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Prediction Dynamic Flooding of Dam Break Using Hydrodynamic Model and Flood Assessment from Classified THEOS Images: A Case Study of Srinagarind Dam, Kanchanaburi Province

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ABSTRACT

The study assumes that Srinagarind Dam of Kanchanaburi province will be collapsed by overtopping with a final breach of trapezoidal shape due to heavy rainstorm. Subsequently, various forms of demolitions and dynamic flooding are identified using Dam Break and Hydrodynamic models of MIKE 11 by applying the principle of the Probable Maximum Flood (PMF). Srinagarind dam is a rock fill dam with it is crest at +185.00 m. from Mean Sea Level (MSL) and store a maximum volume of 17,745 million m^3 of water at the beginning of the break. For dynamic flooding prediction, the maximum breach outflow discharge was 11,307 m^3/s with a velocity of 5.39 m/s at 62.29 hours after the break. In addition, the maximum discharge, water level and flood duration at cross sections of main rivers are also extracted for flood zone identification into 4 zones which include: at Ban Phu Thong Maeo, Wang Dong Sub-district, Mueang Kanchanaburi district, the discharge is 14,231 m^3/s, the maximum water level is +56.56 m. (MSL.) at 68.28 hr. after dam failure. At Ban Nuea, Ban Nuea Sub-district, Mueang Kanchanaburi district, the discharge is 14,081 m^3/s and the maximum water level is +41.59 m. (MSL.) at 83.21 hr. after dam failure. At Ban Wang Khanai, Wang Khanai Sub-district, Tha Muang district, the discharge is 13,244 m^3/s and the maximum water level is +37.671 m. (MSL.) at 93.24 hr. after dam failure. At Ban Luk Kae, Ban Don Khamin Sub-district, Tha Maka District, the discharge was 12,047 m^3/s, the maximum water level is +18.92 m. (MSL.) at 107.26 hours after dam failure. Land use land cover (LULC) types are affected by the flood after the dam-break which would cover an area of 1,172.21 sq.km. (1) Urban and built-up area (City, town, and commercial areas), (2) Paddy field, (3) Field crop, (4) Orchard, (5) Horticulture, (6) Pasture and farm house, (7) Evergreen forest, (8) Deciduous forest, (9)

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Natural water bodies (10) Reservoir (11) Rangeland, and (12) Mine and pits are included. The most affected LULC is paddy field that covers an area of 331.88 sq.km. (28.32%). The second and third largest affected LULC are rangeland and field crop areas that covered an area of 285.66 sq.km. (24.37%) and 267.02 sq.km. (22.78%). The least affected LULC is a deciduous forest that covers area of 15.49 sq.km. (1.32%).

Keywords: Prediction Dynamic Flooding; Dam Break; Hydrodynamic Model; Srinagarind Dam.

1. Introduction

1.1 Background problem

Kanchanaburi province is the economic and tourist center of the western region of the country. Gross Provincial Product (GPP) of Kanchanaburi province between 2002 and 2009 continued to increase. In 2009 GPP of Kanchanaburi province was 72,954 million Bath and per capita income was 92,923 Bath [1].

However, Kanchanaburi province was affected by natural disasters such as droughts and floods. These disasters affected the socio-economic values of the province. Natural flood caused by heavy rain storms which occurred during September to November in rainy season affected to many districts including Mueang Kanchanaburi, Sri Sawat, Dan Makham Tia, Nong Prue, Bo Phloi, Huai Krachao, and Sai Yok districts [2]. Thus, many dams which located in Kanchanaburi province play important roles for flood control.

The Srinagarind Dam has been developed as the multi-proposed project which provides significant benefits include: irrigation for agriculture, hydroelectricity, flood control, water transportation, and control of water quality, fisheries and tourism. It is located at the Khwae Yai River, Tha Kradan Sub-District, Si Sawat District, Kanchanaburi province. The dam is a center impervious core rock fill dam with a maximum height of 140 m. and a crest length of 610 m. The reservoir capacity is 17,745 million m^3 at the maximum retention level of 180 m. from Mean Sea Level (MSL) [3] is shown in Fig. 1.

However, if heavy rainstorm producing a Probable Maximum Flood occurs over the Srinagarind dam, it will impact to dam as overtopping. In addition, if the dam is broken, it will be affected to socio-economic value of the Kanchanaburi province. There for the, study on prediction a dynamic flooding using Dam Break and Hydrodynamic model for land use damage assessment and an evacuation center identification and human migration routing using Location/ Allocation model in assumption case for dam-break with over-topping by heavy rainstorm at Srinagarind dam, Kanchanaburi province is very important. In addition, this research will fulfill the recommendation of previous research works in the field of dam break in Thailand such as Lotinun (1997) [4]; Aksornrat (2000) [5]; Soisangwan (2004) [6], and Waniphongphan (2008) [7]. Because all these research works conducted only dam break and hydrodynamic flood. None touched on the socio-economic impacts of such phenomenon.

Fig. 1. Srinagarind dam in Kanchanaburi province, Thailand. [3]

1.2 Research Objectives

(1) To predict the extent of a dynamic flooding if Srinagarind Dam will be broken with overtopping using Dam Break and Hydrodynamic models.

(2) To assess the flood damages from classified THEOS images.

1.3 Basic assumptions

Basic assumptions in this study are set up as follows:

(1) Srinagarind dam could be broken as overtopping by heavy rainstorm.

(2) Under dam break model, Srinagarind dam, which constructed with clay core zone and rock fill materials, is assumed to only one material having the same properties the use as clay core zone.

(3) The worst case scenario is applied for hydrodynamic model after dam-break based on Probable Maximum Flood (PMF).

1.4 Scope of the study

Scope and limitations of the study can be briefly explained as follows:

(1) Study area which covers an area downstream of Srinagarind dam but within Kanchanaburi province only.

(2) Water flow condition resulting from dam break is analyzed by a Dam Break Model and Hydrodynamic Model of MIKE 11 for simulation flooding phenomena at the specified periods of time from Srinagarind dam to the lowest part of Kanchanaburi province.

1.5 Study area

1.5.1 Kanchanaburi province

Kanchanaburi province, which is the third largest province in Thailand, situated 129 km. from Bangkok and covers a total area of 19,483 sq. km. It is located at the latitude 13° 43′ 20″ to 15° 39′ 39″ North and the longitude 98° 10′58″ to 99° 53′ 31″ East. The province is subdivided into 13 Districts including Mueang Kanchanaburi, Tha Muang, Phanom Thuan, Tha Maka, Dan Makham Tia, Sai Yok, Thong Pha Phum, Sangkhla Buri, Si Sawat, Bo Phloi, Nong Prue, Huai Krachao, and Lao Khwan (Fig. 2). Based on the Department of Provincial Administration [8], it consists of 98 Sub-districts and 959 villages.

1.5.2 Srinagarind dam

The Srinagarind dam is located at the Khwae Yai River, Kanchanaburi province, about 190 km. northwest of Bangkok, Thailand (Fig. 2). The dam is a rock-fill dam with a cross section as shown in Fig. 4 with a maximum height of 140 m. and a crest length of 610 m. The construction began in 1973 and received the first filling on 1977 (Fig. 1 and Fig. 3). The downstream slope angle is 1: 1. 8 and the upstream slope angles are 1:2.2 and 1:2. The reservoir capacity is 17,745 million m^3 at the maximum retention level of 180 m. (MSL). The dam consists of 5 fill zones (Fig. 4) including: (1) the impervious core made of clayey sand, (2) the filter material is obtained from a river alluvium, (3) the transition zone is obtained from the foundation excavation and a query of quartzite, (4) the rock fill material is a hard durable limestone with the maximum size of 0.70 (small size) and (5) the rock fill material is a hard durable limestone with the maximum size of 1.50 m. (large size). The static properties of the dam materials are as shown in Table 1.

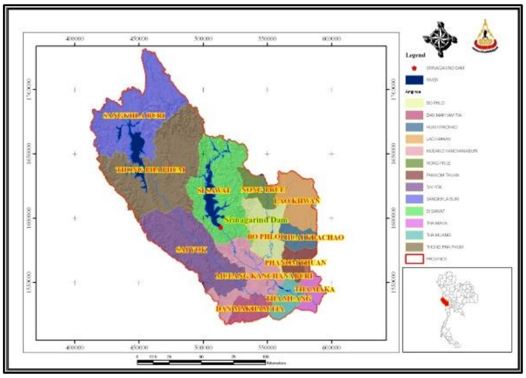

Fig. 2. Location and administration bound-aries of the study area.

Fig. 3. Srinagarind dam in Kanchanaburi province, Thailand.

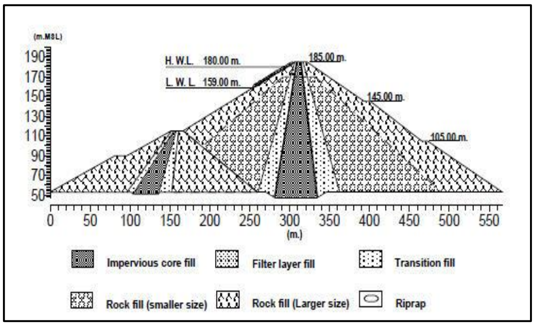

Fig. 4. Cross section of Srinagarind dam. [9]

Table 1. The static properties of the dam materials. [10]

Description	Unit Weight (T/m^3)			Cohesion (t/m^2)	Coefficient of Internal Friction
	Dry	Wet	Sat.		
Impervious	1.8	2.03	2.13	4	0.3
Filter	2	2.04	2.25	-	0.7
Transition Rock fill	2	2.04	2.25	-	0.7
(Smaller) Rock fill	1.8	1.82	2.13	-	0.65
(Larger) Rock fill	1.75	1.77	2.09	-	0.8

2. Materials and Methods

2.1 Data and Equipment

Data used for this research involve spatial data (remotely sensed data, GIS data, hydrological data, topographic data and in situ data.) and non-spatial data (demographic data and socio- economic data, National Rural Development (NRD) - 2C data. For equipment, GPS and a notebook are used as hardware while GIS, Remote Sensing and Hydrology software are applied in this study (Table 2.).

2.2 Methodology

Methodological framework of this research is schematically displayed in Fig. 5. This includes:

(1) Data collection and preparation.

(2) Dam break development and dynamic discharge evaluation by a Dam break Model.

(3) Dynamic water flow evaluation by a Hydrodynamic Model.

(4) Land use and land cover assessment.

The detail of each research methodology is separately described in the following sections.

Table 2. Data and Equipment.

Data and Equipment	Source/Remarks
1.1 Primary datasets	
Field survey by GPS	Kanchanaburi Province
THEOS	Geo-Informatics and Space Technology Department Agency(Public organization)
1.2 Secondary datasets	
Topographic map	Royal Thai Survey Department (RTSD)
River cross section	Electricity Generating Authority of Thailand (EGAT)
GIS dataset	Department of Environmental Quality Promotion
Demographic data	Department of Provincial Administration
NRD data	Community Development Department
2.1 Hardware	
GPS	Laboratory, TU
Notebook	Personnel
2.2 Software	
ArcGIS	Laboratory, TU
ENVI	Laboratory, TU
MIKE 11	Laboratory, TU

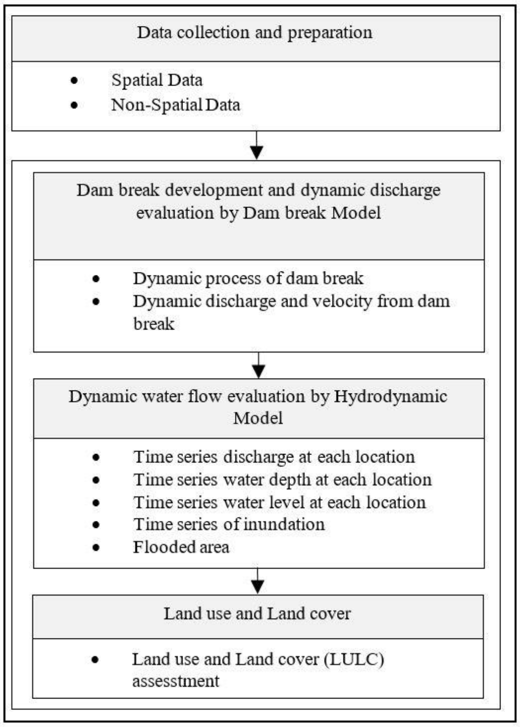

Fig. 5. Framework of methodology.

2. 3 Dam- break development and dynamic discharge and velocity evaluation by Dam Break Model

Under this component, dam- break development and dynamic discharge and velocity evaluation are analyzed by Dam Break of MIKE 11 based on the assumption of Srinagarind dam will be broken by heavy rainstorm as Probable Maximum Flood. In practice, this component is divided into 6 steps for explanation dam break process and evaluation of dynamic discharge and velocity as following.

Step 1 Dimensionless unit hydro-graph analysis

In this step, dimensionless unit hydrograph analysis is conducted by using Modified Snyder method. In practice, basin time lag (t_p) is firstly calculated by using the properties of the basin and watershed area (A), the length of the river (L), the length of the river from the point closest to the center of gravity of the catchment area (L_c) and the average slope (S) in the watershed area by using following Eq. (2.10):

$$t_p = a\left(\frac{LL_c}{\sqrt{S}}\right)^b \qquad (2.1)$$

where:

t_p is the basin time lag time (hours)

L is a length of the river (km.)

L_c is a length of the river from the center of gravity of the watershed area (km.)

S is an average slope in the watershed area

a and b are regression coefficients for the basin

Then, peak flow (q_p) is calculated using following equation [11]:

$$q_p = cA\left(t_p\right)^{-d} \qquad (2.2)$$

where:

q_p is peak flow (m^3/s)

A is area of catchment (sq. km.)

t_p is basin time lag (hours)

c and d are regression coefficients for the basin

In this study, unit hydrograph analysis is calculated based on Bangsomboon's research (1991) [1]. Which works well for small watershed in the Western and Southern parts of Thailand. For the Mae Klong River basin the following equations were used [11].

$$t_p = 0.169\left(\frac{LL_c}{\sqrt{S}}\right)^{0.499} \qquad (2.3)$$

$$q_p = 0.65A\left(t_p\right)^{-0.399} \qquad (2.4)$$

Then, base length time and duration time for a unit hydrograph are calculated using following equations are used [11].

$$T = 3\left(1 + \frac{t_p}{24}\right) \qquad (2.5)$$

where:

T is base length time (day)

t_p is basin time lag (hours)

$$t_r = \frac{t_p}{5.5} \quad (2.6)$$

where:

t_r is duration time (hour) of rain storm.

Based on Snyder's theory [11] the maximum time period for peak flow of water discharge is $(t_p + t_r/2)$. The derived outputs include initial time/discharge, peak time/discharge and end time/discharge are used to create unit hydrograph. After that, series of time and discharge are read from hydrograph chart for normalization of time and discharge by division of their values with maximum time and maximum discharge, respectively. The output is a dimensionless unit hydrograph that will be used in the next step.

Step 2 Analysis of Probable Maximum Flood (PMF) Hydrograph

The Probable Maximum Flood of Srinagarind Dam (7,100 m^3/s) which was calculated by EGAT is used for PMF hydrograph analysis. In practice, normalized time series and normalized discharge series of dimensionless unit hydrograph from previous step are firstly multiplied by basin time lag (t_p) and possible maximum discharge, respectively. Then series of calculated discharge are corrected by the modified direct runoff factor as shown in following Eq. (2.11):

$$\text{Modified runoff factor} = \frac{(\Delta t \times Total\ Q)}{A} \times 100 \quad (2.7)$$

where:

Δt is time step (second)

Total Q is total value of calculated discharge (m^3/s)

A is basin area (sq. m)

After that series of corrected discharge with the modified direct runoff factor is normalized by the maximum corrected discharge values again. Then, new series of normalized discharge are multiplied by possible maximum discharge (7,100 m^3/s) again. Finally, new derived time and discharge series are used to plot Probable Maximum Flood (PMF) graphs.

Step 3 The shape of the breached dam collapsing

In this step, MIKE 11 developed by Danish Hydraulic Institute (DHI) [13] is used to simulate dam break process. In this study, it is assumed that Srinagarind Dam is collapsed by overtopping with final breach's shape of trapezoid. Probable Maximum Flood (PMF) from previous step is used to calculate downstream water movement in hydrodynamic model.

Step 4 Engelund- Hansen Equation analysis

After the crevasse shape of dam is defined, the spread of crevasse erosion using the theory of resistance of water's flow through crevasse is applied using Engelund-Hansen method as [13]:

$$\frac{dH_b}{dt} = \frac{q_t}{L_b\,(1-\varepsilon)} \quad (2.8)$$

where:

H_b is the breach level (m.)

q_t is the sediment transport rate (m^3/s)

ε is the porosity of the sediment

L_b is the breach length in the direction of flow (m.)

t is time (s)

Step 5 Breach Development

In theory, the development of cross sectional of crevasse will not increase if critical shear stress of water is less than or equal to critical shear stress of material. The latest cross section of crevasse will be then formed as trapezoid shape for dam break [13].

The latest cross section of crevasse is calculated based on the level of bed or ridge in order to set the cross sectional area. The coefficient of proportionality, x, (side erosion index) relates the increase in breach width, W_b, to depth (H_b) is calculated by using following Eq. (2.13):

$$\frac{dW_b}{dH_b} = 2x \qquad (2.9)$$

In general, the side erosion index is in the order of 0.5-1.0 [13].

The outputs from step 3 to 5 are used to explain dam break process.

Step 6 Calculation of discharge and velocity through dam

Amount of water flow through dam as discharge and its velocity are calculated using following Eq. (2.13):

$$h_1 + \frac{V_1^2}{2g} - \frac{\zeta}{2g}\left(\frac{Q_s}{A_s}\right)^2 = h_2 + \frac{V_2^2}{2g} \qquad (2.10)$$

where:

- h_1 is upstream water level (m.)
- h_2 is downstream water level (m.)
- v_1 is velocity upstream (m/s)
- v_2 is velocity downstream (m/s)
- A_s is area Section (m^2)
- Q_s is discharge (m^3/s)
- ζ is head loss coefficient
- g is gravitational acceleration (9.8 m/s)

2.4 Dynamic water flow evaluation by Hydrodynamic Model

Under this component, dynamic water flow includes time series of discharge, water depth and water level in the downstream are calculated based on hydrodynamic model of MIKE HD [13] in the following steps:

Step 1 Dam characteristics

Before calculation of flow of hydraulic in downstream, basic characteristics of dam is firstly quantified included location and capacity of dam. In addition, cross section of downstream river includes Khwae Yai and Mae Klong Rivers are also compiled and prepared in GIS format.

Step 2 Characteristics initial condition and boundary of upstream and downstream

In this step, condition of water in upstream that is derived from Dam Break model is set up based on the highest discharge. In addition, the condition of riverside of Khwae Yai and Mae Klong rivers in downstream is also set up.

Step 3 Calculate water flow in channel system using Saint Venant Equation

In this step, water flow in channel system is calculated by considering the characteristics of unsteady flow pattern. This method applies Saint Venant Equation that includes Continuity and Momentum Equations as shown in following Eq. (2.13):

Continuity equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad (2.11)$$

Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0 \qquad (2.12)$$

where:

Q is discharge (m^3/s)
A is flow area (m^2)
q is lateral inflow (m^3/s)
h is stage above datum (m.)
C is Chezy resistance coefficient
R is hydraulic or resistance radius (m.)
α is momentum distribution coefficient

The main outputs in this step include time series of discharge, water level, water depth and flooded area and zones. In this study, the first location of cross section along the main rivers (Khwae Yai and Mae Klong), where is flooded, is selected for the flooding zone identification.

Step 4 Sensitivity Analysis

Sensitivity analysis of discharge in different time step is performed based on Manning's n roughness coefficient of channel as [13]:

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \qquad (2.13)$$

where:

Q is discharge (m^3/s)
A is flow area (m^2)
R is hydraulic or resistance radius (m.)
S is slope
n is Manning'n roughness coefficient

In this study, sensitivity analysis is conducted by varying of Manning'n roughness coefficient between 0.030 and 0.040 for simulation of downstream flow.

Step 5 Stability Condition Calculations

Under this step, stability condition (C_r) is calculated for validation time step (Δt) and average distance between cross section (Δx) using following Eq. (2.13):

$$C_r = \frac{\Delta t(V + \sqrt{gy})}{\Delta x} \qquad (2.14)$$

where:

C_r is Courant value
Δt is time step (s)
Δx is average distance between cross section (m.)
g is gravitational acceleration (9.8 m/s)
y is water depth (m.)

The optimum value of stability condition should be less or equal 10 [13]:

Step 6 Velocity Condition Calculations

Velocity condition of water flow after dam-break is also calculated using following Eq. (2.13):

$$Velocity\ condition = \frac{V\Delta t}{\Delta x} \qquad (2.15)$$

where:

- V is velocity of water (m/s)
- Δt is time step (s)
- Δx is average distance between cross section (m.)

The optimum value of velocity condition should be less or equal 1 [13].

3. Results

3.1 Analysis of a Dimensionless Unit Hydrograph

A modified method of Snyder is applied for dimensionless unit hydrograph analysis. In this study existing equation for basin time lag (t_p) and peak flow (q_p) calculation of Mae Klong River Basin based on Bangsomboon, N. (1991) [12] are used for generation unit hydrograph as shown in Eq. (2.3) and Eq. (2.4), respective.

Basically, Mae Klong River Basin covers area (A) of 10,996.68 sq. km. with 336.46 km. length (L). The length from point closest to the center of gravity of the catchment area (L_c) is 139.73 km. and the average slope (S) in the catchment area is about 0.036. Hence, basin time lag (t_p) and peak flow (q_p) of Mae Klong River Basin are derived as following [11]:

$$t_p = 0.169\left(\frac{336.46x139.73}{\sqrt{0.036}}\right)^{0.499} \quad \text{or}$$

$$t_p = 82.85 \ \text{ hours.}$$

$$q_p = 0.65 \times 10{,}996.68(82.85)^{-0.399}$$

or

$$q_p = 1{,}230.66 \ \text{ m}^3\text{/s}$$

Subsequently, base length time *(T)* and duration time (t_r) for a unit hydrograph derivations were then calculated using Eq. 5 and Eq. 6, respective as follows:

$$T = 3\left(1 + \frac{82.85}{24}\right) \quad \text{or}$$

$$T = 13.35 \ \text{ days} \quad \text{or}$$

$$T = 320.55 \ \text{ hours}$$

$$t_r = \frac{82.85}{5.5} \quad \text{or}$$

$$t_r = 15.06 \ \text{ hours.}$$

In addition, $t_r/2$ equals 7.53 hours.

Thus, the derived outputs include initial time/discharge, peak time/discharge and end time/discharge are used to create unit hydrograph as shown in Table 3. After that, series of time and discharge are read from hydrograph chart for normalization of time and discharge by division of their values with maximum time and maximum discharge, respectively. The output is a dimensionless unit hydrograph as shown in Fig. 6. This output was then used for probable maximum flood analysis.

Table 3. Basic time and discharges for the unit hydrograph creation.

Stage	Time (hour)	Discharge Q_p (m^3/s)
Initial	0	0
Peak	90.38	1,230.66
End	320.55	0

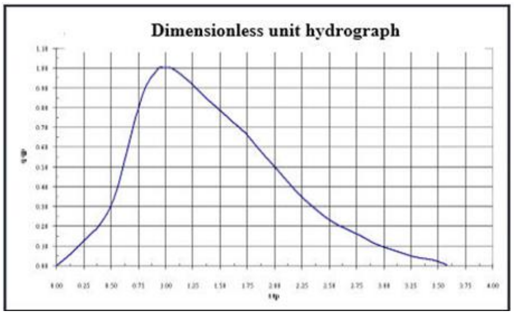

Fig. 6. Dimensionless unit hydrograph of Mae Klong River basin.

3.2 Probable maximum flood (PMF) Analysis

In this study, the possible maximum discharge of Srinagarind Dam of EGAT which equals to 7,100 m^3/s is used for PMF analysis. Normalized time series and normalized discharge series of dimensionless unit hydrograph are firstly multiplied by basin time lag (t_p) and possible maximum discharge, respectively. Then series of calculated discharge are corrected by the modified direct runoff factor (Eq.7) as shown in Table 4. After that the first adjustment of time and discharge are normalized again and calculated new series of time and discharge based on probable maximum flood as shown in Table 5. The hydrograph of probable maximum flood (PMF) is displayed in Fig. 7.

Table 4. First adjustment of time and discharge series based on characteristics of the basin (Basin time lag and PMF) and modified direct runoff factor

Series	Normalized Value of Time and Discharge based on Dimensionless Unit Hydrograph		First Adjustment of Time and Discharge		Modified direct runoff factor
	Time (t/t_p)	Discharge (q/q_p)	Time*Basin time lag (hr.)	Discharge * PMF (m^3)	Discharge
0	0.00	0.00	0.00	0.00	0.00
1	0.13	0.06	11.00	432.69	14.71
2	0.26	0.13	22.00	951.92	32.37
3	0.39	0.21	33.00	1500.00	51.00
4	0.53	0.34	44.00	2480.78	84.35
5	0.66	0.60	55.00	4326.94	147.13
6	0.79	0.87	66.00	6201.95	210.89
7	0.92	0.99	77.00	7067.34	240.32
8	1.00	1.00	82.85	7100.00	241.43
9	1.06	0.99	88.00	7090.42	241.11
10	1.19	0.94	99.00	6692.34	227.57
11	1.32	0.87	110.00	6201.95	210.89
12	1.59	0.73	132.00	5250.02	178.52
13	1.72	0.67	143.00	4788.48	162.83
14	1.85	0.58	154.00	4153.86	141.25
15	1.99	0.50	165.00	3605.78	122.61
16	2.12	0.42	176.00	3000.01	102.01
17	2.25	0.34	187.00	2451.93	83.37
18	2.39	0.28	198.00	1990.39	67.68
19	2.52	0.22	209.00	1586.54	53.95
20	2.65	0.18	220.00	1326.93	45.12
21	2.78	0.15	231.00	1067.31	36.29
22	2.92	0.11	242.00	778.85	26.48
23	3.05	0.08	253.00	605.77	20.59
24	3.18	0.06	264.00	432.69	14.71
25	3.31	0.04	275.00	288.46	9.80
26	3.45	0.02	286.00	201.92	6.86
27	3.54	0.01	293.84	86.53	2.94
28	3.58	0.00	297.00	0.00	0.00
				81661.94	

3.3 Analysis of Srinagarind dam break by water overtopping with a trapezoid shape

Refer to assumption of the study, it is assumed that Srinagarind Dam will be collapsed by overtopping with final breach's shape of trapezoid.

Table 5. Second adjustment of time and discharge series based on PMF.

First Adjustment		Normalization		Second Adjustment by PMF	
Time (hr.)	Discharge (cu. m /s)	Time	Discharge	Time (hr.)	Discharge (cu. m /s)
0.00	0.00	0.00	0.00	0.00	0.00
11.00	14.71	0.13	0.06	11.00	432.69
22.00	32.37	0.26	0.13	22.00	951.93
33.00	51.00	0.39	0.21	33.00	1500.01
44.00	84.36	0.53	0.34	44.00	2480.78
55.00	147.13	0.66	0.60	55.00	4326.95
66.00	210.89	0.79	0.87	66.00	6201.96
77.00	240.32	0.92	0.99	77.00	7067.34
82.85	241.43	1.00	1.00	82.85	7100.00
88.00	241.11	1.06	0.99	88.00	7090.42
99.00	227.57	1.19	0.94	99.00	6692.34
110.00	210.89	1.32	0.87	110.00	6201.96
132.00	178.52	1.59	0.73	132.00	5250.03
143.00	162.83	1.72	0.67	143.00	4788.49
154.00	141.25	1.85	0.58	154.00	4153.87
165.00	122.61	1.99	0.50	165.00	3605.79
176.00	102.01	2.12	0.42	176.00	3000.02
187.00	83.37	2.25	0.34	187.00	2451.94
198.00	67.68	2.39	0.28	198.00	1990.40
209.00	53.95	2.52	0.22	209.00	1586.55
220.00	45.12	2.65	0.18	220.00	1326.93
231.00	36.29	2.78	0.15	231.00	1067.31
242.00	26.48	2.92	0.11	242.00	778.85
253.00	20.59	3.05	0.08	253.00	605.77
264.00	14.71	3.18	0.06	264.00	432.69
275.00	9.80	3.31	0.04	275.00	288.46
286.00	6.86	3.45	0.02	286.00	201.92
293.84	2.94	3.54	0.01	293.84	86.54
297.00	0.00	3.58	0.00	297.00	0.00

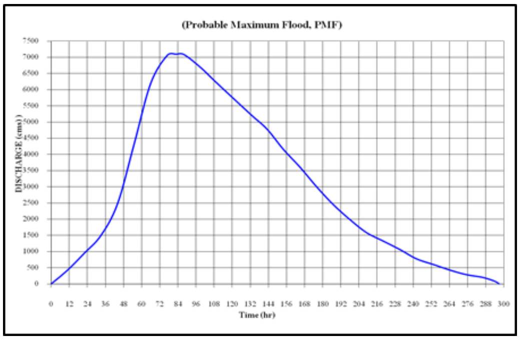

Fig. 7. Hydrograph of Probable Maximum Flood (PMF) of Mae Klong River basin.

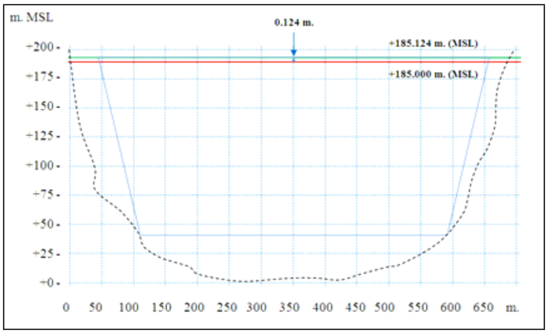

Fig. 8. Shape of dam-break at the start time of the breach development.

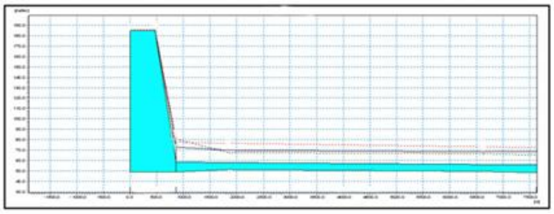

Fig. 9. The profile plot of water levels at the start time when a dam collapses.

3.4 Analysis of Srinagarind dam break by water overtopping with a trapezoid shape (continued)

In this study, MIKE 11 software is used to simulate the unsteady flow of a dam-break process. Herein, Srinagarind dam has an impervious core rock fill dam with crest +185.00 m. (MSL.) and maximum volume 17,745 million cu. m. is collapsed. Subsequently, various forms of demolitions are identified and downstream water movement is also calculated by applying probable maximum flood (PMF).

The collapse of dam model in the case of a gradual destruction is studied by parameter effect analysis. That means monitoring destruction of dam as changing parameters: Side Slope (SS), Side Erosion Index (x), and Initial Breach Width (B).

(1) Initial of dam cracking. Start time for dam cracking begin at hours of 21.44 and velocity of water through the dam is about 1.11 m/s at the water level about +185.21 m (MSL.). The width of the crevices below dam height is 0.24 m. Level of top dam crest is +185.12 m. (MSL.) and Level of bottom dam crest is +185.00 m. (MSL.). The depth of fissure is 0.12 m. (Fig. 8) In addition, side view of dam-break process with water level is shown in Fig. 9.

(2) The peak of discharge through the dam-breaks occurred at hours 83.73 and velocity of water through the dam is 5.39 m/s at the water level about +77.32 m (MSL.). The peak discharge is 11,307 m^3/s. The top breach width is 465.35 m. The width of the crevices below dam height was 250.038 m. Level of top dam crest is +77.34 m. (MSL.) and Level of bottom dam crest is +68.66 m. (MSL.). The depth of fissure is 8.67 m. (Fig.10) In addition, side view of dam-break process with water level is shown in Fig. 11.

(3) The time for development of dam-break until the final breach trapezoid is 295.99 hours. Velocity of water through the dam is 0.733 m/s at the water level about +56.53 m. (MSL.). The discharge is 10.3 m^3/s. The top breach width is 514.26 m. The width of the crevices below dam height is 257.241 m. Level of top dam crest is +56.48 m. (MSL.) and Level of bottom dam crest is +56.43 m. (MSL.). The depth of fissure is 0.05 m. (Fig. 12) In addition, side view of dam-break process with water level is shown in Fig. 13.

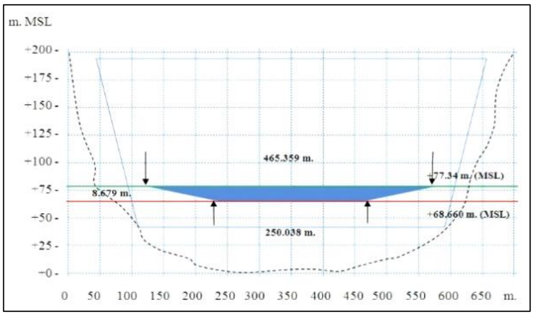

Fig. 10. Shape of dam-break at peak discharge breach development (time at hours 83.73).

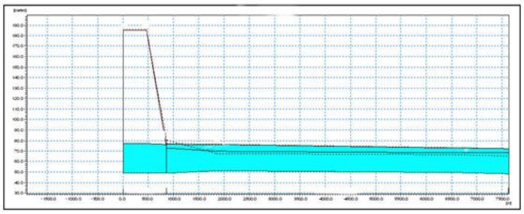

Fig. 11. The profile plot water levels at discharge flowing through the dam collapse at peak water (time at hours 83.73).

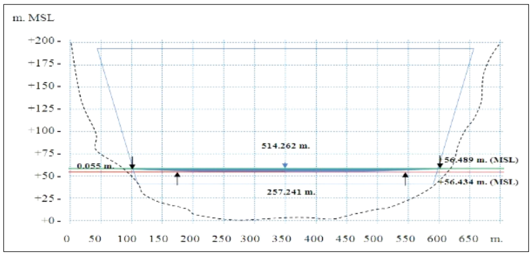

Fig. 12. Shape of dam-break at final breach development (time at hours 295.99).

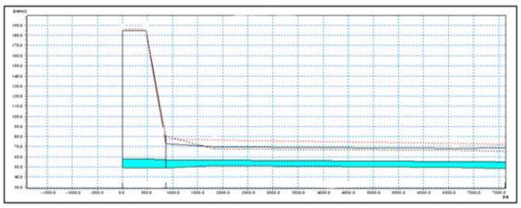

Fig. 13. The profile plot of water levels at develops dam-break until the final breach trapezoid (time at hours 295.99).

3.5 Dynamic water flow evaluation after dam-break

3.5.1 Condition of water in upstream and downstream

Basically, the condition of water in upstream at the km. 0+000 is the amount of water obtained from the simulation of the Srinagarind Dam break based on the probable maximum flood (PMF).While the condition of downstream of Mae Klong river at km. 134+03 (MK-16 At Ban Luk Kae) to mean sea level +13.5 m. (MSL.) is the highest level of the final cross section of Mae Klong river in Kanchanaburi province.

In addition, Manning' n roughness coefficient is defined equal to 0.035 and a sensitivity analysis is evaluated by varying Manning' n roughness coefficient between 0.030 and 0.040.

3.5.2 Dynamic water flow calculation

Dynamic water flow in channel system of Khwae Yai River and Mae Klong River is calculated by Saint Venant Equations that includes Continuity and Momentum Equations as shown in Eq. 11 and 12, respectively. The result of dynamic flow include discharge, water level, water flood, time of discharge, time of water level, time of water flood, duration of inundation at each cross section of two rivers (KY-1 to KY-24 and MK-1 to MK-16). Pattern of dynamic discharge and water level at selected cross section of Khwae Yai River and Mae Klong River are displayed in Fig. 14 and Fig. 15, respectively. In addition, pattern of water depth and water flood at selected cross sections along rivers are shown in Fig. 16 and Fig. 17.

3.6 Sensitivity analysis on the Manning'n roughness coefficient change

Sensitivity analysis for water flow at downstream is carried by varying of Manning' n roughness coefficient between 0.030 and 0.040 for comparison of water level changes at each cross-section. Pattern of water level at each cross section along Khwae Yai River and Mae Klong River from

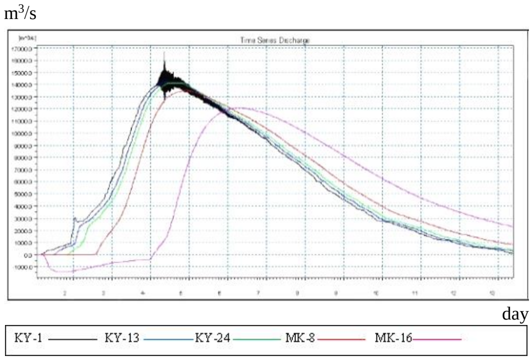

Fig. 14. Pattern of a dynamic discharge at selected cross sections.

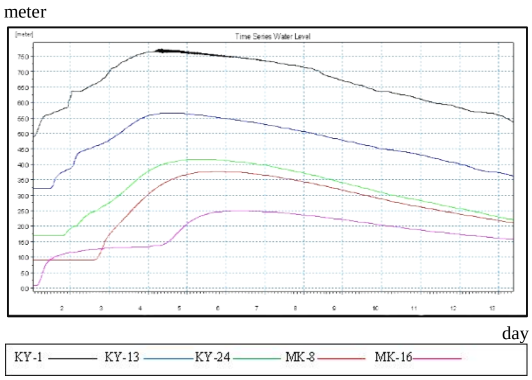

Fig. 15. Pattern of water level at selected cross sections.

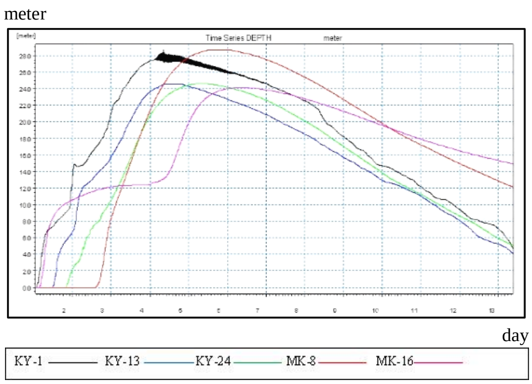

Fig. 16. Pattern of water depth at selected cross sections.

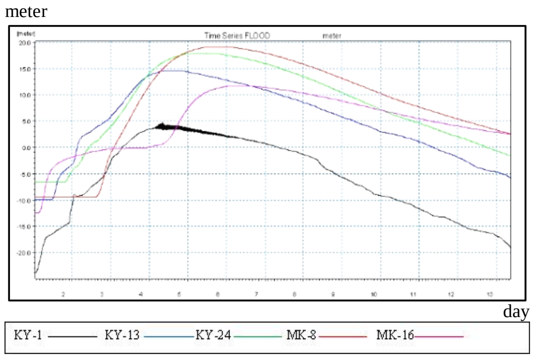

Fig. 17. Pattern of water flood at selected cross sections.

sensitivity analysis by varying of Manning'n roughness coefficient is displayed in Fig. 18.

3.7 Stability Condition Calculation

The result of stability condition (*Cr*) is calculated for validation time step (Δt) and average distance between cross section (Δx) equal 1.14 Theoretically, an optimum value of stability condition should be less or equal 10 [13].

3.8 Velocity Condition Calculation

Velocity condition of water flow after dam-break is calculated equal 0.18. Theoretically, an optimum value of velocity condition should be less or equal 1 [13].

3.9 Affected land use and land cover types

Land use and land cover types covering projected flooding area are classified from THEOS data in 2010 using ENVI image processing software. Herewith standard supervised classification with maximum likelihood classifier is applied for extraction LULC categories according LLD land use classification system at second level included: (1) Urban and built-up area (City, town, and commercial areas), (2) Paddy field, (3) Field crop, (4) Orchard, (5) Horticulture, (6) Pasture and farm house, (7) Evergreen forest, (8) Deciduous forest, (9) Natural water bodies, (10) Reservoir, (11) Rangeland and (12) Mine and pits.

In addition, accuracy assessment for LULC classification is conducted using overall accuracy and Kappa analysis.

The most affected LULC is paddy field which covered area of 331.88 sq.km. (28.32%). The second and third affected LULC are rangeland and field crop which covered area of 285.66 sq.km. (24.37%) and 267.02 sq.km. (22.78%). The least affected LULC is deciduous forest that covers area of 15.49 sq.km. (1.32%). These results imply about the interaction between water flood and landform. Most of LULC types situated in flood plain or undulate landform are affected by flood after dam-break.

Detail of affected LULC in each flood zone (Zone I to Zone IV) is presented in Table 6. Distribution of affected LULC types in each flood zone (Zone I to Zone IV) is displayed in Figs. 19, 20, 21 and 22, respectively. The most affected LULC in each flood zone is diverse: Zone I is evergreen forest, Zone II is field crop, Zone III is rangeland and Zone IV is paddy field. These results infer about the interaction between watershed form and LULC. In upper watershed, most dominant LULC is natural forest, while in middle watershed, most dominant LULC are field crop and rangeland and the lower watershed is dominant covered by paddy fields.

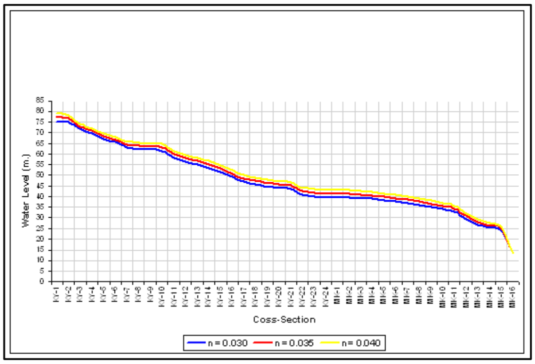

Fig. 18. Pattern of water level at each cross section of Khwae Yai River and Mae Klong River from sensitivity analysis by varying of manning roughness coefficient.

Table 6. Areas and percentages of effected land use and land cover by flood after Srinagarind dam-break.

LULC		Flood Zone				
ID	**Class name**	**I**	**II**	**III**	**IV**	**sq.km.**
1	City, Town, Commercial	0.14	3.92	19.53	5.42	29.01
2	Paddy field	0.02	3.33	50.64	277.88	331.88
3	Field crop	8.98	62.63	77.66	117.76	267.02
4	Orchard	0.16	18.83	45.36	10.64	74.98
5	Horticulture	0.27	24.90	51.30	1.27	77.74
6	Pasture and farm house	0.00	0.00	0.35	25.61	25.96
7	Evergreen forest	9.35	6.52	1.78	0.00	17.66
8	Deciduous forest	3.77	8.22	3.48	0.02	15.49
9	Natural water body	5.76	3.25	14.01	10.90	33.92
10	Reservoir	0.00	0.19	0.21	2.26	2.67
11	Rangeland	0.04	27.67	80.37	177.58	285.66
12	Mine and pit	0.00	0.00	0.02	10.21	10.23
	Total	**28.51**	**159.46**	**344.70**	**639.54**	**1,172.21**

4. Discussion

As results, 1D Hydrodynamic model of MIKE 11 can provide basic need information include discharge, water level and water depth and water flood for dynamic flood prediction. This information is useful for flood warning and evacuation planning.

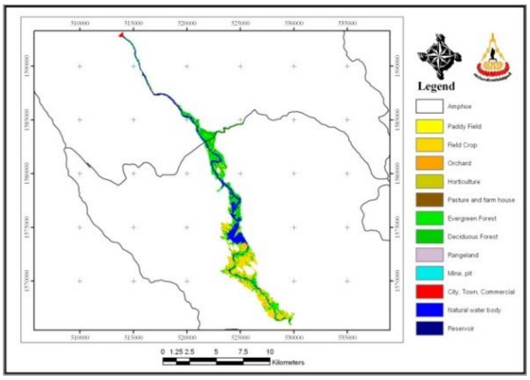

Fig. 19. Affected land use and land cover types of flood Zone I.

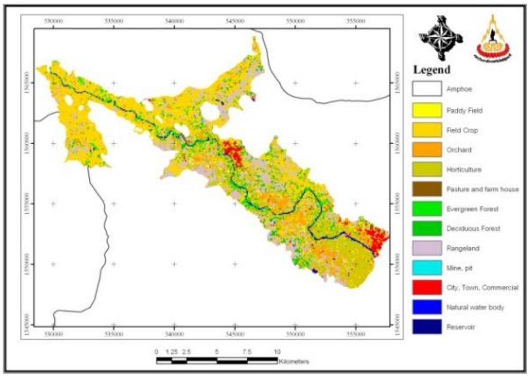

Fig. 20. Affected land use and land cover types of flood Zone II.

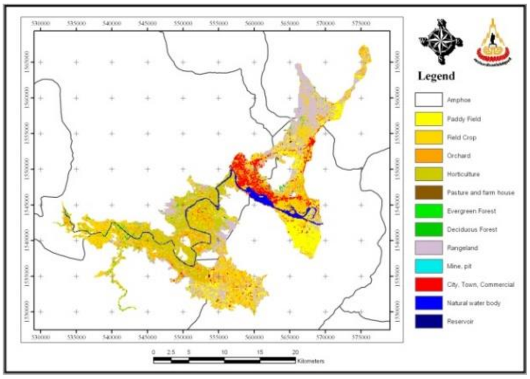

Fig. 21. Affected land use and land cover types of flood Zone III.

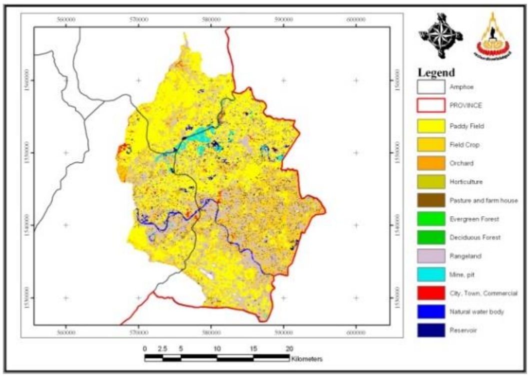

Fig. 22. Affected land use and land cover types of flood Zone IV.

However, the precision of simulation data which are derived from 1D Hydrodynamic depend on detail of cross section, DEM and PMF. Especially, expansion of flood from the main river depends on resolution of DEM.

Furthermore, Manning' n roughness coefficient plays an important role for water level change. In this study, it is found that when Manning' n roughness coefficient varies from 0.035 to 0.030, average water level change decrease 1.54 m. In contrary, when Manning' n roughness coefficient changes from 0.035 to 0.040, average water level change increase 1.40 m. This change directly effects to flood extent.

LULC assessment which consist of LULC types depends on data availability and data quality. Furthermore, based on derived results of affected LULC types and Flood Zone (Zone I to IV), they imply about the relationship between water flood and landform, the relationship between watershed form (upper middle and lower) and LULC.

5. Conclusion

The maximum breach outflow discharge is found 11,307 m^3/s with velocity of 5.39 m per second at 62.29 hours after dam-break with overtopping. In addition, the maximum discharge and water level with duration at selected cross sections of Khwae Yai River and Mae Klong River which are used to classify flood zone (Zone I to IV) can be summarized as following.

For Flood Zone I: KY-13 cross section of Khwae Yai River (Ban Phu Thong Maeo) the maximum discharge is 14,231.43 m^3/s and the maximum water level is +56.56 m. (MSL.) at 68.28 hours after dam-break.

For Flood Zone II: KY-24 cross section of Khwae Yai River (Ban Nuea) the maximum discharge is 14,080.67 m^3/s and the maximum water level was +41.59 m. (MSL.) at 83.21 hours after dam-break.

For Flood Zone III: MK-8 cross section of Mae Klong River (Ban Wang Khanai) the maximum discharge is 13,243.90 m^3/s and the maximum water level is +37.67 m. (MSL.) at 93.24 hours after dam-break.

For Flood Zone IV: MK-16 cross section of Mae Klong River (Ban Luk Kae) the maximum discharge is 12,046.65 m^3/s and the maximum water level is +18.91 m. (MSL.) at 107.26 hours after dam-break.

Furthermore, flood also affected 8 districts of Kanchanaburi province after dam-break included (1) Dan Makham Tia (2) Tha Muang (3) Tha Maka (4) Sai Yok (5) Bo Phloi (6) Phanom Thuan (7) Mueang Kanchanaburi and (8) Si Sawat districts. Total area of flood was 1,172.21 sq.km. and covers area of Flood Zone I to IV about 28.55, 159.50, 344.80 and 639.36 sq.km., respectively.

LULC types are affected by flood after dam-break it covers area of 1,172.21 sq.km. The most affected LULC is paddy field covers area of 331.88 sq.km. (28.32%). The second and third affected LULC are rangeland and field crop that cover area of 285.66 sq.km. (24.37%) and 267.02 sq.km. (22.78%). The least affected LULC is deciduous forest that covers area of 15.49 sq.km. (1.32%).

6. Recommendation

(1) Another type of dam-break (Piping) should be assumed for a dynamic flood prediction after dam-break. Then the result can be compared with this study to identify a worst case between overtopping and piping.

(2) High resolution Digital Elevation Model (DEM) with a horizontal spatial of 1 m in height steps of 10 cm should be applied for dynamic flood prediction instead of moderate resolution DEM.

(3) 2D Hydrodynamic model of MIKE 21 and MIKE Flood have been developed. It should be applied for increasing the precision of flood simulation data.

(4) Very high spatial resolution of remotely sensed data e.g. GeoEye, WorldView, IKONOS, OrbView-3 or QuickBird should be applied for detailed LULC classification and LULC assessment. It is very important for socio-economic damage and value loss evaluation after flood.

Acknowledgements

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