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A Comparison of the Design Peak-Flow Estimated by the Simulated and Observed Storm Hydrographs

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Abstract:- The estimation of peak-flow corresponding to a given return period is a significant factor in designing hydraulic structures. There are many methods used to estimate the peak-flow. In this study, two methods were used to estimate the peak-flow. Firstly, an appropriate probability distribution function is fitted to the recorded annual maximum (AM) wadi-flow series to determine wadi-flow rate with a certain exceedance probability. Secondly, in the absence of long-term wadi-flow data, peak-flows simulated by rainfall-runoff model are used. In this study, these two methods were used to estimate the differences between the design peak-flows in the Wadi Al-Khoud catchment area. For the use of hydrological modelling, Intensity-Duration- Frequency (IDF) curves were developed by using General Extreme Value Probability Distribution (GEV) function. Kolmogorov-Smirnov test was used for testing the goodness of GEV fit with observed data. Comparison of IDF curves developed for the Wadi Al-Khoud area and the once presented in Highway Design Manual in Oman (2010) indicated that the difference between the IDF curves becomes larger as the return period increases. Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) and the rainfall-runoff model (HEC-HMS) were used for delineating the catchment area and simulating rainfall-wadi flow relation. The 10-year peak-flow estimated by the observed wadi-flow records is 503.37 m^3/s, which is much different from the average peak-flows of the simulated 10 scenarios (2877.82 m^3/s). This difference can be attributed to the absence of the long-term rainfall and wadi-flow data for the probability estimations and the inability to capture the spatial distribution of the rainfall over a large catchment area as Wadi Al-Khoud catchment.

***Keywords**:- Annual Maximum Wadi-Flow, GEV Distribution, HEC-HMS, IDF Curves, Peak-Flow, Wadi Al-Khoud*

I. INTRODUCTION

The safety of hydraulic structures such as bridges, spillways and urban drainage systems requires hydraulic engineering. In particular, design flood events, which explain magnitude–frequency relationship at a particular site in a specific region, is necessary for the planning, design and operation of hydraulic structures (Pegram & Parak, 2004). The return period for designing a hydraulic structure is prime important because it governs the economic aspects of the design, safety, size and the nature of the hydraulic structure. Peak-flow estimation requires long-term records for the particular study region.

Probability distribution functions are fitted to the maximum-recorded flood series to determine flood discharges of different probabilities. However, choosing the best-fitted probability distribution function is often controversial. In addition, as parameters are estimated from the sample data, any error in the recorded data will propagate through the results. Furthermore, the short length of observed data, outliers and missing data lead to uncertainty in the extrapolation of floods estimated by the flood frequency method (Saghafian *et al*., 2014).

Beven (2003) and Maskey (2004) reported that rainfall–runoff modeling can also be considered as one of the approaches for designing flood events.

In the absence of long-term wadi-flow data, peak-flow simulated by rainfall-runoff models with the appropriate rainfall inputs can be considered in designs. Rainfall-runoff models are capable of handling non-linear relationships between the hydrological processes, which makes them useful in hydraulics studies. It requires the use of design rainfall events with appropriate antecedent conditions of the catchment. Alternatively, in perennial catchments, continuous simulation can be carried out with sufficiently long historical or simulated rainfall to reduce the uncertainty of the antecedent moisture conditions (Zeng *et al*., 2016).

Runoff occurs whenever rain intensity exceeds the infiltration capacity of the soil, providing there are no physical obstructions to surface flow. Surface runoff modeling is used to understand catchment yields and responses, estimate water availability, changes over time and forecasting (Vaze, 2012). Rainfall-runoff models can produce results over space and time and representations of internal flow processes. Associated parameters of the various hydrological processes can be calibrated and verified with known site conditions, rainfall inputs, and observed wadi-flow records (Beven, 2003). Catchment urbanization, land-use and land-cover are some dominant factors affecting the performance of rainfall–runoff modeling. Parameters in governing equations that relate to the land-use type can be altered at different times to account for the urbanization history of the specific area.

Among the various rainfall-runoff models available, HEC-HMS model is commonly used. The software includes many traditional hydrologic analysis procedures such as

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infiltration, unit hydrographs and hydrologic routing. Uncertainties in the model structure, input data and calibrated model parameters affect the outputs of hydrological models (Refsgaard & Storm, 1996). In addition, there are many limitations to obtain reliable results such as the wind and its effect on the rainfall distribution, evaporation loss and temperature effects.

Sultanate of Oman is located in the southeastern part of the Arabian Peninsula, which is covered with different landforms. Oman is characterized by hyper-arid, through the arid and semi-arid environments that are experienced in different parts of the country. The surface runoff in the wadis evaporates in few days due to the hot atmosphere, which is common in the whole of Oman. Absence of the frequent rainfall events and long-term data is one of the major barriers for hydrological modelling in Oman.

This study considered the above two methods to compare the design peak-flow estimated by the hydrological model and observed wadi-flow data. The first method used the observed wadi-flow data and their probability distribution for developing peak-flow frequency relationships. The second method used the rainfall-runoff model developed using HEC-HMS software and HEC-GeoHMS software. The results were discussed for understanding the difference between their estimations and the applicability of these methods in arid watersheds in Oman.

A. Study Area

Sultanate of Oman is on the southeastern side of the Arabian Peninsula, (Figure 1). The land area of Oman is approximately 309500 km^2. Watersheds in Oman are characterized as semi-arid to hyper-arid and receive spatially variable rainfall that fluctuates based on each area's physiographic properties and rainfall season. The country's rugged landscape patterns include mountain ranges such as; Al-Hajar Mountains (average of elevation is 1220 m) and Qara Mountains (average of elevation is 915 m), coastal plains such as; Salalah Plain (64 km) and Al-Batinah Plain (240 km) and deserts which is covering approximately 62400 km^2. The annual rainfall in Oman is less than 100 mm, which is significantly less than the global annual rainfall of 1123 mm (Gunawardhana & Al-Rawas, 2016).

Wadi Al-Khoud is located in Muscat and is considered one of the largest wadi basins in the Sultanate, (Figure 1). Wadi Al-Khoud catchment has an area equal to 1660 km^2 and the longest wadi-length is about 92 km. The maximum flow recorded in Wadi Al-Khoud was 2329 m^3/s in 1950. Wadi Al-Khoud is characterized by a high range of relief with elevation ranges from 2462 m above sea level and with slopes reached to 76% at the hill slope to sea level downstream. The wadi channel bed materials are mainly gravel and sands with weak base flow mainly downstream originated from groundwater seepage. Land cover is a mainly bare rock with sparse vegetation (Abdel-Fattah *et al.*, 2018). There are 7 rainfall gauges with annual average rainfall in 1994-2004 ranges from 53 mm at Al-Khoud station to 175 mm at Al-Afia station (OMRM, 2008).

B. Data Collection

Simulation of the wadi-flow with respect to the different return periods requires rainfall input obtained from intensity-duration-frequency (IDF curves). Because, catchment scale IDF curves are not available, in this study, they were developed by using hourly rainfall records observed near the catchment outlet (Al-Khoud station in Figure 1).

Hourly averaged wadi-flow data observed at the same location as the rainfall was used to calibrate and validate the hydrological model. In addition, these wadi-flow data were used for the frequency analysis. A common period of wadi-flow and rainfall data were used, which spans from 1996 to 2013. Figure 1 shows all the rainfall stations are within the study area. Due the availability of the data, only the Musbit, Al-Buri, Jabal Al-Hayl and Al-Khoud rainfall stations were used to calibrate and validate the hydrological model (Table 1).

II. METHODOLOGY

A. Frequency analysis

Statistical flood frequency analysis is a technique used by hydrologists to predict flow values corresponding to specific return periods or probabilities in a particular area. The frequency analysis is also used to develop the IDF curves by fitting the AM rainfall in different duration to a particular probability distribution function.

In this study, the development of the IDF curves is based on the AM rainfall series and the GEV distribution function. The AM hourly rainfall events were extracted for 18 years of observation. When the rainfall observations were available only on the daily scale, K-NN method was used for disaggregation of coarse resolution data into the hourly scale (Uraba *et al.*, 2019). The 1-hr rainfall time series were aggregated to 2-, 4-, 8-, 10-, 12-, 14- and 16-hours total rainfall to represent the duration in IDF plots. For the frequency analysis: 2-, 5-, 10-, 25-, 50- and 100-years return periods were selected. Rainfall intensities developed for the above durations and return periods were then compared with the Highway Design Manual (2010) in the Muscat area, (Figure 2).

The GEV distribution is a family of probability distributions that combines the Gumbel, Fréchet and Weibull distributions. It is used to model the smallest or largest value among a large set of independent, identically distributed random values representing observations. GEV makes use of three parameters: location parameter (μ), scale parameter (σ), and shape parameter (k). The cumulative distribution of the GEV function can be calculated by using Equations 1 and 2.

$$G\,(x) = exp^{\left\{-(1-\frac{\mu(x-\kappa)}{\sigma})^{\frac{1}{\kappa}}\right\}} \qquad (1)$$

Where x is the return value (intensity of the rainfall) in mm/hr. To calculate x for a specific return period, Equation 2 can be used.

$$G(x) = 1 - \frac{1}{T} \qquad (2)$$

Where *T* is the return period in years.

Different values of κ lead to the three extreme value distributions as follows:

$\kappa > 0$ corresponds to the Fréchet distribution.
$\kappa < 0$ corresponds to the Weibull distribution, and
$\kappa = 0$ corresponds to the Gumbel distribution.

The Kolmogorov-Smirnov test was used to assess the goodness of fitted distribution with the probability distribution of the rainfall observations (Smirnov, 1939). The test is based on the greatest vertical distance (D) between the cumulative distribution of the fitted and the observed rainfall series. The null-hypothesis, which states that the two data set values are from same distribution is rejected if the D is greater than the critical value at a chosen significance level (α) and if the probability (P) of the cumulative distribution of the hypothesized distribution is smaller than the critical value at the same α. The significance level of $\alpha = 0.05$ was used in this study and the critical value is constant (0.31).

Similar to the frequency analysis of the rainfall data, AM wadi-flow was extracted and used with GEV distribution to calculate the relationship between the peak-flow and the return period. The 10-year return period was selected to compare the peak-flow difference from two methods.

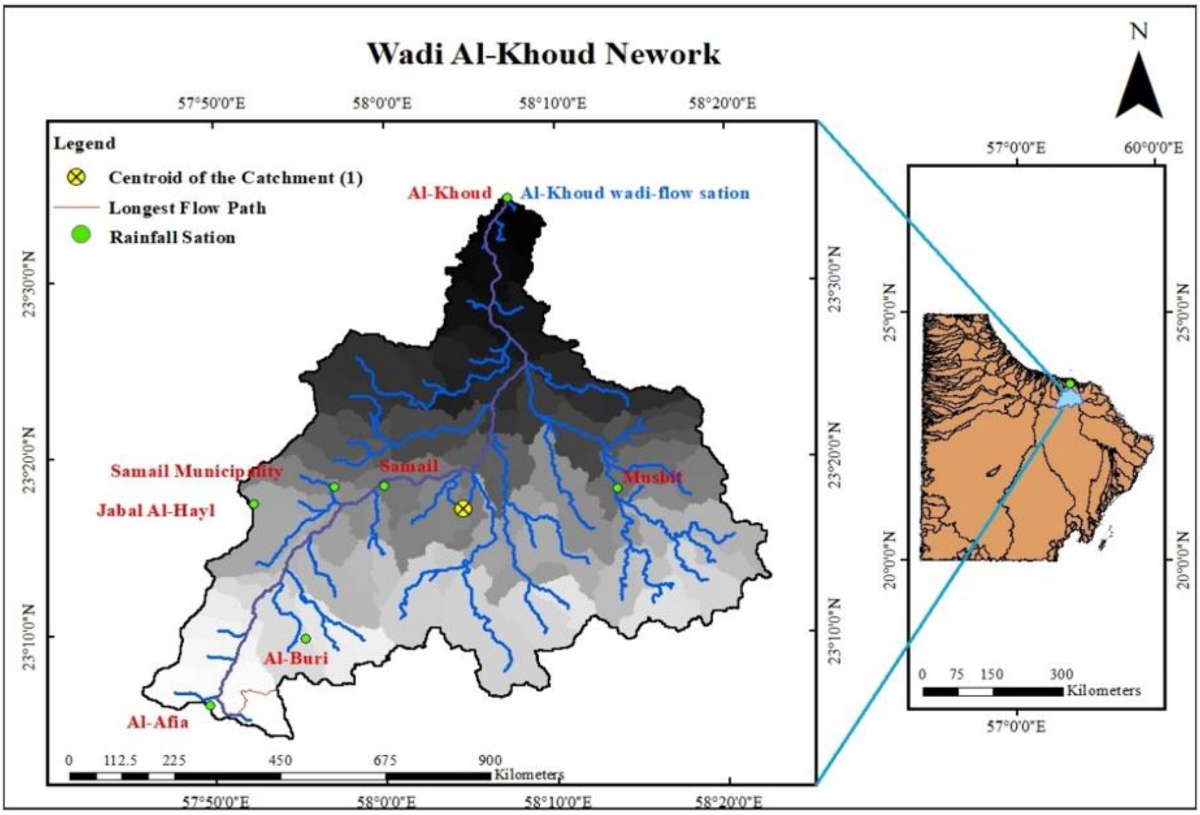

Fig 1. Location and wadis network of Wadi Al-Khoud.

Table 1. The duration of the rainfall-runoff model events.

Event	Start Date	Start Time	End Date	End Time
Event 1	18 Mar 2007	00:00	19 Mar 2007	12:00
Event 2	01 May 2013	15:00	02 May 2013	12:00
Gonu Event	06 Jun 2007	20:00	08 Jun 2007	23:00

B. Rainfall-Runoff Model

A rainfall-runoff model was developed using HEC-HMS for the Wadi Al-Khoud catchment area. Wadi-flow was simulate using rainfall derived from the IDF curves. SRTM (Shuttle Radar Topography Mission) DEM (Digital Elevation Model) data at 90 m resolution was obtained from the USGS Earth Explorer website. This data was analyzed using HEC-GeoHMS and ARC-GIS 9.2 software to delineate the catchment area and the wadi-network.

Because the HEC-HMS model is a lump hydrological model the rainfalls from several rain gage stations were averaged to the centroid of the catchment area using the Inverse Distance Weighting (IDW) method. This method assigns a weight to the station based on the inverse distance from the gage to the centroid of the catchment area (Equation 3).

$$W(r) = \frac{\frac{1}{d(r)^2}}{\frac{1}{d(a)^2}+\frac{1}{d(b)^2}+\frac{1}{d(c)^2}} \qquad (3)$$

Where *W(x)* is the weight of a specific rainfall gage station, *d(r)* is the distance from the specific rainfall gage station to the centroid in m, and *d(a), d(b)* and *d(c)* are the distances from the other rainfall gage stations to the centroid in m. Then the rainfall average $(\overline{P})$ of all weighted rainfall gage stations was calculated by using this formula:

$$\bar{P} = Wa\,Pa + Wb\,Pb + Wc\,Pc + Wr\,Pr \qquad (4)$$

Where *Pa*, *Pb*, *Pc* and *Pr* are the recorded rainfall in each gage station in mm.

Precipitation loss is one of the main factors that influence direct runoff in the basin. The loss models in HEC-HMS normally calculate the runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. In this study, SCS-CN method was selected to estimate direct runoff. This model estimates precipitation excess as a function of cumulative precipitation, soil cover, land-use and antecedent moisture condition using the following equation:

$$Q = \frac{(P-I_a)^2}{P-I_a+S} \qquad (5)$$

Where Q is the accumulated precipitation excess at time *t* (inches), *P* is the accumulated rainfall depth at time *t* (inches), I_a (inches) = 0.2 *S*, *S* is the potential maximum retention (inches), which is measuring the ability of a catchment to abstract and retain storm precipitation and can be expressed as the following equation:

$$S = \frac{1000}{CN} - 10 \qquad (6)$$

Where *CN* is used to represent the combined effects of the primary characteristics of the catchment area, including soil type, land-use and (AMC).

The transformation prediction models in HEC-HMS convert calculated excess rainfall to a time varying direct runoff. In this study, the SCS-UH method was chosen. This model requires T_{lag} to be calibrated. The SCS suggests that the T_{lag} may be related to time of concentration as in Equation 7.

$$T_{lag} = 0.6\,T_C \qquad (7)$$

Where T_C is the time of concentration in minutes, which can be calculated using the Kirpich formula (Kirpich, 1940):

$$T_C = K\,L^{0.770}\,S^{-0.385} \qquad (8)$$

Where *K* is a unit conversion coefficient, which is equal to 0.0078, *L* is the main channel flow length in m and *S* is the dimensionless main channel slope. These parameters were found by the catchment delineation using DEM and ARC-GIS software.

C. *Calibration and verification*

In this study, *CN*, T_{lag} and I_a were calibrated using observed wadi-flow data. Two storm events recorded during the study period were selected for calibration and Gonu Event was used for validating the parameters (Table 2). In this study, Percent Error in Peak (*PEP* in Equation 9) and Peak-Weighted RMS Error (PW-RMS in Equation 10) were used as objective functions for calibration. In addition, the Nash-Sutcliffe efficiencies (*NSE* as in Equation 11) method was used to measure the efficiency of the simulated and observed data in the validation event.

PEP is a goodness of fit test that measures only the goodness of fit of the computed hydrograph peak to the observed peak, which can be expressed as the following formula:

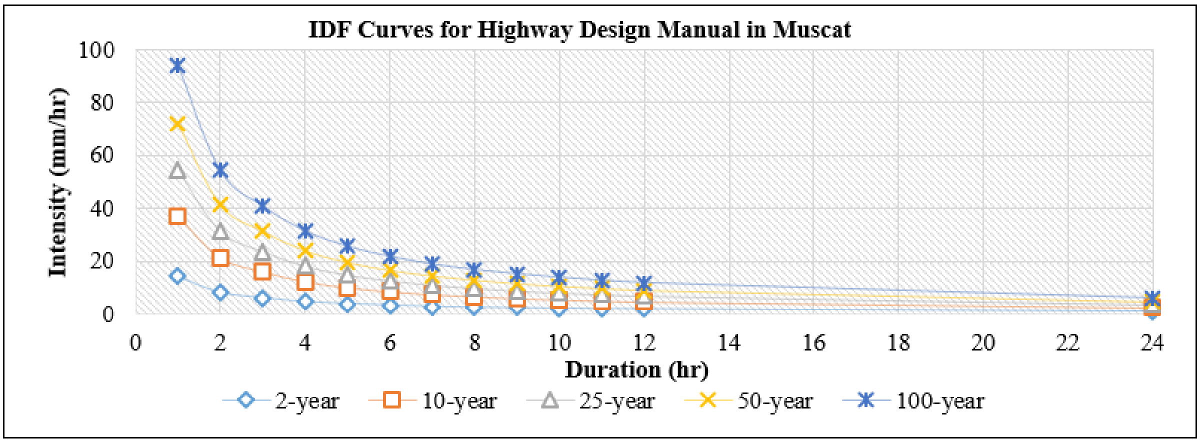

Fig 2. The standard IDF curve for Muscat area (Highway design manual, 2010: Sultanate of Oman).

$$PEP = 100\,\frac{q_S(peak)-q_O(peak)}{q_O(peak)} \qquad (9)$$

Where *qo (peak)* is the observed peak-flow in m^3/s and *qs (peak)* is the calculated peak-flow in m^3/s.

PW-RMS function uses all ordinates, square differences and wights the squared differences.

$$E = \left\{\frac{1}{NQ}\left[\sum_{i=1}^{NQ}(q_O(i) - q_S(i))^2 \left(\frac{q_O(i) - q_O(mean)}{2q_O(mean)}\right)\right]\right\}^{1/2} \quad (10)$$

Where E is the PW-RMS Error, NQ is the number of computed hydrograph ordinates, $q_o(i)$ is the observed flows in m^3/s, $q_s(i)$ is the simulated flows that computed with a selected set of the model parameter in m^3/s and $q_o(mean)$ is the mean of observed flows in m^3/s.

For *NSE* method, an efficiency of $NSE = 1$ corresponds to a perfect match of modelled discharge to the observed data, whereas an efficiency less than zero ($-\infty < NSE < 0$) occurs when the observed mean is a better predictor than the model. *NSE* can be calculated by Equation 11.

$$NSE = 1 - \frac{\sum(q_O - q_S)^2}{\sum(q_O - \overline{q_o})^2} \quad (11)$$

Table 2. Rainfall events selected for calibration and validation.

`	**Date**	**Calibration / Validation**	**Peak-flow (m^3/s)**	**DRO volume (m^3)**
Event 1	18/3/2007	Calibration	781.9	14995.4
Event 2	1/5/2013	Calibration	93.9	3258.6
Gonu Event	6/6/2007	Validation	988.3	47991.3

III. RESULTS AND DISCUSSION

A. *IDF Curves*

Frequency analysis was carried using the recorded AM rainfall for 18 years at Wadi Al-Khoud station to develop IDF curves. Table 3 shows the estimated parameters of the GEV distribution. As shown, κ values for all return periods are greater than 0 with average equals to 0.5, which corresponds to a Fréchet distribution and leads to the distribution being upper bounded. The fitted GEV distributions match with the distributions of AM rainfall values because none of the fitted distributions rejects the null-hypothesis. It can be noted that GEV is fitted almost perfectly in the provided data according to the Kolmogorov-Smirnov method test statistics (Table 3). The null-hypothesis indicates that the fitted GEV function and the AM rainfall have approximately the same distribution.

The IDF curves developed for the Wadi Al-Khoud station are shown in Figure 3. It can be seen that the rainfall intensities generally decrease with an increase in duration for a given return period. In general, larger hydrological structures such as dams and bridges are designed for higher return periods while small hydrological structures such as culverts and drainage gutters are designed for low return periods. The uncertainty of the IDF curves becomes higher when long return periods are used because of the climate changes in the future. However, the uncertainty can be reduced if long duration rainfall data are used in calculations.

Evidently, a comparison of Figure 2 and Figure 3 shows significant differences between rainfall intensities for all durations. Figures 4 show that the difference between two curves increases as the return period increases. In addition, these differences for larger durations are higher than those for smaller duration. For example, the 21% difference estimated for a 2-year return period in 1-hour duration increases to 54% for a 100-year return period, and the 24 % difference estimated for a 2-year return period increases to 81% for a 100-year return period for 12-hour duration. These results, therefore, concluded that the design of hydraulic structures should depend on the site specific IDF curves as much as possible.

B. *Peak-flow frequency analysis using wadi-flow data*

Figure 5 shows the relationship between the peak-flow rate and the return period estimated using observed wadi-flow data. The results of μ, σ and κ parameters are 114.12, 55.94 and 0.45 respectively. κ value is greater than 0, which indicates that the data corresponds to the Fréchet distribution. Kolmogorov-Smirnov test indicates that fitted GEV distributions match with the distributions of wadi-flow data. The fitted distribution did not reject the null-hypothesis because D equals 0.24 and the critical value at $\alpha = 0.05$ is equal to 0.31, which is greater than D.

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Table 3. GEV distribution and Kolmogorov-Smirnov test results.

Duration	κ	σ	μ	D	P	Reject (null-hypothesis)
1-hour	0.43655	9.29520	7.50	0.18388	0.51812	No
2-hour	0.44496	5.61620	4.67	0.08770	0.99693	No
4-hour	0.47673	2.89480	2.59	0.10847	0.96842	No
6-hour	0.47389	1.95080	1.91	0.13487	0.85646	No
8-hour	0.48669	1.51370	1.50	0.13350	0.86436	No
10-hour	0.53606	1.16990	1.18	0.13892	0.83203	No
12-hour	0.54960	0.97030	0.98	0.14025	0.82370	No
14-hour	0.56597	0.82768	0.84	0.14249	0.80934	No
16-hour	0.56643	0.72438	0.73	0.14165	0.81476	No

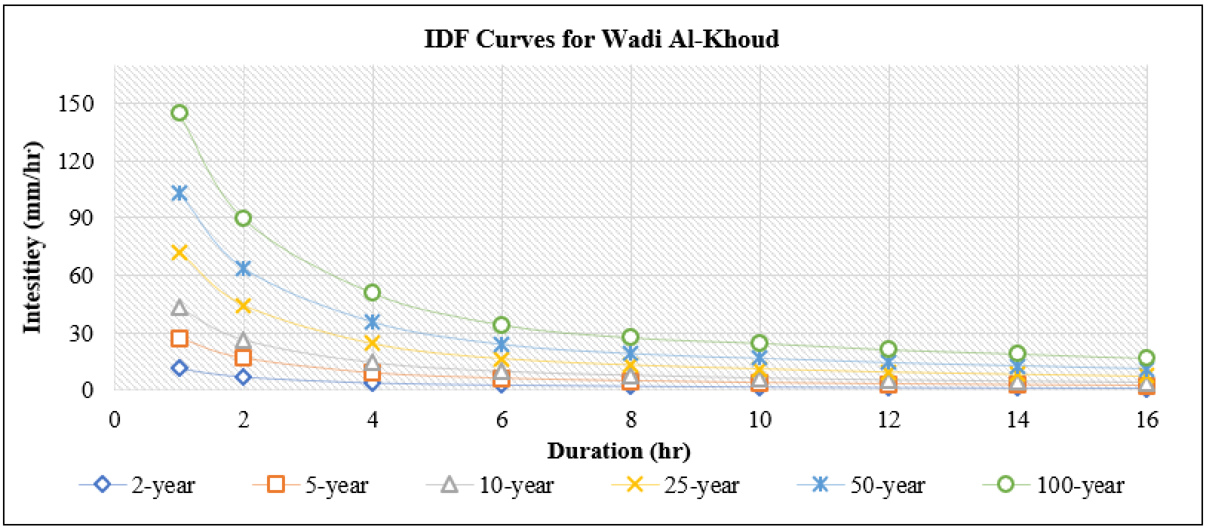

Fig 3. IDF Curves for Wadi Al-Khoud station.

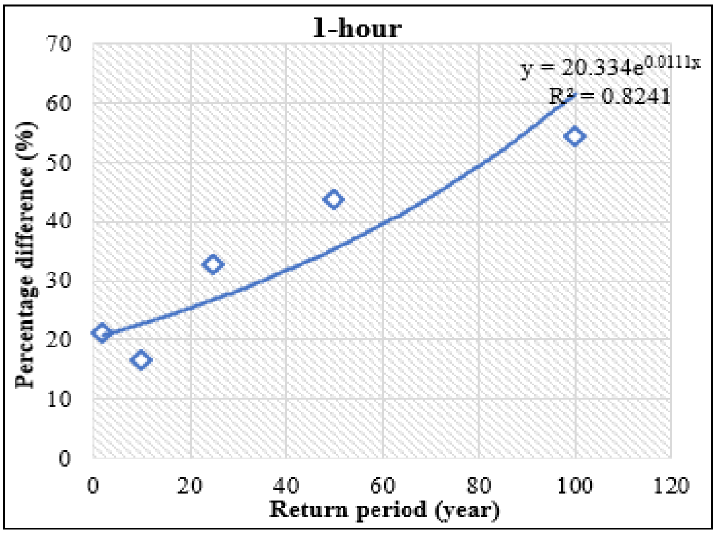

Fig 4a. The differences between the IDF curves for Wadi Al-Khoud station and Highway Design Manual for 1-hour duration.

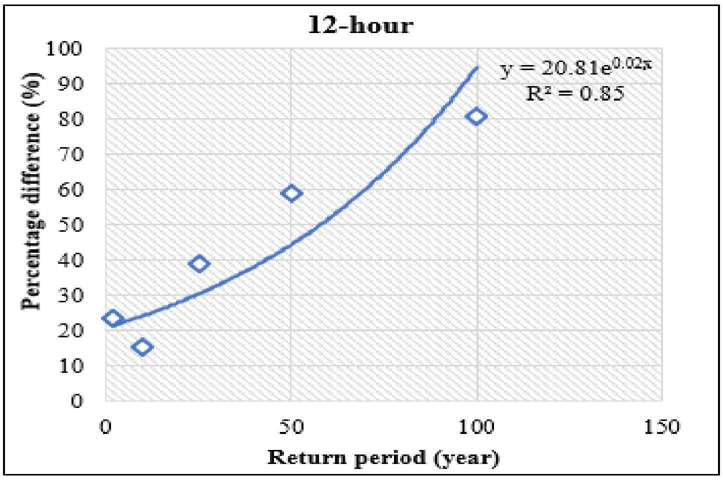

Fig 4b. The differences between the IDF curves for Wadi Al-Khoud station and High way Design Manual for 12-hour duration.

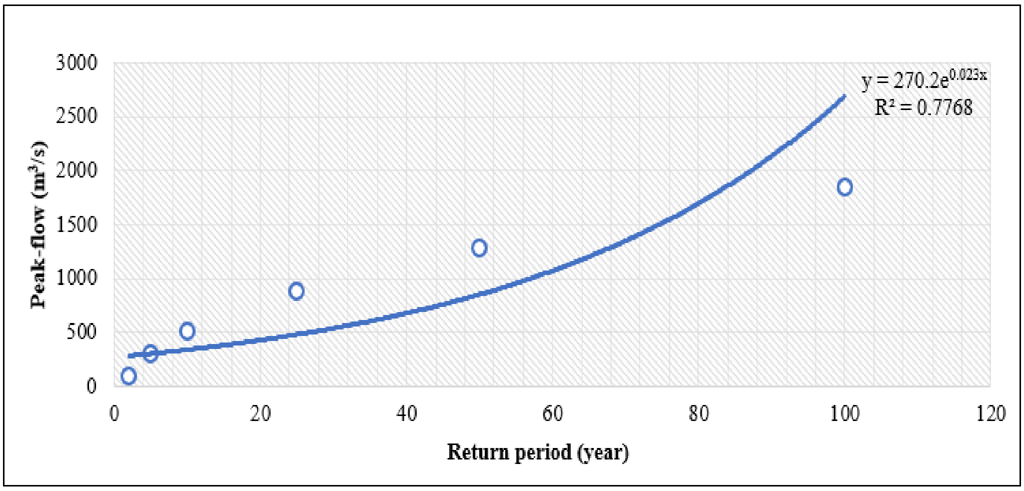

Fig 5. The relationship between the peak-flow rate and the return period estimated using observed AM wadi-flow data.

C. Rainfall-Runoff Model

Two rainfall events and the resulting wadi-flow hydrograph were used to calibrate the I_a, CN and T_{lag} parameters of the HEC-HMS model. The results of the hydrological model showed a reasonable match between the observed and simulated wadi-flow series. The level of agreement between the simulations and observations was assessed in terms of the shape of the hydrograph, timing of peaks (time series) and total water volume. According to Figures 6, there are some differences in peak-flow rates and water volume, which can be attributed to the hydrogeological and meteorological complexities in this significantly large catchment area.

The optimized values of CN and T_{lag} showed a close match between two events (Table 4). However, optimized I_a values were about 18% different from items its average of the events. In addition to the previously stated reasons, this difference between the calibrated values could be due to the variation of the antecedent moisture conditions of the two events. Relatively high CN value constrained for the study area can be attributed to the steep topography and the low-permeability rocks dominate in the upper catchment area.

The set of calibrated parameters were verified using the observed wadi-flows during the Cyclone Gonu event in 2007. This event was recorded as the most extreme event in recent decades in Muscat area. Figure 7 shows that the observed and simulated hydrographs for Gonu Event. The *NSE* of the simulation is approximately 0.8, which depicts reasonable applicability of the constrained parameters in the study area.

Even though the difference between the observed and the simulated peak-flows is very small (2.5%), a significant difference in the total volume was estimated (47.2%). This difference can also be attributed to the inability of the rain gage network to capture the spatial distribution of the rainfall during an extreme event such as Gonu in the catchment area.

D. Direct Runoff Hydrograph

The time of concentration of the catchment area was calculated using Equation 8 and found to be 690 minutes, which is approximately equal to 12 hours. Accordingly, this 12-hour duration was selected for developing synthetic storm hyetographs using IDF curves. Among many possibilities of a ranging hourly rainfall during 12-hour period, 10 common scenarios were selected and they were used with the calibrated model parameters of the HEC-HMS model to simulate resulting hydrographs.

The 10-year return period was used to compare the simulated peak-flows and the peak-flow estimated by the observed wadi-flow data. Table 5 shows the extracted 10-year hourly rainfall intensities from the IDF plot. One of the interesting characteristics of the rainfall in arid areas such as Muscat is that a significantly large amount of rainfall of the event happens within a short period of time causing frequent flash food. According to the distribution of hourly rainfall in Table 5, approximately, 67 % of the total rainfall happens during the first hour and 83% of that happens during the first two hours. The remaining 17% of the rainfall distributes during other 10-hours, which is completely lost for infiltration. Accordingly, first two events alone produce the direct runoff. Peak-flows and the total direct runoff volume extracted from the simulated hydrographs are shown in Table 6. The average peak-flow of the 10 scenarios is 2978 m^3/sec with the highest estimated peak-flow of 3449 m^3/sec.

Event	***CN***		**I_a (mm)**		**T_{lag} (min)**	
	IV	OV	IV	OV	IV	OV
Event 1	60	90	7	10.3	185	177.6
Event 2	60	89.7	7	7.1	185	187.7

Table 4. Optimized parameter values of each event.

IV = Initial value, OV = Optimized value.

Table 5. Rainfall intensities for the hyetographs.

Time (hr)	**Incremental rainfall (mm/hr)**
1:00	43.08
2:00	9.72
3:00	4.20
4:00	0.08
5:00	0.42
6:00	0.98
7:00	1.02
8:00	1.98
9:00	0.62
10:00	0.78
11:00	0.37
12:00	0.33

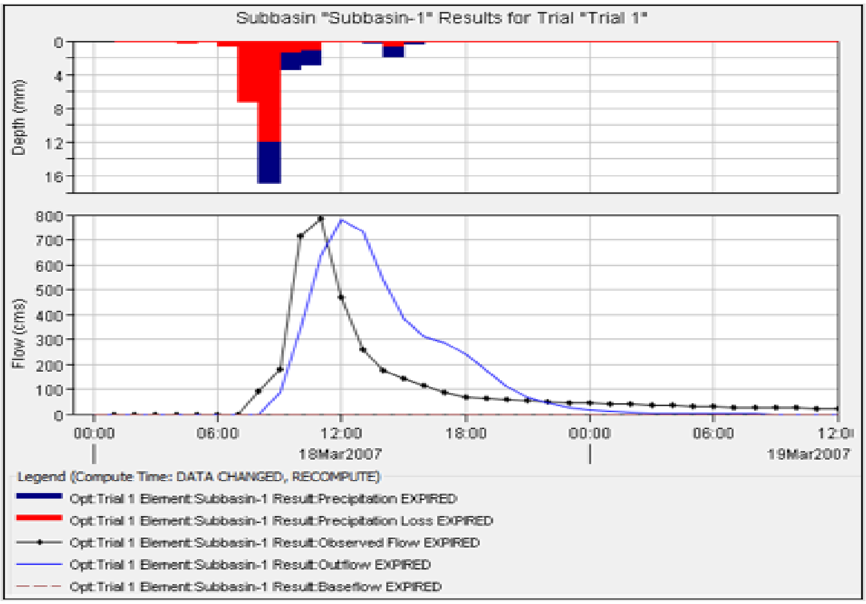

Fig 6a. Comparison between the simulated and observed direct runoff hydrograph and the gross rainfall hyetograph for Event 1.

Table 6. Peak-flow and the volume for each scenario.

Scenario	Peak-flow (m^3/s)	Volume (m^3)
1	2799.30	60414120
2	3370.40	60414480
3	3128.40	60414480
4	**3449.00**	**60415560**
5	2484.40	60414120
6	3227.30	60414840
7	2246.60	60414480
8	2307.20	60414480
9	3245.60	60415200
10	2520.00	60414120

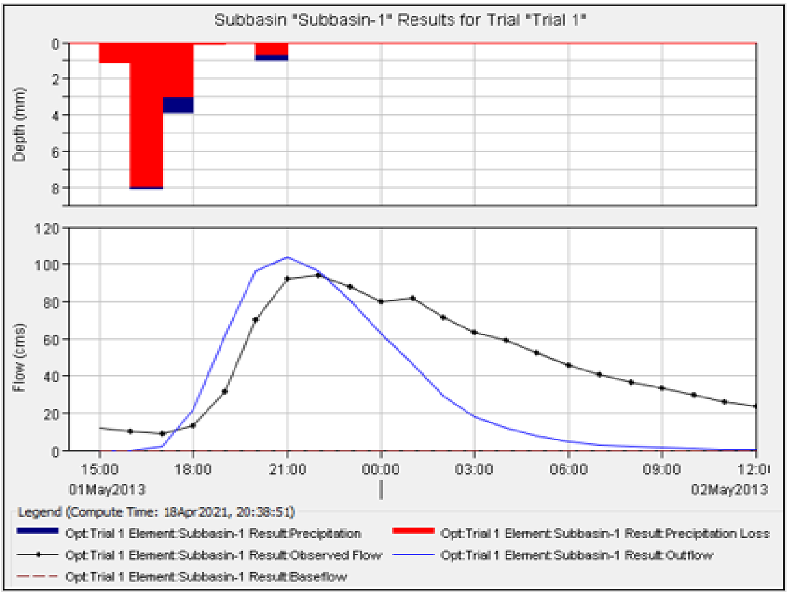

Fig 6b. Comparison between the simulated and observed direct runoff hydrograph and the gross rainfall hyetograph for Event 2.

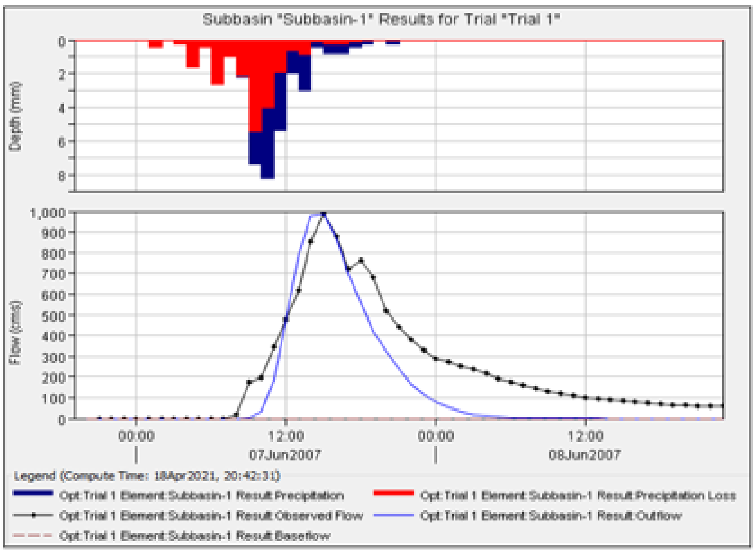

Fig 7. Comparison between the simulated and observed direct runoff hydrograph and the gross rainfall hyetograph for Gonu event.

E. Comparison between the observed and simulated peak-flow

According to Figure 5, the 10-year peak-flow estimated by the observed wadi-flow is about 503 m^3/sec, which is much different from the estimated peak-flow using the IDF curves (2978 m^3/sec). Despite the fact that these two estimations were made by entirely two different methods, the following reasons also may have contributed to the uncertainties of the estimations:

- inability to capture the spatial distribution of the rainfall over a large catchment area. Pilgrim *et al*., (1988) stated that greater spatial variations of rainfall occur in the arid areas than in humid regions, resulting in an error of the sparsely gaged data being unrepresentative of average rainfalls over a catchment area. Kwarteng *et al*. (2008) demonstrated that there is nearly 50% of rainfall variation in different part of Oman.
- absence of the long-term rainfall and wadi-flow data for the probability estimations.
- use of only one station for developing the IDF curves.
- inability of the developed hydrological model to simulate certain site conditions, such as the antecedent moisture condition. This study used the event based hydrological modeling method, in which the soil moisture content is maintained constant by the specified *CN* group. This shortcoming can be addressed by using a detailed hydrological modeling method. such as the soil moisture accounting method. However, absence of a detailed data set to calibrate the required parameters of this method limits its application.

IV. CONCLUSION

Hydraulic designs require the peak-flow rate corresponding to a particular return period. This study demonstrates the difference of estimated peak-flows by two methods in Wadi Al-Khoud catchment area. The first method used the observed wadi-flow fitted with the GEV distribution to estimate the peak-flow and frequency relationship. However, in the absence of long-term wadi-flow records required in probability analysis, commonly abundant rainfall records are used with the hydrological modeling tool to estimate the peak-flow rate resulting from a rainfall event with a certain return period. In this study, this second method was applied by developing the IDF curves and a HEC-HMS hydrological model for the Wadi Al-Khoud catchment area. A marked difference was estimated between the IDF curves in this study area and the ones presented in the Highway Design Manual (2010) for Muscat area. Furthermore, results showed that this difference increases as the return period and the duration increases. It is therefore recommended to use site specific IDF curves in hydrological modeling whenever possible. Similarly, peak-flow rates estimated for 10-year return period show a significant difference between the two methods. The peak-flow rate estimated by the observed wadi-flow is about 503 m^3/sec, which by the hydrological model with the average of wadi-flow rates by 10 synthetic hyetographs is about 2878 m^3/sec. Possible reasons for this variation were discussed. Implication of these two methods in hydraulic designs requires further studies. Accordingly, it will be able to quantify the uncertainties of estimations by these two methods and prepare a set of guidelines to be used in ungauged basins with limited data.

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