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Integration of Spatially Hydrological Modelling on Bentong Catchment, Pahang, Peninsular Malaysia Using Distributed GIS-based Rainfall Runoff Model

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Abstract

With the advance of GIS technology, hydrology model can simulated at catchment wide scale. The objective is to integrate National Resource Conservation Service (NRCS) Curve Number (CN) with kinematic wave and manning's equation using GIS to develop a simple GIS-based distributed model to simulate rainfall runoff in Bentong catchment. Model was built using Spatial Distributed Direct Hydrograph (SDDH) concept and applying the time area (TA) approach in presenting the predicted discharge hydrograph. The effective precipitation estimation was first calculated using the NRCS CN method. Then, the core maps that consists of digital elevation model (DEM), soil and land use map in grid. DEM was used to derive slope, flow direction and flow accumulation while soil and land use map used to derive roughness coefficient and CN. The overland velocity and channel velocity estimation derived from combination of kinematic wave theory with Manning's equation. To capture the time frame, the travel time map was divided into isochrones in order to generate the TA histogram and finally. The creation of SDDH using the TA histogram which will lead to the estimation of travel time for the catchment. Simulated hydrograph was plotted together with the observed discharge for comparison. Six storm events used for model performance evaluation using statistical measure such as Nash-Sutcliffe efficiency (NSE), percent bias (PBIAS) and coefficient of determination (R^2). SDDH model performed quite well as NSE gave result ranging from 0.55 to 0.68 with mean of 0.6. PBIAS indicate that the model slightly over predicted compared to observed hydrograph with result ranges from -46.71 (the most over predicted) to +4.83 (the most under predicted) with average of -20.73%. R^2 ranges between 0.55 to 0.82 with mean of 0.67. When comparing the time to peak, (tp), min, and peak discharge, (pd), m^3/s, results gave NSE_{tp} 0.82, $PBIAS_{tp}$ 0.65, R^2_{tp} 0.32, NSE_{pd} 0.95, $PBIAS_{pd}$ 14.49 and R^2_{pd} 0.98, respectively. Results indicate that the integrated distributed model is successfully applied to the catchment.

Keywords: rainfall runoff; NRCS CN; GIS; spatial distributed; time area

1. Introduction

Since hydrologic variables are varied within a catchment and it heavily involved with topographic factor, the usage of GIS as a tool in modeling is a great hand for modelers. The capability of GIS to handle and manipulate large amount of data is the main strength. Furthermore, there are more computerized data available for modelers to use them incorporated with remotely sensed type of data to perform a GIS based hydrologic models. Often, hydrologist or scientist faces problem in modelling a catchment or basin wide area as the problems ranges from the insufficient of data which cost by either insufficient gauging stations or data loses due to broken equipment to the high cost to develop and produce a rainfall runoff model. It is a common problems to having such limited gauging stations to capture the observed data collection such as precipitation or discharge data that crucial in rainfall runoff modelling. There is a crucial need to get result from a capable and trusted model for the ungauged area or area that having insufficient data. However, there a hydrologic models that offer solution for modeling in ungauged watershed such as National Resources Conservation Service Curve Number (NRCS CN). Nonetheless, it is strongly advised to perform calibration and validation towards the ungauged watershed models before applying it to any projects or studies. Therefore, a GIS-based hydrologic model by using Spatial Distributed Direct Hydrograph

(SDDH) method proposed here can be a big hand to environmentalist, engineers or planners as they can use it for any development planning in the study area. Previously, many hydrological modelling are based on empirical in lumped model condition. With the advance in GIS technology, it can easily be incorporated with hydrological model especially in rainfall-runoff modelling. In lumped model, the parameters required are being averaged and lumped into one model simulation. In real world, it is not particularly being able to capture and represents the hydrological process occurred within a catchment or basin-wide basis. Then came the concept of Hydrological response Unit (HRU) which divide one catchment into several sub-catchment but it is still in lumped modelling mode.

National Resources Conservation Service Curve Number (NRCS CN) method developed by United States Department of Agriculture. The model was easy to apply and it has been proven in many modelling applications in predicting the excess rainfall such as in Agricultural Non-Point Source Management Model (AGNPS) (Young *et al*., 1985); sediment transport model (Her and Heatwole, 2010); mapping of flood hazard area (Diakakis, 2011); soil loses and erosion (Huang *et al*., 2006) and many other models. The NCRS CN model provides an empirical relationship to estimate initial abstraction (I*a*) and runoff (*Q*)as a function of soil class and land use (Pradhan *et al*., 2010). The rainfall runoff relationship in this model can be interpreted by several main factors such as initial abstraction (*Ia*), direct runoff (Q) or effective precipitation (*Pe*) and actual retention (F). The curve number (CN) is basically an index developed to represent the storm water runoff within a catchment or drainage area. Combination of land use and soil type and antecedent soil moisture condition (AMC) was used to estimate the CN. The number for CN ranges from 0 to 100 which indicate the degree of water to flow to become direct runoff as the result of excess precipitation. Previously, the traditional method to create CN is taking a lot of work and time but nowadays, with the advantages of GIS technology, the process of creating CN can be done in more easy way.

Time area (TA) method is a popular method in hydrology to plot discharge versus time hydrograph. It is a graph that shows the cumulative drainage area that contributes to runoff during time and is derived from the sum of the incremental sub-areas of the watershed (Bourletsikas *et al*., 2006). The result interpretation of TA technique is very close to distributed modelling as it capable in the sense of calculating the differences of arrival time of flow for each sub catchment to the catchment outlet (Gad, 2013). Meanwhile, according to Saghafian *et al*., (2002), the TA method has the capability to act as a distributed model by combining the non-uniform excess rainfall and spatially variable watershed characteristic. The main component of TA method is the time contour or known as isochrones. It is the main basic idea to split the watersheds or catchment into specific zones depending on the time required for the water to flow downwards to catchment outlet (Nash and Sutcliffe, 1970; Singh, 1996; Matei, 2012; Odeh *et al*., 2015).

In Malaysia, flash floods is a common problem especially in the rainy season (Sulaiman *et al*., 2010; Sulaiman *et al*., 2017). This study intend to utilize the capability of GIS in performing a fully distributed hydrological based model using the combination of kinematic wave approximation and manning's equation with the time-area method to produce hydrographs. NCRS CN method will be applied to produce the effective precipitation runoff input. Our intention is to model the rainfall runoff in a real extreme event that might happen within the Bentong catchment as the area was prone to be having a flash flood as a result of heavy intense rainfall (Al-Mamun *et al*., 2008). This model basically relied on the capability of GIS to perform a spatially distributed hydrologic model using several event based rainfall to simulated the rainfall runoff.

2. Materials and Methods

2.1 Study area

Bentong catchment is located in Pahang River Basin which is the largest river basin in Peninsular Malaysia. Bentong catchment can be divided into main catchment but in this study, it was break down to eight sub catchments as shown in Fig. 1. The elevation from DEM shows that the height above mean sea level range from 10 to 2660 meter. Most of the catchment area with more 50% of total area (203 km^2) are covered with forest area. There are five main river in the catchment which Bentong River, Perting River, Chamang River, Repas River and Panjuring River. Bentong catchment received average annual rainfall of 2400 mm (Al-Mamun *et al*., 2008). During these past few years, flood has become more frequent and destructive within Bentong catchment. Heavy precipitation either it is monsoon or convective is known to be the basic cause that trigger the flood to occur which resulted large concentration of runoff, which exceeds river capacity. In recent years, as a result of rapid and uncontrolled development within river catchment, the runoff has increased and deteriorated river capacity; this has also in turn resulted in an increase in the flood frequency and magnitude (Al-Mamun *et al*., 2008).

2.2 Data

In this study, the main source of watershed spatial background data used is the Digital Elevation Model (DEM) which represents the elevation point over an area gathered from SRTM at grid size of 30m x 30m. Topographic over a basin played major contribution to flood hydrology. Previously, the elevation data always depends on contour map but through advance of technology, an alternative was being provided by using digital terrain model that can be directly used with geographic information system (GIS). By using this tools, the speed and efficiency over a basin wide area can be increased. In this study, Shuttle Radar (SRTM) was being used as the main source of elevation data provider. It was downloaded for the entire watershed. The resolution of this DEM is 3 arc seconds in the geographic projection (WGS84 datum) which then re-project to the Rectified Skew Orthomophic (RSO) projection used in Malaysia. Fig. 1 shows the DEM of Bentong watershed.

Some of the main output from the DEM extraction such as flow direction and flow accumulation will be used to create other input such as channel network, flow length and the velocity estimation of the watershed. Flow direction will be able to determine the direction of flow by finding the direction of steepest descent from each cell in the grid using the capabilities of GIS. Meanwhile, the flow accumulation will calculate the number of upstream cells that flow into each cell. The process of watershed attribute extraction was done in GIS using the extension of ArcHydro (Zhang *et al*., 2010).

Besides, channel-related attributes, the other main input into the model is land use map. The map will be processed to create the CN value and Manning's roughness coefficient which will be used in the estimation of velocity. In this study, a land use map of year 2000 was used gathered from the Department of Agriculture of Malaysia (DOA), and digitized as shown in Fig. 2.

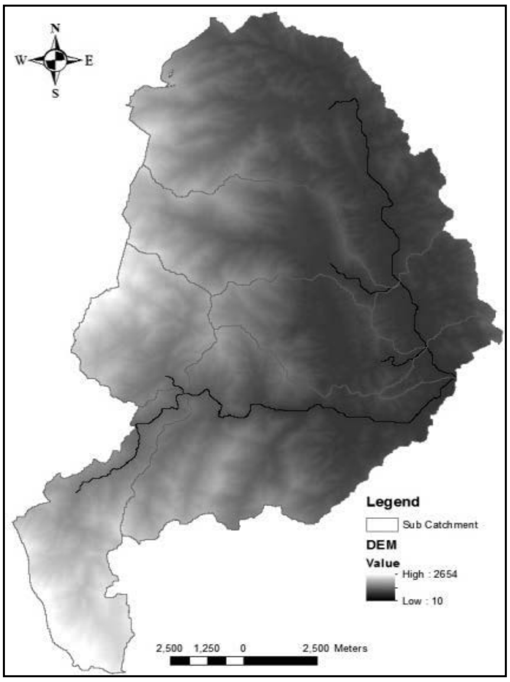

Figure 1. DEM of Bentong catchment

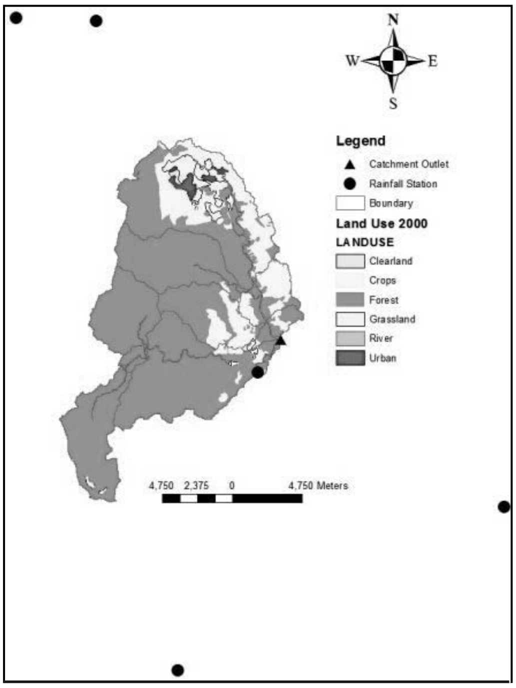

Figure 2. Year of 2000 land use map of Bentong with rainfall station and catchment outlet

2.3 Time area concept

The TA methods were developed in by considering the importance of the time distribution of rainfall on runoff (Singh, 1992). According to Kull and Feldman (1998), nowadays in 40 to 60% of Corp of Engineers (USA) projects, TA method is used as a rainfall-runoff model. The basic idea of these methods is the time-area histogram, which indicates the distribution of partial watershed areas contributing to runoff at the watershed outlet as a function of travel time (Ajward and Muzik, 2000). These areas are bounded by specific time period or known as isochrones. Since the time-area diagram is a graph of cumulative watershed area whose time of travel is less than or equal to a given value, i.e: t = iΔt, where i = 1, 2 ... n, plotted against the value of t (Saghafian *et al.*, 2002; Shokoohi, 2008; Soulis *et al.*, 2015). Mathematically, the general equation of TA method which gives a net hydrograph due to an effective rainfall is as follows:

$$Q_j = \sum_{k=1}^{j} I_k A_{j-k+1} \qquad \text{Eq.1}$$

Where:
j = time step
Q = discharge
I = effective rainfall intensity
A = area between two consecutive isochrones

2.4 NRCS CN Method

The NRCS rainfall-runoff relationship is given by the following equation:
where:

$$P_e = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{Eq.2}$$

where *Pe* is the effective rainfall or runoff depth (mm) at time-step t, *Ia* is the initial abstraction (mm) which is defined as 0.2S, P is the total rainfall (mm) for the storm event, S is the depth of effective available storage within the catchment (mm). The parameter S describes how fast a watershed saturates and starts producing runoff and was physically equal to maximum available storage. The equation of I_a can be written as given by NCRS standard value

$$I_a = 0.2S \qquad \text{Eq.3}$$

And therefore, equation 4 can be written as

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S} \, where \, (P > 0.2S) \qquad \text{Eq.4}$$

This is the simplify version of NRCS CN equation although there are few attempt to modified the equation such as by adding the slope factor to the

CN equation (Huang *et al*., 2006) or adjustment on the I_a input (Akbari *et al*., 2016) and adding the effect of soil moisture accounting into the standard equation (Michel *et al*., 2005). However, in this study the standard value (0.2) was used. The P_e (mm) is in grid form at 30m * 30m. The watershed storage factor, S, can be calculated as

$$S = \frac{25400}{CN} - 254 \quad \text{Eq.5}$$

In NRCS CN, the selection of curve number is based on antecedent moisture content. Each AMC represents the storm event condition which NCRS provided three level of AMC which AMC I, AMC II and AMC III respectively. In easy way, in this study, the selection of AMC judged by the season of the storm event occurred. AMC I represent the dry season, AMC II represent the normal condition while AMC III represent the wet season. Taking the CN II lookup table as the base of the calculation, AMC I and III can be calculated as

$$CN(I) = \frac{4.2CN(II)}{10-0.058CN(II)} \quad \text{Eq.6}$$

$$CN(III) = \frac{23CN(II)}{10+0.13CN(II)} \quad \text{Eq.7}$$

Taking the AMC rules from the NRCS (2002), all event used in this study considered as AMC III except for event #1 which dated on 17.02.2000 classified as AMCI.

2.5 Rainfall events

In this study, six rainfall events will be used to be used in the model as shown in Table 1. There is only one rainfall station located exactly in the Bentong catchment. To gather more rainfall input, rainfall reading from nearest station also been used. There were extra four rainfall stations outside of the catchment to be used for rainfall interpolation to get the total rainfall for the whole catchment on selected events. The total rainfall at each station were then interpolated using Inverse Distance Weight (*IDW*) using GIS spatial interpolation.

Then, based on equation 4, the *Pe* can be calculated using the interpolated total rainfall for each event. After that, the interpolated effective precipitation grid will be created by diving the grid of effective rainfall with duration of rainfall event in (second, *s*) as shown in Table 1 to finally produce the rainfall intensity grid then be used to create the rainfall intensity grid, *Ie* (m/s). Mathematically, rainfall intensity be written as

$$I_{e\,Pe} = \frac{P_{e\,Grid}}{t} \quad \text{Eq.8}$$

The six events were analyzed on a 15 min time-step. To serve the purpose of this study, the main criteria for event selection is that all rainfall station must experience rainfall on that particular storm date, thus, if any rainfall station recorded no rainfall on that event, the event on that day is neglected. As the result, only six events in year 2000 provided sufficient input for the purpose of this study. Table 1 shows the observed rainfall characteristic and discharge at the catchment outlet. The min, max and mean rainfall are gathered through interpolation of five rainfall station. Rainfall used as the input to create effective rainfall estimation while discharge used in the validation of SDDH model.

To test the accuracy of the model, the simulated results were compared to the observed values from the gauging station which located in Kuala Marong, Bentong.

2.6 Baseflow separation

The baseflows were separated to retrieve direct runoff hydrographs from total streamflow records of selected storm events. For these separation, the straight line method and the recursive digital filter method (Eckhardt, 2005) were first applied. Then after conducting several cases of model calibration with these different baseflow separation methods, the final reasonable baseflow removal method was selected on the basis of calibration results. Equation 9 shows the formula of recursive digital filter which uses two parameters of the recession constant, a (0.980 or 0.995),

Table 1. Characteristics of the observed rainfall events and discharge that used as input for model development and validation process, respectively.

Date	Rainfall					Discharge	
	Min (mm)	Max (mm)	Mean (mm)	Duration (s)	AMC	Peak (m^3/s)	Time to peak (min)
17.02.00	22.30	31.10	29.03	13500	I	10.52	330
24.03.00	32.10	69.30	51.31	35100	III	16.47	405
26.04.00	21.03	40.10	30.26	18900	III	25.29	300
17.11.00	29.42	52.00	42.69	14400	III	31.92	405
19.12.00	26.52	61.70	45.35	14400	III	17.18	390
22.12.00	44.94	93.19	56.88	28800	III	76.04	480

and the maximum value of the baseflow index, BFImax (0.80; for perennial streams with porous aquifers and 0.50; for ephemeral streams with porous aquifers). Also, for the practical uses of this recursive digital filter method, the Web based Hydrograph Analysis Tool (WHAT) system which provides an efficient tool for hydrologic model calibration and validation (Lim *et al*., 2005) was used.

$$b_t = \frac{(1-BFI_{max})*\alpha+b_{t-1}+(1-\alpha)*BFI_{max}*Q_t}{1-\alpha*BFI_{max}} \qquad \text{Eq.9}$$

Where b_t is the filtered base flow at the t time step; b_{t-1} is the filtered base flow at the $t-1$ time step; BFI_{max} is the maximum value of long term ratio of base flow to total streamflow; α is the filter parameter; and Q_t is the total streamflow at the t time step.

2.7 Development of Spatial Distributed Direct Hydrograph (SDDH)

The Bentong catchment will be divided into several travel time zones. Each zone represents the part of catchment, which drains the unit excess rainfall to the outlet at specific time interval. In this spatial distributed model, there are two types of flow that are considered for a defined stream network flow which are overland flow and channel. All of the spatial grid calculation used in this study were done using Raster Calculator function in ArcGIS 10.3. The extent of all required grid were set and maintained to 30m * 30m.

In this study, the travel time method will be using the basis of Manning's roughness coefficient. The roughness coefficient prepared from the land use map of Bentong catchment in the year of 2004. The manning's coefficient value that used were from the suggested value by Engman (1986) and Chow *et al.* (1998). This study used manning's velocity equation to compute and prepares the velocity grid. The grid will represent the velocity of flow in each cell. The runoff velocity for areas with overland flow can be estimated using a kinematic wave approximation. Overton and Meadows (1976) had previously given the depth of flow at equilibrium as an Eq.10. The detail mathematical equation derivation of overland flow travel time by applying the kinematic wave approximation of the momentum equation and the continuity equation is well explained in Melesse (2004).

$$y = \left(\frac{I_{e\,Pe}\,nx}{\sqrt{S_o}}\right)^{0.6} \qquad \text{Eq.10}$$

Where:
y=depth of runoff flow at equilibrium (m)
i_e=effective rainfall excess intensity (m/s)
n=Manning's roughness coefficient
x=distance along the flow plane (m)
S_o=slope (m/m)

Then, by applying the manning's equation, the overland velocity can be calculated as

$$V_o = \frac{(I_{e\,Pe}\,x)^{0.4}\,S_o^{\;0.3}}{n^{0.6}} \qquad \text{Eq.11}$$

Where V_o is overland flow velocity in (m/s)

The distance along the flow plane was estimated based on the distance of each grid cell from the closest ridge cell. In GIS it can be calculated simply using the Spatial Analyst function. For cells identified as ridge cells (known as factor x in Eq.8), the distance was assigned to be half the length of a grid cell to be at 15 m (Kilgore, 1997). However, a value of 42 m for the ridges (x) factor was implemented based on the grid size used in this study. All grid cells size in this study are set to be the same at 30 m * 30 m as the DEM size was also in the same size. Based on the grid, the longest length in the grid is hypotenuse, c which follow the Pythagorean Theorem rules; *hypotenuse*, $c = \sqrt{a^2+b^2}$, a is height of grid and b is the length which both are 30 m. The land use of Bentong sub basin was used to estimated the Manning's roughness coefficient for the overland cells. The input of i_e, (m/s) is effective rainfall precipitation intensity that already calculated using Eq.10. The input of S_o, (m/m) can be produced from the DEM using the Surface Analysis in ArcGIS.

Then, the second type of velocity that need to be consider is the channel flow velocity, V_c (m/sec). It is computed using the continuity equation for a wide channel and manning's equation. For channel flow, an assumption was made. The characteristic of the open channel flow is assumed n the wide channel and the hydraulic radius is approximated by the depth of flow assuming the depth of flow is much smaller than the channel width. The detail explanation derivation of the equation of 14 to 16 is well explained in Melesse (2004). The equation for channel flow velocity, manning's equation and continuity equation for wide channel shown as below

Manning's equation

$$V_c = \frac{S_o^{\;1/2}}{n}y^{2/3} \qquad \text{Eq.12}$$

Continuity equation for wide channel

$$Q_c = V_c By \qquad \text{Eq.13}$$

Then, by combining equation 1 and 2, the channel flow velocity can be calculated as

$$V_c = \frac{{S_o}^{0.3}.{Q_c}^{0.4}}{n^{0.6}.B} \qquad \text{Eq.14}$$

Where y is the depth of flow (m), Q_c is the cumulative discharge (m^3/s), B is the channel width (m) while n and S_o are defined as in equation 1. Channels are assumed to have a rectangular cross-section with y depth of flow and an effective width, (B) of 1 m on average value. The calculation of discharge are determined from the total inflow into the cell, i.e., the rainfall excess intensity of a cell and inflows from upstream cells are summed and multiplied by cell grid size ($900m^2$).

$$Q_c = I_{e_{Pe}}.flow\ accumulation\ grid.\ cell\ size\ (900m^2) \qquad \text{Eq.15}$$

Where I_e is rainfall intensity in (m/s) as in Eq.10 and the cell size is 30m*30m (grid size used in this study).

The total velocity is the combination of overland velocity and channel velocity and in GIS, it can be written as

$$V_{total} = V_o(from\ Eq.13) + V_c(from\ Eq.16) \qquad \text{Eq.16}$$

The travel of time of each cell to the outlet can be calculated using the flow length function in GIS.

2.8 Time-area diagram (Isochrones)

Relationship between travel time and area in watershed can be presented by the time area diagram (TAD). TAD can be produced by the cumulative travel time for each cell to the outlet of the watershed. To achieved that, first, the total of travel time along the respective flow path that follows the flow direction; a cumulative travel time map is gathered. The arithmetic procedure explained in above section will handle this task (the flow length function). Flow length function available in the Hydrology Tools in ArcToolBox using ArcGIS 10.3 and the main required input to perform this task is flow direction. To address the effect of land use towards the discharge prediction, the total of velocity in Eq.18 applied in the flow length procedure as the weight as in Eq.19

$$Weight_{Vtotal} = \frac{1}{V_{total}} \qquad \text{Eq.17}$$

Where V_{total} as shown in Eq.18.

Then, the base time or the time interval isochrones selected for analysis is justified and it will produced the TAD. In this study, a 15-minutes time interval was selected for all storm event. The flowchart of the work procedure of SDDH model as shown in Fig. 3.

2.9 Model performance evaluation

To test the reliability between simulated and observed hydrographs, the three statistical models used to perform the job. They are Nash-Sutcliffe efficiency (NSE), percent bias (PBIAS) and coefficient of determination (R^2). NSE is a common statistical measure to assess accuracy of a hydrograph. It is a normalized statistic that determines the relative magnitude of the residual variance compared to the measured data variance (Nash and Sutcliffe, 1970). In this study NSE was used to specify how well the plot of observed versus simulated data to fit the 1:1 line. The equation of NSE shown in Eq.18.

$$NSE = 1 - \left[\frac{\sum_{i=1}^{n}\left(Y_i^{obs} - Y_i^{sim}\right)^2}{\sum_{i=1}^{n}\left(Y_i^{obs} - Y^{mean}\right)^2}\right] \qquad \text{Eq.18}$$

Where Y_i^{obs} is the ith observation for the constituent being evaluated, Y_i^{sim} is the ith simulated value for the constituent being evaluated, Y^{mean} is the mean of observed data for the constituent being evaluated, and n is the total number of observations.

The range of NSE is from $-\infty$ and 1.0 (1 inclusive). For NSE the most optimal value is equal to 1.0. Values between 0.0 and 1.0 are generally viewed as the level of acceptability of performance by the model but this strictly depends on the objective of specific study. Values that indicate below or equal to zero ($x \leq 0.0$) indicate that the value of observed mean is a better predictor than the simulated value and it can be defined as the simulated model result is not an acceptable performance. Many hydrologic models used NSE as their performance indicator such as Ajmal *et al.* (2016); Her and Heatwole (2010); Patil *et al.* (2008) and many more. According to Moriasi *et al.* (2007), a particular model is considered satisfactory if NSE>0.5.

Percent bias or PBIAS also used in this study to measure the average tendency of the simulated data to be larger or smaller than their observed counterparts (Gupta *et al.*, 1999). The optimal value of PBIAS is 0.0. According to Gupta *et al.* (1999), positive value indicate model underestimation bias while negative values indicate model overestimation bias. PBIAS can be calculated using equation 19.

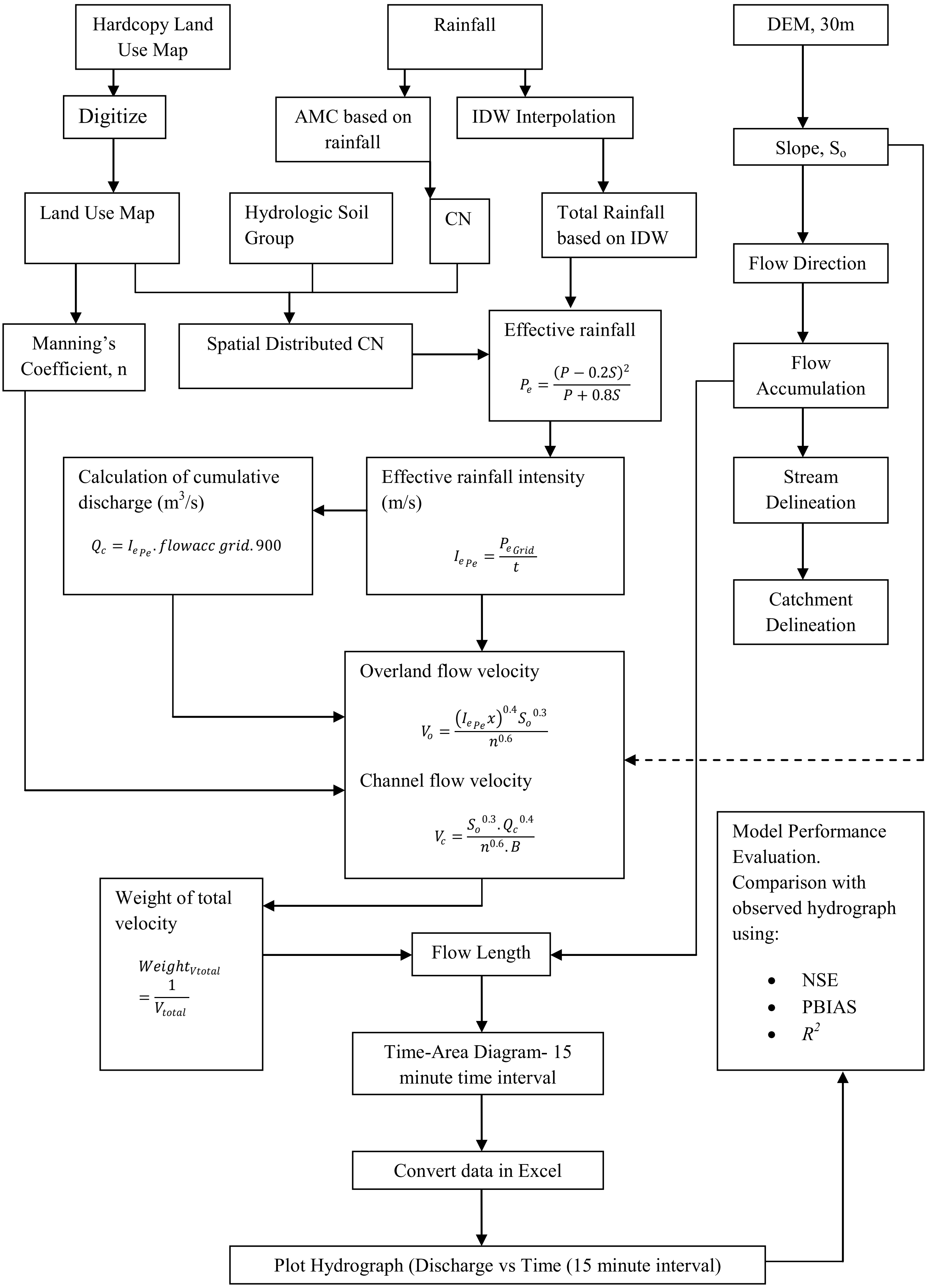

Figure 3. Flowchart of deriving the SDDH model to produce hydrograph

$$PBIAS = \left[\frac{\sum_{i=1}^{n}\left(Y_i^{obs} - Y_i^{sim}\right)*(100)}{\sum_{i=1}^{n} Y_i^{obs}}\right] \quad \text{Eq.19}$$

Where PBIAS is the deviation of data being evaluated, expressed as a percentage. PBIAS was selected

The last statistical measure used to evaluate the model performance in this study is coefficient of determination (R^2) which can be described as the degree of co-linearity between simulated and observed data (Moriasi *et al*., 2007). It ranges from -1 to 1. According to (Santhi *et al*., 2001; Van Liew *et al*., 2003), high values in R^2 indicate the model had less error variance with values of 0.5 or greater than 0.5 normally considered as acceptable.

3. Results and Discussion of SDDH Model

In this study, a fully usage of distributed GIS was applied to Bentong catchment to finally produce a TA map that can be translated into discharge (m^3/s) vs time (15 minute interval). All of the calculation were based on grid size of 30 meter for all parameter input. The estimation of the effective precipitation was done using NCRS CN method and the base flow of observed data corrected with the recursive digital filter method (Eckhardt, 2005). The Time area class was then from the TAD and converted into simulated discharge Vs Time by multiplying the class with mean of effective precipitation for each storm event. Finally, the simulated hydrograph will be compared to the observed hydrograph to validate the accuracy of the model. The final output of the study is the time-area map based on 15 minutes isochrones. It justify the travel time taken for water to flow through each grid cell towards the stream. A 15 minutes time interval was used to classify the time area histogram map and later transform to a Spatial Distributed Direct Hydrographs (SDDH). This study will focused on the preliminary test on the model on the six rainfall events as shown in Table 1. The manning's coefficient value, distance to ridges and slope input are kept the same for the six events while the input of rainfall intensity varies based on the storm amount and duration. The performance of the model was evaluated using three statistical (Nash-Sutcliffe model efficiency, percent bias and coefficient of determination) by comparing the hydrographs between observed and simulated result. Prediction of peak discharge and time to peak were used for comparison.

3.1 Simulated SDDH VS observed hydrograph

The comparison of hydrograph was done for six rainfall events. Figs. 4 to 9 show the comparison of simulated and observed discharge based on six rainfall events.

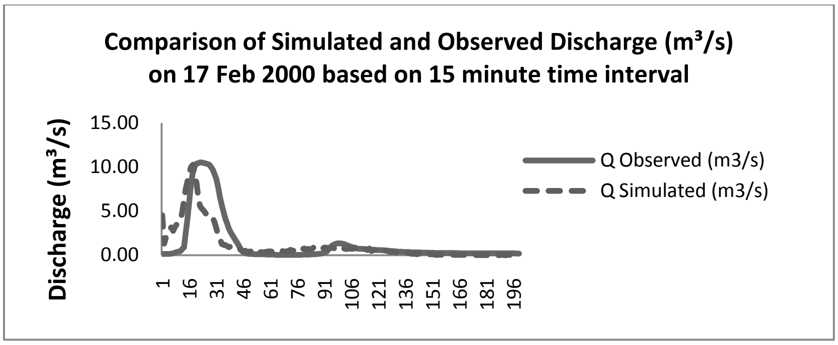

Figure 4. Comparison between simulated and observed hydrograph on 17 Feb 2000 event

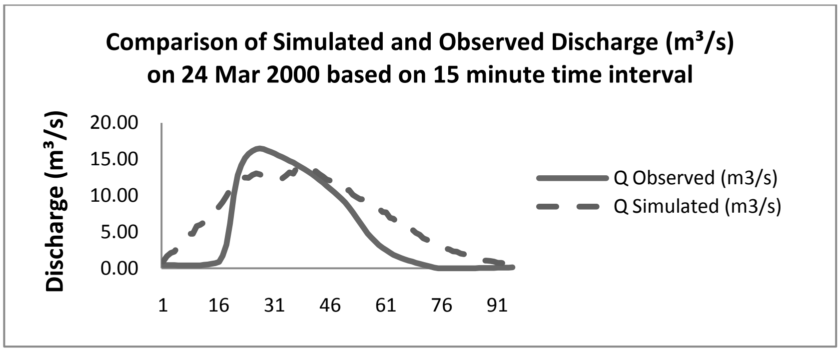

Figure 5. Comparison between simulated and observed hydrograph on 24 Mar 2000 event

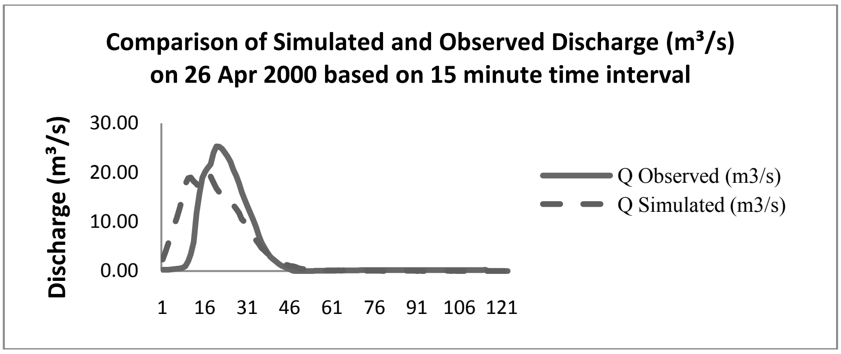

Figure 6. Comparison between simulated and observed hydrograph on 26 Apr 2000 event

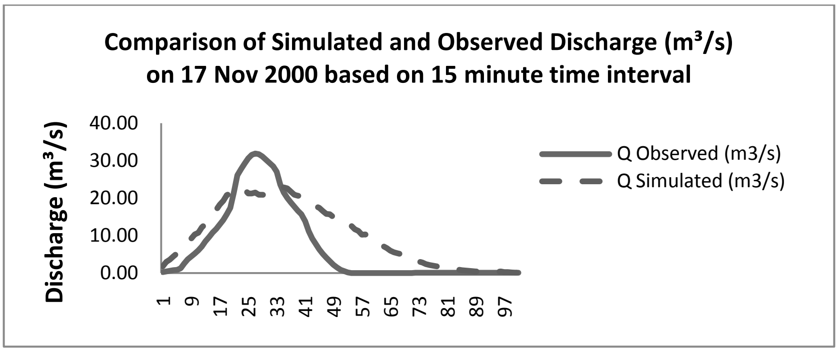

Figure 7. Comparison between simulated and observed hydrograph on 17 Nov 2000 event

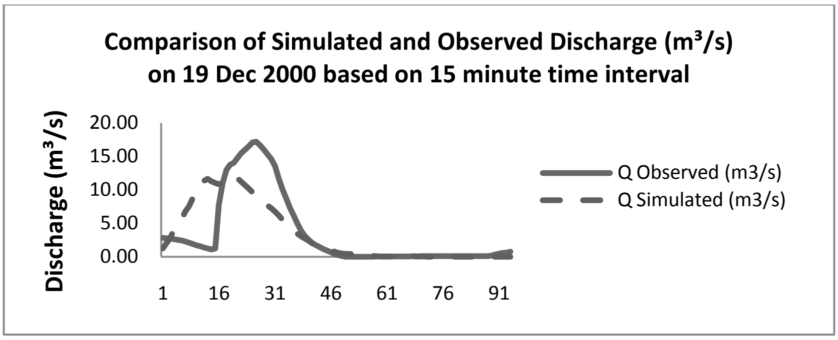

Figure 8. Comparison between simulated and observed hydrograph on 19 Dec 2000 event

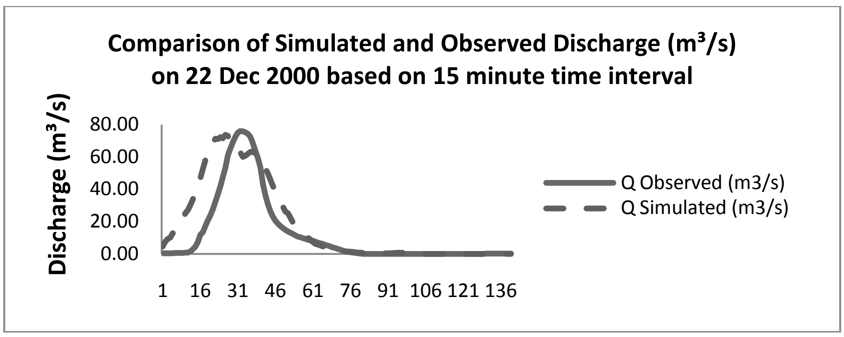

Figure 9. Comparison between simulated and observed hydrograph on 22 Dec 2000 event

3.1.1 Storm #1 (17 February 2000)

The storm #1 has a minimum of 22.30 mm, maximum 33.10 mm and with mean of 29.03 mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 330 min and 10.52 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #1 gave a result of NSE=0.55, PBIAS=2.41 and R^2=0.56. The simulated time to peak and peak discharge are both slightly under predicted with prediction errors of 18.18% and 2.4%, respectively. The hydrograph for this event shown in Fig. 4. Hydrograph showed that the simulated discharge predicted an early time to peak about 60 minute from the observed time to peak.

3.1.2 Storm #2 (24 March 2000)

The storm #2 has a minimum of 32.10 mm, maximum 69.30 mm and with mean of 51.31mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 405 min and 16.47 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #2 gave a result of NSE = 0.68, PBIAS = -36.09 and R^2 = 0.82. The simulated time to peak and peak discharge are both slightly over predicted and under predicted with prediction errors of -37.04% and 15.78%, respectively. The hydrograph for this event shown in Fig. 5. Hydrograph showed that the simulated discharge predicted a late time to peak about 150 minute from the observed time to peak.

3.1.3 Storm #3 (26 April 2000)

The storm #3 has a minimum of 21.03 mm, maximum 40.10 mm and with mean of 30.26 mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 300 min and 25.29 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #3 gave a result of NSE = *0.64*, PBIAS = -3.87 and R^2 = 0.64. The simulated time to peak and peak discharge are both slightly under predicted with prediction errors of 15.00% and 22.82%, respectively. The hydrograph for this event shown in Fig. 6. Hydrograph showed that the simulated discharge predicted an early time to peak about 45 minute from the observed time to peak.

3.1.4 Storm #4 (17 November 2000)

The storm #4 has a minimum of 29.42 mm, maximum 52.00 mm and with mean of 42.69 mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 405 min and 31.92 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #4 gave a result of NSE= 0.60, PBIAS = -44.92 and R^2 = 0.70. The simulated time to peak is over predicted and peak discharge found under predicted with prediction errors of -29.63% and 28.36%, respectively. The hydrograph for this event shown in Fig. 7. Hydrograph showed that the simulated discharge predicted a late time to peak about 120 minute from the observed time to peak.

3.1.5 Storm #5 (19 December 2000)

The storm #5 has a minimum of 26.52 mm, maximum 61.70 mm and with mean of 45.35 mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 390 min and 17.18 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #5 gave a result of NSE = 0.55, PBIAS = 4.83 and R^2= 0.55. The simulated time to peak and peak discharge are both found under predicted with prediction errors of 23.08% and 29.39%, respectively. The hydrograph for this event shown in Fig. 8. Hydrograph showed that the simulated discharge predicted an early time to peak about 90 minute from the observed time to peak.

3.1.6 Storm #6 (22 December 2000)

The storm #6 has a minimum of 44.94 mm, maximum 93.19 mm and with mean of 56.88 mm of rainfall respectively based on the interpolation of rainfall station using IDW. The observed time to peak and peak discharge on that event are 480 min and 76.04 m^3/s, respectively, Based on SDDH model, the overall performance on Storm #6 gave a result of NSE = 0.61, PBIAS = -46.71 and R^2 = 0.76. The simulated time to peak and peak discharge are both found under predicted with prediction errors of 18.75% and 3.23%, respectively. The hydrograph for this event shown in Fig. 9. Hydrograph showed that the simulated discharge predicted an early time to peak about 90 minute from the observed time to peak. A summary of results from this study shown in Tables 2, 3 and 4, respectively.

Table 2. Summary of SDDH model performance via statistical test

Event	Nash-Sutcliffe	Percent Bias	Coefficient of Determination
	E	PBIAS	R^2
17.02.2000	0.55	2.41	0.56
24.03.2000	0.68	-36.09	0.82
26.04.2000	0.64	-3.87	0.64
17.11.2000	0.60	-44.92	0.70
19.12.2000	0.55	4.83	0.55
22.12.2000	0.61	-46.71	0.76
Mean	**0.61**	**-20.73**	**0.67**

3.2 Discussion on SDDH Model

SDDH model was developed by a deterministic type of modeling using distributed GIS to solve the governing equation used in developing the model. Numerous equation used in this study but main equation used to develop the model are based on eq.11 to create overland velocity grid, eq.14 to create channel velocity grid, eq.15 to create cumulative discharge grid and eq.16 to create total velocity grid and eq.17. Final calculation to create the travel time grid was performed using the flow length function using weightage from eq.17. Calculations were made in grid form using raster calculator function in ArcGIS 10.3. Then using the time area concept, the histogram from the time travel grid was reclassified into 15 minutes interval in Ms Excel to create the simulated hydrograph for each different rainfall event. The simulated hydrographs were evaluated using NSE and PBIAS as in eq.18 and 19, respectively. Based on the summarize results, Storm#1 and #5 produce the lowest NSE with both having 0.55. The best NSE came from Storm #2 with 0.68 and R^2 = 0.82. Even with high accuracy of R^2, Storm #2 found to be over predicted when compared to observed hydrograph with PBIAS = -36.09. Surprisingly, Storm #2 also appeared to be having the longest rainfall duration compared to other storm with mean intensity of 0.00146 mm/s. Furthermore, the worst prediction also came event that had the highest intensity which is Storm #5 with average rainfall intensity of 0.00315 mm/s. However, although Storm #5 produced the worst in terms of NSE, it did produce a third best place in terms of percentage of bias with (after Storm #1 and Storm #3, respectively) with 4.83%. The possible explanation of this finding is on the NRCS CN model parameterization itself. Based on the study catchment, it can justified that this catchment can be categorized as a hilly forested catchment (forest area more than 50% of the total catchment area) with elevation from the DEM range from 10 m to 2660 m above the sea level. The slope factor might contribute to this output as the original NCRS CN method did not include slope parameter in the model. The NCRS CN method was originally based on estimation of runoff for agricultural watersheds with slope near about 5% (Deshmukh *et al*., 2013). Based on GIS analysis, the slope in the watershed was found to be at average of 36%. Previous study also by (Deshmukh *et al*., 2013) stated that steeper slopes did affect the surface runoff as it can reduce the initial abstraction (Chaplot and Bissonnais, 2003), decrease in infiltration (Philip, 1991) and reduction of the recession time of overland flow (Evett and Dutt, 1985). Huang *et al*., (2006) had done a reviewed on various studies on the effect of soil slope on the runoff. Despite this findings, this current study in Bentong catchment still performed well with overall performance of SDDH model resulted mean NSE of 0.61, mean R^2 of 0.67 and PBIAS at -20.73 (over predict).

Table 3. Percent of error for each storm event based on time to peak (min) and peak discharge (m^3/s)

Storm Event	Time to Peak (min)		Peak Discharge (m^3/s)		Percent of Error (%)	
	Simulated	Observed	Simulated	Observed	Time to Peak	Peak Discharge
17.02.2000	270	330	10.27	10.52	18.18	2.40
24.03.2000	555	405	13.87	16.47	-37.04	15.78
26.04.2000	255	300	19.52	25.29	15.00	22.82
17.11.2000	525	405	22.87	31.92	-29.63	28.36
19.12.2000	300	390	12.13	17.18	23.08	29.39
22.12.2000	390	480	73.59	76.04	18.75	3.23
Sum	2295	2310	152.25	177.43	8.34	101.97
Mean		385		29.57	1.39	17.00

Table 4. Overall SDDH model performance based on prediction of time to peak (min) and peak of discharge (m^3/s)

	Nash-Sutcliffe	Percent Bias	Coefficient of Determination
	E	PBIAS	R^2
Time to Peak (min)	0.82	0.65	0.32
Peak Discharge (m^3/s)	0.95	14.19	0.98

In terms of prediction of time to peak and peak discharge, Strom #1 produce the best result with percentage of error for both resulted at 18.18% and 2.40%, respectively. But still, the best prediction under predicted the real value from observed hydrograph. The worst prediction in terms of time to peak and peak discharge came from Storm #2 and Strom #5, respectively. There is very large delay in time to peak for Storm #2 with almost 150 min late compared with the observed while SDDH model predict less 5.05 m^3/s of discharge compared to observed peak discharge in Strom #5. Overall performance of SDDH model in terms of time to peak and peak discharge shown in table which indicate the model performed well except the result of R^2 for prediction of time to peak with accuracy of 0.32. However, in term of NSE, it c ategorized as very good model as it resulted NSE of 0.82.

4. Conclusions

A SDDH model using time-area method to produce hydrograph successfully developed and utilized in the Bentong catchment. The application of GIS was used to develop the model as it proven it this study that GIS can handle complicated modeling approach due to its power of spatial analysis technique. The prediction of effective rainfall and direct runoff performed using the standard NRCS CN method. CN method proved to be convenient to use as the model require certain parameters that can be easily generated using GIS. The model basically based on DEM, land use and soil map. Rainfall used as the input for the calculation of velocity and estimation of effective rainfall. Observed discharge used to create hydrographs that in the end will be compared towards the hydrograph produced using the SDDH model. Result indicate that the model have a good accuracy based on overall performance (NSE = 0.61; PBIAS = -20.73; R^2 = 0.67). The efficiency of time to peak and peak discharge also produced a good result with NSE of 0.82 and 0.95, respectively. Results from this study can be used for flood prediction for Bentong catchment in future and can be used for engineers, environmental and town planners as well as engineers for guidelines before any development being done within the catchment.

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