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# Hydrological Impacts and Design Solutions for Downstream Tank Affected by Upstream Tanks Breaches in Cascade Systems

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**Abstract:** Cascade tank systems play a crucial role in water management in Sri Lanka, particularly in districts of Anuradhapura, Vavuniya, and Mannar. However, the failure of the upstream tanks can trigger the failures in downstream tanks. This study assesses the hydrological impacts of such failures and proposes design considerations. The study area is Sinnakunchikulam, Mannar district. Four flood scenarios were analysed. Scenario 1 estimated inflows based on the gross catchment area of the downstream tank, while Scenario 2 considered the net catchment area of downstream tank and outflow from the upstream tanks. Scenarios 3a and 3b analysed breach flow using an empirical breach width formula and an assumed fixed width. Among the evaluated scenarios, Scenario 3b was identified as the most critical condition based on the analysis results. The impact on downstream tank is influenced by water depth, breach area and stored water volume of the upstream tanks, the spill length and attenuation capacities of the downstream tank. Under Scenario 3, the breach outflow calculation of upstream tanks situated immediate to the downstream tank, is sufficient to prevent overtopping of downstream tank. The minimum freeboard is recommended by the highest afflux from Scenarios 1 and 2 unless Scenario 3b exceeds them, in which case it is based on the maximum afflux from Scenarios 1, 2, or 3a. To mitigate sliding failure under overtopping conditions, the earth dam must be designed with stability measures that account for the water level reaching the bund top. The proposed bund top level is determined by selecting the highest value among three criteria added to the Full Supply Level (FSL). These criteria are: (1) the combined height of the minimum freeboard, wave height, wave ride, and an additional one-foot safety margin; (2) the afflux calculated under Scenario 3b, multiplied by a safety factor of 1.2; and (3) a nominal freeboard of four feet.

**Keywords:** Cascade tank system, dam breach analysis, flood scenario, freeboard design

## 1. Introduction

The tanks are engineered water storage systems that play a crucial role in modern water resource management. Typically formed by constructing dams across rivers, these artificial lakes store the water during rainfall seasons to ensure a reliable supply for domestic consumption, agriculture, industry, and hydropower generation.

Beyond water storage, tanks significantly contribute to flood control by capturing excess runoff during the intense rainfall events, thereby reducing downstream flood risks and protecting communities, infrastructure, and ecosystems. The controlled release of water from reservoirs also helps maintain environmental flows, supporting aquatic life, sustaining biodiversity, and preserving the health of rivers and wetlands. Additionally, tanks often serve as recreational hubs, supporting boating, fishing, and eco-tourism, which contribute to local economies and community well-being.

### 1.1 Cascade Tank Systems in Sri Lanka

In Sri Lanka, cascade tank system (network of interconnected minor tanks) plays a vital role in irrigation, groundwater recharge, and flood regulation. These traditional water management structures enable efficient water distribution in monsoon-dependent regions by allowing water released from upstream tanks to be captured by downstream tanks. While these tank systems provide significant hydrological and socio-economic benefits, they also pose risks in the event of dam failure.

### 1.2 Dam Breach Risks in Cascade Systems

In a cascade system, dam breaches caused by overtopping, piping failure, structural degradation, or extreme hydrological events can trigger a sudden and uncontrolled release of water. This surge may destabilize downstream tanks, potentially leading to a chain reaction of failures. The resulting consequences include

- Disruption of sediment transport
- Loss of agricultural land
- Damage to infrastructure

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### 1.3 Dam Breaches in Cascade

#### 1.3.1 Rajawewa and Valathapitty Tank Breach [1]

In January 2011, heavy rainfall during the North-East Monsoon resulted in critical water levels in the Rajawewa Tank, which is located within the catchment area of the Valathapitty Tank in the Sammanthurai Divisional Secretariat, Ampara District, Eastern Province, Sri Lanka. Ultimately, the Rajawewa Tank was breached. There was considerable debate at the time regarding the cause of the breach. One group attributed it to overtopping, while another claimed that local farmers had deliberately cut the bund at a higher elevation to release excess water and prevent further overtopping. However, this sudden release of water caused the water level in the Valathapitty Tank to rise rapidly, leading to overtopping and eventually breaching the Valathapitty Tank as well. The breach resulted in

- Damage to approximately 100 acres of paddy land
- Inundation of 3,000 acres of paddy land for a week
- Road blockages and transportation disruptions

#### 1.3.2 Puthukulam Tank Breach and Mitigation at Uvadiyakulam Tank [2]

In November 2024, heavy rainfall during the North-East Monsoon significantly impacted Northern Province, Sri Lanka causing several tanks, including Puthukulam and Uvadiyakulam, to reach their spill levels. Puthukulam Tank suffered a breach due to piping failure near the sluice, resulting in a final breach with a top width of 14 m, a bottom width of 11.1 m, and a depth of 4.85 m. The tank emptied completely within 2.5 hours.

Consequently, the Uvadiyakulam Tank, with the water level just one foot below the spill level, began filling rapidly. In response to the risk of overtopping, three sluices were promptly opened to discharge excess water, despite uncertainty regarding the adequacy of the available spillway capacity. Within two hours, the tank began spilling approximately one foot, a process that continued for five hours. Eventually, the tank was restored to a safe condition.

### 1.4 Hydrological and Engineering Considerations in Cascade Systems

The severity of a dam-breach flood depends on several factors, including the breach geometry, the water depth, the storage capacity of upstream tanks, the spillway capacity, and the ability of downstream tanks to attenuate the resulting flood wave. These factors highlight the critical need for proactive risk assessment and the implementation of effective mitigation strategies to enhance dam safety. Design approaches typically consider worst-case scenarios to ensure preparedness. In flood studies for cascade systems, several key factors must be considered. Catchment area estimation is crucial, as the gross catchment area directly influences inflows, impacting spillway design and freeboard requirements. Soil conditions must be assessed in their saturated state to account for worst-case scenarios. Additionally, breach flow assessments must incorporate the influence of upstream tanks to achieve accurate modelling and effective flood management.

### 1.5 Study Objectives

This study aims to assess the hydrological consequences of the upstream tanks breach within a cascade tank system and its impact on the downstream tank. Additionally, it seeks to develop design considerations to minimize the effects of overtopping on downstream tank ensuring enhanced flood resilience and dam safety.

### 1.6 Study Area

This study focuses on the irrigation system in the Sinnakunchikulam area, located within the Madu Divisional Secretariat Division, Mannar District. The cascade system comprises five interconnected tanks: Adanchikulam, Kunchukulam, Puthukulam, Thampanaikulam, and Uvadiyakulam. The flow diagram of the cascade system is presented below.

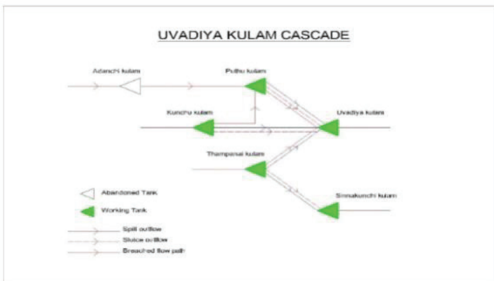


**Figure 1- The tanks and flow path in the cascade**

As shown in Figure 1, the Adanchikulam Tank has been abandoned due to its location within a designated forest conservation area, which imposes restrictions on its use for irrigation purposes. Consequently, the tank is no longer actively utilized within the irrigation system.

This methodology primarily focuses on the remaining functional tanks in cascade specially Kunchukulam, Puthukulam, Thampanaikulam and Uvadiyakulam.

Regarding the stream path, water first flows from the Adanchikulam catchment towards Puthukulam. Simultaneously, excess water spilling from Kunchukulam reaches Puthukulam through its spillway. Eventually, the overflow from Puthukulam drains into Uvadiakulam via its spillway. Additionally, excess water from Thampanaikulam also flows into Uvadiakulam.

However, water released through the sluices of Kunchukulam and Puthukulam flows directly to Uvadiakulam, whereas sluice discharge from Thampanaikulam does not contribute to Uvadiakulam.

In the event of a dam breach, water from Kunchukulam and Puthukulam will reach Uvadiakulam. The impact of a breach at Thampanaikulam, however, depends on the location of the failure, water may either flow into Uvadiakulam or be diverted towards Sinnakunchikulam.

Table 1 summarises the characteristics of each tank and its associated catchment. The catchment area of a tank refers to the geographical region from which all rainfall and surface runoff drain into the tank.

Gross catchment area: The total upstream area that theoretically contributes runoff to the tank, including areas intercepted by upstream tanks.

Net catchment area: The actual area that directly contributes runoff to the tank, excluding portions where runoff is intercepted or stored by upstream tanks.

The gross longest path is defined as the longest natural flow path within the gross catchment, measured from the farthest point to the tank. Similarly, the net longest path is the longest flow path within the net catchment area, excluding the influence of upstream tanks. The gross curve number (CN) is derived based on the land use, soil type, and hydrologic condition of the entire gross catchment area. In contrast, the net curve number is calculated using only the net catchment area.

Likewise, the gross slope and net slope refer to the slope along the longest flow path within the gross and net catchment areas, respectively. These parameters are critical for hydrological modelling.

**Table 1 - Tank and catchment characteristics**

| **Name** | | **Puthukulam** | | **Kunchukulam** | **Thampanaikulam** | **Uvadiyakulam** |
|---|---|---|---|---|---|---|
| Catchment area ($km^2$) | G | 8.24 | | 0.98 | 1.364 | 14.13 |
| | N | 2.67 | | ………….. | …… | 4.52 |
| Longest path (m) | G | 4545 | | 1611 | 2229 | 7,154.00 |
| | N | 4545 | | ……….. | …….. | 3,357.00 |
| Curve Number (CN) | G | 78.15 | | 79.74 | 80.88 | 79.23 |
| | N | 77.93 | | …….. | ……. | 80.71 |
| Slope | G | 0.007578 | | 0.01419 | 0.0124 | 0.01 |
| | N | 0.007578 | | …….. | ……. | 0.01 |
| Capacity ($m^3$) at | FSL | 381,018.0 | | 121,072.00 | 203,031.00 | 542,730.00 |
| | BTL | 751,806.0 | | 276,247.00 | 649,662.00 | 1,354,225.00 |
| Spill length (m) | | 70 | | 15 | 100 | 80.00 |
| FSL (m MSL) | | 48.30 | | 48.3 | 44.7 | 38.12 |
| BTL (m MSL) | | 49.6 | | 49.6 | 46.2 | 39.60 |
| Top width(m) | | 3.0 | | 3.0 | 3.0 | 3.0 |
| Earth dam type | | Homogeneous | | Homogeneous | Homogeneous | Homogeneous with Core wall constructed at Seepage occurred location |
| Earth dam material | | Sandy Clay | | Sandy Clay | Sandy Clay | Sandy Clay |

## 2. Literature Review

Several studies have investigated dam-break dynamics within cascade systems, highlighting the complex hydrological interactions between upstream failures and downstream reservoirs. Hu et al. (2020) developed a Cascade Reservoir Breaching Simulation (CRBS) model for sequential reservoir failures in cascade systems. The model integrates flood routing and flood-regulating calculations, enabling the simulation of hydraulic interactions between upstream and downstream reservoirs. Built upon the Dam Breach Analysis model (DB-IWHR), the CRBS model was applied to a case study involving the Tangjiashan barrier lake and a downstream reservoir (Reservoir II). The simulation results indicated that the calculated peak outflow was approximately 10% higher than the recorded data, which was considered to be within an acceptable range. The study highlighted the model's capability to evaluate breach-related risks and recommended risk mitigation measures, such as establishing early warning systems to pre-emptively reduce downstream storage levels, and constructing an additional discharge tunnel to enhance outflow capacity. Overall, the CRBS model is demonstrated to be an effective tool for risk assessment and flood control in cascade reservoir systems.

Cai et al. (2019) proposed a Bayesian Network (BN)-based probabilistic model to assess flood risks in cascade reservoir systems. The model incorporated expert judgment, historical records, and computational methods to estimate prior probabilities and original conditional probability tables. A sensitivity analysis was carried out to identify likely sequential dam-break failure paths. To reduce potential misperceptions in the probability estimates, the Dam Breach Analysis Model (DB-IWHR, 2014) was used together with flood regulation computations to simulate the dam breaching process. The method was applied to the Bala–Busigou–Shuangjiangkou (BL–BSG–SJK) cascade reservoir system in the Dadu River Basin, China, where three continuous failure paths were identified. A refined BN model was developed using updated evidence to estimate the failure probability associated with each path. The study highlighted the potential of BN models to support probabilistic risk analysis.

Chen et al. (2018) developed a risk-based real-time flood control operation model for cascade reservoir systems under emergency conditions by integrating natural disaster scenarios and forecast uncertainties in inflows and parameter estimation. The model comprises three key modules: (i) simulation of emergency scenarios such as upstream dam break, earthquakes, and spillway failure; (ii) Monte Carlo simulations to handle uncertainties and stochastic variations in inflow and system response; and (iii) risk analysis to evaluate dam overtopping probabilities and operational schedules. The methodology was applied to the Shuangjiangkou cascade system in the upper reaches of the Daduhe River Basin, China. Results showed that the integrated flood risk increases with rising initial reservoir water level, the degree of inflow uncertainty, the return period of extreme floods, and a decrease in release capacity. The study identified maximum initial water levels under different operational strategies, including 2447 m above sea level under the release capacity model and 2444.5 m above sea level under the command model in the case of an upstream dam breach. The findings offer a practical framework for adaptive decision-making in real-time reservoir operations during emergencies and highlight the importance of incorporating both emergency scenarios and uncertainty analysis into reservoir risk management.

## 3. Methodology

This study evaluates the hydrological impacts of the upstream tanks breach in the catchment of Uvadiyakulam Tank, located in the Mannar District. The methodology includes data collection and scenario-based flood analysis to evaluate the impact of the upstream tank's breach.

### 3.1 Data Collection

Key data were collected and analysed to support the study, including:

- **Digital Elevation Model (DEM):** Hydrological features such as streamlines, slopes, watershed boundaries, and catchment areas were delineated using the Hydrology tool set in ArcGIS.
- **Area-Capacity Relationship:** The area-capacity relationship for each tank was established to determine storage capacity.
- **Rainfall:** Rainfall data were sourced to evaluate the water balance and guide flood studies.

### 3.2 Flood Studies

Flood studies play a critical role in designing the spillway of a tank and ensuring the safety of the tank bund. Accordingly, detailed flood assessments were conducted for all tanks within the cascade. Four primary scenarios were analysed to determine the most critical or worst-case flood condition.

#### 3.2.1 Scenario 1: Inflow calculation by using Gross Catchment Area

This scenario follows existing design guidelines by utilizing the gross catchment area.

##### 3.2.1.1 Return period rain fall

The maximum daily rainfall data recorded over a 30-year period were arranged in descending order and assigned corresponding return periods. The Gumbel Extreme Value Distribution was employed to establish a statistical relationship between rainfall magnitude and return period. Based on this correlation, design rainfall values were estimated for various return periods. For this cascade system, the design rainfall corresponding to the 50-year and 100-year return periods was adopted for further analysis.

##### 3.2.1.2 Inflow calculation using SCS Dimensionless Unit Hydrograph

The SCS dimensionless unit hydrograph method was applied to estimate inflows using both the actual curve number and a hypothetical maximum curve number of 100 for comparative analysis. Both the SCS equation and Kirpich's equation were utilized in the calculation.

**1. Time of Concentration ($t_c$) Calculation [3]**

The time of concentration ($\mathbf{t_c}$) in hours was calculated using the SCS model in SI units:

$$t_c = \frac{227\, L^{0.8}\, (\lambda+1)^{0.7}}{10^5 \sqrt{S}} \quad \dots (1)$$

where:

- L = Length of the longest flow path (m)
- S = Average watershed slope (%)
- λ = Curve number function, defined as:

$$\lambda = \frac{1000}{CN} - 10 \quad \dots (2)$$

CN= The Curve Number, determined based on soil type and land use, was used in the analysis. For the worst-case condition, a CN value of 100 was adopted to calculate the Time of Concentration.

The time of concentration was also determined by using the Kirpich's Equation as follows [4,5]

$$t_c = \frac{0.01947\, L^{0.77}}{S^{0.385}} \quad \dots (3)$$

where $t_c$= time of concentration (min) L = Length of the longest path (m), S = slope of the catchment

**2. Lag Time ($t_p$) Calculation**

The lag time ($\mathbf{t_p}$) was determined as:

$$t_p = 0.6\, t_c \quad \dots (4)$$

where $t_p$ (hrs.) is the duration between the centroid of effective rainfall and the peak of the hydrograph.

**3. Time to Peak ($T_p$) Calculation**

The time to peak ($\mathbf{T_p}$ in hrs) is calculated as:

$$T_p = \frac{t_r}{2} + t_p \quad \dots (5)$$

where $t_r$ is the duration of effective rainfall (taken as 1 hour**).**

**4. Base Time ($T_b$) Calculation**

The total base length of the hydrograph ($\mathbf{T_b}$ in hrs) is given by:

$$T_b = 2.67\, T_p \quad \dots (6)$$

**5. Peak Discharge ($Q_p$) Calculation**

The peak discharge ($\mathbf{Q_p}$ in m3/s) is estimated using:

$$Q_p = 2.08\, \frac{A}{T_p} \quad \dots (7)$$

where: A = Catchment area ($km^2$)

Using the coordinates of the SCS Dimensionless unit hydrograph provided in the following table, the unit hydrograph was developed. Subsequently, the flood hydrograph was derived by applying the unit hydrograph in conjunction with the Type II rainfall pattern, which represents the most critical condition compared to other rainfall types, such as Type I, Type IA, and Type III [3].

**Table 2 - Coordinate of SCS unit hydrograph [3]**

| t/Tp | q/qp | t/Tp | q/qp | t/Tp | q/qp |
|---|---|---|---|---|---|
| 0.0 | 0.00 | 1.1 | 0.99 | 2.4 | 0.147 |
| 0.1 | 0.30 | 1.2 | 0.93 | 2.6 | 0.107 |
| 0.2 | 0.10 | 1.3 | 0.86 | 2.8 | 0.077 |
| 0.3 | 0.19 | 1.4 | 0.78 | 3.0 | 0.055 |
| 0.4 | 0.31 | 1.5 | 0.68 | 3.2 | 0.040 |
| 0.5 | 0.47 | 1.6 | 0.56 | 3.4 | 0.029 |
| 0.6 | 0.66 | 1.7 | 0.46 | 3.6 | 0.021 |
| 0.7 | 0.82 | 1.8 | 0.39 | 3.8 | 0.015 |
| 0.8 | 0.93 | 1.9 | 0.33 | 4.0 | 0.011 |
| 0.9 | 0.99 | 2.0 | 0.28 | 4.5 | 0.005 |
| 1.0 | 1.00 | 2.2 | 0.207 | 5.0 | 0.000 |

#### 3.2.1.3 Flood routing (Modified Pul's Method) [5]

This study examines the effect of a flood wave entering a tank. By utilizing the storage-elevation relationship, the study predicts the variation of reservoir elevation and outflow discharge over time. This routing analysis is essential for designing the appropriate spillway capacity

$$\frac{(I_1 + I_2)}{2}\Delta t - \frac{(O_1 + O_2)}{2}\Delta t = S_2 - S_1 \quad \text{... (8)}$$

where, $I_1$ and $I_2$ are inflows (m³/s), $O_1$ and $O_2$ are outflows (m³/s), and $S_1$ and $S_2$ are storages (m³). This method provides outflow and afflux.

### 3.2.2 Scenario 2: Current condition using outflow of reservoirs in upper catchment and net catchment area

This scenario accounts for the outflow from upper catchment tanks and the inflow from net catchment area, providing a more accurate representation of inflow conditions. Therefore, calculations, as in Scenario 1, were performed for the net catchment area. The inflow from the net catchment area was then added to the outflow from the upper catchment tanks.

### 3.2.3 Scenario 3: Inflow scenarios considering upstream tank breaching

When the upstream tank breaches, it significantly alters the inflow conditions of the downstream tank. The sudden release of water creates a surge, which is analysed to assess its impact on the tank storage and spillway discharge of the downstream tank.

In this scenario, the inflow is volume of water released due to the tank breach.

#### 3.2.3.1 Breach dimension estimation

The initial breach bottom width was assumed to be 5 m, 10 m, 15 m, and 20 m, with a side slope of 1V:1H. As expected, the breach flow increased with widening; however, the rate of widening was limited due to the rising breach flow reducing the water level. Consequently, breach widths beyond 20 m were not considered. Further widening primarily occurs through scouring effects rather than direct lateral expansion.

Additionally, followings formula was used to find the breach dimension. [6]

**1.Frohelic's Formula (1987)**

$$B^* = 0.47\, k_o S^{*0.25} \quad \text{... (9)}$$

Where B*: Dimensionless average breach width = Bavg /$h_b$, ko: 1.4 for overtopping, 1.0 for piping, S*: Dimensionless Storage at failure = S/$h_b^{3}$, Bavg: Average breach width (m),
S: Storage (m³) at failure,

$$Z = 0.75\, K_c (hw^*)^{1.57}\, (W^*)^{0.73} \quad \text{… (10)}$$

Z = breach side slope factor (1V: ZH), $h_w^*$ = $h_w/h_b$, $h_w$ = height of water above breach bottom (m), $h_b$ = depth of breach (m), W* = (W crest +W bottom)/2$h_b$,

W crest = top dam width (m), W bottom = bottom dam width (m), Kc= 1.0 for without core wall, 0.6 for core wall

**2.Frohelic's Formula (1995)**

$$B_{avg} = 0.1803\, k_o V_w^{0.32} h_b^{0.19} \quad \text{… (11)}$$

where $B_{avg}$: Average breach width (m) ko: 1.4 for overtopping, 1.0 for piping, $V_w$: Storage at failure (m³), $h_b$: final breach depth (m), side slope factor Z =1.4 for overtopping and 0.9 for piping (1V: ZH)

**3.Johnson and Illes's Formula**

$$0.5\, h_d \leq B \leq 3h_d \quad \text{… (12)}$$

where B: Breach width (m), $h_d$: height of dam (m)

**4.Singh and Snorrason's Formula**

$2\,h_d \leq B \;\leq 5h_d$ … (13)

where B: Breach width (m), $h_d$: height of dam(m)

**5.FERC Formula**

$2\,h_d \leq B \;\leq 4h_d$ … (14)

$$0.25 \leq Z \leq 1$$

where B: Breach width (m), $h_d$: height of dam (m), Z: Breach opening slide slope factor (1V: ZH)

**6.Reclamation Formula**

$B = 3h_w$ … (15)

where B: Breach width (m), $h_w$: height of water (m)

**3.2.3.2 Breach flow calculation**

The flow rate from the breached tank was determined using the weir flow equation, which assumes that the breach behaves like a broad-crested weir.

$Q = C_b B H^{1.5}$ ... (16)

where: Q = breach discharge ($m^3/s$)

- $C_b$ = discharge coefficient
- B = breach width (m)
- H = depth of water above the breach (m)

During the initial breach formation phase, the out flow is not considered. However, once the breach is fully developed, the outflow is taken into account to determine the maximum discharge.

The storage changed during breach is due to inflow from catchment, out flow from sluice and spill and breached out flow.

$\frac{ds}{dt} = \; I(t) - O_{outlet}(S) - \; O_{breached} \;\; (S)$ ... (17)

S: Storage ($m^3$) I(t): Inflow ($m^3$), $O_{outlet}$: Out flow through sluice or spill ($m^3$), $O_{breached}$: breached out flow ($m^3$), t: time (min)

The assumption is made that the inflow into the tank is negligible and the outflow through the sluice or spillway is also considered negligible then re-write the above equation is

$\frac{ds}{dt} = \; - O_{breached} \;\; (S)$ . … (18)

Then numerically

$\frac{\Delta s}{\Delta t} = \frac{-\,(O1+O2)}{2}$ … (19)

Then $S1 - S2 = \frac{(\,O1+O2)\Delta\, t}{2}$ … (20)

$S1 - \frac{O1\,\Delta t}{2} \;= S2 + \frac{O2\Delta t}{2}$ .. . (21)

The outflow was calculated for different depths. Then, S + OΔt/2 was determined, and a graph was plotted between outflow (O) and S + OΔt/2.

Initially, the storage at FSL or BTL and the outflow were estimated. Next, $S_1$ - $O_1\Delta t/2$ was computed and equated to $S_2 + O_2\Delta t/2$. Using the graph, the outflow ($O_2$) was determined. The process, with a 60-second time interval, was repeated until the tank was fully emptied.

**3.2.3.3 Downstream tank response**

In the worst-case scenario, it is assumed that all the water released from the breach directly enters the downstream tank without any losses. This means that the volume of breach outflow is equal to the volume of inflow into the downstream tank, and the breach flow hydrograph is taken as the inflow hydrograph to the downstream tank. This assumption represents the most severe impact scenario, ensuring that the downstream tank experiences the highest possible water levels and flood risks.

The variation in the downstream tank's water level was analysed to assess its impact on spillway discharge, potential overtopping, and dam stability.

As the breached flow entered the water spread area of the downstream tank, it increased storage and raised the water level. If the water level exceeded the spillway crest level, spill discharge occurred. The spill discharge and afflux were computed by using the Modified Pul's Method [4].

$\frac{(I_1 + \,I_2)}{2}\,\Delta t - \;\frac{(O_1 + \,O_2)}{2}\,\Delta t = \; S_2 \; - S_1$ ... (22)

For the calculation, the breached outflow of upstream tanks is considered the only inflow while the outflow of downstream tank is assumed to occur solely through the spillway.

For the Uvadiyakulam catchment, where three upstream tanks exist, it is assumed that the breach outflow from each tank reaches Uvadiyakulam simultaneously to produce the maximum inflow. When analysing breach outflows for these tanks, the breach width is maintained simultaneously to study its relationship with total discharge.

#### 3.2.3.4 Scenario 3a: piping or structural failure of the upstream tank

In this scenario, the upstream tank failure is assumed to occur due to piping or structural failure. For a conservative approach, the water level is considered at the full supply level (spill level) at the time of failure. Simultaneously, the downstream reservoir water level is also assumed to be at the full supply level.

#### 3.2.3.5 Scenario 3b: overtopping failure of the upstream tank

In this scenario, the upstream tank failure is assumed to occur due to overtopping, meaning the water level is considered at the bund top level. Simultaneously, the downstream reservoir water level is assumed to be at full supply level (spill level).

## 4. Results and Discussion

The flood studies were analysed under different scenarios to identify the critical conditions. The primary outputs considered in the analysis include storage change, outflow, and afflux. Among these, afflux is the most suitable parameter for assessing tank conditions, as it directly reflects the potential for overtopping, spilling, and determining the freeboard requirements. Therefore, the results highlight afflux as a key indicator for evaluating the safety and performance of the tanks under various flood scenarios.

### 4.1 Scenario 1

Under Scenario 1, the afflux values for tanks K, P, T and U were evaluated for 50-year and 100-year return periods using three hydrological methods: the standard Curve Number (CN) method, an extreme runoff condition with CN = 100, and the Kirpich's empirical method.

The results, summarized in Tables 3 and 4, demonstrate a clear increase in afflux as the return period increases. This upward trend is consistent across all tanks and methods, indicating the heightened flood risk associated with more intense rainfall events.

**Table 3 - Afflux (m) under scenario 1 with return period 50 years**

| Tank Name | CN | CN=100 | Krilich | Condition |
|---|---|---|---|---|
| K | 0.51 | 0.60 | 0.59 | Spilling |
| P | 0.61 | 0.82 | 0.93 | Spilling |
| T | 0.19 | 0.23 | 0.24 | Spilling |
| U | 0.67 | 0.91 | 1.04 | Spilling |

**Table 4 - Afflux (m) under scenario 1 with return period 100 years**

| Tank Name | CN | CN=100 | Krilich | Condition |
|---|---|---|---|---|
| K | 0.55 | 0.65 | 0.64 | Spilling |
| P | 0.66 | 0.88 | 1.00 | Spilling |
| T | 0.20 | 0.25 | 0.26 | Spilling |
| U | 0.72 | 0.98 | 1.12 | Spilling |

Among the methods applied, the standard CN method produced the lowest afflux values, representing catchment conditions. The CN = 100 condition assumes maximum runoff potential, such as in cases of saturated or impervious surfaces, resulting in noticeably higher afflux. The Kirpich's method, an empirical time-of-concentration formula, often yielded the highest afflux values. This variation across methods highlights the importance of considering multiple approaches when assessing flood impacts in order to capture a range of possible outcomes.

Tank-specific analysis shows that Tank U consistently exhibits the highest afflux values under all scenarios, ranging from 0.67 m to 1.12 m. This suggests a greater vulnerability to elevated inflows, likely due to its catchment characteristics such as larger area. In contrast, Tank T experiences the lowest afflux values, ranging from 0.19 m to 0.26 m. Tanks K and P fall between them.

### 4.2 Scenario 2

Under Scenario 2, the afflux values for downstream tanks P and U were estimated for 50-year and 100-year return periods using three hydrological methods:

**Table 5 - Afflux (m) under scenario 2 with return period 50 years**

| Tank | CN | CN=100 | Kirpich | Condition |
|---|---|---|---|---|
| P | 0.62 | 0.82 | 0.91 | Spilling |
| U | 0.85 | 1.03 | 1.13 | Spilling |

**Table 6 - Afflux (m) under scenario 2 with return period 100 years**

| Tank | CN | CN=100 | Kirpich | Condition |
|---|---|---|---|---|
| P | 0.67 | 0.86 | 0.97 | Spilling |
| U | 0.92 | 1.10 | 1.21 | Spilling |

The results (Tables 5 & 6) show a consistent increase in afflux with increasing return periods. all values indicate a "Spilling" condition.

When comparing these results with those from Scenario 1 (Tables 3 and 4), it is evident that afflux values are generally higher for U tank and lesser for P tank in Scenario 2. This suggests that the inflow depends catchment characteristics such as area, longest path & slope of the gross and net catchment.

### 4.3 Scenario 3

In this scenario, breach outflows were computed for different breach widths in the tanks within the upper catchment of Uvadiyakulam. Scenario 3a assumed the tank water level to be at full supply level at the time of the breach, whereas scenario 3b assumed the bund top level as the water level.

**Table 6 - Afflux (m) under scenario 3a**

| No. | Breach width | Afflux | Condition |
|---|---|---|---|
| 1 | 5 m | 0.69 | Spilling |
| 2 | 10 m | 0.80 | Spilling |
| 3 | 15 m | 0.87 | Spilling |
| 4 | 20 m | 0.90 | Spilling |
| 5 | Froehlich 1987 | 0.82 | Spilling |
| 6 | Froehlich 1995 | 0.75 | Spilling |
| 7 | Johnson &Illes | 0.75 | Spilling |
| 8 | Singh & Snorrason | 0.86 | Spilling |
| 9 | FERC | 0.85 | Spilling |
| 10 | Reclamation | 0.75 | Spilling |

**Table 7 - Afflux (m) under scenario 3b**

| No. | Breach width | Afflux | Condition |
|---|---|---|---|
| 1 | 5 m | 1.24 | Spilling |
| 2 | 10 m | 1.4 | Spilling |
| 3 | 15 m | 1.54 | Overtopping |
| 4 | 20 m | 1.64 | Overtopping |
| 5 | Froehlich 1987 | 1.69 | Overtopping |
| 6 | Froehlich 1995 | 1.63 | Overtopping |
| 7 | Johnson &Illes | 1.42 | Spilling |
| 8 | Singh & Snorrason | 1.64 | Overtopping |
| 9 | FERC | 1.62 | Overtopping |
| 10 | Reclamation | 1.42 | Spilling |

Tables 6 and 7 present the afflux values under Scenario 3a and Scenario 3b, respectively, highlighting the influence of breach width and empirical breach models on downstream hydraulic response. In Scenario 3a, afflux values range from 0.69 m to 0.90 m across varying breach widths and empirical methods, with all conditions resulting in spilling. The increase in afflux with breach width indicates that larger breaches contribute to higher water levels at downstream tank. however, the downstream tank remains within safe limits under these moderate conditions. In contrast, Scenario 3b exhibits significantly higher afflux values, ranging from 1.24 m to 1.69 m, reflecting more severe upstream breach scenarios. Notably, afflux exceeds the downstream tank bund elevation in several cases, leading to overtopping conditions, particularly for breach widths of 15 m and above and empirical formulas such as Froehlich (1987& 1995), Singh & Snorrason (1982,1984) and FERC. This transition from spilling to overtopping signifies a critical shift in flood risk, necessitating urgent structural or operational mitigation. Among the empirical models, Froehlich (1987) yields the highest afflux in Scenario3b.

Furthermore, it was identified that the afflux at the downstream tank is influenced by the water depth, storage volume and breach area of the upstream tanks, as well as the spill capacity and detention capacity of the downstream tank. Therefore, establishing the correlation between afflux and these influencing variables is essential. The identifying such relationships would enable the prediction of flood impacts and support the development of early warning systems for downstream areas.

## 5. Conclusion

Afflux values exhibit an increasing trend with the rise in return period from 50 to 100 years, reflecting the impact of more extreme flood events. Assuming a Curve Number (CN) of 100, which represents zero infiltration and maximum surface runoff, results in a further increase in afflux values. Among the hydrological methods, Kirpich's equation generally predicts higher inflows and afflux compared to the SCS-CN approach, even at CN = 100. This suggests that Kirpich's method inherently estimates rapid runoff response, leading to higher inflows.

A comparison between Scenario 1 and Scenario 2 reveals that the hydrological response varies by location. For instance, Puthukulam exhibits a slight decrease in afflux under Scenario 2, possibly due to differences in catchment characteristics, whereas Uvadiyakulam shows an increase in afflux, indicating greater vulnerability. These contrasting responses demonstrate that the severity of each scenario is location-specific and influenced by factors such as catchment area, slope, and tank detention capacity. Consequently, it is not possible to determine a single scenario as the worst case. Instead, the criticality of each scenario depends on the distinct hydrological characteristics of the catchments associated with the tanks within the cascade system.

It was also observed that breach flow increases proportionally with increasing breach area. Empirical formulas show variation in afflux predictions, with Froehlich (1987) generally yielding the highest estimates.

In Scenario 3, when the upstream tank is at the Bund Top Level (BTL) under Scenario 3b, the resulting breach flow is higher than when the tank is at Full Supply Level (FSL) under Scenario 3a. This indicates that water depth and storage volume have a significant influence on breach discharge. Based on the analysis, Scenario 3b represents the most critical condition among the four scenarios considered.

The impact on downstream tank due to an upstream tanks breach depends on water depth, breach area, stored water volume in the upstream tank at failure, spill capacity and attenuation capacity of downstream tank.

## 6. Recommendation

Under Scenario 3 in a cascade tank system, the breach outflow from upstream Tank 1, which is directly situated above the downstream tank, is determined to be sufficient for evaluating the overtopping risk at the downstream tank. Therefore, it is not necessary to compute the breach outflow from upstream Tank 2 located within the catchment of Tank 1 for the purpose of assessing overtopping at the downstream tank. Nevertheless, it remains essential to evaluate the breach outflow from Tank 2 and its potential impact on Tank 1, as this may induce overtopping or failure at that level. This hierarchical assessment strategy is critical to mitigate the risk of progressive failure propagation within the cascade system.

To determine the minimum required freeboard, flood studies for downstream tank should be conducted using either Kirpich's equation or the SCS Curve Number (CN) method with a CN value of 100, under both Scenario 1 and Scenario 2, and for selected return periods. Additionally, breach modelling should be performed under Scenario 3a and Scenario 3b to analyse potential failure conditions and their impact on downstream tank.

The maximum afflux obtained from the four scenarios can be considered as the minimum required freeboard. However, considering construction costs and the reduced probability of overtopping when using either Kirpich's equation or the SCS method with a Curve Number (CN) of 100 for a return period of 100 years or more, the risk of overtopping is significantly minimized. Therefore, any potential overtopping is more likely to be caused by unforeseen factor, such as spillway blockage by aquatic plants or debris. Taking these aspects into account, the final minimum freeboard is selected as follows.

If the afflux under the scenario 1 or 2 is greater than scenario 3b, the minimum free board has to be taken the maximum of the afflux value of the scenario 1or 2.

Otherwise, the minimum freeboard is the maximum afflux value of the Scenario 1 or 2 or 3a.

1. Minimum free board = maximum afflux under Scenarios 1 & 2 (if afflux under Scenarios 1 & 2 > Scenario 3b) … (23)

2. Minimum free board = maximum afflux under Scenarios 1, 2 & 3a (if afflux under Scenarios 1 & 2 < Scenario 3b) … (24)

The bund top level is determined by selecting the greatest value among following three approaches. In this way, the earth dam is ensured to be capable of withstanding extreme hydrological events and dam breached thereby minimizing the risk of overtopping and stability failure. At the same time, cost-efficiency is also promoted.

1.According to Section 6.6.6 and 7.2.5 of Technical Guidelines for Irrigation Works by Ponrajah [7], the Bund Top Level (BTL) is determined using the following equation:

BTL = FSL +Minimum free board from equation 22 and 23 + Wave height and riding + nominal free board … (25)

In this case, a nominal freeboard of 1 foot (approximately 0.33 m) is considered and also recommended to 0.5 m to 1.0 m. This additional margin serves as a basic allowance for unknown factors.

2.The bund top level is determined by adding the FSL and afflux under scenario 3b multiplied by safety factor 1.2.

BTL=FSL+1.2 x Afflux under Scenario 3b… (26)

A safety factor of 1.2 is justified since the afflux is theoretically calculated, while the actual afflux during breach flow may vary. To ensure design safety, this factor is applied to derive a conservatively higher afflux, accounting for potential uncertainties such as water level fluctuations due to turbulence and wind, long-term embankment settlement, and possible human errors in surveying or levelling.

3.According to Section 5.9 of Design of Irrigation Headworks for Small Catchments by Ponrajah [8], the nominal freeboard is the vertical distance between the Bund Top Level (BTL) and the Full Supply Level (FSL), primarily determined by the fetch, and a range of 3 to 6 feet is recommended. However, the Department of Agrarian Development adopts a standard minimum nominal freeboard of 4 feet for all newly constructed minor tanks, irrespective of the calculated value, to ensure structural safety, allow for settlement, and enhance long-term durability.

BTL=FSL +1.32, m (Nominal free board) … (27)

The highest value obtained from Equations 25, 26, and 27 should be adopted as the Bund Top Level (BTL).

Furthermore, the stability of the earth dam should be checked against sliding failure, considering the water level at the bund top level. This ensures that the dam remains stable under extreme conditions, preventing potential stability failures during overtopping events.

Increasing the spill length is a feasible alternative to raising the bund for flood control. However, the implementation of a spillgate system is not considered practical in this case. The majority of the minor tanks are located in remote areas, where farmers with limited technical knowledge are responsible for operation and maintenance. Additionally, the logistical challenges of operating the spillgate at night, in the event of an overtopping condition, further complicate its practicality. Therefore, the installation of a spillgate is not recommended.

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Abbreviations

BN: Bayesian Network
BTL: Bund Top Level
CN: Curve Number
CRBS: Cascade Reservoir Breaching Simulation
DB- IWHR: Dam Breach Model Institute of Water Resources and Hydropower Research
FB: Freeboard
FERC: Federal Energy Regulatory Commission
FSL: Full Supply Level
G: Gross
K: Kunchukulam
MFB: Minimum Free Board
MSL: Mean Sea Level
N: Net
P: Puthukulam
Sc: Scenario
T: Thampanaikulam
U: Uvadiyakulam