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The Applicability of Real Time Flood Forecasting Model : A Case Study in Thailand

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Summary : Severe flood damages in the southern region of Thailand caused by torrential rains or typhoons were found every year. Rainfall and water level were observed from a few monitoring stations in the region on daily basis. A real time flood forecasting model was formulated and applied to this region for flood disaster prevention. The model composed of two major components, the rainfall-runoff model and the hydrodynamic model. The rainfall-runoff model, a storage function model, was used to calculate the direct runoff due to rainfall. The hydrodynamic model, a vertically integrated model for mass and momentum conservation, was used to simulate flow in the river network. The model was able to predict water level and discharge at any location in the basin.
Comparison between observed and simulated water levels and discharges from the numerical schemes indicated the applicability of the proposed model with satisfactory results, although time interval of observed data was longer than flood discharge arriving time. However, higher accuracy of the forecasting required more frequency of the observed rainfall and water level data both in time and space. Telemetering system was also recommended for the implementation of efficient flood forecasting system.

Keywords : Real time forecasting, Flood forecasting, Rainfall-runoff model, Hydrodynamic model, Numerical analysis

1. INTRODUCTION

This study aims to set up a real time flood forecasting model for the southern region of Thailand. The area was frequently subjected to torrential rains and typhoons causing enormous casualties and damages.

Utaphao river basin, a strategy agricultural area for rubber plantations in the south of Thailand, experienced severe floods every year especially in November 1988. Klong Utaphao is the main river in the basin and drains water from Amphoe Sadao to Amphoe Hat Yai and Songkhla Lake as shown in Fig. 1. Total drainage area is 2,305 km^2, the climate of the basin is influenced by two seasonal monsoons and tropical depressions. Annual average rainfall in the basin is about 1,800 mm and the peak period is normally in November.

There were totally 10 gauged sub-basins as shown in Fig. 1. Additional 5 ungauged stations were delineated to link with the hydrodynamic model. Due to the constraint in manual recording system, rainfall and water level data were limited on daily basis and data during peak discharges or water levels were only partially available.

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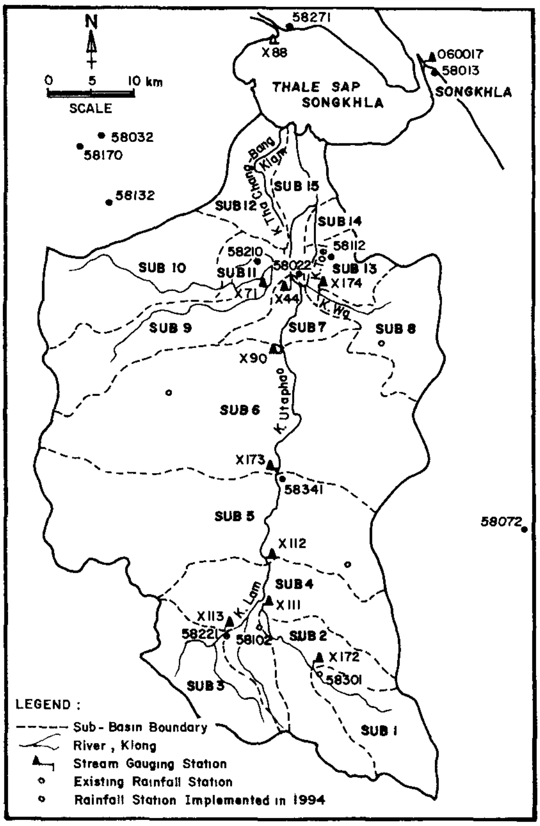

Fig. 1 Utaphao River Basin

2. RAINFALL-RUNOFF MODEL

Storage function model was selected as the rainfall-runoff model in this study. The basic equation of the model was the mass balance equation as follows;

$$\frac{dS}{dt} = I(t) - O(t) \quad \text{.......}(2.1)$$

where S = Storage, $I(t)$ = Inflow, $O(t)$ = Outflow and t = time

This model, as shown in Fig. 2, was considered as a deterministic, conceptual, lumped type model which required few input data and appropriate for simulation of rainfall-runoff process in rural catchments (Nielsen and Hansen, 1973). Numerical calculation of this equation was an explicit finite difference. The basic operation of the model was by continuously accounting four different and mutually interrelated storage tanks which represented physical elements of catchment. Output of the model were the runoff, groundwater level, temporal variation of the soil moisture content, groundwater recharge and other hydrometeorological parameters.

For model calibration, main parameters were the upper limit to the amount of water in the surface storage (U_{max}), the upper limit to the amount of water in the root zone (L_{max}), the overland flow runoff coefficient (C_{QOF}) and time constants for interflow routing through two linear reservoirs in series (CK1 and CK2). The first estimation of the model parameters was based on the general physical characteristics of soils and land uses, then trial and error method was used to determine the best fitted curve between the calculated and observed values within the feasible physical limits. Water balance, timing and peaks of the hydrograph were the major points in the consideration.

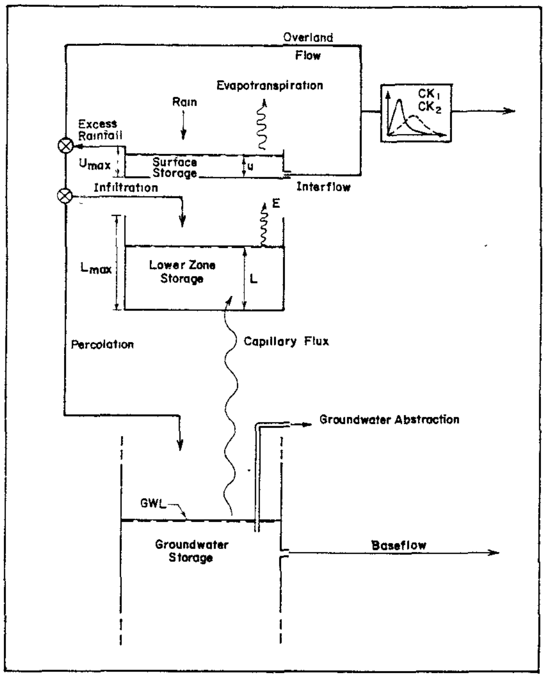

Fig. 2 Rainfall-Runoff Model

For each of the 15 sub-basins, time series of mean aerial rainfall were calculated using rainfall data from 13 stations in and nearby the basin by Theissen polygon method. The selected period of calibration and verification was from 1985 to 1992 which covered the most severe flood year, normal years and dry years as shown in Fig. 3

The calibration and verification results showed the applicability of the model in general, but there were several peaks which were apparently out of the acceptable range. This was attributed to the nonuniform distribution of the rainfall stations in the basin causing the uncertain aerial rainfall input to the model.

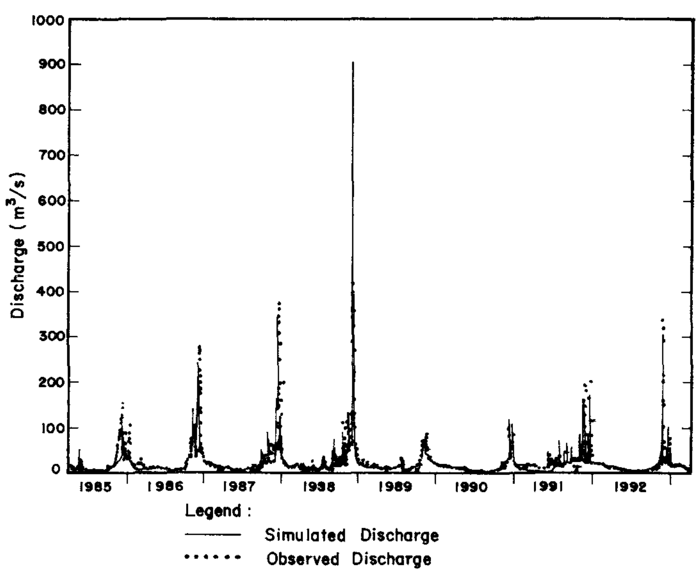

Fig. 3 Calibration for Sub-basin No. 7 at Station X-44 during 1985 - 1992

3. HYDRODYNAMIC MODEL

The governing equations for mass and momentum conservation in the model were as follows;

$$\frac{\partial(\rho Hb)}{\partial t} = -\frac{\partial(\rho Hb\bar{u})}{\partial x} \quad (3.1)$$

$$\frac{\partial(\rho Hb\bar{u})}{\partial t} = -\frac{\partial(\alpha\rho Hb\bar{u}^2 + \frac{1}{2}\rho gbH^2)}{\partial x} \quad (3.2)$$

where ρ = density, H = depth, b = width, $\bar{u}$ = average velocity, α = vertical velocity distribution coefficient, t = time and x = distance.

Integration and transformation of Eqs. 3.1 and 3.2 to a series of implicit finite difference equations in a computational grids consisting of alternating discharge and depth domains yielded the following equations;

For continuity equations :

$$\frac{\partial Q}{\partial x} = \frac{1}{\Delta 2x_j}\left[\frac{1}{2}(Q_{j+1}^{n+1} + Q_{j+1}^{n}) - \frac{1}{2}(Q_{j-1}^{n+1} + Q_{j-1}^{n})\right] \quad (3.3)$$

$$\frac{\partial h}{\partial t} = \frac{1}{\Delta t}(h_j^{n+1} + h_j^{n}) \quad (3.4)$$

For momentum equations :

$$\frac{\partial Q}{\partial t} = \frac{1}{\Delta t}(Q_j^{n+1} - Q_j^{n}) \quad (3.5)$$

$$\frac{\partial}{\partial x}\left(\alpha\frac{Q^2}{A}\right) = \frac{1}{\Delta 2x_j}\left[\frac{1}{2}\left(\alpha\frac{Q^2}{A}\right)_{j+1}^{n+1/2} - \left(\alpha\frac{Q^2}{A}\right)_{j-1}^{n+1/2}\right] \quad (3.6)$$

$$\frac{\partial h}{\partial x} = \frac{1}{\Delta 2x_j}\left[\frac{1}{2}(h_{j+1}^{n} + h_{j+1}^{n+1}) - \frac{1}{2}(h_{j-1}^{n} + h_{j-1}^{n+1})\right] \quad (3.7)$$

The methodology for the numerical determination was based on the full St. Venant's equations which involved an implicit finite difference scheme so called a 6-point Abbot-scheme as shown in Fig. 4.

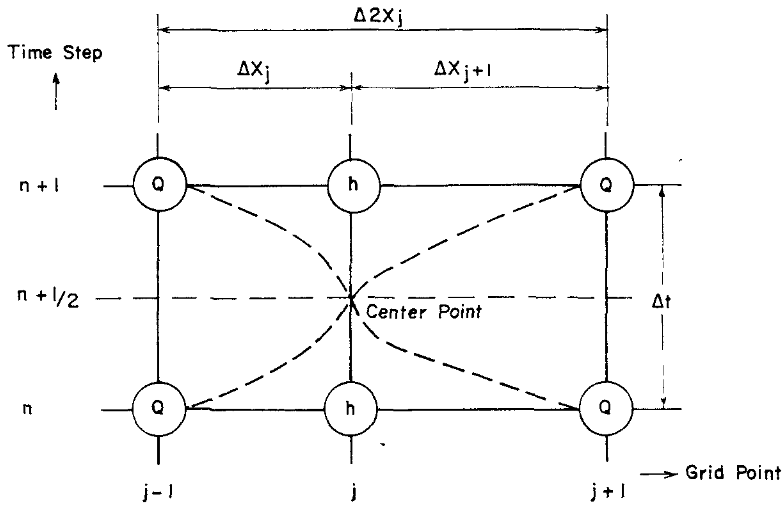

Fig. 4 Centering of Continuity Equation in 6-Point Abbott Schemes

There were 27 river cross-sections available along the main river of approximately 100 km long. A simple schematizing model which provided the most stable simulation conditions for the main river and tributaries was selected. The upstream boundary of the model was the simulated discharge from rainfall-runoff model at station X-172 while the downstream boundary was the observed water levels at station X-88 in Songkhla Lake.

The model calibration was carried out using data during the flood period from October to December in 1988 and 1991. The model was initially calibrated for only channel flow in 1991 because water levels did not exceed the embankments of the river then calibration for flood plain was done in 1988 because the high water levels exceeded the embankment. The calibration provided the variation of the roughness factor along the main channel and the relative resistance numbers on flood plains. The calibration showed a good result in general for both the magnitude and timing of peak discharge, however, only the timing of peak water level was found in a good agreement, the magnitude was out of the acceptable range for some stations. The model was verified using data at the downstream boundary station in 1986, a normal flood year with sufficient data. The magnitude and timing of peak discharge and water level were in an acceptable range as shown in Fig. 5 for station X-44.

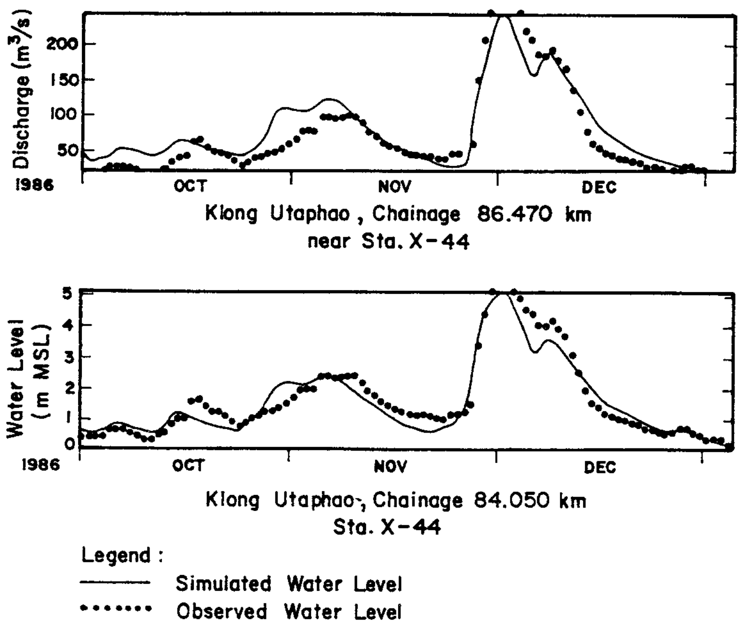

Fig. 5 Verification of Hydrodynamic Model at Station X-44 in 1986

4. REAL TIME FLOOD FORECASTING

For the accuracy of the forecasting, corrections of the errors in the simulation were done before and until the time of forecast (TOF) by an updating procedure. This procedure compared the simulation results with the real time observations until TOF, identified phase and amplitude of errors and made correction accordingly. During the forecasting period, quantitative precipitation and water levels at the model boundaries were predicted.

It was found from the calibration of updating parameters that the available daily observed discharges and water levels were not sufficient to correct the phase error due to the small size of basin yielding a quick response for rainfall. However some trial forecasts were performed for the severe flood in 1988 without using any parameters for the phase error. The results in the forecast period in comparison with the simulation results without updating revealed that the forecast water levels at Hat Yai (X-44) had no improvement when parameters for the phase error was not detected.

Sensitivity of forecasts to the quantitative precipitation forecasts (QPF) was conducted to investigate the effects of QPF on the forecast of water levels. Fig. 6 showed the forecast using the recorded rainfall in the forecast period compared to the forecasts using recorded rainfall ± 50%. It was found that the differences of forecast water levels were +0.3 m. and -0.2 m. for +50% and -50% of recorded rainfall respectively.

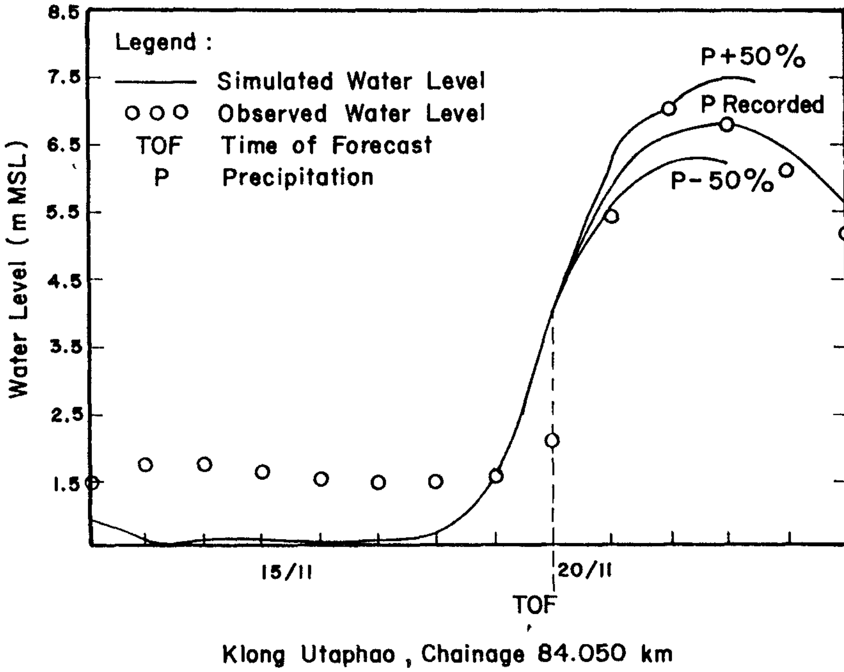

Fig. 6 Trial Forecast, without updating, using different Rainfall Forecast (Time of Forecast (TOF) was on November 20, 1988)

5. CONCLUSIONS

The applicability of the proposed model was found with satisfactory results. The deviation between the simulated and observed discharges and water levels in many situation was mainly attributed to the uncertain mean aerial rainfall input. Because the interval of the recorded data were longer than the discharge arriving time which was about 6 hours, parameters for the phase error could not be detected.

It was recommended to install more rain gauges in the basin. Frequency of the observation for discharges and water levels should be improved from the present daily basis to 6-hourly or even better. This could be accomplished by the implementation of a real time Telemetering system in the basin.

6. REFERENCES

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