



DAM BREAK ANALYSIS OF SOMASILA DAM USING HEC-RAS

¹K. Bhaskar, ²Dr. P. Mynar Babu

¹Post Graduate Student (M. Tech, Water Resources Engineering), ²Assistant Professor(c)

^{1,2}Department of Civil Engineering

^{1,2}University College of Engineering Kakinada (Autonomous), JNTUK, Kakinada, Andhra Pradesh, India

Abstract: Large storage dams bring irrigation, drinking water and flood moderation, yet an uncontrolled release from them can devastate settlements lying downstream. This paper presents a hypothetical breach study of Somasila Dam, an earth-cum-masonry dam on the Penna (Pennar) River in SPSR Nellore district of Andhra Pradesh, carried out with the two-dimensional unsteady module of HEC-RAS. A terrain model built from a 30 m DEM, roughness values assigned from a 10 m land-use/land-cover map, and a reservoir elevation–volume curve were combined with the design flood hydrograph supplied by the state water resources department. The reservoir was assumed to stand at Full Reservoir Level (+100.58 m) with all spillway gates jammed shut. Breach geometry and formation time were derived from five regression approaches (Froehlich 2008, Froehlich 1995, Von Thun and Gillette, Xu and Zhang, and MacDonald and Langridge-Monopolis), and both overtopping and piping modes were run for each, giving ten simulations. Breach outflow peaked at about 97,649.44 m³/s for Froehlich (1995) under overtopping and at 76,355.35 m³/s for the same method under piping. At four reference locations downstream, maximum depths of 16.02 m (overtopping, MacDonald et al.) and 13.70 m (piping, Xu and Zhang) and maximum velocities of 3.45 m/s and 4.26 m/s were obtained, while the earliest arrival of the flood front was only 1.05 h after breaching. The outcomes show that the choice of breach-prediction method and failure mode alters depth, velocity and warning time appreciably, and the inundation maps identify ten villages, led by Somasila, Kammavaripalle and Uppalapadu, as the most exposed.

IndexTerms - Dam break analysis, Somasila Dam, HEC-RAS 2D, breach parameters, overtopping, piping, inundation mapping.

I. INTRODUCTION

Dams are built to hold back river flow so that water can be used for irrigation, municipal supply, power and flood regulation. The same stored volume becomes a hazard if the structure fails, because a sudden release sends a steep flood wave along the valley within hours. Records of past failures around the world show heavy loss of life, damage to roads, canals and buildings, and long-lasting loss of farmland. For this reason, dam safety programmes require a rational estimate of what would happen downstream before an emergency actually occurs [1], [2].

Dam break analysis answers that question in a sequence of steps. A credible failure mechanism is first chosen, the size and speed of breach growth are estimated, the resulting outflow hydrograph is calculated, and the flood wave is routed through the downstream terrain to obtain depth, velocity, arrival time and flooded area [2]. The products of such a study, particularly inundation maps and travel-time estimates, feed directly into emergency action plans, evacuation planning and land-use control in the valley below the dam.

The Hydrologic Engineering Center's River Analysis System (HEC-RAS), distributed by the U.S. Army Corps of Engineers, is among the most widely applied tools for this purpose. From version 5 onward it solves unsteady flow on a two-dimensional computational mesh, couples storage areas to 2D flow areas through hydraulic connections, and lets the analyst describe a breach by overtopping or piping, so that the entire chain from reservoir to floodplain can be modelled in one environment [1]. Its results can be mapped directly in RAS Mapper without a separate GIS post-processing step.

Somasila Dam impounds one of the largest reservoirs in the Pennar basin and supports an irrigated area of several hundred thousand acres, supplies drinking water, and passes Krishna water on towards the Kandaleru reservoir. Its failure would therefore be a regional emergency, which makes a quantified breach assessment for the dam both relevant and overdue. The present paper reports such an assessment and, in addition, compares how five different breach-prediction methods change the flood that is simulated.

II. OBJECTIVES OF THE PRESENT STUDY

The study is organised around the following aims:

- To construct a 2D HEC-RAS model of the Somasila reservoir and the river corridor and floodplain below the dam.
- To simulate dam failure under both overtopping and piping modes, with breach properties taken from five empirical regression methods.

- (c) To obtain the breach outflow hydrographs and the maximum flood depth, flow velocity, water-surface elevation and flood arrival time at selected downstream locations.
- (d) To delineate the areas and villages likely to be inundated, and to compare the methods and failure modes so that their influence on flood hazard is clear.
- (e) To provide hazard information useful for emergency preparedness and flood-risk management downstream of the dam.

III. LITERATURE REVIEW

Reviews of the subject describe dam break modelling as a chain of linked decisions, in which the selected failure mechanism, the estimated breach dimensions and the downstream routing method each add uncertainty to the final flood map [2]. Overtopping failure begins with water running over the crest and cutting the downstream face, whereas piping begins with internal erosion that opens a conduit through the embankment; the two produce differently shaped outflow hydrographs [2]. Peramuna et al. [3] surveyed numerical approaches to flood-wave propagation and noted the growing preference for 2D schemes, because the wave can spread laterally over the floodplain instead of being confined to a pre-drawn channel; they also stressed that terrain accuracy, roughness and boundary conditions govern the reliability of predicted depths and velocities.

Because the dimensions of a future breach cannot be known, analysts rely on regression equations fitted to recorded failures. Catterick et al. [4] compared the commonly used empirical models and cautioned that their ranges of applicability depend on dam type, reservoir size, head and failure mechanism, so that the selected equation can noticeably change the hydrograph. Wahl [5] reached a similar view by quantifying the scatter in breach-width and failure-time predictions. These findings motivate the multi-method comparison adopted here.

Several Indian studies have applied HEC-RAS with GIS support to individual dams. Bharath et al. [6] combined HEC-RAS and HEC-GeoRAS to analyse Hidkal Dam in Karnataka for piping and overtopping and examined the sensitivity of the results to breach width and formation time. Sharma and Mujumdar [7] modelled Ajwa Reservoir through an ArcGIS–HEC-GeoRAS–HEC-RAS chain and demonstrated how hydraulic output is turned into floodplain maps. Ramola et al. [8] studied Pulichintala Dam in Andhra Pradesh, tested the influence of Manning's roughness, breach width, formation time and the probable maximum flood, and reported peak discharges of roughly 121,369 m³/s at the dam and 84,043 m³/s some 85 km downstream, illustrating attenuation of the wave by channel and floodplain storage. The Central Water and Power Research Station has prepared dam break and emergency-action inputs for several Telangana reservoirs, including Konda Pochamma Sagar, Gandhamalla and Kurumurthyra [9]–[11], using 2D modelling to show how floodwater spreads over the terrain.

International applications point in the same direction. Dam break and inundation studies with HEC-RAS have been reported for the Old Aswan Dam on the Nile [12], Sefid-Roud Dam in Iran [13], a hypothetical failure used to illustrate risk management [14], Kibimba Dam in Uganda [15], Drăcșani Lake in Romania, where breach-size choices were compared [16], Kalyani Dam in India [17] and Kenyir Dam in Malaysia [18]. Across these works, HEC-RAS repeatedly proves capable of reproducing flood-wave travel, but most studies adopt one breach method. What is less common is a side-by-side evaluation of five regression approaches under both failure modes for a large Indian earth dam, and that is the gap this paper addresses for Somasila Dam.

IV. METHODOLOGY

A. Study Area and Project Details

Somasila Dam is located across the Penna River near Somasila village in Ananthasagaram Mandal of SPSR Nellore district, Andhra Pradesh, at about 14°29'15" N and 79°18'25" E (Fig. 1). The project, begun in the mid-1970s and finished in 1989, controls a catchment of 48,645 km². The main dam is a zoned earth-cum-masonry structure 352 m long with an 18 m wide top at +105.95 m and a maximum height of about 38 m above ground. The reservoir has a Full Reservoir Level (FRL) of +100.58 m, a Maximum Water Level of +101.80 m, a minimum drawdown level of +82.30 m and a gross capacity of about 78 TMC, with a water-spread of 212.3 km² at FRL. An ogee spillway of 236.21 m length, controlled by 12 radial gates of 15.24 m × 14.17 m, can pass 19,680 m³/s at FRL and 22,375 m³/s at MWL (Table I).

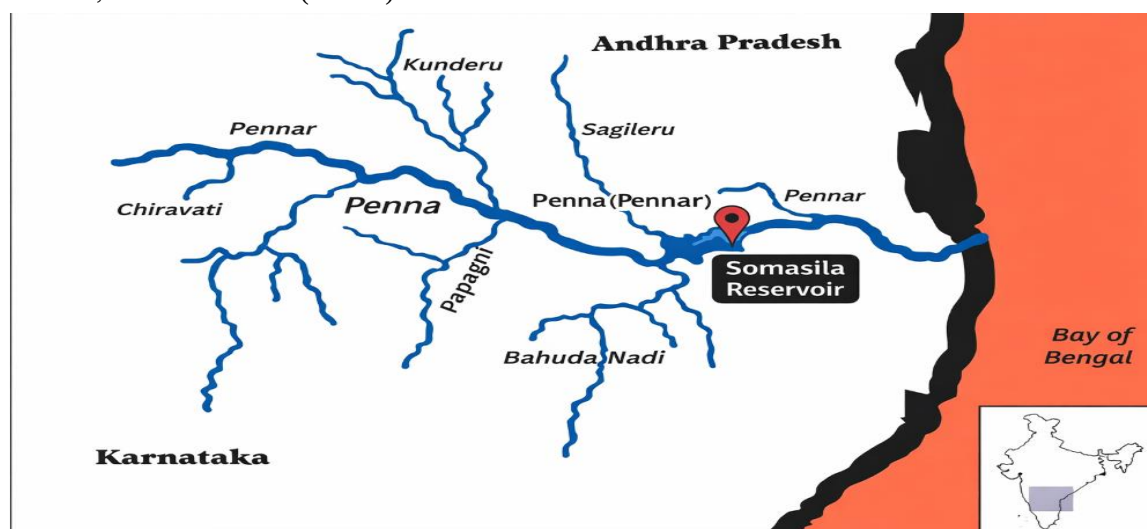


Fig. 1. Location of Somasila Dam on the Penna (Pennar) River, Andhra Pradesh.

Table I. Principal features of Somasila Dam and reservoir

Parameter	Value
Type of dam	Earth-cum-masonry
Length / top width / top level	352 m / 18 m / +105.95 m
Full Reservoir Level / Maximum Water Level	+100.58 m / +101.80 m
Minimum drawdown level (dead storage level)	+82.30 m
Gross storage at FRL	≈ 78 TMC (live storage ≈ 70.4 TMC)
Water-spread area at FRL	212.285 km ²
Spillway	Ogee, 236.21 m long, 12 radial gates
Spillway capacity at FRL / MWL	19,680 m ³ /s / 22,375 m ³ /s
Catchment area at dam site	48,645 km ²

B. Input Data

Terrain was represented by a 30 m DEM downloaded from the Bhoonidhi portal of the Government of India and clipped to the study window in ArcGIS (Fig. 2). Surface roughness was derived from the 10 m Esri land-use/land-cover product for 2023, which separates crops, water, trees, flooded vegetation, built area, bare ground and grassland (Fig. 3). Manning's n was assigned class by class: crops 0.055, water 0.028, trees 0.100, flooded vegetation 0.050, built area 0.065, bare ground 0.030 and grassland 0.050, so that every mesh cell carries a roughness value. The reservoir is represented by the published elevation–volume curve. Inflow to the reservoir is the flood hydrograph furnished by the Water Resources Department (Chief Engineer, SDSO Hydrology, Vijayawada), whose peak is about 34,816 m³/s.

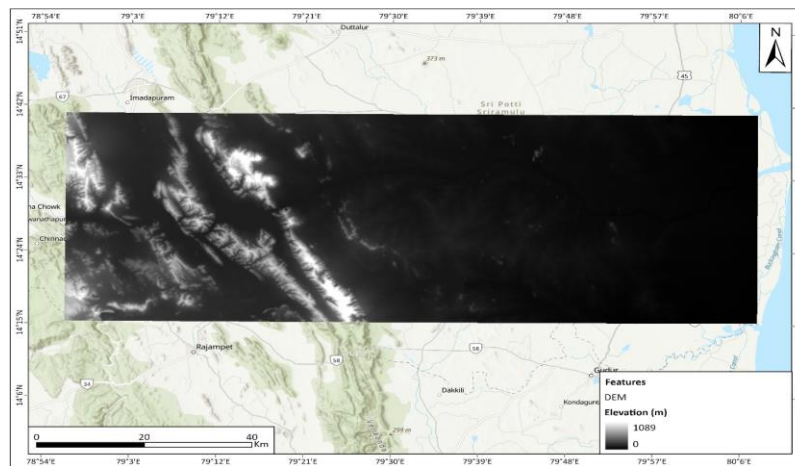


Fig. 2. Digital elevation model of the study area.

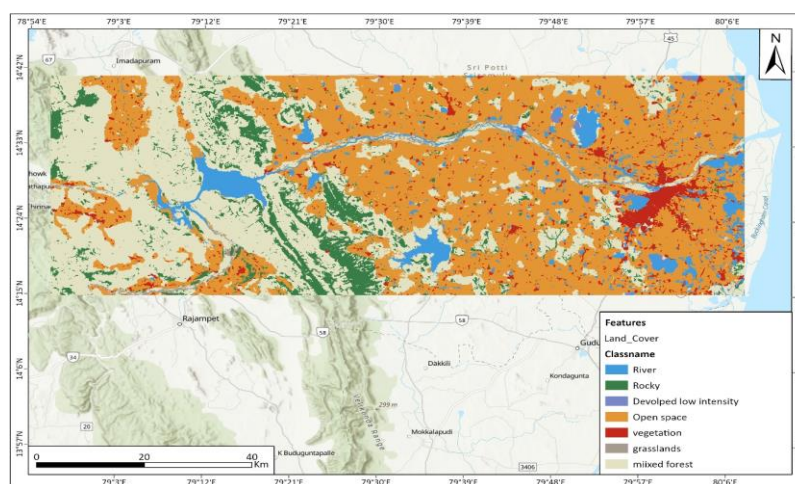


Fig. 3. Land-use/land-cover map used to assign Manning's roughness.

C. HEC-RAS 2D Model Set-up

The terrain and land-cover layers were imported through RAS Mapper. A storage area enclosing the reservoir was drawn and given the elevation–volume relation, and a separate 2D flow area was digitised for the valley below the dam. The dam was traced as a storage-area-to-2D-area connection from left to right, and its crest profile, width and weir coefficient were entered in the geometry editor. A computational mesh of 50×50 cells was generated over the downstream flow area. Reference points were placed along the river at four stations, designated Area 1 to Area 4 from nearest to farthest, to record depth, velocity, water-surface elevation and arrival time (Fig. 4). The upstream boundary is the reservoir inflow hydrograph, and the downstream boundary is a normal-depth condition (value 0.08) applied along a boundary-condition line just outside the 2D area.

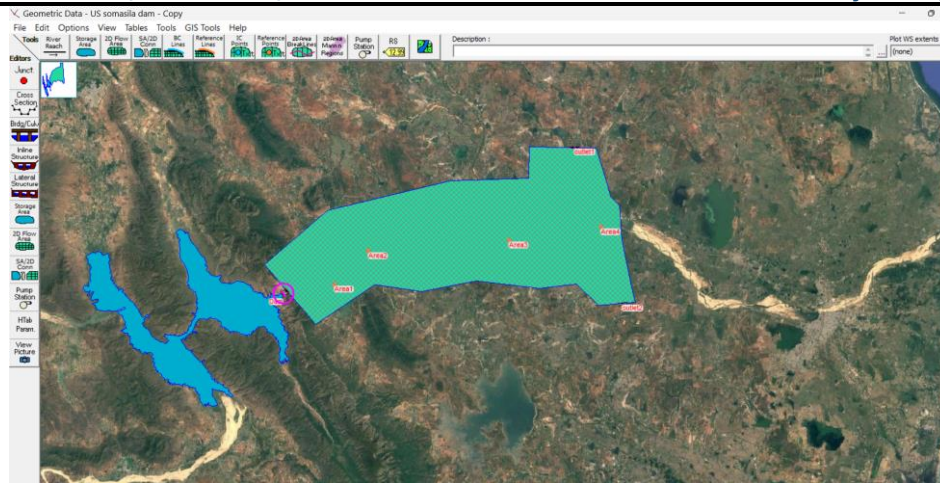


Fig. 4. Model geometry showing reservoir storage area, dam connection and downstream 2D flow area.

D. Failure Scenarios and Breach Hydraulics

The adopted scenario places the reservoir at FRL +100.58 m while the maximum recorded flood of 19,680.21 m³/s arrives with every spillway gate inoperable, a deliberately severe but plausible combination. Two failure modes were examined. In piping, erosion opens a circular conduit through the embankment, treated as orifice flow, which turns into a growing trapezoidal opening governed by weir flow once the top of the dam is reached. The orifice discharge is

$$Q = C A \sqrt{2gH} \quad (1)$$

where Q is discharge, C the orifice coefficient, A the flow area of the opening, g gravitational acceleration and H the energy head measured from the centreline of the opening. In overtopping, a notch starts at the crest and widens into a trapezoid whose flow follows the broad-crested weir relation

$$Q = C_1 b H^{1.5} + C_2 m H^{2.5} \quad (2)$$

in which b is the bottom width of the breach, m its side slope (horizontal to vertical), H the upstream head over the breach invert, and $C_1 = 1.7$ and $C_2 = 1.35$ the coefficients for the rectangular and triangular parts of the section [24]. Breach growth was prescribed with the sine-wave progression, which opens the breach quickly at first and more slowly as it approaches its final size, in preference to a linear development. The “user-entered data” option of HEC-RAS was used, with failure location, mode, bottom elevation, width, side slopes and formation time supplied from the regression methods described next.

E. Empirical Breach-Parameter Methods

Five regression approaches were used to fix the final breach dimensions and the formation time. Froehlich (1995) [19] fitted equations to 63 embankment failures, and Froehlich (2008) [20] updated them with 74 cases. In both, the average breach width is a power function of reservoir volume at failure (V_w) and breach height (h_b), with a factor K_0 that is larger for overtopping. The 2008 version is

$$B_{ave} = 0.27 K_0 V_w^{0.32} h_b^{0.04}, \quad t_f = 63.2 \sqrt{(V_w / (g h_b^2))} \quad (3)$$

with $K_0 = 1.3$ for overtopping and 1.0 for piping, recommended side slopes of 1.0H:1V (overtopping) and 0.7H:1V (piping), and t_f in seconds. The 1995 expressions use $K_0 = 1.4$ for overtopping, side slopes of 1.4H:1V and 0.9H:1V respectively, and give the formation time in hours. Von Thun and Gillette [21] relate the average width to the water depth above the breach bottom through $B_{ave} = 2.5 h_w + C_b$, where C_b depends on reservoir size, and give separate formation-time relations for erosion-resistant and easily eroded dams. Xu and Zhang [22] built their equations on a database of 182 earth and rockfill dams, of which 45 and 28 cases supported the width and time relations respectively. MacDonald and Langridge-Monopolis [23] derived the eroded volume and failure time from the breach formation factor, the product of the outflow volume and the water depth above the final breach invert; for earthfill dams

$$V_{eroded} = 0.0261 (V_{out} h_w)^{0.769}, \quad t_f = 0.0179 V_{eroded}^{0.364} \quad (4)$$

with a side slope of about 0.5H:1V. Following the guidance on these methods, breach height was taken as the full height from the crest to natural ground, and the Xu and Zhang formation-time equation, which includes initiation and post-erosion phases that HEC-RAS does not count as breach time, was not used for timing.

F. Flood Routing and Mapping

The breach outflow was routed over the 2D mesh with the diffusion-wave form of the unsteady equations available in HEC-RAS. Maximum depth, velocity and water-surface elevation grids, together with arrival-time maps, were exported from RAS Mapper, and inundation extents were overlaid on village locations to identify settlements reached by the flood.

V. RESULTS AND DISCUSSIONS

A. Breach Outflow Hydrographs

Fig. 5 and Fig. 6 compare the breach outflow produced by the five methods. Under overtopping, Froehlich (1995) gives the largest peak, 97,649.44 m³/s, followed by Froehlich (2008) at 85,552.87 m³/s; MacDonald et al. reaches about 76,355 m³/s and Xu and Zhang about 74,639 m³/s, the latter only near the end of the simulated window, while Von Thun and Gillette stays lower and starts later. Under piping, the two Froehlich methods again produce the fastest-rising hydrographs, with peaks of 74,199 m³/s at 06:00 h (2008) and 76,355 m³/s at 07:00 h (1995) on 1 September 2019, whereas the Von Thun and Gillette and MacDonald et al. equations release at most about 37,300 m³/s and Xu and Zhang about 54,373 m³/s, each only after the breach starts to release water 14 to 17 hours into the simulation (Table II). A delayed or slowly growing breach therefore means a much smaller, later peak.

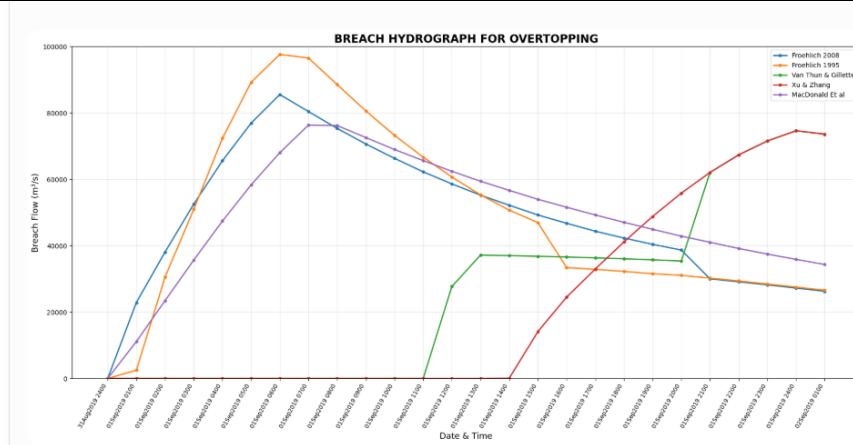


Fig. 5. Breach outflow hydrographs for overtopping failure.

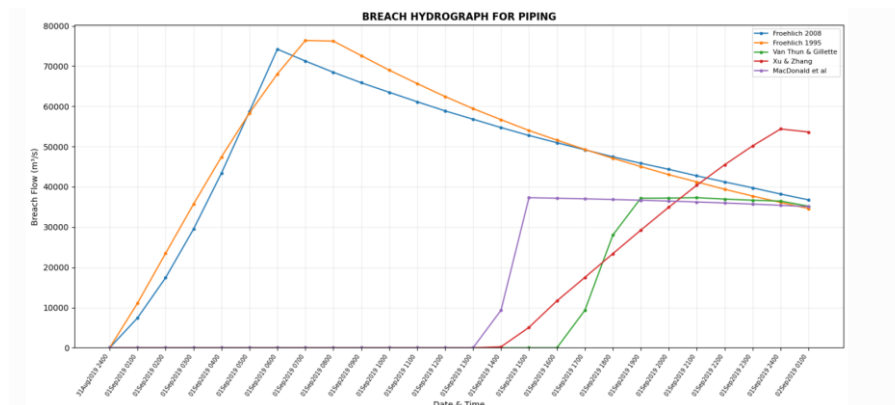


Fig. 6. Breach outflow hydrographs for piping failure.

Table II. Peak breach outflow for piping failure from the five methods

Method	Peak outflow (m³/s)	Time of peak
Froehlich (2008)	74,199.28	01 Sep 2019, 06:00
Froehlich (1995)	76,355.35	01 Sep 2019, 07:00
Von Thun and Gillette	37,302.9	01 Sep 2019, 21:00
Xu and Zhang	54,372.93	01 Sep 2019, 24:00
MacDonald et al.	37,302.90	01 Sep 2019, 15:00

B. Flood Characteristics at Downstream Locations

Table III lists the maximum depth and velocity at the four reference locations. For the overtopping runs, the deepest flooding occurs at the first station, 16.02 m for MacDonald et al. and 15.70 m for Xu and Zhang, and depth then falls steadily toward Area 4 for most methods, matching the spreading and flattening of the wave. The Froehlich (2008) overtopping run illustrates the pattern: depth drops from 10.18 m at Area 1 to 2.61 m at Area 4, velocity from 3.41 to 0.84 m/s and water-surface elevation from 74.19 m to 43.53 m. The Froehlich (1995) overtopping run is less regular, because 9.22 m of depth is recorded at Area 4 even though the intermediate stations are shallower, which probably reflects the local terrain at that station.

Table III. Maximum flood depth (m) and velocity (m/s) at the reference locations

Method	Area	Depth, overtopping	Depth, piping	Velocity, overtopping	Velocity, piping
Froehlich (2008)	1	10.18	8.60	3.41	1.45
	2	7.42	9.83	1.85	3.17
	3	4.48	10.19	1.59	2.81
	4	2.61	9.71	0.84	0.75
Froehlich (1995)	1	10.82	13.07	3.40	4.26
	2	7.42	9.67	1.91	1.82
	3	6.01	8.86	2.27	2.97
	4	9.22	10.18	0.62	2.15
Von Thun and Gillette	1	12.53	8.98	2.27	3.46
	2	8.84	8.61	3.45	2.06
	3	7.22	7.50	1.72	1.53
	4	4.26	4.19	0.85	0.78

Method	Area	Depth, overtopping	Depth, piping	Velocity, overtopping	Velocity, piping
Xu and Zhang	1	15.70	13.70	2.55	2.16
	2	12.10	10.26	2.12	2.20
	3	8.21	8.28	1.87	1.71
	4	3.57	0 (not reached)	1.23	0 (not reached)
MacDonald et al.	1	16.02	3.55	2.73	1.47
	2	11.47	7.20	2.69	2.19
	3	9.30	4.66	2.14	1.07
	4	10.80	2.16	1.84	0.27

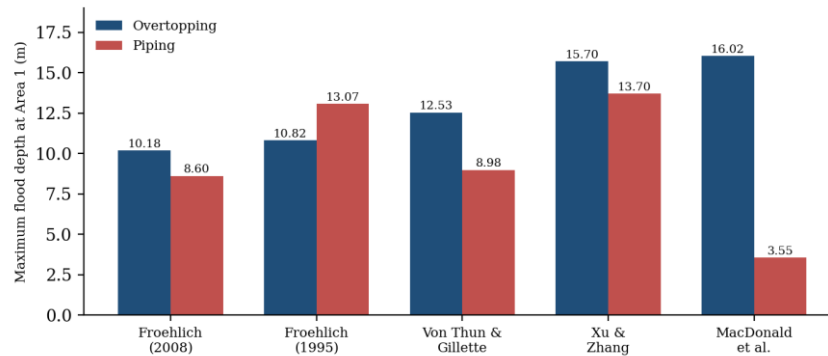


Fig. 7. Maximum flood depth at Area 1 for each breach method and failure mode.

Fig. 7 shows how strongly the predicted depth at the nearest station depends on the method. Among overtopping runs the span is from 10.18 m to 16.02 m, and among piping runs it is from 3.55 m to 13.70 m, so the choice of equation alone can change the flood depth at one location by a factor of nearly four. Velocities lie mostly between about 0.3 and 4.3 m/s. The highest, 4.26 m/s, appears at Area 1 for piping with Froehlich (1995), and the highest overtopping velocity, 3.45 m/s, appears at Area 2 for Von Thun and Gillette. In general, velocities are greatest in the first two stations, where the valley is narrow and the discharge largest. The mode comparison is also not uniform: with Froehlich (2008), piping gives deeper water than overtopping at Areas 2, 3 and 4 (by 2.41 m, 5.71 m and 7.10 m), while MacDonald et al. predicts higher overtopping depths at every station, by 12.47 m, 4.27 m, 4.64 m and 8.64 m. Both depth and velocity should therefore be read together when judging hazard.

C. Flood Arrival Time

Warning time depends mainly on how quickly the breach develops. For overtopping, the flood front reaches Area 1 after between 7.03 h (Froehlich 1995) and 15.18 h (Xu and Zhang), and Area 4 after 16.30 to 23.37 h. The piping scenario with Froehlich (2008) is the most critical for evacuation, since the front arrives at Area 1 only 1.05 h after the start of breaching and at Area 4 after 9.19 h; the other piping runs give 8.16 to 15.17 h at Area 1 (Table IV). In the Xu and Zhang piping run the flood did not reach Area 4 at all during the simulated period. Such short lead times at the nearest settlements show that early-warning and evacuation arrangements for the first few kilometres below the dam must be decided at the dam itself and cannot rely on downstream gauges.

Table IV. Flood arrival time (h) at the nearest (Area 1) and farthest (Area 4) stations

Method	Overtopping, Area 1	Overtopping, Area 4	Piping, Area 1	Piping, Area 4
Froehlich (2008)	9.21	23.11	1.05	9.19
Froehlich (1995)	7.03	16.30	8.16	20.11
Von Thun and Gillette	14.22	22.32	14.28	22.41
Xu and Zhang	15.18	23.37	15.17	not reached
MacDonald et al.	8.18	20.36	9.32	23.11

D. Inundation Extent and Affected Villages

The inundation maps from all ten runs show the flood following the Penna valley below the dam and spreading on to the adjoining low-lying land, with the widest flooding on the flat ground towards the downstream end. Taking the runs together, ten villages emerge as the most severely affected: Somasila, Kammavaripalle, Uppalapadu, Rajupalem, Venkatareddipalle, Bedusupalle, Kachidevarayapalle, Devarayapalle, Koturu and Bandarupalle. Most of them lie close to the river in the first reaches below the dam, where the model gives the deepest water and highest velocities, and where the earliest arrival times occur.

E. Discussion

Three points follow from the comparison. First, the scale of the outflow, whose peaks lie in the range of roughly 74,000 to 98,000 m³/s for the faster-developing breaches, is several times the spillway capacity of 22,375 m³/s, which shows how different a breach flood is from any operational release. Second, because the Froehlich (2008) equations rest on the larger data set and update the 1995 relations, and because their peaks are close to those of the other approaches, Froehlich (2008) was treated as the critical method for the dam; its piping run also gives the shortest warning time. Third, the divergence between methods in depth, velocity, peak time and arrival time confirms the uncertainty in breach prediction noted in the literature [4], [5]. A conservative emergency plan should therefore be based on an envelope of several methods and modes rather than a single run. The study has limits: the 30

m DEM and the single design flood restrict the resolution of local depths, no field data on past floods were available for calibration, and results at four reference stations have been used to describe a continuous flood surface.

VI. CONCLUSIONS

A 2D HEC-RAS model of Somasila Dam was used to simulate overtopping and piping failures with five empirical breach methods. Froehlich (1995) gave the largest outflow, 97,649 m³/s for overtopping and 76,355 m³/s for piping, while the maximum depth of 16.02 m (overtopping, MacDonald et al.) and 13.70 m (piping, Xu and Zhang) occurred at the nearest station and the highest velocity was 4.26 m/s. The earliest flood arrival, 1.05 h, was obtained for piping with Froehlich (2008). Results varied significantly between methods, so decisions on emergency planning should use an envelope of scenarios. The ten villages listed above should receive priority in evacuation planning, and future work may include finer terrain data, calibration against recorded floods and a sensitivity study on roughness and mesh size.

REFERENCES

- [1] G. W. Brunner, *HEC-RAS River Analysis System: Hydraulic Reference Manual*. Davis, CA, USA: U.S. Army Corps of Engineers, Hydrologic Engineering Center.
- [2] P. Pandya and R. Jain, "A literature review on dam break analysis," *Journal of Computational Analysis and Applications*, vol. 33, no. 6, pp. 1587–1593, 2024.
- [3] A. Peramuna et al., "Review on model development techniques for dam break flood wave propagation," *WIREs Water*, 2024, doi: 10.1002/wat2.1688.
- [4] S. B. Catterick, C. Iliadis and V. Glenis, "A review of empirical dam breach models," *River*, pp. 1–23, 2026, doi: 10.1002/rvr2.70065.
- [5] T. L. Wahl, "Uncertainty of predictions of embankment dam breach parameters," *Journal of Hydraulic Engineering*, vol. 130, no. 5, 2004.
- [6] A. Bharath, A. V. Shivapur, C. G. Hiremath and R. Maddamsetty, "Dam break analysis using HEC-RAS and HEC-GeoRAS: A case study of Hidkal Dam, Karnataka State, India," 2021.
- [7] P. Sharma and S. Mujumdar, "Dam break analysis using HEC-RAS and HEC-GeoRAS – a case study of Ajwa Reservoir," *Journal of Water Resources and Ocean Science*, vol. 5, no. 6, pp. 108–113, 2017, doi: 10.11648/j.wros.20160506.15.
- [8] Ramola et al., "Dam break analysis of Pulichintala Dam, Andhra Pradesh, using HEC-RAS," 2021.
- [9] Central Water and Power Research Station, "Dam break analysis and providing inputs for Emergency Action Plan for Konda Pochamma Sagar Reservoir, Kaleshwaram Project, Telangana," technical report.
- [10] Central Water and Power Research Station, "Dam break analysis and inputs for Emergency Action Plan for Gandhamalla Reservoir, Telangana," technical report.
- [11] Central Water and Power Research Station, "Dam break studies and emergency action planning for Kurumurthyra Reservoir, Telangana," technical report.
- [12] A. M. Helwa, M. H. Elgamal and A. H. Ghanem, "Dam break analysis of Old Aswan Dam on Nile River using HEC-RAS."
- [13] K. Khosravi, M. Rostaminejad, J. R. Cooper, L. Mao and A. M. Melesse, "Dam break analysis and flood inundation mapping: The case study of Sefid-Roud Dam, Iran."
- [14] A. P. Phyto, H. Yabar and D. Richards, "Managing dam breach and flood inundation by HEC-RAS modeling and GIS mapping for disaster risk management."
- [15] M. Kiwanuka et al., "Dam breach analysis of Kibimba Dam in Uganda using HEC-RAS and HEC-GeoRAS."
- [16] L.-M. Albu, A. Enea, M. Iosub and I.-G. Breabă, "Dam breach size comparison for flood simulations: A HEC-RAS based, GIS approach for Drăcșani Lake, Sitna River, Romania."
- [17] B. Balaji and S. Kumar, "Dam break analysis of Kalyani Dam using HEC-RAS."
- [18] F. Che Ros, L. M. Sidek, A. Z. A. Razad, N. N. Nik Ibrahim and M. N. Ahmad, "Dam break analysis for hydroelectric dam: Case study of Kenyir Dam, Terengganu, Malaysia," 2009.
- [19] D. C. Froehlich, "Embankment dam breach parameters revisited," in *Proc. Water Resources Engineering Conf.*, ASCE, 1995, pp. 887–891.
- [20] D. C. Froehlich, "Embankment dam breach parameters and their uncertainties," *Journal of Hydraulic Engineering*, vol. 134, no. 12, pp. 1708–1721, 2008.
- [21] J. L. Von Thun and D. R. Gillette, *Guidance on Breach Parameters*. Denver, CO, USA: U.S. Bureau of Reclamation, Mar. 1990.
- [22] Y. Xu and L. M. Zhang, "Breaching parameters for earth and rockfill dams," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 135, no. 12, pp. 1957–1970, 2009.
- [23] D. MacDonald and J. Langridge-Monopolis, "Breaching characteristics of dam failures," *Journal of Hydraulic Engineering*, vol. 110, no. 5, pp. 567–586, 1984.
- [24] V. P. Singh, *Dam Breach Modeling Technology*. Dordrecht, The Netherlands: Kluwer Academic Publishers, 1996.