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Hydrological investigation of a failure of the Huai Sai Kamin dam, Sakon Nakhon province, Northeastern Thailand

Peangta Satarugsa[1,*], Natthawiroj Silaratana[1], Kiattisak Sonpirom[1] and Vichai Sriboonlue[2]

[1]Department of Geotechnology, Faculty of Technology, Khon Kaen University, Khon Kaen, Thailand
[2]Department of Agricultural Engineering, Faculty of Engineering, Khon Kaen University, Khon Kaen, Thailand
[*]Corresponding author: peangta@kku.ac.th

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Abstract

The present study aimed to analyze hydrological data relating to the failure incident of the Huai Sai Kamin dam, occurring on 28 July 2017 following continuous heavy rainfall from the Sonca tropical depression. Structural and hydro-meteorological data related to the dam failure incident were assembled. From these data the incident rainfall return periods and excess rainfalls were estimated using flood flow simulation. Unit hydrographs and the channel routing procedures were conducted in order to derive inflow and outflow hydrographs. Rainfall analysis showed that a daily rainfall rate at 410 mm generated during the depression was equal to a 52-year return period. The reservoir routing indicated the amount of excess rainfall water in the reservoir and above the spillway to be 0.6 million cubic m, while the water contained in the reservoir was 3.0 million cubic m when an overtopping of the dam crest occurred. At the overtopping incident, the spillway outflow rate was 25.24 m^3/s. This rate was three-time lower than the original maximum design rate of 85 m^3/s. Although the outflow rate from the spillway was at its original maximum rate, the dam failure would still have occurred 6 or 7 h later. The original outflow rate at the spillway was slower, likely due to the spillway blockade by large amounts of water weed and debris. The diminished spillway capacity and the large amounts of water weed reflected inadequate dam design, construction and maintenance.

Keywords: Dam failure, Overtopping, Spillway, Hydrological investigation

1. Introduction

The Huai Sai Kamin (HSK) is an earth-filled dam, situated in Sakon Nakhon province, the Northeastern part of Thailand (Figure 1). It was built for irrigation purposes with a water storage capacity of 2.4 million cubic m. The dam embankment rests on Quaternary alluvial soils created by the erosion of sandstone, siltstone and mudstone from the Phu Phan Mountains. The dam embankment was constructed by the compaction of nearby natural soils, composed mainly of sand, silt and clay [2]. Based on the Unified Soil Classification System (USCS), the soil used for constructing the HSK embankment is clayey sand (SC) [2]. The catchment area of the dam is around 50 km^2. Within the HSK catchment area are four additional earth-filled dams, named Huai Nam Bo (HNB), Huai Sai Ton Bon 1 (HSTB1), Huai Sai Ton Bon 2 (HSTB2), and Huai Sai Ton Bon 3 (HSTB3) (Figure 1). Table 1 provides a summary of structural data of the HSK and these four dams.

On the morning of 28 July 2017, the HSK suddenly failed to hold water, and consequently raised the flood level in its downstream area i.e. Sakon Nakhon town. The continuous heavy rainfall generated by the tropical depression, Sonca, resulted in water overtopping of the dam crest. This water overtopping caused the dam embankment to breach at two locations. One location was close to the left end of the dam embankment with a size of 20 m wide, and 4 m deep. The other location was at the left main channel outlet with the size of 50 m wide, and 8-9 m deep. A spillway was partially broken [2].

Several questions have been raised following the failure incident at the HSK dam. The questions raised concerned the adequacy of spillway design and/or maintenance, other elements that may have had facilitated the embankment to breach at the aforementioned two zones and the assurance of a proper rehabilitation of the HSK. The fact that other nearby dams did not break although they were also hit by the same amount of rainfall is of note. The failure incident of the HSK has furthermore prompted the dam safety awareness among the people who live in the areas downstream of several other dams. Consequently, the people requested dam owners to provide them the dam safety guarantee that should prevent another disaster similar to the HSK. Thus, the main objective of this study was to analyze hydro-meteorological that may delineate some hydrological factors contributing to the HSK failure incident. These hydrological accounts of the HSK failure could serve as a useful data source for a proper and suitable design of the HSK dam rehabilitation and safety restoration.

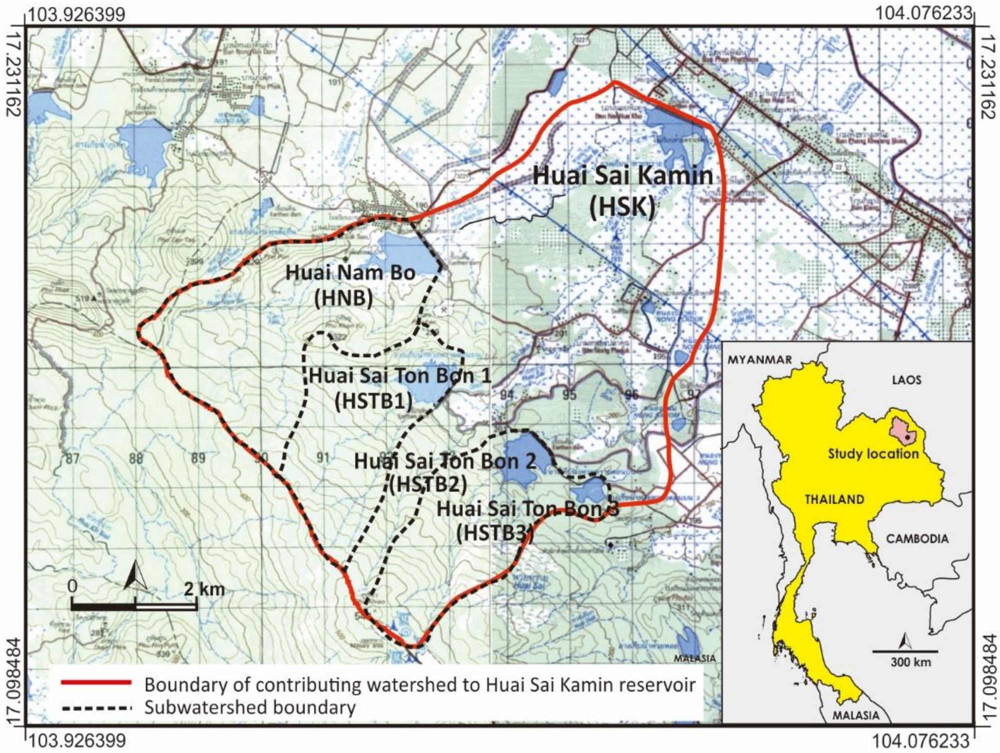


Figure 1 Topographic map showing the locations and catchment areas of the Huai Sai Kamin dam and the four dams [1]. An insert at the bottom right corner is an index map showing Thailand and study location.

Table 1 Structural data of the dams [3-4].

Properties	HSK	HNB	HSTB1	HSTB2	HSTB3
Years built	1953-1956	1953-1956	1987-1988	1986	1985
Catchment area	50 km^2	10 km^2	6.2 km^2	5.8 km^2	0.6 km^2
Water storage area	1.44 km^2	1.37 km^2	0.6 km^2	0.51 km^2	0.21 km^2
Storage capacity	2.4 x 10^6 m^3	2.25 x10^6 m^3	2.1 x 10^6 m^3	2.1 x 10^6 m^3	0.21 x 10^6 m^3
Embankment height	8.3 m	17 m	13.5 m	12.9 m	8.0 m
Embankment length	1,300 m	950 m	575 m	690 m	290 m
Crest width	5 m	6 m	6 m	6 m	5 m
Base width	56.5 m	74 m	90 m	80 m	70 m
Dam crest	155.0 m*	173.5 m*	207.8 m	209.8 m	213.6 m
Storage design level	154.0 m*	171.5 m*	206.0 m	208.0 m	212.5 m
Spillway crest	154.2 m*	171.8 m*	206.8 m	208.8 m	212.8 m
Conduit level	151.2 m*	167.0 m*	200.0 m	202.0 m	209.0 m
Diameter of conduit; capacity	0.6 m x 2; 0.6 m^3/s	0.6 m; 0.3 m^3/s	0.5 m; 0.5 m^3/s	0.5 m; 0.5 m^3/s	0.3 m; 0.15 m^3/s
Spillway; capacity	Box; 85 m^3/s	Morning groly; 15 m^3/s	2 Boxes; size 2 x 2.5 m	2 Boxes; size 2.5 x 2.5 m	Conduit; diameter 1.2 m

*Elevation is different from the topographic map shown in Figure 1.

2. Materials and methods

Hydro-meteorological data were gathered together with maps, the original dam design and construction data. The dam site reconnaissance was undertaken as was the interview of local witnesses concerning the timeline leading to the incident. The hydro-meteorological data were subjected to the following analysis.

2.1 Incident rainfall return periods

The tropical depression, Sonca, was the primary cause of the failure of the HSK dam on 28 July 2017. The hourly rainfall pattern during the critical three days presented in Figure 2 showed a 1-h peak of 57.5 mm on 28 July, 04:00-05:00 and a 24-h peak of 410 mm from 19:00 of 27 July to 19:00 of 28 July. We estimated return periods of the incident rainfall for several rainfall durations to assess the heaviness of the rainfall event.

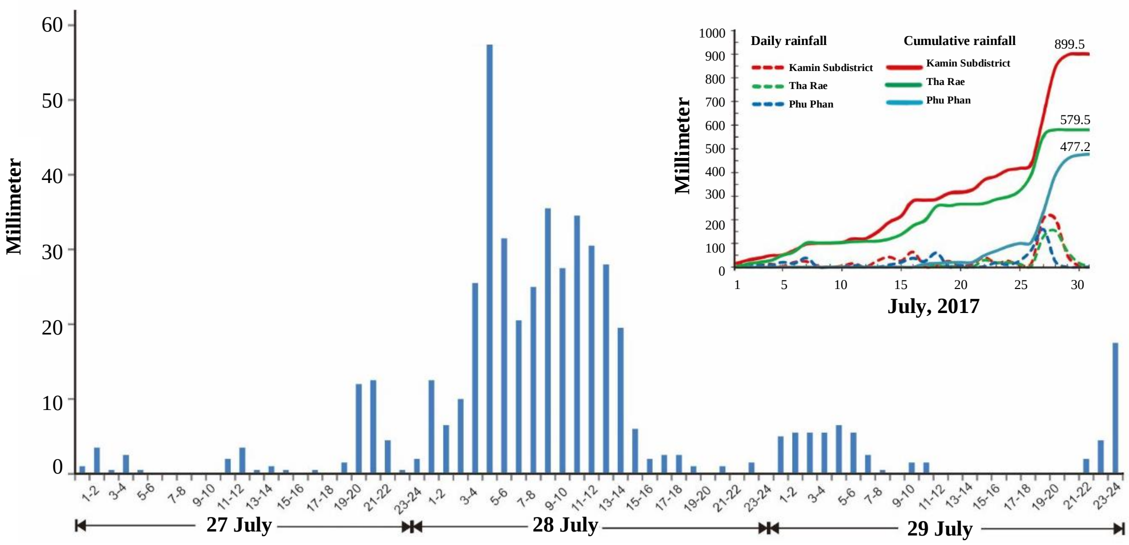


Figure 2 The 72-h rainfall from 00:00 of 27 July 2017 to 24:00 of 29 July 2017 from the Kamin subdistrict rain gauge station, the maximum amount of 24-h continuous rainfall from 19.00 of 27 July to 19.00 of 28 July is 410 mm, the maximum amount of 12-h continuous rainfall from 02:00 to 14:00 of 28 July is 345.5 mm, an insert graph shows daily and cumulative rainfalls measured from nearby three rain gauge stations, the Tha Rae rain gauge located about 8 km downstream, and the Phu Phan rain gauge located about 20 km upstream the HSK reservoir [5].

The annual maximum rainfall values from 1990 to 2015 for the Sakon Nakhon weather station were extracted from the Meteorology Department records.. The maximum values for 1, 2, 3, 6, 12, and 24 h rainfall durations were subjected to statistical analyses using the 2-parameter extreme value type I (Gumbel) distribution procedure. The Gumbel procedure is based on an extreme value theory suitable for the data set of less than 30 years [6]. The two parameters are the local parameter, α, and the scale parameter, β, which can be evaluated from the average and standard deviation values of the annual maximum data from Chow *et al.* (1988) [7] as:

$$\beta = \sqrt{6}\ \frac{SD}{\pi} \tag{1}$$

$$\alpha = \bar{x} - 0.5772\beta \tag{2}$$

where $\bar{x}$ is an average value, and SD is the standard deviation. Return periods of the incident rainfall that cause the dam breaching can be calculated from Chow *et al.* (1988) [7] as:

$$T = 1/(1 - e^{(-e^{\left(-\frac{I-\alpha}{\beta}\right)})}) \tag{3}$$

where T is a return period and I is rainfall intensity.

2.2 Flood flow simulation

The inflow into HSK reservoir was the actual cause of dam crest overtopping and subsequent failure. The inflow simulation was thus crucial to an analysis of the dam failure. The inflow into the HSK reservoir came from its watershed. The watershed of HSK was divided into two parts, the upstream and the downstream. The upstream part was hilly with a fairly steep slope and a bushy landscape. The lower part was flat with its majority being the paddy field. There were four reservoirs located at the boundary between these two parts, namely HNB, HSTB1, HSTB2, and HSTB3 listed from northwest to southeast (Figure 1). Three sub-watersheds were delineated for HNB, HSTB1 together with HSTB2 plus HSTB3. As the sub-watershed of HSTB3 was small, it was merged with the HSTB2. The whole watershed was, therefore, separated into 4 sub-watersheds including the HSK sub-watershed itself (Figure 1). Since there were four sub-watersheds with three of them situating in the upstream part, the flow simulations of the three upper sub-watersheds were first carried out, followed by the HSK sub-watershed.

The flood flow simulation started with the incident rainfall (Figure 2) which could be separated into two parts, lost and excess rainfall. The excess rainfall was evaluated and transformed into stream flow value that flows into each individual corresponding reservoir. The inflows into the upstream reservoirs were routed through the reservoirs, while the outflows were routed through the corresponding channels down to the HSK reservoir. The total inflow to the HSK reservoir was composed of the flows from HNB, HSTB1, and HSTB2 plus HSTB3 along with their courses and the outflow from the Huai Sai Khamin sub-watershed itself. This inflow was routed through the HSK reservoir to become the outflow of the reservoir, which was the key factor causing the dam to breach.

2.3 Excess rainfall estimation

The excess rainfall pattern was estimated from the total rainfall pattern (Figure 2) by the Curve Number method, originated from the Soil Conservation Service, US Department of Agriculture, known also as the SCS-CN method. This method requires one parameter, curve number (CN), that depends on the soil, land use, and antecedent moisture content of the corresponding watershed [8]. Three categories of CN involved in the calculation depend on the moisture of the watershed; CN1, CN2, and CN3 are for antecedent dry, moderate, and wet watershed, respectively. The CN2 values of a watershed can be evaluated from its soil and land use, usually the reading from the prepared table available [9]. By considering the antecedent moisture condition of the watershed, CN2 are converted to CN1 or CN3 using Hawkins et al. (2009) [9] as:

$$CN1 = \frac{4.2CN2}{10-0.058\,CN2} \tag{4}$$

$$CN3 = \frac{23CN2}{10+0.13\,CN2} \tag{5}$$

In the SCS-CN method, hydrological soils are classified into four types, namely A, B, C, and D. The ranking is from high seepage loss soils from A type to nearly impervious soils for D. The derived spatial mean CN values are dimensionless and vary between 0 and 100. The CN values is converted to potential storage capacity of watershed in millimetres, *S*, by Hawkins *et al.* (2009) [9] as shown in Equation 6 (Eq.6). Then the excess rainfall, *R*, can be calculated from total rainfall, *P*, as Eq.7:

$$S = \frac{25400}{CN} - 254 \tag{6}$$

$$R = \frac{(P-0.2\,S)^2}{P+0.8\,S} \tag{7}$$

The total rainfall, *P*, to be used in the Eq.7, must be a set of cumulative rainfall; if the *P* of the first step is less than *0.2S*, then *R* equals to 0. This means that if the initial rainfall abstraction by the watershed is higher than the initial rainfall then there is no runoff at the initial time step.

The CN2 value for each of the sub-watersheds was derived from prepared tables as illustrated by Hawkins *et al.* (2009) [9]. The CN2 value for upstream sub-watersheds was 77 as the area had poor cover woodland with shallow soil of type C. The CN2 value for sub-watershed of HSK was 78, given that it covered most of the downstream area and about one-fifth of the upstream area, therefore cultivating land with conservation and the C soil type. Due to highly wet antecedent conditions, CN3 was chosen to be used in calculating each sub-watershed. By using Eq.5, the CN3 value for each of the three upstream sub-watersheds was 88, and 89 for the HSK sub-watershed. The potential storage capacities, *S*, evaluated from Eq.6 was 34.6 mm for the one upstream and 31.4 mm for the downstream. The values of excess rainfall, *R*, were then calculated from Eq.7. Considering the incident rainfall pattern, shown in Figure 2, we decided to simulate the stream flows using rainfall data from 00:00 to 15:00 of 28 July 2017.

2.4 Unit hydrograph evaluations

A 1-h unit hydrograph was synthesized from the information on topography and shape of each of the four sub-watersheds, using the following two equations [8 & 10] as:

$$t_l = C_t\left(\frac{LL_c}{\sqrt{s}}\right)^a \tag{8}$$

where C_t and a are local climatic parameters, t_l is lag time, s is mean slope of the main channel, L is length of the main channel, while L_c is the length along the main channel from the nearest point to the centroid of the sub-watershed to the outlet, and

$$u_p = C_pAt_p^b \tag{9}$$

where C_p and b are local climatic parameters, u_p is peak flow, A is an area of sub-watershed, and t_p is time to peak.

The values of climatic parameters of the whole HSK watershed are C_t=2.262, a=0.242, C_p=2.746, and b=-1.104, which belong to the parameters of Nam Songkram basin [10]. Eq.8 was used to estimate the lag time of each of the sub-watersheds from their values of L, L_c, and S (Table 2) and parameters C_t and a. Time to peak, t_p, in Eq.9, is the lag time, t_l, plus a half of duration (D) of unit hydrograph. Since we developed unit hydrographs of 1-h duration, therefore, $t_p = t_l+0.5$ (h). From sub-watershed areas, A, as in Table 2, including the values of C_p and b, we estimated peak flows, u_p. A complete unit hydrograph can be derived from u_p and t_p by Akan and Houghtalen (2003) [11] as:

$$u = u_p\left[\frac{t}{t_p}e^{\left(1-\frac{t}{t_p}\right)}\right]^{n-1} \tag{10}$$

where u is a set of discharges of a unit hydrograph corresponding to a time series, t, and n is a shape parameter relating to C_p by Meadows and Blandford (1983) [12] as:

$$n = 1.0685+0.1175\,C_p+0.782\,C_p^2 \tag{11}$$

The values of t_l, t_p, u_p, and n are shown in Table 2 for HNB, HSTB1, HSTB2 plus HSTB3, and HSK sub-watersheds. The parameters in Table 2 were used to estimate a 1-h unit hydrograph for each of the sub-watersheds. We allowed 24 time steps for each unit hydrograph.

Table 2 The physical and unit hydrographic parameters of the HSK sub-watersheds.

Parameters	Sub-watersheds			
	HNB	HSTB1	HSTB2+3*	HSK
A (km^2)	10.13	6.25	6.40	26.25
L (km)	6.88	4.10	4.46	11.29
L_c (km)	2.73	1.89	1.73	4.31
s	0.0352	0.0701	0.0748	0.0299
t_l (h)	6.90	5.12	5.08	8.85
t_p (h)	7.40	5.62	5.58	9.35
u_p (m^3/s)	3.055	2.552	2.635	6.165
n	7.288	7.288	7.288	7.288

*HSTB2+3 is HSTB2 plus HSTB3.

2.5 Inflow and outflow hydrographs

A graph of streamflow that discharges into a reservoir is called an inflow hydrograph, while one that flows out from the reservoir is an outflow hydrograph. An inflow hydrograph was the result of adding base flow to direct runoff hydrograph. A direct runoff calculation was done by summing the product of excess rainfall with unit hydrograph ordinates as in the following equation by Akan and Houghtalen (2003) [11] as:

$$Q_{d,n} = \sum_{m=1}^{n} R_m u_{n-m+1} \tag{12}$$

where $Q_{d,n}$ is direct runoff ordinate at n^{th} time step, R_m is excess rainfall at m time step, u_{n-m+1} is unit hydrograph ordinate at n-m+1 time step.

After considering the incident rainfall pattern shown in Figure 2, we chose a rainfall period from 00:00 to 15:00 of 28 July 2017 which were 15 time steps for R and 24 time steps for u (Figure 3A). To obtain inflow hydrographs to the corresponding reservoirs, the direct runoff hydrographs for all sub-watersheds were calculated with Eq.12, and the base flows of 5 m^3/s and 10 m^3/s were added to the ordinates of the upstream and downstream sub-watersheds, respectively.

The inflow hydrographs for the 3 upstream reservoirs were routed through the corresponding reservoirs to yield the outflow hydrographs. This procedure is called reservoir routing [11]. There are two types of reservoir routing, i.e. hydraulic routing and hydrologic routing. We chose the hydrologic routing due to its simplicity and accuracy. The hydrologic routing makes use of the water balance concept and reservoir characteristics. The water balance concept gives the relationships between inflow, outflow, and water storage at two time steps 1 and 2 as:

$$G_2 = I_1 + I_2 - 2Q_1 + G_1 \tag{13}$$

where I and Q are inflow and outflow, respectively, and G can be expressed as:

$$G = \frac{2S}{\Delta t} + Q \tag{14}$$

where S is water volume stored above spillway crest, Δt is length of time step.

Based on the spillway design and the S-Q relationship in Eq. 13, we obtained two reservoir features, namely the relationships between height (H) with water storage (S), above spillway crest, and between height (H) with outflow (Q) over spillway crest. Figure 3A shows the H-S relationship, while Figure 3B shows the H-Q relationship of all reservoirs. The inflow hydrograph to each upstream reservoir was routed through the correspondent reservoir using Eqs. 13 and 14 to generate the outflow hydrograph of each reservoir.

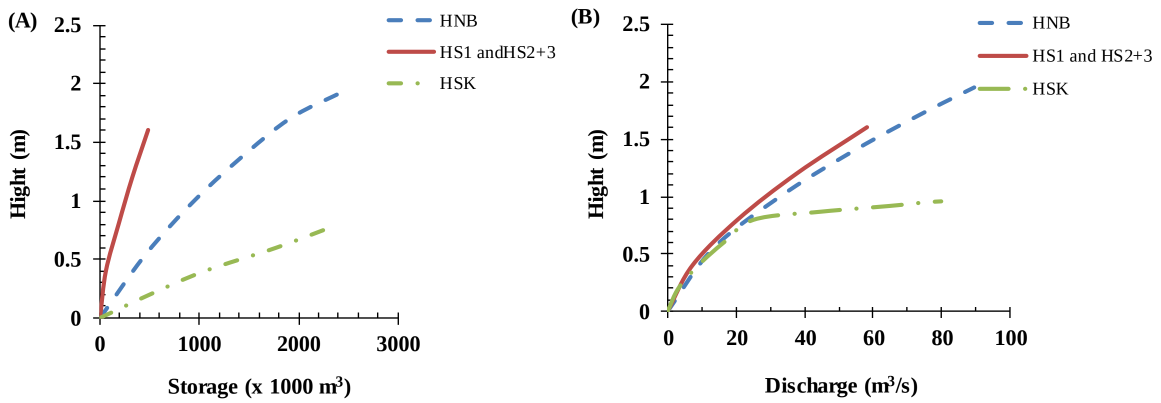


Figure 3 (A) The relationship between the height and storage above the spillway crest, (B) the relationship between the height of the water surface above the spillway crest and the outflow discharges of the four reservoirs.

2.6 Channel routing procedure

The outflows from individual upstream reservoirs flowed along its water course down to the HSK reservoir. These outflows were routed through their water courses to the downstream reservoir by a channel routing method. By following the reservoir routing, we also used hydrologic routing, Muskingum method, instead of the hydraulic method [10]. The channel routing equation can be expressed for inflows (I) and outflows (Q) at two time steps (1 and 2) by Akan and Houghtalen (2003) [11] as:

$$Q_2 = C_1 I_1 + C_2 I_2 + C_3 Q_1 \quad (15)$$

where C_1, C_2 and C_3 are functions of the channel characteristics and the length of time step as,

$$C_1 = \frac{KX + 0.5\,\Delta t}{D}\ ;\ C_2 = \frac{-KX + 0.5\,\Delta t}{D}\ ;\ C_3 = \frac{K - KX - 0.5\,\Delta t}{D}\ ; D = K - KX + 0.5\,\Delta t \quad (16)$$

where Δt is the length of time step which was 1 h for this simulation, K and X are the channel flow parameters. K is approximately equal to the time it took for the flood wave to move from the upper bound to the lower bound stream section. It can be approximated from,

$$K = \frac{L}{c} \quad (17)$$

where L is the length of the stream section, and c is the celerity of the flood wave, and the unit of K is the same as Δt.

The X value for natural rivers is normally in the range of 0.2 to 0.3, however we used the formula to estimate X by Cunge (1969) [13] as:

$$X = 0.5\left[1 - \frac{Q_0}{B_0 c s L}\right] \quad (18)$$

where Q_0 is the referent discharge, B_0 is referent width of stream, c is celerity, s is mean stream slope, and L is length of the stream section.

The values of Q_0, B_0, s, and L for the three stream sections between HNB-HSK, HSTB1-HSK, and HSTB2 plus HSTB3-HSK were approximated from their geomorphic parameters and are shown in Table 3. By assuming c=2 m/s, the K and X values were estimated from Eqs. 16 and 17, also shown in Table 4. Once the channel routing parameters, K and X, were known and Δt was assigned to be 1 hr, then C_1, C_2, C_3, and D were determined from Eq. 16 and presented in Table 4.

Table 3 Channel flow parameters for the stream sections.

Section	L (km)	K (h)	Q_0 (m^3/s)	B_0 (m)	s	X
HNB-HSK	5.25	0.73	45	15	0.002857	0.45
HSTB1-HSK	7.875	1.09	40	13	0.00254	0.46
HSTB2 plus HSTB3-HSK	7.35	1.02	40	13	0.003129	0.47

Table 4 Channel routing parameters for the three stream sections.

Section	D	C_1	C_2	C_3
HNB-HSK	0.901	0.919	0.191	-0.110
HSTB1-HSK	1.089	0.923	-0.004	0.082
HSTB2 plus HSTB3-HSK	1.045	0.935	0.023	0.43

Each outflow from the three upstream reservoirs became the inflow to the corresponding downstream channel section. The inflows of the three channel sections were routed using the Muskingum channel routing method, Eq. 15, to the HSK reservoir. The inflow into HSK from the three channels together combined with the HSK sub-watershed itself created the total inflow of HSK. We simulated the outflow of HSK as if there were no failure of the embankment using reservoir routing procedure Eqs. 13 and 14 including HSK reservoir characteristics in Figure 3.

3. Results and discussion

Table 5 is a timeline of the HSK dam failure. Assumptions, interpretations and time constraints for our hydrological analyses were based on the evidence summarized in Table 5.

Table 5 Timeline of the HSK dam failure.

Date	Events
18 July 2017	Water in the dam was 105% above its maximum capacity, i.e., the water level was 100 centimeters below the dam crest [14]
27 July 2017	Overflow at the spillway, i.e., the water level was approximately 70 to 80 centimeters below the dam crest [15]
At approximately 08:00 and before 10:00 of 28 July 2017	Water flew over the top of the dam and the dam was breached [16]
At approximately 10:00 of 28 July 2017	Water level of the dam was about 10 centimeters below the dam crest. Water levels between upstream and downslope sides at the dam's breached sites were nearly identical [16-17]
29 July 29 2017	The Royal Irregation Department began to plug the breach with soils from nearby area [16-17]
22 August 2017	A completion of a temporary dam rehabilitation.

3.1 Rainfall return periods

The tropical storm, Sonca, was presumed to be the cause of HSK dam breach and a subsequent flooding of the Sakon Nakhon town. We analyzed the rainfall data of 27-29 July 2017 together with rainfall patterns. We used the extreme value type I method to categorize the degree of rainfall heaviness. In the last row of Table 6, the smallest return period was 3 years for the duration of 1-h between 04:00 and 05:00 of 28 July while the largest was 52 years of 24-h duration between 19:00 of 27 July and 19:00 of 28 July.

Table 6 Maximum rainfall variables and return periods of the incident rainfalls.

Variable	Rainfall duration					
	1-h	2-h	3-h	6-h	12-h	24-h
$\bar{x}$ (mm/h)	57.27	35.94	26.85	15.03	8.26	4.80
SD (mm/h)	12.96	9.30	9.10	6.55	3.40	1.83
β	10.10	7.25	7.10	5.11	2.65	1.43
α	51.43	31.75	22.75	12.08	6.73	3.97
P (mm)	57.5	89.0	114.5	197.5	345.5	410.0
I (mm/h)	57.50	44.50	38.17	32.92	28.79	17.08
T (year)	3.05	3.76	4.44	10.50	49.16	51.90

3.2 Excess rainfall

The cumulative rainfall in column 3 of Table 7 was computed from total hourly rainfall, shown in column 2. The cumulative total rainfall was converted by Eq. 7 to cumulative excess rainfall of both upstream and downstream sub-watersheds as in columns 4 and 5 respectively. Finally, the cumulative excess rainfall values yielded the hourly excess rainfall as in columns 6 and 7. To produce the direct runoff hydrographs that flow into the corresponding reservoirs, the excess rainfall of the upstream sub-watershed was to be used with the unit hydrographs of HNB, HSTB1, and HSTB2 plus HSTB3, while the downstream one was to be used with HSK sub-watershed.

Table 7 Rainfall on HSK watershed from 00:00 to 15:00 of 28 July 2017.

Time (h)	Total rainfall (mm)	Cumulative rainfall (mm)	Cumulative excess rainfall		Hourly excess rainfall	
			Upstream (mm)	Downstream (mm)	Upstream (mm)	Downstream (mm)
0-1	12.2	12.2	0.7	0.9	0.0	0.0
1-2	5.9	18.1	2.7	3.2	2.0	2.3
2-3	9.9	28.0	7.9	8.9	5.2	5.6
3-4	25.1	53.1	26.4	28.0	18.4	19.1
4-5	57.5	110.6	77.7	80.2	51.3	52.2

Time (h)	Total rainfall (mm)	Cumulative rainfall (mm)	Cumulative excess rainfall		Hourly excess rainfall	
			Upstream (mm)	Downstream (mm)	Upstream (mm)	Downstream (mm)
5-6	31.3	141.8	107.3	110.1	29.7	29.9
6-7	20.2	162.0	126.8	129.6	19.5	19.6
7-8	24.8	186.8	150.8	153.8	24.0	24.2
8-9	35.2	222.0	185.3	188.4	34.4	34.6
9-10	27.4	249.4	212.2	215.3	26.9	27.0
10-11	34.1	283.5	245.8	249.0	33.6	33.7
11-12	30.5	314.0	275.9	279.2	30.1	30.2
12-13	27.6	341.6	303.3	306.6	27.4	27.4
13-14	19.3	360.9	322.4	325.8	19.2	19.2
14-15	5.7	366.6	328.1	331.4	5.6	5.6

3.3 Unit hydrographs

The physical parameters of the sub-watersheds were used to estimate the unit hydrograph variables i.e. t_l, t_p, u_p, and n from Eqs. 8 and 9 as shown in Table 2. From these variables, the 1-h unit hydrographs for all four sub-watersheds were created as shown in Figure 4. The two upstream unit hydrographs, HSTB1 and HSTB2 plus HSTB3, are alike due to their close positions and similar areas and shapes. Another upstream unit hydrograph was for HNB, which has a higher peak (3.03 m^3/s/cm) than the other two. Among the four sub-watersheds from the same rainfall, the downstream unit hydrograph for HSK with the highest peak (6.14 m^3/s/cm) can produce the highest flow.

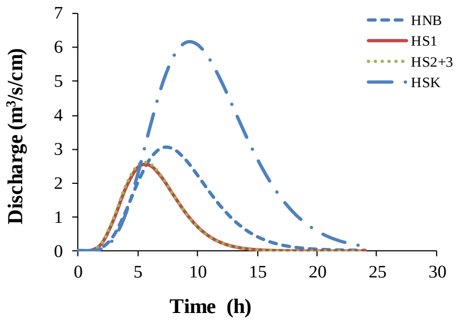


Figure 4 One-h unit hydrographs of the sub-watersheds. The peaks of HSK, HNB, HSTB1 and HSTB2 plus HSTB3 are 6.14, 3.03, 2.52 and 2.59 m^3/s/cm, respectively.

3.4 Inflow and outflow of the three upstream reservoirs

An inflow hydrograph from each sub-watershed to its reservoir was generated from its unit hydrograph and the hyetograph of excess rainfall from 00:00 to 15:00 of 28 July 2017 (Table 7). The inflow hydrographs to HNB, HSTB1, and HSTB2 plus HSTB3 are shown in Figure 5 together with their outflow hydrographs. The inflow hydrograph peak for each individual upstream reservoir is always higher than the outflow.

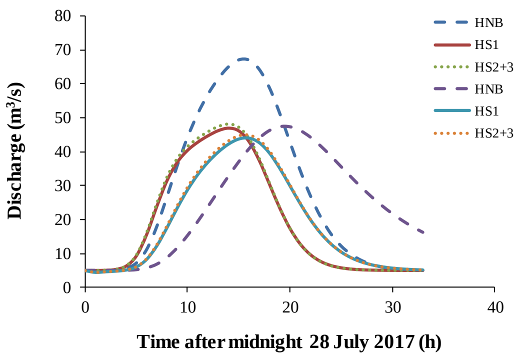
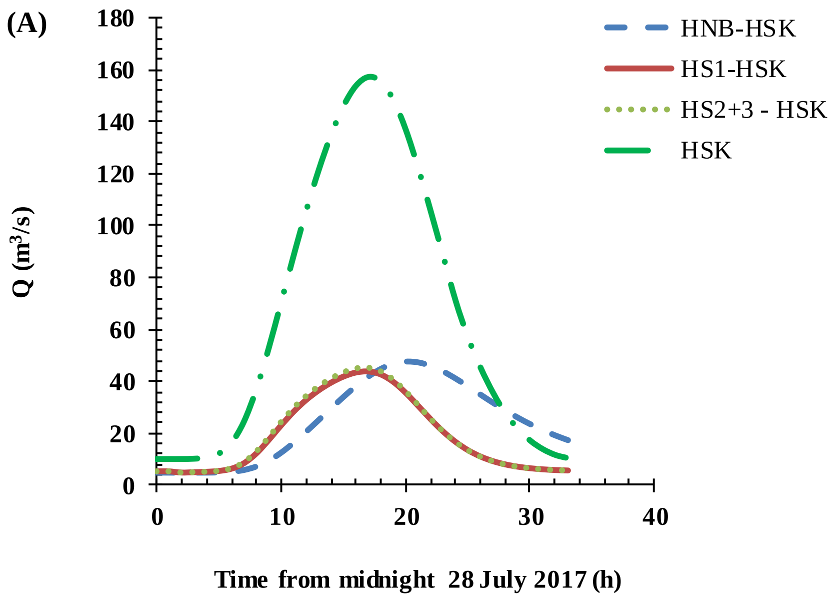


Figure 5 Inflow and outflow hydrographs of the three upper reservoirs, inflow graphs are those with higher peaks. The maximum amount of excess rainfall water stored in the HNB, HSTB1 and HSTB2 plus HSTB3 reservoirs above their spillways were about 1.3, 0.24, and 0.24 million cubic m, respectively.

3.5 Situation of Huai Sai Khamin during the flood

The total inflow into the HSK reservoir was comprised of outflows from three channel sections (HNB-HSK, HSTB1-HSK, and HSTB2 plus HSTB3 - HSK) and the outflow from HSK sub-watershed itself. Figure 6A shows these four components while Figure 6B shows the summation as the total inflow flood hydrograph. This inflow hydrograph created a flood and broke the embankment. The outflow hydrograph was simulated using reservoir routing Eqs. 13 and 14 to approximate the dam overtopping time. We approximated the overtopping time to be between 09:00 and 10:00 of 28 July, given that the dam crest was 0.8 m higher than the spillway crest and the *H-Q* relationship (Figure 3B) for the spillway rating curve of HSK. The dash line in Figure 6B represented the water level above the spillway crest, whereas the solid line represented the dam crest. Our analysis of flow over HSK spillway found the overtopping discharge at 25.24 m^3/s. However, the original design flood for the dam was 85 m^3/s. This design Figure was three times larger than the actual discharge at 25.24 m^3/s. In accordance with the dam original design Figure, the overtopping would have had happened between 15:00 and 16:00 on 28 July.



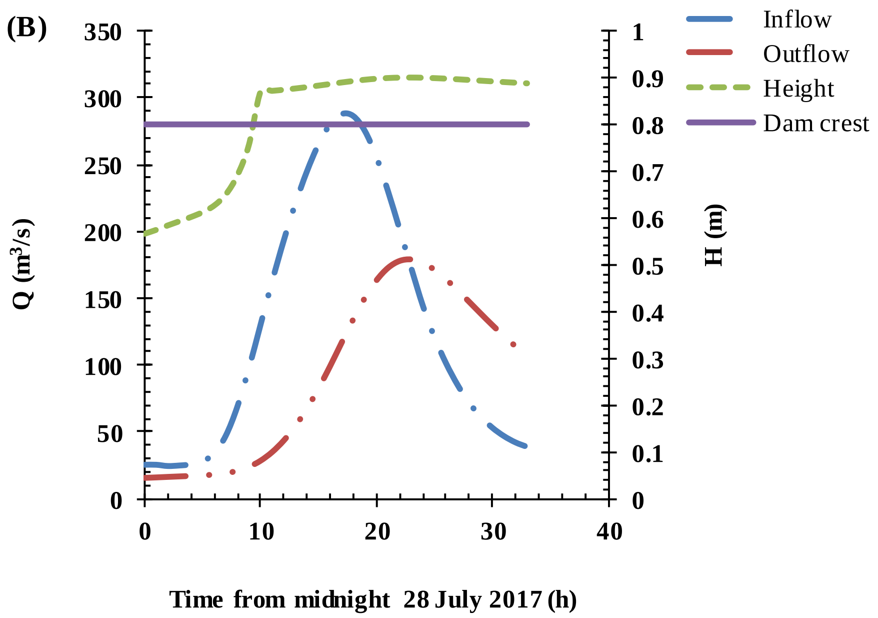


Figure 6 (A) Four portions of inflow to HSK, (B) inflow and outflow hydrographs with the height of water level above the spillway crest in relation to the dam crest. If the dam did not break, the maximum amount of excess rainfall water stored in the HSK reservoir above its spillway, and before the inflow smaller than the outflow was 5.5 million cubic m.

The evidence suggesting spillway blockage during the HSK dam failure episode comes from our site visits and viewing of videos and pictures showing the presence in large quantities of dense water weed and water plants [2, 15-17]. Apparently, large quantities of dense and large debris diminished the spillway capacity. The pictures of HSK dam taken before its failure incident showed also that the HSK reservoir contained deep-root water weeds and a large number of big dead trees and timbers, thereby suggesting that the trees were not completely removed from the reservoir during construction [2]. The HSK dam was built and completed at the same time as the HNB (Table 1). The HSK, however, failed after 61 years with the 52-year return period of rainfall, while the HNB has survived. This can be taken as evidence to suggest that the design, construction and/or maintenance of HSK dam may have been compromised.

4. Conclusion and recommendations

The analysis of hydrological data indicates an inadequate spillway capacity to be one of the factors contributing to a failure incident of the HSK dam. The outflow rate from the spillway during the breaching incident was 25.24 m^3/s. This outflow rate was three times lower than the original maximum outflow rate of 85 m^3/s. However, it is noteworthy that even if the outflow rate from the spillway was at its original maximum rate, the dam failure would still have occurred around 6 or 7 h later. The dam was breached by the strike of 410 mm rainfall within 24 h. This amount of rainfall was a 52-year return period. In contrast, there was no overtopping of the four dams, HNB, HSTB1, HSTB2, and HSTB3. These four dams are all situated within the HSK catchment and they were also affected by the similarly large amount of rainfall. For HNB, HSTB1 and HSTB2 plus HSTB3, the maximum amount of excess rainfall water stored in their reservoirs above their spillways were about 1.3, 0.24, and 0.24 million cubic m, respectively. The highest water level below the dam crests for HNB, HSTB1 and HSTB2 plus HSTB3 were about 0.5, 0.1, and 0.1 m, respectively. As the present study has focused solely on hydrological data, it is recommended that geological, geotechnical, and engineering conditions should be investigated to acquire all knowledge crucial for a proper HSK dam restoration.

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