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A unified approach for hydrological modeling of arid catchments for flood hazards assessment: case study of wadi Itwad, southwest of Saudi Arabia

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Abstract

A unified and systematic approach for hydrological modeling is presented for assessing flood hazards in arid and semi-arid regions with wadi Itwad as a case study. The approach consists of the following steps: (1) estimation of the design storm duration, (2) estimation of the design rainfall depth, (3) development of the design storm hyetograph, (4) setup of the watershed hydrological system, (5) infiltration tests for estimate of soil characteristics, (6) land use and land cover analysis for curve number estimation, (7) how to incorporate dams (if any) and their reservoir characteristics for reservoir routing analysis, and (8) performing hydrological modeling under different scenarios, namely: lumped with and without the effect of dams and routed scenarios. An emphasis is made on the comparison between the SCS unit hydrograph and the unit hydrograph (UH) estimated for stream flow data. The UH method produces hydrographs that are typical of arid regions. It is characterized by a steep rising limb with a short time to peak, a rapid recession to zero base flow, and less flood volume due to transmission losses. The scenarios show that the lumped case overdesigns the protection schemes, while the lumped central basin with detention dams storing the flood water completely leads to underdesign in the downstream area. The routed scenario is more realistic, since it considers the detention dams with an overflow. The runoff volume (66.6 MCM) of the 10-year return period of the routed scenario is in the order of magnitude within the observed historical data (50 to 100 MCM) reported in the literature. The study concludes that the proposed approach is robust and takes into account the Saudi arid environment while performing flood risk assessment in KSA.

Keywords Arid region · Rainfall–runoff modeling · Unit hydrograph theory · Flash floods · Detention dams · Wadi Itwad · Saudi Arabia

Introduction

Since the flash floods that attacked Jeddah City in 2009 and 2011 (Elfeki and Bahrawi 2017; Elfeki et al. 2018; Azeez et al. 2019), the Saudi Arabian authorities have decided to make flood protection schemes to cities and new communities that are vulnerable to flood hazards. Therefore, for any urban development in the Kingdom of Saudi Arabia (KSA), a hydrological study is a must to check whether any urban development is subjected to flash floods or not, and if so, the designer has to put the mitigation measure for flood protection.

In Saudi Arabia, there are about 509 dams distributed over the KSA (Al Qahtani and Matter 2018). The purpose of these dams is mainly for flood protection, irrigation, rainwater harvesting, and groundwater recharge.

There is a great deal of projects in the KSA related to flash flood protection schemes. These projects require hydrological studies of the catchments. Engineers and hydrologists often rely on assumptions and theories that are derived from environments that are not arid (e.g., the Soil Conservation Service (SCS) type II hyetograph for rainfall storm distribution, the SCS unit hydrograph theory, SCS-curve number (CN) tables, etc.) due to the scarcity of data and fundamental research in arid regions. Moreover, they also often encounter the existence of dams in catchments and there is no specific approach

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to handle these dams in the hydrological analysis. The common approaches for handling such catchments, in the engineering consultation offices, are either neglecting the existence of these dams and consider the catchment as a whole which would lead to an overdesign (high factor of safety) or subtracting the area of the sub-catchments of these dams which would lead to underdesign (low factor of safety). Since both cases are not realistic, a realistic alternative approach must be considered. Therefore, the motivation of the current study is covering the abovementioned aspects and to present a unified approach that tackles these issues and provides a more realistic approach for flood mitigation studies.

Since the beginning of the twenty-first century, many researchers face the challenge of the paucity of hydrologic data for the arid and semi-arid regions (Sui and Maggio 1999; Merzi and Aktas 2000; Zerger and Smith 2003; Guzzetti and Tonelli 2004; Sanyal and Lu 2006; He et al. 2003; Fernadez and Lutz 2010; Masoud et al. 2014; Masoud 2015; Elfeki et al. 2017). This makes the hydrological analysis to be a challenging task in arid and semi-arid environments. There are enormous studies in hydrological modeling worldwide, among them Baduna Koçyiğit et al. (2017a, 2017b)) used the Hydrologic Engineering Center–Hydrologic Modelling System (HEC-HMS) to estimate the hydrological parameters of the Kocanaz watershed in Turkey. They concluded that for small watersheds such as Kocanaz, it might be better modeled as a single basin without losing estimation capacity and with minimum computational costs. The results also showed a slight improvement in the peak discharge in the case of using sub-basins. Akay et al. (2018) used HEC-HMS to simulate hydrologic processes in a watershed in the western Black Sea region in Turkey. The results showed that multiple stream gauging stations for calibration and validation provide relatively good results for runoff hydrograph; however, the prediction of the peak discharges is not improved. These studies rely on the synthetic unit hydrograph theories in the HEC-HMS software which may be not applicable in every region. Therefore, the current study shows the differences that may occur when using the SCS unit hydrograph theory and a unit hydrograph that is developed from a catchment (Yiba basin; Albishi 2015) in the vicinity of the study basin (wadi Itwad).

A research team from the Department of Hydrology and Water Resources Management and the Water Research Center at King Abdulaziz University in Jeddah, KSA, has devoted several research projects to setup a scientific hydrological database for the hydrological analyses in the KSA.

Elfeki et al. (2014) have developed design hyetographs for the 13 regions in the KSA. These hyetographs are different from the SCS type II hyetograph that is commonly used in the KSA. These developed hyetographs are recommended to be used in the design of flood protection schemes in the KSA. Ewea et al. (2016b) have developed the intensity–duration–frequency curves and equations for the bases of the storm water design in Saudi cities. Albishi et al. (2017) have developed empirical equations for flood analysis based on the flood measurements, the so-called Ari-Zo model. The model provides parameters which are different from the parameters established by the well-known SCS method (USDA-SCS 1963). Some of their results are used in the current study and compared with the SCS method. Ewea et al. (2016a) have studied the influence of the hyetograph models on the runoff hydrograph on one of the basins that attacked Jeddah City in 2009. The results show that the developed hyetograph from rainfall data is different from the SCS hyetograph types (I, II, III, IA). The SCS type II hyetograph, which is commonly used in the KSA, shows an overestimate in the runoff peak by 68% and also an overestimation in the time to peak by about 57% when compared with the developed hyetograph from rainfall data in Makkah Al-Mukkramah region. However, in the Ewea et al. (2016a) study, they used the SCS method for the unit hydrograph theory. Therefore, in the current study, a focus is made in the comparison between the SCS method unit hydrograph and the unit hydrograph derived from stream flow data.

In this paper, a unified and systematic approach is presented for hydrological modeling in flood mitigation studies. The approach used rational aspects for the estimation of each parameter in the modeling process rather than relying on assumptions and theories from the environment that is not arid.

A focus is made on some hydrological issues related to the modeling of a catchment in arid or semi-arid regions, namely: the choice of the design rainfall duration; estimation of the design rainfall depth; selection of the design rainfall hyetograph; estimation of SCS-CN from infiltration tests together with well-known infiltration models: Philip (1957a, 1957b, 1958) and Horton (1933, 1939) and some empirical formulas that relate SCS-CN to infiltration parameters; inclusion of dams in the analysis and reservoir routing; and taking into account flood routing.

In addition, an emphasis is made to the comparison between the SCS unit hydrograph theory and the unit hydrograph estimated for stream flow data, in an arid catchment in the vicinity of the study area. The reason is to estimate the discrepancies between the two methods since most flood studies in Saudi Arabia rely on the SCS unit hydrograph theory.

Moreover, the different scenarios for hydrological modeling processes based on lumped and semi-lumped approaches with the inclusion of dams are analyzed to evaluate the results and come up with the realistic scenario for practical applications.

The Itwad catchment (Sorman et al. 1994), which is located in the southwestern part of the KSA, is selected, since it contains two big dams, as a case study to implement such an approach.

Material and methods

Study area

Wadi Itwad basin is located in Asir region southwest of Saudi Arabia within the Tihama escarpment of the Arabian Shield (Fig. 1a). The wadi is in the region between latitudes 17° 31′ and 18° 16′ N and longitudes 42° 06′ and 42° 43′ E (Sorman et al. 1994). Wadi Itwad basin is considered as one of the high elevated basins where its elevation ranges from 2 m (amsl) at the Red Sea coast to 2962 m (amsl) at the water divide areas with mean elevation of about 1045 m (amsl) as shown in Fig. 1b (left image). Wadi Itwad upstream starts from the Sarawat Mountains, characterized by steep slopes in the west, and the lower stream area is in the Tihama–Asir region along the Red Sea coastal area. The steep slopes helped in the formation of many middle and upstream branches. The type of rocks, structure, relief, natural vegetation, and climate have also contributed in controlling the characteristics of wadi Itwad basin. The basin has a total area of 1698.63 km^2.

From a geological point of view, the main rock units in wadi Itwad consist of basement (Precambrian) and sedimentary rocks, intruded by associated plutonic rocks, and are overlain by Paleozoic and Mesozoic sedimentary rocks (Sorman et al. 1994). Precambrian volcanic rocks constitute the main geology of the area and Quaternary sedimentary deposits in places are recent geological features. These deposits lie most often at lower elevations of the Tihamah area, which has evolved during the Oligocene epoch and has continued to the recent geologic time. The geology of the area is presented in Fig. 1b (right image). Close to the downstream area, there are intrusive rock outcrops of gabbro and diabase in the forms of densely sheeted dike swarms and thick dikes. The basement rocks outcrop all over the eastern margin of the area and form the hilly pediment of the Asir Mountains. Tertiary deposits at the downstream-most part, and a large patch of basalt and andesite rocks of Jeddah group occurs, which includes pillow lava, flow breccia, and tuff interbedded. Quaternary deposits alluvium fills in the form of Quaternary deposits (sand, gravels, and conglomerates) from the lowest elevations along the wadi main channel with increasing thickness and width toward the downstream.

Methodology

The methodology presented herein consists of several steps. The following is a short listing of these steps and the reader can refer to the flowchart in Fig. 2:

1. Statistical analysis of storm durations and proposing an approach to come up with the design storm duration.
2. Estimation of the design rainfall depth using rainfall frequency analysis and statistical tests.

3. Analyzing the temporal rainfall pattern and development of the design of the storm hyetograph.
4. Catchment delineation and setup of the hydrological wadi system.
5. Infiltration tests for estimation of soil characteristics and analyzing the results using two infiltration models, namely: Philip and Horton models.
6. Estimation of land use and land cover for curve number (CN) estimation based on results from the infiltration tests and empirical equation developed by Van Mullem (1989).
7. Including dams in the catchments and their reservoir characteristics in the hydrological system.
8. Hydrological modeling of the catchments system under different scenarios, namely: lumped, with and without the effect of dams, and routed scenarios.
9. In the current study, an emphasis is made on the comparison between the SCS unit hydrograph theory and a unit hydrograph (UH) estimated for stream flow data in an arid watershed in the vicinity of wadi Itwad.

The following paragraphs explain each item intensively and discuss the steps in detail and provide new insight on some issues that are always encountered in arid regions and how they can be tackled. During the description of the methodology, there will be a discussion associated with each item.

Design storm duration analysis

Estimation of the design storm duration has been performed through the analysis of the measured storm durations from some stations in the Asir region. Fifty-nine storms are recorded with their corresponding durations (Elfeki et al. 2014). These durations are presented as frequency histograms and as a cumulative distribution in Fig. 3. The most frequent duration (mode) is 3 h which is common for producing flash floods. Common probability distribution functions have been fitted to the data, namely: normal, log normal, Gumbel, exponential, gamma, and beta. The Kolmogorov–Smirnov test has been utilized to find out the best distribution (Table 1). It has been shown that all distributions could not be rejected at 10% and 5% significant levels. However, the best distribution is beta since it gives the minimum value of the maximum differences over other distributions (Kite 1977).

Design rainfall depth

Time series of maximum rainfall depth is collected from four stations in the study area. Figure 4 (left column) shows the time series of the stations. The stations have a record length starting from the year 1965 up to 2015. The maximum observed value from each station and its corresponding year are 198.6 (1998), 186.8 (1998), 68 (1988), and 119.9 (1997) mm
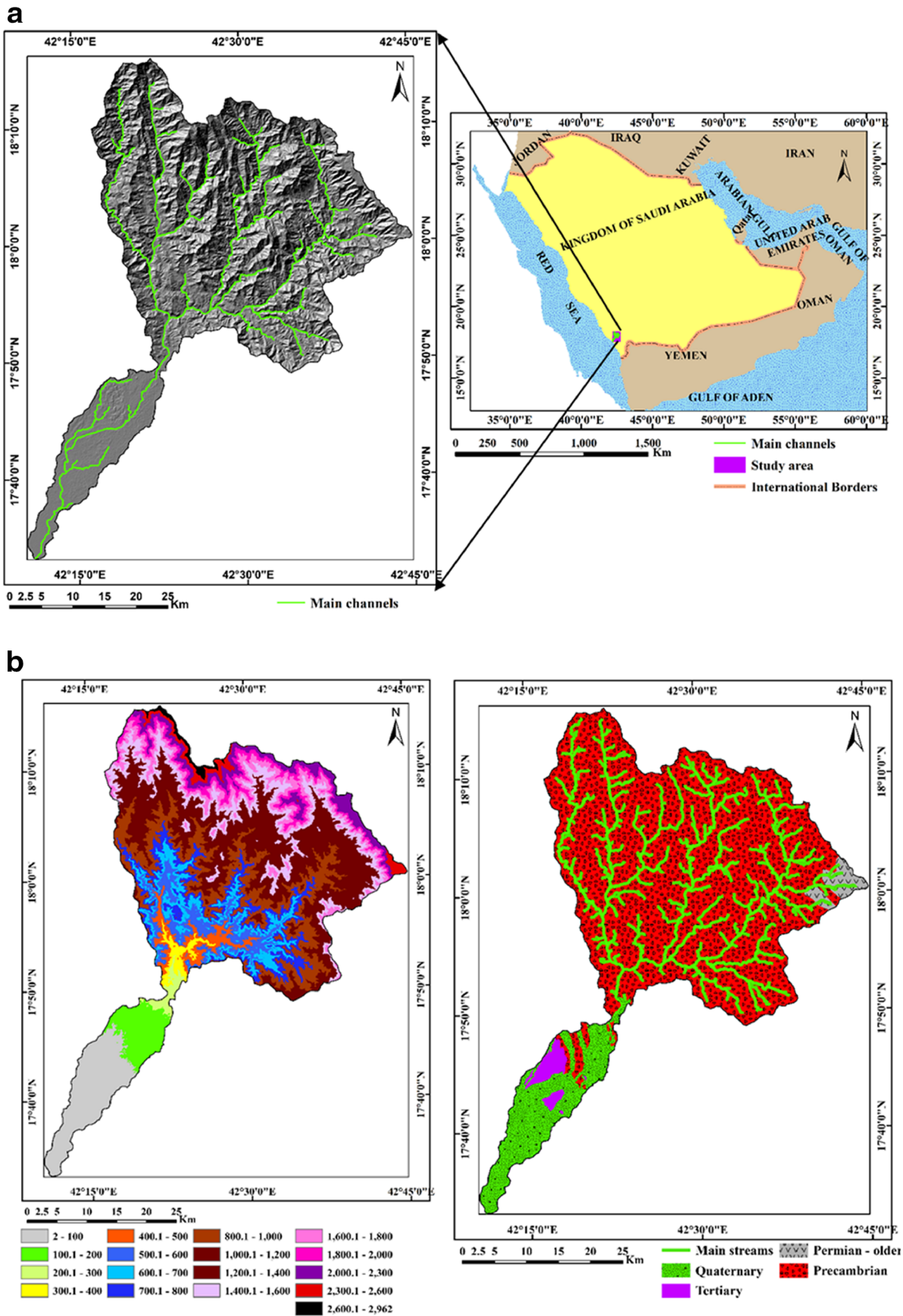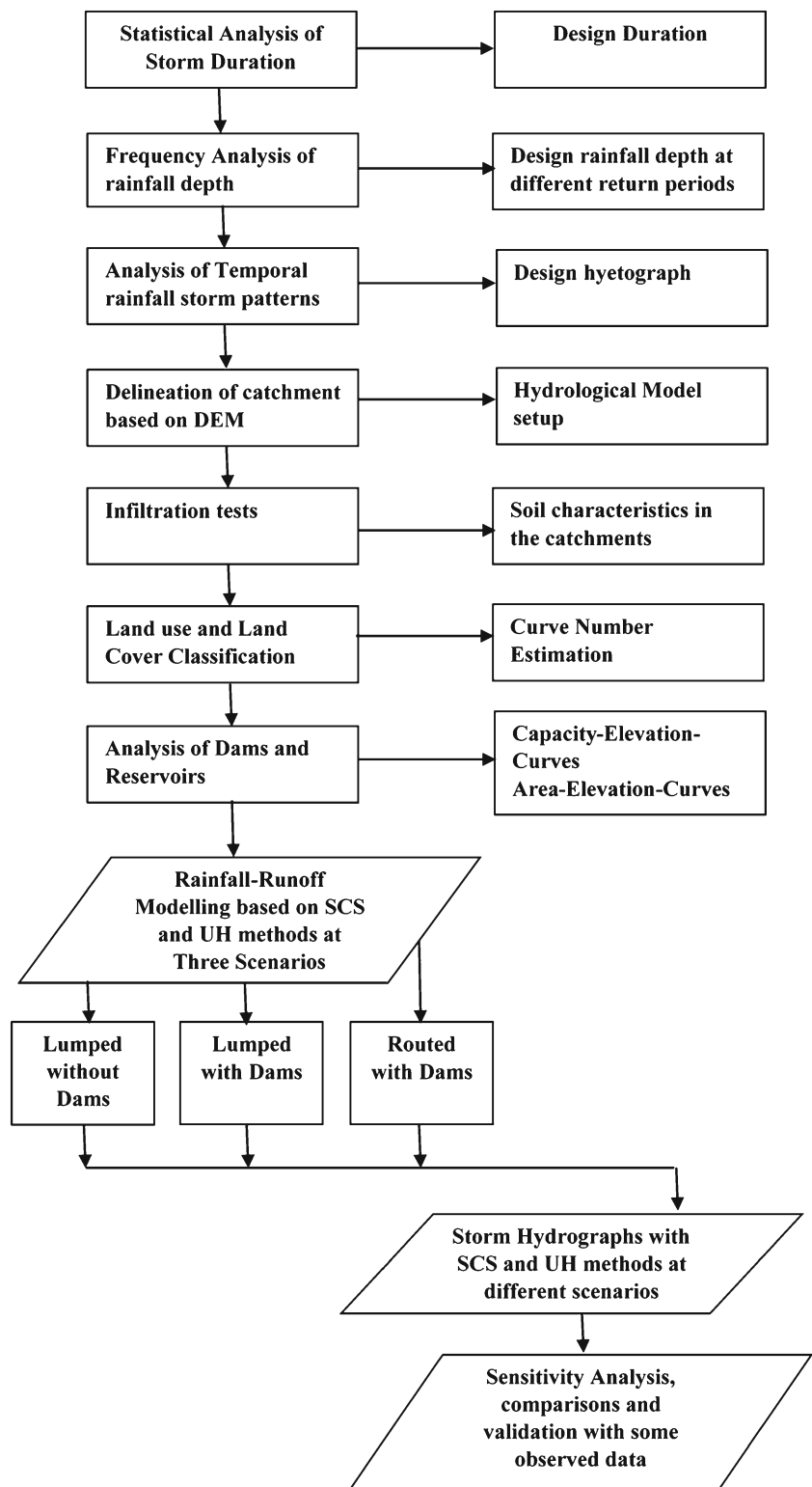
Fig. 1 a Location of the study area. b Digital elevation model of the Itwad basin (left) and geology of the Itwad basin (right)

Fig. 2 Flowchart of the proposed methodology

for stations 28, 31, and 503 and Abha airport, respectively. The above values show that there was an extreme event in 1998 over the study area. The mean rainfall depth for each station is represented by the blue line in Fig. 4 (left column) which reads 52.7, 44.5, 30.4, and 41.4 mm for stations 28, 31, and 503 and Abha airport, respectively. The frequency analysis of the maximum daily rainfall depth is made using different probability distributions, namely: generalized extreme value (GEV), normal distribution, log normal, 3-parameter log normal, and Gumbel distributions. The root mean square error
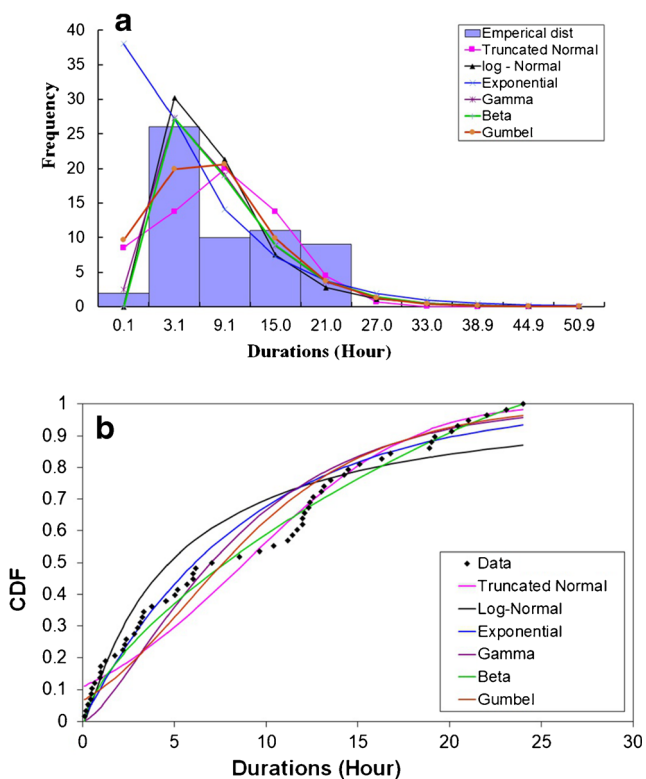
Fig. 3 Frequency histogram (a) and cumulative distribution function (CDF) of storm durations (b) and fitting by different probability distribution functions in Asir region

(RMSE) is used as a measure of the best fit (Kite 1977; Niyazi et al. 2014; Elfeki et al. 2017; Noor and Elfeki 2017; Kamis et al. 2018). The RMSE is given by,

$$\text{RMSE} = \left(\frac{1}{n} \sum_{i=1}^{n} \left[\widehat{R}_i - R_i \right]^2 \right)^{0.5} \tag{1}$$

where,

R_i is the measured rainfall depth (mm),
$\widehat{R}_i$ is the estimated rainfall (mm), and
n is the number of data points.

Table 2 shows the values of the RMSE for the tested distributions. The best distribution is GEV which provides the minimum RMSE for Abha airport and stations 28 and 503, while for station 31, the three-parameter log normal is the best. Figure 4 (right column) shows the results of

the frequency analysis of the maximum daily rainfall depth fitted to the best probability distribution. Table 3 shows the expected rainfall for different return periods and their corresponding exceedance probability for the stations. The design rainfall for the wadi is estimated by two methods, namely: the arithmetic mean method and the inverse distance squared method. The table shows that the arithmetic mean method provides less value between 1 and 8 mm for 2- to 100-year return periods, respectively. The inverse distance squared method seems to provide more realistic results since it takes into account the effect of the distance weighting when calculating the spatial average over the catchment (Niyazi et al. 2014).

Design hyetograph

Elfeki et al. (2014) have developed design hyetographs for the storm patterns in the KSA. These hyetographs are based for detailed sub-day storm data. Ewea et al. (2016a) have shown the effect of the difference between the design hyetographs in the KSA and the SCS type II hyetographs. Therefore, in this study, the KSA hyetographs in the Asir region are adopted. Figure 5a shows the dimensionless mass curve for the storm pattern in Asir region (Elfeki et al. 2014), and a dimensionless equation (Eq. 2) was fitted to the data to be able to stablish temporal rainfall distribution (hyetographs) for any design rainfall depth and duration. Figure 5b shows the mass curve for the design rainfall presented in Table 3 for different return periods (2, 5, 10, 25, 50, and 100 years).

$$\frac{r(t)}{R} = \frac{1 - a^{(t/D)}}{1 - a} \tag{2}$$

where

$r(t)$ is the cumulative rainfall depth (mm) at time t,
R is the total rainfall depth of the storm (mm),
t is the elapsed time since the storm begins (min),
a is a fitting parameter, and
D is the storm duration.

Table 1 Results of the Kolmogorov–Smirnov test

Distribution	Normal	Log normal	Exponential	Gamma	Beta	Gumbel
Max difference	0.129	0.156	0.149	0.146	0.064*	0.123
Tabulated value (10% level)	0.161	0.161	0.161	0.161	0.161	0.161
Tabulated value (5% level)	0.178	0.178	0.178	0.178	0.178	0.178

*Is the best distribution

Fig. 4 Time series of the maximum rainfall depth (left column) and fitting probability distribution to the maximum rainfall data (right column)
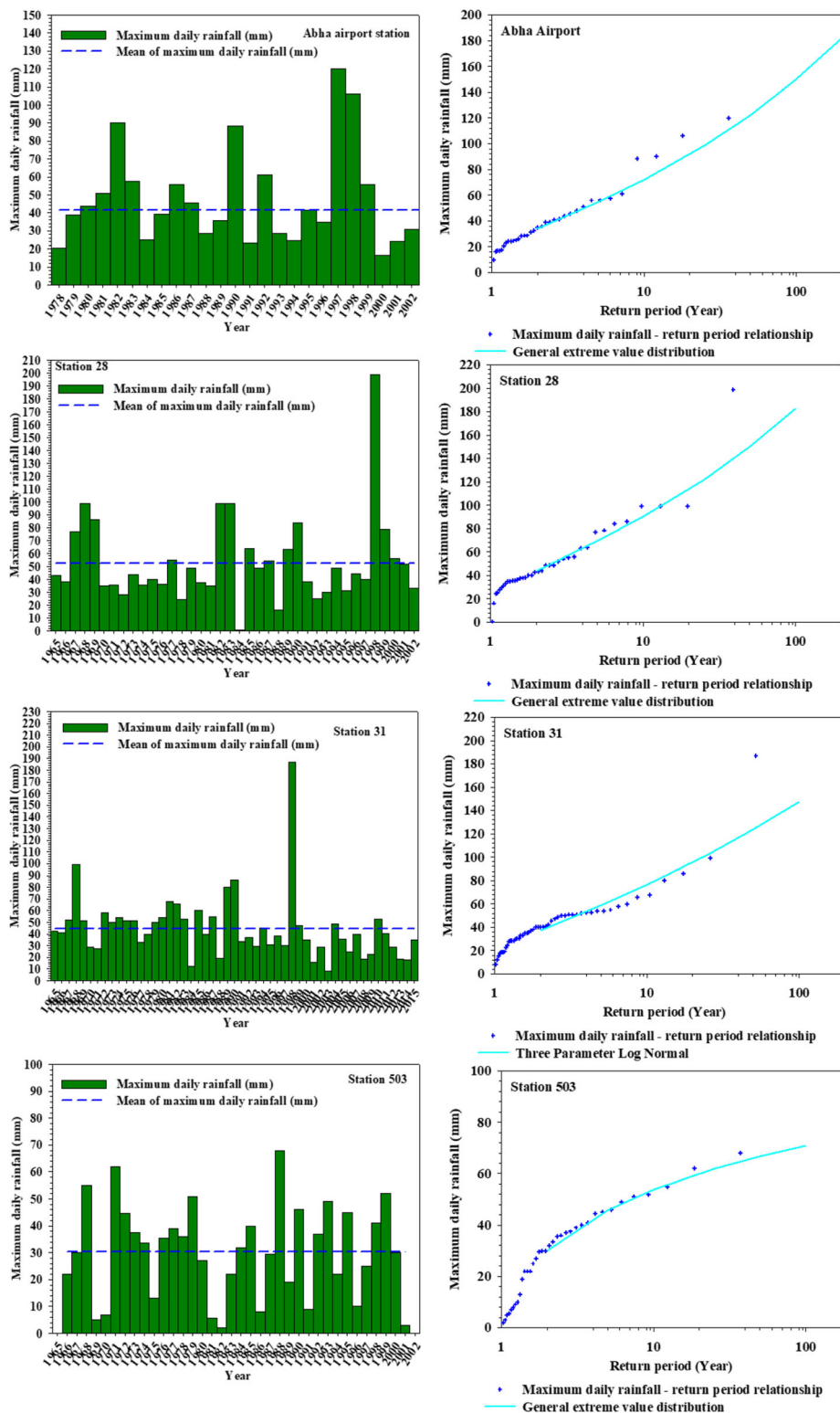
Catchment delineation and the hydrological setup of the wadi system

Advanced Spaceborne Thermal Emissions and Reflections (ASTER) data is used to create digital elevation models (DEM) with 30-m resolution, and geographic information systems (GIS) are applied to extract the drainage tributaries and estimate geometric parameters, textures, and relief characteristics of Itwad basin and its sub-basins. The Watershed Modeling System (WMS) and HEC-HMS are connected to

Table 2 Root mean square error (RMSE) in millimeters for different rainfall distributions at the rainfall stations in the study area

Type of distribution function	Stations			
	28	31	503	Abha airport
Generalized extreme value (GEV)	10.7*	10.51	2.34*	5.33*
Normal	16.5	14.56	2.61	10.33
Log normal	10.88	10.03	5.31	5.7
3-Parameter log normal	10.85	9.622*	2.57	6.1
Gumbel	12.4	11.36	3.88	6.52

*Is the best distribution

the GIS to assess the relationships between rainfall and surface runoff based on some measured rainfall data, infiltration test data, and physical characteristics of the study wadi to determine the flood hydrographs of wadi Itwad and its sub-basins taking into account the effect of the existing dams in the study area. Figure 6 (left image) shows the hydrological setup of the basin and its sub-basins, locations of the dams in the study area, and the locations of the rainfall stations. The catchment is divided into three sub-basins: Itwad sub-basin (right sub-basin), Dil sub-basin (middle sub-basin), and Maraba sub-basin (left sub-basin). In the study area, there are two dams, namely: Itwad and Maraba dams. These dams are included in the hydrological analysis as will be explained later in the paper.

Infiltration tests

Infiltration tests were performed in the wadi alluvium (Al-Ammawi et al. 2010). In the current paper, the infiltration test data was used to estimate the soil parameters and therefore provided estimation of the CN in the wadi alluvium. Five infiltration tests are carried out at some locations in the wadi alluvium as shown in Fig. 6 (right image), and the results of the tests are plotted in Fig. 7a and b. The figure shows a high degree of variability between the tests which is a typical feature in arid regions (Berndtsson 1987). Two famous models are used to fit to the infiltration data, namely: Philip (1957a,
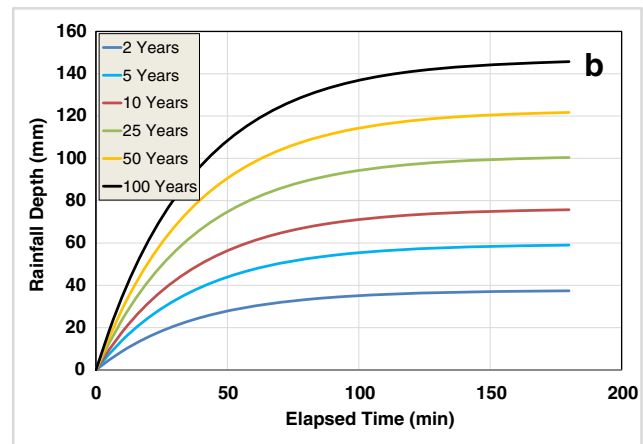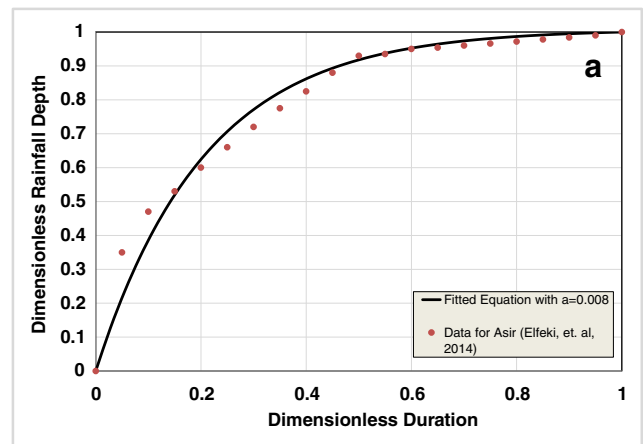
Fig. 5 Dimensionless mass curve for design storms in Asir region (a) and mass curve for different return periods (b)

1957b, 1958) and Horton (1933, 1939) models. The Philip infiltration equation is given by,

$$f(t) = \frac{S}{2\sqrt{t}} + K_{p} \tag{3}$$

Where,

$f(t)$ is the infiltration rate (cm/min) at time t (min),
S is the soil sorptivity (cm/min$^{0.5}$)

Table 3 Design rainfall based on the best distribution for the stations in the study area

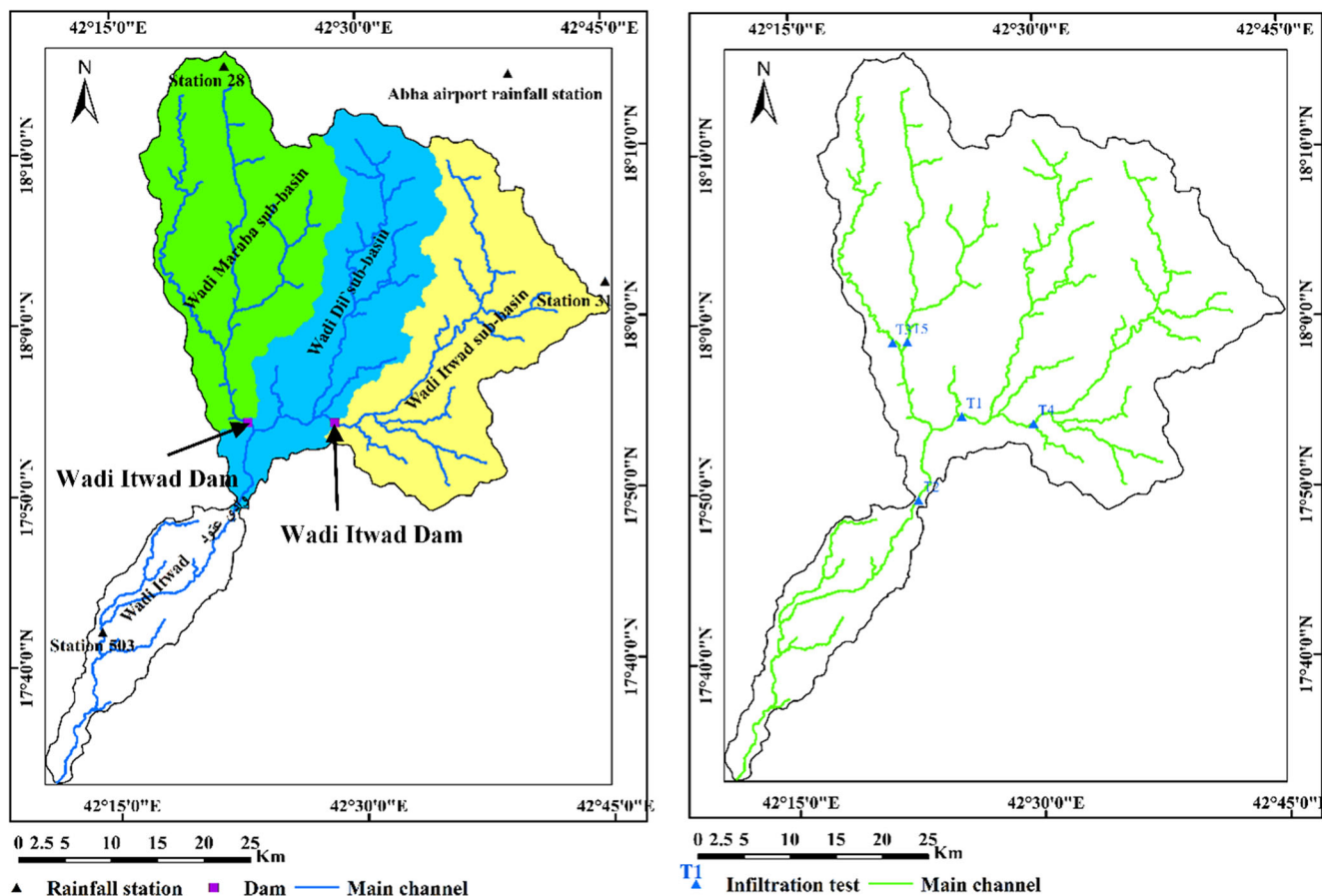|	Probabilities %	50	80	90	96	98	99
	Return period (years)	2	5	10	25	50	100
Meteorological stations							
28		44.1	69.4	90.1	121.7	150.0	182.9
31		37.1	58.8	76.3	102.0	123.5	147.2
503		30.2	45.9	53.9	62.0	66.8	70.7
Abha airport		34.2	55.0	72.2	98.5	122.2	149.9
Arithmetic mean method		36.4	57.3	73.1	96.1	115.6	137.7
Inverse distance squared weighting method		37.4	59.0	75.7	100.4	121.7	145.7

Fig. 6 Hydrological setup of the basin and its sub-basins, dam locations and rainfall stations (left image), and locations of the infiltration tests (right image)

K_p is the hydraulic conductivity of the soil (cm/min), and
t is the time since infiltration starts (cm).

The Philip cumulative infiltration depth, $F(t)$, which is an integration of Eq. 3, is given by,

$$F(t) = S\sqrt{t} + K_p t \tag{4}$$

Equation 4 is fitted to the infiltration test data and the parameters S and K_p are estimated using the least square method (LSM) between the observed data (Fig. 7b) and Eq. 4. The LSM is formulated by the following objective function (Eq. 5) that needs to be minimized to obtain the fitting parameters S and K_p.

$$\text{Obj}(S, K_p) = min \sum_{i=1}^{n} \left(\widehat{F}(t_i) - \left[S\sqrt{t_i} + K_p t_i \right] \right)^2 \tag{5}$$

Where,

$\widehat{F}(t_i)$ is the observed cumulative infiltration (cm),
t_i is the time corresponding to the observed cumulative infiltration (min), and
n is the number of observations.

The Horton infiltration equation is also tested for fitting the infiltration data. The Horton model is given by,

$$f(t) = f_c + (f_o - f_c)e^{-kt} \tag{6}$$

Where,

$f(t)$ is the infiltration rate (cm/min) at time t (min),
f_o is the initial infiltration rate (cm/min),
f_c is the ultimate infiltration rate (cm/min), i.e., equilibrium infiltration rate after the soil has been saturated,
k is the decay coefficient (1/min), and
t is the time since infiltration starts (min).

The Horton cumulative infiltration depth, $F(t)$ in centimeters, which is the integration of Eq. 6, is given by,

$$F(t) = f_c t + \frac{(f_o - f_c)}{k} \left(1 - e^{-kt}\right) \tag{7}$$

Equation 7 is fitted to the infiltration test data and the parameters f_o, f_c, and k are estimated using the LSM. A similar formalism can be made as in Eq. 5 to read,
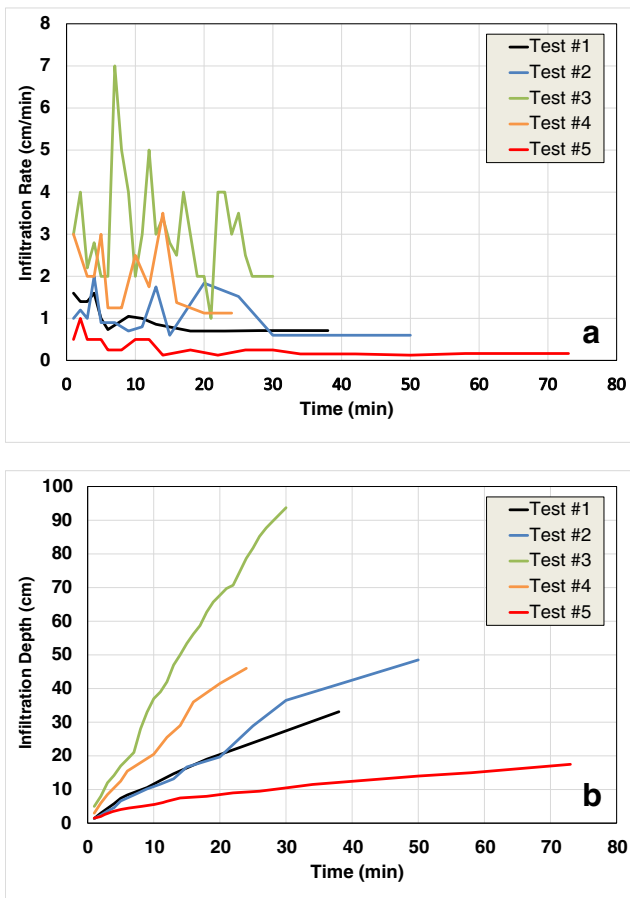
Fig. 7 Field-measured infiltration tests: infiltration rates (a) and cumulative infiltration depth (b) versus time for the five infiltration tests (from test #1 up to test #5)

$$\mathrm{Obj}(f_o, f_c, k)$$
$$= min \sum_{i=1}^{n} \left(\widehat{F}(t_i) - \left[f_c t_i + \frac{(f_o - f_c)}{k} \left(1 - e^{-kt_i}\right) \right] \right)^2 \quad (8)$$

SCS-curve number estimation

The traditional approach to estimate SCS-CN is normally based on the SCS-CN tables (USDA-SCS 1963, 1985). In this traditional approach, the CN values are often subjective and depends on person opinion. Therefore, in the current study, an alternative approach is followed. The estimation of SCS-CN from the alluvium of the wadi bed is made from the following empirical equation that relates the hydraulic conductivity of the soil to the CN (Van Mullem 1989),

$$K = 21,462 exp(-0.16061 CN) \quad (9)$$

where K is the saturated hydraulic conductivity in in/h and CN is the curve number.

The above equation has a correlation coefficient of 0.993 and it is valid of CN between 55 and 94 (Van Mullem 1989). This range of values is within the range of values observed in the KSA (Farran and Elfeki 2019; Farran and Elfeki 2020a, 2020b).

Adjusting the equation for metric units and reformulating for CN, the formula reads,

$$CN = -6.2263 ln(K/13,083) \quad (10)$$

where K is in m/day.

Equation 10 is used to estimate CN from the results of the infiltration tests. For the Philip model, the value of K_p corresponds to the saturated hydraulic conductivity and consequently substituted in Eq. 10 to provide $CN(K_p)$, while for the Horton model, the value of f_c that corresponds to the saturated hydraulic conductivity is also substituted in Eq. 10 to estimate $CN(f_c)$.

Since there is no data to estimate CN for the wadi basement, a value is obtained from the literature (Masoud et al. 2013). Therefore, the final value of wadi Itwad catchment is estimated from the weighted average of the CN of the alluvium and the CN of the basement, weighted by their percentages.

Dams and reservoirs in the study area

There are two dams in the study area: Itwad and Maraba dams. These dams are built for flood control and groundwater recharge. The shape of the catchment area of the dams is shown Fig. 4. The images of the dams are displayed in Fig. 8a and b. The reservoirs of the dams are shown in Fig. 8c and d. Table 6 provides the data of the dams. The data is obtained from the construction company website (http://www.sajco.com) and the Ministry of Environment, Water and Agriculture (https://www.mewa.gov.sa/). The dam reservoir characteristic curves are presented in Fig. 8e and f. Both the elevation–surface area curve and the elevation–captivity curve are displayed. The figure shows that Itwad dam is at higher elevation than Maraba dam.

Dam reservoirs are analyzed using the approach presented by Mohammadzadah-Habili et al. (2009). They have provided a dimensionless equation for height–capacity curves of dam reservoir. The formula is of exponential form, which reads,

$$\frac{V(h)}{V_{max}} = \left(e^{(ln2)(h/h_{max})} - 1 \right)^{1/N} \quad (11)$$

Where,

$V(h)$ is the reservoir volume as a function of the height above the reservoir bottom at the dam site,

V_{max} is the maximum reservoir volume,

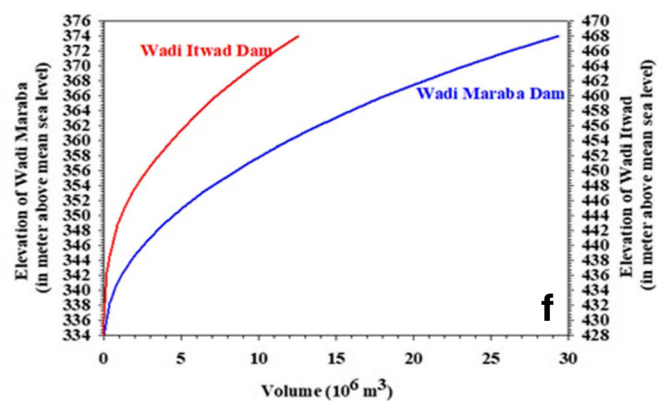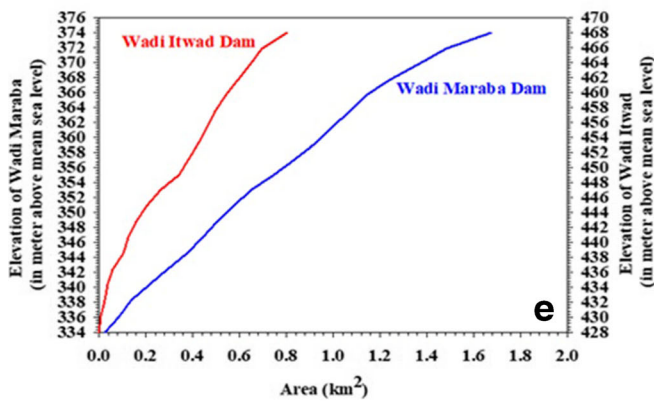h_{max} is the maximum reservoir height, and

Fig. 8 Dams in the study area: a image of Maraba dam and b image of Itwad dam. Source: http://www.sajco.com/project/maraba-dam/, http://www.sajco.com/project/itwad-dam/. c Google Earth Satellite image of Maraba reservoir. d Google Earth Satellite image of Itwad reservoir. e The area–elevation curves for both dams. f Capacity–elevation curves for both dams

N is a fitting parameter of the equation to the reservoir data which is called the reservoir coefficient.

The reservoir coefficient, N, is related to the reservoir shape factor, M, through the relationship given by Mohammadzadah-Habili et al. (2009) as,

$$N = 1.0751M^{-0.9063} \tag{12}$$

The value of M is used to classify the reservoir type according to Borland and Miller (1958) into four categories. When M is in the range of 3.5–4.5, then the reservoir is type I (gorge);

when M is in the range of 2.5–3.5), the reservoir is type II (hill); when M is in the range of 1.5–2.5, the reservoir is type III (floodplain foothill); and when M is in the range of 1.0–1.5, the reservoir is type IV (lake).

Hydrological analysis

In this section, a detailed hydrological analysis is presented. The catchment is treated based on three scenarios. Two of these scenarios are chosen based on the common practice in the engineering offices in Saudi Arabia. In the case of dams
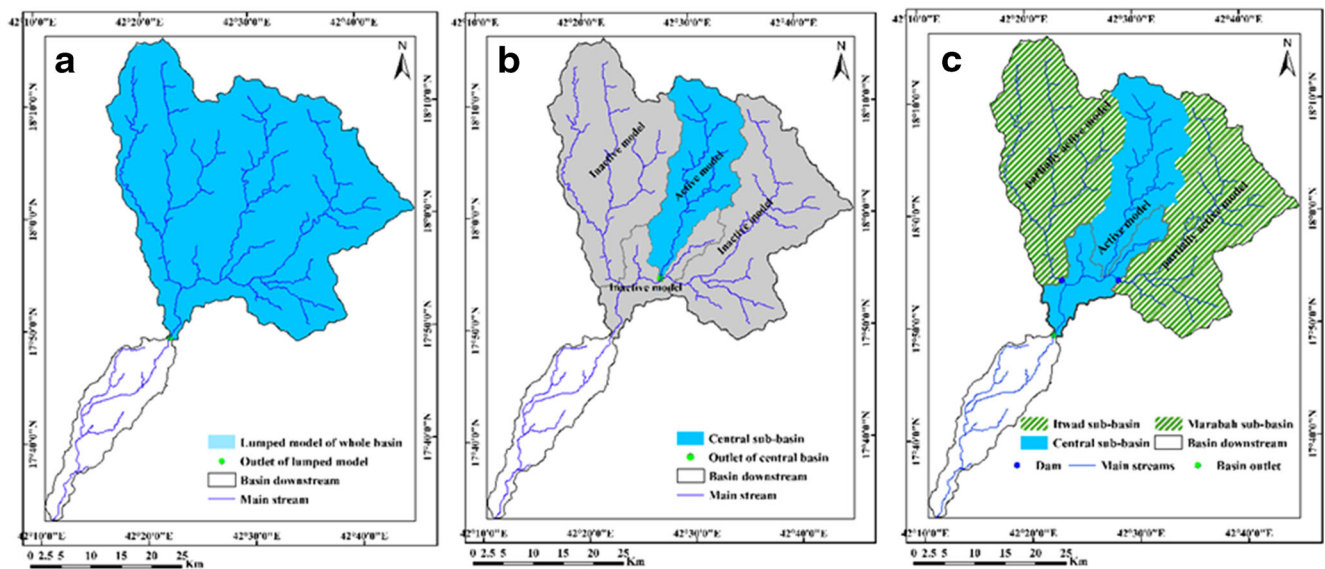
Fig. 9 Scenarios of the hydrological setup of the catchment: a lumped case: the whole basin active, b central basin active and the rest is inactive, and c central basin active and the rest is partially active
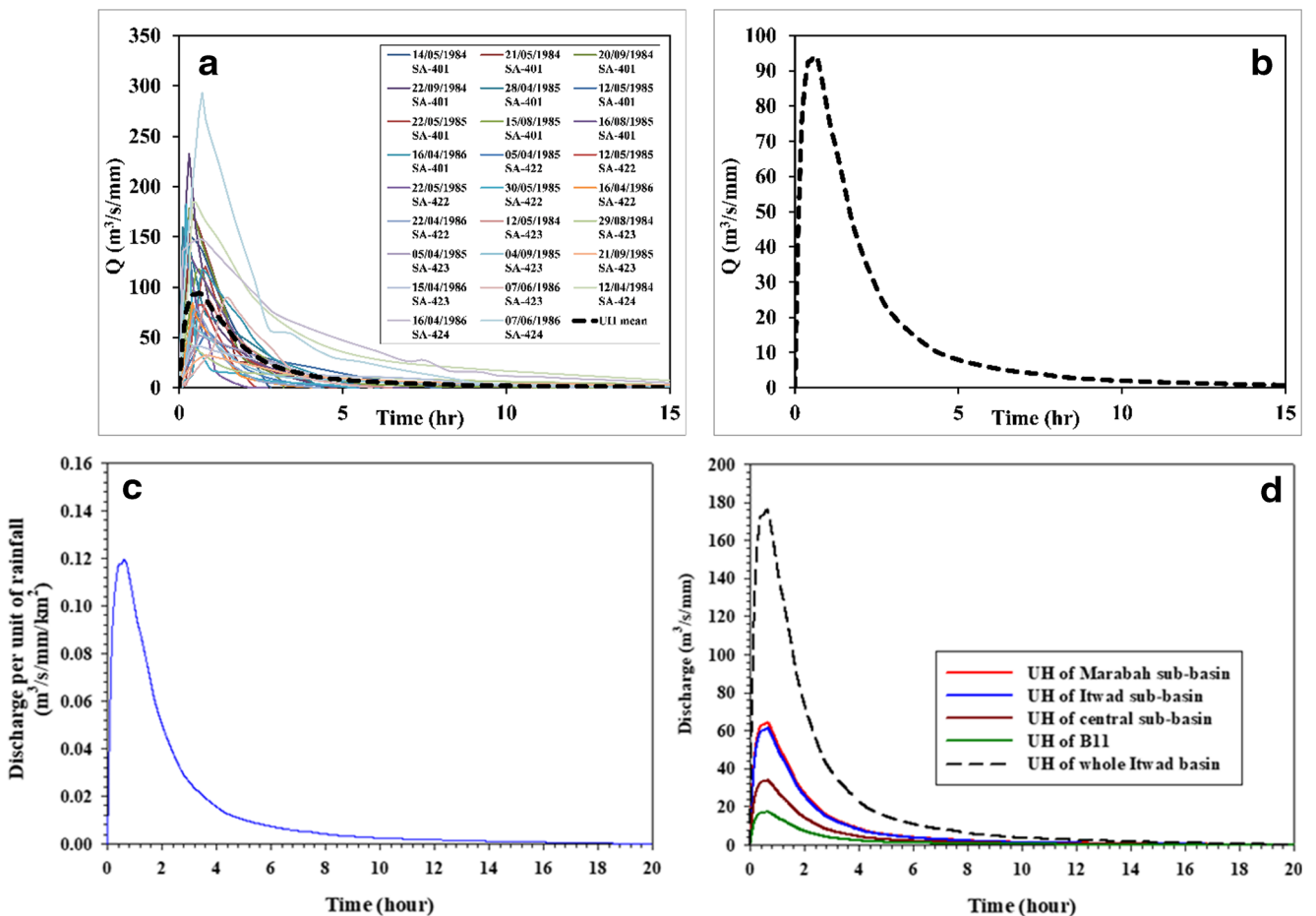
Fig. 10 Unit hydrograph of Yiba basin for 1 h storm duration (modified from Albishi 2015): a UH of each recorded storm at each station in the Yiba basin, b the mean of UHs from the recorded storms, c normalized unit hydrograph of Yiba basin, and d the estimated UHs for each sub-basin of wadi Itwad catchment

Fig. 11 Fitted infiltration equations to field-measured infiltration tests in the wadi alluvium (left column: Philip equation and right column: Horton equation) from test #1 up to test #5

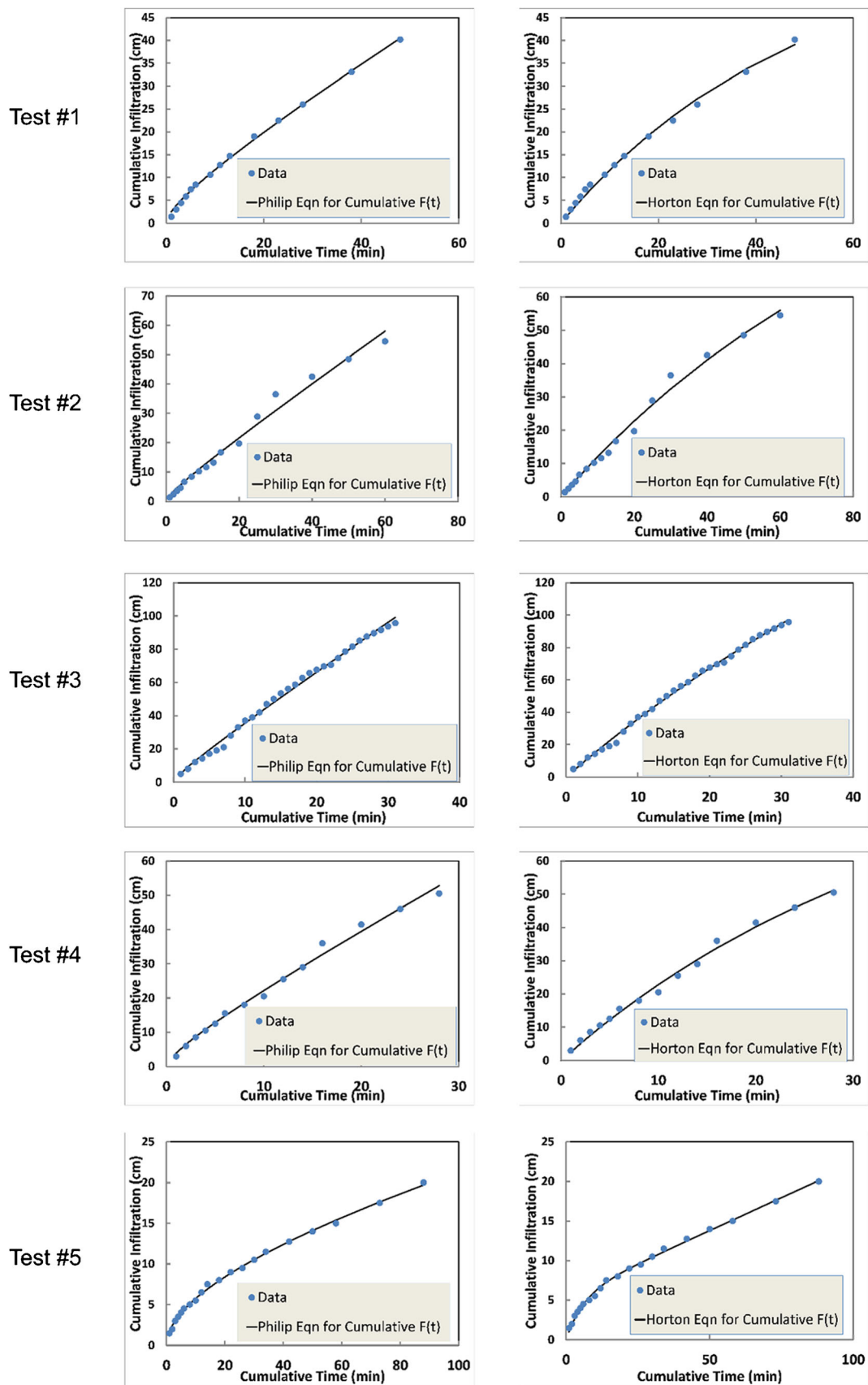located in the main catchment, it is common practice to either neglect the effect of these dams (i.e., consider the catchment as lumped without dams, this is considered as the first scenario) or consider that the dams are catching the flood water from their sub-catchments and the flood water in these sub-catchments is stored in the dam reservoir and no overflow occurs (this is the second scenario and called lumped with dams). Then the active catchment is the central basin and the rest of the catchment is inactive. Both of the aforementioned scenarios are unrealistic. Therefore, in the current study, a third and more realistic scenario is presented. In this third scenario, the central basin is active and the dam sub-catchments are partially active (i.e., reservoir routing is implemented and the outflow over dam spillway is estimated). A sketch of the three scenarios is shown in Fig. 9. In the current study, a comparison between two types of unit hydrographs has been analyzed. These unit hydrographs are the SCS unit hydrograph in HEC-HMS software (US Army Corps of Engineers HEC-HMS, http://www.hec.usace.army.mil/software/hec-hms/) and a unit hydrograph derived from stream flow data in a basin in Asir Province (Yiba basin; Farran and Elfeki 2019) located in the vicinity of the study

area. Details of the derivation of the unit hydrograph from stream flow data is presented in the following section.

The unit hydrograph

The UH is defined as the surface runoff hydrograph resulting from one unit of rainfall excess uniformly distributed spatially and temporally over a watershed for a specified duration. Two unit hydrographs are used in the analysis: the unit hydrograph of the SCS method and the unit hydrograph that is derived from Yiba basin which is located in Asir region within the study area. Yiba basin has a total area of 2830 km^2; however, the basin is divided into 4 sub-basins for areas that ranges between 322 to 2350 km^2 (Albishi et al. 2017). The unit hydrograph of Yiba basin is derived from streamflow data of 25 storms collected from four runoff stations located at the outlet of the sub-basins in Yiba catchment (Albishi 2015). Figure 10a and b show the derived UH of each storm and the mean unit hydrograph based on the methodology of Boorman and Reed (1981). In order to transfer the UH of Yiba basin to Itwad basin and its sub-basins, the UH of Yiba is expressed in discharge per unit of the drainage area by
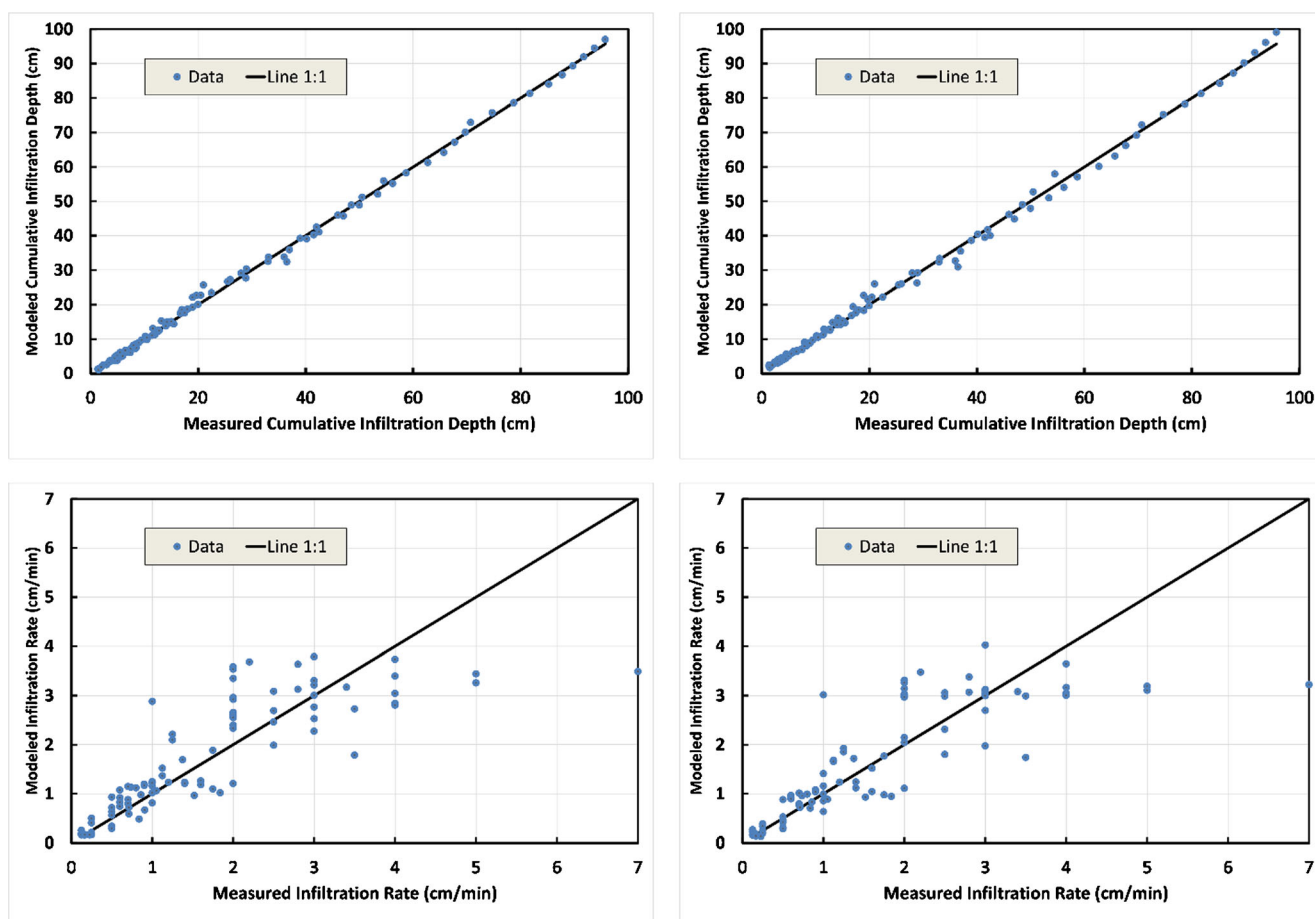

Fig. 12 Field-measured cumulative infiltration depth (top) and infiltration rate (bottom) versus modeled cumulative infiltration depth and infiltration rate according to Philip model (left column) and Horton model (right column)

dividing the UH of Yiba basin by the corresponding catchment area in square kilometers as shown in Fig. 10c. This graph is called the normalized UH. The corresponding UH for Itwad basin and its sub-basin is presented in Fig. 10d. The UHs of the Itwad basin and its sub-basin (Fig. 10d) are going to be used to calculate the storm hydrograph for each return period and will be compared with the one estimated from the UH of the SCS method in HEC-HMS software (US Army Corps of Engineers HEC-HMS, http://www.hec.usace. army.mil/software/hec-hms/). The results of the aforementioned methodology is discussed in the following section.

Discussion of the results

Results of the infiltration tests

The results of the fitting of the Philip model are presented in Fig. 11 (left column) and the corresponding parameters are displayed in Table 5 for each test. The correlation coefficient between the modeled and measured infiltration depths is very
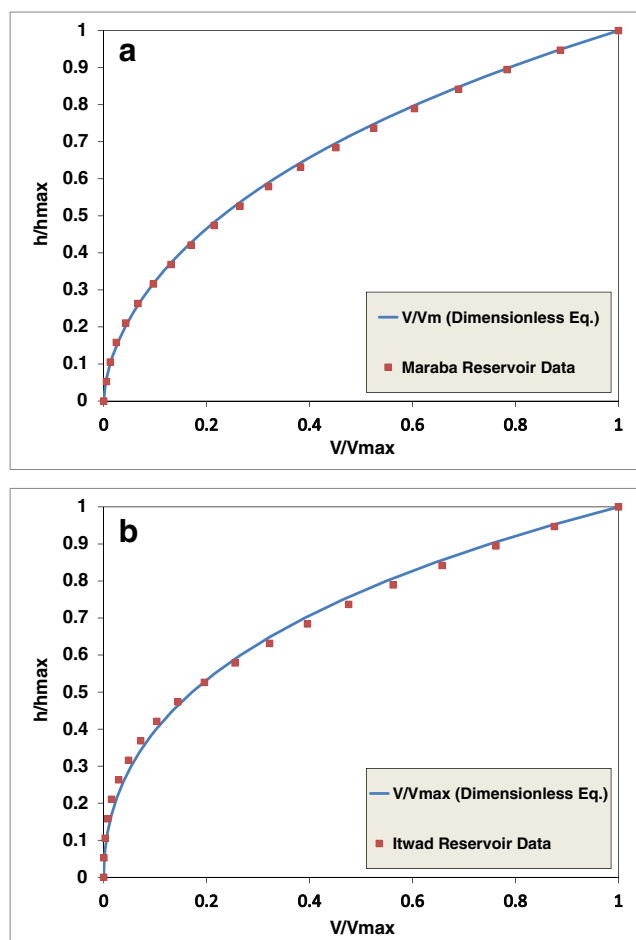
Fig. 13 Fitting dimensionless height–capacity curve for Maraba reservoir data (a) and Itwad reservoir (b)

high: it almost reaches 1. However, the parameters show a high degree of variability between the tests. This reflects the high degree of spatial variability of the soil parameters in the area. The sorptivity, S, varies between 0.45 and 0.99 m/day$^{0.5}$, while the hydraulic conductivity varies between 0.66 and 39.4 m/day. This is a typical issue in arid regions to have such a high degree of heterogeneity in the wadi alluvium (Farran and Elfeki 2019, 2020a).

The fitting of the Horton model is presented in Fig. 11 (right column) and the corresponding parameters are displayed in Table 5 for each test. The correlation coefficient between the modeled and measured infiltration depths is very high: it almost reaches 1. Figure 12 shows the agreement between measured and calculated infiltration rates and cumulative infiltrations as produced by both models. The correlation coefficient over all the tests for the Philip model is 0.998 and 0.817 for cumulative infiltration depths and infiltration rate, respectively. However, for the Horton model, it is 0.999 and 0.83 for both cumulative infiltration depths and infiltration rates, respectively. Both models fit the data pretty good. The correlation coefficient is higher for the equations describing the cumulative infiltration. The reason is that the integration process accumulates the data and then performs better.

However, the parameters of the Horton model show a high degree of variability between the tests as observed in the Philip model. The f_o varies between 15.4 and 55.3 m/day, while the f_c varies between 0.12 and 2.4 m/day, and the decay coefficient, k, varies between 15.4 and 237.8 day^{-1}. The ultimate infiltration rate, f_c, is approximately the hydraulic conductivity of the soil (~ 0.12 to 2.4 m/day). The high degree of variability in the parameters is confirmed by the Philip model as well. Even so, the range of hydraulic conductivity estimated by the Philip model (0.66 to 39.4 m/day) is different from the Horton model (~ 0.12 to 2.4 m/day). It is also obvious from Table 5 that there is no matching in the trend between both infiltration models. The reason is twofold: firstly, each model has its own parameters, e.g., the Philip model has two parameters, while the Horton has three parameters so these parameters influence the fitting procedure and may produce different results. Secondly, the Philip model seems to give more realistic results since the value of K_p is relatively higher than f_c in the Horton model that accounts for the transmission losses in ephemeral streams.

Results of the CN estimation

The values of CN are estimated from Eq. 10 based on K_p and f_c. It is obvious from Table 6 that CN(K_p) is lower than CN(f_c). The average values for both cases reflect high to moderate degree of infiltration rates. This in turn
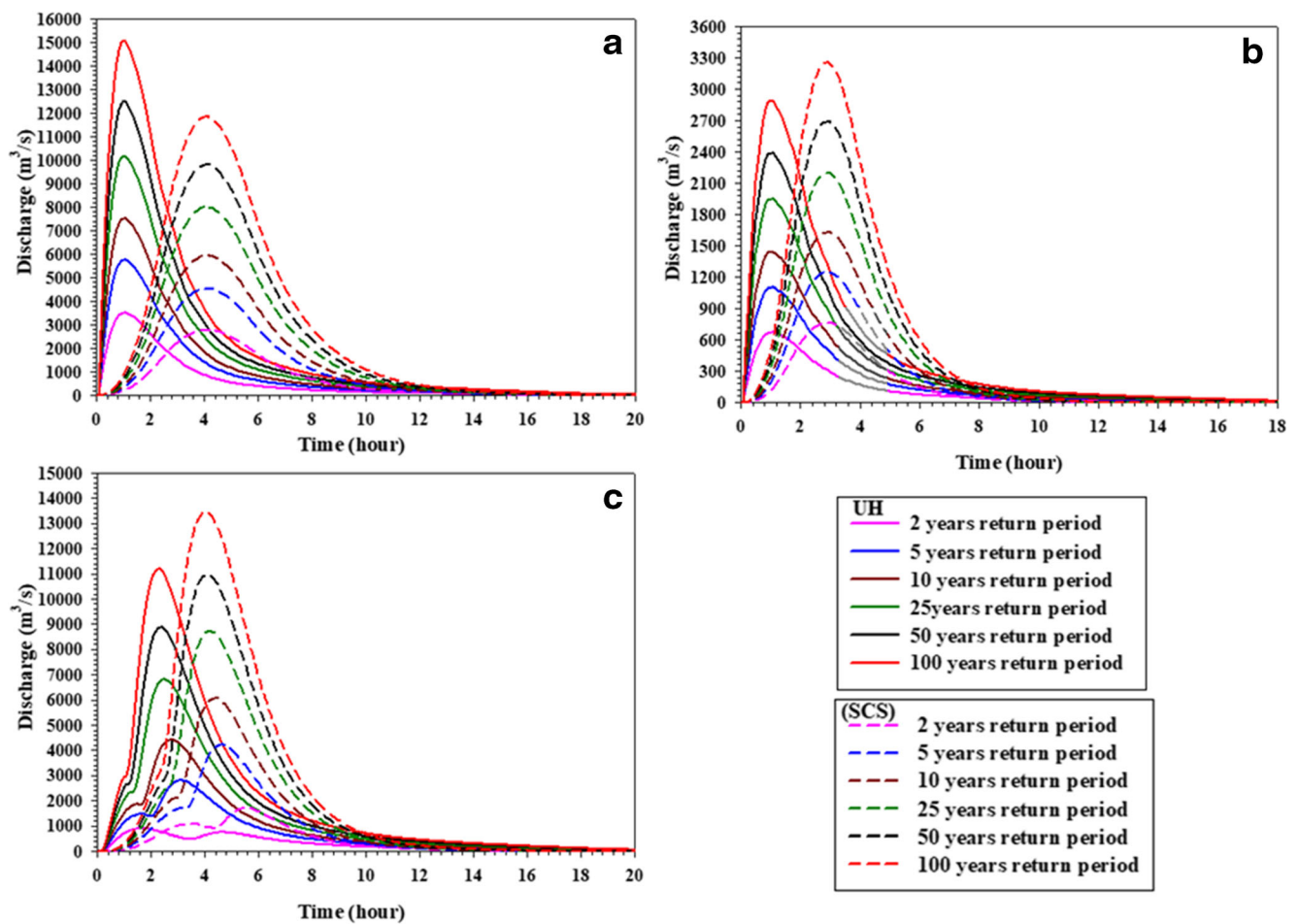
Fig. 14 Comparison between design flood hydrographs based on derived UH and SCS methods under various scenarios: a lumped whole basin case, b lumped central basin, and c routed hydrographs for sub-basins

accounts for the transmission losses that happen during the flood movement in the wadi bed.

>The percentage of the alluvium in the catchment was estimated to represent 20% of the catchment area, while the percentage of the rock outcrops represents 80% of the catchment. Since we do not have any measured values to estimate the CN for the rock outcrops, a value of 97 is used for basement (Masoud et al. 2013). Based on these values, the weighted average CN is 86.7 in the case of the Philip equation

(Table 5), while the weighted average CN is 90.5 for the case of using the Horton model (Table 5). The traditional (subjective) approach to estimate CN is based on the CN table (USDA-SCS 1963, 1985) and uses the matrix information for the hydrologic soil groups (HSG) and the land use/land cover (LULC). This traditional approach led to a weighted average value of 76.1. It is more objective to use the value of 86.7 rather than the subjective value of 76.1 since part of the first value is based on measured infiltration data. On the other hand, 86.7 is more rational than 90.5, since the value of 86.7 accounts for a moderate value of transmission losses.

Results of reservoir analysis

Figure 13 shows the dimensionless curves of the height–capacity curves for both dam reservoirs, and the fitting reservoir parameter, N, and the shape factor, M, are given in Table 7 for both reservoirs. Also the reservoir shape factor M shows that the reservoirs of both dams are of type III (floodplain foothill). This information is useful for further studies of reservoir routing and

Table 4   Data summary of the dams in the study area

Item	Wadi Itwad dam	Wadi Maraba dam
Height (m)	37	38
Storage (m³)	10,000,000	9,000,000
Dam length (m)	267	250
Spillway crest length (m)	170	170
Dam type	Earth fill	Earth fill
Purpose	Flood control	Groundwater recharge
Catchment area (km²)	516	536.4
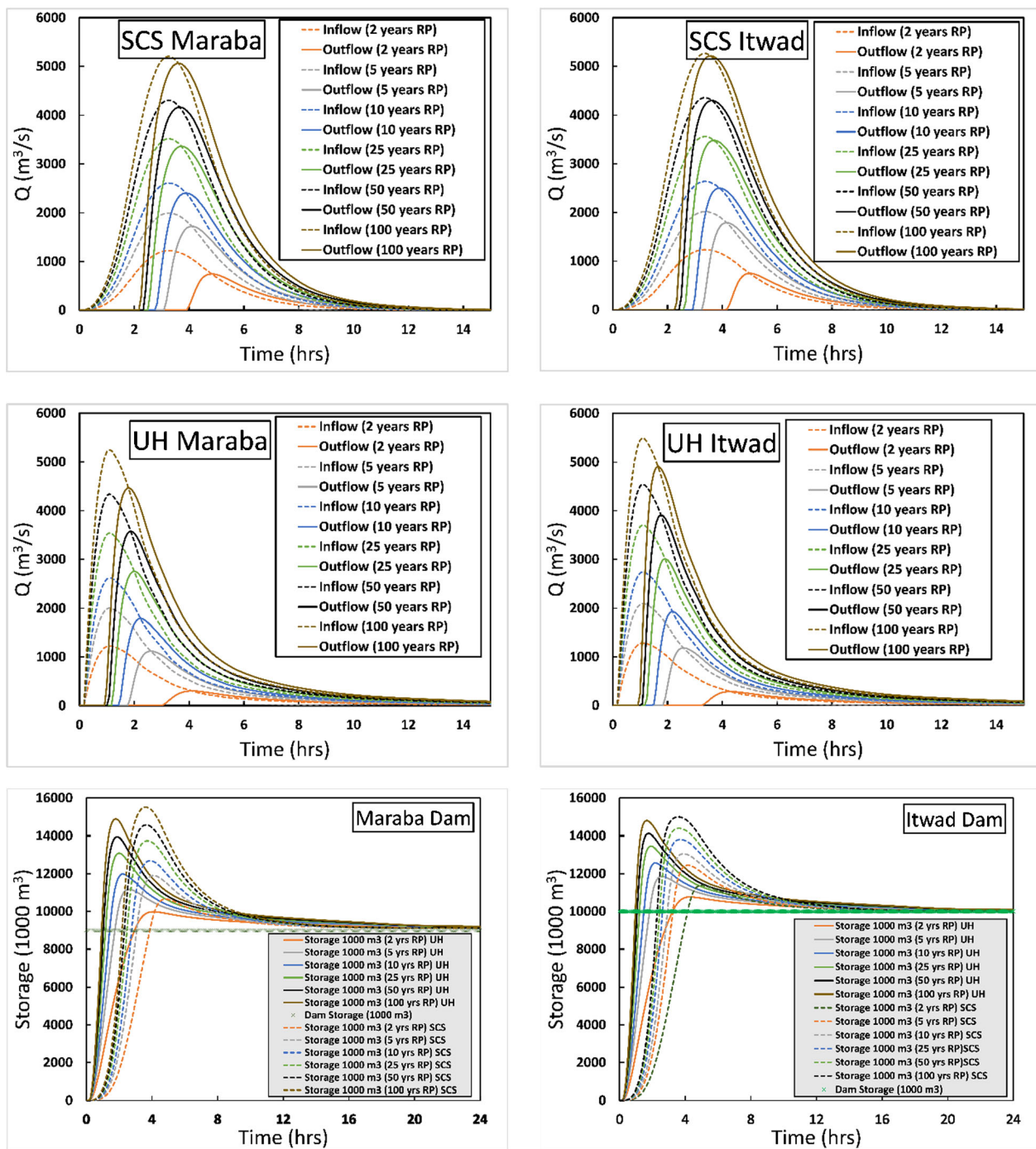
Fig. 15 Results of inflow and outflow hydrographs from the dam spillway and the change of storage over time in the dam reservoir under both SCS and UH models: left column, the Maraba dam; and right column, the Itwad dam

downstream flood studies (for more details, see Kamis et al. 2018).

Results of the hydrological analysis

The results of the proposed scenarios and the use of the two UHs are presented in Fig. 14. The figure shows a comparison

between design flood hydrographs based on the unit hydrograph and SCS method at different return periods and under various scenarios: (a) lumped whole basin case, (b) lumped central basin, and (c) routed hydrographs for sub-basins. The summary of the results is presented in Table 8.

The general analysis of the scenarios shows that the UH method produces hydrographs that are very typical of arid

Table 5   Fitting parameters of Philip and Horton infiltration equations

Infiltration test	Location		Philip parameters		Correlation coefficient	Horton parameters			Correlation coefficient
	Latitude (deg)	Longitude (deg)	S (m/day$^{0.5}$)	K_p (m/day)		f_o (m/day)	f_c (m/day)	k (1/day)	
T1	17.90689	42.42144	0.72	8.2	0.999	18.5	1.2	32.9	0.998
T2	17.82525	42.37569	0.45	11.7	0.993	18.1	0.14	15.4	0.996
T3	17.98	42.35214	0.98	39.4	0.998	55.3	0.23	19.7	0.999
T4	17.8985	42.49467	0.99	20	0.996	37.4	0.12	38.7	0.997
T5	17.98058	42.36711	0.63	0.66	0.999	15.4	2.4	237.8	0.999

regions when compared with the SCS method. The storm hydrographs that are developed by the UH method are characterized by a steep rising limb with a short time to peak, a rapid recession to zero base flow, and less flood volume due to transmission losses. These features are not presented by the SCS method.

The results of the lumped basin scenario are presented in Fig. 14a. The UH method produces higher peaks than the SCS method for different return periods. The peak flow ranges between 3529 and 15,104 m^3/s for 2- to 100-year return periods, respectively, based on the UH method, while it ranges between 2784 and 11,846 m^3/s for 2- to 100-year return periods, respectively, based on the SCS method. This lumped case leads to overdesign of the protection measure in the area since the dams are not included in the analysis.

The results of the lumped central basin scenario are displayed in Fig. 14b. In this case, the UH method shows also less peaks than in the case of the SCS method for different return periods. The peak discharge ranges between 675.9 and 2892.5 m^3/s for 2- to 100-year return periods, respectively, based on the UH method; however, it ranges between 763.4 and 3252 m^3/s for 2- to 100-year return periods, respectively, based on the SCS method. This central basin scenario leads to underdesign of the protection schemes in the area since the dams are assumed to store the flood water and they do not contribute their flow in the downstream area through the spillway or dam outlets which is not realistic.

Table 6   Estimation of CN from the empirical equation for wadi alluvium based on the Philip, CN(K_p), and Horton, CN(f_c), models

Infiltration test	K_p (m/day)	CN(K_p)	f_c (m/day)	CN(f_c)
T1	8.2	45.9	1.2	57.9
T2	11.7	43.7	0.14	71.3
T3	39.4	36.1	0.23	68.2
T4	20	40.4	0.12	72.2
T5	0.66	61.6	2.4	53.6
Average	16.0	45.5	0.8	64.6

The results of the scenario for routed hydrographs for sub-basins are shown in Fig. 14c. In this case, the UH method shows less peaks than in the case of the SCS method for different return periods. The maximum discharge ranges between 895.3 and 11,197 m^3/s for 2- to 100-year return periods, respectively, based on the UH method; however, it ranges between 1713.8 and 13,463 m^3/s for 2- to 100-year return periods, respectively, based on the SCS method. This case is the realistic case, since it takes into account the effect of the dams and the flow is routed over the spillway after storage of the dam capacity. This would lead to a rational design case. The design values in this case fall between the two above cases.

It is worth mentioning that both dam storage capacities (see Table 4) are different from the estimated runoff volume from the 2-year rainfall data (see scenarios in Table 8). This is because the dam storage capacities in Table 4 belong to each dam, while the runoff volume in Table 4 belongs to the basin as a whole under various scenarios. Figure 15 shows the results of the dam reservoir routing for both SCS and UH models and the changes of the dam reservoir over time. It is obvious from these graphs that the UH models show features that are common in arid regions, i.e., the steep rising limb of the inflow hydrographs, short time to peak, steep lowering limb, and long tails. Also, the storage–time curve shows that in the UH model, the reservoir is filled faster than in the case of the SCS method. The figure shows the convergence of the curves toward the reservoir capacity at late times. The curves show that the reservoir capacities are designed for a return period of a maximum of 5 years.

Regarding the magnitude of the peak flows and flood volumes obtained in the study area, it seems the values are within the range of some observations in wadi Itwad and in some wadis in the vicinity of Itwad basin. Walter (1989) has shown a unique event in wadi Ghat located in Asir area with a peak discharge of 3219 m^3/s. Al-Turki (1995) has reported peak discharges of some wadis in the Asir region. In wadi Itwad, there was an

Table 7 Dam reservoir characterization

Dam reservoir	N (reservoir parameter)	M (shape factor)	Reservoir type	Standard classification
Itwad reservoir	0.5	2.33	Floodplain foothill	Type III
Maraba reservoir	0.6	1.90	Floodplain foothill	Type III

exceptional flood of 4000 m³/s; in wadi Baysh, there is a flood of 10,000 m³/s; in wadi Damad, there is a flood of 4000 m³/s on the 23rd of April 1990 (flood volume of 385,240 million cubic meters (MCM)); in wadi Hali, there is a flood of 6000 m³/s on the 13th of February1987, and in the same wadi, there was a flood of 2280 m³/s on the 10th of May 1990. In wadi Dogah, there was a peak flood of 668 m³/s on the 17th of February 1983 (flood volume of 122,030 MCM). The observed peak discharges indicated that the region experienced extreme floods.

In wadi Itwad during normal years, the runoff volume ranges between 50 and 100 MCM (Al-Turki 1995). The simulated volumes presented in Table 8 are within the observed range in the data up to 10-year return period for the UH method.

Regarding the time to peak, Table 8 shows the time to peak for both the UH method and the SCS method. The time to peak in the case of the UH method is less than in the case of the SCS method. This proves that the UH method is more realistic than the SCS method, since this short time to peak is typical for flash floods (see examples of some events in Walter 1989 and Farran and Elfeki 2020a).

Conclusions

The proposed unified approach presented in the current study for flood risk assessment is systematic. It shows a step-by-step hydrological analysis and covers various aspects in the hydrological modeling used in flood assessment studies. The following conclusions can be drawn from the study:

– The correlation coefficient between observed and estimated infiltration shows that both the Philip and Horton models are pretty high (e.g., 0.998 and 0.817 for cumulative infiltration depths and infiltration rate, respectively, for Philip, and 0.999 and 0.83 for cumulative infiltration depths and infiltration rates, respectively, for Horton). The correlation coefficient is higher for the equations describing the cumulative infiltration. The reason is that the integration process accumulates the data and then

performs better. Both models are reasonably good to represent the infiltration tests.

– The fitting parameters in the infiltration models show high variability for both models (Philip and Horton) which is typical in arid zones; however, the Philip model provides more realistic results since it produces relatively higher hydraulic conductivity that justifies the transmission losses which is a typical phenomenon in arid basins.

– The empirical equation of Van Mullem (1989) for CN estimation from hydraulic conductivity could provide a plausible estimation of CN of the alluvium, and therefore, it can help provide a more objective CN estimation rather than using the SCS-CN tables in the undeveloped wadis (such as Itwad wadi).

– The dam reservoirs were analyzed and the height–capacity curves for both dam reservoirs and the fitting reservoir parameter, N, and the shape factor, M, are evaluated. The shape factor M shows that the reservoirs of both dams are of type III (floodplain foothill). This information is useful for further analysis of reservoir routing, reservoir sedimentation upstream of the dam, and flood studies in the downstream area of the dam.

– The proposed approach considers two hydrograph generation methods: one is based on the common SCS method and the other is based on the UH theory derived from a catchment in the vicinity of the study area (Yiba catchment). The analysis shows that the UH method produces hydrographs that are very typical of arid regions when compared with the hydrographs generated by SCS method. It is characterized by a steep rising limb with a short time to peak, a rapid recession to zero base flow, and less flood volume due to transmission losses.

– The studied scenarios show that the lumped case leads to overdesign of the protection schemes, while the lumped central basin with detection dams that store the flood water completely leads to underdesign in the downstream area. The routed scenario is more realistic, since it considers the detention dams with over flow from the spillways.

– The magnitude of the estimated flood peaks and the runoff volumes of the 10-year return period based on the UH method of the case with routed scenario are within the

Table 8 Summary of the hydrological scenarios

Return period	Hydrological setup																	
	Lumped						Central basin						Routing sub-basins					
	SCS			UH			SCS			UH			SCS			UH		
	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)	Q_p (m³/s)	Time to peak (min)	Outflow volume (MCM)
2	2784	245	47.92	3529	60	40.2	763.4	175	9.2	675.9	60	7.7	1713.8	325	28.8	895.3	100	21.1
5	4546	245	78.23	5770.6	60	65.2	1247	175	15.0	1105.1	60	12.5	4228.3	275	59.1	2808.2	185	46.5
10	5935	245	102.11	7541.6	60	85.7	1629	175	19.6	1444.3	60	16.3	6076.1	265	83.0	4428.5	165	66.6
25	8009	245	137.76	10,193	60	115.6	2198	175	26.4	1952	60	22.0	8727.9	250	118.7	6828.7	150	96.5
50	9810	245	168.72	12,500	60	141.6	2693	175	32.3	2393.9	60	27.0	10,973	245	149.6	8892.4	140	122.5
100	11,846	245	203.71	15,104	60	170.9	3252	175	39.0	2892.5	60	32.0	13,463	240	184.6	111,197	135	151.9

observed values in some historical data reported in the literature. For example, the predicted peak discharge (4428.5 m³/s) and the predicted runoff volume (66.6 MCM) of the 10-year return period of the routed scenario are in the order of magnitude within the observed historical data of 4000 m³/s and 50 to 100 MCM, respectively, as reported in the literature.

– The study concludes that the proposed methodology is robust and takes into account the Saudi arid environment while performing flood risk assessment in the KSA.

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