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COMPARISON OF PEAK DISCHARGE ESTIMATION METHODS IN NORTHERN JEDDAH IN WESTERN SAUDI ARABIA

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Understanding the accuracy of common methods for estimating peak discharge is essential because hydraulic structures are designed based on these discharges. The Wadi Marwani basin in Jeddah, Saudi Arabia was selected as the study area for this investigation. The catchment area of the Wadi Marwani is 2875 km2, and the total length is 99 km. To recommend the most accurate method for estimating peak discharges, four methods were applied to the measured peak discharges in 2-, 5-, 10-, 25-, 50- and 100-year return periods. These methods were: the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS), the probabilistic rational method (PRM), the modified Talbot method (MTM), and regional flood frequency analysis (RFFA) regression equations. The hydrologic and morphometric parameters of each method differed in accuracy for estimating peak discharges. The root mean square error (RMSE) was used to measure the accuracy of the four methods. The RMSE values of peak discharges estimated by the PRM, HEC-HMS, MTM, and RFFA ranged from 0.019 to 0.038, 0.029 to 0.112, 0.068 to 0.198, and 0.007 to 0.205, respectively. The MTM and RFFA equations displayed much higher errors. The results of this study are consistent with the most comprehensive analyses of measured and modeled peak discharge in northern Jeddah in western Saudi Arabia.

INTRODUCTION

The rainfall-runoff relationship plays a key role in many aspects of watershed management, and especially in the design of flood protection measures (Wheater et al., 1993). Rainfall and basin parameters are the two main factors that affect the rainfall-runoff process. In arid regions, the rainfall is converted to runoff, causing flash floods because the upper parts of basins have a high rainfall intensity with short duration, steep topography, and bare soil (Sen, 2008).

The estimation of peak discharge is one of the most common problems faced by hydrologists and engineers when designing hydraulic structures (Quraishi and Al-Hassoun, 1996). Peak discharge is the maximum flow rate for a particular stream during a storm event. There are many methods for estimating peak discharge including regression equations, the probabilistic rational method (PRM), and the graphical peak discharge method.

A number of detailed hydrologic models have been developed to estimate the peak discharge and runoff hydrograph for a given rainfall distribution such as the hydrologic engineering center model (HEC-1), the technical release-20 model (TR-20), the technical release-55 model (TR-55), and the Hydrologic Engineering Center-Hydrologic Modeling system (HEC-HMS; Tummala, 2003). To hydraulic structure designing at a site where measured peak discharge data are available, engineers must choose a peak discharge estimation method based on a form of flood frequency analysis or one of the methods based on designed rainfall (Pilgrim, 1987).

The Ministry of Transportation (MOT) in Saudi Arabia did not have experience in designing hydraulic structures in 1970 and consulted foreign experts, who suggested an empirical formula for estimating peak discharge in the region. Wilson-Murrow (1971) was one of the foreign consultants, and the company suggested that the southern United States is similar in geographical characteristics to mid-northern Saudi Arabia. The Talbot method was used to estimate peak discharge, and two independent variables were considered: the drainage area and the discharge coefficient. The consultant suggested a modification of the Talbot method (MTM) to suit mid-northern Saudi Arabia and the MOT later generalized this modification for all parts of the region for estimating peak discharge.

This study determined the most accurate model for peak discharge estimation. To achieve this, four models, namely the HEC-HMS, PRM, MTM, and regional flood frequency analysis (RFFA) regression equations, were applied to measure the peak discharge of 2-, 5-, 10-, 25-, 50-, and 100-year return periods in the Wadi Marwani basin.

SELECTION OF THE STUDY AREA

The Ministry of Agriculture and Water (MOAW) constructed nine runoff stations in western Saudi Arabia. Umm Addar station (no. 401) is located at the outlet of the Wadi Marwani basin. This wadi was selected as the study area because of its location and the availability of its rainfall measurements. It extends between longitudes 39‘ 05’ and 40‘ 14’ E and latitudes 21‘ 55’ and 22‘ 50’ N (Figure 1) and crosses the Makkah-Medina highway. This wadi also discharges toward the Red Sea, threatening the King Abdullah University for Sciences and Technology (KAUST). The Wadi Marwani is approximately 1645 m above mean sea level (amsl) in the east and 100 amsl at the outlet, as shown in Figure 2. The Wadi Marwani has a catchment area of 2875 km2 and a total length of 99 km.

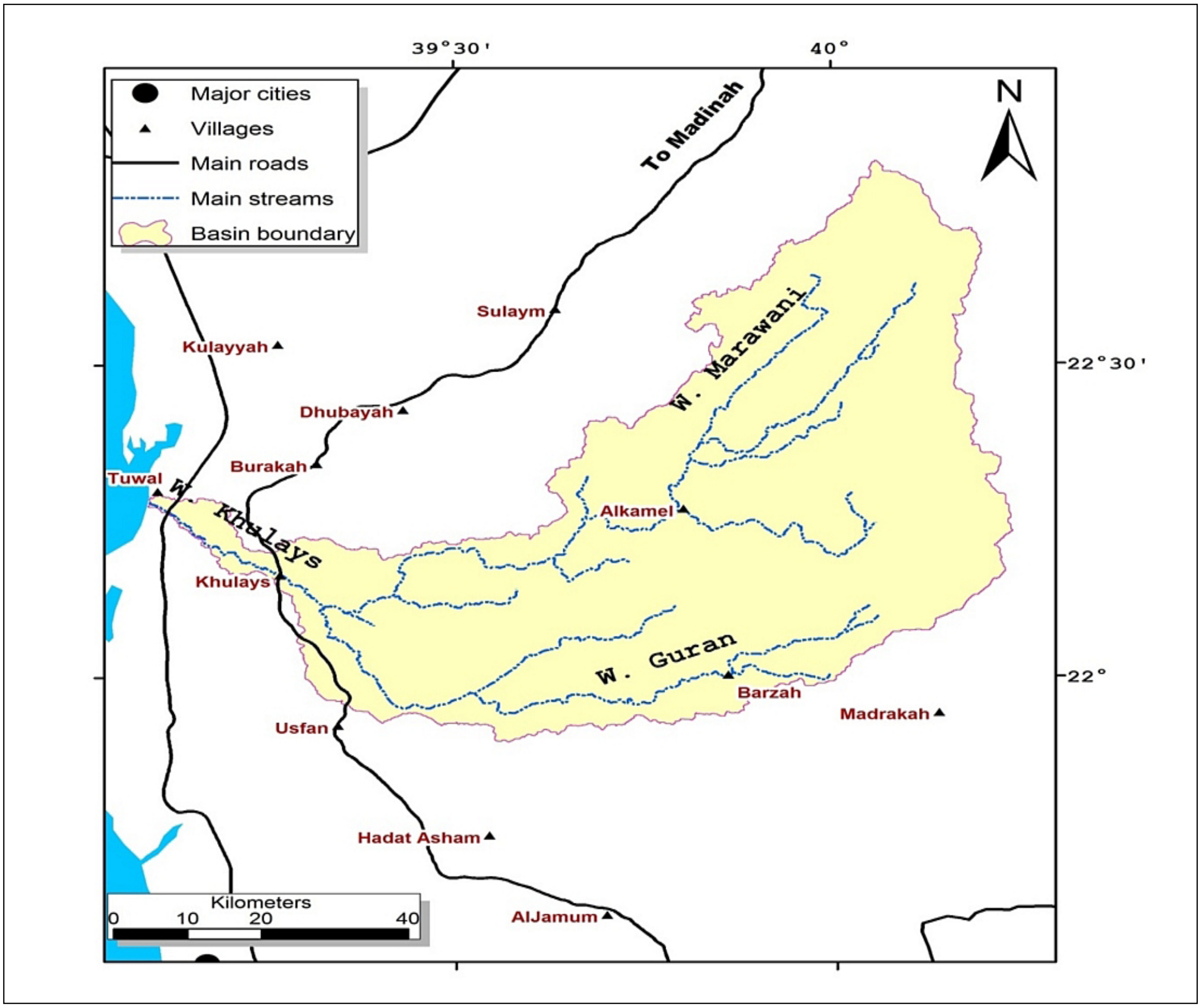

Figure 1. Location of the study area.

MATERIALS AND METHODS

Sub-basin Delineation

Sub-basin delineation was conducted using a 30-m digital elevation model (DEM) in the watershed modeling system (WMS), which was obtained from King Abdul-Aziz City for Sciences and Technology (KACST). The flow directions were obtained using the WMS. The sub-basins were created based on the location of tributaries in the study area, and were then converted to 36 polygons and 18 reaches as shown in Figure 3. The WMS was used to calculate the drainage area upstream of the points of interest.

Measured Peak Discharge

The extreme value type 1 (EVI) distribution is the optimal distribution of peak discharge for different regions in Saudi Arabia (AL-Turbak and Quaraishi, 1987; Sorman and Abdulrazzak, 1993; Quaraishi and Al-Hassoun, 1996). In this study, the EVI was used for the frequency analysis of measured peak discharge. The return periods of peak discharge for the study area are presented in Table 1. Given the record lengths, the estimated 50- and 100-year peak discharge are likely to be subject to a high degree of extrapolation and measurement error, and hence the peak discharge values for these return periods should be used with caution.

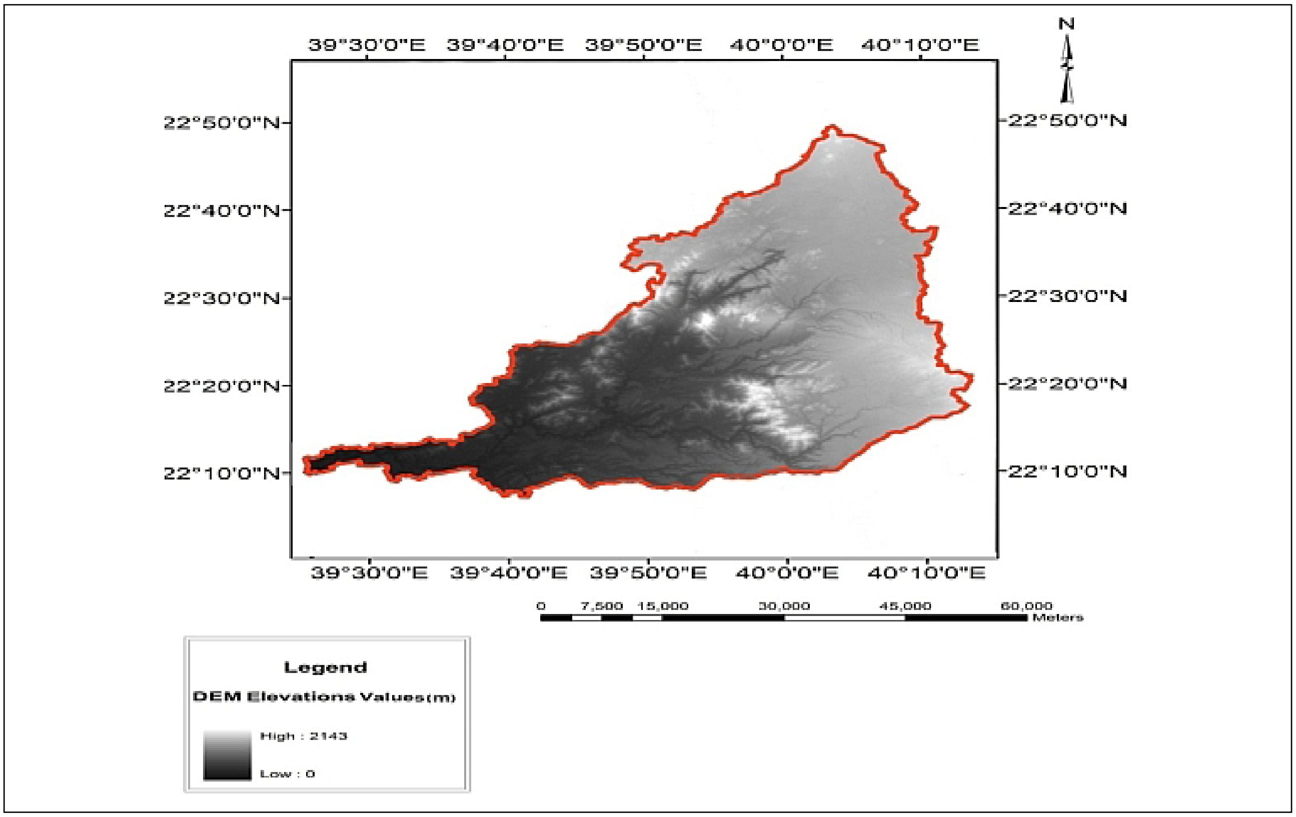

Figure 2. A digital elevation model (DEM) of the study area.

Applications of the Four Methods

This study determines the most accurate model for estimating peak discharge in the Wadi Marwani basin. To achieve this objective, four methods, namely the HEC-HMS, PRM, MTM, and RFFA regression equations, were applied to 2-, 5-, 10-, 25-, 50-, and 100- year return periods. The

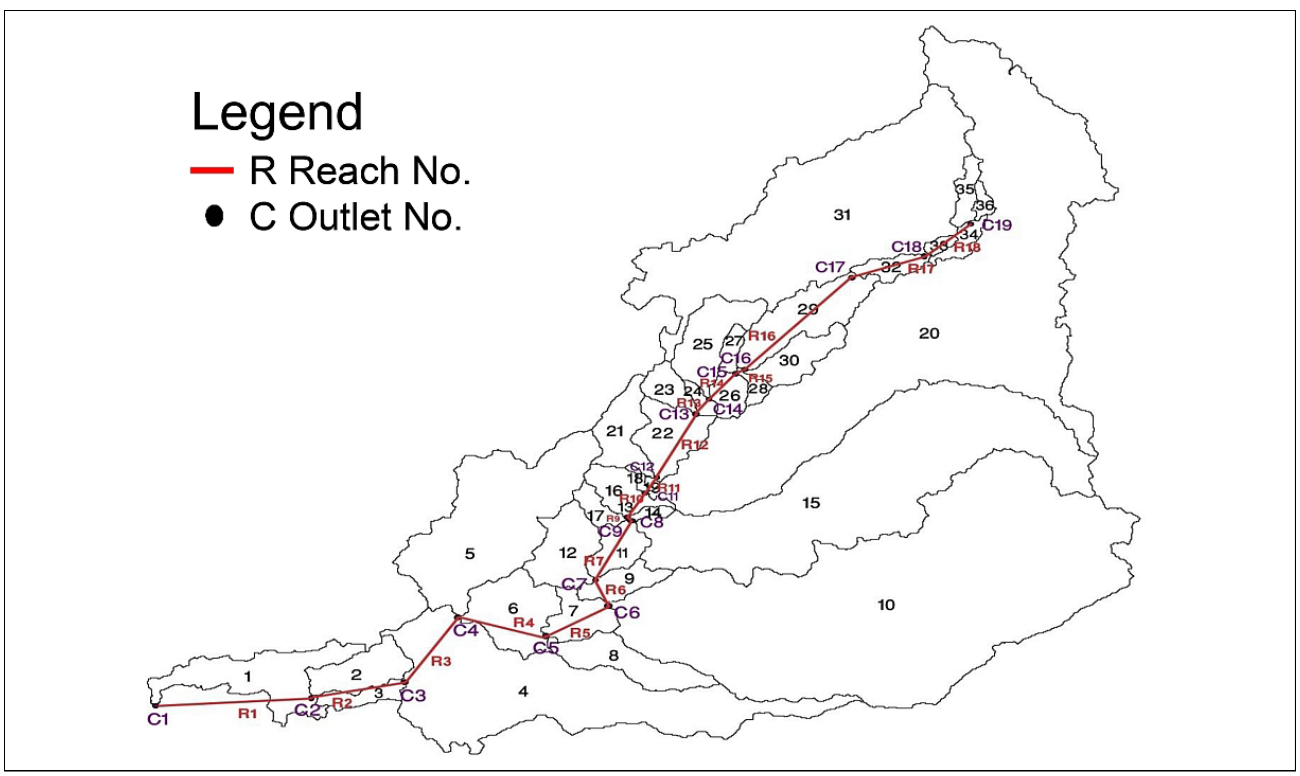

Figure 3. Sub-basin delineation of the study area.

Table 1. Measured peak discharge values for each return period.

Return period (y)	Peak discharge (m3 / sec)
2	234
5	1067
10	1618
25	2315
50	2833
100	3346

conceptual model for this study was based on the water balance at the soil surface during rainfall. Three hydrologic processes were considered: rainfall, infiltration, and runoff. Evaporation and evapotranspiration were negligible in this study.

The HEC_HMS program

The HEC-HMS was used in this study because it is considered a foundation for future hydrologic software and is a public domain program supporting documentation The project file in the HEC-HMS is considered a main data file. The HEC-HMS was run for return periods of 2, 5, 10, 25, 50, and 100 years using the soil conservation service curve number (SCS CN) method to estimate the infiltration losses with an average antecedent moisture condition (AMC II) based on the following equations (SCS, 1985):

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{1}$$

where Q = direct runoff (mm), P = rainfall depth (mm), and S = potential maximum retention after runoff begins (mm).

S is related to the physical characteristics of the landscape with the curve number (CN) shown in the following equation (SCS, 1985):

$$S = \frac{25400}{CN} - 254 \tag{2}$$

The SCS CN is a function of the hydrologic soil group, land use, land cover, and antecedent moisture conditions (Ponce et al., 1996).

CN generation

The first step in determining a CN within a geographic information system (GIS) is to overlay the land use and land cover data sets and create individual polygons containing a single description of land use and land cover. Next, a data table is developed from the SCS CN tables to match the curve numbers to each polygon. Applying these steps to the datasets of available tables resulted in the distribution of the curve numbers shown in Figure 4. Curve numbers ranged from 80 to 94 , indicating that a high peak discharge may be produced with low infiltration. The sub-basins have more types of hydrologic soil groups; the area-weighted curve number is generated. The composite curve numbers CCN for SCS for AMC II were estimated using the following equation:

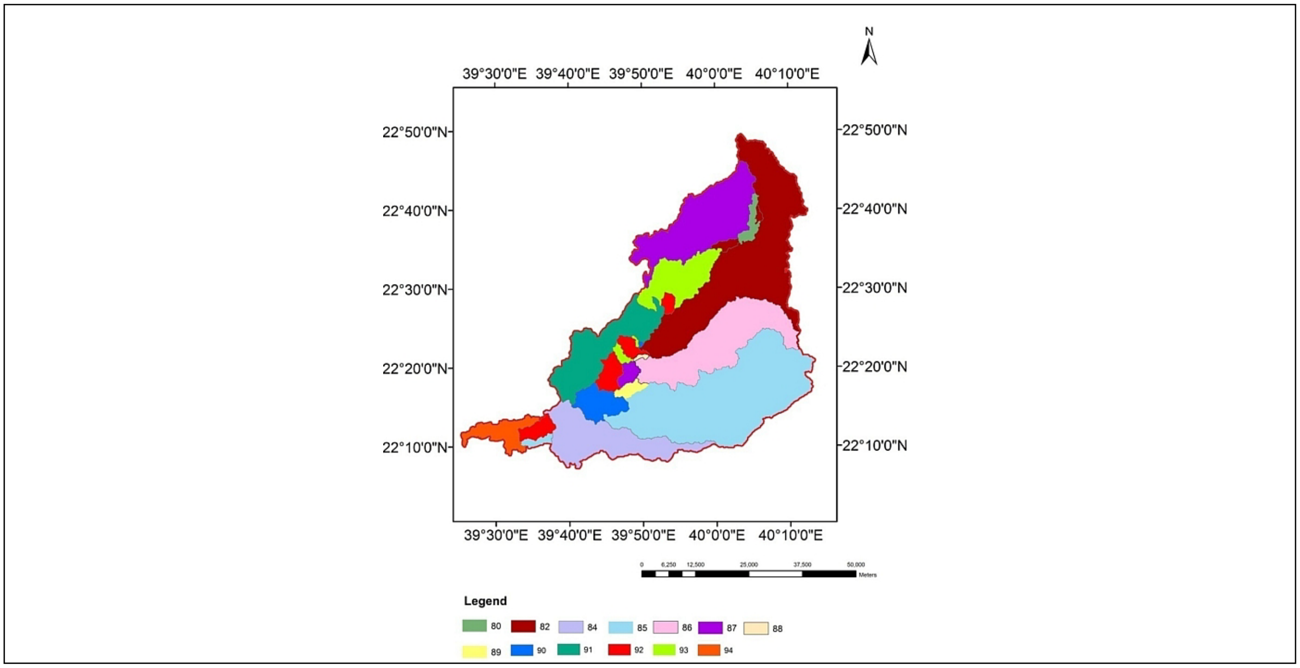

Figure 4. Curve number values within the study area.

$$CNN = \frac{\sum_{i-1}^{n} CN_i A_I}{\sum A_i} \tag{3}$$

where CNI is the curve number for each sub-basin, A_i is the area of each polygon.

Maximal daily rainfall depth

The maximal daily rainfall depth is essential for designing hydraulic structures. The maximal daily rainfall depth from the rainfall gauging stations was used to determine the return periods of daily rainfall depth. The maximal daily rainfall data series for this study was determined based on EVI, which is one of the most widely used analytical tools for evaluating extreme values, and is therefore the most suitable tool for estimating one-day maximal rainfall depth (Hershfield, 1961; Islam and Kumar, 2003; Subyani, 2009).The maximal daily rainfall depth for each return period is presented in Table 2.

Clark unit hydrograph

The Clark unit hydrograph (UH) was used as the transfer function based on the following equations (Clark, 1945):

$$Ot = CAIt + (1 - CA)Ot - 1 \tag{4}$$

$$C_A = \frac{\Delta t}{R + 0.5\Delta t} \tag{5}$$

where R is the watershed storage coefficient. The watershed storage coefficient is approximated by the lag time of the watershed (Sarma, 1969). The lag time is the difference in time between the center of the mass of effective rainfall and the center of the mass of direct runoff produced by the effective rainfall. The SCS CN method observed that the lag time t_l is generally shorter than the time of its concentration t_c, and in most cases (SCS,1985):

Table 2. Maximal daily rainfall depth for the study area using EV1.

Return periods (y)	Maximal daily rainfall depth (mm)
2	21.16
5	36.48
10	46.63
25	59.46
50	68.97
100	78.41

$$t_1 = 0.6 t_c \tag{6}$$

where t_c is the time of concentration. The SCS lag-time formula is used to estimate t_c as follows (SCS,1985):

$$t_c = 0.057 \frac{L^{0.8} \left(\frac{1000}{CN} - 9\right)^{0.7}}{\sqrt{S}} \tag{7}$$

where L is the basin length (km), CN is the curve number of the basin, and S is the slope of the basin (SCS,1985).

Reach routing

The Muskingum-Cunge method was used for reach routing based on the following equation (USACE, 2000):

$$O_t \left[\frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1-X)} \right] I_{t-1} + \left[\frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1-X)} \right] I_t + \left[\frac{2(1-X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1-X)} \right] (O_{t-1}) + \left[\frac{2\frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1-X)} \right] (q_L \Delta x) \tag{8}$$

where O_t and O_{t-1} are the outflow at times t and t-1 (m3/ sec); I_t and I_{t-1} are the inflow at times t and t-1 (m3/ sec); C1, C2, C3, C4 are the coefficients of routing; Δx is the distance increment (m); X is the weighting factor (dimensionless); Δt is the time difference between times t-1 and t; K is the storage constant (sec); and q_L is the lateral inflow.

The parameters K and X are given as follows (Cunge, 1969; Ponce, 1978):

$$K = \frac{\Delta_X}{c} \tag{9}$$

$$X = \frac{1}{2} \left\{ 1 - \frac{Q}{B S_O c \Delta x} \right\} \tag{10}$$

where B is the top width of the water surface (m), S_o is the channel slope, and c is the wave celerity (m/sec), The longest reach (R1) cross sections were measured across the channel using

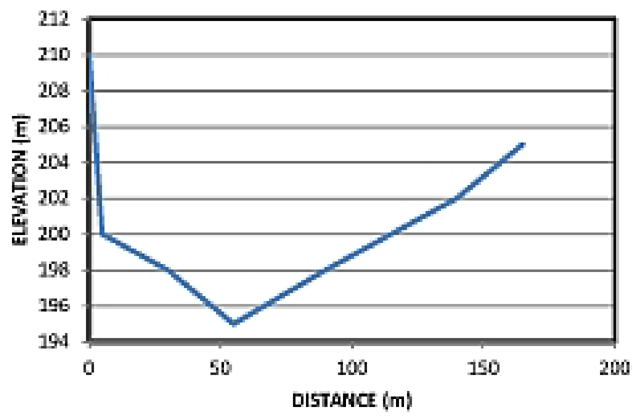

Figure 5. An 8-point cross-section measured for reach R1.

measurement tape, and depths were recorded at eight representative points using GPS (Figure 5).

The reach data from the field measurements were inputted into the HEC-HMS program to enable calculating the routing coefficients K and X. The 0.1 hr was used in this study as the time step (Δt). Once Δt is chosen, the HEC-HMS computes the value of the distance step (Δx) as follows:

$$\Delta x = C\Delta t \tag{11}$$

Then, the values of Δt, Δx, and Equations (9) and (10) are used to calculate the routing coefficients K and X from the reach cross-section data.

The probabilistic rational method (PRM)

To estimate the peak discharge using the PRM for the selected return periods, the measured peak discharges were applied based on Table 1. The estimation of peak discharge using PRM is given by (Pilgrim and Cordery, 1993):

$$Q_Y = 0.278 C_Y I_{tc} A \tag{12}$$

where Q_Y is the peak flow rate (m3/s) for each return period presented from measured peak discharge in Table 1, C_Y is the runoff coefficient (dimensionless) for each return period, A is the area of catchment (km2), and I_{tc} is the average rainfall intensity (mm/h) for a design duration of time of concentration (t_c) hours for each return period.

Intensity duration frequency (IDF) curve development

The rainfall intensity data were taken from the Ministry of Water and Electricity (MOWE, 2011). To develop the IDF curve for the study area, the following steps were taken:

(1) The annual maximum rainfall depths were extracted from historical rainfall records for durations of: 10, 20, and 30 min and 1, 2, and 24 hr.

(2) The rainfall intensities were calculated by dividing the rainfall depth by the recorded duration.

(3) Frequency analysis was applied to the annual data based on: EV1 distribution for different return periods, as shown in Table 3.

(4) The intensity duration frequency (IDF) curves were constructed by plotting the obtained rainfall intensities versus rainfall durations for different return periods as shown in Figure 6.

Table 3. Rainfall intensity (mm/hr) for different return periods using the EV1 distribution

Duration	2-y	5-y	10-y	25-y	50-y	100-y
10 min	40.23	61.63	75.8	93.7	106.98	120.16
20 min	29.06	46.46	57.97	72.52	83.32	94.04
30 min	22.05	36.43	45.95	57.98	66.9	75.76
1 hr	11.9	22.4	29.34	38.12	44.63	51.1
2 hr	6.3	13.17	17.72	23.46	27.72	31.95
24 hr	0.91	1.45	1.81	2.27	2.61	2.94

The times of concentration were calculated using the Bransby-Williams formula (Pilgrim and Cordery, 1993):

$$t_c = 14.6 L A^{-0.1} S^{-0.2} \tag{13}$$

where L is the basin length (km), A is the drainage area (km2), and S is the basin slope (m/m).

Runoff coefficient

Values of the dimensionless runoff coefficient (C_Y) were calculated using following equation (Pilgrim and Cordery, 1993):

$$C_Y = Q_Y / (0.278 I_{tc} A) \tag{14}$$

The frequency factor (FFy) represents the variation in the runoff coefficient across average recurrence intervals. The frequency factors were computed by dividing C_Y by the dimensionless runoff coefficient for the 10-year return period (C10).

The calculated runoff coefficient (C) can be related to the basin characteristics by mapping them over a region (Pilgrim and Cordery, 1993). For this study, there was no available map of C in the Wadi Marwani basin, and an average C value was used. The values of C for each sub-basin were estimated based on available tables, soil classifications, and GIS data. These values ranged from 0.55 with high runoff flow and dominant wadi bed coverage to 0.95 with hard rock cover and a large runoff flow. The weighted average of C values was 0.75. The estimated peak discharges were computed based on Equation 12 for selected return periods.

The modified Talbot method (MTM)

The MTM was developed by Wilson-Murrow (1971) and included three categories of Saudi Arabian watersheds :

(1) Medium watersheds of 400–1258 hectares.

(2) Large watersheds of 1258–35944 hectares.

(3) Regional watersheds of more than 35944 hectares.

The basic formula is given by Wilson-Murrow (1971):

$$Q = KCA^n R_f F_f \tag{15}$$

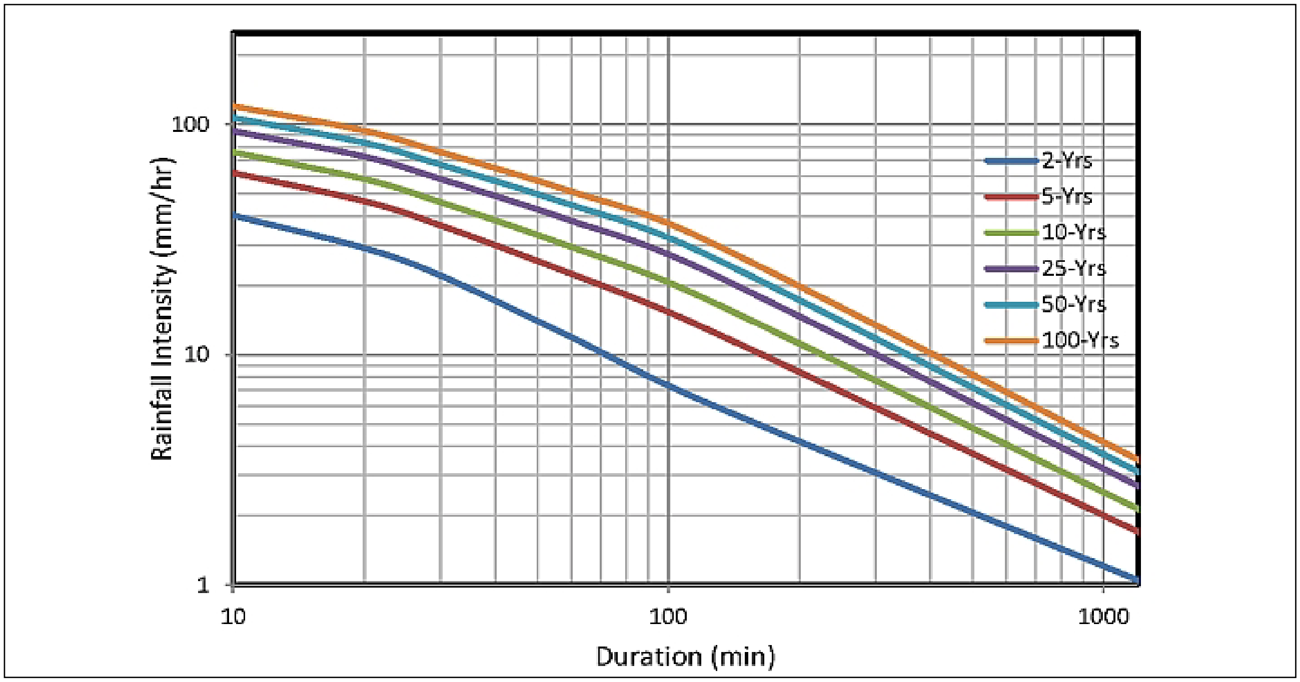

Figure 6. The IDF curve for the study area using EV1 distribution.

where Q is the peak flow rate (m3/s); *K* is a constant having values of 0.558, 3.561, and 10.166 for medium, large, and regional watersheds, respectively; *C* is the coefficient of discharge equal to the sum of c1, c2, and c3 (Table 4); *A* is the drainage area in hectares; *n* is an exponent that depends on the size of the drainage area with values of 0.75, 0.5, and 0.4 for medium, large, and regional watersheds, respectively; R_f is the rainfall factor that was suggested to be 1.5 for medium watersheds and 1.4 for both large and regional watersheds; and F_f is the frequency factor, which depends on the storm frequencies given in Table 5.

The RFFA regression equations

A regional flood frequency analysis is necessary when the available data are limited for frequency analysis. Flood estimates for ungauged basins should be based on regional analyses of data from gauged basins.

Farquharson et al. (1992) developed general relationships for eight separate world regions using catchment areas (km2) as the sole independent variables in Saudi Arabia and Yemen (Figure 7), the developed equation for estimating main annual flood (MAF) in m3/sec is as follows

$$MAF = 0.991A^{0.771} \tag{16}$$

RESULTS AND DISCUSSION

The measured peak discharges were applied to modeled peak discharges from the HEC-HMS, PRM, MTM, and RFFA regression equations for the return period of 2, 5, 10, 25, 50, and 100 years at the outlet of the study area (C1). The comparisons between the measured and modeled peak discharge values are presented in Table 6 and Figure 8.

The root mean square error (RMSE) was used to measure the accuracy of the four modeled peak discharges. A lower RMSE indicates greater overall accuracy. The RMSE is computed as the square root of the sum of the squared differences between the logarithms of the observed and modeled peak discharges. The RMSE values for modeled peak discharges are shown in Table 7.

Based on RMSE values, the PRM (Figure 9) was the most accurate model for computing peak discharge in the Wadi Marwani basin. The RMSE values of peak discharge estimated by the PRM ranged from 0.019 to 0.038. The accuracy of the PRM resulted from:

(1) The calibration of dimensionless runoff coefficient, which depends on the frequency of the measured peak discharge data, rainfall intensity, and drainage area of the basin.

(2) The estimation of average runoff coefficient, which accounts for basin characteristics including slope, soil infiltration, vegetation cover, and surface storage.

(3) The IDF curve, which was developed based on rainfall data taken from the MOWE (2011) and the return period using EVI.

(4) The accurate time of the concentration formula (Bransby-Williams), which assumes that the time of concentration equals the rainfall duration.

The HEC-HMS results in Figure 10 were the next most accurate after the PRM. The RMSE values for peak discharge estimated by the HEC-HMS ranged from 0.029 to 0.112. The accuracy of the HEC-HMS resulted from:

(1) The selection of the infiltration method (SCS CN), which uses the CN to estimate the losses during rainfall based on the MOAW general soil map (1986), soil classifications, average antecedent moisture conditions, and the morphometric parameters for the basin that were extracted from the GIS data.

(2) The Maskingum-Cunge reach routing method, which uses the physical characteristics of the stream (cross-section data and roughness) from field measurements to estimate routing coefficients.

(3) The Clark UH, which uses the time of concentration and basin storage coefficient to calculate the runoff hydrograph at outlet of the basin. An SCS lag time formula and basin storage were used to estimate the time of concentration and lag time of the study area.

Table 4. Discharge Coefficient Values for MTM (Wilson, 1971).

Coefficient	Values	Drainage nature
C_1	0.30	Mountainous area
	0.20	Semi-mountainous
	0.10	Low land
C_2	0.50	S >15%
	0.40	10%<S <15%
	0.30	5%<S <10%
	0.25	2%<S <5%
	0.20	1%<S <2%
	0.15	0.5%<S <1%
	0.10	S <0.5%
C_3	0.30	W = L
	0.20	W = 0.4L
	0.10	W = 0.2L

Table 5. Drainage Storm Frequency Factor for MTM (Wilson, 1971).

Return periods (year)	F_f
5	0.60
10	0.80
25	1.00
50	1.20
100	1.40

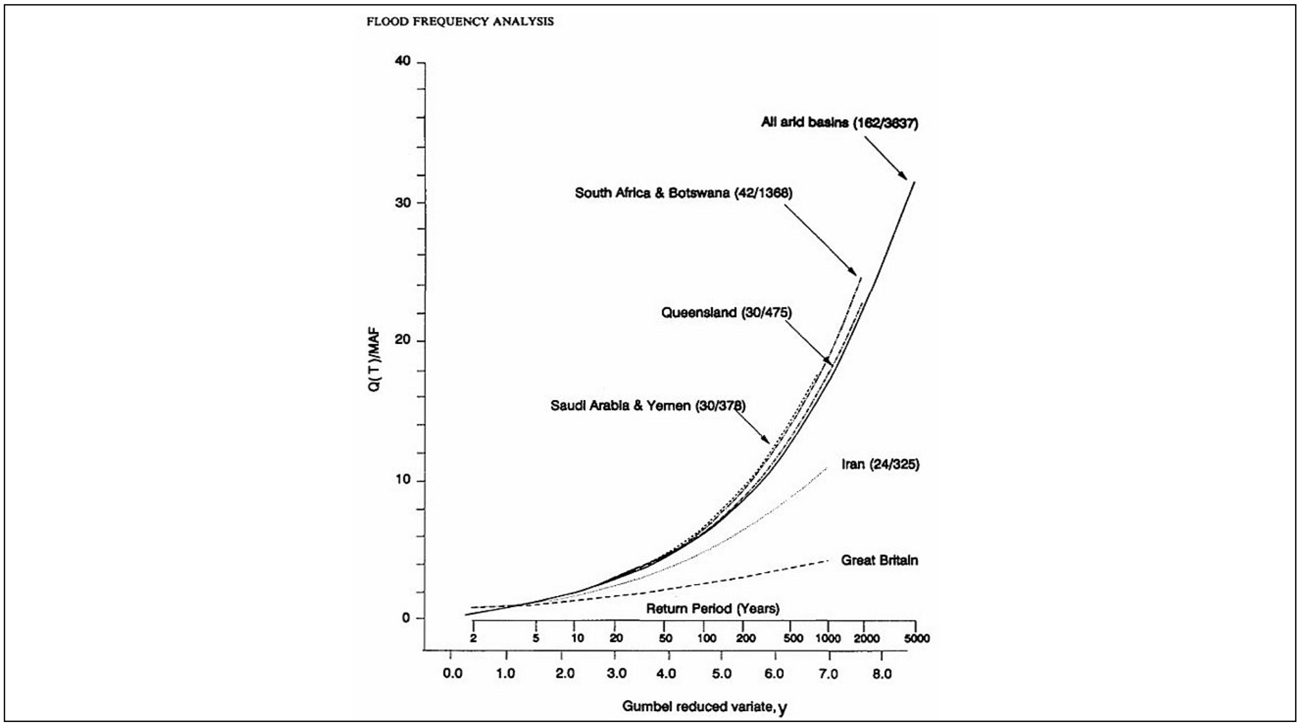

Figure 7. Worldwide regional flood frequency curve (Farquharson et al. 1992).

(4) The SCS type II design storm for a 24-hour rainfall distribution (short duration, high-intensity rainfall) for various return periods (2, 5, 10, 25, 50, and 100 years) of maximal daily rainfall depth based on the (EVI) distribution.

The MTM and RFFA equations have a much higher rate of error. The errors in the MTM (Figure 11) resulted from the assumptions of the model, which does not account for the land cover or land use. The RMSE values of estimated peak discharge by MTM ranged from 0.068 to 0.198. It was also found that the RMSE increased in the peak discharge estimated by the MTM when the return period of the designed storm increased, indicating that this method is appropriate for estimating peak discharge for a 10-year return period or less. The errors in RFFA results (Figure 12) occurred because this method only accounts for the catchment area and ignores all other topographic and storm characteristics of the basin. The RMSE values for estimated peak discharge using RFFA ranged from 0.007 to 0.205. The results of this study are consistent with the most comprehensive analyses of measured and modeled peak discharges in northern Jeddah in western Saudi Arabia.

CONCLUSION

The estimation of peak discharge is a common problem faced by hydrologists and engineers when designing hydraulic structures. Four methods were used to estimate peak discharge in the

Table 6. Comparison between the measured and modeled peak discharges for selected return periods at the outlet of Wadi Marwani basin.

Return periods (y)	Measured peak discharge (m3/sec)	Modeled peak discharge (m3/sec)			
		HEC-HMS	PRM	MTM	RFFA
2	234	218	223	-	230
5	1067	824	1004	911	460
10	1618	1302	1482	1212	547
25	2315	2006	2184	1515	918
50	2833	2519	2689	1818	1146
100	3346	3132	3161	2122	2064

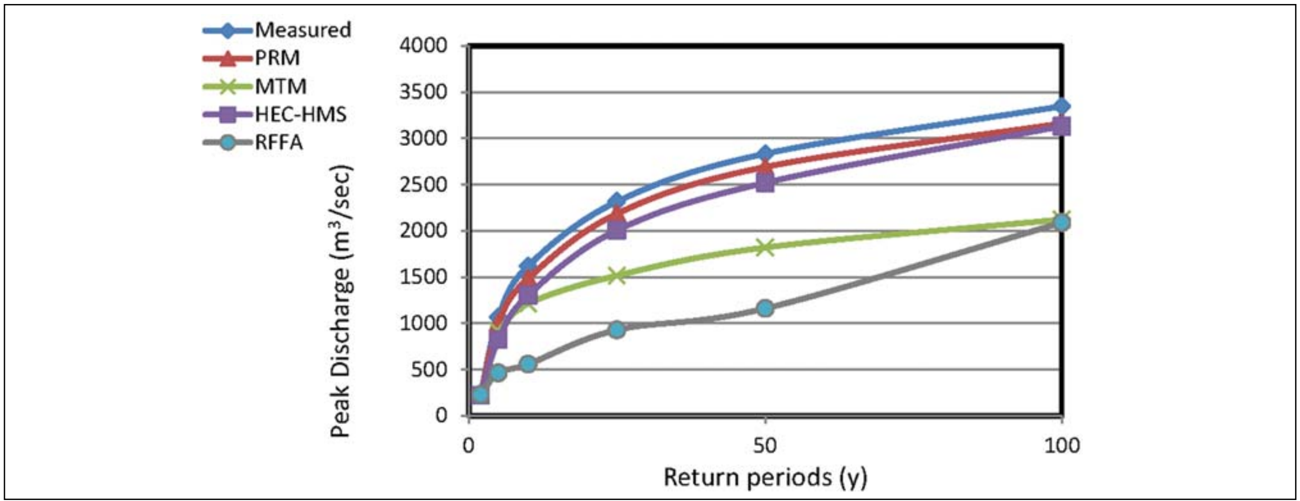

Figure 8. A comparison between the measured and modeled peak discharges in Table 6.

Table 7. RMSE values from a comparison between modeled and measured peak discharges for the selected return period.

Return periods (y)	HEC-HMS	PRM	MTM	RFFA
2	0.029	0.019	-	0.007
5	0.112	0.026	0.068	0.362
10	0.094	0.038	0.125	0.463
25	0.062	0.025	0.184	0.397
50	0.051	0.023	0.192	0.388
100	0.029	0.025	0.198	0.205

study area: the HEC-HMS, PRM, MTM, and RFFA regression equations. The measured peak discharges taken from the MOAW (2011) were compared with modeled peak discharges for 2-, 5-, 10-, 25-, 50- and 100-year return periods. The RMSE was used as the overall measure of accuracy, where a low RMSE indicated a high accuracy.

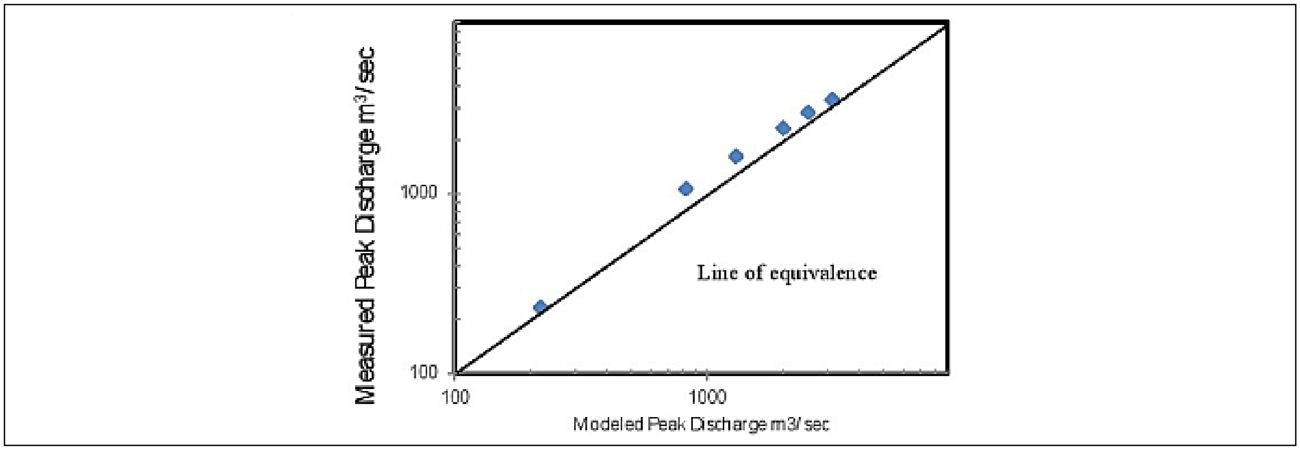

Figure 9. Modeled peak flows from the HEC-HMS plotted against the measured peak discharges for different return periods.

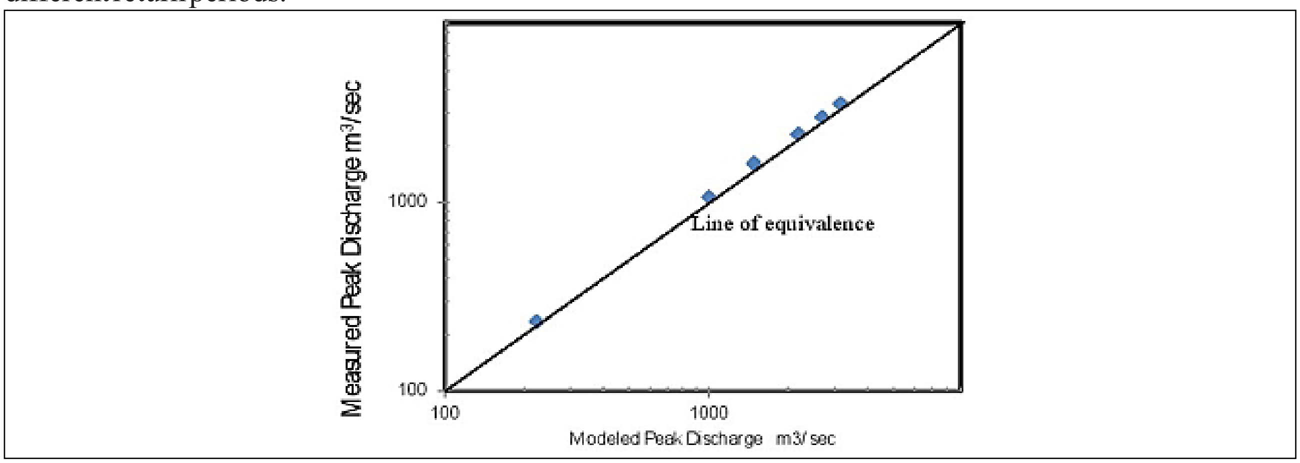

Figure 10. Modeled peak discharges from the PRM formula plotted against the measured peak discharges for different return periods.

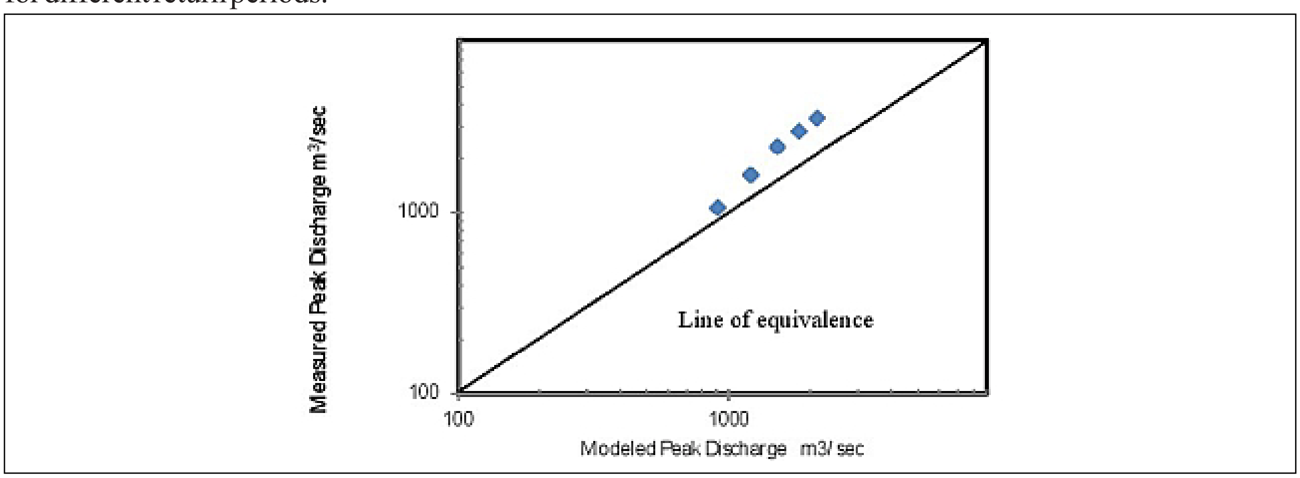

Figure 11. Modeled peak discharges from the MTM formula plotted against the measured peak discharges for different return periods.

The PRM was an accurate model for computing the peak discharge in the study area. The RMSE of peak discharge estimated by the PRM ranged from 0.019 to 0.038. The accuracy of this model resulted from the selection of the time of concentration formula, the calibration of the runoff coefficient, and the IDF curve, which was developed in this study. The Bransby-Williams formula was used to estimate the time of concentration.

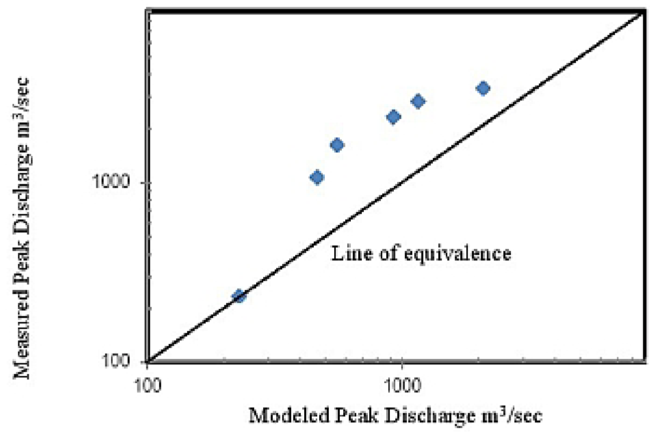

Figure 12. Modeled peak discharges from the RFFA formula plotted against the measured peak discharges for different return periods.

The HEC-HMS was the second most accurate model for computing the peak discharge in study area. The RMS values of peak discharge estimated by the HEC-HMS ranged from 0.029 to 0.112. . The accuracy of the HEC-HMS resulted from accurate of maximal rainfall depth distribution (EVI), the selected method for estimating infiltration losses (SCS CN method), the timing parameters that were input into the Clark UH which uses as a transfer method, and the accurate selected time of concentration formula (SCS lag time).

The MTM and RFFA equations had much higher errors. The error in the MTM resulted from the assumptions of this method, which did not take land cover and land use into consideration. The RMSE values of estimated peak discharge by MTM ranged from 0.068 to 0.198. The error in the RFFA resulted because the catchment area is the only independent variable in RFFA. The RMSE values of estimated peak discharge using RFFA ranged from 0.007 to 0.205. The results of this study are consistent with the most comprehensive analyses of measured and modeled peak discharges in northern Jeddah in western Saudi Arabia.

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