
HYDRODYNAMIC ASSESSMENT OF PIPING-INDUCED DAM-BREAK AND DOWNSTREAM FLOOD HAZARD CLASSIFICATION AT ZOBE DAM, NIGERIA. AN INTEGRATED HEC-HMS AND HEC-RAS MODELING APPROACH

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ABSTRACT

Dam infrastructure occupies a central position in water resource management; and dam failure can produce severe consequences, especially in data-scarce regions where observed hydrological and failure data are limited. This study assesses a hypothetical piping failure of Zobe Dam, an earthen embankment with a documented history of seepage on the Karaduwa River in Katsina State, Nigeria, utilizing coupled HEC-HMS and 2D HEC-RAS modelling software. A terrain-delineated HEC-HMS model comprising five subbasins and two Muskingum-Cunge-routed reaches was used to generate inflow hydrographs for 30-, 50-, and 100-year design storms. Global Curve Number 250 (GCN250)-derived curve numbers and Soil Conservation Service (SCS)-derived lag times were used for hydrologic simulation, while breach parameters were estimated using Froelich (2008) formulation. Breach initiation was fixed at the documented Full Supply Level of 493.00 m across all scenarios. Peak basin-outlet discharge increased from 4,616.3 to 5,333.4 m³/s between the 30- and 100-year events, while peak breach discharge increased from 2,377.24 to 2685.35 m³/s. Flood depth, velocity, and inundation extent were from RAS Mapper and mapped in QGIS, with downstream hazard classified using the Australian Institute for Disaster Resilience (AIDR) Guideline 7-3 depth-velocity framework. Hazard classification remain unchanged across the three scenarios: Garhi was classified H6; Hakazama, Safana and Kunamawa as H5; and Dogon Ruwa and Makwanshi as H2/H3. The Water Treatment Plant and Bodole remained outside the modeled inundation extent. Sensitivity testing indicated limited variation in routed and breach discharges within the tested parameter ranges, while floodplain roughness produced a localized change in hazard classification at Safana. Garhi and the downstream road corridor linking Dutsin-Ma toward Kankara were identified as priorities for emergency preparedness and evacuation planning.

Keywords: Dam breach; piping failure; flood hazard mapping; HEC-RAS; HEC-HMS; QGIS

1. INTRODUCTION

Dams provide water security, irrigation, hydropower, and flood control, but can also become sources of catastrophic loss when they fail. More than 2,000 dam failures have been recorded globally over the past century, causing over 200,000 deaths (Nasser et al., 2025; Hariri-Ardebili, 2018). Earthen embankment dams are especially prone to failure because of their lower structural rigidity relative to concrete or masonry dams (Planès et al., 2016; Tang et al., 2022), and their collapse generates flood waves capable of altering downstream morphology and devastating riparian communities (Zhou et al., 2026). Failure typically results from piping, overtopping, seepage, or seismic loading (Chen et al., 2023). Piping, or internal erosion, occurs when seepage through the dam or its foundation progressively entrains fine soil particles, enlarging internal voids until a mass collapse of the overlying material sharply increases outflow (Saliba et al., 2019; de Paula Barreto et al., 2024). Because piping failures develop internally and can remain undetected until collapse is

imminent, they pose a distinct and often underappreciated risk relative to overtopping, which is more readily anticipated from reservoir stage alone (Chen et al., 2023). Understanding this mechanism, and quantifying the flood wave it would generate, is essential for downstream disaster preparedness.

Historical events such as the Teton (1976) and Banqiao (1975) dam failures catalyzed the development of formal breach-hydrograph and inundation-mapping methods (Bezerra and Eleuterio (2026). An estimated one to two significant dam-breach events occur globally each year, disproportionately affecting populations near the failed structure (Rana et al., 2022); the 2015 Swar Chaung sluice-gate breach in Myanmar and the 2023 Derna cascading dam failures in Libya illustrate how quickly such disasters unfold when settlements lie close to the source, and hydraulic simulation is the principal means of quantifying breach discharge, depth, velocity, and inundation extent in advance of an actual failure. While empirical regression relationships developed by MacDonald and Langridge-Monopolis (1984) and Froehlich (1995, 2008) now allow breach geometry and discharge to be estimated quantitatively from dam and reservoir characteristics. Combined with GIS-based terrain processing, coupled hydrologic-hydraulic software such as HEC-HMS and HEC-RAS enables engineers to simulate breach initiation, progression, and downstream flood propagation even where ground-survey data are limited (Peker et al., 2024). Most published applications of this coupled approach emphasize overtopping scenarios, however, with piping treated, at best, as a secondary comparison; site-specific, piping-focused assessments using locally derived design-storm hydrology remain comparatively scarce, particularly for embankment dams in West Africa. This emphasis partly reflects the greater modelling uncertainty associated with piping failure, which depends on poorly constrained internal erosion processes, initiation conditions, and breach progression that are difficult to characterize without detailed geotechnical and dam monitoring data; however, overtopping failure can be more readily represented using hydraulic loading and established breach-parameter relationships. Zobe Dam, an earthen embankment impounding the Karaduwa River in Katsina State, Nigeria, illustrates this gap directly. The dam has a documented history of foundation seepage (ESIA Report for Zobe Dam and Associated Infrastructures Rehabilitation Works, 2020), a condition mechanistically linked to piping failure (Magawata et al., 2020), yet no piping-specific dam-break analysis using locally derived design-storm hydrology has previously been conducted for the dam. Downstream, several rural settlements, farmland, and a water treatment plant serving the wider Dutsin-Ma and Katsina area remain without a quantitative, hydraulically modeled understanding of the flood hazard they would face in a breach event.

Accordingly, this study evaluates the downstream consequences of a hypothetical piping-induced breach of Zobe Dam under 30-, 50-, and 100-year design-storm conditions using coupled HEC-HMS and two-dimensional HEC-RAS modeling. The specific objectives are to: (i) model the piping-failure scenario using HEC-RAS 2D hydraulic modeling; (ii) estimate 30-, 50-, and 100-year design-storm runoff hydrographs using HEC-HMS with Gumbel frequency analysis; (iii) simulate the resulting flood wave and characterize peak flow, depth, and velocity; (iv) map the spatial distribution of downstream flood risk against defined hazard thresholds; and (v) translate the results into recommendations for dam safety and emergency planning. The analysis estimates breach hydrographs, flood depths, velocities, inundation extent, and spatial hazard levels, with particular attention to the implications for downstream settlements and emergency preparedness, and offers a replicable methodology for comparable earthen dams across Nigeria's Sokoto-Rima Basin. The study is confined to Zobe Dam using HEC-RAS 6.6 for two-dimensional unsteady-flow simulation of a piping failure, with breach parameters from the Froehlich (2008) method.

2. METHODOLOGY



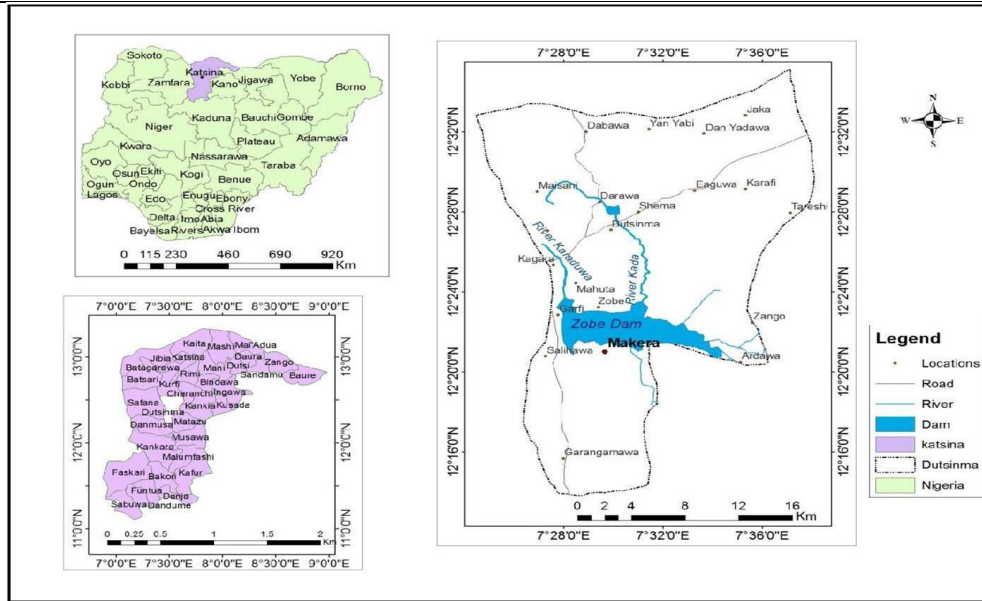
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Figure 1: Overall study workflow, from data acquisition through terrain-based HEC-HMS delineation and design-storm hydrograph generation, HEC-RAS breach and unsteady-flow simulation, to GIS-based flood hazard classification.

2.1 Study Area

Zobe Dam is located at Makera, approximately 65km south of the city of Katsina in Katsina State, Nigeria, between longitudes $7^{\circ} 27' 57.12''$ and $7^{\circ} 34' 4.68''$ E and latitudes $12^{\circ} 20' 34.62''$ and $12^{\circ} 23' 27.48''$ N. The reservoir has a design capacity of $177 \times 10^6 \text{ m}^3$, fed by the Gada and Karaduwa rivers, and was built to irrigate an 8,000ha command area and supply water to Katsina town and surrounding villages. The embankment is a 2.7km curved earth-fill structure with a 110m free-overflow spillway, reinforced at chainage 0+950 by a concrete intake-tower culvert, impounding a reservoir approximately 15km long and 36.9 km^2 in surface area. The dam supplies water for irrigation, domestic use, livestock, and fisheries (Jamda Saidu et al., 2019), emphasizing the scale of disruption a breach would set on the surrounding population.

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Source: Department of Geography and Planning, University of Jos, Plateau State, Nigeria.

Figure 1. Dutsinma showing Zobe Dam (Insert: Nigeria and Katsina State).

Figure 2: Location of Zobe Dam within Dutsin-Ma LGA, Katsina State (Source: Department of Geography and Planning, University of Jos).

Table 2 summarizes the principal structural and hydrological characteristics of Zobe Dam, as documented by the Sokoto-Rima River Basin Development Authority, which formed the core input data for the HEC-RAS geometric model.

Feature	Value
Embankment length	2.75 km
Embankment height	18.9 m
Dam crest width	5.00 m
Upstream slope	1:3.2 (rip-rap, $d = 1.00$ m)
Downstream slope	1:2.2 (rip-rap, $d = 0.35$ m)
Freeboard	2.21 m
Catchment area	2,527 km ²
Active storage capacity	170 x 10 ⁶ m ³
Dead storage capacity	7.0 x 10 ⁶ m ³
Maximum flood storage capacity	263 MCM
Reservoir surface area	4,500 ha
Full storage level (FSL)	493.00 masl
Dead storage level (DSL)	484.60 masl
Spillway No. 1 crest length	110 m
Spillway No. 2 crest length	78 m
Maximum overflow capacity	1,087 m ³ /s

The catchment-wide curve number and time of concentration originally reported for the full 2,527 km² catchment are substituted, for the delineated Zobe sub-catchment used in the HEC-HMS model, by the subbasin-specific values in Table 2a (Section 2.4).

Table 2a. Subbasin curve numbers, derived from the GCN250 global curve-number dataset, and SCS lag times used in the HEC-HMS model.

Subbasin	CN	Lag time (hrs)
S1	70.5	5.9
S2	70.8	5.5
S3	70.0	7.1
S4	73.6	2.8
S5	70.1	5.9

The Karaduwa catchment lies within the Basement Complex of Katsina State, underlain by granite-migmatite and gneiss overlain by laterite and sandy-gravelly loam (ESIA Report for Zobe Dam and Associated Infrastructures Rehabilitation Works, 2020; Kankara, A., & Shazalli, N. U. 2020); permeable alluvial sand and gravel deposits extend across the river channel and adjacent floodplain, strata recognized as preferential pathways for internal erosion and directly relevant to the seepage vulnerability motivating this study. The catchment is dendritic, covering area of about 2,527km² as a tributary of the River Bunsuru, with average rainy-season discharge of 356 m³/s above the dam (Wang et al., 2022). The region receives 600-800 mm of annual rainfall governed by ITCZ migration, concentrated in a May-September wet season (mean 750 mm, peak in August) followed by an eight-month dry season (Geen et al., 2020).

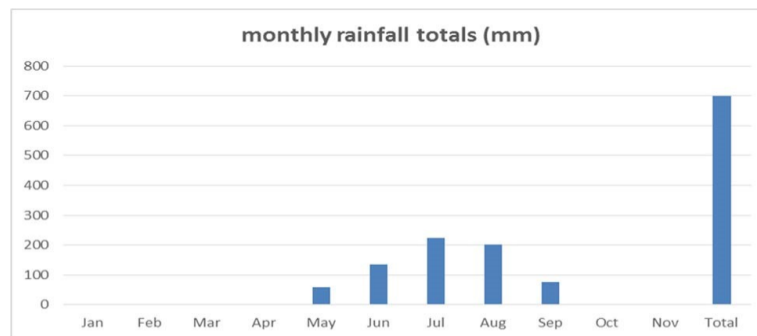


Figure 3: Mean monthly rainfall depth (mm) at the Katsina gauge station (Source: ESIA Report for Zobe Dam and Associated Infrastructures Rehabilitation Works, 2020).

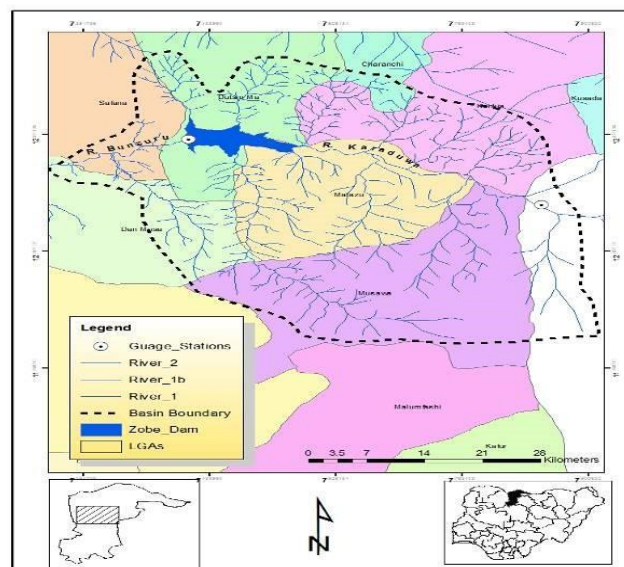


Figure 4: Drainage network of the Karaduwa-Bunsuru catchment showing the Zobe Dam reservoir (Source: Sokoto-Rima River Basin Development Authority).

2.2 Data Sources and Preparation

Table 1 summarizes the data sources used to construct the hydrologic and hydraulic models. Geometric data (dam dimensions, operating levels) were obtained directly from the Sokoto-Rima River Basin Development Authority. A 30m resolution SRTM DEM tile (N12_E007_1ARC_V3.TIF) was obtained from USGS Earth-Explorer and clipped to the study area using ArcGIS Spatial Analyst to provide the terrain surface for the HEC-RAS 2D model. Land-cover information from Landsat imagery and satellite base-map interpretation informed the Manning's roughness assignment described in Section 2.6. These datasets collectively provided the hydrological, topographical, and hydraulic information required to develop, in terms of parameters, and evaluate the joined HEC-HMS and HEC-RAS modelling framework

Data	Format	Source	Use
Rainfall records	Time series	Fudma meteorological station	Gumbel frequency analysis / design storms
Digital Elevation Model	Raster (30m)	USGS EarthExplorer (SRTM)	Terrain surface for HEC-RAS 2D mesh
Dam and reservoir geometry	Engineering records	Sokoto-Rima River Basin Development Authority	HEC-RAS geometric model, breach parameters
Land use / land cover	Raster	Landsat imagery, Google satellite basemap	Manning's roughness assignment
Manning's roughness (n)	Literature-based estimate	Assigned per land-cover class (Section 2.6)	Channel and floodplain roughness input

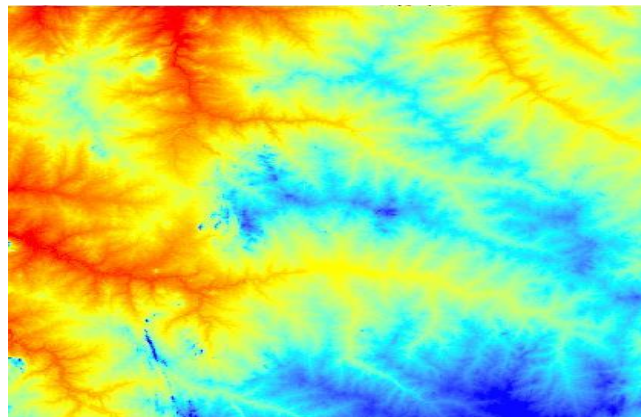


Figure 5: DEM of the Zobe Dam catchment derived from SRTM 30 m data (red = high elevation, blue = low elevation).

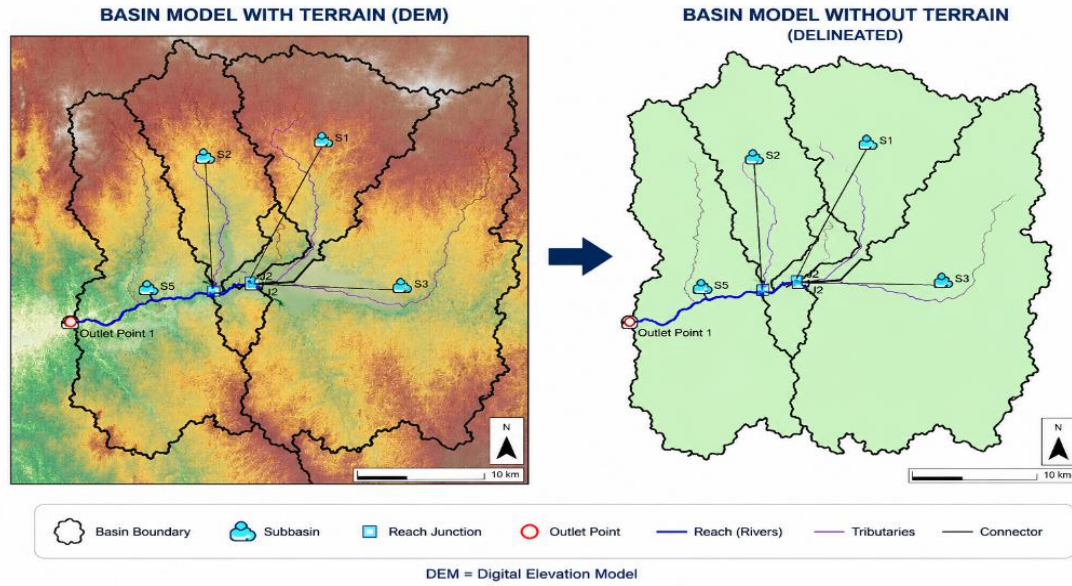


Figure 6: HEC-HMS basin model schematic showing the five terrain-delineated subbasins (S1-S5), the two Muskingum-Cunge-routed reaches (R1, R2), junction elements, and the basin outlet (Sink-1).

2.3 Frequency Analysis and Design Rainfall

Design rainfall depths for 30-, 50-, and 100-year return periods were estimated from the annual maximum rainfall series recorded at the Fudma meteorological station using Gumbel's extreme-value distribution:

$$XT = x - \bar{x} + K \cdot \sigma$$

$$\text{Where } K = \left(\frac{y_T - y_n}{S_n} \right)$$

$$y_T = - \left[- \ln \left(\ln \left(\frac{T}{T-1} \right) \right) \right]$$

where $\bar{x}$ and σ are the sample mean and standard deviation of the annual maximum series and K is the frequency factor for return period T , y_T is the reduced variate, y_n the reduced mean and S_n is the reduced standard deviation. Annual maximum daily depths were first disaggregated to shorter sub-durations (5 minutes-24 hours) using the empirical relation $P_t = P_{24}(t/24)^{1/3}$, and Gumbel's method was then applied at each duration to obtain return-period design rainfall depths for the 30-, 50-, and 100-year return periods. The design rainfall depths were converted to 24-hour design-storm hyetographs at five-minute resolution and used as meteorological input to HEC-HMS (Section 2.4).

2.4 Rainfall-Runoff Modelling

The Zobe sub-catchment was delineated directly from the SRTM DEM using HEC-HMS's terrain-processing tools, yielding five subbasins (S1-S5) connected through two routed reaches (R1, R2) to a single basin outlet (Figure 6). No streamflow gauge exists on the Karaduwa River, so model parameters were derived from physical watershed characteristics and not adjusted to observed hydrographs, consistent with standard practice for ungauged catchments (Section 2.8).

Curve Numbers were obtained per subbasin from GCN250 (Jaafar & Ahmad, 2019), a global 250m curve-number grid combining ESA CCI land cover with the HYSOGs 250m hydrologic soil group dataset, area-averaged for each subbasin by zonal statistics; values ranged from 70.0 to 73.6 (Table 2a), consistent with the mixed cropland, grassland, and scattered woodland cover of this Sudan-savanna catchment. The SCS Curve Number method was selected because it provides a practical and widely adopted approach for estimating rainfall losses in ungauged catchments where observed infiltration measurements are unavailable. GCN250 provides a physically based, spatially distributed alternative to assigning a single empirical CN in such data scarce settings and has shown a reasonable agreement with independent runoff products and global scales, including application in

Africa (Jaafar et al., 2019; Sujud & Jaafar, 2022). However, its use at the local watershed like the Karaduwa carries uncertainty because the derived CNs inherit uncertainties from the developed global land-cover and hydrologic-soil-group datasets and also the generalized CN lookup relationships. Hence, the resulting values were therefore physically valid estimates. The SCS Curve Number loss method,

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

where $S = \left(\frac{25400}{CN}\right) - 254$ (mm), was applied with these subbasin-specific CN values.

Subbasin lag times were computed from the SCS lag equation,

$$L = \frac{l^{0.8}(S + 1)^{0.7}}{1900 Y^{0.5}}$$

where l is the hydraulic length (ft), Y is the average subbasin slope (%), and $S = (1000/CN) - 10$, using length and slope values from the terrain delineation; resulting lag times ranged from 2.8 to 7.1 as presented in (Table 2a) across the five subbasins, applied in the SCS unit-hydrograph transform (PRF-484).

Name	CN	L(km)	Y	S	L(ft)	Y(%)	Tc	Lag(Hr)	Lag(min)
S1	70.47	29.60	0.07	4.19	94984.10	7.25	9.90	5.94	356.54
S2	70.82	26.79	0.07	4.12	85967.45	7.04	9.19	5.51	330.85
S3	70.02	38.24	0.08	4.28	122700.43	7.80	11.86	7.12	426.95
S4	73.55	11.42	0.06	3.60	36649.89	6.02	4.66	2.79	167.67
S5	70.13	30.78	0.08	4.26	98783.66	7.93	9.86	5.92	355.09

Channel routing for R1 and R2 used the Muskingum-Cunge method, which derives routing behavior from channel geometry, slope, and roughness rather than from calibration against observed inflow-outflow pairs. Reach length and slope were taken from the terrain delineation (R1: 12.126 km, 0.00091; R2: 3.111 km, 0.00546). Because the SRTM resolution was not enough to directly resolve channel cross-sectional geometry, two cross-sectional profiles were sketched along each reach using Google Earth Pro v7.36, based on the visible channel trace and elevation profile and the resulting dimensions were averaged for each reach. The Channel cross-sections approximated as trapezoidal, with bank height and top width; gave a representative geometry of approximately 94m and 68m bottom width and 2H:1V assumed side slopes, for R1 and R2 respectively. Manning's n was initially set to 0.035 for a natural earth channel with light vegetation (Chow, 1959). Both this roughness assumptions and the sketched cross-sectional geometry are potential sources of uncertainty in the Muskingum-Cunge routing; their effect on simulated peak discharge was tested and reported (Section 3.6), together with the other parameter sensitivity checks performed in this study.

The three design storms from Section 2.3 were applied as a 24-hour Frequency Storm at a five-minute computational time step, with the simulation window to 72 hours to allow each hydrograph to fully recede to near-baseflow conditions. The resulting hydrographs at the basin outlet were used as HEC-RAS unsteady-flow boundary conditions (Section 2.6). Internal computational consistency was confirmed by verifying that the unit-hydrograph volume equaled 1mm for each subbasin; this check confirms only that the model computes runoff correctly given its inputs. Accordingly, the hydrological model was verified through parameter consistency, comparison with published ranges, and evaluation of the simulated hydrograph characteristics and not direct calibration against observed streamflow.

2.5 Breach Scenario and Breach-Parameter Estimation

Overtopping accounts for approximately 70.9% of documented dam-breach incidents and piping for 14.3% (Zhong et al., 2021); reflecting on Zobe Dam's seepage history (Section 2.1) and not its overall failure-mode base rate, this study models a piping failure occurring during normal reservoir operation. Piping is characterized by a discrete erosional conduit through the embankment interior, in contrast to the progressive crest-downward erosion of overtopping, and this distinction governed the parameters of the HEC-RAS breach module.

Because detailed geotechnical information required to predict the precise onset of piping was unavailable, a hypothetical piping-induced breach was assumed to initiate when the reservoir water surface reached the documented Full Supply Level (FSL = 493.00 m, Table 2). This assumption represents a “sunny-day” piping failure initiated at the normal operating level, transitioning into the design-storm inflow, rather than an overtopping-triggered failure. The same breach-initiation criterion was applied across the 30-, 50-, and 100-year scenarios to maintain a consistent failure condition and allow the effect of convergent storm magnitude on breach response to be compared. The breach invert was set to 489.47 m, which is the dam's documented foundation level. Accordingly, the study evaluates the downstream consequences of an assumed FSL-initiated piping failure subjected to subsequent storm loading, and not predicting the actual onset or probability of piping failure.

Reliable projection of peak discharge and inundation extent requires an accurate characterization of breach geometry and development time. A substantial body of regression research, summarized by Wahl (1998), has produced competing formulations for these parameters, compiled in Table 3; because each equation was justified to a different, generally modest sample of historical failures, and can diverge by a factor of two or more for a given dam, practitioners increasingly favor comparative application of several formulas rather than reliance on one, an approach followed here via the HEC-RAS Breach Parameter Calculator (Section 3.2).

Reference	Case Studies (n)	Regression Relation (SI units)
Johnson & Illes (1976)	-	$0.5hd \leq B \leq 3hd$ (earth-fill dams)
Singh & Snorrason (1982, 1984)	20	$0.5 \text{ m} \leq \text{breach depth} \leq 0.61 \text{ m}; 0.25 \text{ hr} \leq tf \leq 1.0 \text{ hr}$
MacDonald & Langridge-Monopolis (1984)	42	$Ver = 0.0261(Vout.hw)^{0.769}; tf = 0.0179(Ver)^{0.564}$
FERC (1987)	-	$B = 2-4xhd; tf = 0.1-1.0 \text{ hr}$ (engineered, compacted)
Froehlich (1987)	43	$B = 0.47.Ko.(S)^{0.25}; tf = 79(S)^{0.47}$
U.S. Bureau of Reclamation (1998)	-	$B = 3hw; tf = 0.011B$
Singh & Scarlatos (1988)	52	B_{top}/B_{bottom} averages 1.29
Von Thun & Gillette (1990)	57	Empirical B, Z, tf guidance
Froehlich (1995b)	63	$B = 0.1803.Ko.Vw^{0.32}.hb^{0.19}; tf = 0.00254.Vw^{0.55}.hb^{0.90}$

Among these, statistical evaluation has repeatedly identified the Froehlich (2008) equations as the best-performing method for earth-fill dams specifically (Temür et al., 2026; Geremew, 2024), the basis for its adoption in this study. The sensitivity of the simulated peak discharge to the adopted breach formation time was tested and reported in (Section 3.6), together with the other parameter sensitivity checks performed in this study.

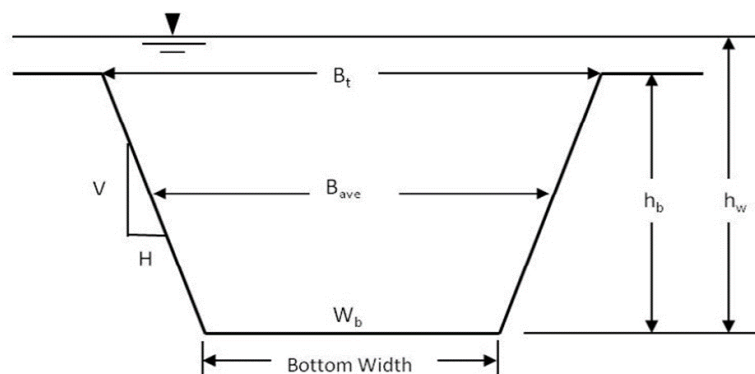


Figure 7: Schematic definition of embankment breach geometry (adapted from the HEC-RAS reference manual).

2.6 Two-Dimensional Hydraulic Modelling

The clipped DEM (Section 2.2) was imported into RAS Mapper at 1/128 rounding precision under the WGS 1984 UTM Zone 32N projection (EPSG: 32632), the correct zone for the study area's longitude (~7.5°E), and the 2D flow area and its connection to the Zobe reservoir storage area were constructed in the geometric editor.

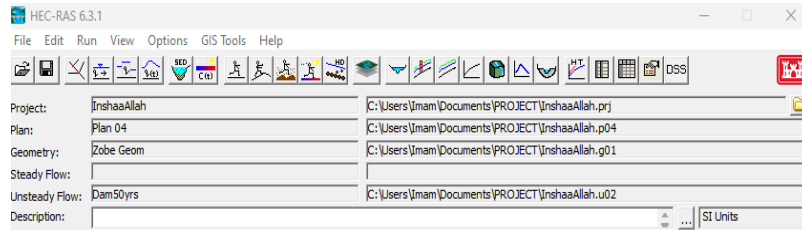


Figure 8: HEC-RAS v6.3.1 project window showing the configured plan, geometry, and unsteady-flow files.

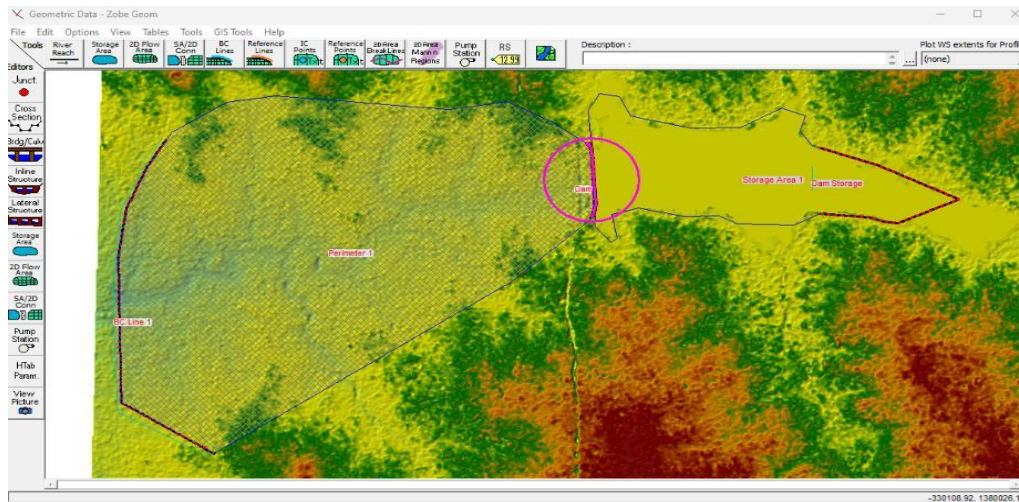


Figure 9: Geometric data editor showing the 2D flow area, storage area, and SA/2D connection (breach location).

A base mesh of cell size of 50x 50m was computationally generated for the polygon of the study area with 44035 grid cells. The selected Manning's roughness coefficients n of 0.035 was set which represent the dominant land-cover conditions within the savannah grassland cover of the Karaduwa-Bunsuru (Section 2.1); but 'Spatially Varied Manning's n on Faces' option was selected; which extracts multiple roughness values along a 2D face rather than depending on a single value at the center. The sensitivity of the resulting depth-velocity hazard classification to this floodplain roughness assumption was tested and is reported in (Section 3.6), together with the other parameter sensitivity checks performed in this study. Unsteady-flow boundary conditions comprised the HEC-HMS design-storm hydrographs at the upstream boundary and normal depth downstream was selected to ensure stable unsteady-flow routing is enough. Simulation used a 30-second computational interval and 5-minute mapping interval over 1 hour of simulated time, within a stable Courant-number range for the HEC-RAS full momentum solver. Depth, velocity, and water-surface-elevation outputs were visualized in RAS Mapper and exported to QGIS for the hazard maps in Section 3.4.

2.7 Flood Hazard Classification

Downstream hazard was classified using the depth-velocity product ($D \times V$) framework of the Australian Institute for Disaster Resilience (AIDR, 2017) Guideline 7-3, Flood Hazard, developed from the vulnerability curves of Cox et al. (2010) and Smith et al. (2014). The adopted classification incorporates the combined influence of flood depth and velocity, thereby providing a more realistic representation of flood risk than either parameter considered independently. This structure defines six hazard classes (H1-H6) from the combined depth and velocity at each location, oriented toward the

stability and safety of people, vehicles, and buildings in floodwater and not directly classifying structural or economic damage assessment specifically.

Class	D x V limit (m²/s)	Limiting depth (m)	Limiting velocity (m/s)
H1	≤ 0.3	0.3	2.0
H2	≤ 0.6	0.5	2.0
H3	≤ 0.6	1.2	2.0
H4	≤ 1.0	2.0	2.0
H5	≤ 4.0	4.0	4.0
H6	> 4.0	-	-

H1 denotes conditions generally safe for people, vehicles, and buildings; H6 denotes conditions under which all building types are considered vulnerable to structural failure and floodwater is unsafe for vehicles and people. Maximum simulated depth and velocity at each downstream location were combined into a single D x V value and classified against these thresholds in QGIS.

3. RESULTS

3.1 Design Rainfall and Rainfall-Runoff Results

HEC-HMS produced peak discharge and time-to-peak at the basin outlet (Sink-1) for the three design storms derived in Section 2.3 (Table 5), confirmed by a unit-hydrograph volume of 1 mm for each subbasin.

Return Period (years)	Simulated Peak Discharge (m³/s)	Time to Peak
30	4,616.3	19h 20min
50	4,773.9	19h 15min
100	5,333.4	19h 15min

Peak discharge increased from 4,616.3m³/s (30-year) to 5,333.4m³/s for the 100-year event, representing an increase of approximately 22% (Table 5). While time-to-peak remains near-constant (~19.25 hours) across all three return periods, indicating that increasing rainfall intensity affected runoff magnitude without altering the watershed's hydrologic response time for these selected rainfall scenarios. The subbasin lag times and reach travel times derived from the terrain delineation (Sections 2.2, 2.4) behavior were consistent, controlled by the catchment characteristics and not storm magnitude. The simulated inflow hydrographs at the Sink-1 were subsequently used as the upstream boundary conditions for the HEC-RAS breach simulations.

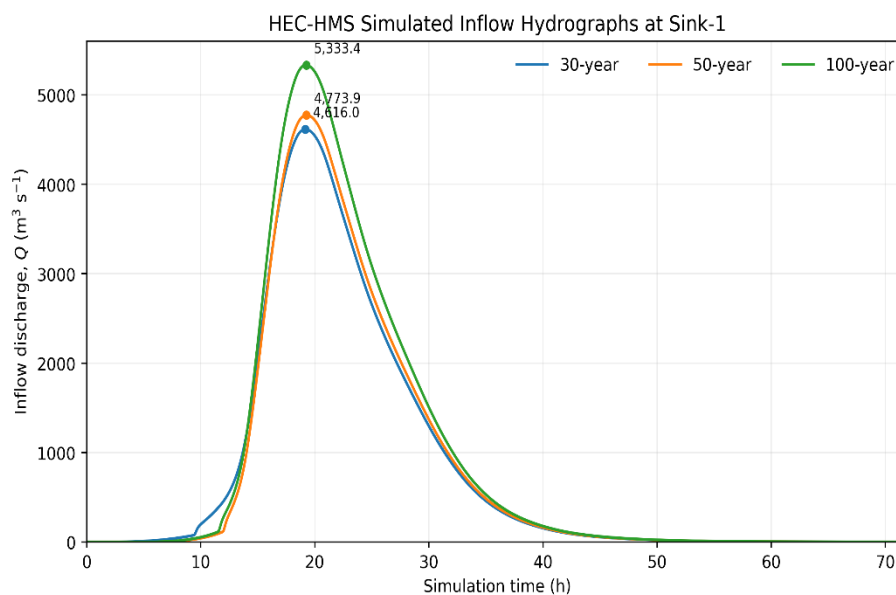


Figure 10: HEC-HMS Simulated Inflow Hydrographs at Sink-1.

3.2 Breach Parameters

Breach parameters were estimated using the HEC-RAS Breach Parameter Calculator based on the empirical relationship of MacDonald and Langridge-Monopolis (1984), Froehlich (1995, 2008), Von Thun and Gillette, and Xu and Zhang formulations listed in Table 3. Following the selection criteria described in section 2.5, the Froehlich (2008) output (Table 6) was adopted for all simulations. Because breach dimensions are primarily by the physical characteristics of the dam and reservoir characteristics not the inflow hydrographs. The Pool Elevation at Failure (493.00m, the documented FSL) and breach invert (489.47m) were similarly held constant across all three scenarios (Section 2.5). This approach ensured that difference in flood response reflected variations of the hydrological inputs not changing breach geometry.

Breach Parameter	Estimated Value (Froehlich, 2008)
Breach bottom width	119 m
Side slopes	0.7 (H:V)
Breach development time	3.73 hrs
Pool elevation at failure (FSL)	493.00 m
Breach invert elevation	489.47 m

Figure 11: HEC-RAS Parameter Calculator comparing breach parameters across regression methods for Zobe Dam.

The adopted breach geometry corresponds to a moderate piping failure scenario and was treated as a hypothetical but internally consistent representation for all scenarios.

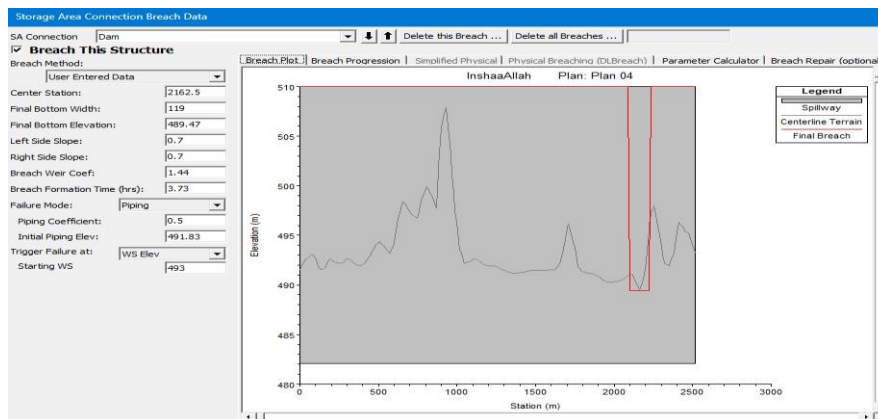


Figure 12: Breach plot for Zobe Dam showing reservoir cross-section, spillway, and final breach geometry.

3.3 Breach Hydrographs and Reservoir Response

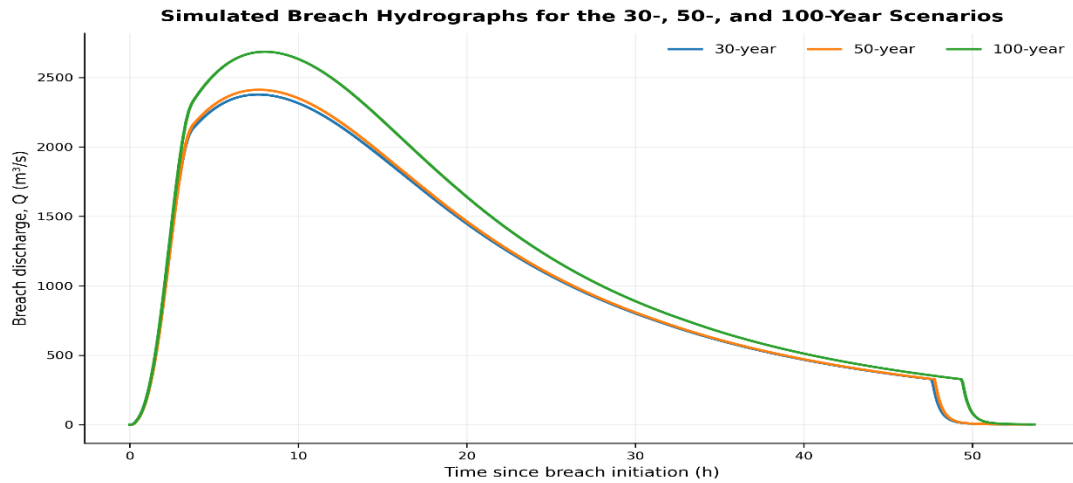


Figure 13a. Breach plot for Zobe Dam showing the outflow Breach Hydrograph for the 30, 50, and 100-year event.

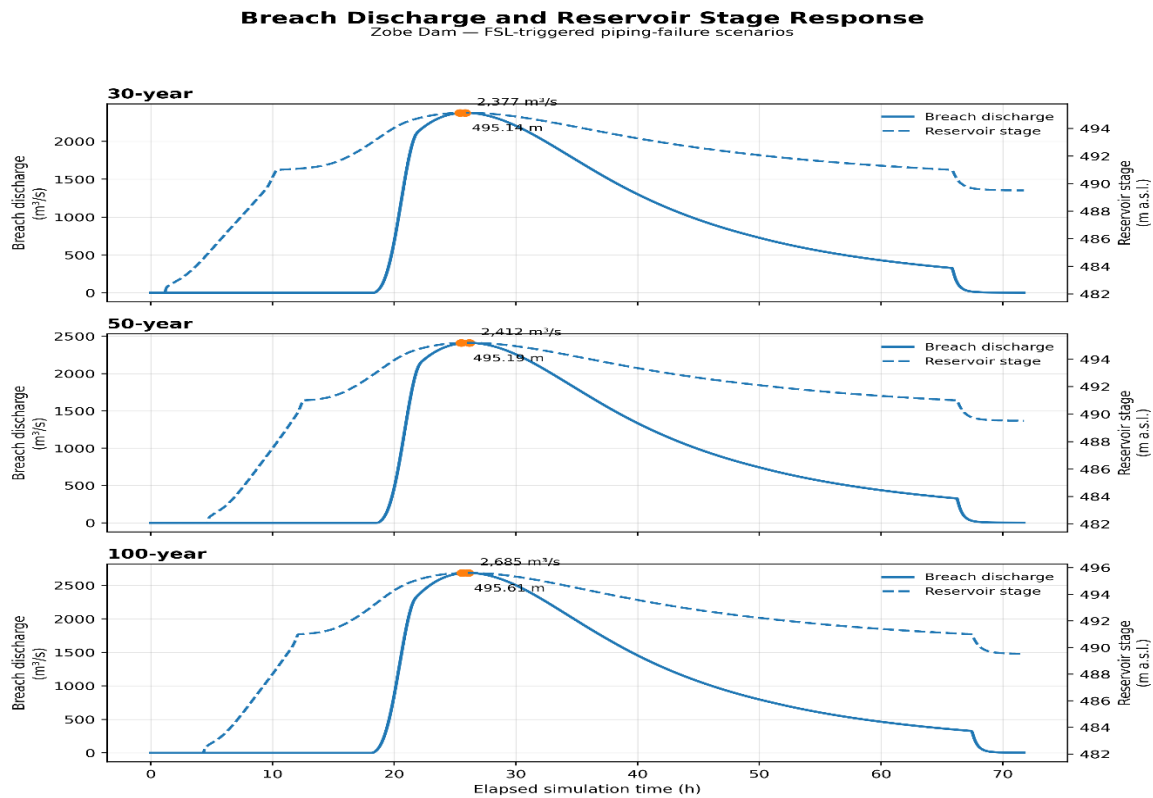


Figure 13b. Breach plot for Zobe Dam showing reservoir stage and Breach Discharge starting at selected Full Supply Level triggered piping scenario.

With the reservoir breach initiating at the documented FSL (493.00m, Section 2.5), Table 7 summarizes the reservoir response by return period.

Return Period	Peak Inflow (m ³ /s)	Maximum Reservoir Stage (m)	Peak Breach Discharge (m ³ /s)
30-year	4,616.3	495.13	2,377.24
50-year	4,773.9	495.19	2,411.69
100-year	5,333.4	495.61	2,685.35

The simulated reservoir response and breach hydrograph for the three design storm scenarios are summarised in Table 7 and plotted in Figure 13a. Peak inflow increased from 4,616.3 m³/s for the 30-year event to 5,333.4 m³/s for the 100-year event, producing a corresponding rise in maximum reservoir water level from 495.13 to 495.61 m. Peak breach discharge also increased with return period, ranging from 2,377.24 to 2,685.35 m³/s.

Despite the increase in inflow, peak breach discharge remained substantially lower than peak reservoir inflow in all simulations as shown in Figure 13c. This behaviour reflects the hydraulic control imposed by the adopted breach configuration and the relatively shallow driving head available above the breach invert (489.47 m) following breach initiation at the documented Full Supply Level (493.00 m) shown in Figure 13b. Consequently, a considerable proportion of the incoming flood volume was temporarily attenuated by reservoir storage before being released through the developing breach, resulting in a moderated breach hydrograph.

The modest increase in breach discharge across the three return periods further indicates that, under a fixed breach geometry and failure elevation, downstream flood response is influenced more by reservoir routing and breach hydraulics than by the incremental increase in design storm magnitude. This observation is consistent with the relatively small differences in maximum reservoir stage among the simulated events. More so, the 30-year simulation produced a volume-accounting error of only 0.05%, demonstrating excellent mass conservation and confirming the numerical stability of the hydraulic model (Section 2.8).

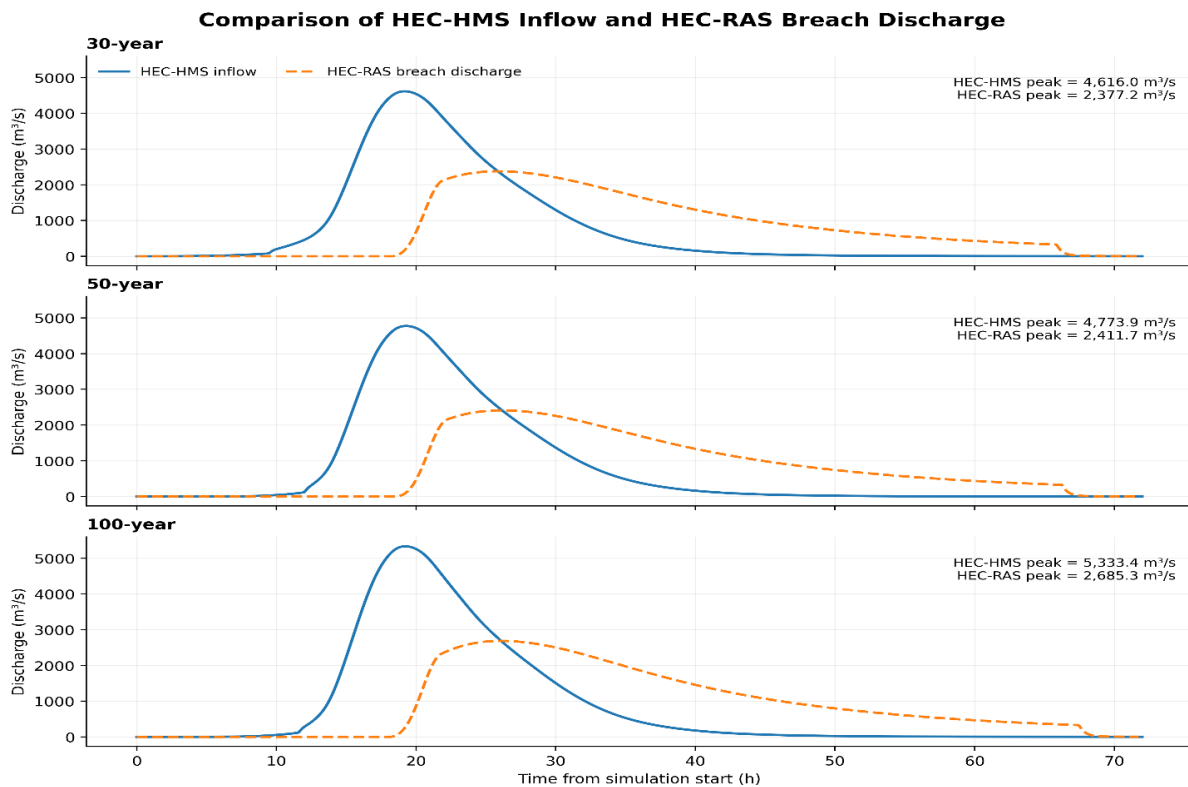


Figure 13c. Breach plot for Zobe Dam showing the comparison of Inflow and Outflow Discharges.

3.4 Flood Inundation Characteristics

Raster data of water depth values and water velocities obtained after flood modeling with HEC-RAS 6.6 were converted into Hazard Index factors using the Spatial Analysis Tools—Reclassify command in the ArcGIS 10.6.1 software. Then, Equation DxV was applied to these raster data with the help of the Spatial Analysis Tools—Raster Calculator command and hazard maps were obtained according to hazard classes according to the limit values in Table 4 of Section 2.7.

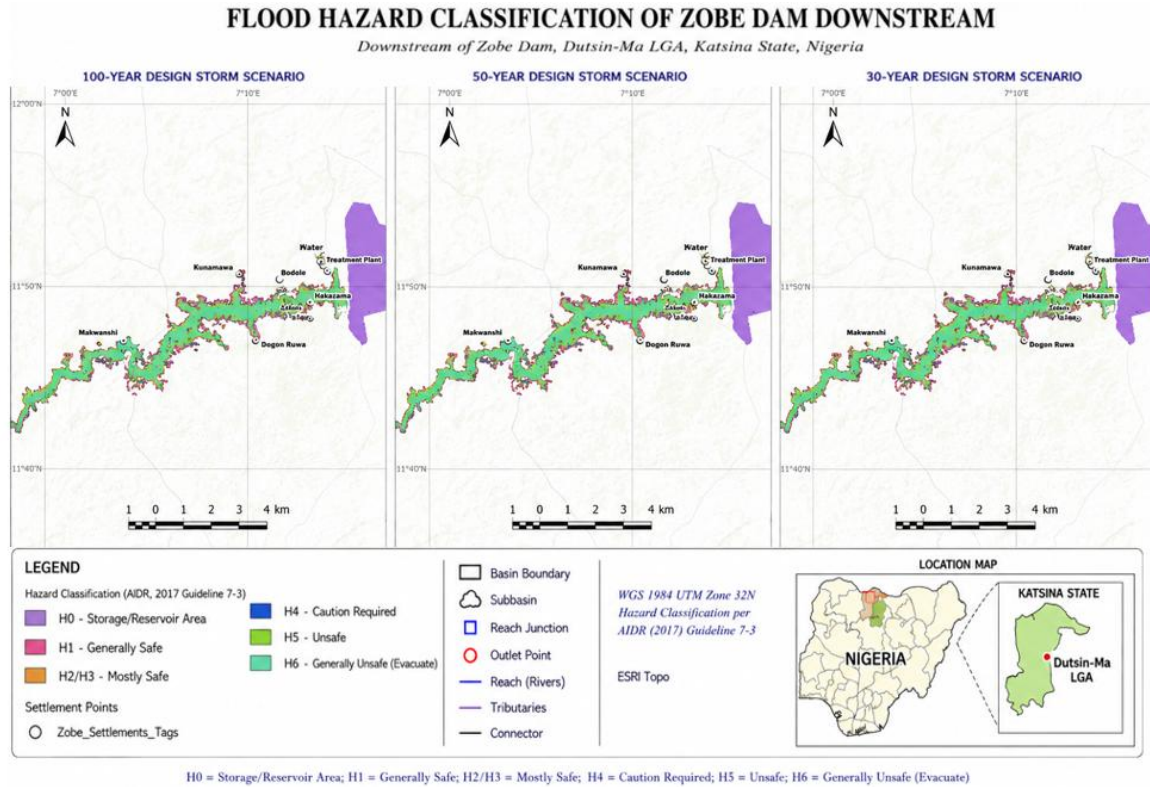


Figure 14: Flood hazard maps, 30-, 50-, and 100-year FSL-triggered design-storm scenarios (QGIS).

Maximum flood depth and velocity were extracted from RAS Mapper at eight downstream locations and are summarized in Table 8. Total inundated area and boundary perimeter, derived from the exported inundation-boundary polygon, are summarized in Table 9.

Location	30-yr (m)	D	30-yr (m/s)	V	50-yr (m)	D	50-yr (m/s)	V	100-yr (m)	D	100-yr (m/s)	V
Garhi	5.95		4.86		5.97		4.87		6.16		4.88	
Hakazama	3.31		0.66		3.34		0.68		3.52		0.86	
Safana	3.68		0.85		3.71		0.87		3.88		1.03	
Kunamawa	4.24		0.55		4.27		0.55		4.26		0.56	
DogonRuwa	2.62		0.15		2.64		0.15		2.83		0.15	
Makwanshi	0.50		0.50		0.60		0.51		0.74		0.56	
W.T.P.	Nil		nil		nil		nil		nil		nil	
Bodole	Nil		nil		nil		nil		nil		nil	

Return Period	Inundated Area (km ²)	Inundation Extend (km)
30-year	37.52	152.4
50-year	37.58	152.6
100-year	38.02	154.9

Maximum flood depth and velocity extracted from RAS Mapper at the eight identified downstream locations are presented in Table 8. Garhi recorded the highest hydraulic intensity across all three return periods, with maximum depth increasing from 5.95 m for the 30-year event to 6.16 m for the 100-year event, while velocity remained almost unchanged at 4.86–4.88 m/s. Hakazama and Safana also showed progressive increases in both depth and velocity with increasing return period. At Kunamawa and Dogon Ruwa, the changes were comparatively small, while Makwanshi showed an increase in maximum depth from 0.50 to 0.74 m and velocity from 0.50 to 0.56 m/s. The WTP and Bodole were not intersected by the simulated inundation extent in any of the three scenarios.

The resulting flood footprint is summarized in Table 9. Inundated area increased from 37.52 km² for the 30-year event to 37.58 km² for the 50-year event and 38.02 km² for the 100-year event.

Over the same return-period range, the flood extent length increased from 152.4 to 154.9 km. Thus, the overall spatial extent of flooding changed only slightly across the three scenarios, with the largest increase occurring between the 50- and 100-year events.

The eight locations represent named settlements and facilities identifiable from the available spatial data and are used here to characterize hydraulic conditions at selected downstream locations. They do not constitute an exhaustive inventory of all settlements within the simulated flood extent. The spatial limitation of the settlement-level assessment is discussed further in Section 4.5.

3.5 Flood Hazard Classification

Depth-velocity products at the eight assessed locations were classified using the AIDR (2017) Guideline 7–3 thresholds described in Section 2.7, with the resulting hazard classes presented in Table 10. Locations falling within the combined H2/H3 range are reported as H2/H3.

Location	30-year Class	50-year Class	100-year Class
<i>Garhi</i>	<i>H6</i>	<i>H6</i>	<i>H6</i>
<i>Hakazama</i>	<i>H5</i>	<i>H5</i>	<i>H5</i>
<i>Safana</i>	<i>H5</i>	<i>H5</i>	<i>H5</i>
<i>Kunamawa</i>	<i>H5</i>	<i>H5</i>	<i>H5</i>
<i>Dogon Ruwa</i>	<i>H2/H3</i>	<i>H2/H3</i>	<i>H2/H3</i>
<i>Makwanshi</i>	<i>H2/H3</i>	<i>H2/H3</i>	<i>H2/H3</i>
<i>W.T.P.</i>	<i>Not inundated</i>	<i>Not inundated</i>	<i>Not inundated</i>
<i>Bodole</i>	<i>Not inundated</i>	<i>Not inundated</i>	<i>Not inundated</i>

Hazard classification remained unchanged across the 30-, 50-, and 100-year scenarios at all assessed locations. Garhi was classified as H6 in all three scenarios, representing the highest hazard category recorded in the assessment. Hakazama, Safana, and Kunamawa consistently fell within H5, while Dogon Ruwa and Makwanshi remained within the combined H2/H3 category. The WTP and Bodole were not inundated in any scenario and were therefore not assigned a hazard class. However, these locations are not the exhaustive inventory settlements within the study area. They represent the names and facilities that could be identified from the available spatial data. Other households, structures and smaller settlements located within or immediately adjacent to the mapped hazard zones would be expected to experience conditions corresponding to their respective positions on the hazard maps, even where they are not individually labelled. The downstream road corridor toward Kankara, which comes from Dutsin-Ma town, passes near Garhi, also falls within the affected area; given Garhi's H6 classification, flooding could severely disrupt road access and vehicles movement, with implications for evacuation, emergency response, and local economic activities.

The stability of the hazard classes across the three return periods reflects the relatively limited variation in flood depth and velocity observed between scenarios (Section 3.4). Despite the increase in design-storm magnitude, none of the assessed locations crossed a hazard-class threshold.

3.6 Sensitivity Analysis

Sensitivity analyses were conducted to assess the influence of selected hydrologic and hydraulic parameters on the modeled response. For HEC-HMS, the effects of channel Manning's roughness and the assumed channel geometry on routed discharge were examined. For HEC-RAS, sensitivity was assessed for floodplain Manning's roughness and breach formation time, while the remaining model parameters were held constant.

3.6.1 HEC-HMS Sensitivity

Changing channel Manning's roughness from $n=0.035$ to $n=0.040$ produced peak routed discharges of 3,921.0 and 3,944.1 m^3/s for R1, respectively, while the R2 peak remained 3,069.5 m^3/s for both cases. The corresponding change in R1 peak discharge was approximately 0.59%.

A $\pm 20\%$ variation in the adopted channel geometry produced Sink-1 peak discharges of 4,773.7, 4,773.9, and 4,773.5 m^3/s for the -20% , base, and $+20\%$ cases, respectively. The results indicate very limited sensitivity of the routed peak discharge to the tested changes in channel roughness and geometry.

3.6.2 HEC-RAS Sensitivity

Floodplain (2D) Manning's n . Because the 2D floodplain used in the HEC-RAS hazard-mapping simulations directly affect the depth-velocity hazard classification, the roughness was increased from $n=0.035$ base value to $n=0.040$ for the 30-, 50-, and 100-year scenarios. The resulting changes in depth and velocity varied among locations (Table 11) as compared, but the hazard classifications remained unchanged for the 30- and 50-year scenarios. In the 100-year scenario, Safana changed from H5 (base: $D=3.88\text{m}$, $V=1.03\text{m/s}$, $DxV=4.00$) to H6 (alternate: $D=4.10\text{m}$, $V=1.08\text{m/s}$, $DxV=4.43$), crossing the H5/H6 threshold; while the other assessed locations (Table 11) although, with increasing depths and velocity but retained their base-case classifications. This indicates that although the overall hazard pattern is relatively stable, locations close to classification thresholds may respond to changes in floodplain roughness.

Breach formation-time sensitivity was assessed using formation times of 2.0, 3.73, and 5.0 h. The corresponding peak breach discharges were 2,601.60, 2,685.35, and 2,764.74 m^3/s , respectively. The difference between the extreme cases was approximately 6.3%, indicating a relatively limited change in peak breach discharge within the tested formation-time range.

4. DISCUSSION

The result of the simulations shows that increasing the design-storm return period increased the hydrologic and breach responses, including flood depth and velocity, but did not change the resulting downstream hazard classification. Peak basin-outlet inflow increased from 4,616.3 m^3/s for the 30-year event to 5,333.4 m^3/s for the 100-year event (15.53%), while peak breach discharge increased from 2,377.24 to 2,685.35 m^3/s (12.96%). The relatively limited increase in breach discharge reflects the FSL-triggered “sunny day” breach adopted in this study, where breach initiation occurs at a fixed reservoir elevation rather than at a storm-dependent peak stage. Despite the increase in flood magnitude, the hazard pattern remained unchanged across the three scenarios. Garhi remained H6, Hakazama, Safana, and Kunamawa remained H5, while Dogon Ruwa and Makwanshi remained within the combined H2/H3 class. The Water Treatment Plant and Bodole were not intersected by the modeled inundation extent in any scenario. Thus, under the assumed FSL-triggered piping failure, increasing the design-storm return period increased flood magnitude but was not sufficient to change the hazard class at the assessed locations.

The limited increase in breach discharge can be explained by the reservoir response following FSL-triggered failure. Reservoir stage increased only from 495.13 m to 495.61 m between the 30- and 100-year scenarios, despite the 22% increase in peak inflow. Once the breach developed, the additional outflow limited further reservoir accumulation. Consequently, the hydraulic head above the breach invert remained within approximately 5.7–6.1 m, resulting in only a 13% increase in peak breach discharge. This shows that, for the failure condition adopted here, the increase in storm magnitude did not translate directly into a similar increase in breach discharge.

The breach parameters also contribute to the resulting hydraulic response. The Froehlich (2008) formulation used in this study gave a breach width of 119 m and a formation time of 3.73 h. As noted by Froehlich (2008); (Zhong et al., 2021) breach characteristics introduce uncertainty into predicted dam-break discharge and downstream flood conditions (Azeez et al., 2020). The simulated breach discharges should therefore be considered within the context of the assumed breach geometry, formation time, and FSL-based initiation condition rather than as a prediction of an actual failure discharge.

Although the increase in breach discharge was relatively modest, Garhi experienced the most severe hydraulic conditions. This indicates that downstream hazard is not controlled by peak discharge alone, but also by the local terrain and the depth and velocity of flow at the exposed settlements (Peker et al., 2024; AIDR, 2017). The unchanged hazard classes across the three scenarios are therefore important from an emergency-planning perspective. Garhi remained the most critical assessed location, while Hakazama, Safana, and Kunamawa consistently remained within H5 and would also require appropriate preparedness measures.

The assessment was based on the named settlements and facilities that could be identified from the available spatial data and does not represent an exhaustive inventory of downstream settlements. Other settlements, buildings, and households located within the same mapped hazard

zones may experience similar conditions depending on their position within the floodplain. The road corridor towards Kankara is also important because flooding around Garhi could disrupt vehicle movement and emergency access. This could affect evacuation and response activities and may also interrupt the movement of agricultural produce, goods, and other services along the corridor. These effects should therefore be considered in downstream emergency and water-resources management planning.

The sensitivity analysis indicates that the hydrologic response was relatively stable to the tested channel roughness and geometry assumptions, with only minor changes in routed peak discharge. This suggests the variation lies within the numerical precision of the simulation rather than reflecting on systematic geometric effect; therefore, peak inflow at the basin outlet is accordingly insensitive to channel cross-sectional width within the tested range. However, in the absence of surveyed channel sections, the adopted cross-sectional dimensions nonetheless remain approximate and are documented here as a source of structural, if empirically minor, uncertainty in the Muskingum-Cunge routing.

The hydraulic results showed greater sensitivity to floodplain roughness, particularly at locations close to hazard-class thresholds. The change in Safana from H5 to H6 under the 100-year roughness sensitivity case demonstrates that individual hazard classifications can be affected by hydraulic parameter uncertainty, even though the broader downstream hazard pattern remained similar. The sensitivity of breach discharge to formation time was comparatively limited within the tested range. These findings are consistent with previous dam-breach studies that identify breach formation parameters as important sources of uncertainty in predicted downstream flows and water levels (Wahl, 1998; Froehlich, 2008).

Generally, the results show that, under the assumed FSL-triggered piping failure, the breach initiation condition and reservoir response had a greater influence on the variation in breach discharge than the difference between the three design-storm return periods. The stable hazard pattern provides a useful basis for identifying persistent downstream priorities for emergency preparedness, evacuation planning, and access management.

5. CONCLUSION

This study assessed the consequences of a hypothetical piping failure of Zobe Dam under 30-, 50-, and 100-year design-storm conditions using coupled HEC-HMS and 2D HEC-RAS modelling, with flood-hazard mapping undertaken in QGIS. The modeled peak inflow increased from 4,616.3 m³/s for the 30-year event to 5,333.4 m³/s for the 100-year event, while peak breach discharge increased from 2,377.24 to 2,685.35 m³/s. Despite the increase in design-storm magnitude, the resulting flood hazard classification remained unchanged across the three scenarios. Garhi was consistently classified as H6, while Hakazama, Safana, and Kunamawa were classified as H5, and Dogon Ruwa and Makwanshi as H2/H3. The Water Treatment Plant and Bodole remained outside the modeled flood extent. Under the assumed Full Supply Level-triggered piping failure, therefore, increasing the return-period increased the flood magnitude but did not change the identified pattern of downstream hazard. Although, sensitivity analyses showed limited variation in HEC-HMS routed discharge and breach discharge within the tested parameter ranges, but floodplain roughness affected individual hydraulic conditions and resulted in a change from H5 to H6 at Safana under the 100-year sensitivity case.

These results identify Garhi and the downstream road corridor that linked Dutsin-Ma toward Kankara as important priorities for flood-warning, evacuation, and emergency-access planning. The potential disruption of this corridor, together with the hazard classification of the affected settlements, has implications for regional water-resources management, especially the continuity of transport, emergency response, and movement of people and goods during a dam-failure event.

Data Availability

The digital elevation model used in this study is publicly available from the United States Geological Survey's EarthExplorer portal (earthexplorer.usgs.gov). Dam-specific geometric and hydrological data were obtained from the Sokoto-Rima River Basin Development Authority and are available from the corresponding author upon reasonable request and with the permission of that authority.

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