JJEES

JJEES (2025) 16 (1): 28-34
ISSN 1995-6681

Jordan Journal of Earth and Environmental Sciences

Flood Hazard Mapping from Dam Break: A Case Study of King Talal Dam

Radwan Al-Weshah[1*,2], Murad Al-Salahat[3], Saif Al-Omari[3]

[1]Civil Engineering Department, The University of Jordan, Amman 11942, Jordan.
[2]Department of Civil Engineering, Faculty of Engineering and Technology, Al-Zaytoonah University of Jordan, Amman 11733, Jordan.
[3]World Vision International, Amman 11855, Jordan.

Received on September 30, 2024; Accepted on November 10, 2024

Abstract

This paper assesses the flood hazard related to dam break analysis for King Talal Dam in Jordan. Probable Maximum Flood (PMF) was estimated by HEC-HMS using the Probable Maximum Precipitation (PMP) values. The PMF of Zarqa River Watershed (ZRW) was determined to be 3823 . Dam breach parameters were estimated using different models. A 2-dimensional unsteady flow simulation was used to route the breach flood in the downstream area of the dam. The maximum breach flows for overtopping and piping scenarios were estimated to be 65456 and 43748 , respectively. The study generated inundation, flood hazard, duration, and velocity maps for the dam's downstream areas. Most of the inundated areas were classified as extremely hazardous. The maximum water depth was approximately 107 meters on some agricultural lands. The paper provides valuable insights into the dynamics of dam breach floods and their effect on downstream regions. These findings help to develop plans and policies to minimize the risk of loss of life and property due to dam failure. The emergency preparedness strategies include the development a telemetry monitoring system for the dam body and spillway, an early warning system for the upstream watersheds, physical evacuation, and emergency response plans for residents downstream of the dam. The flooded areas include schools, hospitals, and other important public and private properties. This study demonstrates the importance of efficient emergency planning to minimize potential damage. The results of this analysis can help local authorities to prioritize their plans based on the observed arrival times and flood durations. It is very important to engage local communities, government agencies, and other stakeholders in the development of integrated flood preparedness strategies and management plans.

Keywords: *Dam Break Analysis, King Talal Dam, Flood Hazard Mapping, HEC-RAS, Flood Hydrology.*

1. Introduction

Failure of dams can result from internal and external factors. Studies show that dam failure events appear as a result of different reasons such as piping, overtopping, and settlement (Dincergok, 2007). Failure incidents until 1985 were 34 percent a result of overtopping, 30 percent because of foundation defects, 28 percent due to piping, and 8 percent for other reasons. Overtopping failure occurs as a result of poor spillway design, which causes a reservoir to fill high with water, especially during heavy rains (Goodarzi, 2013). In piping failure mode, a breach starts to emerge when water percolates through the dam's body. Then, a critical amount of flow and materials will come out through the piping hole until the breach becomes fully formed. This type of failure occurs in fair weather (sunny days). The analysis of a dam break involves identifying and detailing the consequences of flooding that occurred due to the dam's failure (Whitham, 1955). This analysis consists of three main tasks: estimation of the breach flow hydrograph, routing of the hydrograph through the downstream region, and creation of inundation maps (Aydemir and Güven, 2017).

The parameters of a dam breach, that have an impact on the flood downstream and the accompanying risks, are called dam breach parameters. Some crucial variables such as breach width, depth, time to breach, and side slope are used to simulate a dam breach and estimate its post-effects (Wahl, 1997). Researchers have predicted dam breach parameters through using many dam failures. The most common studies are MacDonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990), Froehlich (1995), Froehlich (2008), and Xu and Zhang (2009). MacDonald and Langridge-Monopolis analyzed 42 events. Their study sample comprised an earthfill with a clay core, and rockfill dams. They established a relationship for the Breach Formation Factor. The factor was estimated by the volume of water that leaves the dam and the depth of water above the dam. Thereafter, the factor was utilized to correlate the amount of the material that was eroded from the dam (MacDonald and Langridge-Monopolis, 1984). In 1995, Froehlich performed a study that used 63 actual failures of embankment dams to determine the dam breach parameters. Based on his analysis, Froehlich concluded that most breaches have a trapezoidal shape with a horizontal bottom. In 2008, Froehlich updated his equations by adding 11 instances of dam failure events to his study. He then used the results of a statistical analysis in a Monte Carlo simulation to estimate the unpredictability of peak flows and water levels that could result from failed embankment dams

* Corresponding author e-mail: r.weshah@ju.edu.jo

(Froehlich, 2008). Von Thun and Gillette (1990) developed a guideline for estimating the parameters using data from Froehlich and MacDonald & Langridge Monopoli's studies. The guideline has a specific focus on estimating the breach side slopes, breach width at mid-height, and time until failure. The researchers found that their approach provided a more accurate fit for historical case study data than previous methods based on the breach formation factor proposed by MacDonald & Langridge Monopolis (Wahl, 1997). In 2009, Xu and Zhang provided empirical equations that possess a concrete basis to estimate the parameters. To elicit these equations, they utilized 182 instances of earthfill and rockfill dam failures. Approximately, half of the cases concerned sizable dams exceeding a height of 15 meters. They established empirical formulas between the characteristics of reservoirs and the breaching parameters using a multiparametric, non-linear regression technique (Xu and Zhang, 2009).

One of the most useful and popular hydraulic models was created by the U.S. Army Corps of Engineers (USACE). It is the Hydrologic Engineering Center's River Analysis System (HEC-RAS). It is employed for several tasks, including modeling water quality and water temperature, sediment transport, and 1-D and 2-D unsteady flow simulations (Brunner et al. 2016; Al-Azzam and Al-Kuisi, 2021; Al-Amoush et al., 2017; Al-Shibli et al., 2017). The most recent version of HEC-RAS offered by the US Army Corps of Engineers is 6.3.1. In order to provide precise hydraulic property tables based on the underlying topography utilized in the modeling process, HEC-RAS now features a 2D flow area pre-processor. Brunner (2018) employed the benchmark examinations devised by the Joint Defra Environment Agency of the UK to appraise the 2D proficiency of HEC-RAS. The outcomes demonstrated that HEC-RAS outclassed other models substantially, such as TUFLOW, MIKE FLOOD, and SOBEK.

The objective of this study is to perform a dam break analysis for the King Talal dam using a 2-D simulation approach to predict the characteristics of the flood wave, including peak flood flow, timing of the peak flood, and routing of the flood for different downstream regions. Additionally, the study generated inundation and hazard maps for overtopping and piping scenarios.

2. Materials and Methods

To create a flood map, this study employed an integration of rainfall-runoff and hydrodynamic models (Al-Salahat et al., 2024). High-resolution geometric data and meteorological and hydrological data were utilized in a rainfall-runoff simulation that estimated the flood magnitude. This integration was followed by simulation of overtopping and piping scenarios. The data are detailed in subsequent sections. The methodology employed in this study is graphically presented in Figure 1.

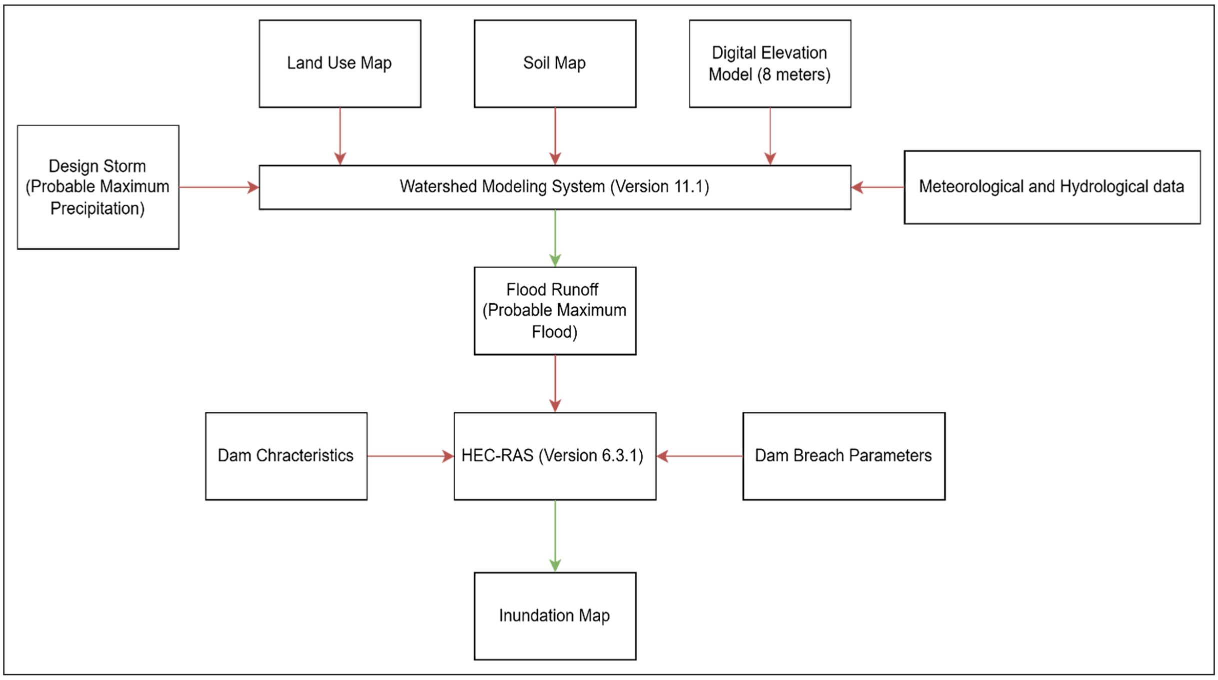

Figure 1. Flowchart of the modeling workflow

2.1 Study Area

Zarqa River Watershed (ZRW) is one of the major watersheds in Jordan that has coordinates of 32°12′ N and 35°35′ E. King Talal Dam (KTD) is located at the outlet of the ZRW with coordinates of 32°12′ N, 35°50′ E as shown in Figure 2. The dam is used for irrigation purposes in the Jordan Valley. The ZRW encompasses several cities and towns such as Amman, Zarqa, and Russeifa. The watershed covers about 4,300 square kilometers. It is a significant agricultural and industrial region and home to a substantial portion of Jordan's population. The KTD was built in 1978 as an earthfill dam with a spillway capacity of 4500 . The total storage capacity of the dam is 88.5 million cubic meters (MCM). The dead storage is approximately 8 MCM. Meanwhile, the live storage amounts to about 80.5 MCM. The reservoir's lake measures about 7.6 km in length and spans a maximum width of 450 meters (El-Radaideh et al., 2017).

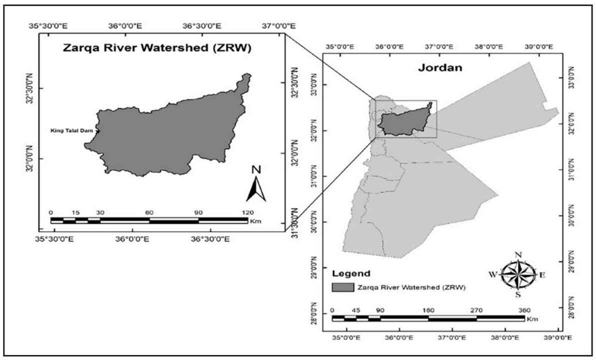

Figure 2. Location of the ZRW and KTD (El-Radaideh et al., 2017)

2.2 Geometric Data

A digital elevation model (DEM) is a depiction of a terrain's topography in a digital format. It provides a graphical illustration of elevation data that can be used to analyze and model various aspects of the terrain. Hydrological modeling, land-use planning, and infrastructure design are some of the most common applications of DEM. In this study, the DEM was generated using the Watershed Modeling System (WMS) with a resolution of 8 meters. Subsequently, it was converted to Triangulated Irregular Network (TIN) of maps using the HEC-RAS model for downstream area of the ZRW. The DEM of the ZRW is illustrated in Figure 3.

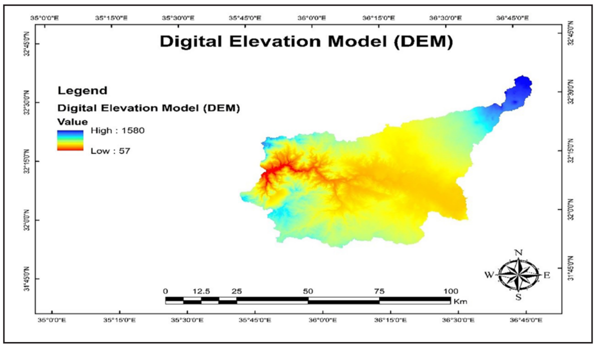

Figure 3. Digital Elevation Model of ZRW (Global Mapper, 2024)

In addition, the soil map was obtained from the Food and Agriculture Organization of the United Nations (FAO, 2022) website. Figure 4 illustrates that there are six types of soil presented in the ZRW. The ZRW is comprised solely of class D soil. According to USDA NRCS (1986), it tends to provide high surface runoff and has limited infiltration capacity when fully saturated.

For land use/cover maps, the Environmental Systems Research Institute (ESRI) website was utilized to maintain the map of the ZRW area. It offers a detailed and high-resolution depiction of land use with a resolution of 10 meters. The map classifies land use into seven major categories, including water, trees, grass, crops, scrubs, urban lands, and bare lands. Figure 5 presents the prominent land use areas of the ZRW.

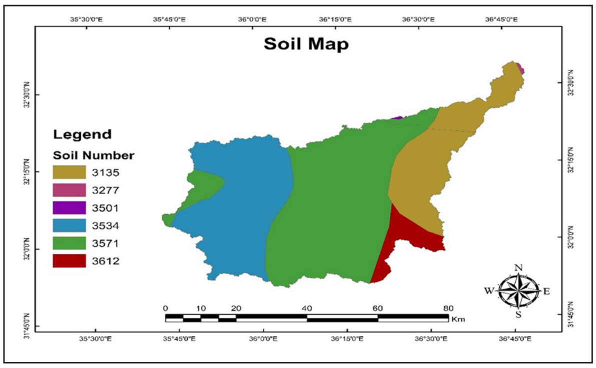

Figure 4. Soil map of ZRW (FAO, 2022)

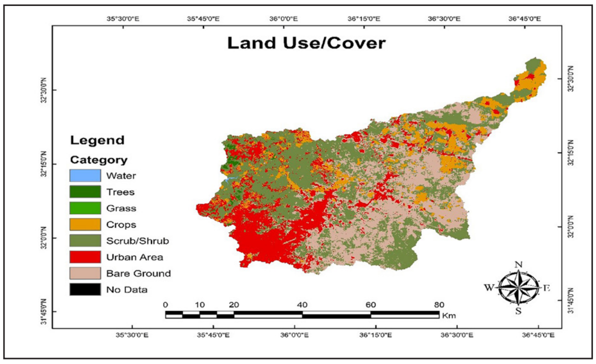

Figure 5. Land use/cover of ZRW (ESRI 2022)

3. Meteorological and Hydrological Data

The metrological and hydrological data were acquired from various governmental institutions. Specifically, the rainfall data for the ZRW were sourced from the Ministry of Water and Irrigation (MWI). The data were a daily rainfall measurement from 1987 to 2018, collected from 42 rainfall stations. However, due to incomplete records, only 36 stations were used in this study. In addition, data on the surface water elevation, storage, and discharge from the KTD were procured from the Jordan Valley Authority (JVA). The characteristics of the KTD such as dam type, crest width, height, and storage capacity were also collected from the JVA. Table 1 shows the characteristics of the KTD.

Table 1. General characteristics of KTD (MWI, 2024)

Type of the dam	Non-homogenous earthfill
Length (m)	350
Crest width (m)	11.5
Height (m)	108
Catchment area (km^2)	4300
Storage at full supply level (MCM)	88.5

4. Hydrologic Modeling

The use of Probable Maximum Flood (PMF) is a common practice to guarantee the safety of hydraulic structure and as a reference for flood disaster management (Alias et al., 2013a). To determine the PMF values, input data of Probable Maximum Precipitation (PMP) are required (Desa et al., 2001). The World Meteorological Organization (WMO) defines the PMP as the upper limit of precipitation that is expected to occur within a specific time and location (WMO, 2012). In this study, the values of the PMP are estimated using the Hershfield statistical method. It is a well-establish technique that has been widely applied (Hershfield, 1961). The PMP can be estimated using the following equation:

$$X_{PMP} = X_n + S_n \times K_m \quad (1)$$

$$K_m = \frac{X_{max} - X_{n-1}}{S_{n-1}} \quad (2)$$

where X_n denotes the annual maximum rainfall average, and S_n represents the standard deviation of annual maximum rainfall. The K_m factor is the frequency factor, while X_{max} is the highest recorded rainfall value. Finally, X_{n-1} and S_{n-1} indicate the mean and standard deviation of the annual maximum rainfall data without the highest recorded value.

The simulation of inflows into the dam was conducted using the HEC-HMS rainfall-runoff model, which is proficient in generating runoff estimations from daily rainfall data (CEIWR-HEC, 2017). The preparation of HEC-HMS projects involved utilizing the WMS software to facilitate the requisite processes. WMS is a comprehensive watershed solution that automates delineation, as well as hydrologic and hydraulic modeling and facilitates floodplain and storm drain mapping. For the simulation, the soil conservation service-curve number (SCS-CN) method was used to estimate losses, the SCS unit hydrograph was employed to transform the inputs, and the Muskingum-Cunge method was used to determine channel routing.

5. Hydraulic modeling

Dam breach parameters were estimated using MacDonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990), Froehlich (1995), Froehlich (2008), and Xu and Zhang (2009) equations. The breach shape was assumed to be a trapezoidal breach as shown in Figure 6. HEC-RAS can estimate the dam breach parameters of the previous models using the Parameter Calculator option. In this research, Parameter Calculator was used to estimate the parameters for the models mentioned above utilizing various variables such as dam characteristics and initial conditions at the time of failure.

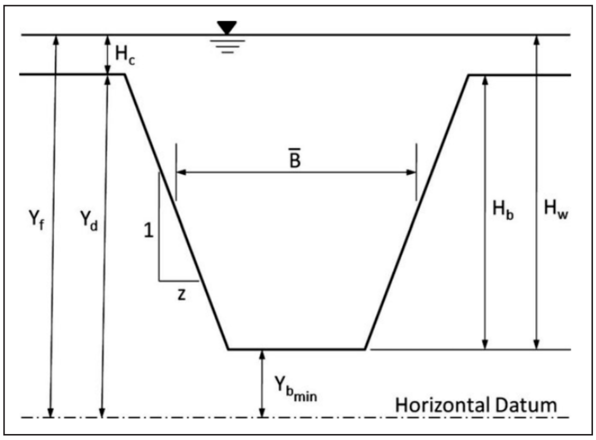

Figure 6. Dam breach shape and parameters (Froehlich, 2008)

In this study, two scenarios were simulated. They are overtopping and piping failures. It was assumed that the dam would fail when water would reach the crest of the dam in the overtopping scenario. In the piping scenario, the failure occurs in fair weather. Accordingly, the date 6/6/2020 was chosen for testing this scenario, as a typical day in the summer. At this date, the height of water and storage volume in the KTD was 99 m and 67.9 MCM. The data of water elevation-storage volume are a daily record of water elevation and storage from 2008 to 2020 for the KTD. Using this data, an elevation-storage volume diagram was formed as shown in Figure 7. A two-dimensional flow area was created using a square grid with a spacing of 75 m. To cover the entire area downstream of the dam, 270,000 square cells were generated. Moreover, a 30-second interval was selected as a time step for the simulation.

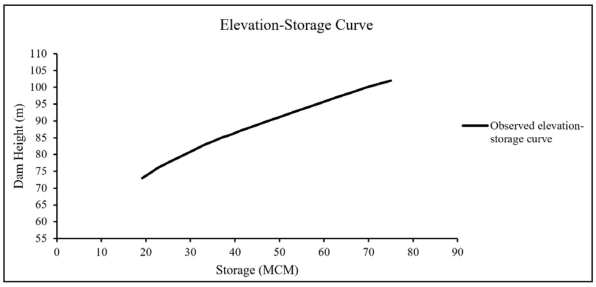

Figure 7. Elevation-Storage curve of the KTD

In this study, a 2D unsteady flow simulation was used to route the PMF hydrograph through the dam utilizing the 2D diffusion wave equations. The HEC-RAS model generates several maps such as inundation, velocity, duration, and arrival time maps. These maps were exported to the ArcGIS and the hazard map was classified according to the depth of the water resulting from the breach flood.

In summary, the paper investigates the hydrological analysis of the water upstream of the dam, the dam failures empirical models, and the hydraulic characteristics of the flood wave in the downstream areas of the dam.

6. Results and Discussion

The study aims to investigate the hydrologic characteristics of ZRW and estimate dam breach parameters for KTD. Also, a 2-D unsteady flow was simulated to predict the resulting flood in the downstream regions of the dam. Finally, flood mapping was performed to visualize the flood in these regions.

6.1 Probable Maximum Flood Hydrograph

Utilizing the features of the WMS software, the basin was divided into 8 sub-basins as shown in Figure 8, the CN values for these basins were entered, the PMP values that were calculated using Hershfield's equation were entered, and the channel routing was done using the Muskingum-Cunge method. After that, the final WMS model was exported to the HEC-HMS. Finally, a PMF hydrograph was computed using the SCS method.

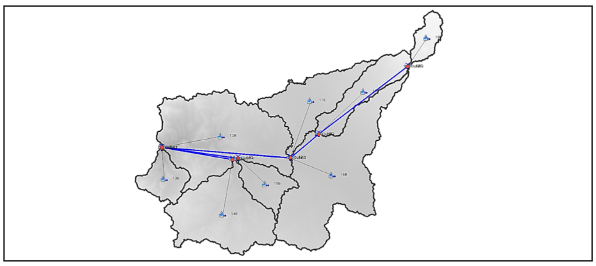

Figure 8. Hydrologic model using HEC-HMS

The CN of the basin was estimated to be approximately 88. Table 2 represents the values of the land use for the basin in general. It is noted from the table that most of the watershed area is composed of shrubs, approximately 40%. In addition, 28.7% of the land is bare, while urban land represents 19.3% of the ZRW.

Table 2. Land use of ZRW

Category	Land use (%)
Water	0.1
Trees	0.6
Grass	0.1
Crops	11.2
Scrub/Shrub	40.1
Urban Area	19.3
Bare Ground	28.7
Total	100

The PMF hydrograph from the HEC-HMS was used in HEC-RAS as an input. Figure 9 represents the PMF hydrograph of the PMP in the ZRW. The maximum flow was approximately 3823.

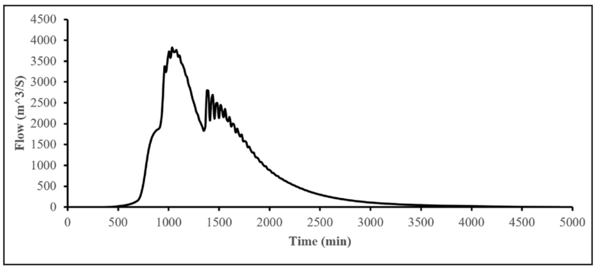

Figure 9. PMF hydrograph of the PMP

6.2 Breach parameters and breach flood hydrographs

Dam failure was simulated for both scenarios, piping and overtopping, and breach hydrograph curves were found at the outlet of the dam. Tables 3 and 4 show the dam breach parameters for overtopping and piping failures that were estimated using the five models mentioned previously. Compared to what is often utilized in HEC-RAS for breach development time, the Xu and Zhang data used in the equation for breach development time cover more of the initial and post-erosion times. Thus, the use of breach formation time would give inaccurate results (Brunner et al., 2016).

Table 3. Dam breach parameters for overtopping failure

Method	Breach Bottom Width (m)	Side Slope (H: V)	Breach Development Time (hrs.)
MacDonald et al.	47	0.5	2.65
Froehlich (1995)	96	1.4	0.83
Froehlich (2008)	70	1	0.68
Von Thun & Gillette	215	0.5	1.85
Xu & Zhang	151	1.08	1.94

Table 4. Dam breach parameters for piping failure

Method	Breach Bottom Width (m)	Side Slope (H: V)	Breach Development Time (hrs)
MacDonald et al	22	0.5	2.28
Froehlich (1995)	60	0.9	0.69
Froehlich (2008)	47	0.7	0.57
Von Thun & Gillette	192	0.5	1.67
Xu & Zhang	68	0.59	2.16

Figures 10 and 11 represent the breach flow hydrograph for overtopping and piping failures using the five equations. Froehlich's 1995 model has the greatest breach rate for overtopping failure, according to the data, with a value of 67782 . Von Thun is next, with a peak flow of roughly 65609 , and Froehlich 2008 results, with a peak flow value of 65456 . After that, the maximum discharge predicted by the Xu and Zhang equation is 35832 . Finally, the MacDonald and Langridge-Monopolis equation produced the lowest result, which amounted to 21499 . The breach flows for the five equations for pipe failure were as follows: 43748 for Froehlich 2008, 42049 for Froehlich 1995, 23534 for Von Thun and Gillette, 17487 for Xu and Zhang, and 13321 for MacDonald and Langridge-Monopolis.

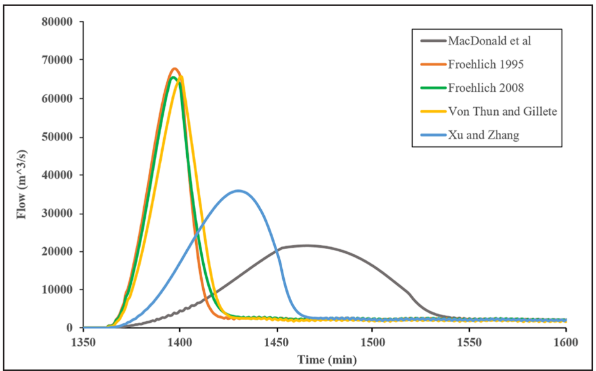

Figure 10. Breach flow hydrographs for overtopping failure

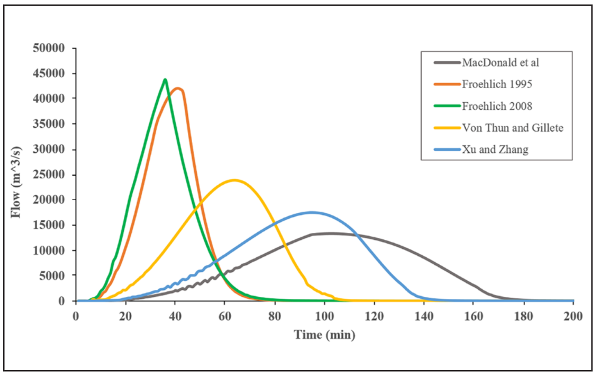

Figure 11. Breach flow hydrographs for piping failure

6.3 Flood Hazard Mapping

There are three urban regions located downstream of the King Talal dam, namely, Deir Alla, Twal Shamali, and Twal Janoobi, with a population exceeding 27,000 (Department of Statistics, 2020). Figure 12 displays the submersion map divided into several groups according to the MILT classification, as previously mentioned. It is noted that the majority of the flooded regions are categorized as H5, which stands for extremely hazardous regions. Deir Alla and Twal Shamali are fully included in this classification, whereas Twal Janoobi is only partially included. The remaining sections of this region are included in the H4 and H3 categories. In addition, Table 5 illustrates the area of the flooded areas for both scenarios. We note that the submerged areas, due to the overtopping scenario, are higher than the submerged areas due to the failure of the piping.

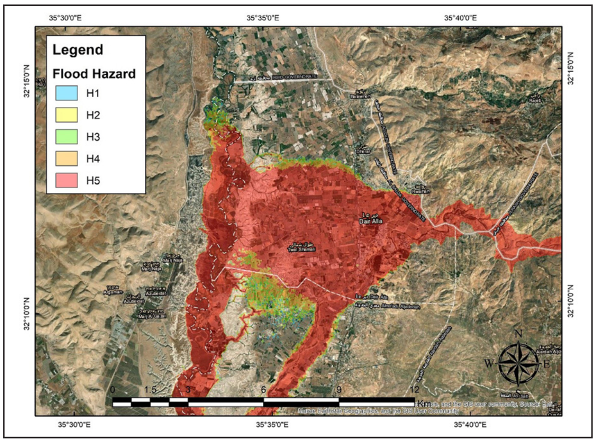

Figure 12. Flood hazard map due to dam failure

Table 5. Inundated area due to dam failure

Method	Inundated area (km^2)	
	Overtopping	Piping
MacDonald et al	163.0	129.7
Froehlich (1995)	162.1	132.5
Froehlich (2008)	161.9	132.6
Von Thun & Gillette	162.0	131.3
Xu & Zhang	160.4	129.8

Figure 13 represents the height of water resulting from the simulation of the dam failure. The maximum height in the Deir Alla region is 41 meters, while it is 25 and 17 meters in Twal Shamali and Twal Janoobi. These areas are considered residential areas. Also, the height of the water in the valleys and agricultural areas exceeds 100 meters.

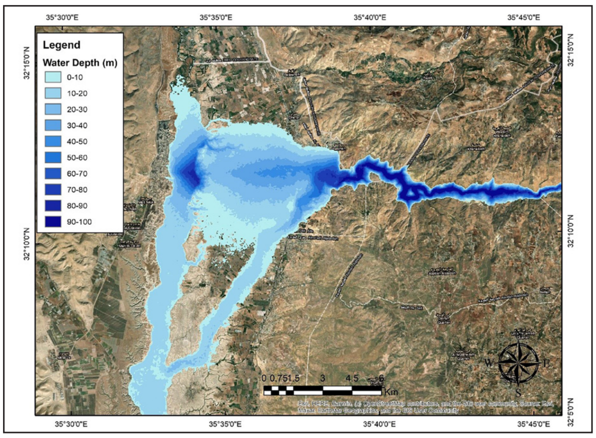

Figure 13. Inundation map for the downstream area of the dam

In terms of the flood's arrival time, Figures 14 and 15 show how long it would take for each urban region for the two failure scenarios. Since the dam's breakdown, the predicted flood takes around two and a half hours to reach Deir Alla and three hours to reach Tawal Shamali and Tawal Janoobi according to MacDonald model. The flood in the other models is comparatively faster than that in the MacDonald model.

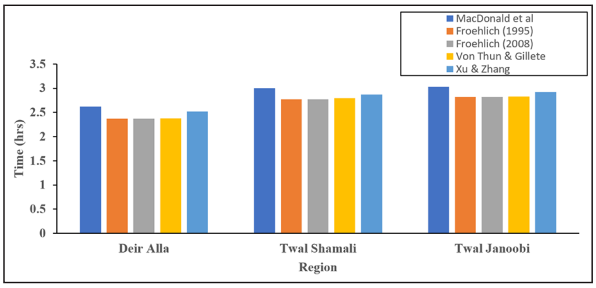

Figure 14. Arrival time for overtopping failure

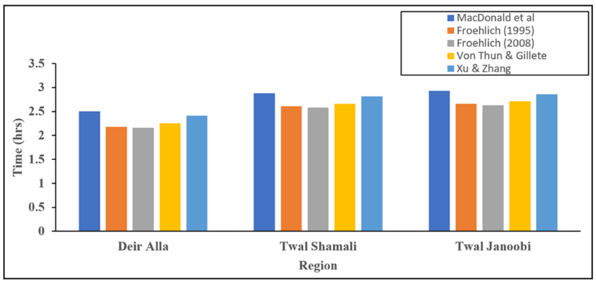

Figure 15. Arrival time for piping failure

Velocity maps provide important details regarding flood flow dynamics and potential erosion. Both the population and the infrastructure could be seriously affected. Figure 16 presents the velocity map that resulted from the simulation. The flood velocity in Deir Alla is about 85 m/s, which is higher than the other two urban regions. Figure 17 depicts how long the flood lasted in each area. Deir Alla's eastern region is subject to floods that last for more than 48 hours. With regard to the remaining areas, the flood continues for a maximum of eight hours. Tawal Shamali and Tawal Janoobi regions are affected by lengthy floods that persist for around 40 hours.

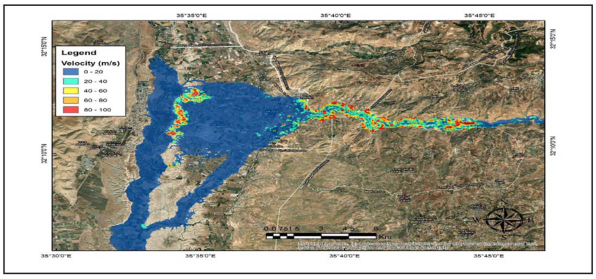

Figure 16. Velocity map due to dam failure

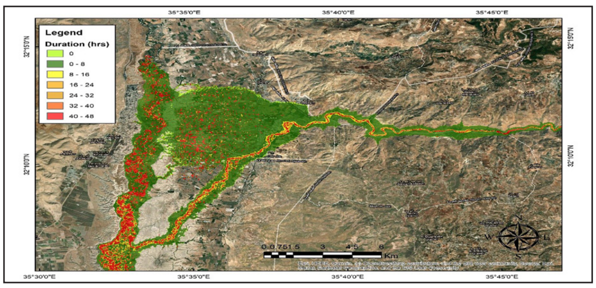

Figure 17. Flood duration period

7. Conclusions

Dam breach parameters were estimated using different models for two failure mode scenarios, namely, overtopping and piping failures. A 2-dimensional unsteady flow simulation was used to route the breach flood in the downstream area of the dam. The maximum breach flows for overtopping and piping scenarios were estimated to be 65456 and 43748 m^3/s, respectively. The study generated inundation, flood hazard, duration, and velocity maps for the dam's downstream areas. Most of the inundated areas were classified as extremely hazardous. The maximum depth of water was approximately 107 meters at some agricultural lands. More specifically, the inundation map shows that the depth of water ranged from 10 to 50 meters in Deir Alla, Tawal Shamali, and Tawal Janoobi regions. The velocity of the flood exceeds 100 meters per second in some steep places and wadis.

Furthermore, the fastest arrival time of the flood is estimated to be 2 hours after the breach occurred due to piping failure. Also, the flood in some regions lasts for about 40 hours. This underscores the importance of good emergency response plans and long-term flood management policies.

Overall, the paper provides valuable insights into the dynamics of dam breach floods and their effect on downstream regions. These findings could aid in developing plans and policies in order to minimize the risk of loss of life and property due to dam failure. The emergency preparedness strategies include developing a telemetry monitoring system for the dam body and spillway, an early warning system for the upstream watersheds, and physical evacuation and response plans for residents downstream of the dam. It is recommended to propose various flood mitigation and flood risk management tools and improvements for major infrastructures like roads, bridges, and hospitals.

The results of this analysis can help local authorities to prioritize their plans based on the observed arrival times and flood durations. It is important to engage local communities, government agencies, and other stakeholders in developing integrated flood preparedness strategies and management plans and raising their public awareness during a flood crisis.

It is recommended to conduct more national dam break studies to investigate possible dam breaks for the major dams in Jordan and link that to the National Crisis Management Center.

References

Al-Azzam, N.; Al-Kuisi, M. (2021). Determination of Flash Floods Hazards and Risks for Irbid Governorates Using Hydrological and Hydraulic Modelling. Jordan J. Earth Environ. Sci. (JJEES) 12, 81–91.

Alias N E, Luo P, Takara, Kaouru. (2013). Probable Maximum Precipitation Using Statistical Method for The Yodo River Basin Journal of Japan Society of Civil Engineers 69, 157-162

Al-Amoush, Hani & Alshabeeb, Abdel Rahman & Al-Adamat, Rida & Al-Fugara, A'Kif & Alayyash, Saad & Shdeifat, Akram & Al-Tarazi, Eid & Rajab, J.. (2017). The Use of GIS Techniques and Geophysical Investigation for Flood Management at Wadi Al-Mafraq Catchment Area. Jordan J. Earth Environ. Sci. (JJEES) 8, 97-103.

Al-Salahat, Murad, Radwan Al-Weshah, and Saif Al-Omari. (2024). 'Dam break risk analysis and flood inundation mapping: a case study of Wadi Al-Arab Dam', Sustainable Water Resources Management, 10: 74.

Al-Shibli, F. M., Maher, W. A., Thompson, R. M. (2017): The Need for a QuantitativeAnalysis of Risk and Reliability for Formulation of Water Budget in Jordan. – Jordan J. Earth Environ. Sci (JJEES) 8, 77-89

Aydemir, A. and Güven, A. (2017). 'Modified risk assessment tool for embankment dams: case study of three dams in Turkey.' Civil Engineering and Environmental Systems. 34 (1), 53-67.

Brunner, G. Benchmarking of the HEC-RAS Two-Dimensional Hydraulic Modeling Capabilities; U.S. Army Corps of Engineers: Davis, CA, USA, 2018; pp. 1–137.

Brunner, G. W., United States Institute for Water Resources (U.S.), and Hydrologic Engineering Center (U.S.). (2016). 'HEC-RAS river analysis system: Hydraulic reference manual.' Davis, CA: US Army Corps of Engineers, Institute for Water Resources, Hydrologic Engineering Center.

CEIWR-HEC; HEC-HMS. Hydrological Modeling System: Application Guide; US Army Corps of Engineers Hydrologic Engineering Center: Davis, CA, USA, 2017; pp. 3.1–3.19.

Department of Statistics. (2020). 'Estimated Population of the Kingdom by Municipality and Sex, at End-year 2020.' Jordan.

Desa M N, Noriah A, Rakhecha P. (2001). Probable Maximum Precipitation for 24 h Duration Over Southeast Asian Monsoon Region-Selangor Malaysia. Atmospheric Research 58(1) 41-54

Dinçergök, T. (2007). 'The Role of Dam Safety in Dam-Break Induced Flood Management.' International Conference on River Basin Management, May 23th -25th. pp. 683-691, Southampton, UK.

El-Radaideh, N., Al-Taani, A. A., & Al Khateeb, W. (2017). Status of sedimentation in King Talal Dam: case study from Jordan. Environmental Earth Sciences, 76, 132.

Froehlich, D. (1995). Peak Outflow from Breached Embankment Dam. Journal of Water Resources in Planning and Management,121, 90-97.

Froehlich, D. (2008). 'Embankment Dam Breach Parameters and Their Uncertainties'. Journal of Hydraulic Engineering, 134, 1708-1721.

Goodarzi, E., Lee, T. S., & Ziaei, M. (2013). Dam overtopping risk using probabilistic concepts – Case study: The Meijaran Dam, Iran. Ain Shams Engineering Journal, 4, 185-197.

Hershfield DM. (1961). Estimating the probable maximum precipitation. Journal of the Hydraulics Division, 87, 99–106

MacDonald, T. and Langridge-Monopolis, J. (1984). 'Breaching Characteristics of Dam Failures.' Journal of Hydraulic Engineering, 110, 567-586.

USDA NRCS. (1986). Urban Hydrology for Small Watersheds. US Department of Agriculture, Soil Conservation Service, Engineering Division: Washington, DC, USA.

Von Thun, J. and Gillette, D. (1990). 'Guidance on breach parameters' (Unpublished internal document, 17 p.). U.S. Bureau of Reclamation, Denver, Colorado, March 13, 17 p.

Wahl, T. L. (1997). Predicting embankment dam breach parameters-a needs assessment. In: Proceedings of the International Association for Hydraulic Research Congress, (pp. 48–53). San Francisco, California: IAHR.

Whitham, G. (1955). 'The effects of hydraulic resistance in the dam-break problem.' Proceedings of the Royal Society A, 227 (1170), 399–407.

World Meteorological Organization (WMO). (2012). Manual for estimation of probable maximum precipitation (PMP). Geneva, Switzerland: World Meteorological Organization.

Xu, Y., Zhang, L.M. (2009). Breaching parameters for earth and rockfill dams. Journal of Geotechnical and Geoenvironmental Engineering, 135, 1957–1970.