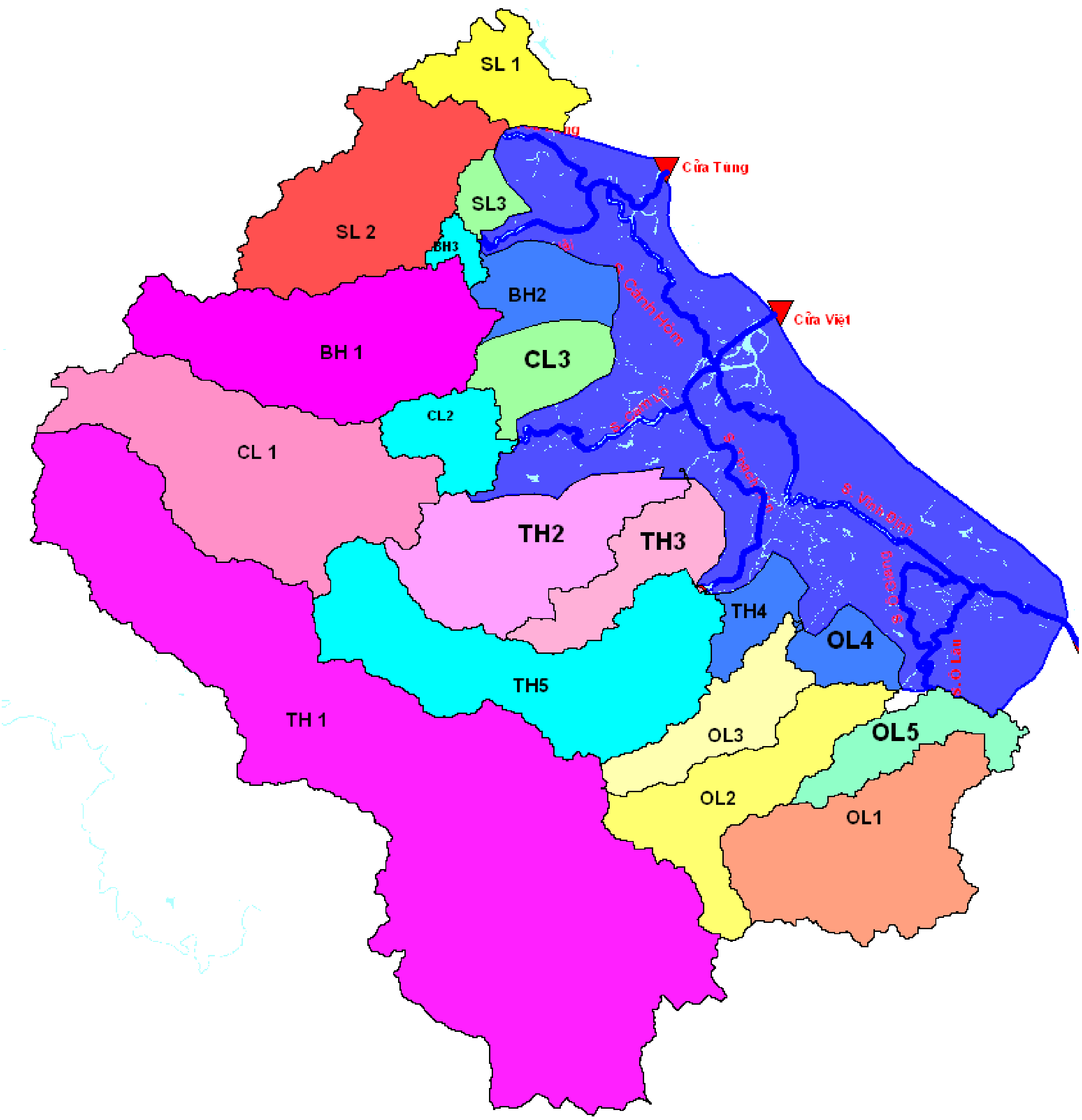


Figure 4: Map of the sub-catchments in the Quang Tri Province including the coastal basin

2.2 Data

For this project rainfall-, discharge and evapotranspiration data is collected by Hanoi University of Science.
Because of the technologic restrictions in the past, the amount of rainfall – and stream flow data is limited for most catchment areas. The flood frequency analysis results will be affected by the short lengths of the available data (Boughton & Droop, 2002). For example, it will make a big difference if there either has been or hasn't been a few huge discharge peaks in the recorded data.
However, there is only one huge peak in the available data, the Hydrologic report (2003) calls the limited data suitable.
The available data is:

- Daily rainfall data for seven gauging stations in and near the Quang Tri Province. These data has been measured over the period 1976-2000.
- Discharge data for one gauging station (Gia Vong station) on the Ben Hai River. For this gauging station annual maximum discharge is available in the period 1977-2004 and for 5 periods six-hourly data is available. Because the discharge is very low in dry periods, only the discharge in wet periods has been measured.
  Six-hourly data is available, but for the days with big floods even hourly discharge data is available.
  The five available periods are:
    - September – November 1983 with a peak discharge of 1280m^3/s.
    - August – November 1990 with a peak discharge of 1660m^3/s.
    - September – November 1995 with a peak discharge of 1420m^3/s.
    - September – December 1998 with a peak discharge of 1090m^3/s.
    - October – December 1999 with a peak discharge of 1060m^3/s.
- Evapotranspiration data for two gauging stations in the Quang Tri Province. This data is available for the period 1975-2004.

3 Statistical methods

Flood frequency analysis

3.1 Introduction

With a statistical method a first approach of the design floods has been made. The available historical discharge data has been analyzed with the tool flood frequency analysis. This analysis is a way to calculate flood discharges of defined probability. The next paragraphs will describe the model, the model application and the output. After the flood frequency analysis design floods are available for the sub-catchment area with historical discharge data. These design floods will be compared with deterministic found design floods in further chapters.

3.2 Input

For flood frequency discharge data or water level data is required. Discharge data is more suitable, because it has a linear connection with the amount of water. Water level data is not linear, because it depends on the shape of the river-bed. So in this study discharge data has been used. Unfortunately there is only one gauging station with discharge data available: Gia Vong (see paragraph 2.2).

3.3 Model description

For the flood frequency analysis a spreadsheet tool has been used: FFA Spreadsheet 2.0. This tool is developed by the 'Cooperative Research Centre for Catchment Hydrology', nowadays known as the eWater CRC (Cooperative Research Centre). eWater is a technology development initiative set up by Australia's water resource management and research sector. FFA Spreadsheet can be used as a macro-file in Microsoft Excel.

3.3.1 Log-Pearson III distribution

The FFA Spreadsheet 2.0 model is based on the Log-Pearson III distribution. This distribution has been recommended for application to flood flows by the U.S. Interagency Advisory Committee on Water Data (1982) (Bedient & Huber, 1988, p. 177) and is part of the gamma distribution family. The assumption is that this distribution also can be used for Quang Tri.
As a formula the distribution function of the log-Pearson III distribution is given as:

$$F(x)=\frac{1}{a\Gamma(b)}\int_0^x \frac{1}{x}\left[\frac{\log(x)-g}{a}\right]^{b-1} e^{-\left\{\frac{\log(x)-g}{a}\right\}} dx \qquad (1)$$

Where $\Gamma(\cdot)$ = the gamma function

α = Shape parameter

β = Inverse scale parameter

γ = Location parameter. This parameter is dependent of α, β and the average of the values.

The parameters in this formula could be estimated with three different methods.

- Method of Moments
    - Direct Method of Moments
    - Indirect Method of Moments

- Maximum Likelihood (ML) Method
- PWM Method (Probability Weighted Moments Method)

The model uses the Method of Moments to estimate the parameters (CRC of Catchment Hydrology, 2001).

3.3.2 Flood frequency model

Three different models can be used for flood frequency analysis: Annual maximum series (AM) model; Partial duration series (PD) or peaks over threshold (POT) model; Time series (TS) model.

The FFA Spreadsheet 2.0 is based on the annual maximum series model. In this model the peak flow in each year of record is considered. However, the use of an AM series may involve some loss of information. Kite's study (as cited in Ramachandra Rao & Hamed, 2000) says: "For example the second or third peak within a year may be greater than the maximum flow in other years and yet they are ignored".

3.4 Model application

3.4.1 Annual maximum values

First step is to select the annual maximum values. The length of the period of record data is important for the reliability of the analysis. For the flood frequency analysis of the Gia Vong gauging station there are 28 years of data (1977 – 2004) available. This amount seems not much, because conclusions for 100 year flood discharges will be done, but for flood frequency analysis this amount of data is enough, although the level of uncertainty increases with a smaller dataset. Of course there may not be any trend in the discharge data, because any trend will break down the randomness. Watching the available data in figure 5 , there doesn't seem to be any trend like increasing or decreasing annual maximums, so it can be assumed this data set is suitable for flood frequency analysis.

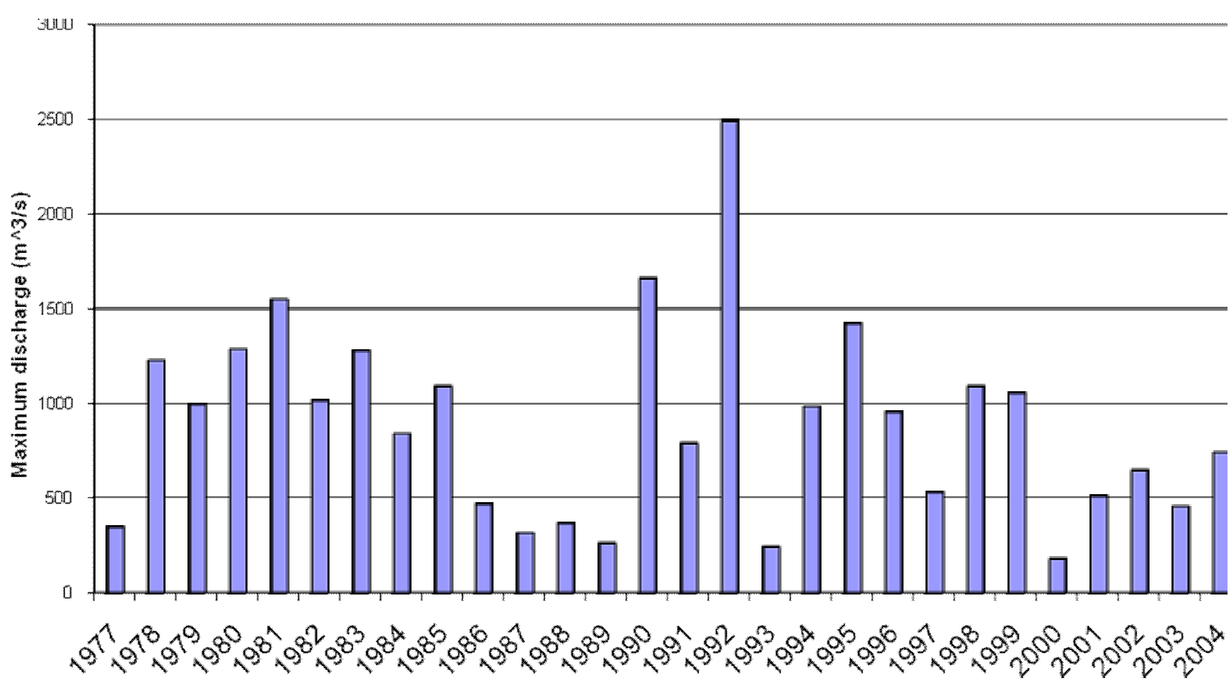


Figure 5: Maximum yearly discharge measured on Gia Vong station.

3.4.2 Ranking

These maximum values are the input for the FFA Spreadsheet 2.0 model. The model ranks the annual series and assigns a probability for each recorded flood. The probability that flood of a defined size or larger will occur in any one year is calculated using the 'Cunnane formula' as:

$$P = \frac{(m-0.4)}{(N+0.2)} \qquad (2)$$

Where P = probability of exceedence for flood peak
m = rank of flood peak in annual series (with the largest flood = 1)
N = number of years of record

This formula suggests that the flood in the middle of the ranked data has a probability of exceedence of 50% every year.

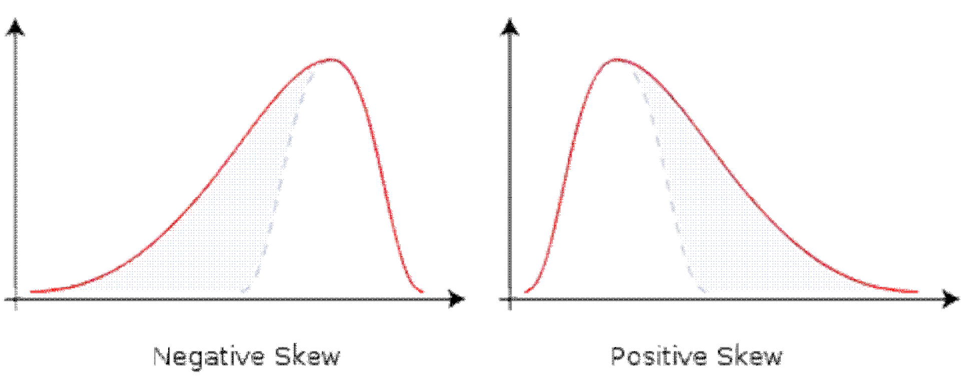
3.5 Output

3.5.1 Prediction (or confidence) limits

The results will be given with lines of 95% prediction limits. So the probability that the real value is between those lines is 95%. These limits have been chosen because 95% is an acceptable value of prediction limits in hydrological studies.
For example the 10-year flood for Gia Vong will give the next values:
0.95: 1294
Q_Y: 1625
0.05: 2041
This means, that it is for 95% sure that the real value of the 10-year flood is bigger than 1294 m^3/s. And it is also for 95% sure that the real value of the 10-year flood is smaller than 2041m^3/s.

3.5.2 Skew and omitting flows

For each distribution a skew factor is calculated by the model. This factor represents the asymmetry of the tails of the distribution. If the skew factor has a big negative value (-0.4 or less according to CRC of Catchment Hydrology, 2001) the low tail of the distribution is 'too asymmetric'. This means the influence of one or more low values of annual maximum discharges is too big. The solution is to omit one or more low flows.
In figure 6 the effect of negative and positive skew is shown. Small or big values in a distribution make the graph asymmetric.



Figuur 6: effect of negative and positive skew on a distribution

Rainfall frequency analysis

3.6 Introduction

September, October and November are the wettest months in the Quang Tri province. In figure 7 the distribution of the average rainfall for the several gauging stations in and near the Quang Tri Province is shown.
The discharge in the rivers is linked to the rainfall in the upstream catchment area. Because of the limited discharge data for the Quang Tri Province, rainfall data has been used to analyze the annual exceedence probability of floods. These estimations will be presented in chapter 4. Input for these estimations is the design rainfall. These paragraphs will show the method to estimate these design rainfalls. This method is called rainfall frequency analysis and is also a statistical method. Rainfalls of defined probability will be know after these paragraphs. This rainfall frequency analysis will be done for one -, two – and three-day rainfall.

3.7 Input

For rainfall frequency analysis rainfall data is required. The data is available for seven rainfall gauging stations (see figure 8) over the period 1976-2000.

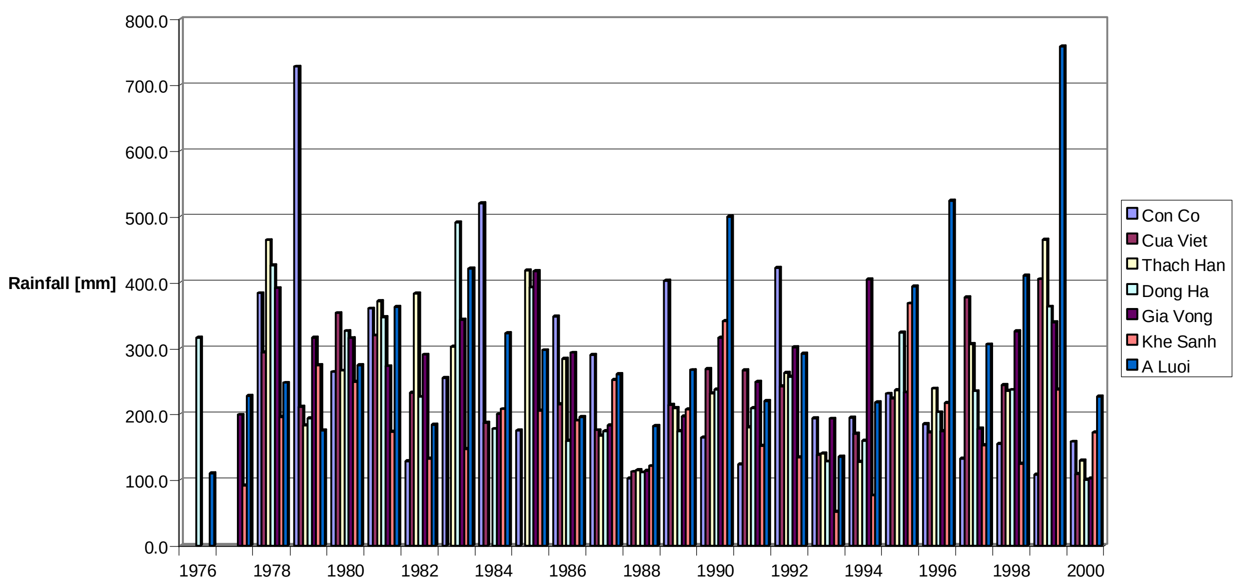


Figure 7: Annual maximum daily rainfall for seven rainfall stations.

3.8 Methodology

The methodology of the rainfall data analysis is similar to the methodology of the flood frequency analysis (see paragraph 3.1). Maximum annual values have to be selected again. This has been done for daily, two-daily and three-daily rainfall. This difference has been made, because in small (sub-)catchment

areas a rainfall of one day can produce the highest peak discharge, but for larger catchment areas it will probably be two or three days.
The maximum values for daily rainfall are shown in figure 7. The annual maximum two- and three-day rainfall graphs are placed in appendix A-1.
Using the *FFA Spreadsheet 2.0 model* based on the Log Pearson III distribution an approach of the design rainfalls has been made. This method has been applied in the Hydrologic Report (2003) for the Quang Ngai province, so it will be applied in this report too.

3.8.1 Storm duration

The annual maximum rainfall has been used for one, two and three day storms. Obviously it is possible that a daily maximum rainfall occurs in October of a specific year, but a two-daily maximum a month later.
All these three storms durations have been analyzed; because every catchment area has its own critical storm duration (see chapter 4 and 5). These storm durations determine the maximum discharge in the rivers.

3.8.2 Short term rainfall

There is only daily rainfall data available, but it is possible that the critical storm duration (see chapter 5) is smaller than one day. So it is necessary to have suitable values for the annual exceedence probability of three-, six- and twelve-hour rainfall.
In several other studies like Hydrologic report (2003) the percentages of the daily rainfall were [0.32], [0.47] and [0.75] for three-, six- and twelve-hour rainfall. These weight factors have also been applied to this study. This means a typical twelve-hour storm produces a water volume of 75% of the total water volume of a 24-hour storm.

3.8.3 Skew

The skew factor is an important characteristic of the rainfall frequency analysis, because it tells something about the 'quality' of the dataset. As in paragraph 3.5 too low values of skew have been corrected by omitting one or more low flows. In table 2 the skew factors of every rainfall station are presented. The underlined values were found after omitting one or more low storms.

Table 2: Skew factors for AEP rainfall graphs.

Rainfall station	1 day	2 days	3 days
A Luoi	0.207	0.113	0.465
Con Co	0.408	0.981	0.942
Cua Viet	-0.312	-0.058	0.071
Dong Ha	-0.098	-0.098	0.033
Gia Vong	-0.166	-0.383	-0.355
Khe Sanh	-0.249	0.019	-0.061
Thach Han	0.107	0.000	0.482

4 Rainfall runoff modeling

Part 1: Calibration.

4.1 Introduction

Using rainfall data and a rainfall runoff model, the behavior of the rainfall runoff of several floods can be simulated. This behavior will be visualized in flood hydrographs. The important quantities are

- The cumulative water volume of the rainfall and flood. This quantity has a direct link with the water quantity in the coastal basin after a big storm.
- The peak discharge of the flood. This quantity is important because of the sizing of dikes, dams etc.

The shape of the hydrographs is also an important factor to criticize the behavior of the rainfall runoff, but not as important as the two quantities mentioned above.

The rainfall runoff model MIKE-NAM has been used to calculate flood hydrographs. MIKE-NAM is a part of the MIKE11 model. MIKE11 is a model for river systems. MIKE-NAM is one of the rainfall runoff model types that can be chosen. The choice for MIKE-NAM has been made on advice of the Hanoi University of Science (Personal communication with N.T. Giang, 2009).

4.2 Input

The required input for MIKE-NAM is listed below.

Daily rainfall data

For this study rainfall data has been used from September 1998 and November 1999. These specific periods have been chosen because a flood occurred in these months. The 1998-flood has duration of 84 hours and the simulated flood of 1999 has duration of 156 hours. The data is available for seven different rainfall gauging stations in or near the Quang Tri province (see figure 7).

Potential EvapoTranspiration data

MIKE NAM requires Potential EvapoTranspiration data as input for the model. The data has been used for the specified periods mentioned above. This data is available for two gauging stations: Khe Sanh and Dong Ha (see figure 7). The Potential EvapoTranspiration in September, October and November is usually between 0 and 6 mm/day. In comparison with the precipitation in these periods, these values are almost negligible. However, the values have been added to the model, because Potential EvapoTranspiration data is required and it will make the simulation complete.

Discharge data

For the calibration of the model observed discharge data is required. For Gia Vong gauging station discharge data is available. For September 1998 and November 1999 six-hourly discharge data has been used. Hourly data is available but only for 24 hours. Unfortunately MIKE NAM requires equal time steps, so six-hourly data has been chosen.

Map of the catchment areas

In paragraph 2.1.3 the map of the subareas is given. This map has been loaded in MIKE-NAM (see figure 8).

The map includes the locations of the gauging stations. Data is available from seven gauging stations. The locations of these gauging stations are shown in figure 8. Five of these stations are located in the Quang Tri Province. One rainfall station called Con Co is located on a small island in the sea and another rainfall station is located in mountains of the Hue Province, south of Quang Tri. The last one is important for the O Lau River. This river is located for a big part in the Hue Province.

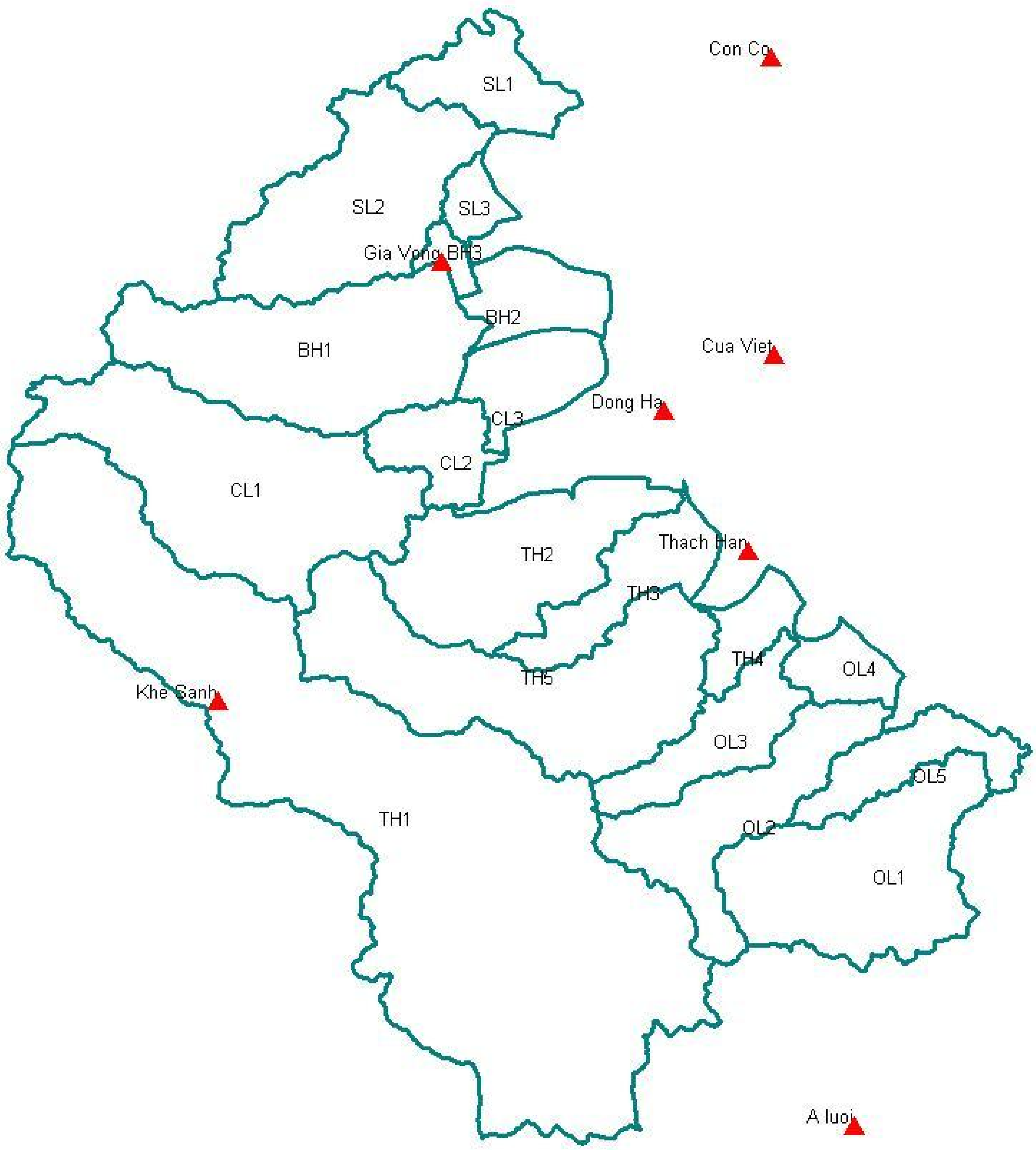
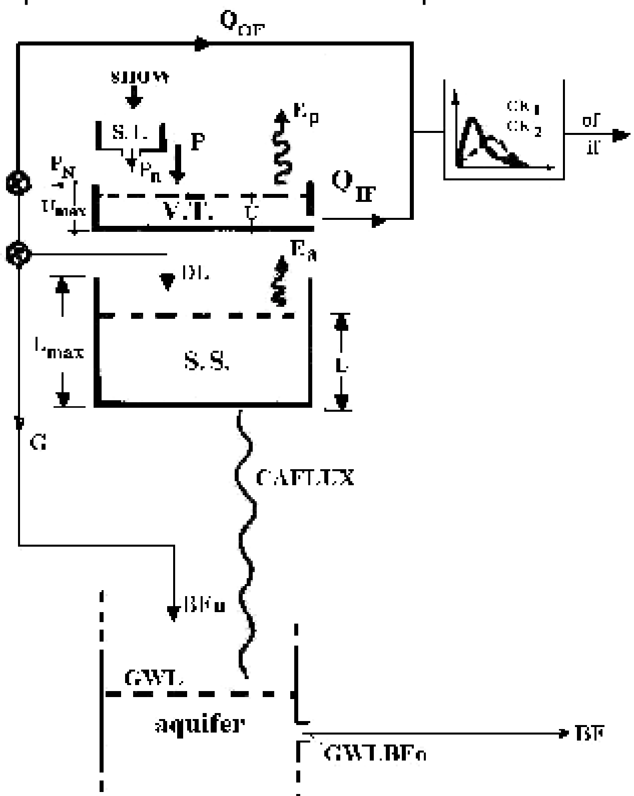


Figure 8: Map of the study area including the 19 sub-catchment areas and the seven gauging stations.

4.3 Model description

The structure of the MIKE-NAM model is presented in figure (x). Some of these parameters, like snot stock, hasn't been used in this research, because they were irrelevant. In the next paragraph the important initial conditions and the parameters will be described.



Figuur 9: The tank structure of the model MIKE11-NAM (Crăciun, 2003).

The parameters of the schema showed in the figure 9 are:

S.T. = Snow stock;
V.T. = Vegetation stock tank
S.S. = Surface stock tank
P = Rain
Ep = Evapotranspiration;
Ea = Evaporation to the surface soil;
U = Humidity of the vegetation;
Pn = Net rain;
QIF = Runoff flow;
L/Lmax = Linear variation of the water stock in the soil
CKi = Time of the hypodermic flow
PN = The excess rain which flow to the hydrographical network or is infiltrate in soil
QOF = The part of PN which go to the surface flow
G = Percolation;
GWLBFo= The distance between the superior level of the aquifer GWL and the water level in the river
BF = Base flow;
CAFLUX = Capillary flux

This chapter continues with the setup of the model. The initial conditions and the parameters will be clarified after the next paragraphs.

4.3.1 Thiessen's Weights Method

Rainfall data for each subarea is necessary, but rainfall data is only available for the seven rainfall gauging stations. Using the Thiessen's Weights Method an interpolation of the daily rainfall in each catchment area has been made. The rainfall in each subarea is an average of the rainfall measured in the rainfall stations. The Thiessen's Weight Method adds for each subarea a weight factor to each rainfall station. The weight factor depends on the location of each rainfall station according to the location of the subarea. The rainfall in some catchment areas depends on only one rainfall station, while the rainfall in other catchment areas depends on two, three or even four rainfall stations. In appendix B-1 a table shows the exact weight factors.

4.3.2 Initial conditions

A few conditions have to be set MIKE NAM before running the model. These are called the *initial conditions*. In table 3 the initial conditions of the parameters are given.

Table 3: Initial conditions for the MIKE-NAM model

NAM Parameter	Nam Parameter Description	Initial condition
U/U_{max} [-]	Relative water content in surface zone storage	1.0
L/L_{max} [-]	Relative water content in root zone storage	0.9
QOF [m^3/s]	Overland flow	0
QIF [m^3/s]	Interflow	0
BF [m^3/s]	Base flow	Depends on the surface area of the sub catchment (see equation 3)

Water content in the surface and root zone storage

The relative water content in the surface and root zone storage influences three important characteristics of the flood hydrographs. These characteristics are visualized in appendix B-2.

The first characteristic is the reaction time of the discharge according to the rainfall. When the relative water content in both layers is almost zero, the soil will absorb a big part of the first rainfall. When the soil is completely saturated this won't be the case, water won't infiltrate in the top layers and will cause a direct flow. An increase in discharge will occur sooner when the soil is saturated.

The second characteristic is the total volume of water passing the (virtual) gauging station during the storm. When the soil is dry, water will infiltrate. This water will find its way to the outlet of the subarea much slower than over land. So the total volume of rainfall water will find its way more gradually to the outlet. However, the initial conditions don't affect the total water volume.

The third characteristic is the peak discharge. When the soil will absorb a part of the first rainfall the peak discharge will be lower. The soil will act as a buffer. This buffer won't be available when the soil is totally saturated.

The values 1 and 0.9 are chosen, because the simulations have been done for the wet months of the Quang Tri Province.

Overland flow and interflow

These parameters simulate the actual flows in the subarea before the storm happened. Overland flow is all the water that finds its way over the surface. Interflow is the flow in the top layers of the soil. These parameters are different for every storm and every sub catchment. These parameters are related to the surface area of the subarea.

Obviously these parameters influence the total water volume passing the outlet during a storm and will have influence on the peak discharge of the flood hydrographs. However, for the simulation these initial conditions are left as zero. The assumption is that only the storms will cause an overland flow and an interflow. Changing these parameters for every storm would make the simulation more complicated and the contribution of these flows is not that big.

Base flow

Base flow is the flow caused by water out the deeper layers of the ground. Just like the overland - and interflow the base flow is dependent of the surface area of the catchment area. To find an approach for the base flow, the flood of September 1998 has been analyzed. Gia Vong discharge gauging station gives a discharge of 12m^3/s before the storm has begun. The assumption is that this discharge is caused completely by base flow. Because the top layers of the soil don't have a direct influence on the base flow, the assumption is that this base flow has a constant value. This assumption has been done to simplify the model and the calculations. Normally this base flow isn't a constant value.

The only known factor which influences the base flow is the surface of the catchment area. The assumption is the base flow has a linear relationship with the surface area of the catchment. The base flows for the other catchments are estimated with:

$$BF_x = \frac{BF_{GV}}{A_{GV}} * A_x = \frac{12}{275} * A_x \qquad (3)$$

Where:

BF_x	= Base flow in catchment x in m^3/s.	
BF_{GV}	= Base flow for Ben Hai 1 subarea in m^3/s.	
A_{GV}	= Surface of Ben Hai 1 subarea in km^2.	
A_x	= Surface of sub-catchment area x in km^2.	

4.3.3 Parameters

MIKE NAM works with several parameters divided into five groups: Surface- and root zone; Groundwater; Snow melt; Irrigation and Initial Conditions. The last group has been clarified in paragraph 4.3.2. Because there is no intensive irrigation during the raining season in Quang Tri, no irrigation parameters have been used in this study. Also the snow melt parameters have been excluded, because the temperature in this province is almost never below 5°C. The meaning of the surface- and root zone parameters and the groundwater parameters is explained in table 4.

Table 4: NAM parameter explanation and ranges.

NAM Parameter	NAM Parameter Description	Unit	Parameter boundaries
U_{max}	Maximum water content in surface storage	mm	10 – 20
L_{max}	Maximum water content in root zone storage	mm	50 – 300
CQOF	Overland flow runoff coefficient	-	0 – 1
CKIF	Time constant for routing interflow	hours	500 – 1000
$CK_{1,2}$	Time constant for routing overland flow	hours	3 – 48
TOF	Root zone threshold value for overland flow	-	0 – 0.7
TIF	Root zone threshold value for interflow	-	0 – 1
TG	Root zone threshold value for groundwater recharge	-	0 – 0.7
CKBF	Time constant for routing base flow	hours	-

4.4 Model calibration & optimization

Rainfall data is available for seven gauging stations, but discharge data is only available for one gauging station: Gia Vong. This station is the outlet of one of the Ben Hai subarea and only the water fallen in this subarea will pass the Gia Vong discharge gauging station (see paragraph 4.2), so there is a direct link between the rainfall and the discharge. The model calibration has been done for this particular area. The parameters of this area will be applied on the other 18 subareas (personal communication with N.T. Giang, 2009). Each subarea has its own characteristics and also its own parameters, so there will be a difference between the output of the calibration and the real values of the parameters of the other subareas.

Two storms have been used for the calibration of the MIKE NAM model:

- 27th of September 1998 till 30th of September 1998 (84 hours) including a two-day storm.
- 2nd of November 1999 till 7th of November 1999 (156 hours) including a five-day storm.

These two storms are very different from each other. Besides both storms have different durations, the 1998-storm has only one peak in the gauged discharge at the Gia Vong Gauging station, while the 1999-storm has more (smaller) peaks.

In the calibration procedure, several model parameters have to be adjusted using trial and error to obtain optimum values. These optimum values are considered as the representative coefficient to determine the runoff within the catchment area (Shamsudin & Hashim, 2002).

4.4.1 Procedure

The MIKE NAM model has an 'auto-calibration' function. This function uses a goal seek method to approach the optimal parameters. The user defines the parameters which have to be calibrated and the model searches for the best set of parameters. The numerical performance measures (as described in the MIKE11 user guide, 2007) which can be used are:

1. Agreement between the simulated and observed average catchment runoff: overall volume error.
2. Overall agreement of the shape of the hydrograph: overall root mean square error (RMSE).
3. Agreement of peak flows: average RMSE of peak flow events.
4. Agreement of low flows: average RMSE of low flow events.

A combination of the first three is chosen here, because it is not important for this research if the simulated low flows don't match with the observed low flows.
MIKE NAM gives a value of ' R^2 ' (with $0 < R^2 < 1$) for every simulated flood with a corresponding observed flood. When a simulated flood matches the observed flood R^2 will approach 1. The goal of the calibration and optimization is to find a set of parameters, which could be applied on both the 1998- and 1999-storm and so will be an approach of the real parameters of the catchment area. This means the set of parameters should give a high value of R^2 for both storms and low errors as described in this paragraph.

4.4.2 Trial and error

The optimal way to calibrate the parameters is to combine the auto-calibration function with a manual calibration. The nine parameters given in table 4 have been calibrated. First both storms have been simulated with parameters used by Shamsudin & Hashim (2002). They did a calibration for a completely different area, with different area characteristics, so logically the parameters didn't fit at all.
The second step was to do an auto-calibration for all the nine parameters for both storms. As expected the R^2 of these simulations were close to 1.

Table 5: Different sets of parameter for MIKE-NAM.

NAM Parameter	Set Shamsudin & Hashim	Best parameters 1998	Best parameters 1999	Used parameters
U_{max}	24	10	24	20
L_{max}	80	116	100	140*
CQOF	0.62	0.703	0.892	0.8
CKIF	1000	849.6	200	500
$CK_{1,2}$	20	10	21.7	17.6
TOF	0.1	0.99	0.71	0.1
TIF	0.1	0.135	0.263	0.4*
TG	0.1	0.789	0.672	0.7
CKBF	1000	1000	1000	1000
R^2 1998	0.753	0.896	0.644	0.761
R^2 1999	0.791	0.843	0.979	0.925

The next step was to find an optimum for each parameter. The assumption is the values of the real parameters are somewhere between the 'perfect parameters' of 1998 and 1999. With a trial and error method an approach of the best parameters has been made. These parameters are presented in table 5. Because most of the parameters are dependent of each other, it is possible that the used parameters are not in the range of the best parameters of 1998 and 1999. For example the L_{max} and TIF (marked with *).

4.4.3 Effects of the parameters

Shamsudin and Hashim (2002) described the effects of some of these parameters on the total runoff volume and on the peak of the runoff. Their conclusions are shown in table 6. Some of the parameters have a huge effect on the simulated discharge, while the effect of some of the parameters is insignificant.

Table 6: Observed effects of NAM parameters by Shamsudin and Hashim (2002)

Parameters	Change	Effects
L_{max}	Increase	Peak runoff decreased Runoff volume reduced
U_{max}	Increase	Peak runoff decreased Runoff volume reduced
CQOF	Increase	Peak runoff decreased Runoff volume increased
TOF	Increase	Peak runoff decreased Runoff volume reduced
CK1 & CK2	Increase	Peak runoff decreased The triangular shape expand horizontally
CKBF	Increase	Base flow decreased
Maximum groundwater depth causing base flow	Increase	Peak runoff decreased Runoff volume reduced

Part 2: Model application

4.5 Introduction

The parameters estimated in paragraph 4.4 have been used again in the next paragraphs.
In the next paragraphs the methodology used in Hydrologic Report (2003) has been applied. The rainfall-runoff model has been used to give an approach of the floods matching the annual exceedence probabilities of rainfall for each subarea. The values estimated in this chapter are not the design floods, but are the discharges resulting from design rainfalls. This is not the same.

4.6 Methodology

4.6.1 Relation between storm and flood return periods

In the chapter ‘Rainfall Frequency Analysis’ the annual exceedence probabilities (AEP) for each rainfall station have been estimated. These AEP’s will be the input for this chapter and its calculations. Although, Viglione and Blöschl (2009) conclude that there is no straightforward relation between storm and flood return periods. Hypothetically if there is only one single storm duration, the flood return period T_Q is always equal to the rainfall return period T_P. The flood return period depends on two variables: T_P and the storm duration. If the storm duration is a constant value, the relationship between T_Q and T_P is linear, so those values should be equal. In more realistic cases where storm durations vary, T_Q will always be smaller than T_P.
To get a good view of the relationship between flood return period T_Q and rainfall return period T_P for the catchment areas in the Quang Tri Province, a study should be done for a large amount of storm and

flood data. The critical storm duration exists where, as output of this study, T_Q/T_P is at maximum. Viglione and Blöschl (2009) have found a maximum value of T_Q/T_P of about 0.4. This means the design rainfall method will give a 40 year flood when using a 100 year storm as an input to the runoff model.

4.6.2 Weighted rainfall

The rainfall in several subareas depends on the rainfall measured in more than one gauging station. A combination of a 100-year rainfall in gauging station X and a 100-year rainfall in gauging station Y won't usually result in a 100-year rainfall in a specific subarea. But the average of the rainfall gauging stations is the most suitable option with the limited information. Just like in paragraph 4.3 the Thiessen's Weights Method will be used to get an average rainfall for each subarea.

4.6.3 Critical storm duration

According the Quang Ngai study (Hydrologic Report, 2003) the first step is to find the 'critical storm duration' for the 'BH1' subarea. For Gia Vong discharge gauging station a flood frequency analysis has been done (see chapter 3). All the water passing the gauging station has fallen in subarea 'BH1'.
The rainfall values matching the storm durations of 3, 6, 12, 24, 48 and 72 hours, estimated in chapter 3, will be used as input for MIKE NAM. The intensity of the storm is assumed to be constant, but in the real world every storm has a peak in the hyetograph. Next the discharge peaks from the model will be compared with the peaks from flood frequency analysis. The peak of the storm duration which is the best approach of the peak of the flood frequency analysis will be used in the design floods estimation.

The second step is to get the critical storm duration for the other 18 subareas. The critical storm duration depends on the response time of the subarea. Several factors influence the response time of a subarea, for example:

- Surface of the subarea
- Average slope of the subarea
- Land use in the subarea
- Water reservoirs in the subarea

Only the factor 'surface' is known. This factor will give, of course in combination with the precipitation values, the amount of water fallen in the catchment area. The other factors only will influence the duration of the rainfall discharge. In this research a simple approach has been chosen: only the surface of the subarea will influence the critical storm duration.

The third step is to run the model with the right critical storm duration for each subarea. This has to be done for an AEP of 50%, 20%, 10%, 5%, 2% and 1%. The output is the design floods for each subarea and for each annual exceedence probability. According to Viglione and Blöschl (2009) the found annual exceedence probabilities of storms are not the same as annual exceedence probabilities of floods.

Part 3: Three-day rainfall method

4.7 Introduction

For the seven sub-catchment areas with a surface area larger than 100km^2 (see table 1) another method has been applied. In Vietnam it is common to work with three-day rainfall for runoff estimations (personal communication with N.T. Giang, 2009) In table 7 the maximum three-day rainfall for the period 1976-2000 is given.

Table 7: Maximum three-day rainfall [mm]

	Con Co	Cua Viet	Thach Han	Dong Ha	Gia Vong	Khe Sanh	A Luoi
1976	-	-	-	466.5	-	-	143.1
1977	-	-	-	-	395.3	227.7	298.5
1978	480.3	489.1	540.6	532.6	623.4	415.8	305.1
1979	1301.3	444.1	314.1	386.1	613.1	570.2	269.3
1980	317.3	548.4	637.4	620.0	654.3	471.1	629.0
1981	450.2	475.1	520.7	410.0	323.6	259.8	470.8
1982	235.7	473.2	495.4	418.8	562.9	266.9	276.2
1983	553.6	-	481.1	681.2	547.9	334.1	744.7
1984	567.4	345.9	-	326.0	309.7	277.3	501.9
1985	270.4	-	578.6	543.7	544.5	216.7	358.8
1986	380.2	251.7	306.3	196.5	384.2	211.8	409.8
1987	462.0	321.5	233.2	245.1	256.8	310.7	397.1
1988	183.3	156.3	214.2	215.1	231.3	203.9	340.6
1989	447.4	294.8	255.2	252.0	259.4	266.4	338.0
1990	247.1	371.8	389.4	376.1	427.6	495.6	1069.0
1991	335.2	487.7	479.7	503.0	544.8	364.4	397.9
1992	442.9	430.5	407.2	391.7	583.2	302.0	415.0
1993	315.4	272.3	205.1	207.8	253.9	81.2	300.2
1994	274.4	250.9	252.5	206.6	429.2	130.1	329.8
1995	275.2	293.2	327.8	445.5	571.3	558.1	979.0
1996	266.7	268.0	336.2	249.5	250.2	366.4	881.2
1997	166.0	531.6	421.8	289.2	276.1	285.4	358.4
1998	285.2	364.9	446.8	355.6	481.6	240.8	1056.6
1999	180.2	926.3	1141.8	714.3	627.3	400.8	1490.8
2000	191.4	191.4	204.6	211.5	156.1	189.7	463.6

4.8 Methodology

First step is to find the ‘typical three-day rainfall’. In chapter 3 the three-day design rainfalls are given. Again the AEP of 1% will be used. A typical three-day rainfall is a rainfall from table 8 that has about the same amount as the design rainfall from table 13.

Because the rainfall has to be estimated for subareas, the Thiessen’s Method (see paragraph 5.4.1) will be used again. For example subarea O Lau 1 is divided in 77.7% A Luoi gauging station and 22.3% Thach Han gauging station.

Because the values from table 8 and table 13 aren’t exactly the same, an increase factor has to be estimated. The increase factor is called K:

$$K = \frac{X_P}{X_T} \quad (8)$$

Where: X_P = the rainfall amount [mm] corresponding to the design frequency (table 8)

X_T = the total rainfall amount [mm] of the typical rainfall event (table 13)

K is different for every rainfall station and every typical three-day rainfall event.

Using K, the values of the hyetographs of each event will be transformed in values corresponding to the design three-day rainfalls (see table 9).

Example
For gauging station Gia Vong the three-day design rainfall with an AEP of 1/100 is equal to 873m^3. In 1980 a three-day storm of 654.3 mm occurred. This gives a K of about 1.33. The daily rainfalls of the storm are known. This daily rainfalls will be multiplied by K. The result is a hytograph of a typical storm.

Table 8: Typical 100-year rainfall for the subareas with a surface > 100km2.

Subarea	BH1	CL1	OL1	OL2	SL2	TH1	TH5
Used year	1980	1979	1999	1999	1980	1995	1999
Rainfall Day 1 [mm]	297.9	239.6	869.5	587.4	297.9	129.3	456.8
Rainfall Day 2 [mm]	420.8	360.2	451.4	412.9	420.8	510.9	314.2
Rainfall Day 3 [mm]	154.2	135.2	424.1	345.4	154.2	422.4	205.9
Total runoff volume [10^6 m^3]	190	195	345	160	160	900	240

It was hard to find a suitable three-day rainfall for TH1, because of the big distance between the gauging stations A Luoi and Khe Sanh. This means it is possible (or even likely) that the peak of the storm won't occur at the same time at both gauging stations. Some storms measured at A Luoi even missed Khe Sanh totally.

The found values of the hyetographs (see table 8) have been used as input for MIKE-NAM.

5 Rational method of estimation design floods from rainfall

5.1 Introduction

The values found in chapter 4 are not the design floods of the catchment areas, but are an approach of the design rainfalls. To get an approach for the real design floods, another method should be applied on the available data. Bell & Songthara Om Kar (1969) give two methods to find an approach for the peak discharge in a flood hydrograph: the rational method and the rational loss-rate method. The rational method looks like:

$$Q_x = C_x I_x A \quad (4)$$

Where: Q_x = the flood peak with given return period x.
I_x = mean intensity of catchment rainfall with return period x and duration equal to the time of concentration (see paragraph 7.2.3).
C_x = a coefficient depending on x, catchment characteristics and the units of the other variables.
A = Area of catchment.

This formula has a few limitations:
(a) It treats runoff as a percentage of the flood-producing rainfall whereas physical considerations suggest that runoff is more appropriately treated as a residual of the rainfall after deductions are made for losses.
(b) The coefficient C_x includes the effects of two unrelated factors (storage and loss), which should be expressed separately.

5.2 Characteristic times

Equation 4 and 5 are based on the *'time of concentration'*. This is a characteristic time of a catchment area. Characteristic times are essentially measures of the speed of response of stream flow to flood-producing rainfall which depends on catchment particulars such as area, stream slopes and stream lengths (Bell & Songthara Om Kar, 1969). Estimates of characteristic times are required for two fundamental purposes:
(a) To provide a means of estimating the critical rainfall duration which is assumed to have some relationship to the catchment response.
(b) To enable the synthesis of a design hydrograph, the dimensions of which depend on the catchment response.
Characteristic times are *rise time; critical lag; time of concentration* and *Critical rainfall (storm) duration.*

Rise time

This is time of the peak discharge, of a single peak flood, after start of rise.

Critical lag

There are several definitions of the critical lag:
(a) The time between the centroid of the hyetograph of excess rainfall and the flood peak.
(b) The time between the centroid of the hyetograph of excess rainfall and the centroid of the flood hydrograph.
The formula for the critical lag in small catchment areas as in the Quang Tri Province is (Bell & Songthara Om Kar, 1969):

$$T_k = MA_m^{0.33} \tag{5}$$

Where: A_m = area in square miles

T_k = critical lag in hours

M = constant varying from 1.0 to 3.0, depending mainly on the channel storage characteristics. Especially vegetation appears to have an influence on M (Bell & Songthara Om Kar, 1969).

Time of concentration

Time of concentration is usually defined as the time of travel of water from the most remote point on the catchment to the outlet or the time from commencement of rainfall excess until the whole area is contributing to flow at the outlet (Bell & Songthara Om Kar, 1969). The formulas of the time of concentration are based on average slope of the catchment area and the distance from the outlet to the most remote part of the catchment.

Critical rainfall (storm) duration

This is the duration of a *total* rainfall, but it can be measured as the duration of the excess rainfall contributing directly to the flood hydrograph. The theoretical analyses of Machmeier and Larson (as cited in Bell & Songthara Om Kar) show that the critical duration is approximately equal to the critical lag.

The critical lag is the most constant characteristic of these values. Bell and Songthara Om Kar (1969) express the characteristic times in percentage of critical lag. The time of concentration is usually 70 (summer) – 100 % (winter) of the critical lag.

5.3 Methodology

Equation 4 will be used to find an approach for the design floods in all the sub catchment areas.

5.3.1 Calculation of critical lag

First an approach of the critical lag for each subarea has to be found. Because all the available area data is in km^2 a small change has been made in the formula

$$T_k = M\left(0.386102 * A_k\right)^{0.33} \quad (6)$$

Where: A_k = area in square kilometers

These calculations has been done for M = 1, M = 2 and M = 3.

Table 9: Critical lag for 19 subareas with M=1, M=2 and M=3

Subarea	A (km^2)	T_k (hours)	T_k (hours)	T_k (hours)
		M = 1	M = 2	M = 3
BH1	275.2	4.7	9.3	14.0
BH2	72.8	3.0	6.0	9.0
BH3	15.6	1.8	3.6	5.4
CL1	334.2	5.0	9.9	14.9
CL2	65.4	2.9	5.8	8.7
CL3	73.1	3.0	6.0	9.0
OL1	238.4	4.4	8.9	13.3
OL2	161.9	3.9	7.8	11.7
OL3	93.6	3.3	6.5	9.8
OL4	44.8	2.6	5.1	7.7
OL5	75.5	3.0	6.1	9.1
SL1	80.5	3.1	6.2	9.3
SL2	225.1	4.4	8.7	13.1
SL3	26.3	2.1	4.3	6.4
TH1	1067.0	7.3	14.6	21.9
TH2	192.2	4.1	8.3	12.4
TH3	95.2	3.3	6.6	9.9
TH4	43.8	2.5	5.1	7.6
TH5	312.0	4.9	9.7	14.6

5.3.2 Selection of critical rainfall duration

The second step is to find the rainfall duration for every critical lag. This is usually the rainfall with the maximum duration but shorter than the critical lag. For example: If the critical lag has a value of 14.6 hours, the chosen rainfall duration is 12 hours.

5.3.3 Selection of rainfall value

In chapter 3 values for the AEP's of rainfall have been estimated. These values will be used as input in this chapter. When for example 12-hour rainfall duration is leading for subarea SL2, the corresponding value of the precipitation of subarea SL2 will be used in the further estimations.

5.3.4 Estimation of design floods

The AEP's have been used to calculate the mean intensity of the rainfall. This is just the amount of water fallen in a specific time period (m/s or mm/s).
For catchment BH1 Q_x is known (see chapter x). Using this knowledge, values for C_x for every return period could be estimated. C_x is based on the catchment characteristics and depends on the return period of the flood. Assumed is that C_x is independent of A. Moreover, the assumption is that other catchment characteristics of the other 18 sub catchments are equal to the characteristics of BH1. This means every C_x could be estimated for every return period and every M.

5.4 Flood hydrographs

With the design floods estimated in paragraph 5.3.4 a visualization of the runoff can be made. Assumed is that M is equal to 2 (Bell & Songthara Om Kar, 1969) and so the hydrographs are based on the peak discharges presented in table 18. For each subarea the own critical storm duration has been used. Because of the limited information the intensity of the storm is to be assumed as a constant value. The hydrograph of the 100-year discharge of a catchment area is a standard used characteristic. The hydrographs for this annual exceedence probability will be given.
Because different critical storm durations have been used for design flood estimation, the return period of the rainfall of the different sub-catchment also will be different for every sub-catchment. For example, if a 12-hour rainfall had been used instead of a 6-hour rainfall, the peak of the flood hydrograph would be very different.
In general it can be assumed, that a shorter critical storm duration will match a longer critical period of return of the rainfall. With a little trial-and-error the correct flood hydrographs can be found.

6 Results and discussion

6.1 Flood frequency analysis result

For sub-catchment area BH1 flood frequency analysis is done. The values of the annual exceedence probability are given in table 10. To show the influence of omitting a low value both the values of zero and one omitted flows are given. The values in the table represent the annual exceedence probabilities, so the change that a discharge of 2700m^3/s or more occurs in any one year is 1%.

Table 10: Design floods for subarea BH1 [m^3/s]

Discharge station		Annual Exceedence Probability - %					
	years	50	20	10	5	2	1
Gia Vong	28	772	1289	1642	1980	2410	2727
Gia Vong (1 Omit)	27	781	1282	1625	1955	2382	2700

In figure 10 the annual exceedence probabilities have been visualized. The dark blue line represents the Log-Pearson III distribution of the used dataset and the light blue lines are the confidence limits. The scale is logarithmic.

In table 10 the difference between the design floods with zero and one low flood omitted are shown. The difference is very limited, so omitting one flow isn't really necessary for this catchment area.

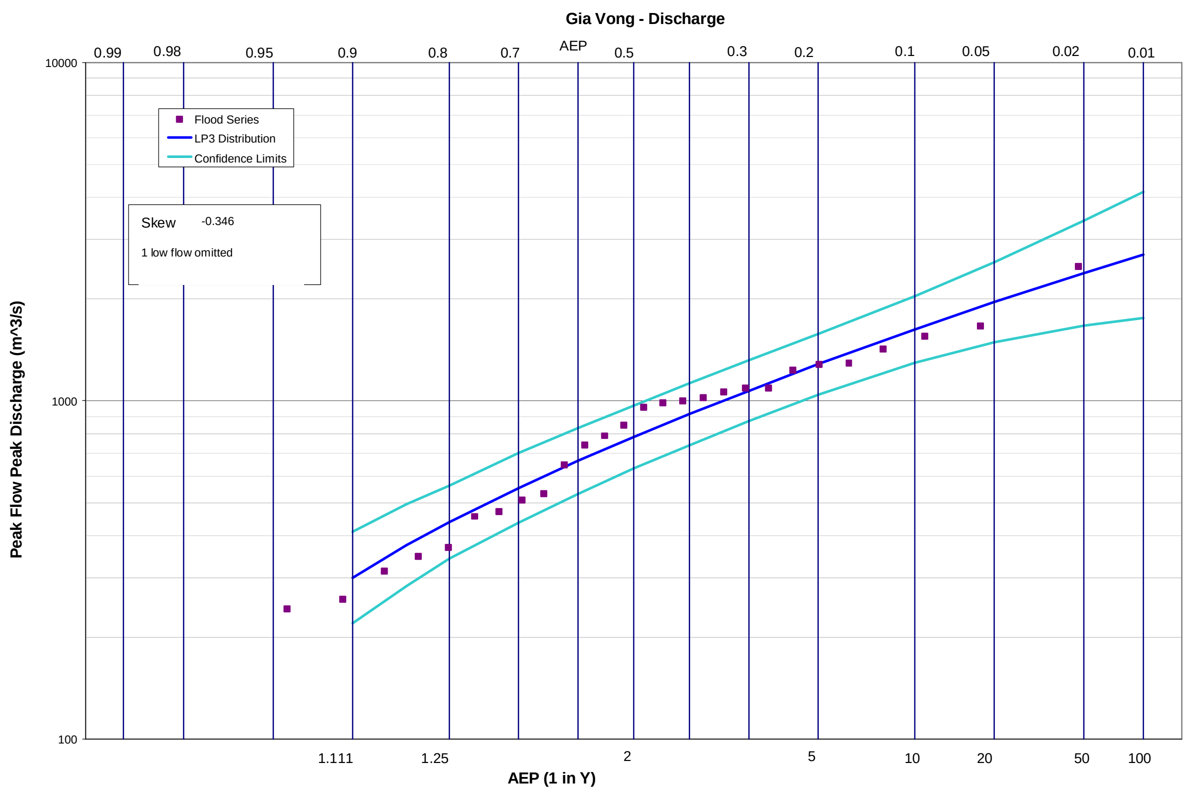


Figure 10: Gia Vong peak discharge annual exceedence probabilities.

6.2 Rainfall frequency analysis

In chapter 3 the methodology of FFA has been applied on rainfall data. The tables of one-day, two-day and three-day rainfall annual exceedence probabilities are shown in table 11, 12 and 13. The graphs with the Annual Exceedence Probabilities of one-day rainfall are placed in appendix A-2.

Table 11: Design rainfalls [mm] for one-day rainfall for seven gauging stations.

Station		Annual Exceedence Probability - %				1day	rainfall
	years	50	20	10	5	2	1
A Luoi	25	269	393	483	575	704	808
Con Co	23	219	351	458	578	760	918
Cua Viet	21	225	300	346	386	435	470
Dang Ha	24	230	327	390	452	531	591
Gia Vong	24	262	335	378	417	463	496
Khe Sanh	24	178	246	288	326	374	407
Thach Han	22	241	341	407	470	552	613

Table 12: Design rainfalls [mm] for two-day rainfall for seven gauging stations.

Station		Annual Exceedence Probability - %				2day	rainfall
	years	50	20	10	5	2	1
A Luoi	25	390	593	741	894	1106	1278
Con Co	23	269	421	559	724	998	1258
Cua Viet	21	302	432	520	606	718	803
Dang Ha	24	321	440	516	589	681	749
Gia Vong	24	363	473	535	588	650	693
Khe Sanh	24	248	341	401	460	535	593
Thach Han	22	322	468	569	669	802	905

Table 13: Design rainfalls [mm] for three-day rainfall for seven gauging stations.

Station		Annual Exceedence Probability - %				3day	rainfall
	years	50	20	10	5	2	1
A Luoi	25	437	706	931	1185	1577	1925
Con Co	23	309	476	623	798	1082	1348
Cua Viet	21	359	506	606	706	838	941
Dang Ha	24	356	501	599	696	824	922
Gia Vong	24	417	561	646	721	810	873
Khe Sanh	24	295	406	477	544	629	693
Thach Han	22	367	539	671	814	1023	1199

6.2.1 Differences between gauging stations

Watching the AEP of 1% in table 12 and 13 a remarkable difference appears. The gap between both values of A Luoi is about 650mm, while the gap between the values of Con Co is only 90mm. There is a simple explanation for these differences. The graphs in appendix A-1 show the maximum rainfall values. Clearly for A Luoi station a third rainfall day had a significant influence on the total rainfall volume, while for Con Co the influence of a third rainfall day was limited. Because the used data is limited not every rainfall station has experienced a three-day storm.

6.2.2 Short duration rainfall

The design rainfall of three, six and twelve hours is based on the design rainfall of one day. This is the limitation of this method. The weight factors have been adopted from another study (Hydrologic report, 2003) and the assumption is, these factors are accurate. The tables of the short duration rainfall are presented in appendix A-3.

6.3 Rainfall runoff modeling part 1: model calibration result

Using the floods of 1998 and 1999 the MIKE NAM model has been calibrated for the BH1 subarea. In table 14 the estimated parameters are shown. Every run of the model has R^2 as an output. This value represents the accuracy of the used parameters for the specified flood. These values are shown in table 15.

Table 14: Best NAM parameters after calibration.

NAM Parameter	Parameter Value
U_{max} [mm]	20
L_{max} [mm]	140
CQOF [-]	0.8
CKIF [hours]	500
$CK_{1,2}$ [hours]	17.6
TOF [-]	0.1
TIF [-]	0.4
TG [-]	0.7
CKBF [hours]	1000

Table 15: Accuracy of the parameters compared to the observed floods.

Flood	R^2
1998	0.761
1999	0.925

When only the overall volume error is important for the research, the optimal parameters will be different from parameters calibrated using the agreement of peak flows.
Figure 9 and 10 show the graphs of the observed discharge compared to the simulated discharge.

During the study it was easier to find high values of R^2 for the 1999-flood than for the 1998-flood. The difference can have several causes:

1. Shorter duration of 1998-storm. The total water volume of the 1998 is lower, so the differences between the observed total water volume and the simulated total water volume are relative bigger than for the 1999-storm.
2. Sharp observed peak of 1998-flood. It isn't natural for MIKE-NAM to simulate sharp peaks (see figure 11 and 12). The 1999-has more peaks, but those peaks are less sharp and also a bit lower.

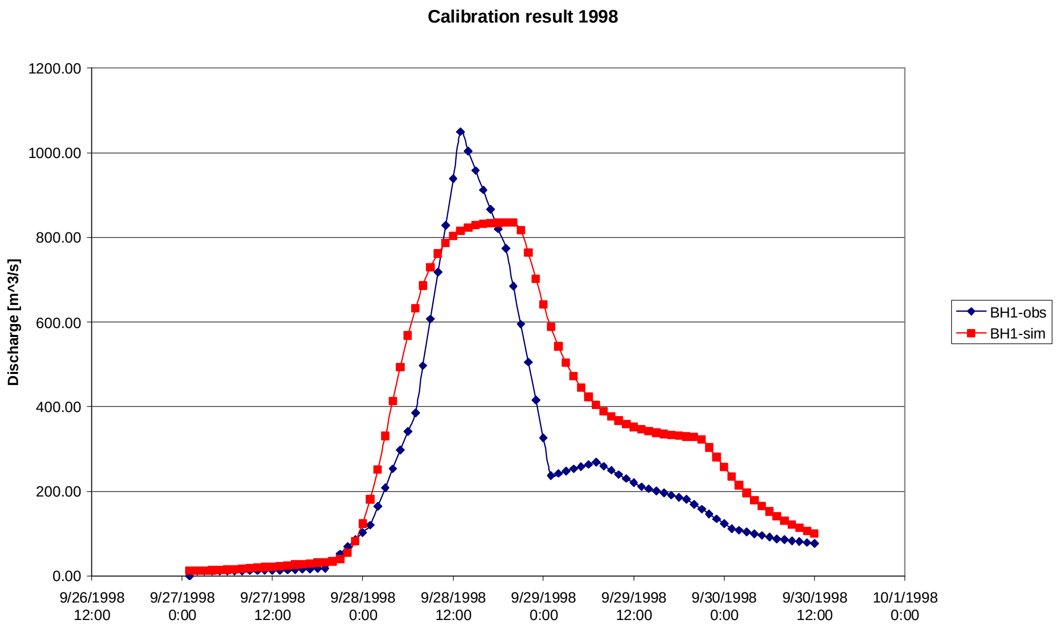


Figure 11: Simulated 1998-flood hydrograph compared to the observed 1998-flood hydrograph.

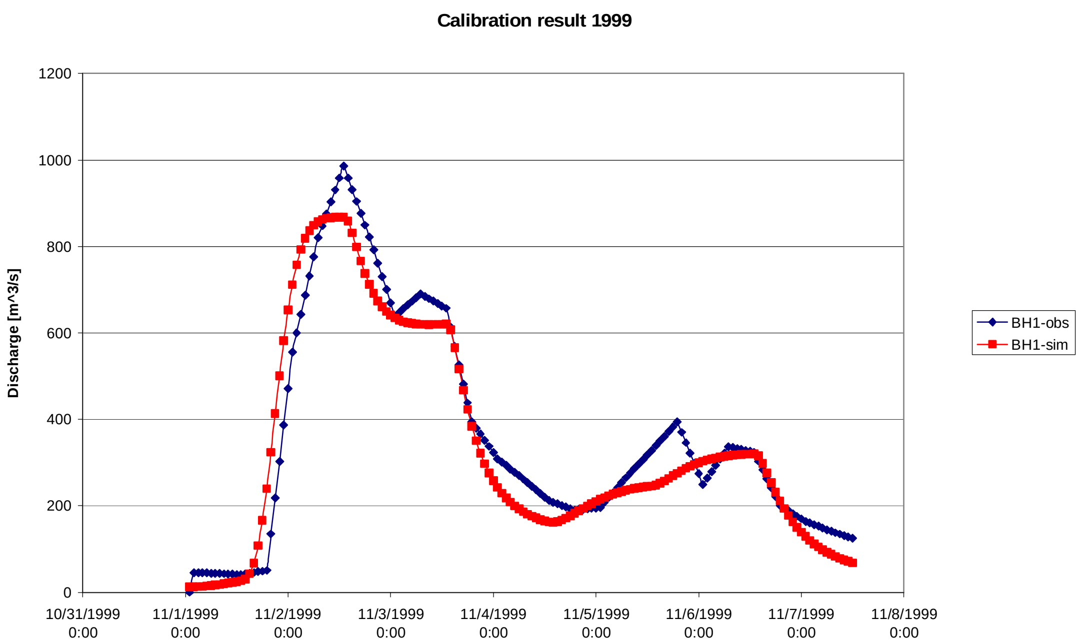


Figure 12: Simulated 1999-flood hydrograph compared to the observed 1999 flood hydrograph

6.3.1 One catchment area with discharge

Unfortunately only one catchment area with discharge data was available. The other 18 subareas have adopted the parameters of BH1 subarea. Each catchment has its own characteristics, so it will also have its own parameters.

6.3.2 Peaks

A characteristic of MIKE NAM is that the simulated peaks are almost always lower than the observed peaks. This can't be changed with the surface- and root zone and groundwater parameters. A solution for this problem is to add an interflow or an overland flow in the initial conditions. But in that case the total water volume of the simulation will be more than the observed total water volume. Probably MIKE NAM is not very suitable for simulation of peak discharges.

6.3.3 Made for long term simulation

According to Willems et al (1999) MIKE 11 has a tradition of modeling long, continuous stream flows. MIKE11 is a time-based model. However, in this research MIKE11 has been used for event-based modeling. This model feature results in lower values of R^2 for the shorter 1998-flood.

6.4 Rainfall runoff modeling part 2: model application result

The method of Hydrologic report (2003) has been applied in this study. The annual exceedence probabilities of rainfall have been translated in matching design floods. However, the return periods of these floods are smaller than the return periods of the storms.

Table 16: Matching discharges [m^3/s] of the AEP of rainfall.

Subarea	AEP (%)	50	20	10	5	2	1
BH1		666	857	969	1071	1190	1276
BH2		162	213	243	271	305	329
BH3		38	48	55	60	67	72
CL1		600	822	955	1075	1226	1330
CL2		153	200	228	254	285	308
CL3		163	223	260	296	341	373
OL1		577	843	1034	1225	1486	1693
OL2		367	526	635	740	878	982
OL3		208	297	356	411	483	537
OL4		99	141	169	195	229	255
OL5		167	238	285	330	388	430
SL1		194	250	284	315	351	378
SL2		543	698	789	872	969	1039
SL3		63	81	92	102	113	121
TH1		2008	2889	3462	4015	4748	5286
TH2		401	576	686	792	926	1027
TH3		206	295	353	409	480	533
TH4		97	138	166	192	225	250
TH5		605	862	1027	1180	1376	1516

Table 17: Discharge [m^3/s] of NAM compared with flood frequency analysis.

Subarea	AEP (%)	50	20	10	5	2	1
BH1 – MIKE-NAM		666	857	969	1071	1190	1276
BH1 – FFA		781	1282	1625	1955	2382	2700

6.4.1 Annual exceedence probabilities

The annual exceedence probabilities of the rainfall are not the same as the annual exceedence probabilities of the discharge. So according to Viglione and Blöschl (2009) the differences in output were expected. However, the study for the Quang Ngai Province (Hydrologic report, 2003) found matching values for AEP's of rainfall and AEP's of discharge. The likely reason for this is the difference in use of rainfall-runoff model. RORB, used in Hydrologic Report (2003), seems to have the feature to translate 100year rainfalls in 100year discharges. MIKE-NAM doesn't have that feature.

6.4.2 Parameters based on only one discharge

The matching floods of the AEP of the rainfalls are based on one set of parameters. This set of parameters is based on a calibration for one catchment area. Each catchment area has its own characteristics, so obviously the parameters won't be (exactly) the same as the parameters estimated for BH1 subarea.

6.5 Three-day rainfall

The advantage of the three-day is, that the shape of the hytographs is more realistic. This leads to more realistic hydrographs. The three-day rainfall method also gives the total water volume of a total storm and not just of 24 hours. This makes this method useful. However, the same problems as in paragraph 6.4 will occur when using this method.

6.5 Rational method result

Because the values of the rainfall runoff model didn't match the results of the flood frequency analysis an additional method had to be added.

With an additional method design floods for the catchment areas have been estimated. The rational method gives three tables of output: for M = 1, M = 2 and M = 3. The most likely value of M is between 1 and 2. In table 18 the values of M = 2 are shown. These values look the most plausible according to Bell & Songthara Om Kar (1969). The tables with design floods for M = 1 and M = 3 can be found in appendix C.

Table 18: Design floods [m^3/s] estimated with the rational method for M=2.

Subarea	AEP (%)	50	20	10	5	2	1
BH1		781	1282	1625	1955	2382	2700
BH2		202	338	433	525	647	739
BH3		55	91	115	139	169	192
CL1		715	1239	1613	1977	2466	2827
CL2		228	380	486	590	726	829
CL3		193	336	439	544	685	795
OL1		679	1264	1736	2240	2986	3605
OL2		433	791	1068	1355	1763	2089
OL3		244	444	595	749	966	1135
OL4		147	266	357	450	580	681
OL5		197	358	480	604	779	915
SL1		228	375	477	576	705	802
SL2		639	1049	1329	1599	1948	2208
SL3		94	154	195	234	285	323
TH1		1596	2896	3887	4903	6351	7491
TH2		457	827	1104	1387	1780	2085
TH3		251	455	611	769	991	1165
TH4		143	260	349	440	566	666
TH5		721	1298	1729	2162	2763	3224

6.5.1 C_x is uncertain

In the rational method the value for C_x plays an important role in the design flood estimation. However, the value of C_x is very uncertain. The value of C_x is a loss factor and there is not so much attention for the real meaning of C_x. During the research C_x has been used as a correction factor and not as a reality based weight factor.

6.5.2 Based on one discharge value

The calculations made in this chapter are mostly based on the output of chapter 3. The information is limited. When more discharge data (for other subareas) would be available, more accurate estimations of design floods can be made.

Flood hydrographs

Flood hydrographs have been drawn for the 100-year floods (floods with an annual exceedence probability of 1%). In figure 13 the flood hydrograph for Ben Hai 1 sub-catchment area is shown. The flood hydrographs for all the nineteen sub-catchment areas have been placed in appendix D. In table 19 the total rainfall matching the flood hydrographs has been placed.

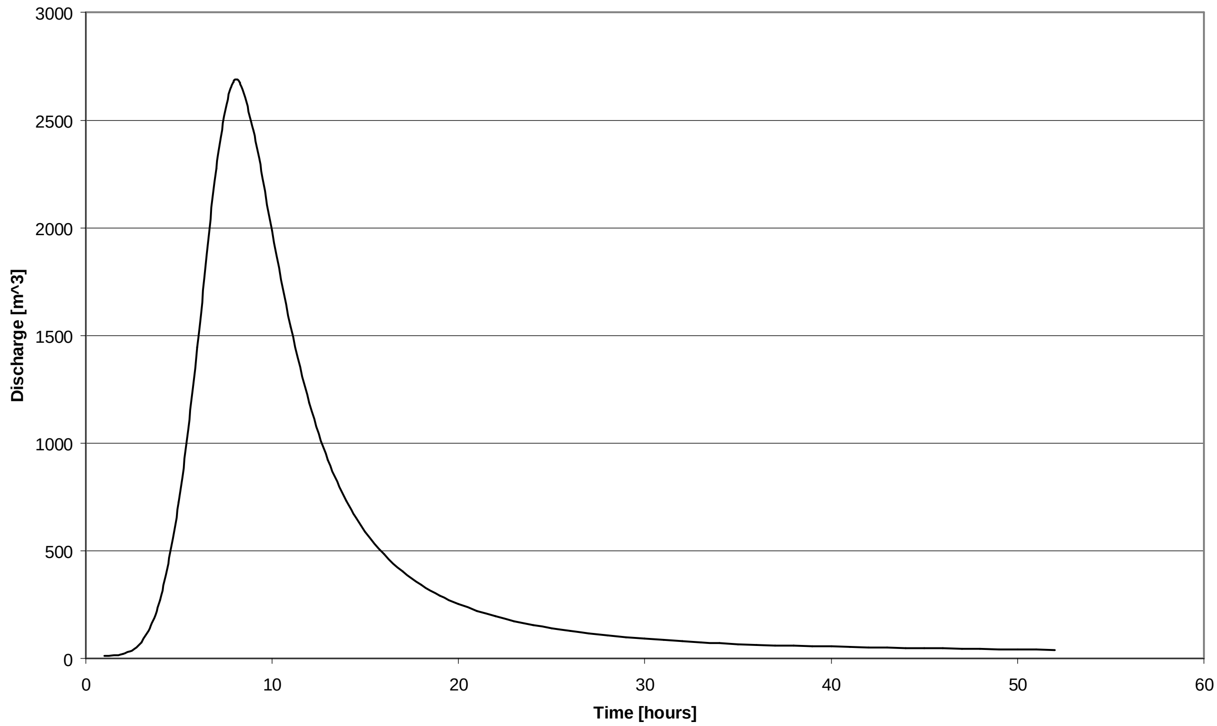


Figure 13: Ben Hai 1 AEP 1:100 total runoff hydrograph based on a six-hour rainfall

Table 19: Total hypothetical rainfall volume as input for the flood hydrographs.

Subarea	Total rainfall volume [10^6 m^3]	Subarea	Total rainfall volume [10^6 m^3]
BH1	99	SL1	29
BH2	27	SL2	81
BH3	6	SL3	10
CL1	103	TH1	426
CL2	27	TH2	76
CL3	29	TH3	43
OL1	132	TH4	21
OL2	76	TH5	118
OL3	42		
OL4	22		
OL5	33		

6.5.3 Hyetographs

Because rainfall data is only available in daily values, the shapes of the hyetographs are unknown. If the total rainfall volume is important, the shape doesn't matter and only the duration and intensity of the rainfalls can be used. But usually a peak in the rainfall affects a peak in the discharge, so a global shape of the hyetographs is required. Therefore the use of hourly rainfall data during storms is necessary.

7 Conclusion and recommendations

In chapter 1 the research goal has been formulated:

The research goal of this study is to estimate the design floods for all the sub-catchment areas in the Quang Tri Province using Flood Frequency Analysis and the rainfall-runoff model MIKE-NAM.

Using 'FFA Spreadsheet 2.0' for flood frequency analysis and MIKE-NAM as a deterministic tool, a big part of the assignment could be done. However, during the process this method appeared to be limited. The main reason for this limitation is that the annual exceedence probabilities of rainfall are not the same as the annual exceedence probabilities of matching discharges. Also with the use of different storm durations the discharges as output of the model were much lower than expected.

The most important conclusion of the research is that the limited available data is a limitation for the accuracy of the research. Hourly rainfall data during storms will improve the accuracy of the results. Discharge data for more than one gauging station in the subarea also will improve the accuracy of the research.

Another important conclusion is that MIKE-NAM isn't suitable for design flood estimations. In de next paragraphs this conclusions will be explained and some recommendations will be given.

7.1 Design floods

According to the theory of Viglione and Blöschl (2009) the results were expected. The period of return of the floods is much shorter than the period of return of the matching rainfalls. As in chapter six was mentioned: to get a good view of the relationship between flood return period T_Q and rainfall return period T_P for the catchment areas in the Quang Tri Province, a study should be done for a large amount of storm and flood data.

In this report a rational method has been used to fix the differences. However, it is assumed there are more complex techniques to find more accurate results for design floods using rainfall data. Probably other, more recent rainfall-runoff models are already using these techniques.
The Hydrologic Report (2003) found accurate values for the discharge. That study used another rainfall-runoff model. Differences between the models can cause differences in the results.

- Further research with complex techniques for design flood estimation.
- Observe the opportunities of other rainfall runoff models and pick the best model for research.

7.2 MIKE-NAM

In this study MIKE-NAM was used for short duration simulations. Usually MIKE-NAM has been used for long term simulations. A characteristic of MIKE-NAM is that the model is better over long duration simulations.

The parameters of the model can fluctuate a lot, meaning there is a wide range of possible values of the real parameters. It is recommended to study the influence of each parameter. The boundaries given by the MIKE11 User Guide (2007) also need to be controlled critically. Because the model will be applied in very different areas, the parameters for each area will be very different.

- Sensitivity analysis for parameters in MIKE-NAM.
- Match the boundaries of the MIKE-NAM parameters with catchment area characteristics.
- A calibration for another subarea (obviously with the discharge data) is recommended.

7.3 Data

Most of the output of this research is based on the measured discharge of the Gia Vong gauging station. However, the other sub-catchment areas have specific characteristics. For more accurate output, discharge data for more locations is necessary. It is also recommended to use hourly discharge data over a long period.

Each storm graph has its own characteristic shape. Sometimes, the duration of a storm isn't even 24 hours. Because only daily rainfall data is available, the shape of the storm graph is unknown. The hydrographs of the discharge are based on daily rainfall data, but some of the characteristic times of the hydrograph are less than 6 hours. Therefore, hourly rainfall data during heavy storms is required to get accurate flood hydrographs.

- Collect more discharge data over more locations and more frequently.
- During storms measure hourly rainfall data instead of daily rainfall data.

Of course it isn't possible to collect data from the past, so using more advanced techniques will be a first option to find more accurate output.

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Appendices

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A. Rainfall frequency analysis

A-1 Annual maximum two-daily and three-daily rainfall

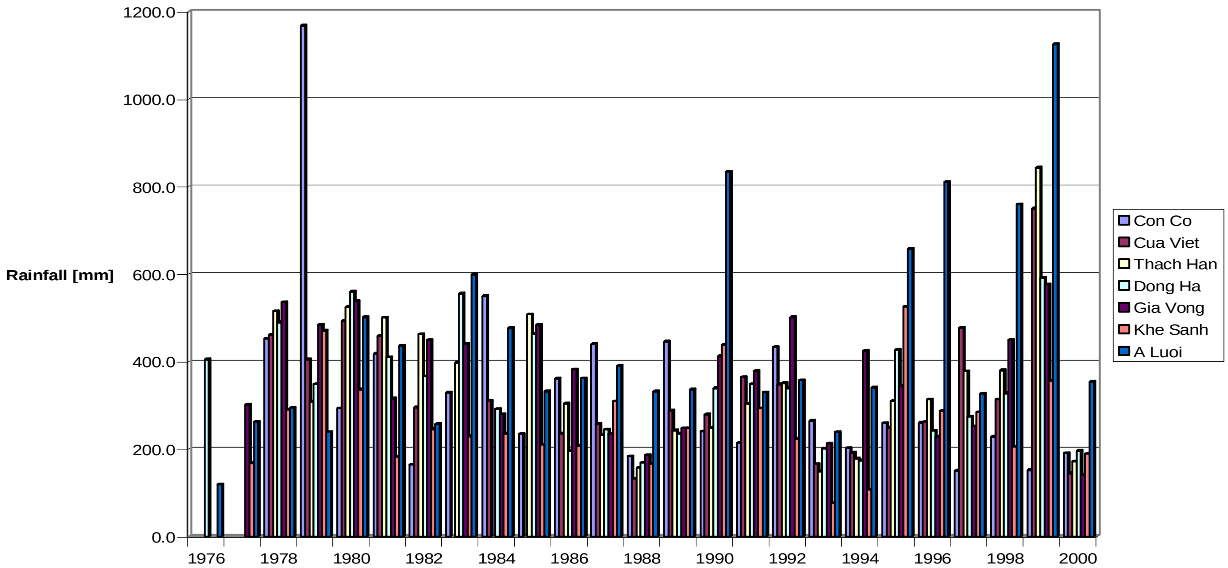

Figure 14: Annual maximum two-daily rainfall for seven rainfall stations

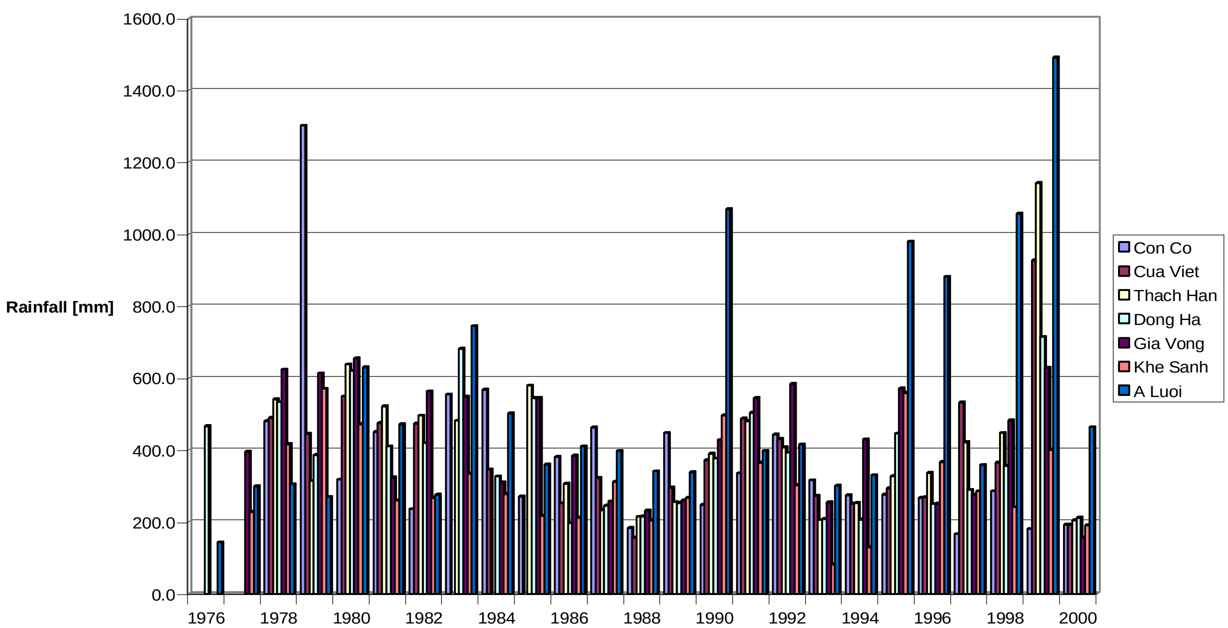

Figure 15: Annual maximum three-daily rainfall for seven rainfall stations

A-2 Gauging stations rainfall frequency analysis

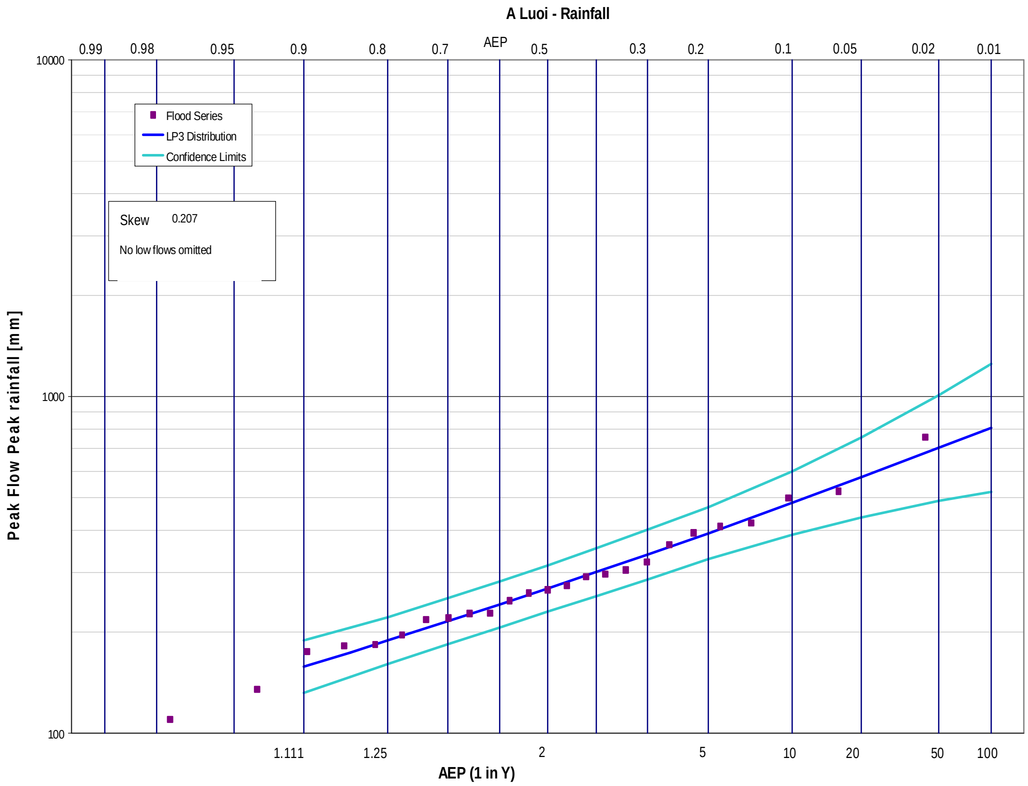

Figure 16: A Luoi daily rainfall frequency analysis

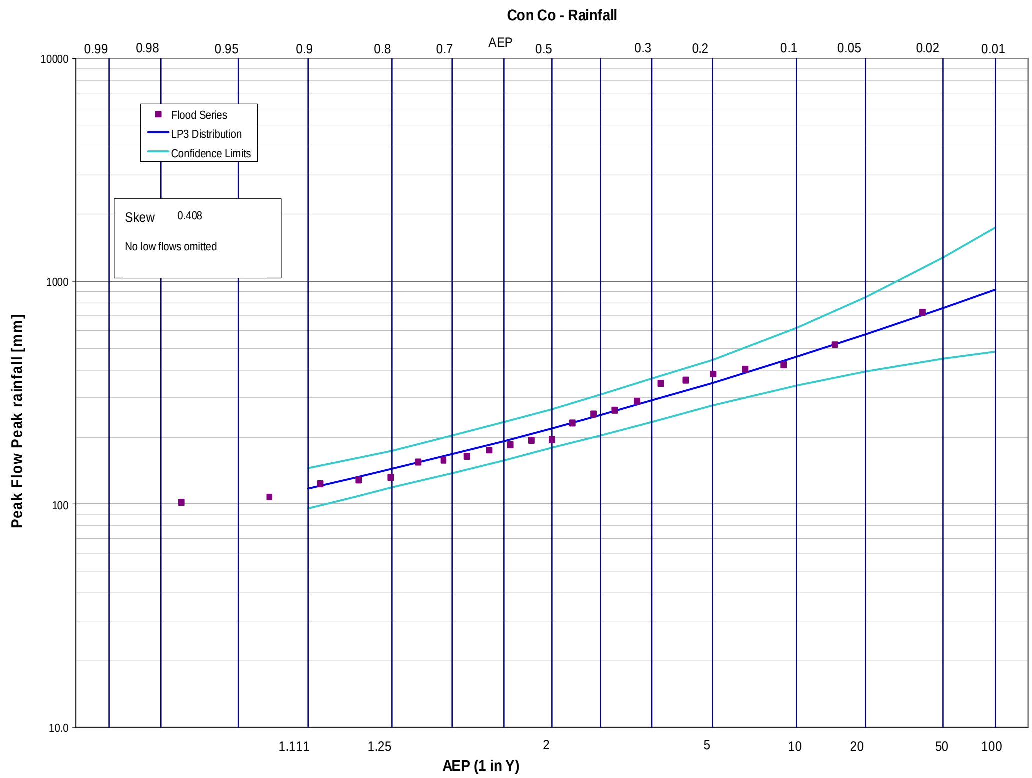

Figure 17: Con Co daily rainfall frequency analysis

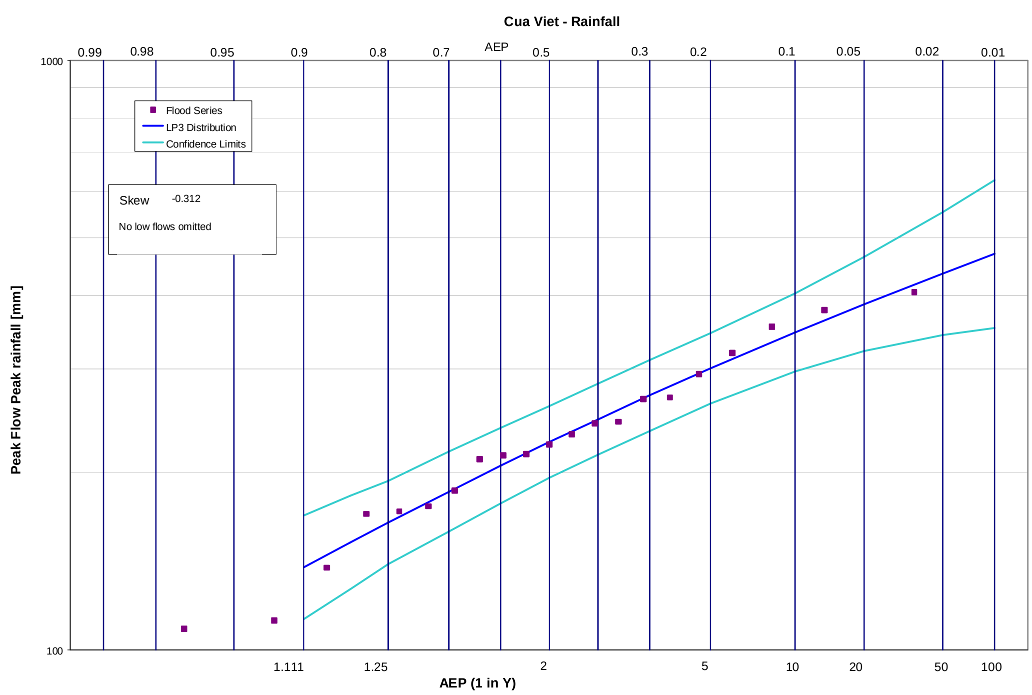

Figure 18: Cua Viet rainfall frequency analysis

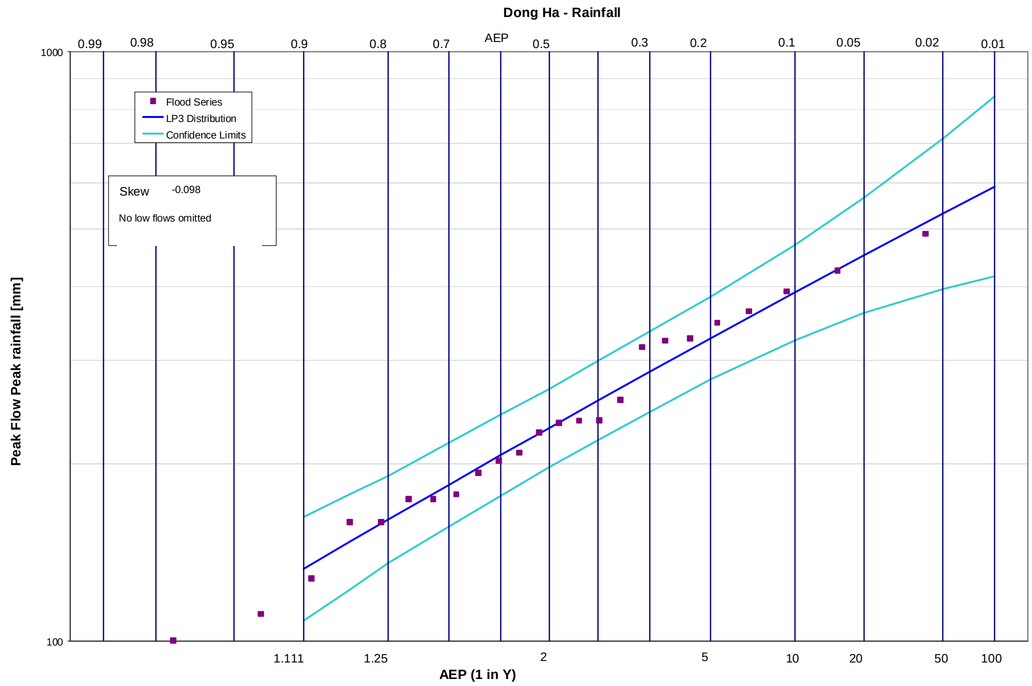

Figure 19: Dong Ha rainfall frequency analysis

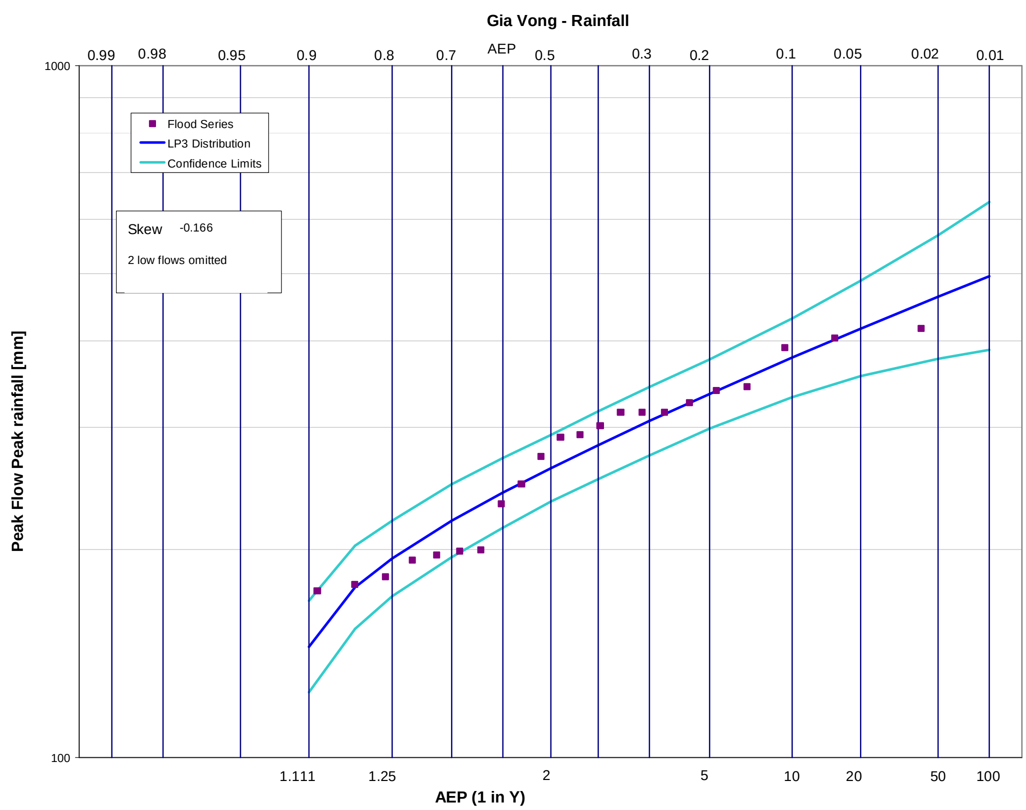

Figure 20: Gia Vong rainfall frequency analysis

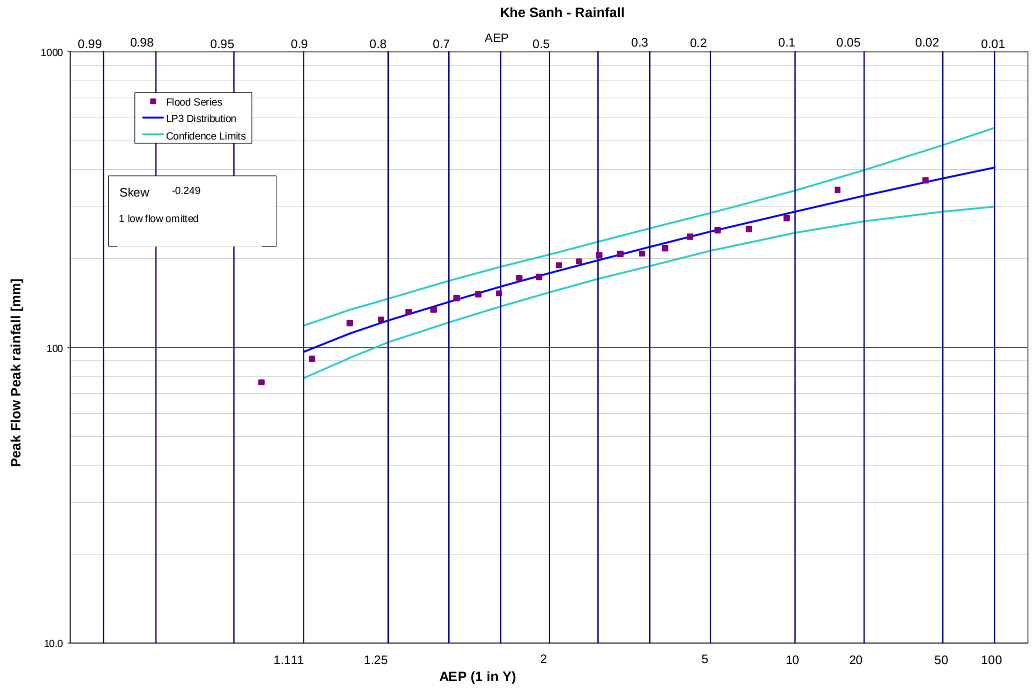

Figure 21: Khe Sanh daily rainfall frequency analysis

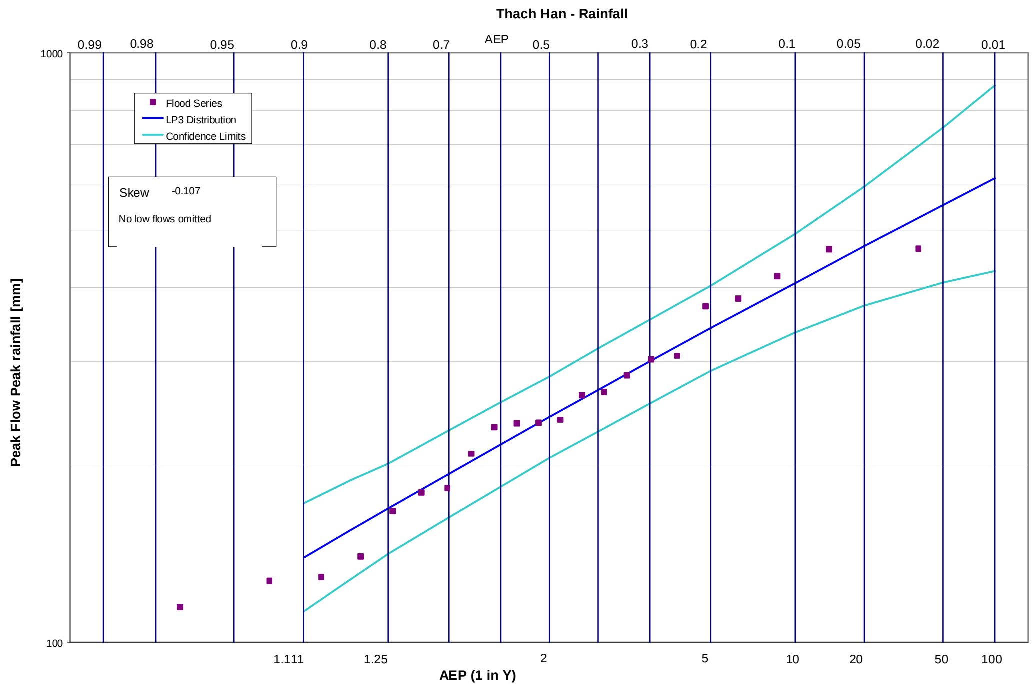

Figure 22: Thach Han daily rainfall frequency analysis

A-3 Short duration rainfall frequency tables

Table 20: Design values [mm] for twelve-hour rainfalls

station		Annual Exceedence Probability - %			12 hour	rainfall
	50	20	10	5	2	1
A Luoi	202	295	362	431	528	606
Con Co	164	263	344	434	570	689
Cua Viet	169	225	260	290	326	353
Dang Ha	173	245	293	339	398	443
Gia Vong	197	251	284	313	347	372
Khe Sanh	134	185	216	245	281	305
Thach Han	181	256	305	353	414	460

Table 21: Design values [mm] for six-hour rainfalls

Station		Annual Exceedence Probability - %			6 hour	rainfall
	50	20	10	5	2	1
A Luoi	126	185	227	270	331	380
Con Co	103	165	215	272	357	431
Cua Viet	106	141	163	181	204	221
Dang Ha	108	154	183	212	250	278
Gia Vong	123	157	178	196	218	233
Khe Sanh	84	116	135	153	176	191
Thach Han	113	160	191	221	259	288

Table 22: Design values [mm] for three-hour rainfalls

Station		Annual Exceedence Probability - %			3 hour	rainfall
	50	20	10	5	2	1
A Luoi	86	126	155	184	225	259
Con Co	70	112	147	185	243	294
Cua Viet	72	96	111	124	139	150
Dang Ha	74	105	125	145	170	189
Gia Vong	84	107	121	133	148	159
Khe Sanh	57	79	92	104	120	130
Thach Han	77	109	130	150	177	196

B. Rainfall runoff modeling part 1: model calibration

B-1 Thiessen's Weights

Table 23: Thiessen's weight values for every subarea.

area Station	Khe Sanh	Gia Vong	Dong Ha	Thach Han	Cua Viet	Con Co	A Luoi
BH1	-	1	-	-	-	-	-
BH2	-	0.8200	0.1800	-	-	-	-
BH3	-	1	-	-	-	-	-
CL1	0.7674	0.2326	-	-	-	-	-
CL2	-	0.8328	0.1672	-	-	-	-
CL3	-	0.4312	0.5688	-	-	-	-
OL1	-	-	-	0.2232	-	-	0.7768
OL2	-	-	-	0.7983	-	-	0.2017
OL3	-	-	-	1	-	-	-
OL4	-	-	-	1	-	-	-
OL5	-	-	-	1	-	-	-
SL1	-	0.9820	-	-	-	0.0180	-
SL2	-	1	-	-	-	-	-
SL3	-	1	-	-	-	-	-
TH1	0.6661	-	-	0.0436	-	-	0.2903
TH2	0.2442	0.0117	0.5821	0.1620	-	-	-
TH3	-	-	0.0117	0.9883	-	-	-
TH4	-	-	-	1	-	-	-
TH5	0.4395	-	-	0.5605	-	-	-

B-2 Initial conditions

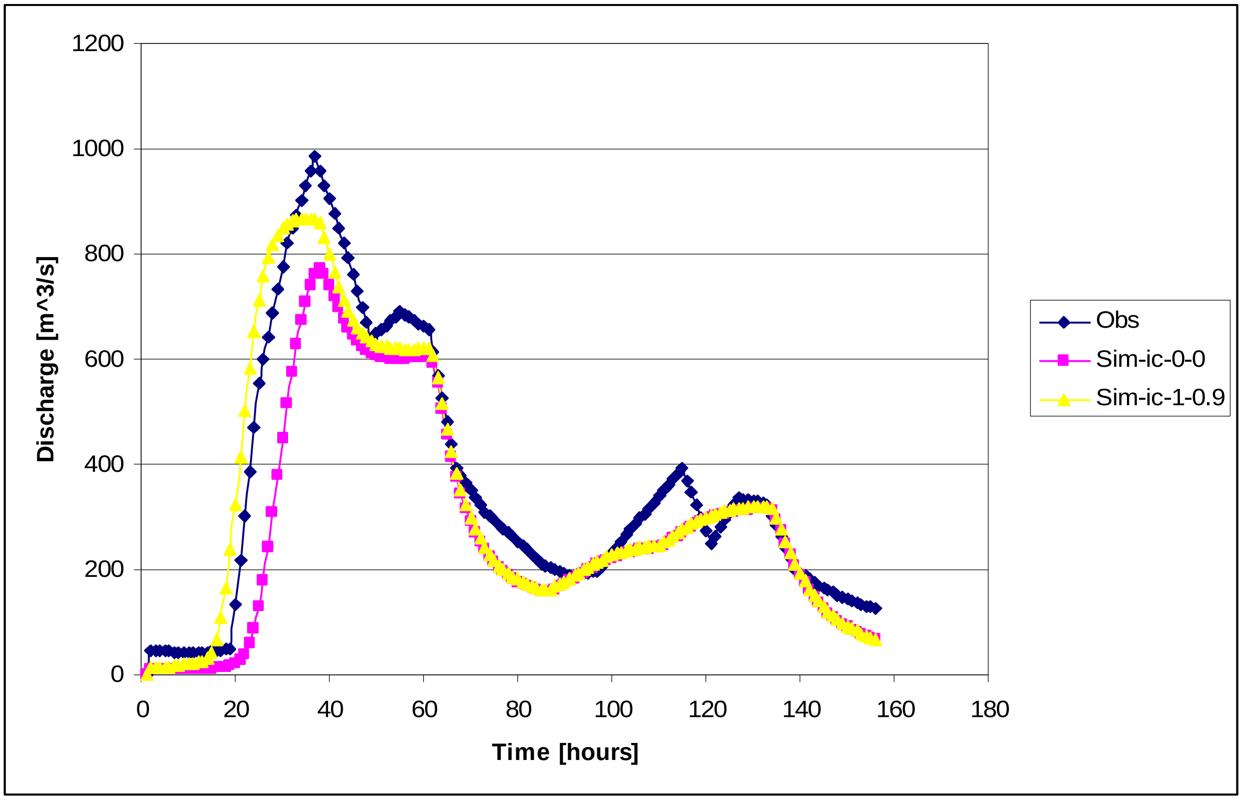

Figure 23: Differences initial conditions U/Umax and L/Lmax

B-3 Calibration Results

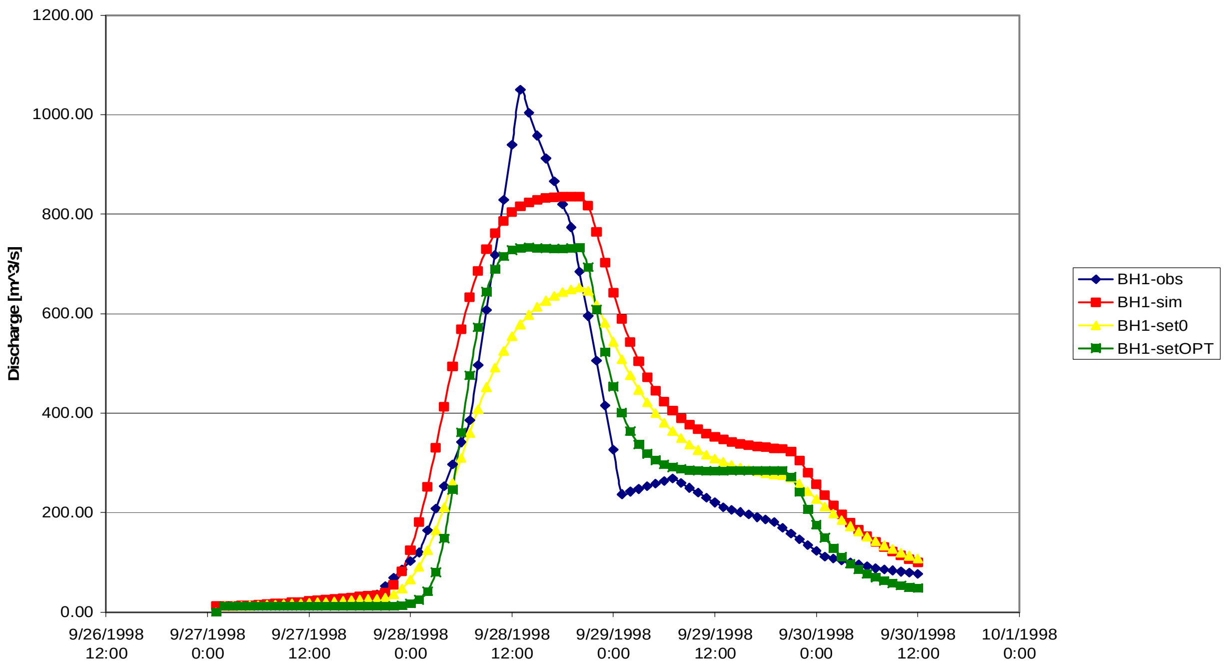

Figure 24: Flood hydrographs of the 1998-flood with: observed rainfall (blue), used parameters (red), parameters found by Shamsudin & Hashim (yellow), the best parameters for this flood (green) according to the value of R^2.

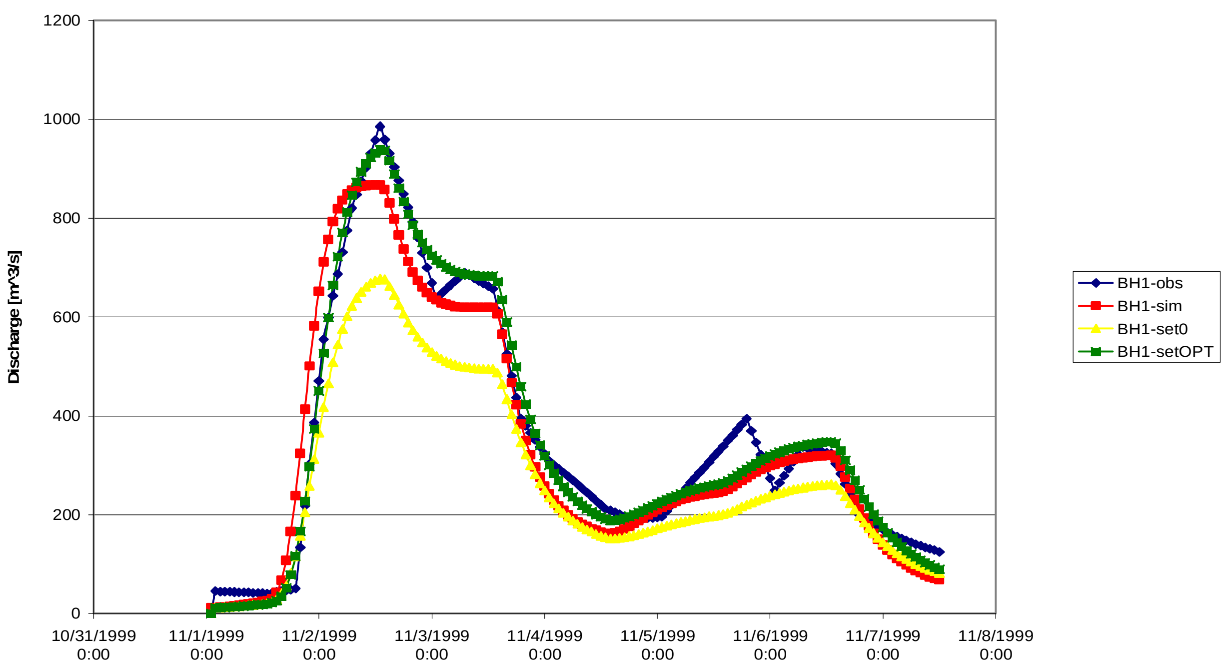

Figure 25: Flood hydrographs of the 1999-flood with: observed rainfall (blue), used parameters (red), parameters found by Shamsudin & Hashim (yellow), the best parameters for this flood (green) according to the value of R^2.

C. Rational method

Table 24: Design floods [m3/s] estimated with the rational method for M=1.

Subarea	AEP (%)	50	20	10	5	2	1
BH1		781	1282	1625	1955	2382	2700
BH2		202	338	433	525	647	739
BH3		44	73	92	111	135	153
CL1		715	1239	1613	1977	2466	2827
CL2		182	303	388	471	580	662
CL3		193	336	439	544	685	795
OL1		679	1264	1736	2240	2986	3605
OL2		433	791	1068	1355	1763	2089
OL3		244	444	595	749	966	1135
OL4		117	212	285	359	462	543
OL5		197	358	480	604	779	915
SL1		228	375	477	576	705	802
SL2		639	1049	1329	1599	1948	2208
SL3		75	123	155	187	228	258
TH1		1758	3191	4282	5400	6996	8252
TH2		457	827	1104	1387	1780	2085
TH3		251	455	611	769	991	1165
TH4		114	208	278	351	452	531
TH5		721	1298	1729	2162	2763	3224

Table 25: Design floods [m3/s] estimated with the rational method for M=3.

Subarea	AEP (%)	50	20	10	5	2	1
BH1		781	1282	1625	1955	2382	2700
BH2		292	479	611	745	922	1058
BH3		96	154	195	235	288	328
CL1		718	1223	1594	1978	2500	2916
CL2		262	430	548	668	826	947
CL3		279	476	621	772	977	1138
OL1		699	1327	1849	2430	3296	4032
OL2		624	1121	1509	1921	2512	2990
OL3		353	628	841	1063	1376	1624
OL4		169	301	403	509	659	778
OL5		284	507	678	857	1109	1310
SL1		329	532	674	817	1004	1148
SL2		639	1049	1329	1599	1948	2208
SL3		108	174	219	265	324	369
TH1		2440	4410	5971	7671	10122	12167
TH2		457	796	1054	1325	1701	2001
TH3		362	645	863	1091	1412	1667
TH4		165	294	394	497	644	760
TH5		706	1267	1705	2176	2844	3392

D. Flood hydrographs

D-1 Flood hydrographs rational method

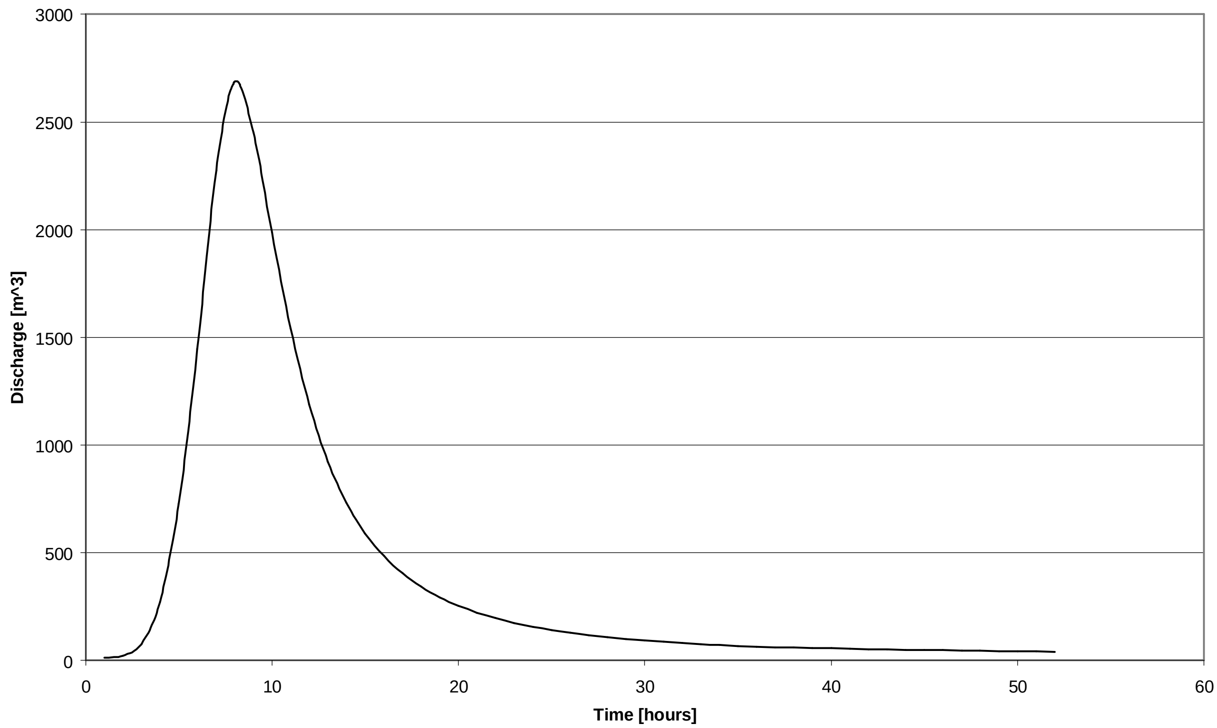

Figure 26: Ben Hai 1 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

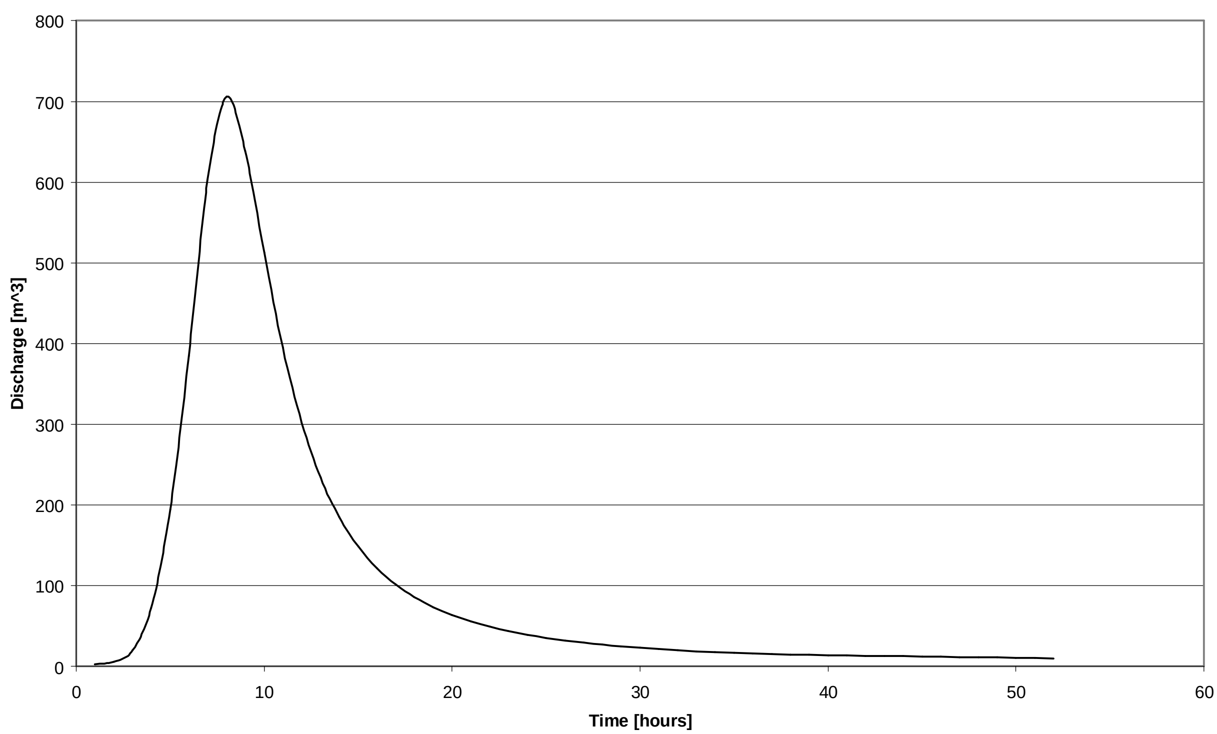

Figure 27: Ben Hai 2 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

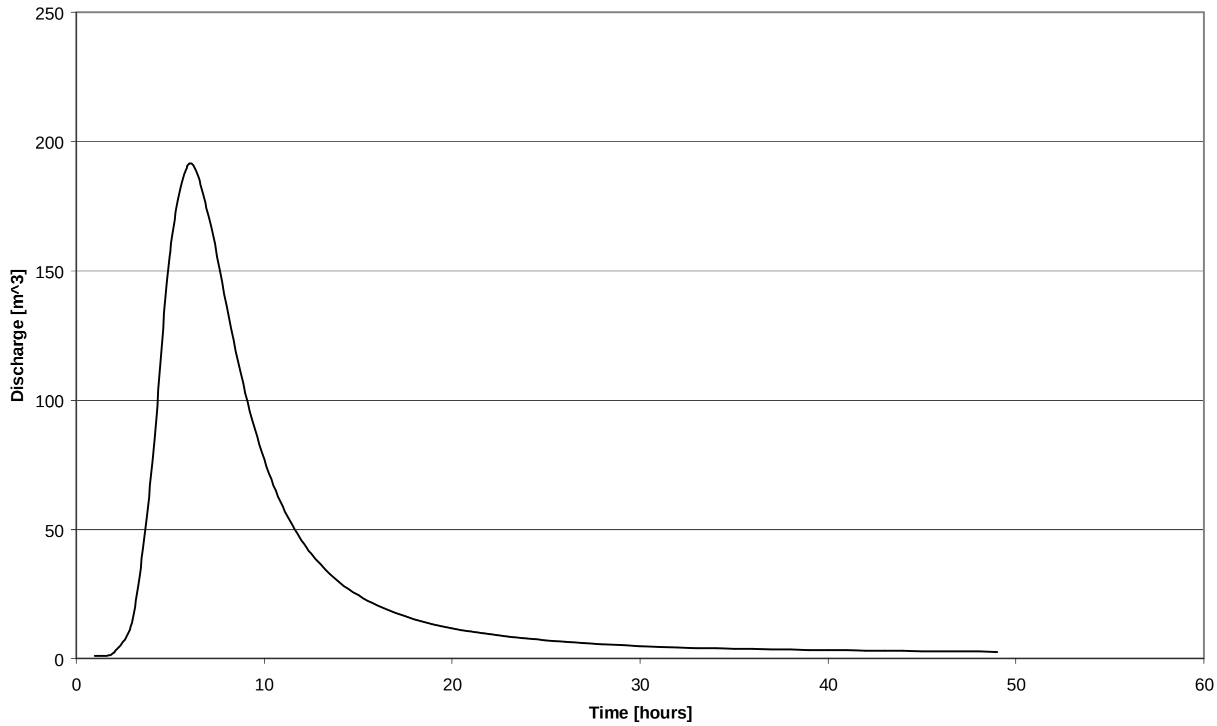

Figure 28: Ben Hai 3 AEP 1:100 total runoff hydrograph based on a three-hour rainfall.

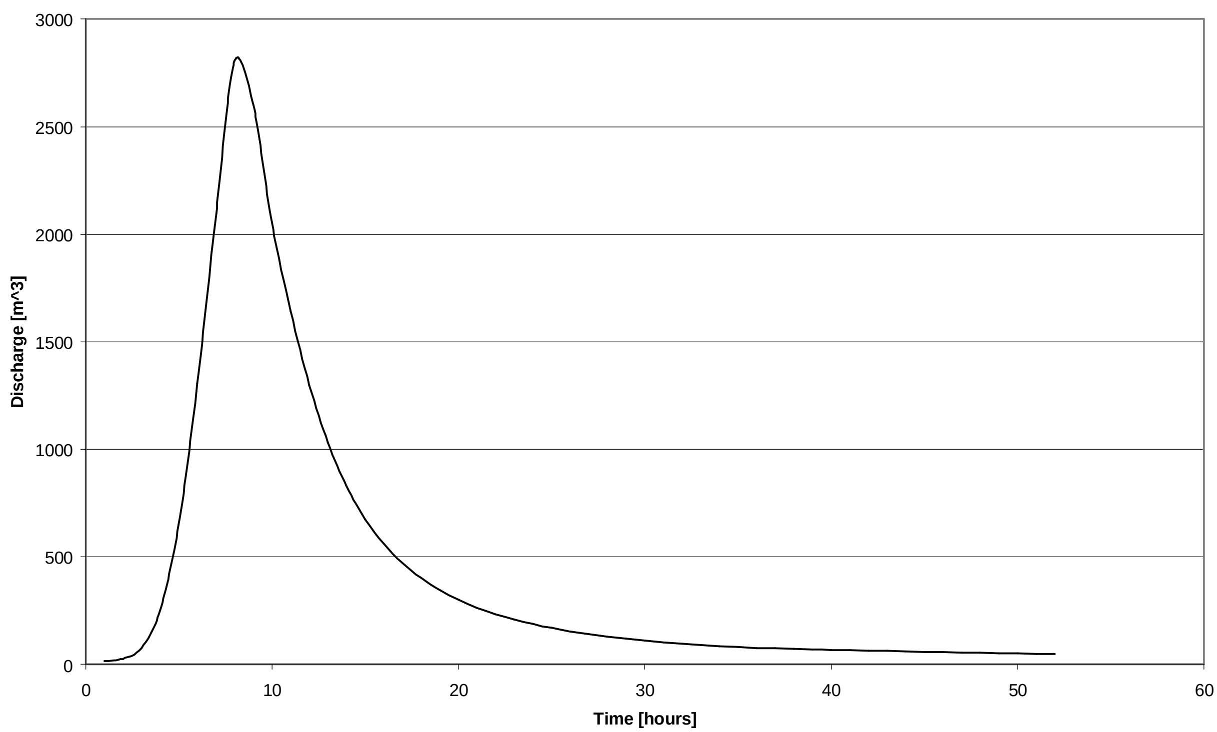

Figure 29: Cam Lo 1 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

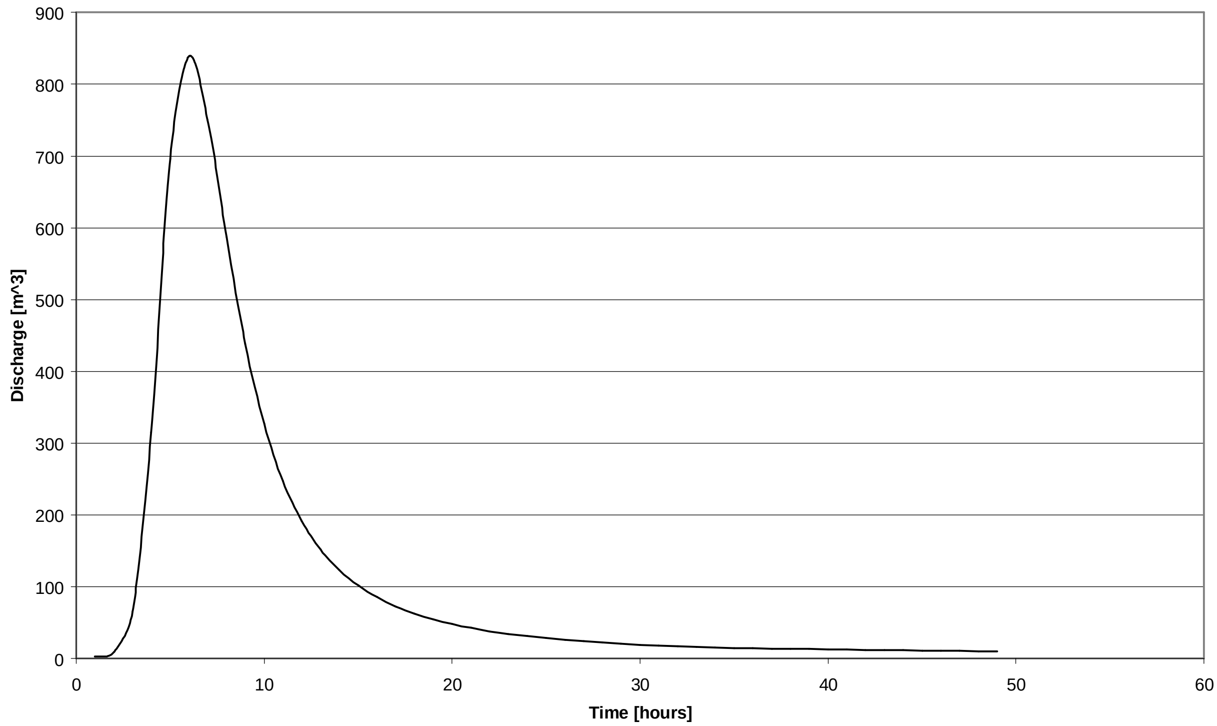

Figure 30: Cam Lo 2 AEP 1:100 total runoff hydrograph based on a three-hour rainfall.

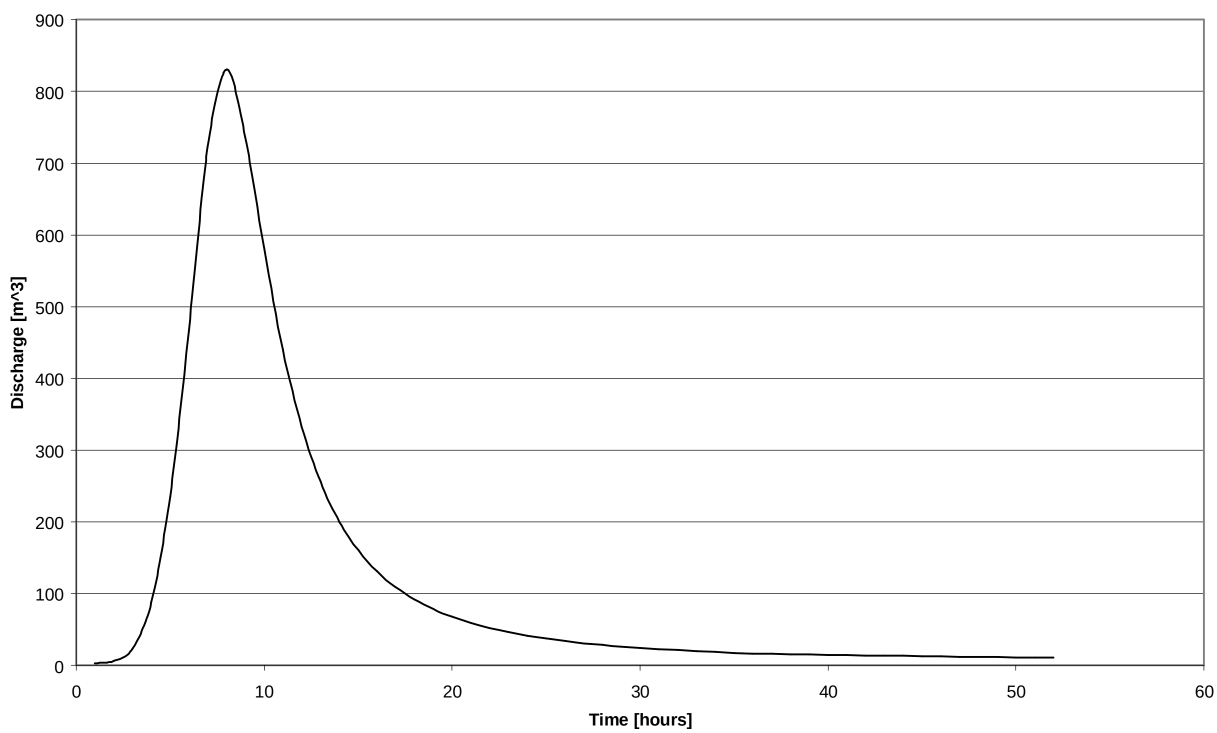

Figure 31: Cam Lo 3 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

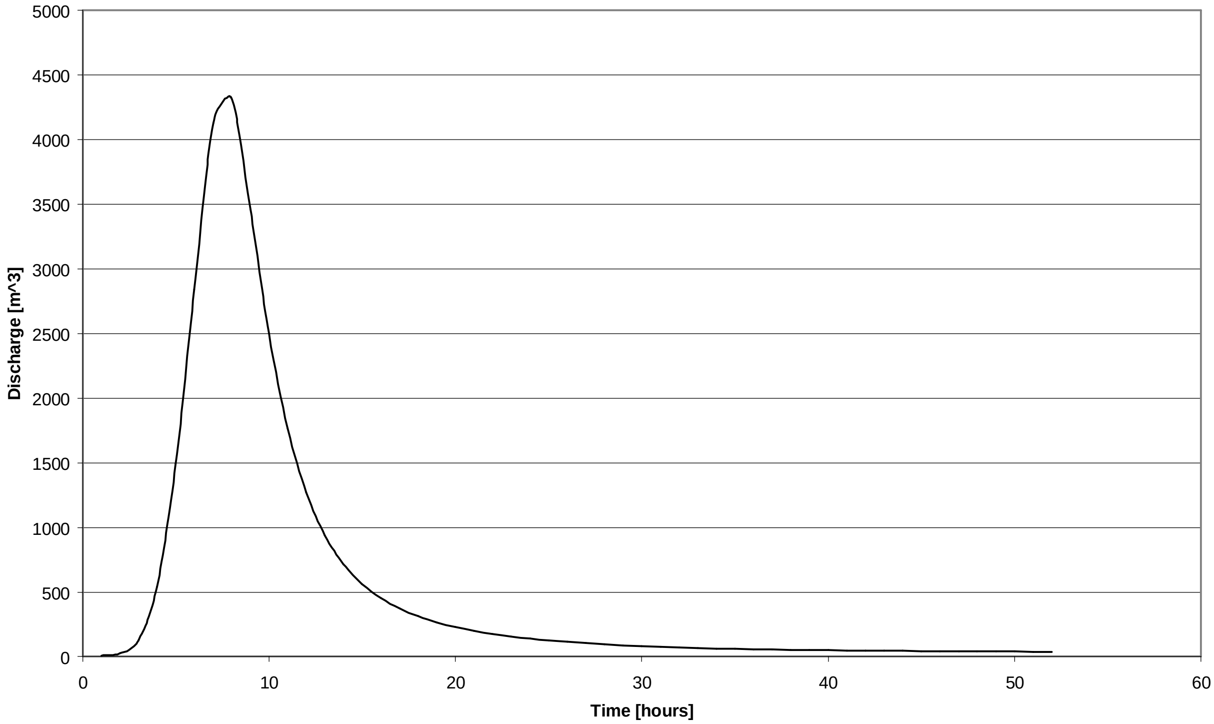

Figure 32: O Lau 1 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

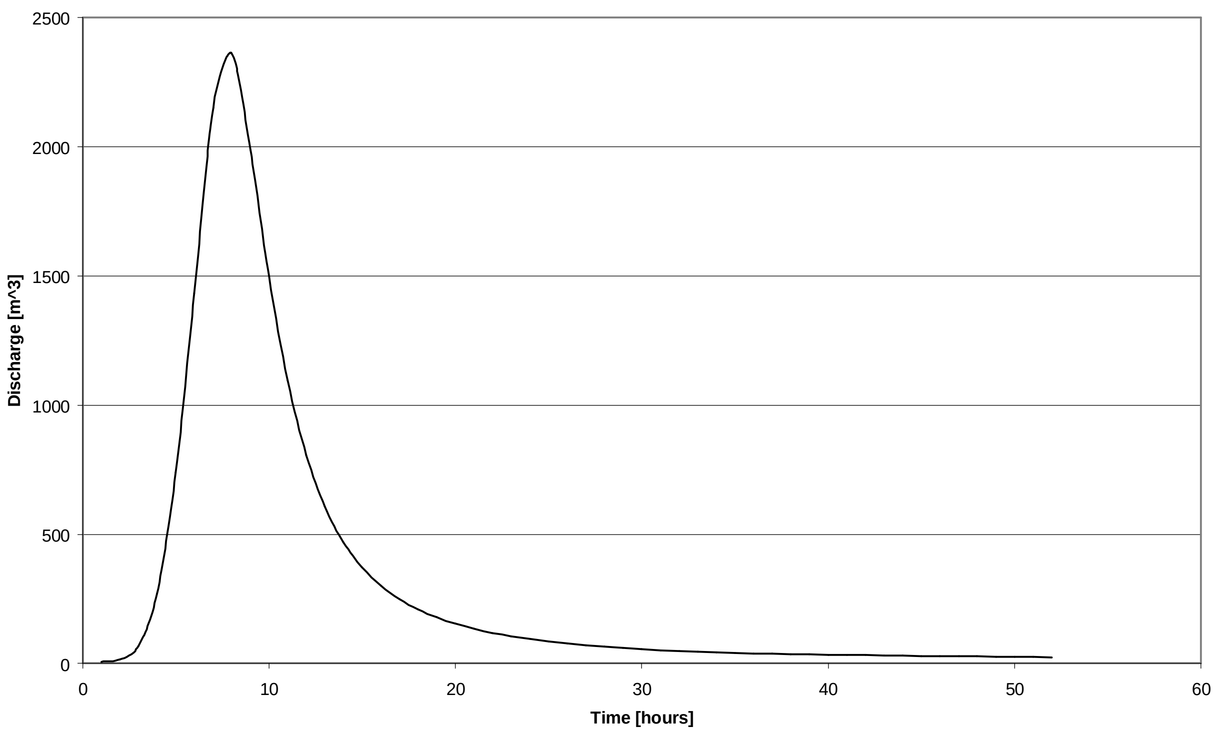

Figure 33: O Lau 2 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

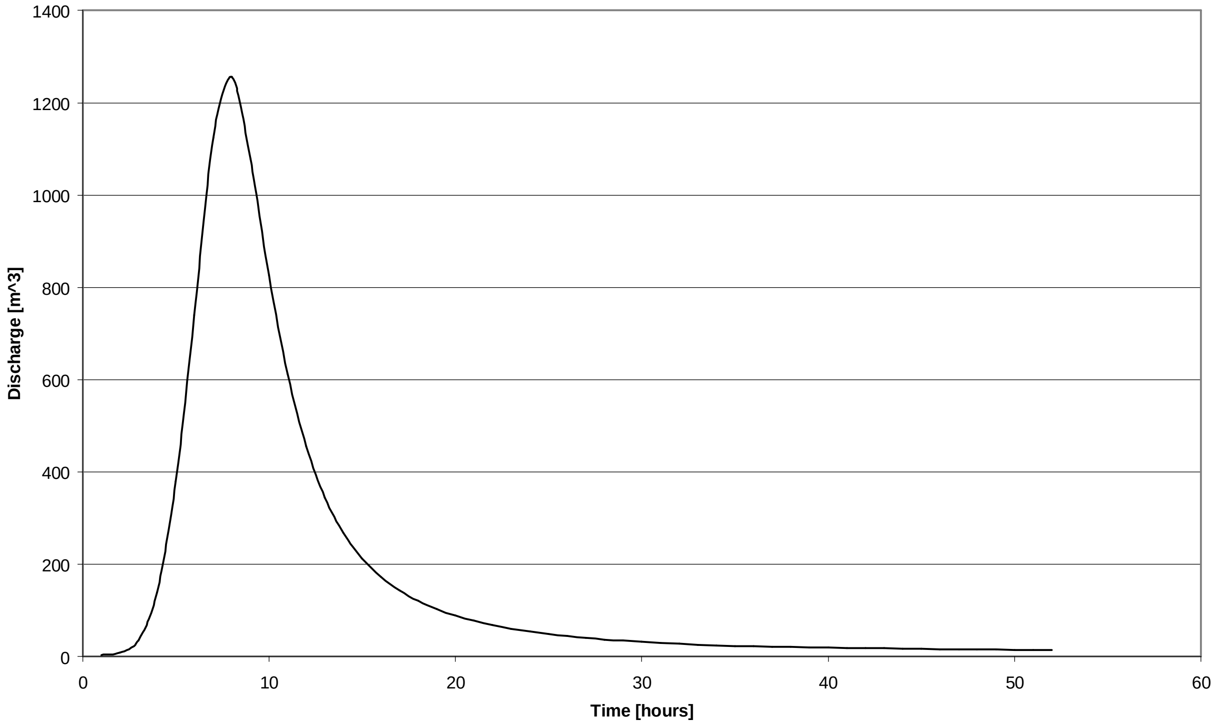

Figure 34: O Lau 3 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

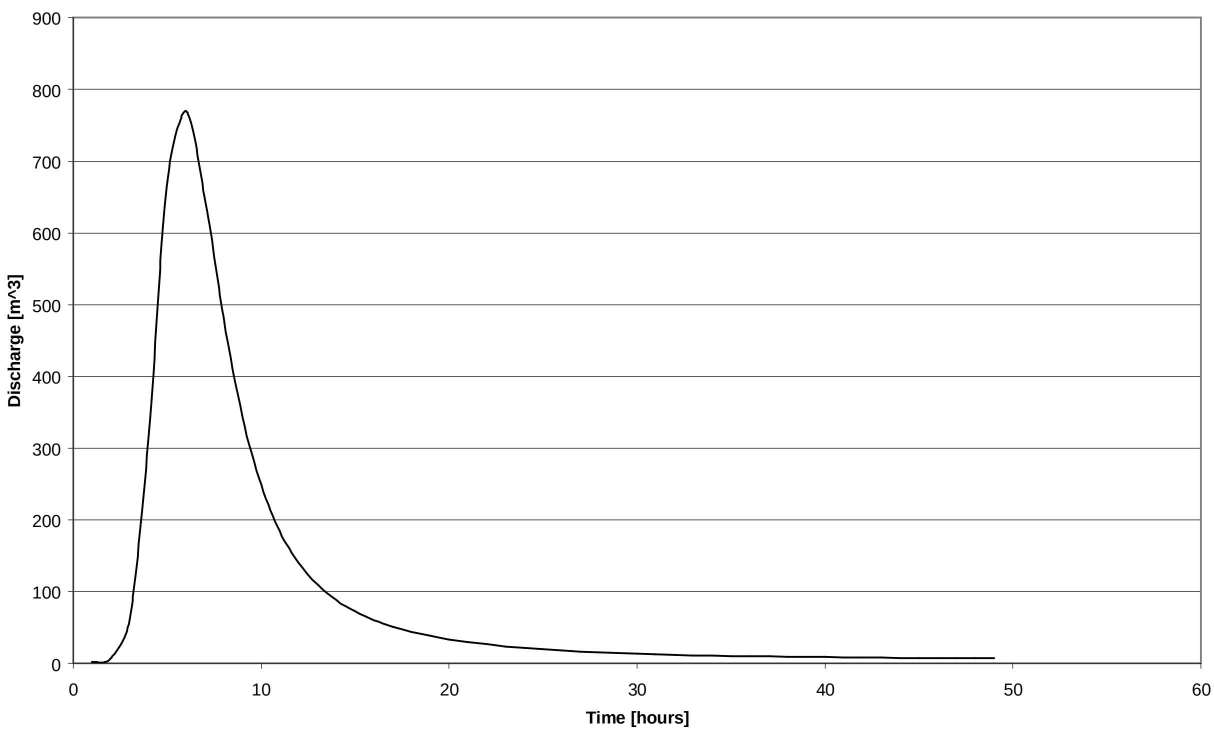

Figure 35: O Lau 4 AEP 1:100 total runoff hydrograph based on a three-hour rainfall.

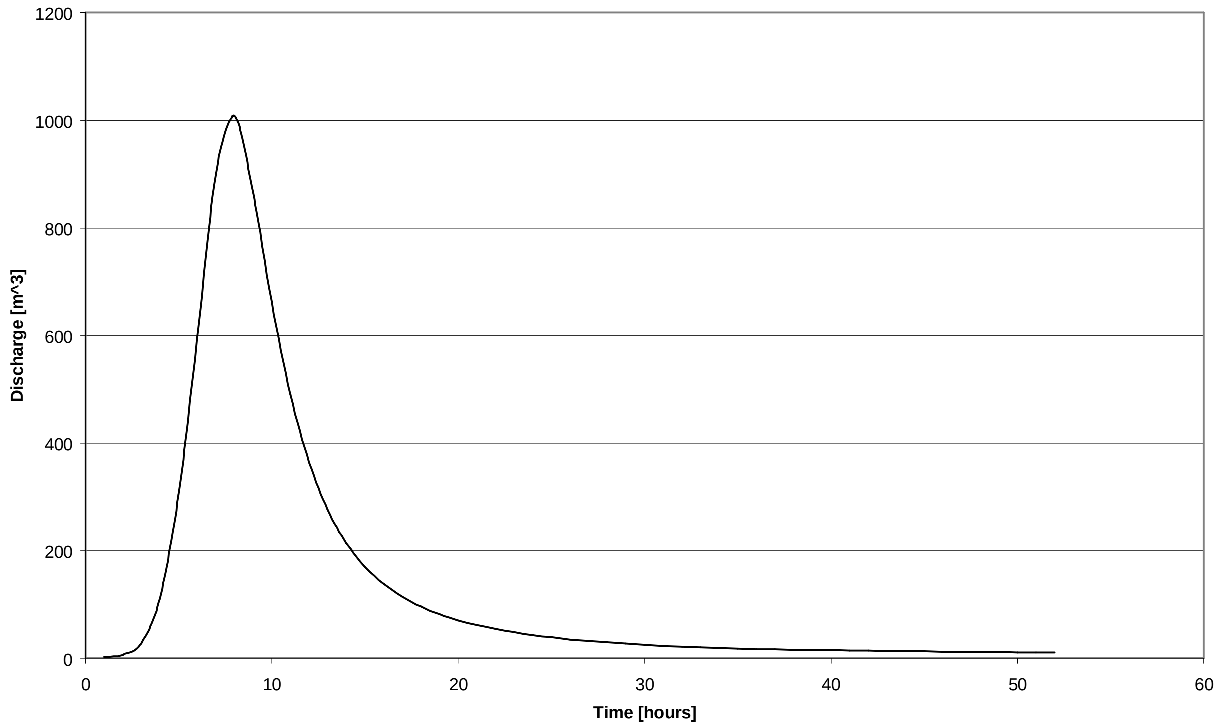

Figure 36: O Lau 5 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

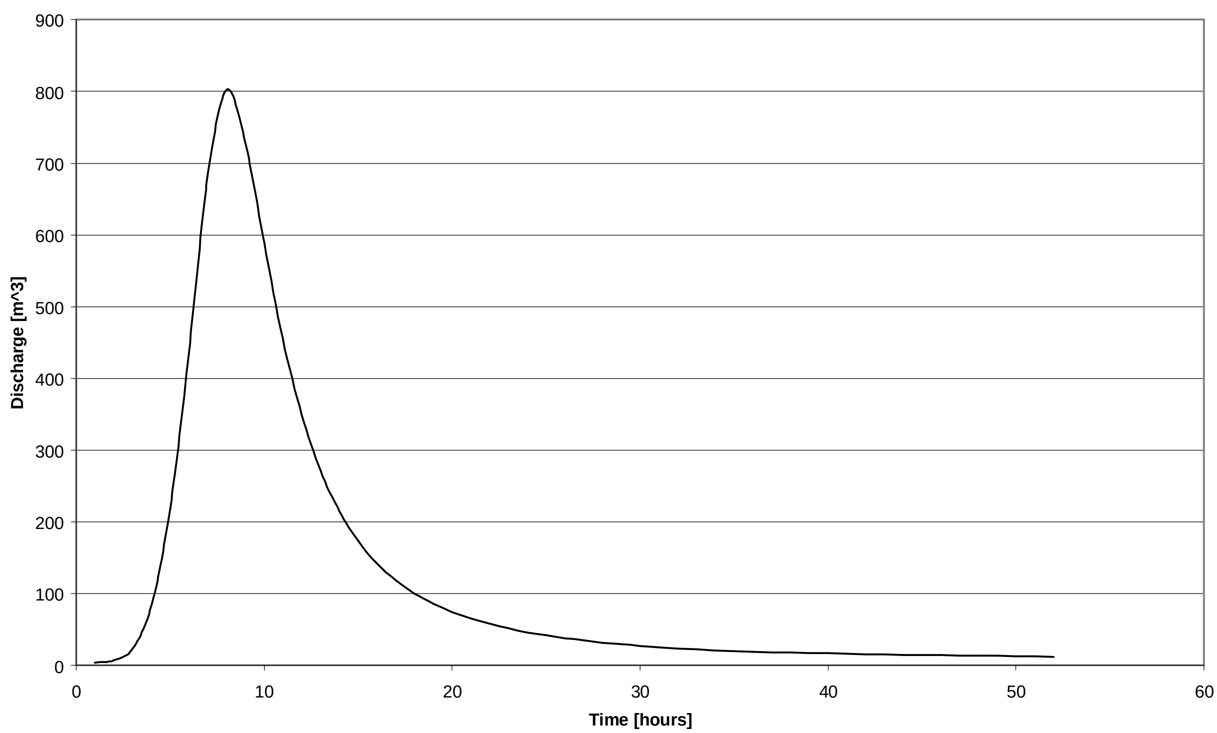

Figure 37: Sa Lung 1 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

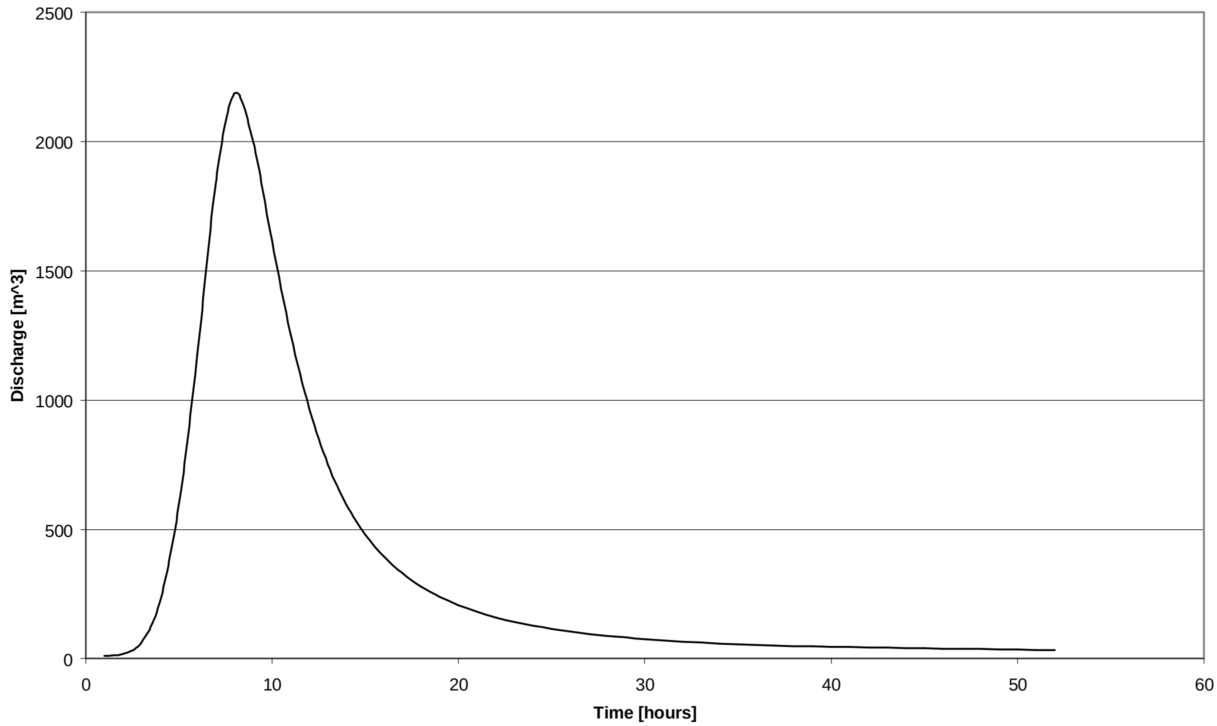

Figure 38: Sa Lung 2 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

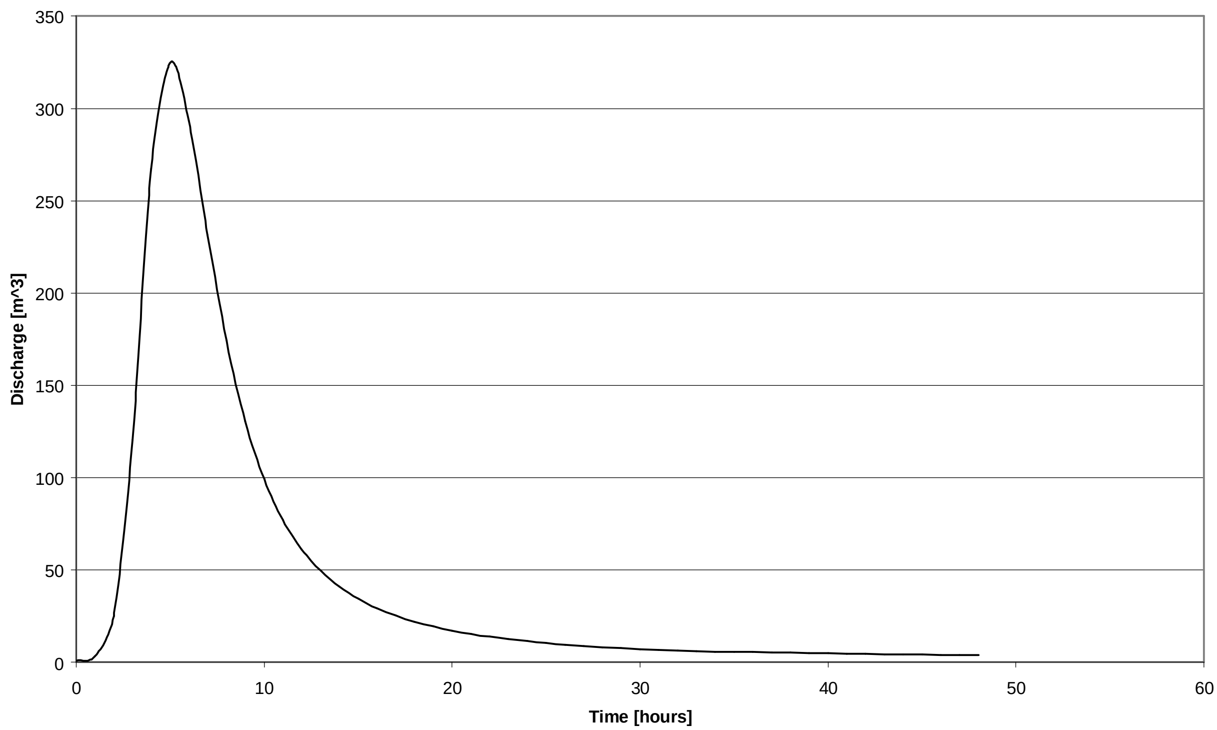

Figure 39: Sa Lung 3 AEP 1:100 total runoff hydrograph based on a three-hour rainfall.

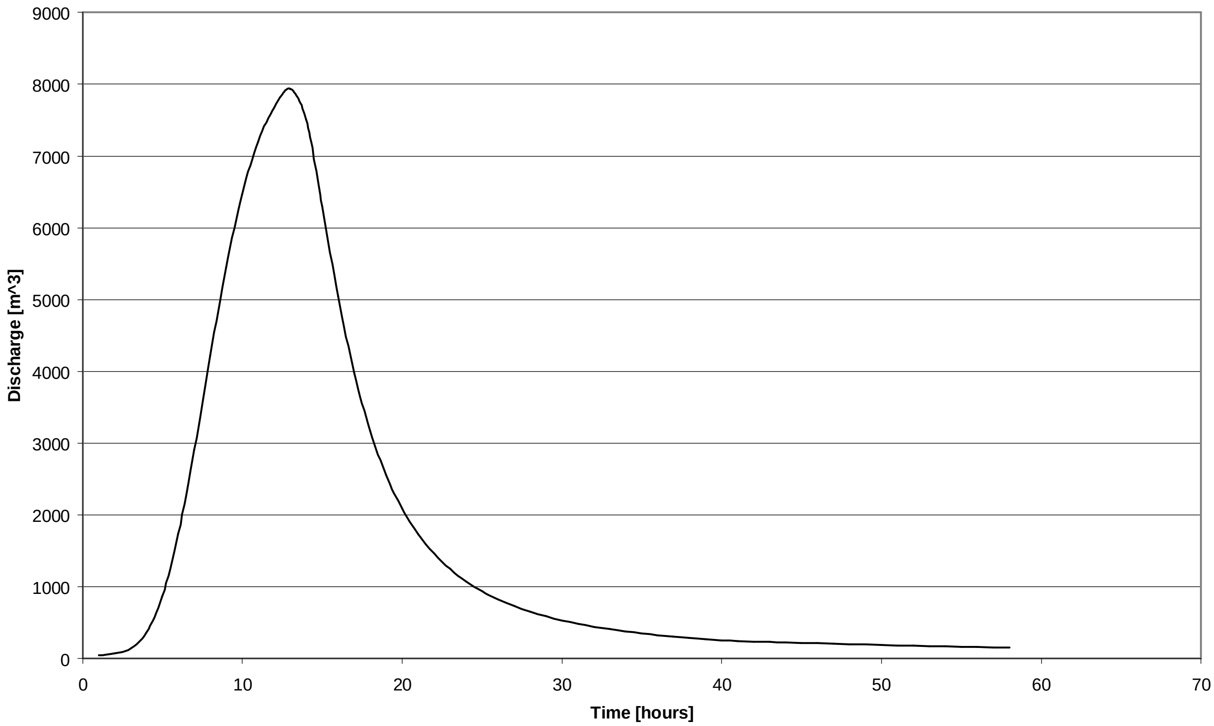

Figure 406: Thach Han 1 AEP 1:100 total runoff hydrograph based on a twelve-hour rainfall.

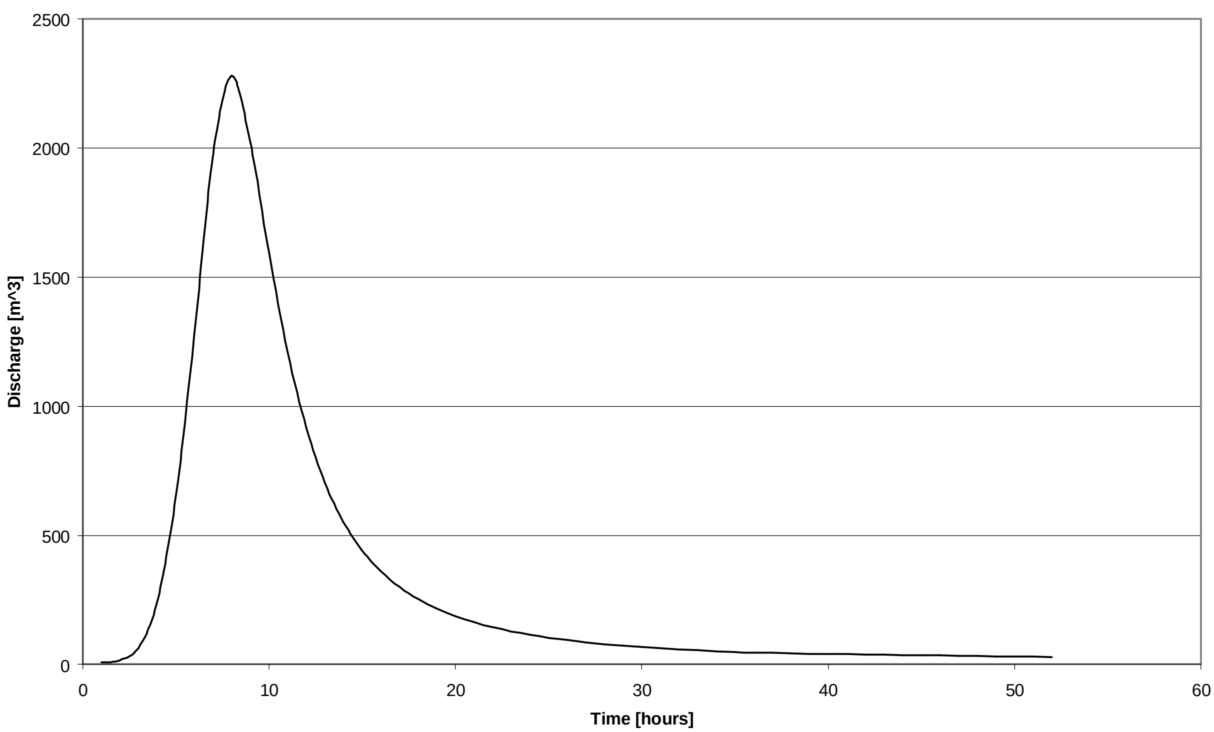

Figure 41: Thach Han 2 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

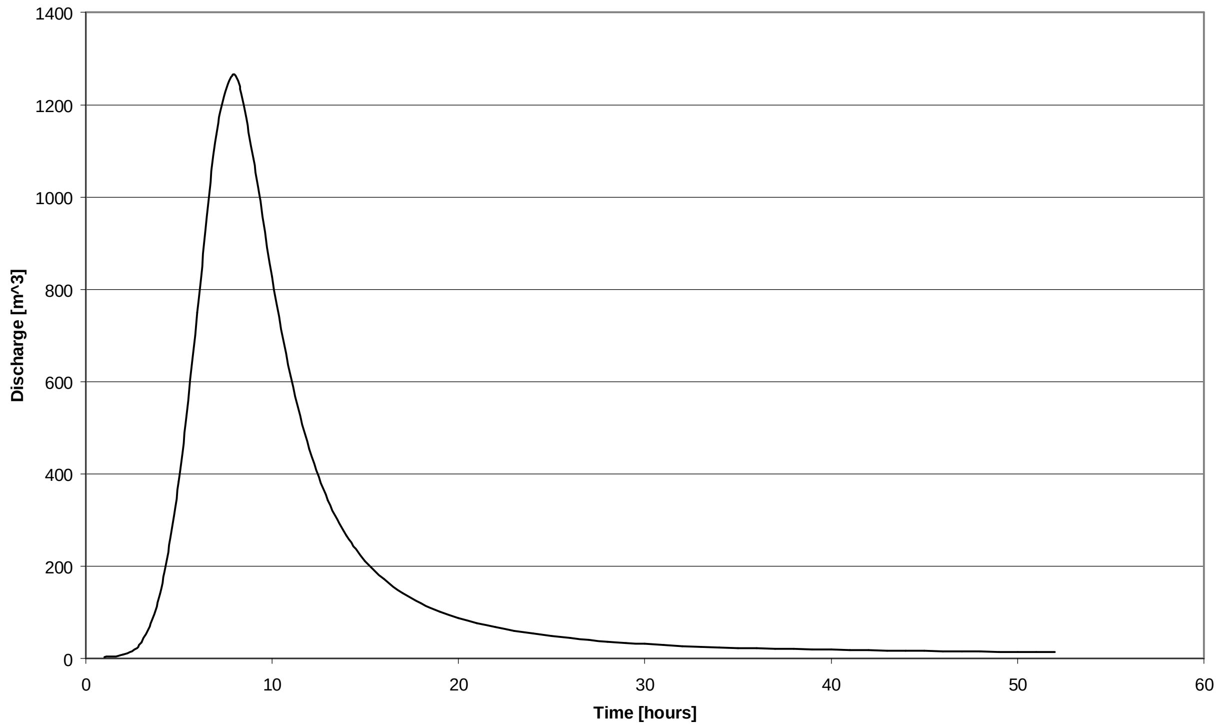

Figure 42: Thach Han 3 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

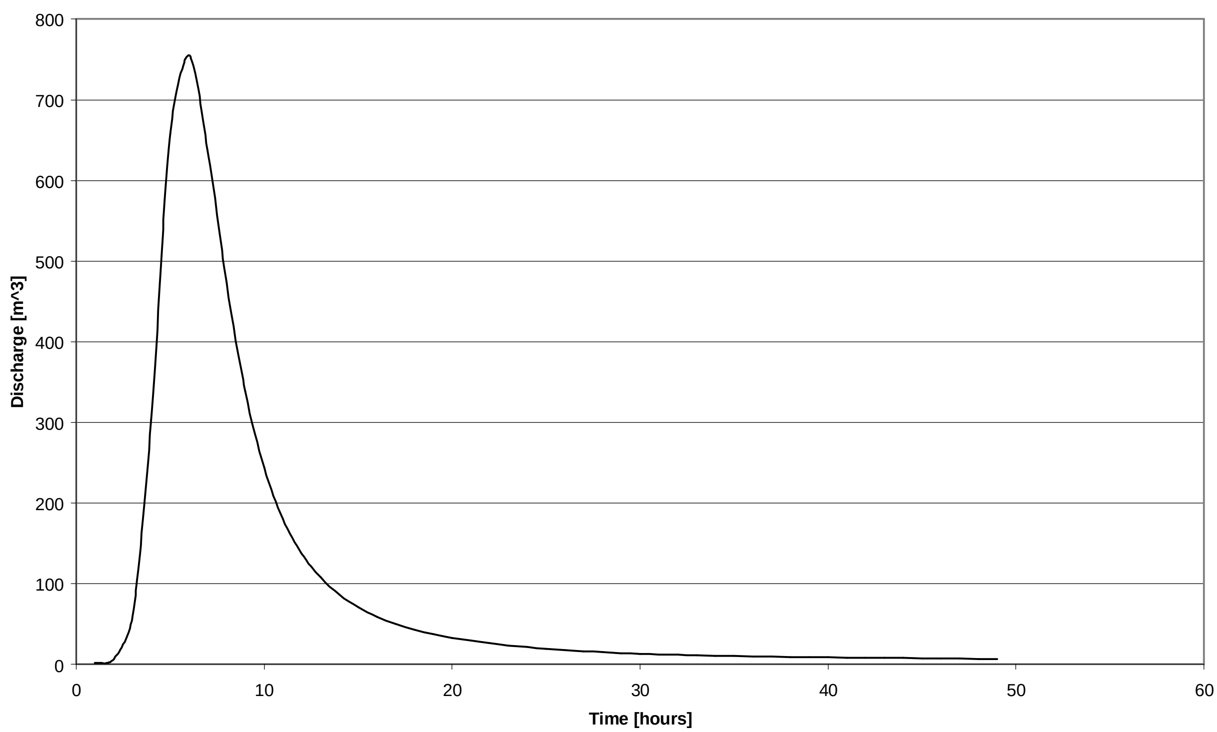

Figure 43: Thach Han 4 AEP 1:100 total runoff hydrograph based on a three-hour rainfall.

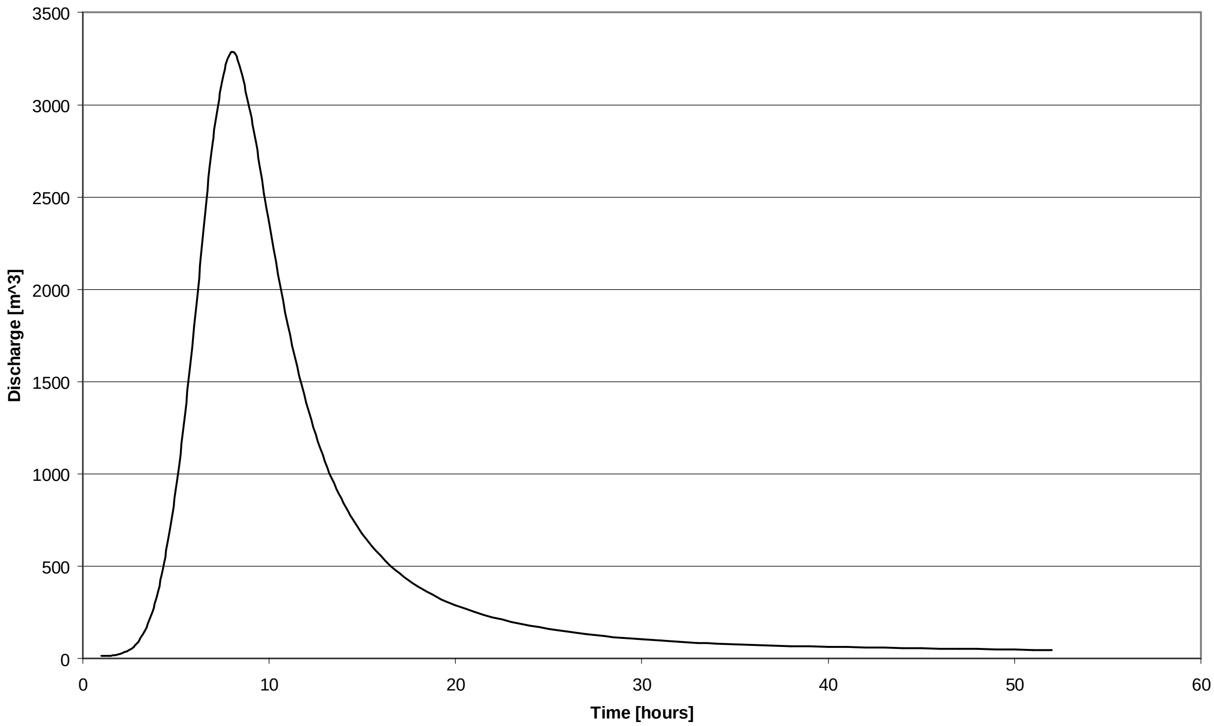

Figure 44: Thach Han 5 AEP 1:100 total runoff hydrograph based on a six-hour rainfall.

D-2 Flood hydrographs three-day rainfall method

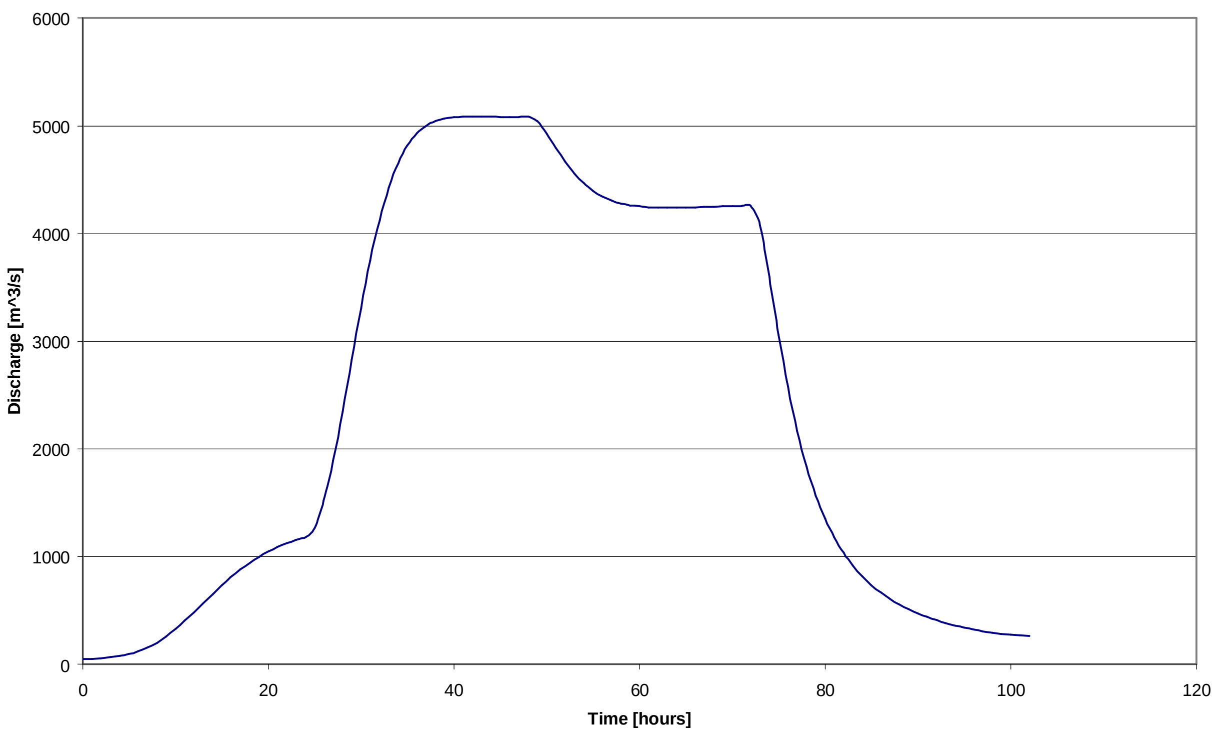

Figure 45: Thach Han 1 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

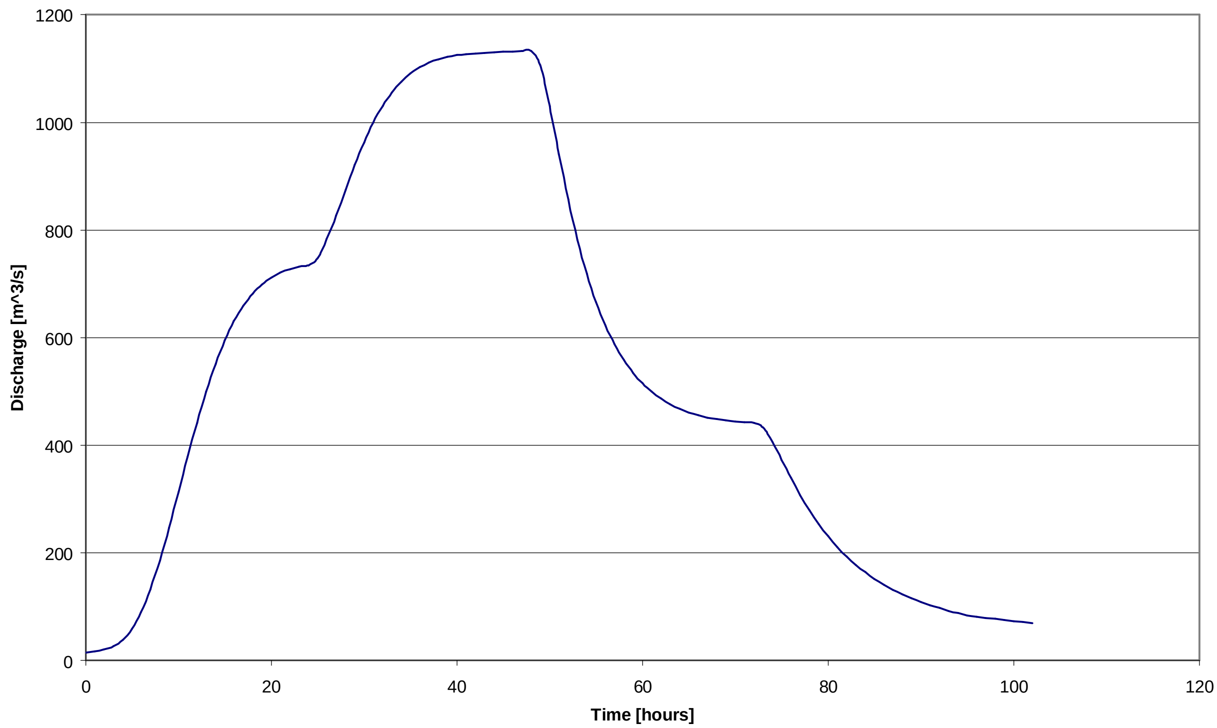

Figure 46: Cam Lo 1 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

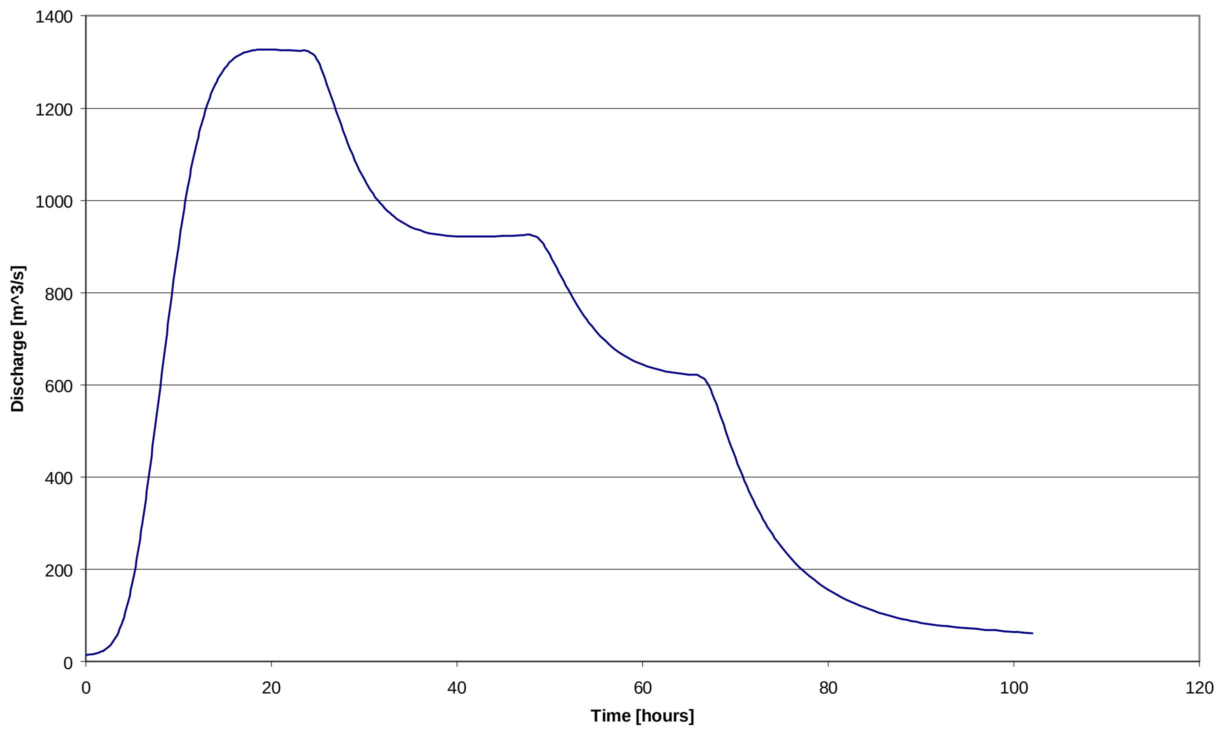

Figure 47: Thach Han 5 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

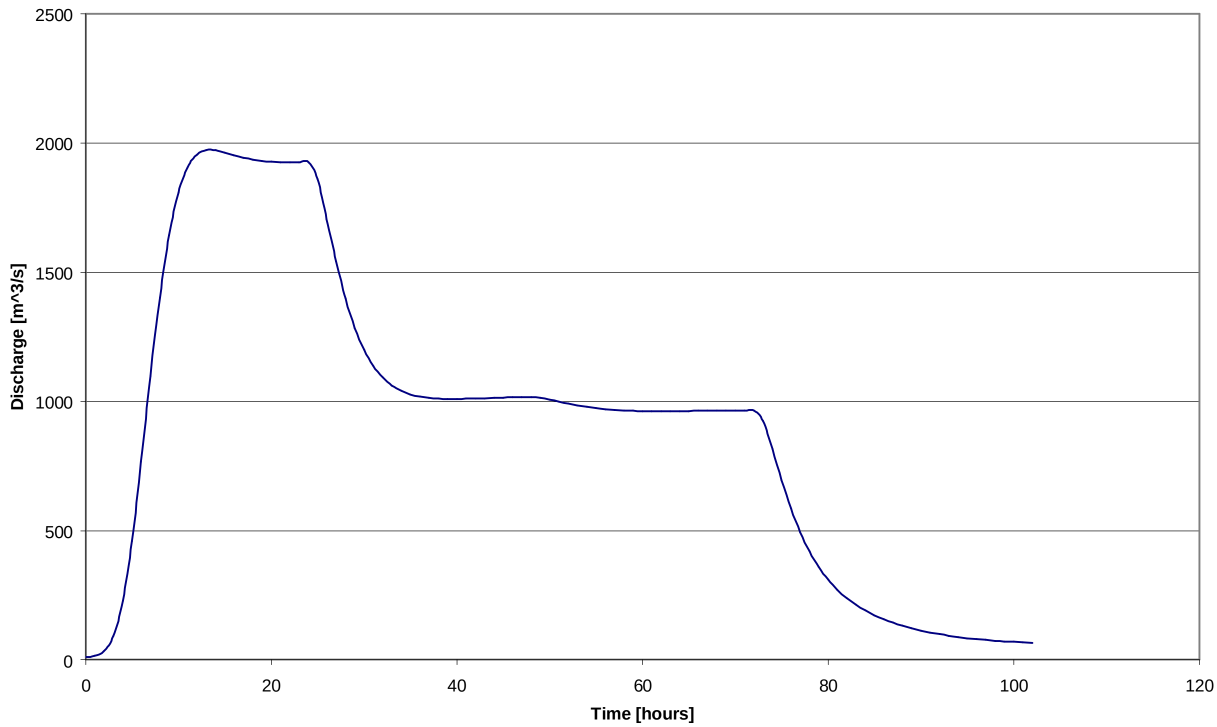

Figure 48: O Lau 1 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

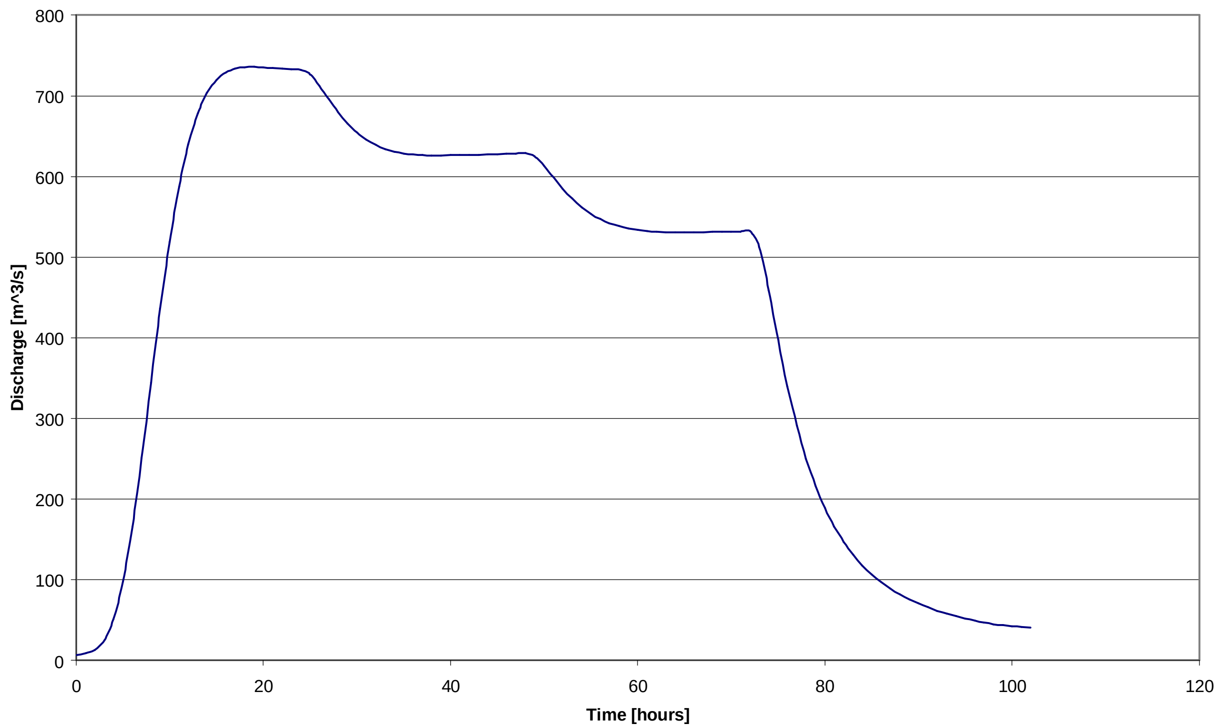

Figure 49: O Lau 2 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

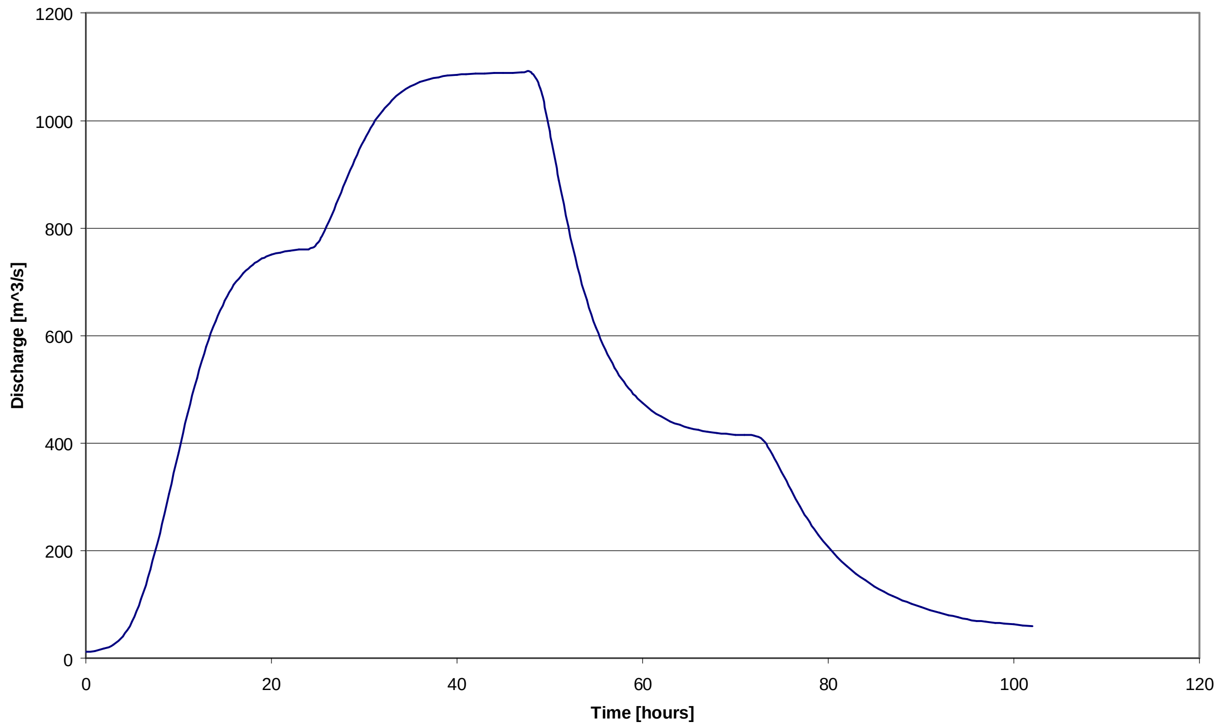

Figure 50: Ben Hai 1 AEP 1:100 total runoff hydrograph based on a three-day rainfall.

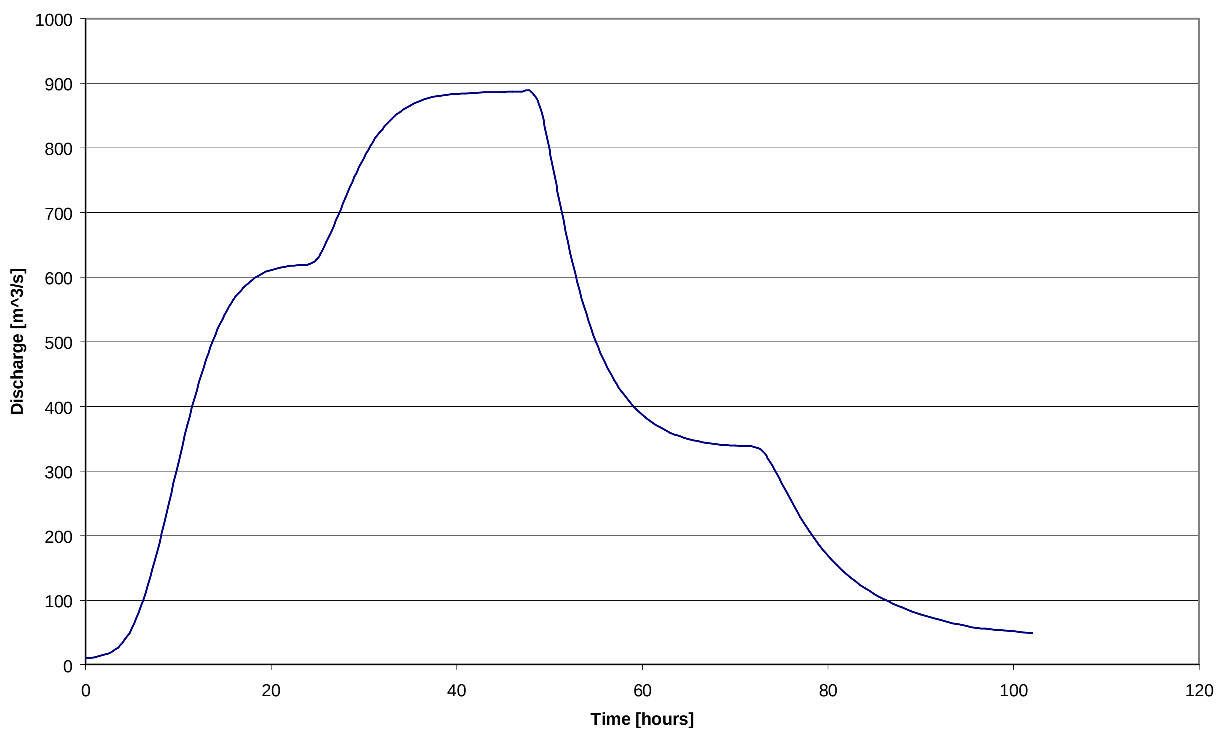

Figure 51: Sa Lung 2 AEP 1:100 total runoff hydrograph based on a three-day rainfall.