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# The Role of Integrated Water Management in Watershed Flood Management

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**Abstract:** The aim of this study, flood management, in a system of a reservoir and ungaged sub-basins was evaluated. By means of frequency analysis and index hydrograph method, flood hydrographs with different return periods for these under study sub-basins were calculated. After determination of inflow hydrograph to dam reservoir, flood routing in reservoir and consequently in river network by three scenarios were conducted. The results have shown that dam reservoir located in one of the sub-basins, functioned well as a flood controller, but because of no watershed analysis as a hydrologic unit and comprehensive consideration in watershed studies, the flood control resultant in the out point of system is in the order of zero.

**Key words:** Integrated flood management, simulation, flood routing, Allah watershed

## INTRODUCTION

Human interference in the rivers natural regimes, scarcity of the water resources and also the ever-rising needs for various water consumption have forced the policy-makers to demand managers of water resources to initiate the coping strategies with development, management and operation of the water resources issues. The water resources development projects normally require a sound and consistent planning in order to provide the set objectives. Considering all the technical aspects and the ultimate implications of the projects is highly complicated that must be analyzed with the awareness of the water system issues and characteristics as well as application of a systematic outline and appropriate tools. The consequences of overlooking the science-based methods, regulations and standards of the water resources management may lead to numerous defects and deficiencies at the time of operation. Reservoir dams are amongst the most important infrastructures in water supplying systems that play a significant role in the national economy through water storage and regulation. The construction of such structures requires plentiful funds which can be supplied only through adopting scientifically approved methods in design and operation of the dams resulting to long-term economic profits.

The integrated or interconnected water resources management is a method with the objective of simplifying the understanding of a watershed issues and their analysis. The distinct water resources management, conversely, is capable of incurring social, economic and environmental losses. The former has been accepted as the dominant water resources management regime of the 21st century (Jonker, 2007). It pursues the coordinated development and management of water resources, land and the related resources to promote and maximize the social and economic benefits without posing any potential threat to the stability of the natural ecosystems. This method includes an integrative view of the watershed in terms of land and water management relations, different functions and water use in the area. Manning and Seely (2005) reported the results for an integrated water resources management conducted in Kuiseb waters head in Namibia and emphasized the integration of the natural resources management especially in sensitive regions such as dry rivers. To ensure the fulfillment of the socioeconomic needs of its third development plan, the national state of South Africa cautiously decided to use the water resources management plans and began to introduce it into the academic syllabi across the country.

The ultimate objective of the integrated water resources management is to create a system that places

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the water resources management practices in an environmental context and relates to the social and economic developments as well as application of the responses given by Kar-Amouz *et al.* (2006a). Some of the indirect consequences of the lack of integrated management emerge with a sometime in future as their consideration in opting for the best alternatives and refusal of those failing the water system is important.

Flood management is an extensive process in flood control that compensates for the losses caused by flood events for the purpose of which structural and non-structural methods are applied. Due to the differences in watersheds in terms of climate, development and topography, it is not possible to suggest any single standard pattern for the use of flood management practices (Kar-Amouz *et al*., 2006b). In large Watershed where different designs with various alternatives and sizes can be executed, the efforts should be directed towards the elimination of the inappropriate options and selection of the best methods.

Assessment of different flood control and management methods indicates a decrease in the maximum flow discharge due to the existence of the reservoir and storage dams. The optimization methods yet have become critical in management and operation of the reservoirs during the last decade. Yeh (1985) and Labadi (2004) presented in their study a comprehensive review and assessment of the optimization methods and the relative models. According to Labadi (2004) despite the fact that the optimization models have been developed well recently, still much of the reservoir planning and operation studies are carried out using simulation models. The improvement of the flood frequency estimation models at unmeasured points and where reliable information is missing is one of the objectives of today hydrology. River flood regime is a product of responses of basin hydrology to the flood-generating process. Unfortunately, any applicable integrative method is restricted to the flood regime at the point of measurements (Cunderlik Juraj and Burn Donald, 2002). The flood performance of Allah River is unknown and complicated which is mainly due to the integration of controlled hydrographs of Rood-Zard tributary and uncontrolled ones of A'la and Talkh tributaries. The present study deals with determining the performance of the Jarreh reservoir dam on one of the controlled branches of Rood-Zard with the obtained flood data and its integration with the flood data of the corresponding tributaries where hydrometric stations are non-existent. Study and design of different projects In Iran are conducted distinctly and unconnected. The lack of integrated management of distinctive projects and unsuitable size of supplies dedicated to meet the objectives of each project has caused major drawbacks. Considering the criticality of the integrated water resources management this work tries to focus on the flood management in a reservoir system and several uncontrolled sub-basins.

## MATERIALS AND METHODS

This study was conducted during the years 2005-2008 at the Department of Hydrology and Water Resources, Faculty of Water Engineering Sciences, Shahid Chamran University, Ahwaz, Iran.

**Study area:** Allah river is originates in a mountainous area 70 km east and Northeast of the city of Ramhormourz and after almost 130 km joins Maroon river in Cham Hashem area where Jarreh river is initiated. The total area of the Allah river basin is approximately 2900 km$^2$, its circumference is 325 km, the length and width of it are 141 and 21 km, respectively, while the Gravilous coefficient of the water head was found to be 1.67 (Hamadi and Nouzarian, 2003). This watershed comprises three main sub-basins including A'la, Rood-Zard and Talkh sub-basin. Figure 1 shows the map of Allah river watershed. A'la and Talkh tributaries have not hydrometric stations and are un controlled discharge currently and no data are available regarding their hydrological performance, combinations and flood hydrograph transfer to the downstream. The run-off of the Rood-Zard sub-basin is regulated by the Jarreh reservoir dam while there is an active measuring hydrometric station, Mashin station at the main stream near the outlet of the dam. The dam is located 20 km northeast Ramhormourz at 44° 43' E and 31° 26' N. The Rood-Zard river joins A'la river at Rood-Zard village downstream Jarreh reservoir dam which ultimately constitutes the Allah river system. The Talkh tributary flows into the Allah River 9 km downstream which is of an area of 657 km$^2$.

**Research methodology:** This research uses the annual maximum flash flood data series and observed flood hydrographs during 1970 to 2006 of the Rood-Zard River at Mashin station. By completing and validating the data, the required statistical tests including randomness, independence and irrelevant data tests were carried out. The frequency analysis and also the degree of different distributions agreement with the maximum flash flood records was performed with the Hyfa hydrological frequency analysis software package. The best statistical distribution in agreement with the data was determined based upon the minimum median deviation of the observed maximum flash flood data and its calculated

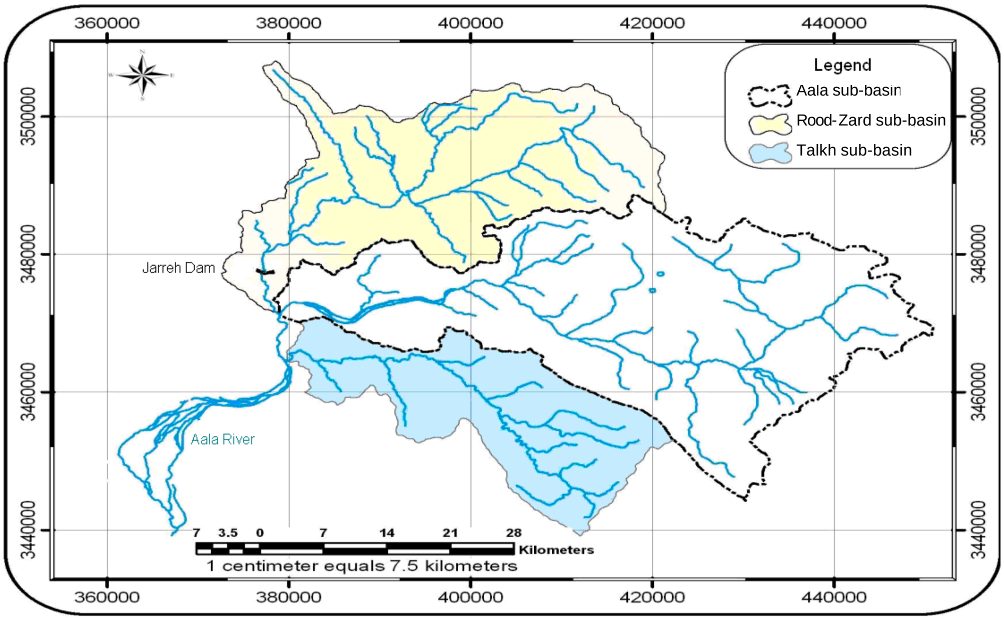



Fig. 1: Allah watershed and its sub-basins

values in distributions, median relative square deviations, Chi-square analysis and comparing the distribution fitting with the observed data (Chow, 1988). Then, based upon the best distribution analysis maximum flood with different return periods was determined. In order to determine the flood hydrograph the Flood Index Method was used. This method requires an adequate number of observed annual maximum flood hydrographs to use. To develop the index hydrograph, all the recorded annual maximum flood events were studied and 9 flood events were selected. The events were picked up based upon the determinants of the hydrographs including maximum discharge, peak time, base flow discharge and base time. Then, with determination of the dimensionless index and maximum discharge with different return periods resulting from the best distribution function, flood hydrograph determinants were specified with different return periods. These periods included 11 different probabilities with different return periods of 2 to 10000 years. After extraction of the flood hydrographs of the Rood-Zard river, the corresponding floods were calculated for uncontrolled tributaries of A'la and Talkh. The calculations were performed through modifying the flood hydrographs of Rood-Zard river relative to the impure rainfall and the sub-basins area, thus, the flood hydrographs for A'la and Talkh rivers were developed with different return periods. By determination of inflow hydrographs of the Jarreh reservoir dam the flood routing was carried out for the reservoir. The spillway of this dam is as high as 497 m MSL functioning at the height of 502 to discharge the flood. In order to perform the flood routing in the reservoir at the required elevation of the spillway, HydroRout software was used (William, 1999). The inflow hydrographs were simulated for 11 probabilities at 5 different spillway elevations and the outflow hydrographs were calculated. The hydrodynamic Mike 11 model (HD) was used for the purpose of flood routing. This model has been developed by Danish Hydraulic Institute (DHI) for simulation of non-steady flows in the rivers, irrigation canals, sediment transport, water quality and other water resources-related disciplines. The hydrodynamic model which is a main part of the Mike 11 modeling system was used as the basis for other models such as flood simulations, dispersion and distributions, water quality and non-cohesive sediment transfer models. The Saint Venant and continuity equations were resolved non-linearly across the grid in specific time steps. After hydrodynamic simulation, the flow hydraulic unknowns including depth, velocity and discharge of the flow were determined. In order to use the model efficiently, it is necessary that initial input be provided to the model and then the boundary conditions of the model be calibrated. The cross-sections of the river are provided to the model as distance-depth coordinates and for each cross-section, the geographical coordinates and its position against the reference are putted into the model. The river reach simulated is 54 km long from Jarreh reservoir dam upon Rood-Zard River to its conjunction point to the Maroon river. The available river cross-sections to be used in this study were 54 cross-sections that were provided by Khuzestan Water and Power Authority (KWPA).

Mike 11 model is not capable of generating reliable flow simulations without appropriate definition of boundary conditions. The flow boundaries included downstream and upstream ends of the river so that the simulated reach would not be connected to another tributary and the flow begin from or end with that specific cross-section (Samadi Boroujeni and Samani, 2002). The boundary conditions of the river depend upon the prevalent physical conditions and by considering the river conditions and the objectives of the hydraulic simulation, the flow boundaries should be defined precisely. In simulating the flow system, the upstream flow conditions were determined in form of hydrographs with different return periods of 2 to 10000 years while the flow conditions of downstream were presented in form of discharge-elevation curve. The inflowing tributaries were introduced to the simulating model as sources. The calibration of the HD model used discharge control method or water level control method. The main parameter in calibration of the model is the bed roughness coefficient (Manning n). Therefore, by using a series of measured discharge records the model was run and the n was selected so that the depth of calculated flow be equal to the depth of observed flow. Since the calibrated n in low flows occurred in the main canal, the roughness coefficient of the plain along the channel was estimated through field observations and the roughness determinations guidelines (Chow, 1959) and finally, by using Luther formula (Hosseini and Abrishami, 2006) the corresponding roughness coefficient for 5 reaches along the main water channel was determined. Also, to ensure the accuracy of the results in addition to the calibration of low flows, the sensitivity analysis was carried out for the flood discharge in which the roughness along the river was taken as an independent variable while the flood hydrograph at the end of the simulation was considered as a dependent variable. After the flood routing of the reservoir and preparation of the flow model, three simulation scenarios were developed for several flow hydrographs. The first scenario suggested that the natural flood flow was routed with the Jarreh dam eliminated. In the second and third scenarios, the flood routing was simulated with the presence of the Jarreh reservoir for both cases of the filled and empty reservoirs.

## RESULTS AND DISCUSSION

The Allah watershed is consisted of three sub-basins of A'la, Rood-Zard and Talkh. By applying the annual maximum flash flood data and observed flood hydrographs based upon the above method, the flood hydrographs for different return periods were developed. The maximum discharge and hydrograph volume for each sub-basin are shown in Table 1. Considering the engineering characteristics of the sub-basins and the peak time of the hydrograph, the flood of the Talkh, Rood-Zard and A'la would be discharged from the watershed, respectively. As noted in the procedures above, to perform the flood routing with different return periods at different spillway elevations, HydroRout model was used. By introduction of the flow hydrograph to the reservoir, part of the flow dissipates and the rest of it exits the reservoir with respect o the spillway elevation. The proportion of maximum discharge of the inflow and outflow hydrographs and the peak time of the outflow hydrograph for different return periods were calculated. The results of the calculations for different spillway elevations are shown in Table 2 and 3. The inflow and outflow hydrographs resulting from running the reservoir routing model for various flood events and differing spillway elevations are shown in Fig. 2a-f.

The results showed that the in the scenario where the reservoir is empty, the entire of flood hydrograph will be dissipated with low return periods. With floods with

Table 1: Maximum discharge and volume of hydrographs for sub-basins under study

| Return period (years) | Rood-Zard sub-basin Maximum discharge (CMS) | Rood-Zard sub-basin Volume (MCM) | Al'a sub-basin Maximum discharge (CMS) | Al'a sub-basin Volume (MCM) | Talkh sub-basin Maximum discharge (CMS) | Talkh sub-basin Volume (MCM) |
|---|---|---|---|---|---|---|
| 2 | 457 | 17.1 | 573 | 26.8 | 304 | 8.5 |
| 5 | 842 | 31.5 | 1055 | 29.4 | 560 | 15.7 |
| 10 | 1158 | 43.3 | 1451 | 67.9 | 771 | 21.6 |
| 20 | 1507 | 56.4 | 1888 | 88.3 | 1003 | 28.2 |
| 25 | 1627 | 60.9 | 2039 | 95.4 | 1085 | 30.4 |
| 50 | 2027 | 76.0 | 2540 | 119.0 | 1349 | 38.0 |
| 100 | 2470 | 92.0 | 3095 | 145.0 | 1644 | 46.0 |
| 200 | 2960 | 111.0 | 3709 | 174.0 | 1970 | 55.0 |
| 500 | 3685 | 138.0 | 4617 | 216.0 | 2453 | 69.0 |
| 1000 | 4298 | 161.0 | 5385 | 252.0 | 2861 | 80.0 |
| 10000 | 6779 | 254.0 | 8494 | 397.0 | 4512 | 127.0 |

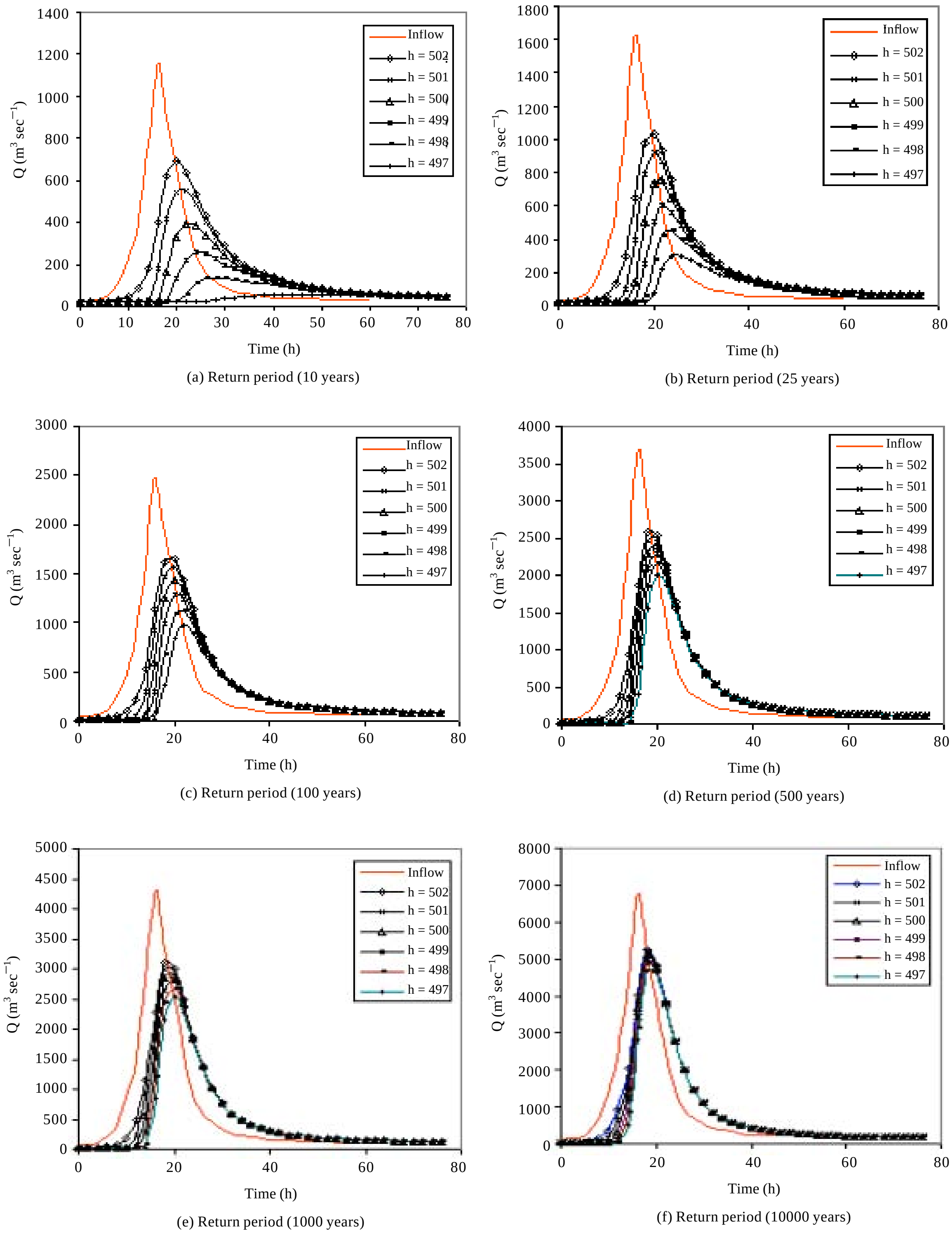



Fig. 2: Inflow and outflow hydrographs to the Jarreh reservoir at different elevations

Table 2: Reductions of maximum inflow at the exit point of the Jarreh dam reservoir (percent)

| Return period (years) | Height (m) | | | | | |
|---|---|---|---|---|---|---|
| | 502 | 501 | 500 | 499 | 498 | 497 |
| 2 | 53 | 77 | 96 | 96 | 96 | 96 |
| 5 | 45 | 60 | 78 | 90 | 96 | 96 |
| 10 | 40 | 53 | 66 | 78 | 88 | 95 |
| 20 | 37 | 45 | 56 | 66 | 76 | 86 |
| 25 | 37 | 43 | 54 | 63 | 73 | 82 |
| 50 | 35 | 39 | 47 | 54 | 62 | 71 |
| 100 | 34 | 37 | 42 | 48 | 55 | 60 |
| 200 | 33 | 35 | 39 | 43 | 48 | 54 |
| 500 | 30 | 33 | 35 | 38 | 41 | 46 |
| 1000 | 28 | 30 | 33 | 35 | 38 | 41 |
| 10000 | 22 | 23 | 24 | 26 | 28 | 31 |

Table 3: Reductions of peak time inflow at the exit point of the Jarreh dam reservoir (h)

| Return period (years) | Height (m) | | | | | |
|---|---|---|---|---|---|---|
| | 502 | 501 | 500 | 499 | 498 | 497 |
| 2 | 6 | 8 | 60 | 60 | 60 | 60 |
| 5 | 4 | 6 | 8 | 14 | 58 | 60 |
| 10 | 4 | 6 | 6 | 8 | 12 | 32 |
| 20 | 4 | 4 | 6 | 6 | 8 | 10 |
| 25 | 4 | 4 | 6 | 6 | 8 | 10 |
| 50 | 4 | 4 | 4 | 6 | 6 | 8 |
| 100 | 4 | 4 | 4 | 4 | 6 | 6 |
| 200 | 4 | 4 | 4 | 4 | 4 | 6 |
| 500 | 2 | 4 | 4 | 4 | 4 | 4 |
| 1000 | 2 | 2 | 4 | 4 | 4 | 4 |
| 10000 | 2 | 2 | 2 | 2 | 2 | 2 |

median return period of maximum outflow will decrease to 86 to 54% of the maximum inflow. In floods with return periods of 500 to 10000 years maximum outflow reduction will be 46 to 31% of the maximum inflow to the reservoir.

In extreme case, where the reservoir flood control is full and floods occur with a return period of 10000 years, the maximum inflow to the reservoir will decrease to 22% while the peaking time will be delayed for 2 h at the exit point. This emphasizes that the dam reservoir functions well locally in controlling floods.

The outflow hydrographs of the sub-basins enter the Allah river system and integrate into each other. Before the hydraulic routing of the river, it was decided to conduct a sensitivity analysis for the flow simulation model. The US Army Corps of Engineers have conducted several studies regarding the sensitivity of the model that confirms the effects of geometrical errors and bed roughness on the water surface profile calculations. They suggested that the geometrical errors are controllable while the bed roughness effects obliterate the validity of the results of the model. The required information of the bed roughness were not available or inflicted by severe measuring errors, thus, it was necessary to run the sensitivity test for the model application. The roughness coefficient for distances of 0, 9, 10, 24 and 54 km was estimated as 0.038, 0.039, 0.040, 0.043 and 0.045%, respectively. The effects of bed roughness on the model

Table 4: Errors in results caused by roughness variations to the initial values

| Roughness variations (%) | RMSE+ | RMSE- | MAE+ | MAE- |
|---|---|---|---|---|
| 16 | 68.2 | 68.2 | 1.4 | -1.4 |
| 12 | 51.5 | 51.6 | 1.1 | -1.1 |
| 8 | 34.1 | 34.3 | 0.7 | -0.7 |
| 4 | 17.2 | 17.3 | 0.3 | -0.4 |

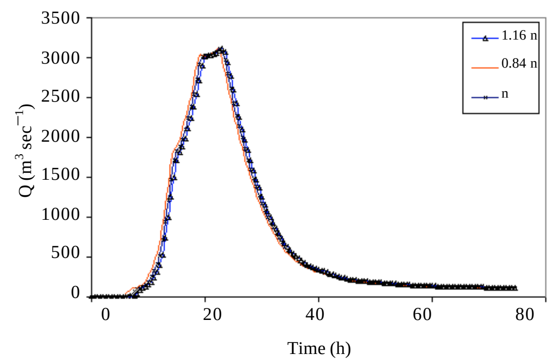



Fig. 3: Flood hydrographs resulting from different values of bed roughness

results were studied through modifying the input data to the model as percentage of initial value and re-running of the model. The roughness coefficient was reduced or increased for 4, 8, 12 and 16% of the initial value. The flow routing for flood hydrographs with return period of 25 years was carried out. The variations of the hydrographs resulting from changing bed roughness to the initial value were calculated. The variations were determined by the Relative Minimum Square Error (RMSE) and Median Absolute Error (MAE). The results of flow routing indicated that the maximum discharge hydrograph would not be affected by increase or decrease of bed roughness while it slightly affects the maximum discharge time. In other words, variations of the roughness in a range of 4-16% imply the restricted hydrographs variations to downstream of the flow. Table 4 shows the statistical analysis of difference resulting from roughness variations to the initial values used in the model.

Figure 3 shows the resulted flood hydrograph due to the roughness variation as of ±16% to the initial bed roughness. The studied hydrograph has a base discharge flow of 72 $m^3 sec^{-1}$ and peaking time of 22 h. With the maximum roughness variation as of 16%, the peaking time of the hydrograph will change only 30 min. By introducing the hydrograph of the studied sub-basins to the flow simulation model, the combined hydrograph was developed.

The flow simulation in this stage was carried out with the first scenario which uses the natural state of the river system without the Jarreh dam. The results of calculations are presented in Fig. 4a-f. The peaking time

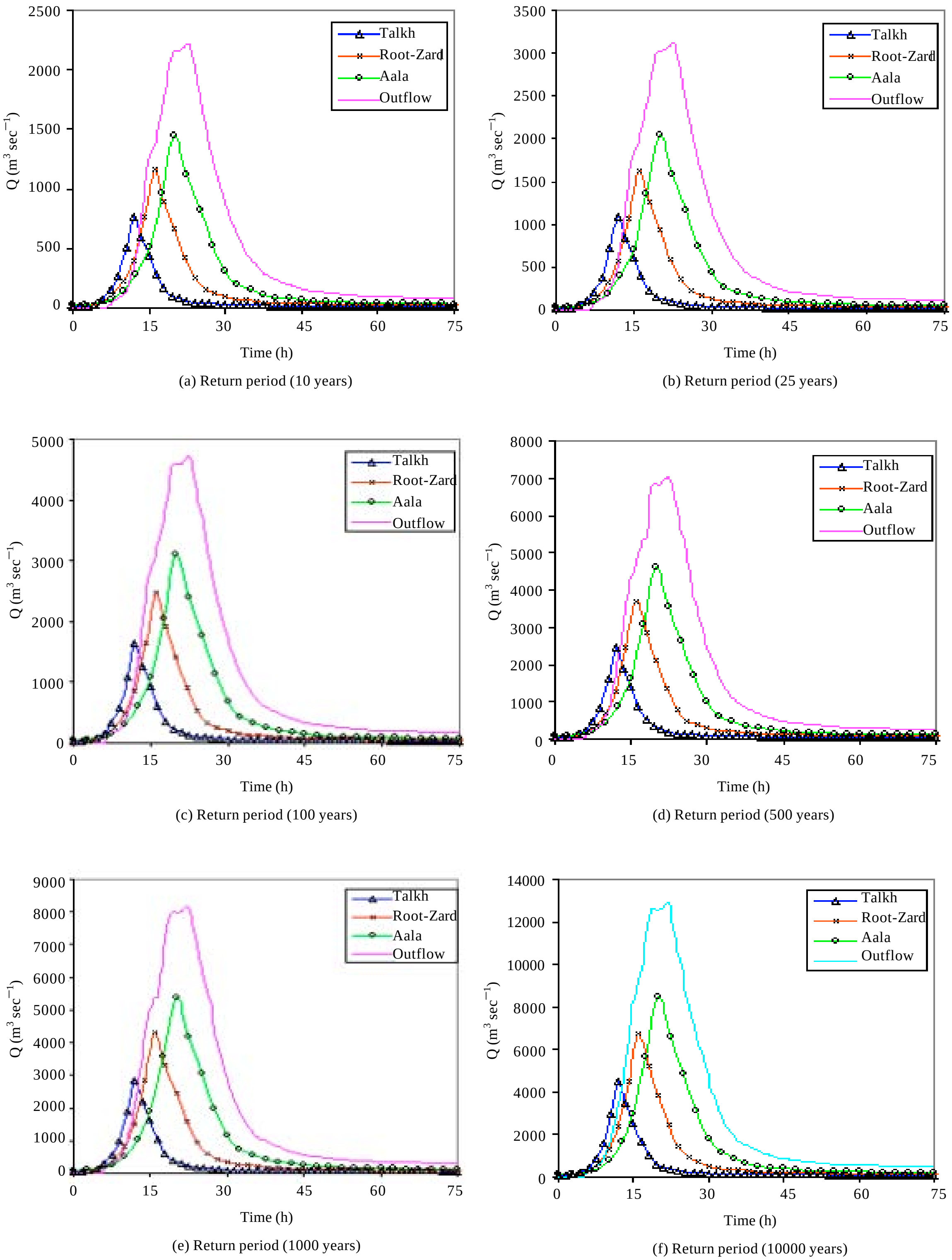
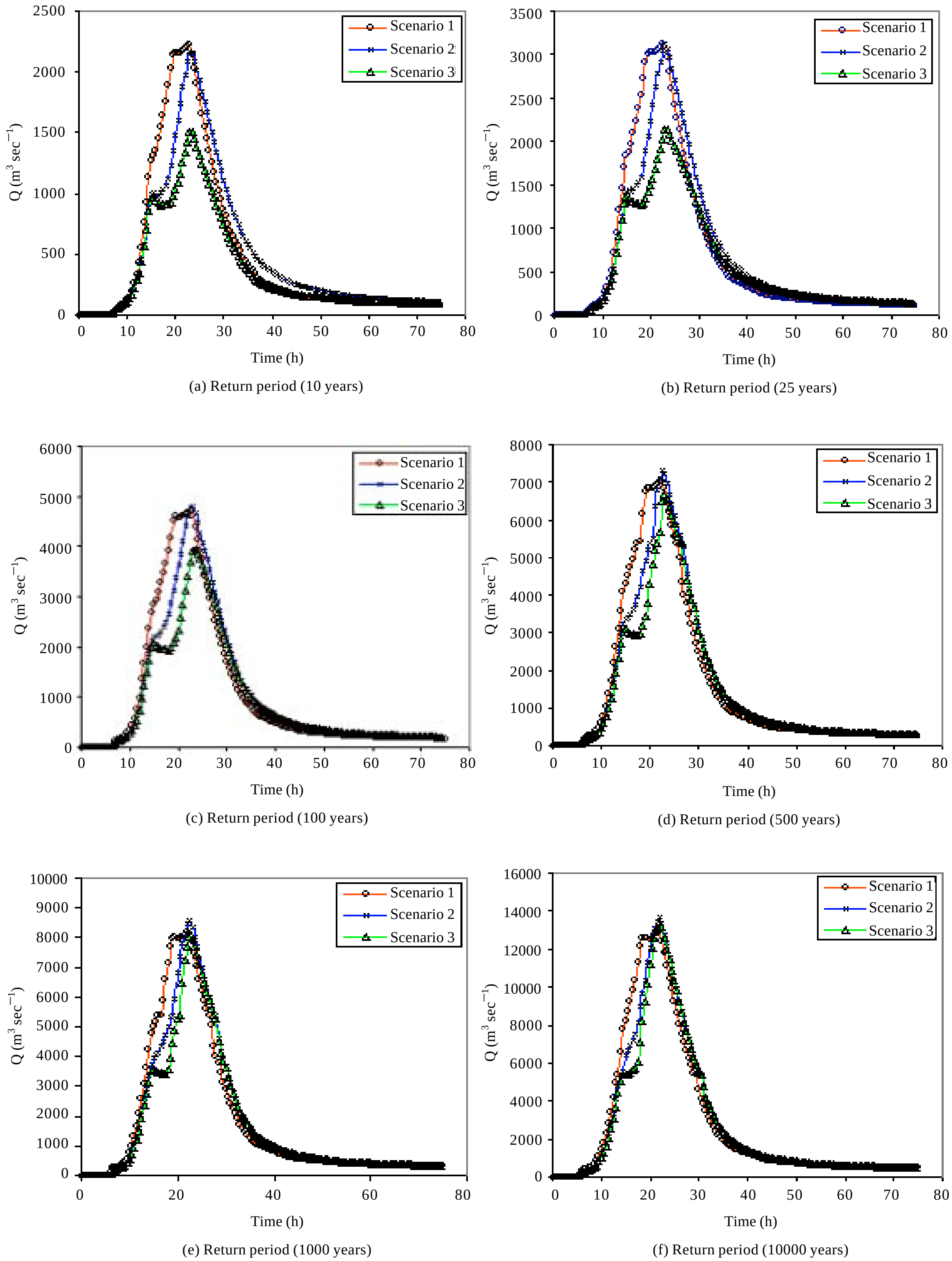



Fig. 4: Flood hydrographs of sub-basins and their combinations at the exit point of the river system




(a) Return period (10 years)

(b) Return period (25 years)

(c) Return period (100 years)

(d) Return period (500 years)

(e) Return period (1000 years)

(f) Return period (10000 years)

Fig. 5: Flood hydrographs of sub-basin outlets for different scenarios

for A'la, Rood-Zard and Talkh tributaries were as 20, 16 and 12 h, respectively. The Fig. 3 and 4 suggest that the maximum flow time of routing hydrographs at the exit point of the basin was about 23 h and more affected by the peaking time of A'la tributary. This can be attributed to the geometrical condition of the river itself. Indeed, the discharge of the flood flow of the tributaries and reduced effects of their maximum flow occur at the exit points of Talkh, Rood-Zard and A'la rivers, respectively. In the second and third scenarios, the outflow hydrographs of the Jarreh reservoir in both cases of full and empty reservoir are considered. The outflow hydrographs were combined with the natural flow from Talkh and A'la tributaries and then routed in the river system. The results are shown in Fig. 5a-f.

As seen from the figures, the both full and empty reservoirs only cause a delay of 30 min to the peaking time of the outflow hydrograph. As the flood occurrence probability decreases, the role of the reservoir in flood control becomes less significant in such a way that a discharge flow with return period of 100 years and beyond, the maximum flood discharges in scenarios will increase in comparison. With natural conditions. With reservoirs.

## CONCLUSIONS

The study area in the present research included the Rood-Zard basin with a controlled flow and two sub-basins of A'la and Talkh tributaries. The Jarreh reservoir dam in the Rood-Zard basin is responsible for local control of the flow in the area. Despite the flood control practiced in this basin, the result of such control at the exit point of the system is almost zero which is mainly due to the lack of system analysis as a hydrological unit and non-integrative watershed management on the research level. In other words, if Jarreh reservoir was constructed and operated on either A'la tributaries the system would demonstrate a more efficient functioning. The effects also can be studied through further researches. Also, any decrease in maximum discharge will lead in a delayed peaking time. However, the delay makes the outflow hydrograph of the reservoir be combined with the maximum hydrograph of the adjacent sub-basin i.e., A'la sub-basin and create a more acute of the first scenario.

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