Flood Hazard assessment for Upper Se San

Appendix 4 to the Annex 1 to the
Draft Stage 2 Evaluation Report

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1 Introduction

This Appendix 4 to Annex 1 deals with flood hazard assessment for flash floods. In the provisional BDP list of projects for Vietnam the Project Drainage and Flood Control Kontum in Upper Se San is mentioned. The type of floods that creates damages here are flash floods. VNMC has proposed this area as focal area for flood risk assessment due to flash floods. From the Field Visit Consultants undertook to Kontum on 22 February, 2008, it was learned that the flooding problems are not around the city but rather in the uplands. Little data could be collected for these areas. Therefore, it was decided to focus in this Focal Area particularly on flood hazard assessment procedures for flash floods using generally applicable techniques.

In Chapter 2 an overview is given of flood hazard assessment procedures for flash floods. A description of the drainage basins in the Upper Se San is provided in Chapter 3. Application of the hazard procedures with a provisional design storm development for the Upper Se San is dealt with in Chapter 4.

2 Flood hazard assessment for flash floods

2.1 General

Flash floods occur in the steep sloped upper reaches of the basins due to intense rainfall after a long rainy period forcing the catchment to respond quickly to the rainfall. Flash floods are short lived, typically in the order of hours, rise and fall rapidly and the flow velocities are very high. Effects of flash floods, when accompanied with landslides, are equivalent to dam break waves. The hydrological monitoring infrastructure in the Lower Mekong basin is such that only a limited number of basins are equipped with rainfall and water level recorders. For most locations only daily rainfall is available. The water level gauging network is generally less dense and the monitoring interval is often too large to properly describe flash flood. Very often only daily water levels is available fully unsuitable for flash floods. With respect to flow discharge measurements usually cover only the lower stages and occasionally include flood data. These conditions have to be taken into account when flood hazard assessment procedures are developed.

In the following three procedures are proposed:

1. Flood hazard derived from rainfall extremes
2. Flood hazard derived from observed flows
3. Flood hazard determined from regional flood statistics

2.2 Outline of procedures

2.2.1 Flood hazard derived from rainfall extremes

When using rainfall extremes for the assessment of the flood hazard the following main steps are involved:

1. Collection of relevant information
2. Database development and field visit
3. Determination of design rainstorms for different return periods
4. Transformation of design rainstorms into design hydrographs
5. Transformation of design hydrographs into design levels.

These steps are elaborated below.

Collection of relevant information

Data required for flood hazard assessment includes:

1. Digital Elevation Model of the area or topographical maps;
2. Soils, geological and past, present and planned land use maps;
3. Data on historical floods;
4. Layout of hydro-meteorological network in the river basin, its operation and maintenance;

5. Collection of time series of rainfall, climatic variables, water levels, stage-discharge measurements, discharges and sediment of the river basin;
6. Data on hydraulic infrastructure, dimensions and operation in times of flood;
7. Survey data of river and flood plain at and upstream and downstream of the reach under investigation and relevant hydraulic parameters including bed material, vegetation, embankment elevation, etc
8. Data on morphological development of the river and its environs.

Database development and field visit

The collected information is next organized in a database consisting of GIS-related and time oriented data. In the database a clear distinction is to be made between original/raw data, data under validation and fully validated data. Thorough validation, correction and completion procedures should be applied to arrive at a reliable set of information. In this data collection and validation process one or more field visits is essential to validate collected information and to assess its sufficiency. As a result additional data may be collected or a (temporary) monitoring program is set up and executed.

Design rainstorms

The determination of design storms involves the following steps:

1. Assessment of the time of concentration T_c of the basin up to the location for flood hazard assessment;
2. Determine basin rainfall from point rainfall for smallest interval available if applicable.
3. Development of intensity-duration-frequency curves for return periods of 2 to 100 years, for durations from 5 min to 24 hours (standard option in MRC-HYMOS data base and data processing system). If no short interval rainfall is available then the short duration intensities are derived from daily rainfall data. This involves the following steps:
   - 3.a Extreme value analysis of annual maximum daily rainfall by fitting EV1 or GEV-distribution to the observed distribution and determine the values for the requested return periods;
   - 3.b Adjustment of the daily values to 24 hr values by applying a correction factor of 1.12 to the daily extremes
   - 3.c Adjustment of the 24 hr values for possible orographical effects, based on regional rainfall characteristics as function of elevation
   - 3.d Select regionally acceptable values for the coefficients b and n in the following intensity duration relation:

$$I = \frac{a}{(D+b)^n} \qquad (2.1)$$

where: I = rainfall intensity (mm/hr)
D = rainfall duration (hr)
a,b,n are constants

Some regional values have been included in the following Table 2.1

Table 2.1 Intensity-duration formula constants (Watkins, et al, 1984)

Location	range of duration	b	n
Phnom Penh	5 min-24 hrs	0.23	0.86
Singapore	15 min-24 hrs	0.5	0.73-0.84
Sarawak	15 min-24 hrs	0.9	0.83-0.88
Malasia East Coast	15 min-24 hrs	0.3	0.54-0.76
Malasia West Coast	15 min-24 hrs	0.5	0.86-1.05
Sumatra	30 min-24 hrs	0.3	0.95
Hong Kong	5 min-24 hrs	1.0	0.91

3.e Computation with the Rainfall Ratio Method for selected return periods the rainfall for durations shorter than 24 hrs. The Rainfall Ratio method assumes that the constants in equation (2.1) apply for all durations less than 24 hours and return periods. In that case the rainfall of duration D for any return period is derived from the 24 hour rainfall extreme from

$$RR_D = \frac{D}{24}\left(\frac{b+24}{b+D}\right)^n \qquad (2.2)$$

where: RRD=rainfall ratio for conversion of 24 hr rainfall into D hr rainfall
D = rainfall duration in hrs

4. In case of single point rainfall correct the point rainfall for spatial distribution by application of an Area Reduction Factor (ARF) as a function of basin area and rainfall duration. For convective rainfall in Cambodia the following function applies:

$$\begin{aligned} &for \quad D \le 8hrs: \quad ARF = 1 - 0.04\,D^{-0.33}A^{0.50} \\ &for \quad D > 8hrs: \quad ARF = 1 - 0.02\,A^{0.50} \end{aligned} \qquad (2.3)$$

For e.g. Jakarta the curves as shown in Table 2.1 are applied.

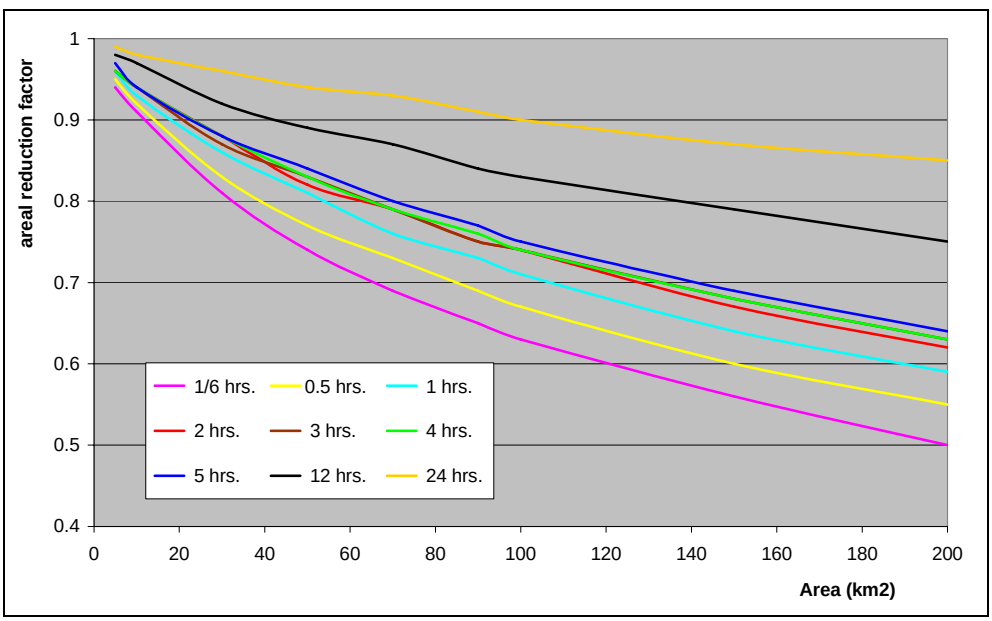

Figure 2.1 Area Reduction Factors as a function of rainfall duration and surface area for rainfall events in and around Jakarta, Indonesia (taken from: NEDECO [1973]).

5. The design rainstorms for selected return periods can be obtained by the incremental intensity of the rainfall totals of the calculated durations up to the time of concentration:

$$I_{incr} = \frac{R_{A,D_j} - R_{A,D_{j-1}}}{D_j - D_{j-1}} \left(\frac{mm}{hr}\right) \quad for: \quad D_j > D_{j-1} \qquad (2.4)$$

   where: I_{incr} = incremental intensity (mm/hr)
   $R_{A,D}$ = areal rainfall of duration D

   The incremental intensities are subsequently arranged around the peak value. The location of the peak value is determined by the storm advancement coefficient *r* which is the ratio of the time to peak relative to the total duration of the design storm T_c. The advantage of this procedure is that up to T_c all design intensities are included in the design storm. Other options are triangular hyetographs, where the occurrence of the peak value is determined by the storm advancement coefficient. When possible the arrangement is according to observed hyetographs in the region. The availability of short duration rainfall then is a necessity.
6. Repeat above step for required return periods to arrive at a set of hyetographs.

From design storm to design hydrograph

The transformation of a design rainstorm into a design hydrograph involves the following steps:

1. Determination of the net rainfall by estimation of losses, to be distributed among the ordinates of the hyetograph;
2. Development of a unit hydrograph of the basin up to the required location
3. Convoluting the net hyetograph to the design hydrograph using the unit hydrograph of step 2.

To determine the rainfall losses, e.g. use can be made of the SCS-Curve Number method, applied for wet antecedent moisture conditions. This method requires information about soils, land use and treatment of the land to arrive at a curve number. The Curve Number method is extensively described in hydrological textbooks (e.g. Viessman,1989)

To develop a unit hydrograph of the basin it is advocated that use is made of the Clark method. This method is based on the routing of a time-area diagram with a time base equal to the concentration time T_c through a linear reservoir, simulating channel storage. For the diagram isochrones are constructed representing points of equal travel time to the segment outlet, see Figure 2.2. The time area diagram fully represents the physical runoff characteristics from (mountainous) catchments. This diagram can be thought of as the outflow from the basin if only translation and no deformation take place of an instantaneous unit supply of rain. Flow paths, slopes and hydraulic roughness determine the celerity and can readily be obtained from a DEM or a topographic map. Beside the concentration time it requires the estimation of a recession coefficient *k* for routing of the time-area diagram It is noted that the output from the reservoir represents the instantaneous unit hydrograph *(IUH).* This has to be transformed into a unit hydrograph of the required interval.

The two parameters T_c and k can be obtained from observed rainfall and discharge hydrographs. The time of concentration is equal to the time interval between cessation of rainfall and the time the hydrograph has receded to its inflection point. Alternatively, it is determined from physical features of the segment as length and slope. The recession coefficient k is determined from the slope of recession part of the surface runoff hydrograph.

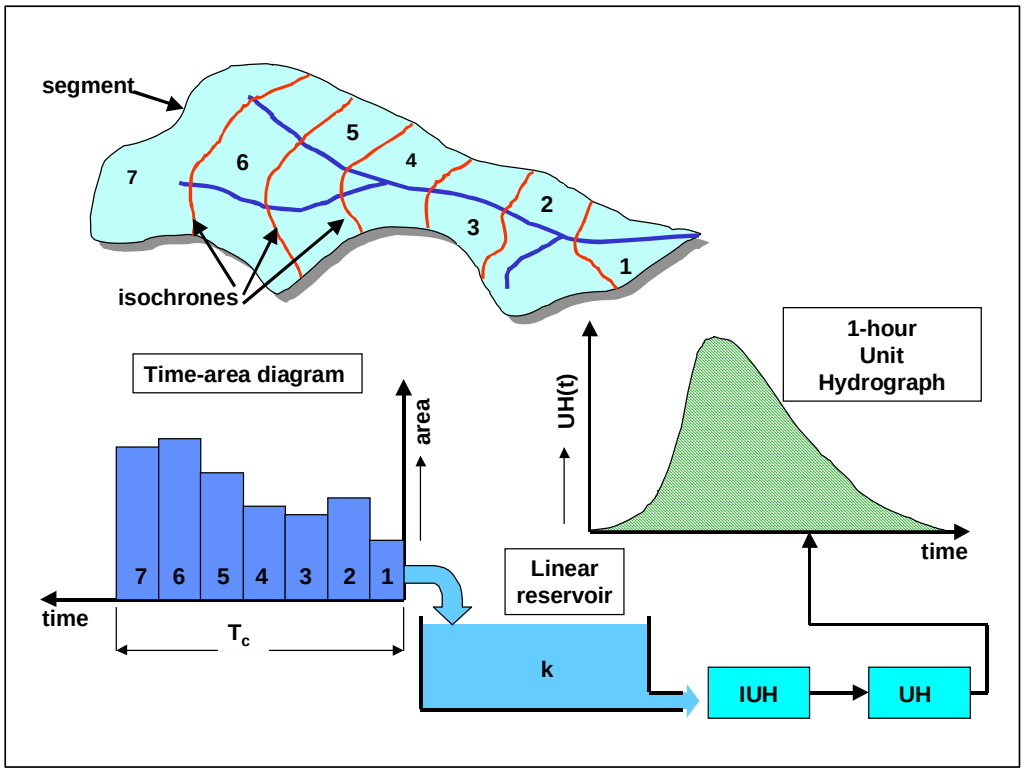

Figure 2.2 Principle of the Clark Unith Hydrograph method

When the unit hydrograph has been established, the net design rainstorm is convoluted to a design hydrograph, which comprises the hydrological hazard with a return period equal to the return period of the design rainstorm. This assumption is valid here in view of the wet antecedent moisture conditions assumed in the computation, reducing the abstractions causing differences between extremities of storms and hydrographs. By repeating this process for design rainstorms of different return periods peak discharge values are obtained for the required locations and a frequency distribution can be obtained.

From design hydrographs to design levels

By means of backwater calculations the peak discharges can be transformed into water levels of required return periods. These backwater calculations require a hydraulic model with the river schematised according to detailed bathymetric data on river and flood plain and hydraulic roughness. The calculations should include planned bank protections as well as these may back up future water levels further upstream

Application to larger areas

Above procedure is applicable to relatively small catchments, say 1,000 km^2. With some modification, this procedure can also be applied to larger areas. Then use is made of a segmented approach to the transformation of excess rainfall into runoff to allow spatial variation in the excess rainfall and runoff characteristics. In that case the basin up to the location of interest is divided into segments and the rainfall-runoff transformation is applied to each segment. The output of each segment is subsequently routed to the area of interest. This routing can be done by simple hydrologic routing methods or the hydraulic model is extended to the outlet of the uppermost segment. This approach is outlined in Figure 2.3.

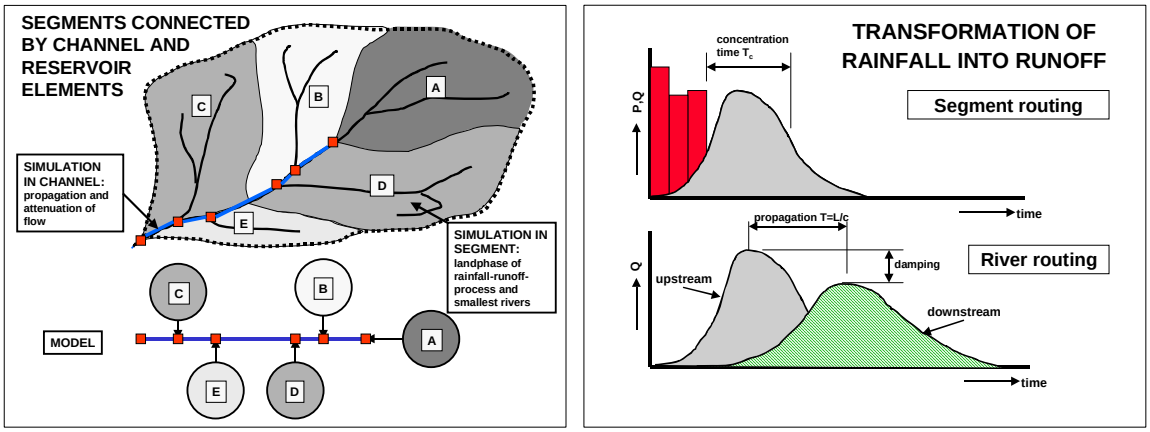

Figure 2.3 Semi-distributed runoff modelling

2.2.2 Flood hazard derived from observed flows

In case discharge series of sufficient length (≥ 15 years) is available then the procedure outlined for the hazard assessment for the city of Chiang Rai for tributary floods (see Appendix 1 to Annex 1) is also applicable:

1. creation of a homogeneous series of annual maximum flows of sufficient length
2. application of extreme value analysis to the annual maximum series to derive frequency distribution of flow extremes
3. derivation of representative dimensionless flood hydrographs from 20 largest flood hydrographs in the series
4. scaling of the representative flood hydrographs to the flood peak of selected return periods
5. routing of the hydrograph through the river section of interest to derive the associated flood levels.

Note that this procedure is only applicable for flash floods if hourly (or smaller interval) discharge series is available or can be created.

2.2.3 Flood hazard derived from regional flood statistics

Adamson (2007) proposed the use of a regional approach. This has been developed for the upper part of the LMB. It involves the following steps:

1. Creation of a regional sample of annual maximum flood peaks by pooling the individual annual maximum values scaled to their individual mean annual flood value.
2. Subsequently, a TCEV (Two Component Extreme Value) distribution is fit to the observed frequency distribution of pooled values (see e.g. Figure 2.4 valid for Upper part of LMB). The TCEV-distribution was chosen to account for the different phenomena creating the discharge extremes, monsoon and monsoon and typhoons, where the latter causes extremes far beyond the monsoon range.

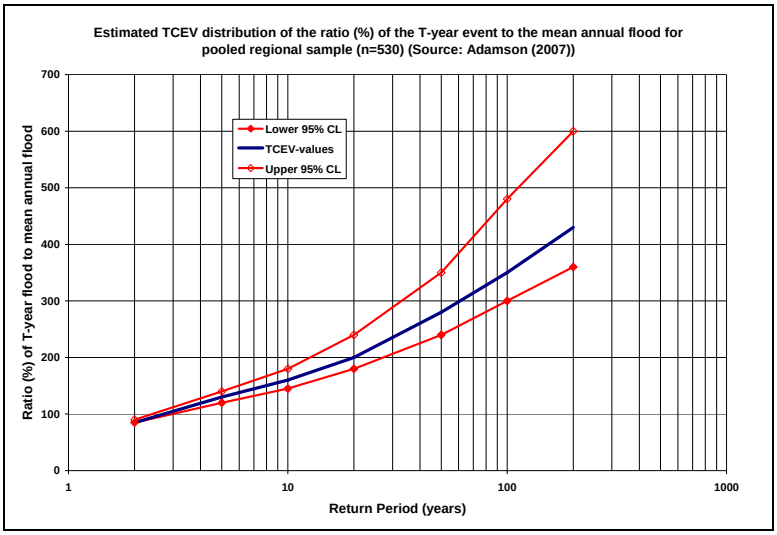

Figure 2.4 TCEV-distribution of the ratio of the T-year event to the mean annual flood for the pooled regional sample (Upper part of LMB) .

3. To apply the TCEV-values to ungauged sites a regional relationship between the mean annual maximum flood discharge and one or more climatic and/or basin characteristics has been developed. For the upper part of the LMB Adamson (2007 found the following relation with drainage area:

$$Q_{ave} = A^{0.75} + err. \qquad (2.5)$$

where: Q_{ave} = mean annual maximum flood (m^3/s)
A = drainage area (km^2)

The error about regression is considerable and it is advised to enter more characteristics in the relation.

4. It is noted that above presented relations are only valid for the upper region of the LMB. To make the procedure applicable to the Upper Se San regional values will have to be collected. Furthermore, corrections have to be made for daily average values to peak values, which in the case of flash floods may be a considerable adjustment in view of the short duration.

3 Basin description

3.1 General

In the provisional BDP list of projects for Vietnam the Project Drainage and Flood Control Kontum is mentioned. The type of floods that creates damages here are flash floods, notably the flood on 3rd November 1996 with a maximum one-day rainfall of 197.4 mm measured at Dak Gley and at Dak To Ve with 159 mm. The maximum discharge was observed at Dak Bla station Kontum with a peak discharge of 3,620 m^3/s. VNMC has proposed this area as focal area for flood risk assessment due to flash floods in Kontum Province.

3.2 Basin description

The Focal Area covers the upper basin of the Se San in Vietnam in Kontum and Gia Lai Provinces, upstream of Yali reservoir. The headwaters of the Se San are formed by the Dak Bla and Krong Po Ko rivers, which join some 15 km downstream of the city of Kontum upstream of Yali, see Figure 3.1.

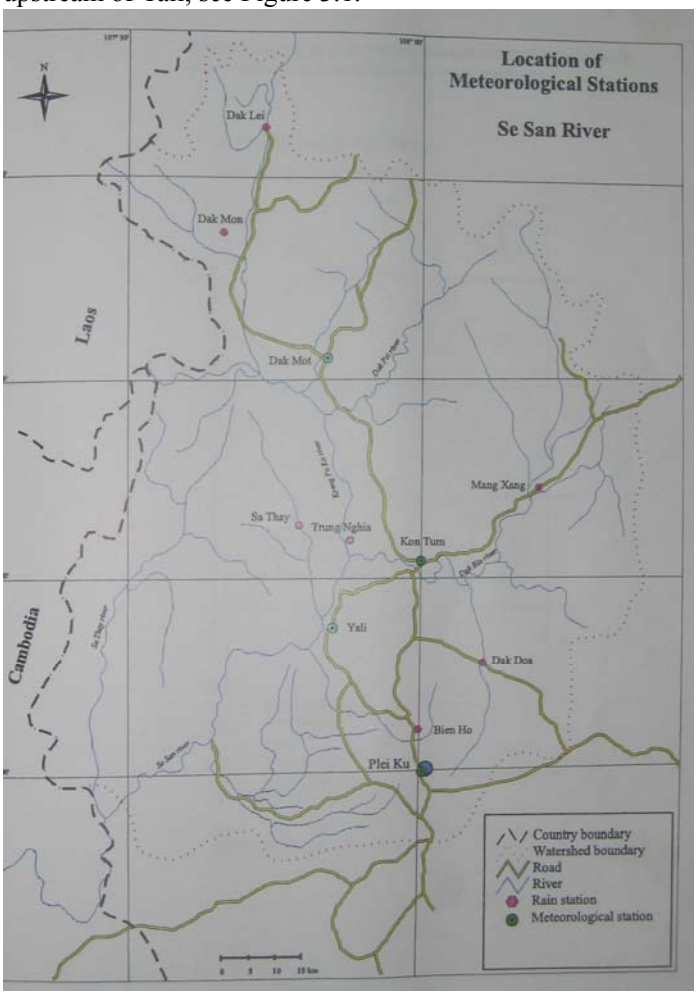

Figure 3.1 Layout of Upper Se San basin (DHI, 2006)

The Dak Bla river is 152 km long and has a basin area of 3,050 km^2 (IWRP, 2003). The river has its headwaters in the Ngoc Co Rinh range at a height of 2,025 m and runs NE-SW to join the Krong Po Ko at Sa Binh. The latter has a basin area of 3,530 km^2 and a length of 121 km.

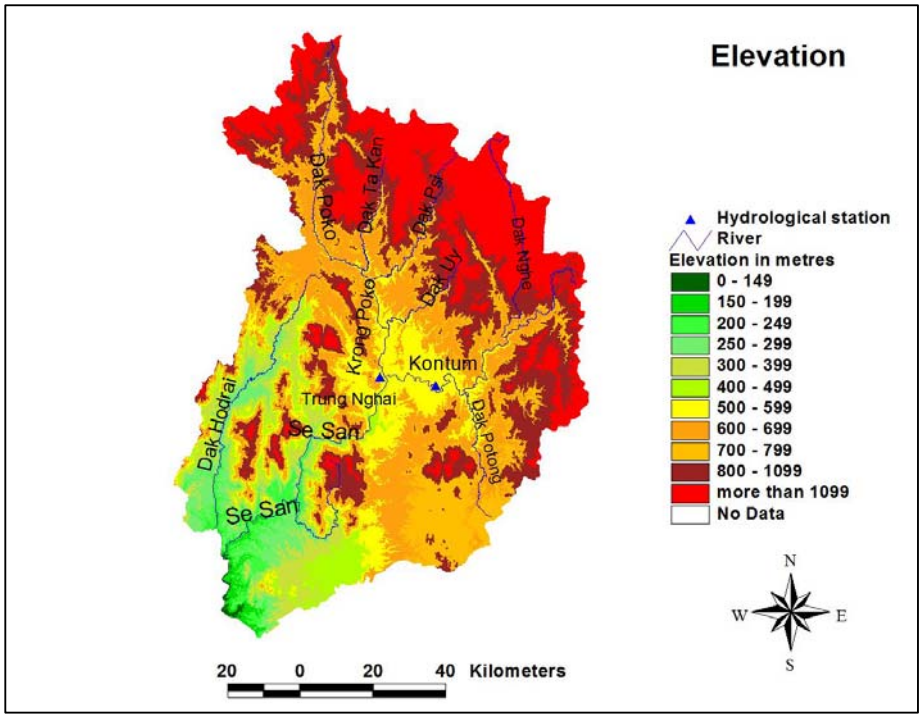

Figure 3.2 Elevation map of Upper Se San River Basin

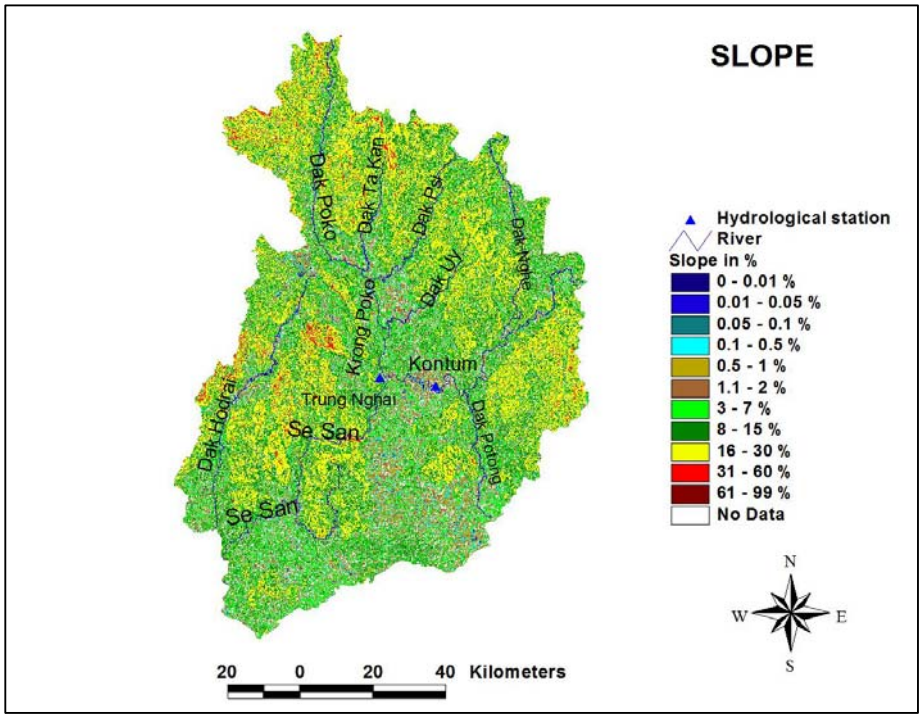

Figure 3.3 Slope map of Upper Se San River Basin

The elevation and slope maps of the Upper Se San are presented in Figure 3.2 and Figure 3.3. Elevations range from 500 m to 2,500 m in the upper reaches with slopes of 10 -30% and locally even higher, typically the conditions for having flash floods. The levels drop to about 200 m at the border with Cambodia. At locations, as e.g. around Kontum, the slopes of the land reduce to about 1%. The mountain areas are covered with forest, whereas in the lower reaches along the rivers agriculture is practised.

The regime of the Se San is affected by storages and weirs. Upstream of Yali at present the control is limited. Some regulation will be achieved with the implementation of the planned Upper Kontum hydropower dam on Dak Bla.

3.3 Problem description

The flood regimes of the Dak Bla and Krong Po Ko are different (IWRP, 2003):

- Krong Po Ko floods are governed by the climate characteristics of the west Truong Son Range with a flood season that starts in June and ends in November.
- Dak Bla is under the climatic effect of the east Truong Son Range, with a flood season that starts and ends one month later than in the Kong Po Ko.

It implies that the area around Kontum is already flooded when the big floods from Dak Bla arrive. Nonetheless, around Kontum city no real flooding problems do exist anymore as flood protection works are in place. Flooding problems are said to occur in the upper areas.

In the IWRP (2003) Report flash flood problems are mentioned for Upper Se San occurring particularly in areas where forest cover has largely been removed and flows have been obstructed by man-made interventions. The report states:

> *"Due to the fact that the watersheds have seriously destroyed and left barren, and during the dry season, droughts have caused the soil layers to become bare. As a result, these soils fail to keep moisture and are strongly eroded in the event of large rains. Moreover, the underground water penetrates into cracked and weathered soil and rock layers, causing land collapse. The streams themselves are flows of sand and mud, which wash many villages away. Flash floods often occur in high mountains and in remote areas in the Central Highlands, where most of the population are minor ethnic groups. In recent years there are more flash floods which are followed by great damages in terms of property and people."*

Implementation of early warning systems is considered as a measure to reduce flash flood damage. Pilot projects for flash flood warning systems are underway for two communes in Kontum province: Mang Ri in Tu Mo Rong District and Tu Mo Rong in the centre of the same district (CECI, 2007). Mang Ri is a southern mountainous commune 18 km from the district centre located in a heavy rainfall region of the Northern Highlands. Natural disasters, causing severe damage are particularly due to flash floods and landslides. Tu Mo Rong is surrounded by mountain ranges at an elevation of 1,100-1,200 masl with slopes of 22%. The land type in the commune is mostly red clay soils. Flash floods and landslides are often experienced here, affecting seriously the development of the commune.

3.4 Hydrological network and data availability

The extent of the hydrological data in the Upper Se San as available in the MRC database is following:

Daily rainfall:

Pleiku	(140703), data 1927-2006 with gaps
Kontum	(140704), data 1923-2006 with gaps
Dak To	(140715), data 2001-2006

Daily evaporation:

Pleiku	(140703), data 2003-2006
Kontum	(140704), data 2003-2006
Dak To	(140715), data 2003-2006

Daily average water level data:

Kontum (Dak Bla)	(440201), data 1966-69, 73, 84-06
Trung Nghia (Krong Po Ko)	(440601), data 1992-93, 95-97

Daily average discharge data:

Kontum (Dak Bla)	(440201), data 1968-71, 84-06
Trung Nghia (Krong Po Ko)	(440601), data 1992-93, 95-97

In addition, DSF rainfall and discharge data exists for the period 1985-2001. It is not known to what extent this is based on observed or simulated data. It is likely that the actual hydrological network is denser than follows from the data availability in the MRC database.

A hydraulic model based on Mike11 has been established by DHI in 2006 covering the Se San till its confluence with Sre Pok, including Dak Bla from Kontum and Krong Po Ko from Trung Nghia. The model was basically established to study the effect of dam development in Vietnam on the flow regime in Cambodia. This model is not suitable for assessing flood hazards in the Upper Se San upstream of the Dak Bla-Krong Po Ko confluence.

3.5 Hydrological characteristics

Rainfall

Average annual rainfall in the Se San varies from 1600 mm in the the western part to 2,500 mm in the upper Se San. An example of the annual variation is given for Pleiku (long-term average 2,150 mm) in Figure 3.4, which ranges between 1,200 mm and 3,200 mm. The seasonal variation can be observed from Figure 3.5. Some 90 % of the rainfall occurs between May and October, with August generally the wettest month.

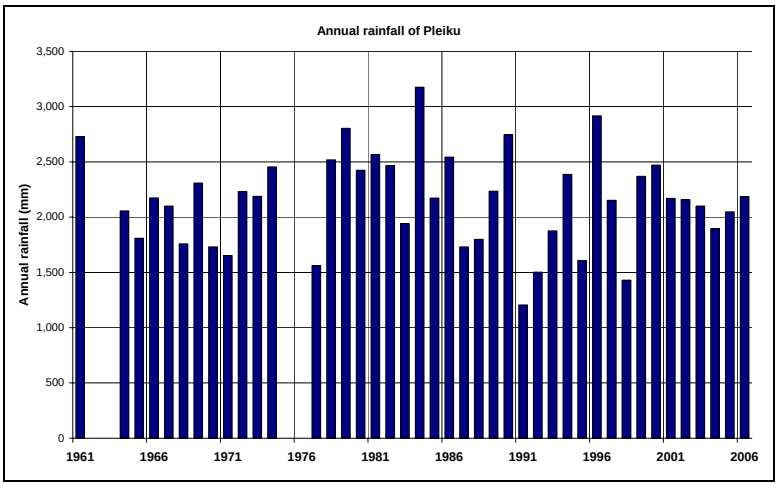

Figure 3.4 Annual rainfall of Pleiku (Vietnam)

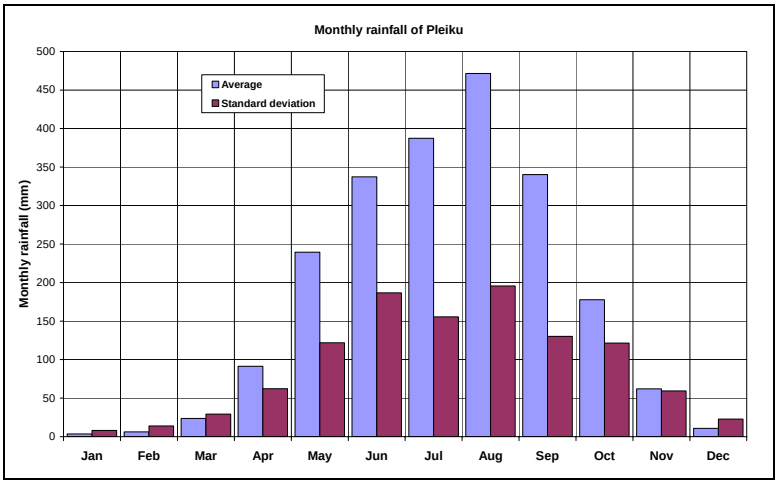

Figure 3.5 Monthly rainfall charcateristics of Pleiku

Though not particularly visible in the long-term rainfall statistics, for the occurrence of extreme floods the landfall of cyclones is of importance. For central and southern Vietnam particularly during the months September to November, those events can be expected as may be observed from Figure 3.6.

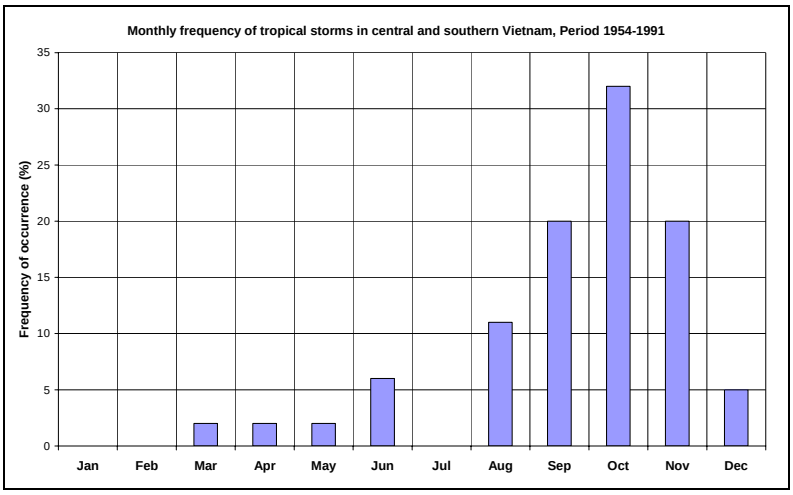

Figure 3.6 Monthly frequency of tropical storms and cyclones in central and southern Vietnam, Period 1954-1991 (ADPC, 2000)

Evaporation

Annual evaporation amounts range from 900 to 1500 mm in the Upper Se San. The values available in the MRC database are likely Piche evaporation data. The seasonal variation of the evaporation is presented in Figure 3.7. Maximum values are observed in December-April and the evaporation is minimal in June-September.

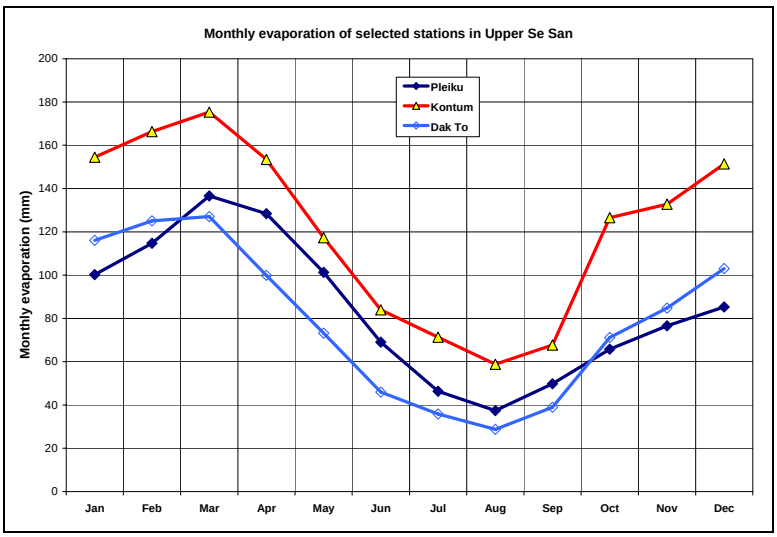

Figure 3.7 Monthly evaporation at selected stations in Upper Se San (2003-2006)

Runoff

The average monthly flow for the Upper Se San at Kontum and Trung Nghia is presented in Figure 3.8. The largest flows are occurring in August-October, i.e a relatively late response to the rainfall pattern. From an analysis of the hydrographs it appears that the base-flow component for the Se San is quite important. Note that the runoff depth from Krong Po Ko (1,200 mm) is larger than that from the Dak Bla (975 mm).

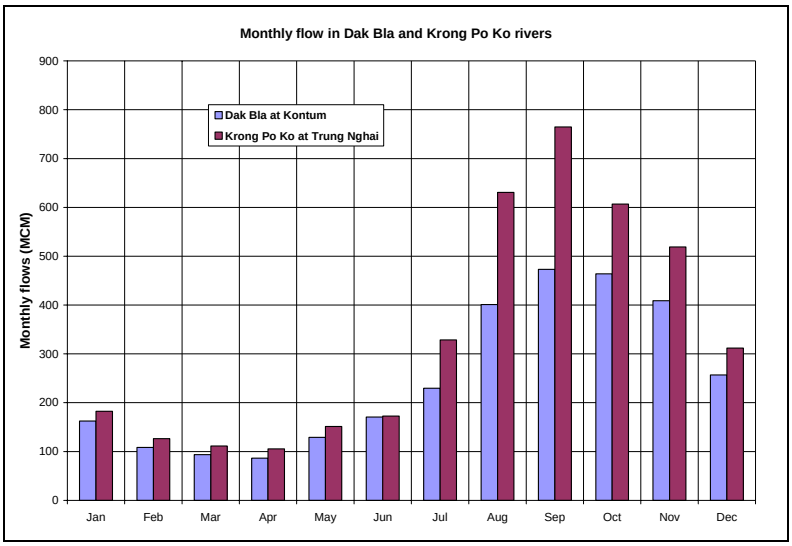

Figure 3.8 Monthly flows of Dak Bla at Kontum and Krong Po Ko at Trung Nghia (Upper Se San)

The discharge hydrographs of the Dak Bla at Kontum are shown in Figure 3.9 with the hydrograph of 1996 highlighted in Figure 3.10. The hydrograph clearly shows that floods on Dak Bla can be very flashy. Peak values from one year to another may differ by a factor 5.

The mean annual flood at Kontum is estimated at 900 m^3/s, which value has been exceeded in the period from September to November several times, see Figure 3.11. Here also the concentration of cyclones around October plays a role. This average value is substantially above the average flood peaks identified in the upper part of the LMB (compare with Adamson, 2007, Figure 23). This implies that the third method for flood hazard assessment described in Chapter 2 needs a regional adaptation.

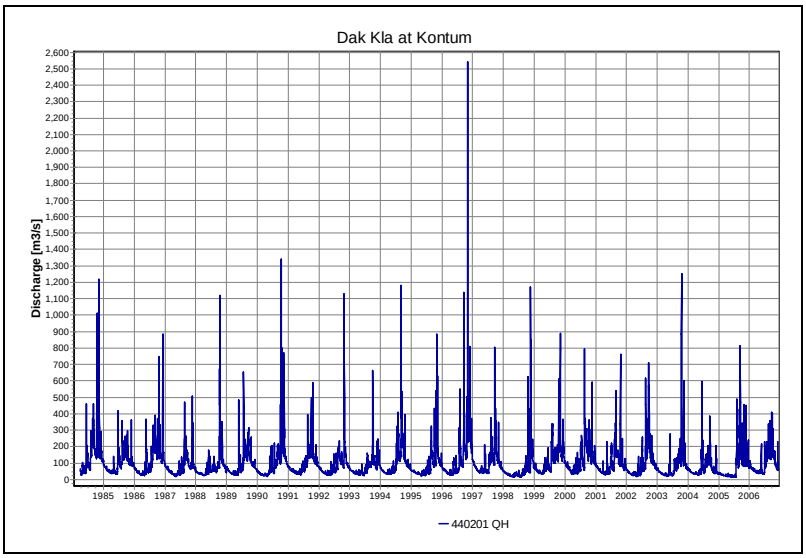

Figure 3.9 Discharge hydrograph of the Dak Bla at Kontum

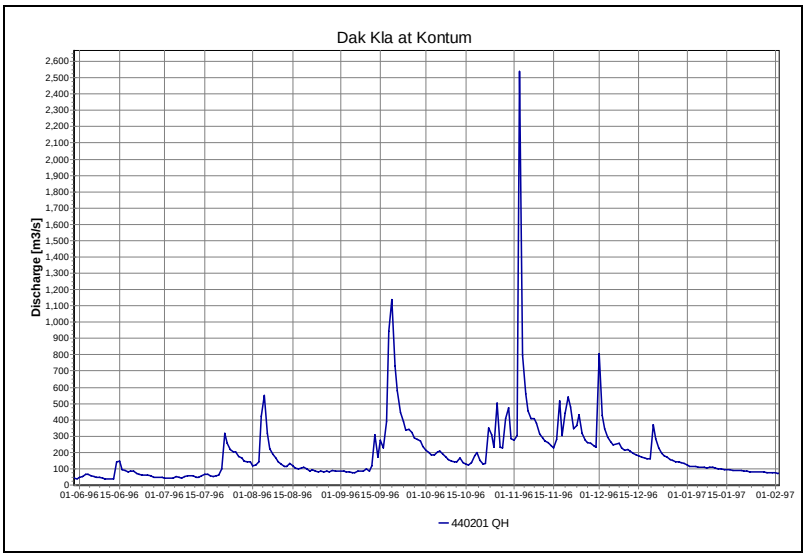

Figure 3.10 Discharge hydrograph of the Dak Bla at Kontum, year 1996

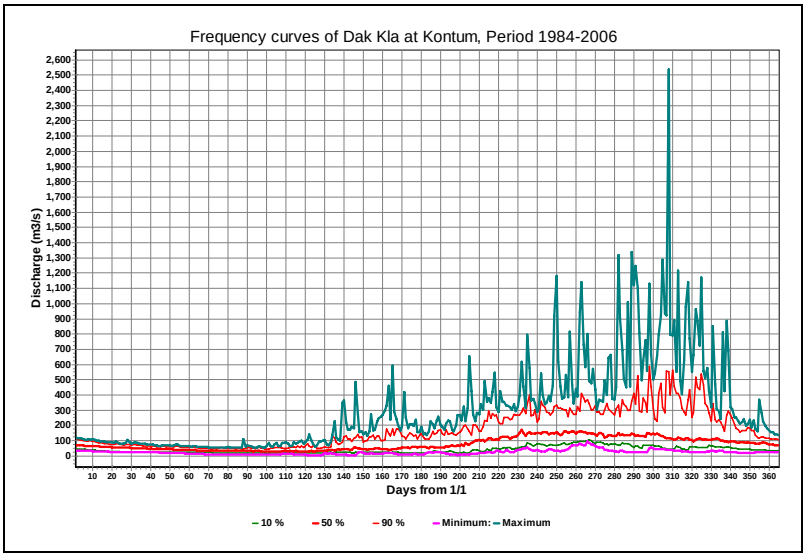

Figure 3.11 Frequency curves of daily discharge of the Dak Bla at Kontum

4 Flood hazard assessment

4.1 General

Following the flood hazard procedure proposed in Chapter 2, the hazard is assessed in three steps:

1. assessment of the meteorological hazard: development of design rainstorms
2. assessment of the hydrological hazard: transformation of design storms into design hydrographs, and
3. assessment of the flood hazard: transformation of design hydrograph into design levels.

The first step will be elaborated in the following subsections. The remaining steps first need a choice to be made on the locations. No such information is available at this stage.

4.2 Daily rainfall extremes

For the development of design storms the first step includes the derivation of annual rainfall extremes. Use has been made of the data available in the MRC database for the Upper Se San. Data here is only available from intervals of 1 day and larger. So the statistics are derived of the annual maximum rainfall. Sufficiently long series of daily rainfall is available for the stations Pleiku and Kontum, from the mid twenties onward. Though not all years were available or complete, those years were selected with observations in the months where the annual maximum values are likely to occur. In this way, series of respectively 57 and 55 years were obtained. The series are shown in Figure 4.1. The series do not show any trend. The daily extremes varied between 60 and 225 mm in Pleiku and 50 and 150 mm in Kontum. The series of Dak To is too short for a statistical analysis of extremes. The annual maximum in Pleiku is seen to have been considerably higher in the past then in Kontum (8 occasions higher then the maximum value for Kontum)

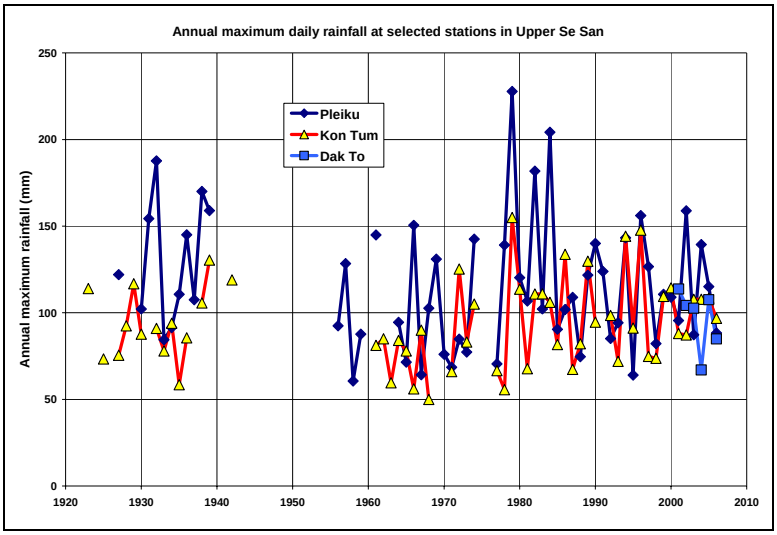

Figure 4.1 Annual maximum daily rainfall at Pleiku, Kontum and Dak To.

Extreme value distributions (EV1 = Gumbel and GEV = General Extreme Value) have been fitted to the observed distribution. The results are shown in Table 4.1and Figure 4.2 and Figure 4.3. A comparison is made in Figure 4.4.

Table 4.1 Parameters of EV1 and GEV distributions fitted to annual maximum daily rainfall and rainfall values for selected return period

Parameters/ Return period (years)	**Pleiku**		**Kontum**	
	EV1	**GEV**	**EV1**	**GEV**
k	-	0.0048	-	0.118
α	30.12	30.26	20.45	22.55
u	98.08	98.11	82.42	83.59
2	109	109	90	92
5	143	143	113	115
10	166	166	128	128
25	194	194	148	144
50	216	215	162	154
100	237	236	176	164
500	285	283	209	183
1000	306	304	224	190

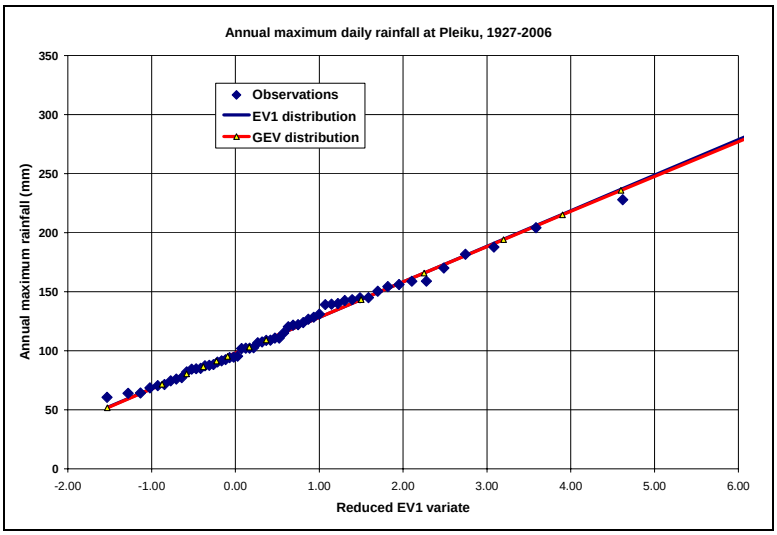

Figure 4.2 EV1 and GEV-fit to annual maximum daily rainfall at Pleiku, Period 1927-2006

The fits and the values for the insignificant values for the shape-parameter k show that for both locations the EV1 or Gumbel distribution is applicable. The daily extreme rainfall values from this distribution are therefore used in the development for the design storm.

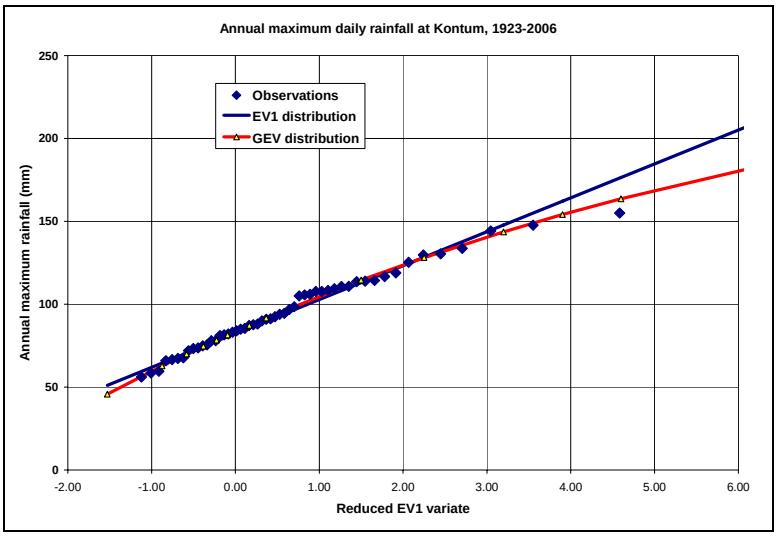

Figure 4.3 EV1 and GEV-fit to annual maximum daily rainfall at Kontum, Period 1923-2006

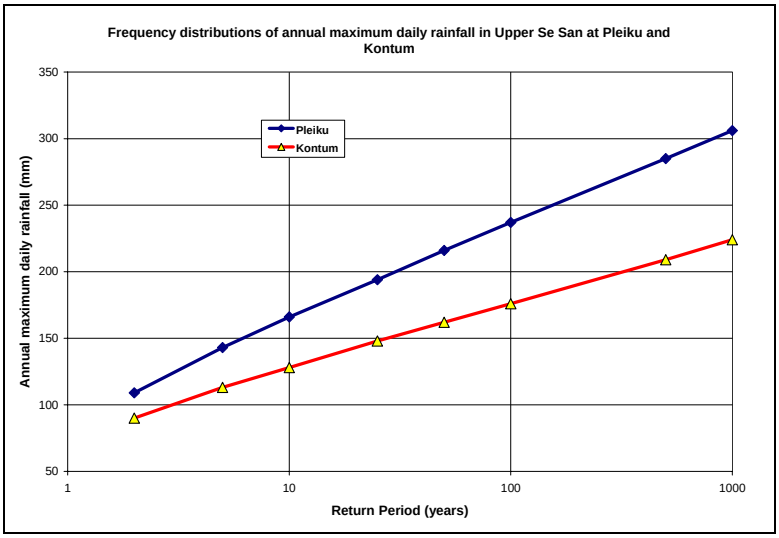

Figure 4.4 Frequency distribution of annual maximum daily rainfall at Pleiku and Kontum

4.3 Design storm

In the example for the development of a design storm the following parameters will be assumed:

- Basin area is 50 km^2;
- Rainfall intensity parameters for dis-aggregation of daily rainfall of Phnom Penh apply;
- Storm advancement coefficient r = 0.3,
- ARF for convective storms in tropical countries apply and.
- Rainfall conditions are similar to Pleiku with orographic adjustment factor of 1.25.

The results are presented in Table 4.2 and Figure 4.5. The calculation of the design storm is indicated in Table 4.3. The result is presented in Figure 4.6. It follows from a re-ordering of

the derived incremental intensities. Here a basic interval of 15 minutes has been assumed. The sensitivity of the design hydrograph for the choice of this interval is to be investigated, once the basin Unit Hydrograph is known. This procedure is also to be repeated for other selected return periods.

Table 4.2 Short duration rainfall (mm) at Pleiku derived from daily rainfall extremes

Duration		**Rainfall ratio**	**Return Period in years**					
	hrs	**RR**	**2**	**5**	**10**	**25**	**50**	**100**
5 min	0.083	0.15	18	23	27	32	35	39
10 min	0.167	0.24	29	38	44	52	58	63
15 min	0.250	0.30	37	49	56	66	73	81
30 min	0.500	0.42	52	68	79	92	102	112
1 hr	1	0.54	66	87	101	117	131	144
2 hr	2	0.65	79	104	121	141	157	172
4 hr	4	0.75	91	120	139	162	181	198
8 hr	8	0.84	103	135	157	183	204	224
12 hr	12	0.90	110	144	167	196	218	239
18 hr	18	0.96	117	153	178	208	232	254
24 hr	24	1.00	122	160	186	217	242	265
Day	**Day**		**109**	**143**	**166**	**194**	**216**	**237**

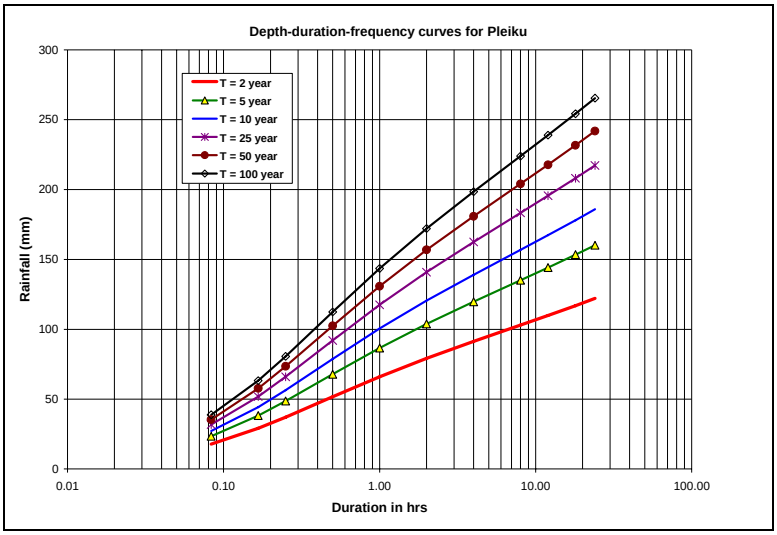

Figure 4.5 Depth-duration-frequency curves for Pleiku using Phnom Penh RR-values

Table 4.3 Computation of design storm ordinates for 15 min intervals

Time (hrs)	0.25	0.50	1.00	2.00
Point Rainfall (mm)	73	102	131	157
Correction for orography	92	128	164	196
ARF	0.55	0.64	0.72	0.78
Areal rainfall (mm)	51	82	117	152
Incremental intensity (mm/hr)	202	127	70	35
No of 15 min increments	1	1	2	4

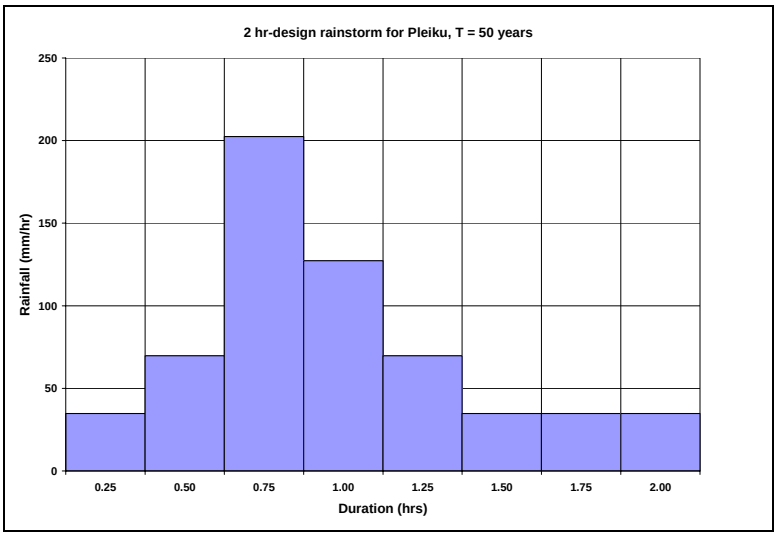

Figure 4.6 2hr-Design Rainstorm for Pleiku, Return Period = 50 years

4.4 Hydrological and flood hazard

In the development of the flood hazard the next step is the transformation of the design storm into the design hydrograph, involves first the assessment of the excess rainfall. For this the Curve Number Method can be used. Subsequently, the unit hydrograph of the basin is determined, for which we propose the use of the Clark Method. Then the excess hyetograph is convoluted by the Unit Hydrograph to arrive at the design floods for the selected return periods. Based on these results a frequency distribution of the floods resulting from the selected rainfall duration (equal to the time of concentration) is prepared.

It is to be verified whether storms of shorter or longer duration lead to higher peaks. This will be dependent on the way the losses are distributed over the rainfall ordinates.

The frequency distribution of flood peaks can be transformed to design flood levels by means of backwater calculations. If storages upstream play a role, then also the full flood hydrograph should be routed through the hydraulic network, with realistic initial values for the storages.

5 Conclusions and recommendations

5.1 Conclusions

Based on the analyses discussed in the previous chapters the following conclusions can be drawn:

1. Floods in the upper reaches of the Se San are very flashy, creating considerable damage. The areas are increasingly sensitive to flash floods due to deforestation.
2. Around Kontum city no flooding problems exist anymore. Problems are encountered upstream of the city on Dak Bla and in the upper reaches of Krong Po Ko and tributaries.
3. Three methods have been identified to assess the flash flood hazard:
   3.a Flood hazard derived from rainfall extremes
   3.b Flood hazard derived from observed flows
   3.c Flood hazard derived from regional statistics
4. In the absence of flow series the flood hazard assessment procedure starting off from rainfall extremes is promising, as it makes use of the physical characteristics of the basin and easily accessible meteorological data.
5. Short duration rainfall statistics were not available for the Upper Se San. Annual maximum daily totals have been used instead. Their statistics were transformed to short duration statistics based on findings elsewhere. Local calibration of the intensity duration equation is needed.
6. Available series of observed flows are generally refer to daily average values. A considerable correction will be needed to adjust this to instantaneous values.
7. The flood hazard derived from regional statistics of the Upper LMB is not applicable in the Upper Se San. Values in Upper Se San seem to be higher. To make the procedure applicable to the Upper Se San regional values will have to be collected, with corrections for daily average values to peak values.

5.2 Recommendations

The application of the flood hazard derived from rainfall is recommended to be applied in the Upper Se San. Focal Areas have to be selected to test the method. It is recommended to use pilot areas where verifiable flood levels are available. A review of short duration rainfall relative to daily rainfall statistics for the Se San region is advocated with due attention to orographic effects and effects of orientation of the mountain ranges.

A regional analysis of peak discharges as doe for the Upper se San (Adamson, 2007) is recommended to introduce an alternative procedure for flood hazard assessment.

6 Literature

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