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# C-1 CONSEQUENCES OF DAM OR LEVEE FAILURE

## C-1.1 Introduction

Flood water can be one of the most destructive forces on earth, especially if caused by an event that unexpectedly overwhelms an existing flood defense or by catastrophic breach of a dam or levee. Relatively recent events, such as flooding caused by Hurricane Katrina (2005) and the tsunami in Japan (2011), have caused thousands of people to lose their life and unknown billions of dollars in damages. By the same token, dozens of floods (some from similarly unexpected events like a dam or levee breach) occur every year with no resulting loss of life and relatively minimal property damage.

Although flooding can have many types of severe consequences, including economic, social, cultural, and environmental, the primary objective of Bureau of Reclamation's (Reclamation) dam safety program and United States Army Corps of Engineers' (USACE) dam and levee safety programs are to manage the risk to the public who rely on those structures to keep them reasonably safe from flooding. Thus, reducing the risk associated with loss of life is paramount. The safety programs of both agencies treat life loss separately from economic and other considerations. Decisions as to whether to invest in dam or levee improvements are based primarily on risk to life by applying the concept of tolerable risks. Since informed decisions based on tolerable risk require estimates of loss of life for potential flood events, the focus of this chapter is on estimating loss of life. Estimation of the magnitude of life loss resulting from a flood requires consideration of the following factors:

- Understanding of the population at risk (PAR) in the potentially impacted area

- Warning and evacuation assumptions for that PAR

- Flood characteristics including extents, depths, velocities, and arrival time (can be heavily influence by failure mode and breach parameters)

- Estimation of fatality rates

The full consideration of all these factors is a complex problem that requires detailed modeling of the physical processes (breach characteristics and flood routing), human responses, and the performance of technological systems (such as warning and evacuation systems, transportation systems and buildings under flood loading). This chapter describes a range of practical approaches to this complex problem that can provide life-loss estimates for use in risk analysis.

## C-1.2 Life Loss Estimation Methodologies and Agency Perspectives

The USACE and Reclamation perform risk analysis for the dams or USACE levees (to assist with risk informed decision making on flood defense infrastructure within each agency's portfolio. While the basic concept of using life loss estimates to help quantify risk is similar between each agencies, the methodologies employed by the two agencies have differences. This chapter is intended for use by both agencies, and is structured in a way that presents general information on life loss estimation.

USACE applies the Hydrologic Engineering Center (HEC)-LifeSim model to estimate life loss as well as direct economic damage. LifeSim is an agent-based simulation model that tracks the movement of people and their interaction with flooding through time. It includes an integrated transportation simulation algorithm to model the evacuation process, and evaluates loss of life based on location of people when the water arrives and important factors related to building, vehicle, and human stability. Fatalities are estimated by grouping people into high or low hazard "zones." Each zone has a corresponding fatality rate, which were developed based on an extensive review and analysis of historic flood events.

Reclamation primarily uses the Reclamation Consequences Estimating Methodology (RCEM) (Reclamation 2015) to estimate dam failure life loss. RCEM is an empirical-based method which relies on 60 dam failure and other flooding cases as a basis. A total of 79 data points have been created from the case histories, and fatality rates are estimated using curves that have been developed from the data. Flooding intensity (the intensity is quantified by DV, which is the maximum depth [D] of flooding multiplied by maximum velocity [V] of the flood flow) and warning time are the key parameters that affect fatality rate selection. Strong emphasis is placed on a team approach to the development of assumptions, fatality rate selection, building the case and identifying sources of uncertainty. RCEM is a revision to the DSO-99-06 (Reclamation 1999) method, also developed by Reclamation. DSO-99-06 was in use by Reclamation from 1999 to 2014, prior to the development of RCEM.

Reclamation has also been developing capability with the Life Safety model. Similar to the LifeSim model used by USACE, the Life Safety model is a simulation model that tracks movement of water and movement of people. Fatalities are estimated based on various factors including building destruction, vehicle toppling and drowning. The Life Safety Model has an integrated transportation model, but does not use empirical-based fatality rates.

One important difference between simulation models (HEC-LifeSim (USACE 2018) and the Life Safety model and the empirical methodology employed in RCEM is that simulation models estimate "direct" loss of life, which is the loss of life caused by direct exposure to the physical effects of the flood. More specifically, direct life loss is that caused by drowning or being crushed by a building that collapses due to flooding. Indirect life loss includes those people that lose their life due to other flood related issues such as stress leading to heart attack or suicide, illness, or prolonged exposure to the elements at an emergency evacuation location (e.g. attic, overpass, etc.).

More information on agency specific approaches to estimating loss of life can be found here:

- HEC-LifeSim (USACE): http://www.hec.usace.army.mil/software/hec-lifesim/

- RCEM (Reclamation): https://www.usbr.gov/ssle/damsafety/references.html

## C-1.3 Summary of Historic Flooding Events

In order to understand the potential for loss of life from flooding and the strengths and weaknesses of the available life loss estimation methodologies, it is important to understand what has led to loss of life during flood events in the past. All flood disasters are unique in many ways. However, there are a few commonalities that are consistent across most flood scenarios when it comes to how many people lose their life. These common factors include the intensity of the flooding and the time available for warning and evacuation. This section summarizes several historic flood disasters and describes the driving factors that influenced the loss of life for each scenario. Many of these events were used to inform the fatality rates used in LifeSim and RCEM. A more complete list of case histories, and how they were used to inform the fatality rates in RCEM can be found here: https://www.usbr.gov/ssle/damsafety/documents/RCEM-CaseHistories20140304.pdf.

### C-1.3.1 Teton Dam (1976)

Teton Dam, constructed, owned and operated by Reclamation, failed during first filling on Saturday June 5, 1976. The dam was located on the Teton River, about three miles northeast of the town of Newdale, Idaho. Teton Dam was a central-core, zoned embankment dam with a 305-foot structural height (not including 100 feet of additional foundation excavation), and contained 251,700 acre-feet of storage at the time of failure. The cause of failure was internal erosion of the core of the dam, initiated within the foundation key trench.

During the night of June 4, water evidently flowed down the right groin, and a shallow damp channel was noticed early on the morning of June 5. Shortly after 7 a.m. on June 5, muddy water was flowing at about 20 to 30 cubic feet per second (ft/s) from talus on the right abutment. At about 10:30 a.m., a large leak of about 15 cubic ft/s appeared on the face of the embankment, possibly associated with a "loud burst" heard at that time. The new leak increased and appeared to emerge from a "tunnel" about 6 feet in diameter, roughly perpendicular to the dam axis and extending at least 35 feet into the embankment. The tunnel became an erosion gully developing headward up the embankment and curving toward the abutment. At about 11 a.m., a vortex appeared in the reservoir, above the upstream slope of the embankment. At 11:30 a.m., a small sinkhole appeared temporarily, ahead of the gully developing on the downstream slope, near the top of the dam. Shortly thereafter, at 11:57 a.m., the top of the dam collapsed, and the reservoir was breached (figure C-1-1).

**Figure C-1-1.—Teton Dam failure.**

Failure of the dam released 240,000 acre-feet in about six hours. Flooding reached the town of Wilford, 8.4 miles downstream, within 30 minutes or so. Six fatalities occurred at Wilford and 120 of 154 homes were swept away. Flooding 12.3 miles downstream at Sugar City arrived at 1:30 p.m. and was described as a 15 foot high wall of water. At Rexburg, 15.3 miles downstream, flooding arrived at 2:30 p.m. and reached a depth of 6 to 8 feet within minutes. The photographs on figures C-1-2 through C-1-4 show the impacts of the flooding in these downstream areas.

Eleven fatalities occurred as a result of the dam's failure, although it is thought by some that the consequences could have been much worse if the dam had failed at night with no warning. Persons were present at the dam while it was failing, and

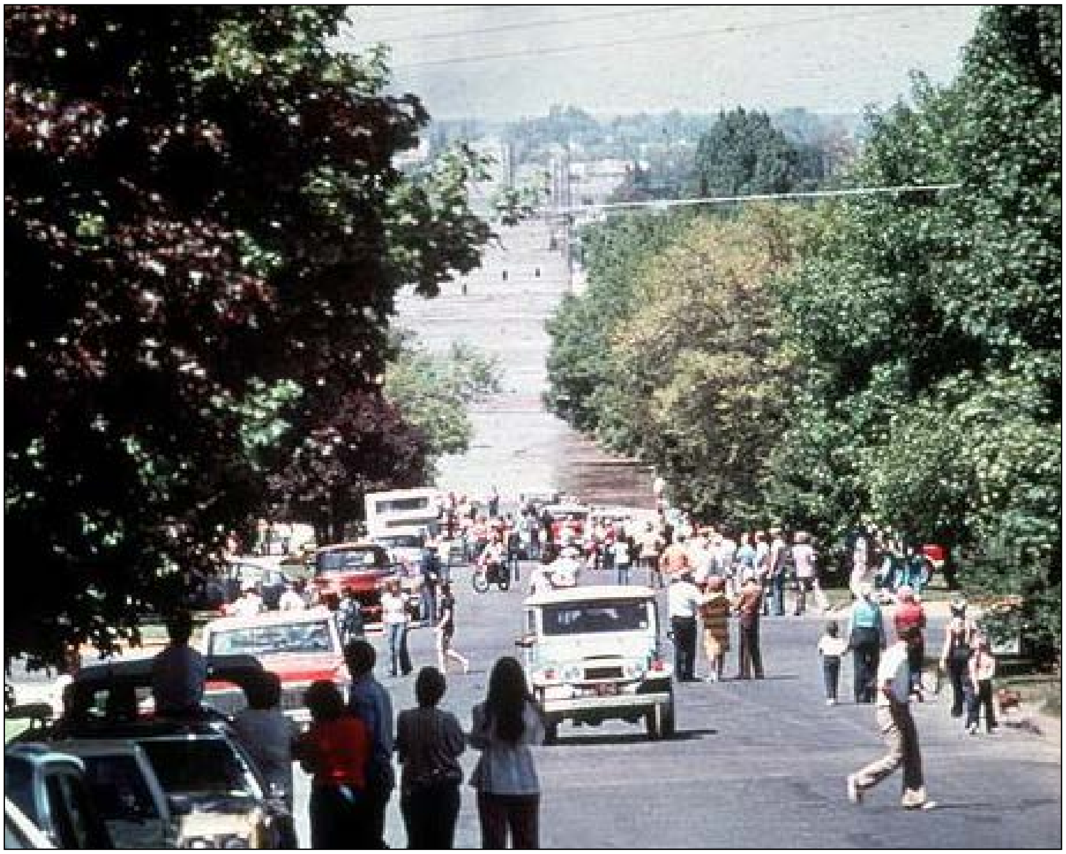

**Figure C-1-2.—Flooding and evacuation at Rexburg, Idaho.**

**Figure C-1-3.—Flood wave propagation across farmland.**

**Figure C-1-4.—Flooding aftermath at Rexburg.**

evacuation of downstream population was ordered thirty minutes to an hour prior to the full development of the breach.  More than 30,000 people in total were evacuated.  Some fatalities occurred when persons who had previously evacuated went back into the flood zone to retrieve possessions.

Out of the 11 fatalities, 6 died from drowning, 3 from a heart attack, 1 from accidental shooting, and 1 from self-inflicted gunshot wounds.  Table C-1-1 summarizes the flooding related information associated with the failure of Teton Dam.

**Table C-1-1.—Summary Table of Teton Dam**

| | |
|---|---|
| Warning time | 30 minutes to 1 hour for Wilford, Sugar City and Rexburg |
| Time of day | Daytime (noon) |
| Failure scenario | Internal erosion |
| Fatalities | 11 (6 direct, 5 indirect) |
| Fatality rate | 0.01 at Wilford, 0.0002 at Rexburg |
| Dam height | 305 feet |
| Reservoir storage | 240,000 acre-feet released during breach |
| Breach formation time | 1:30 |
| Downstream distance to PAR | 2.5 miles to Teton Canyon, 8.4 miles to Wilford, 15.3 miles to Rexburg |
| Maximum DV | About 1,600 $ft^2/s$ in Teton Canyon with fast rate of rise, 180 $ft^2/s$ at Sugar City, 30 $ft^2/s$ at Rexburg |

Note:  $ft^2/s$ = square feet per second.

Maximum dam failure discharge was about 2.3 million $ft^3$/s at Teton Canyon, 2.5 miles downstream from the dam. At Wilford, the flood is estimated to have attenuated to 1,060,000 $ft^3$/s.

The Teton Dam failure is a good example of a daytime failure with effective warning and evacuation. Flooding intensity was high in many locations, but overall fatality rates were fairly low due to the successes of the evacuation. A number of factors contributed to the effective warning and evacuation. The dam failure occurred during the weekend, in good weather. The closest residential population (Wilford) was more than 8 miles downstream, and the associated flood wave travel time allowed greater opportunity for warning/evacuation. Local media coverage of the event was excellent via a radio broadcast from the dam site. Also, most of the downstream population were members of the Mormon Church; church leaders took and active role in spreading the warning and coordinating the evacuation. The closeness of the community contributed significantly to the successful evacuation. It is important to note that almost half of the fatalities that occurred from this event were not from drowning, but were probably related to the stress of the evacuation.

### C-1.3.2 Francis Dam (1928)

St. Francis Dam (figure C-1-5) was located about 37 air miles north-northwest of downtown Los Angeles, California. The arched concrete gravity dam was constructed to augment the Los Angeles water supply.

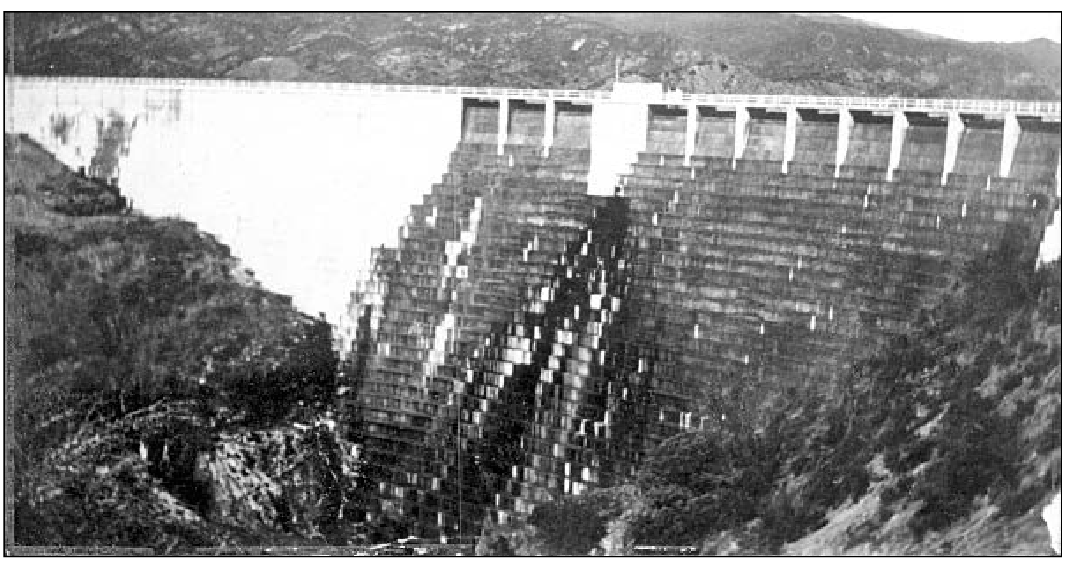

**Figure C-1-5.—St. Francis Dam before failure.**

St. Francis failed at about midnight, March 12-13, 1928. The flood traveled from the dam, 54 miles to the Pacific Ocean, in a five and one-half hour period during the early morning hours of Tuesday, March 13. The dam was completed in 1926, and was 2 years old when it failed. Failure of this young dam was caused by sliding on weak foliation within the schist comprising the left abutment, suspected of being part of an old landslide. The breached dam is shown on figure C-1-6.

**Figure C-1-6.—The breached St. Francis Dam.**

St. Francis Dam had a height of 188 feet, and the reservoir volume at the time of failure was about 38,000 acre-feet. The reservoir was about 3 feet below the crest of the parapet at the initiation of dam failure.

The failure sequence for this dam can be considered a worst-case scenario. Failure occurred in the middle of the night when many people would have been asleep, and darkness prevented people from observing the events that were occurring. The dam failed suddenly with no warning being issued before failure, and the entire contents of the reservoir drained in less than 72 minutes. The dam tender was unable to alert anyone of the danger. He and his family lived in the valley downstream from the dam and perished in the flood.

The Ventura County Sheriff’s Office was informed at 1:20 a.m. Telephone operators called local police, highway patrol, and phone company customers. Warning was spread by word of mouth, phone, siren, and by law enforcement traveling in motor vehicles.

Flooding was severe through a 54-mile reach from the dam to the ocean. The leading edge of the flooding moved at about 18 miles per hour near the dam and 6 miles per hour closer to the ocean. There were about 3,000 people at risk and about 420 fatalities, although the number of fatalities reported varies significantly. The fatality rate for the entire reach was about 0.14. It was much higher than this near the dam and much lower as the flood approached the Pacific Ocean. The dam was not rebuilt.

Two downstream areas, Powerhouse No. 2 and Edison Construction Camp, are of particular interest when it comes to understanding how the intensity of flooding resulting from this breach led to relatively high loss of life. The Powerhouse No. 2 located in the San Francisquito Canyon, about 1.4 miles downstream from the dam. The flood arrived at this location as a wall of water, about five minutes after the dam had failed with an estimated maximum flood depth of 120 feet and peak discharge of 1.3 million $ft^3/s$. The 60-foot tall concrete powerhouse was "crushed like an eggshell" and the area swept clean. The photographs on figures C-1-7 and C-1-8 show the powerhouse before and after the dam failure. Warning time was zero. Twenty-eight workers and their families lived at the site. There were three survivors. Table C-1-2 summarizes the flooding related information associated with the failure of St. Francis Dam.

**Table C-1-2.—Summary Table of St. Francis Dam**

| | |
|---|---|
| Warning time | Zero at Powerhouse No. 2 and the Edison Construction Camp |
| Time of day | After midnight |
| Failure scenario | Sudden failure |
| Fatalities | Unknown at powerhouse No. 2, 84 at Edison Camp, estimate of total flood fatalities ranges from 420 to more than 600 |
| Fatality rate | > 90% at Powerhouse No. 2, 56% at Edison Camp |
| Dam height | 188 feet |
| Reservoir storage | 38,000 acre-feet |
| Breach formation time | Instantaneous |
| Downstream Distance to PAR | 1.4 miles to Powerhouse No. 2, 18.6 miles to Edison Camp |
| Maximum DV | 2,960 $ft^2/s$ at Powerhouse No. 2 |

Another area of interest was the Edison Construction Camp (figure C-1-9) located 18.5 miles downstream where 150 men slept in tents along the banks of the river. The flooding at this location was described as a 60-foot wall of water. An effort to issue advance warning to the site was unsuccessful. As the flood approached, a night watchman became alerted and attempted wake the sleeping men, but it was mostly too late. An estimated eighty-four fatalities occurred at this site.

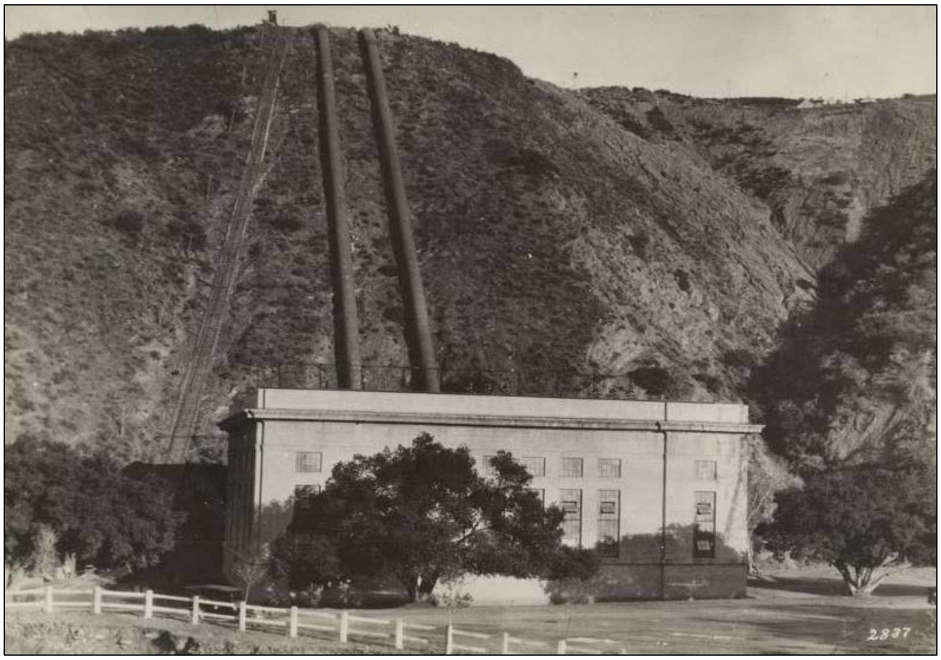

**Figure C-1-7.—Powerhouse No. 2 before its collapse.  (University of Southern California Digital Archive (c)2004).**

**Figure C-1-8.—Location of Powerhouse No. 2, area swept clean after flooding.**

**Figure C-1-9.—Aftermath of flooding at the Edison construction camp.**

Farther downstream, at the towns of Fillmore and Santa Paula, there was very intense flooding close to the river channel, but most of the developed areas at these communities were subjected to flooding that was much less severe.

This dam failure case is a classic example of very high intensity flow which can result from the sudden failure of a concrete dam (and which can also occur with a rapidly developing seismic overtopping breach of an embankment dam).  In the upper portions of the flooded reach, the leading edge of the flood coincided with the peak discharge, creating a “wall of water.”  This type of flood condition is extremely destructive and is associated with high fatality rates.

### C-1.3.3 Baldwin Hills Dam (1963)

Baldwin Hills Dam was an embankment structure that consisted of the main dam and three interconnected dikes, which formed a “ring” that enclosed the reservoir, as shown on figure C-1-10.  The dam which stored municipal water, was located in Los Angeles, California, and was 232 feet high with a crest length of 650 feet. Failure occurred on Saturday December 14, 1963 due to subsidence leading to internal erosion and piping.  Baldwin Hills Dam was 12 years old at the time of its failure.

The dam failed at 3:38 p.m. on a sunny, Saturday afternoon.  Seepage from the dam was detected at 11:15 a.m., and the process of issuing warning was well in advance of the breach.  Initially, there was an attempt to draw down the reservoir level and flooding from the releases began affecting residential streets at about

**Figure C-1-10.—Baldwin Hills Dam.**

12:20 p.m.  At 1:45 p.m., the decision was made to issue evacuation orders to downstream residents.  Neighborhoods were cordoned off and warning was strongly issued via emergency alert broadcasts, helicopters with bullhorns and by police officers going door to door.

Immediately downstream from the dam was a narrow flood channel, approximately 50 to 75 feet wide.  Numerous houses were damaged or destroyed in this area, but no fatalities occurred due to a successful evacuation.  At about 0.4 miles downstream of the dam was the large apartment complex community known as Village Green.  At Village Green, the flow spread laterally east and west, with an approximate width of 0.5 miles.  All of the five fatalities resulting from the failure of Baldwin Hills Dam occurred in the vicinity of Village Green, including three persons traveling together in a vehicle when overtaken by the flood.  Figures C-1-11 and C-1-12 show some of the downstream areas impacted by the dam breach.

A fire department helicopter was responsible for rescuing 18 people caught in the flooding at Village Green.  At least six of these persons may have died if they had not been rescued.

**Figure C-1-11.—Flooding downstream of Baldwin Hills Dam.**

**Figure C-1-12.—Flooding immediately downstream of Baldwin Hills Dam.**

The pre-evacuation PAR in the affected area was estimated at 16,500. At least 1,000 people are thought to have remained in the flood zone. Maximum breach discharge estimated to have been 35,000 to 40,000 $ft^3/s$. Flooding was reported to have been up to 30 feet deep initially, and maybe 5 to 8 feet deep further downstream with a velocity of 20 miles per hour (29 ft/s).

Damage in the Village Green area was extensive, but many structures remained standing after the flood. The narrow flood channel immediately downstream of the dam experienced high intensity flooding, although no fatalities occurred in this area.

The Baldwin Hills Dam failure illustrates an example of dam failure flooding in an urbanized area with a reasonably good warning and evacuation effort. Although the flooding was severe at impacted locations, the overall fatality rate was low. Table C-1-3 summarizes the flooding related information associated with the failure of Baldwin Hills Dam.

**Table C-1-3.—Summary Table of Baldwin Hills Dam**

| | |
|---|---|
| Warning time | 1 hour, 50 minutes |
| Time of day | Daytime |
| Failure scenario | Subsidence of foundation leading to internal erosion |
| Fatalities | 5 |
| Fatality rate | 0.0003 |
| Dam height | 232 feet |
| Reservoir storage | 738 acre-feet |
| Breach formation time | About 4:30 p.m., assuming that initial seepage discovered at 11:15 a.m. was the initiation of the breach |
| Downstream distance to PAR | Beginning immediately downstream of the dam and extending for 3 miles when considering the extent of potentially lethal flood flow. |
| Maximum DV | 147 $ft^2/s$ based on an account of 5-foot deep flooding moving at 20 miles per hour. May have been higher in the narrow channel just below the dam. |

## C-1.3.4 Laurel Run Dam (1977)

Laurel Run Dam was located on a stream known as Laurel Run in west-central Pennsylvania, near the town of Johnstown. Figure C-1-13 show the location of the dam. The earthen dam was 42 feet high with a 623-foot crest length, and the reservoir typically held about 300 acre-feet of storage. Due to embankment overtopping, 450 acre-feet of storage was reported to be in the reservoir at the time of its failure.

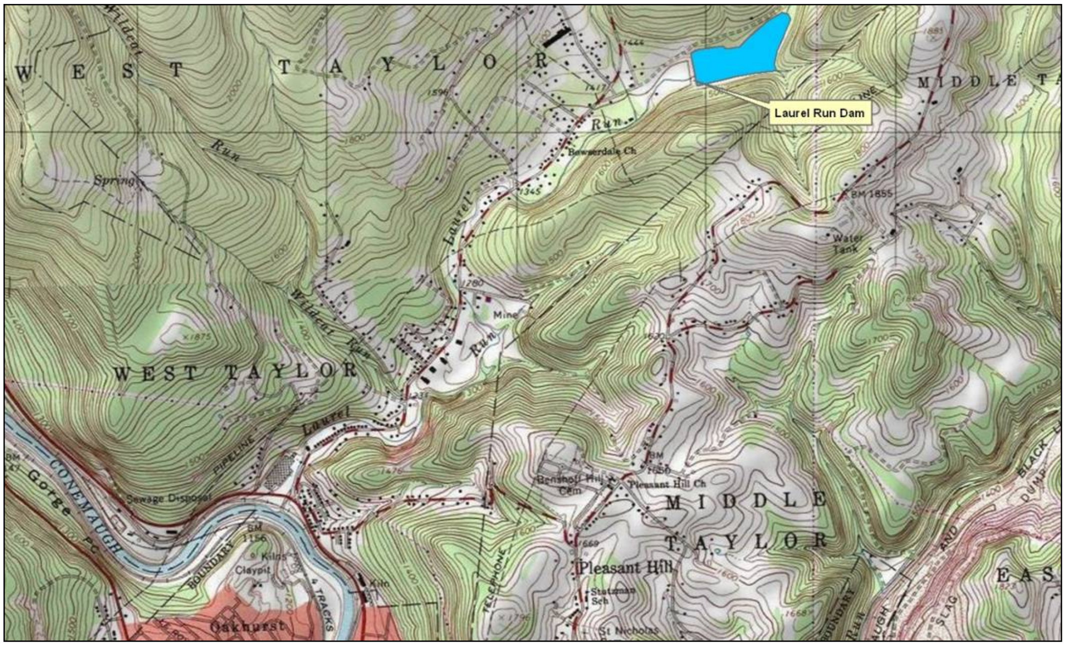


**Figure C-1-13.—Laurel Run Dam location map.**

Laurel Run had the largest reservoir of the seven dams, which failed between July 19 and 20, 1977, in the vicinity of Johnstown, Pennsylvania, as a result of heavy rainfall and flooding (figure C-1-14). The failure of Laurel Run caused the greatest number of fatalities from this hydrologic event. Failure is claimed to have failed at 2:35 a.m. on morning of July 20 after a period of locally heavy rain. A rainfall total of 11.82 inches occurred in 10 hours, and this was estimated to be between a 5,000- to 10,000-year event. About 41 people were killed in the town of Tanneryville, located in a three-mile long valley, immediately downstream of the dam. Most residents were asleep when the dam failed, and no warning was issued. In addition, the rain and night-time conditions limited any escape. Many of the homes in Tanneryville were either damaged or destroyed.

Another dam, Sandy Run Dam, was also responsible for several deaths. Overall, there were more than 70 deaths in the area resulting from the effects of this regional flood. The town of Johnstown along the Conemaugh River, famous for the flooding from the 1889 failure of South Fork Dam, was heavily impacted by floodwaters. Damage to Johnstown was extensive, but without fatalities. The area experienced widespread power outages the night of the flood. Telephone service was intermittent in some communities as well. Laurel Run Dam was not rebuilt.

A hydraulic re-creation done by Chen and Armbruster (1980) estimates velocities at the downstream end of Laurel Run to have been 24 ft/s. Peak breach discharge was estimated to have been approximately 56,000 $ft^3/s$. A gage below Laurel Run

**Figure C-1-14.—Remains of Laurel Run Dam.**

Dam, at Coopersdale Bridge in Tanneryville, indicated that the flood had attenuated to 37,000 $ft^3/s$ maximum discharge.  Figure C-1-15 shows the post-flooding condition at Tanneryville.

**Figure C-1-15.—Flooding aftermath at Tanneryville.**

The dam failure flood destroyed many buildings, but the area was not completely swept clean. Flood velocity along Laurel Run was estimated to have been about 24 ft/s. Some information is available in a United States Geological Survey (USGS) report which cites maximum stage at various locations along Laurel Run, but it is difficult to establish estimates of actual flood depths due to limited ground surface elevation data along the Laurel Run stream.

This case history is a good example of a dam failure that occurs at night with no warning, and with adverse weather conditions. Often, with larger, high profile dams, monitoring of the reservoir and an increased level of awareness can result from high reservoir inflow conditions such as would occur during periods of prolonged rainfall. When estimating fatalities for high profile dams due to hydrologic conditions, advance warning and a high percentage of evacuation are often assumed, and this can result in lower fatality rates than would be assumed for static and seismic failure modes. However, for smaller dams/reservoirs such as Laurel Run, drainage basin inflows may be flashy, meaning that the reservoir may rise rapidly and may be undetected until overtopping occurs and the dam breaches. For these situations, fatality rates can be assumed to be high, based on the assumption of minimal warning and high intensity flood flow. Table C-1-4 summarizes the flooding related information associated with the failure of Laurel Run Dam.

**Table C-1-4.—Summary Table of Laurel Run Dam**

| | |
|---|---|
| Warning Time | No warning |
| Time of Day | Dam failure at 2:35 am |
| Failure Scenario | Overtopping |
| Fatalities | 41 from failure of the dam, more than 70 regionally |
| Fatality Rate | 0.27 |
| Dam Height | 42 feet |
| Reservoir Storage | 300 acre-feet, 450 acre-feet at time of failure |
| Breach Formation Time | Unknown |
| Downstream Distance to PAR | Tanneryville was located along a 3-mile valley between the dam and the Conemaugh River confluence. |
| Maximum DV | Unknown |

## C-1.3.5 New Orleans – Hurricane Katrina (2005)

In 2005, Hurricane Katrina caused one of the worst catastrophes in recent U.S. history resulting in more than 1,100 fatalities in Louisiana alone. The photographs on figures C-1-16 and C-1-17 show some of the impacts of the flooding. The paper “Loss of Life Caused by the Flooding of New Orleans after Hurricane Katrina: Analysis of the Relationship Between Flood Characteristics


**Figure C-1-16.—Levee failure caused by Hurricane Katrina.**

**Figure C-1-17.—Flooding from Hurricane Katrina.**

and Mortality" by Jonkman et al. (2009) presented an analysis on the loss of life caused by Hurricane Katrina in the city of New Orleans, Louisiana. The following discussion presents some of the ideas and findings of that paper.

Data on the locations, conditions, and characteristics of 771 of the fatalities were available for the study. Of these 771 fatalities that had data associated with them, it was determined that approximately 1/3 of those fatalities either occurred in hospitals or shelters within the flooded area or outside of the flooded area altogether. This meant that 2/3 of these fatalities occurred within the flooded areas and were mostly due to drowning.

Due to the warnings that went out prior to Hurricane Katrina making landfall, it is estimated that 430,000 vehicles had left the metropolitan area using the primary roads. In addition, another 10,000 to 30,000 vehicles left the area by secondary roads. This means an estimated 1.1 million people left the area prior to landfall, which equates to 80 to 90 percent of the PAR in the area.

The Hurricane Katrina study looked at age, gender, and race and the role they played in the fatalities. There were 853 fatalities that had some data available for these comparisons. Of most significance was the amount that age factored in to the fatalities. Of the 829 fatalities that the age was known, most were elderly. The report states that less than 1 percent of these fatalities were children (0-10 years old) and only about 15 percent were less than 51 years of age. This means that nearly 85 percent of the fatalities were over the age of 51, 60 percent were over the age of 65, and almost 50 percent were older than 75.

The data also showed that gender and race did not play a significant role in the Hurricane Katrina fatalities. The ratio of fatality rates for men and women were similar to the percentage of men and women that resided in the area before the hurricane. A similar comparison was found for race.

A second study by Jonkman and Kelman (2005) researched fatalities for small-scale river flooding in the United States and Europe. Their findings showed that males have a higher mortality rate in those situations. This was attributed to males taking unnecessary risks during those flood events. Their study also showed that the fatality rates for the elderly did not show that they were more at risk. These findings contradict the results for Hurricane Katrina that show age does have an effect on fatality rate and gender does not. This can be explained by the large-scale and unexpected flooding that took place in New Orleans. During a large-scale event like Hurricane Katrina, people (males in particular) are less likely to partake in risky behaviors due to the extreme circumstances and survival is more related to endurance in these extreme conditions. This helps explain the high fatality rate for the elderly in New Orleans.

Of the 771 recorded fatalities in the metropolitan area, 624 (81 percent ) were inside the flooded areas and 106 of those were determined not to be a direct impact of the flooding since they were found in hospitals and shelters. The remaining 518 fatalities that were recovered (67 percent of total recovered) were attributed to direct impact of the flooding (drowning, physical trauma, or building collapse). Of these fatalities, it was determined that many were near large breaches in the levees and therefore, were in areas that experienced deeper water levels.

The highest fatality rates computed in the metropolitan area were in the St. Bernard bowl (Lower 9th Ward), which had rates of 5 percent to 7 percent . This is a low-lying area that is near two large breaches in the levees. This agrees with past research that shows fatality rates are usually highest near breaches as well as areas that experience deep water levels, fast rising waters, and the collapse of buildings. In the Lower 9th Ward, the two large breaches allowed water to enter the area with great force, causing many buildings to collapse.

The study concluded that fatality rates were highest: (1) near breaches in the levees due to the combination of depth, velocity, and less reaction time and (2) in areas with the greatest flood depths. One difference between this study and similar studies by Jonkman et al. (2009) in Europe was that the impact of how quickly the water rose was insignificant in determining the fatality rate. Finally, the study concluded that the fatality rates for Hurricane Katrina were in line with historic events. The overall fatality for this and the historic events analyzed by Jonkman et al. (2009) is approximately 1 percent of the PAR.

### C-1.3.6 Quail Creek Dike (1989)

Quail Creek Dike, along with Quail Creek Dam, impound the waters of Quail Creek Reservoir, an off-stream storage facility located in Washington County, Utah, near the town of St. George. Construction of the dike was completed in 1985. The dike, which was 78 feet high, failed on January 1, 1989 at 12:08 a.m. About 25,000 acre-feet of water was released from the reservoir which had a capacity of 40,000 acre-feet. Figure C-1-18 shows the breached dike. Based on eye-witness accounts, the first indication of failure was observed the previous day, although seepage related issues had been a concern for some time.

The breach released a flood that surged down the Virgin River in waves which were 10 to 40 ft high, inundating parts of St. George and several other small towns, including Bloomington. Three small bridges were swept away, along with a 98-year-old irrigation dam. The flood also disintegrated half a mile of Utah Route 9, where water thundered through a narrow highway cut adjacent to a bridge about a mile downstream. The flood surge wiped out utility lines at the crossing, including a newly-completed 8-inch gas line.

**Figure C-1-18.—View of the breached Quail Creek Dike.**

Prior to the breach, the Washington County Water Conservancy District, which owns the project, worked for 12 hours to stanch a leak at the toe of the embankment. It initially was spilling 25 gallons per minute. Late in the afternoon of December 31, Washington County Water Conservancy District officials advised the county emergency management director to prepare for downstream evacuations based on unprecedented observations of muddy seepage. The seepage increased to 600 gallons per minute by about 11:00 p.m. and the dike was breached shortly after midnight. No fatalities occurred. Residents located 15 miles downstream had been warned and evacuated. Late in the afternoon on the December 31, County emergency managers called for downstream evacuations. 1,500 people were evacuated. There were no fatalities.

The 80-foot wide breach was reported to have formed in two hours and released a peak discharge of 60,000 $ft^3/s$. Flood depths close to the dam were estimated to have been 61 feet high, traveling at 18 ft/s DV equal to 1,098 $ft^2/s$). 20,000 acre-feet of storage were drained in five hours. Flooding followed the course of the adjacent Virgin River. Flood flows reached Bloomington, 16 miles downstream, in four hours with 5 foot flood depths (DV equal to about 29 $ft^2/s$).

This dam failure case history is very significant in that it represents the best possible outcome. The in-progress failure of the dike was observed and monitored. Advance warning was issued, and the downstream population was evacuated without incident. Flooding through downstream areas was significant and of high enough intensity to have produced fatalities. The overall fatality rate of zero is a best-case scenario. Table C-1-5 summarizes the flooding related information associated with the failure of Quail Creek Dike.

**Table C-1-5.—Summary Table of Quail Creek Dike**

| | |
|---|---|
| Warning time | Adequate warning was issued, evacuations were ordered well in advance of the breach |
| Time of day | Night time |
| Failure scenario | Static failure, internal erosion |
| Fatalities | 0 |
| Fatality rate | 0 |
| Dam height | 28 feet |
| Reservoir storage | 40,000 acre-feet |
| Breach formation time | Unknown, but increased seepage leading to the breach occurred for about 12 hours |
| Downstream distance to PAR | 16 miles |
| Maximum DV | 1,098 $ft^2/s$ downstream of dam, 29 $ft^2/s$ at Bloomington |

# C-1.4 General Loss of Life Methodology Overview

## C-1.4.1 Life Loss Estimation: Selecting Scenarios

Failure scenarios for dam safety risk analysis are typically identified from the findings of a Potential Failure Mode analysis. Failure modes usually fall into three categories: static, seismic and hydrologic. Within each category, there may be specific details for a failure mode, such as: overtopping due to a 50,000 year inflow, liquefaction and slumping of a dam crest due to seismic loading, or internal erosion due to seepage induced piping along the outlet works conduit. There are many possible, site specific potential failure modes for dams and levees and these are just a few examples. In addition to the basic scenario selection, relevant sub-scenarios can be developed to aid in sensitivity analysis and to estimate ranges of possible outcomes. Life loss estimates based on the evaluation of sub-scenarios can take the form of a highly developed probability distribution, or can be simplified into high, middle and low-end estimates.

Depending on the needs of the study, subscenarios can be based on:

- **Time of day.—**The time of day affects where people may be located and can affect the ability of the PAR to respond to warning and to effectively evacuate. Historically, more fatalities have occurred during night time flood events, due to people sleeping, darkness, decreased ability to spread warning and a slower evacuation response.
- **Weekday/weekend.—**The day of the week can, in some cases, have an effect on life loss estimates. Recreational areas such as campgrounds, or along rivers where fishing or boating are popular, will see higher PAR numbers on weekends.

- **Seasonal variation.—**For areas with significant recreational (transient) PAR, there may be large differences in numbers of PAR present between summer and winter months.

Additional sub-scenario sensitivity analysis can be performed by evaluating variations in initial reservoir levels for dams or river stage for levees. Variations in breach parameters, such as breach width and breach formation time can also be evaluated as sub-scenarios. Figure C-1-19 shows reservoir levels over a several year period for an example of a dam where initial reservoir sub-scenarios may be valuable. The failure scenario is based on a static condition, or a "sunny day failure." The failure mechanism is internal erosion. Typically, a sunny day failure will use an initial reservoir level at top of active conservation, or top of joint use if the joint use designation exists for a particular dam. For this example, the dam has a top of joint use elevation of 6769 feet. As can be seen in figure C-1-19, the reservoir level (in blue) reaches joint use elevation 6769 feet every year, but only stays at that level for a short time. During winter months, the reservoir drops to elevation 6760 feet. Risk analysis sub-scenarios for the sunny day failure condition might include a scenario with the reservoir at top of joint use elevation 6769 feet, and one with the reservoir at an average annual level (in red) of about 6764.5 feet, or some other exceedance level. Note that in practice, an estimation of average annual reservoir level should contain as many years of record as possible. The several years of data depicted on figure C-1-19 example is shown only for clarity.

Also note that this example represents the case of a very large storage reservoir, where the several feet of difference between joint use pool and average annual levels represents significant differences in storage volume. Most internal erosion failures occur when the reservoir is full, or nearly full. The use of varying levels as sub-scenarios has greatest application to static failure of large reservoirs, and may be even more useful for seismic failure scenarios which could be considered likely to occur throughout a wider range of reservoir levels.

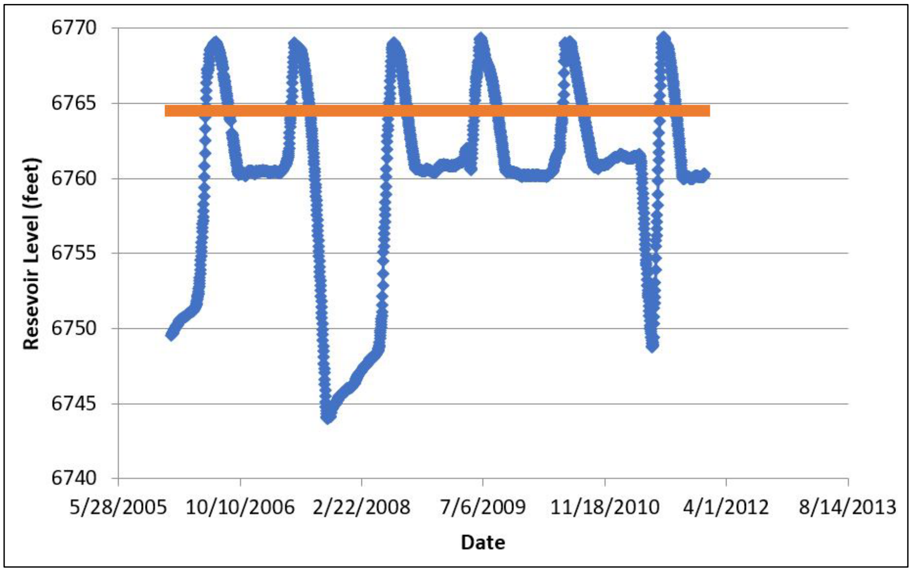


**Figure C-1-19.—Example reservoir level fluctuation plot.**

## C-1.5 Flood Inundation Modeling

Flood inundation modeling is a critical part of the consequence estimation process. The flood inundation analysis provides estimates of the inundation areas, the intensity (depths and velocities) of flooding, and flood wave travel times. Inundation analysis should be performed by a specialist who has a broad understanding of hydraulic modeling, dam safety, consequence assessments, and Geographic Information Systems (GIS).

Often, when conducting a risk analysis, an inundation study may exist for a particular dam or levee. An assessment of the existing study should be made to decide whether the study results can adequately represent the scenarios to be evaluated during the risk analysis. The following items should be considered when assessing the adequacy of an existing inundation study:

- **Failure scenario.—**Is the failure scenario portrayed in the existing study comparable to the desired scenarios for the new study? For example, a new inundation study may be justified if the current study seeks to evaluate a sunny day failure with normal reservoir levels, but the existing inundation study is based on a Probable Maximum Flood (PMF) inflow where the inflow volume of the flood increases the breach outflow significantly over sunny day conditions.

- **Breach parameters.—**Are the breach parameters for the existing study realistic? Are they significantly different from the desired breach parameters of the failure scenario to be evaluated by the risk analysis? An example might be a situation that involves a large concrete gravity-arch dam. The existing inundation assumed failure of the entire dam, all the way to the foundation. Recent finite element structural analysis indicates that the dam, when subjected to the most severe of loading conditions would only breach to the upper one-third of its height. In a situation like this, a new inundation study may be justified.

- **Downstream conditions.—**There are many examples of older inundation studies that were performed with one-dimensional (1D) hydraulic models where the downstream terrain contains populated areas that are very flat. The modeling cross sections may extend over very wide areas, sometimes exceeding several miles in width. The cross sections may even contain vertices or bends in the cross sections which extend uphill in order to artificially create a "lip" in the cross section so that it will hold water. Two-dimensional (2D) hydraulic models do a more accurate job of modeling flood flow over wide flat flood plains, but 2D models did not begin to be used for flood inundation applications until about the late 1990's. For these cases, a new inundation study, using 2D modeling and appropriate terrain data may improve the accuracy in estimating overall flood extent, the intensity of flooding, and travel times and may be warranted (or beneficial). Figure C-1-20 provides an example in which a 2D model would be more appropriate.


**Figure C-1-20.—Example of a 1D inundation study where a 2D study would be most appropriate.**

Often, when time and budget are limited, available inundation information must be used along with judgment-based assumptions, to approximate downstream impacts from flooding.  This is especially true for more basic levels of risk analysis studies.  High level risk analysis almost always justifies using appropriately developed inundation information.

## C-1.5.1 One-Dimensional and Two-Dimensional Hydraulic Modeling for Flood Inundation Analysis

The following discussion contains general information regarding 1D and 2D hydraulic modeling for flood inundation applications.

1D hydraulic models have traditionally been the standard for flood inundation applications.  Recently, 2D modeling has become more common practice when the conditions of the study are such that 1D modeling cannot properly capture certain aspects of the flood characteristics, especially multi-directional flows and large volume floodplains.  Details of when 1D or 2D modeling should be applied to support consequence estimation are provided below.  1D modeling uses a river centerline to define the flow path, and cross sections to define the channel geometry.  An example of a 1D model layout is shown on figure C-1-21.

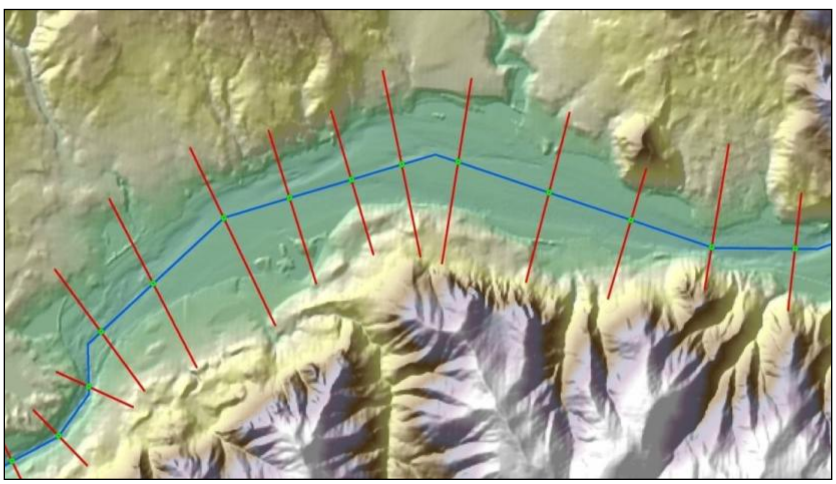

**Figure C-1-21.—1D hydraulic model layout.  River centerline shown in blue and cross sections in red.**

1D modeling is typically applicable in the following situations:

- River systems where dominant flow directions and forces follow the general river flow path (i.e. well-defined channels).
- Steep streams that are highly gravity driven and have small overbank areas.

- River systems that contain numerous bridges, culvert crossings, weirs, dams and other gated structures, levees, pump stations, etc., and these structures impact the computed stages and flows within the river system.

- Medium to large systems (50 or more miles long) where the time required for the flood wave to fully propagate through the system is days or weeks. While 2D modeling can be used here, the time required to run 2D models for these situations can be restrictive.

- Areas where the available data does not support the potential gain of using a 2D model. For example, if detailed overbank and channel bathymetry does not exist, or the only data available includes detailed cross sections at representative locations, many of the benefits of the 2D model will not be realized.

When a 1D model is run, the output is a 1D water surface profile as shown on figure C-1-22.

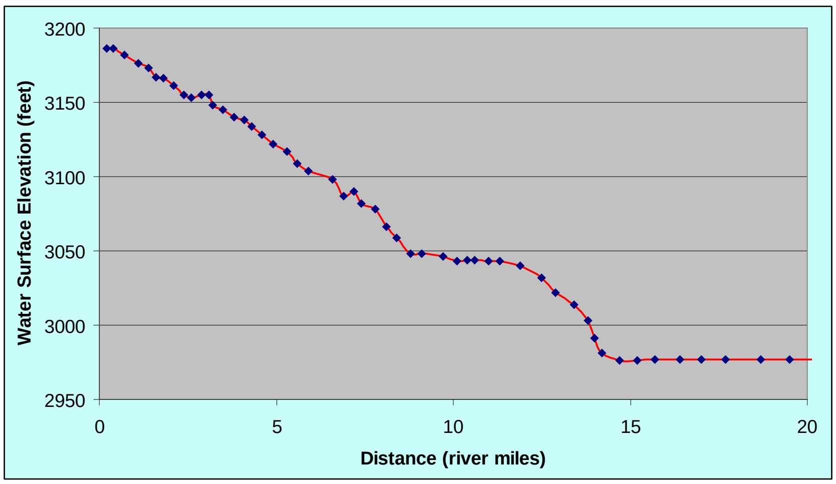


**Figure C-1-22.—1D hydraulic model maximum water surface profile output.**

The 1D model calculates a single water surface elevation for each cross section. In order to create a flood inundation boundary, this 1D result is imposed on a 2D surface. This is accomplished by interpolation. Before the advent of GIS systems, inundation boundaries were hand-drawn on topographic maps, using contour lines to aid in the delineation of the flooding extent. Modern techniques make use of GIS technology. Typically, a Triangular Irregular Network (TIN) methodology is employed to develop the interpolated flood inundation boundary as shown on figure C-1-23.

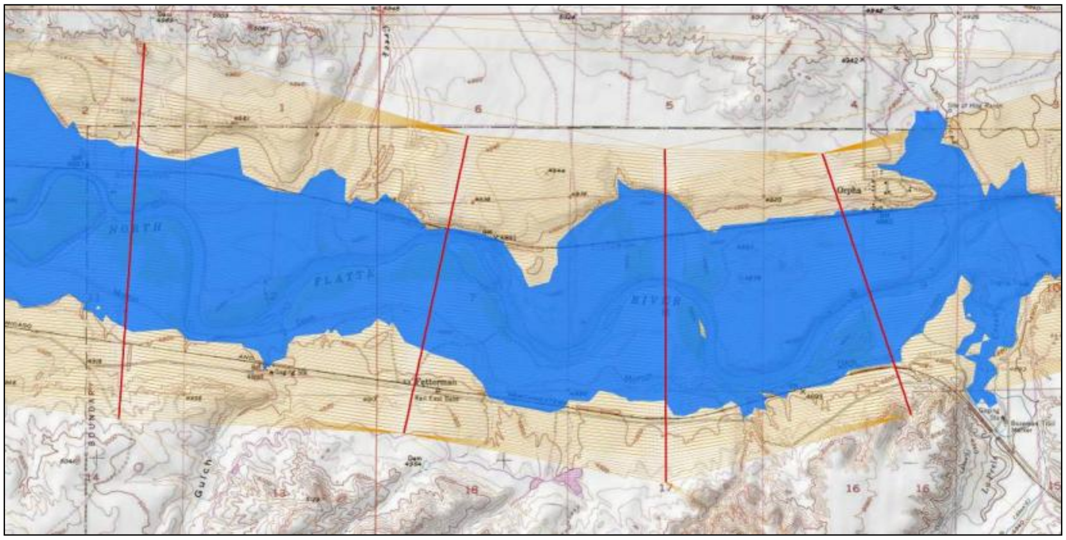


**Figure C-1-23.—TIN surface and interpolated maximum inundation boundary.**

Advantages of 1D hydraulic models are:

- Relatively short model run time – typically minutes to hours
- Long reaches are more easily accommodated
- Hydraulic structures such as dams, culverts, bridges, and levees can be easily included

1D model disadvantages are:

- Does not calculate velocities that are not parallel to the stream centerline
- Does not appropriately handle lateral spreading of flows in very flat flood plains
- Inundation extents are interpolated
- Conditions are averaged over an entire cross-sectional area

2D models have significant differences when compared to 1D models. A 2D model does not use a river centerline or cross sections. Instead, it represents a continuous terrain surface and flow introduced to the model follows the path of least resistance, letting gravity and momentum direct its progression. Every inundated point in a 2D model is a calculated point, so no interpolation is performed. An example of 2D flood inundation modeling is shown on figure C-1-24.

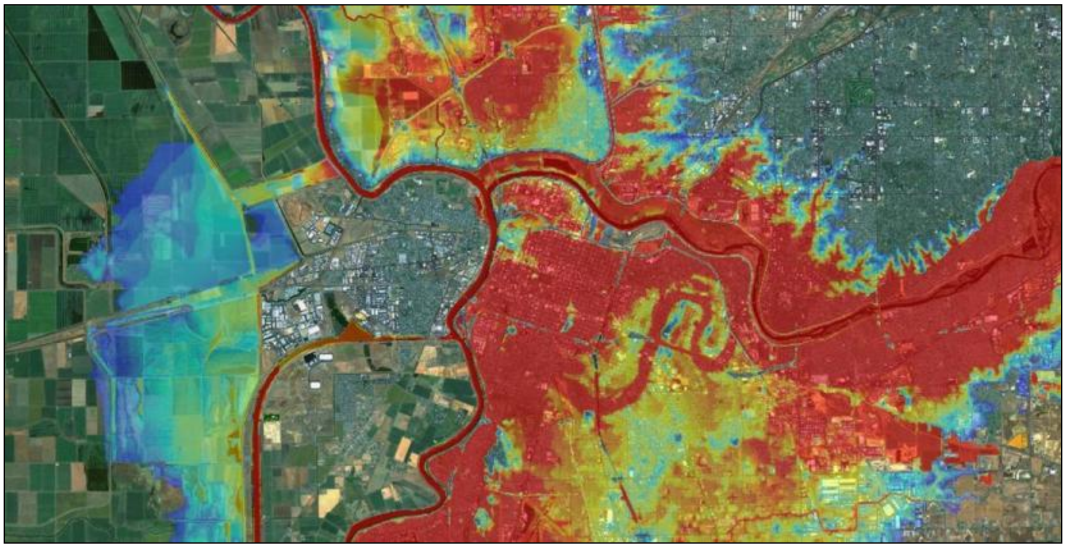

**Figure C-1-24.—2D flood inundation example.**

The inundation depicted on figure C-1-24 is 2D in that there is a high degree of out of bank flow, lateral spreading and spilt flow. Advantages of 2D hydraulic models:

- 2D modeling works better for areas with flat terrain where lateral spreading of flow is significant (alluvial fans, areas behind levees, etc.).
- Complex split flow situations (including highly braided streams) are more accurately handled with a 2D model.
- Bays and estuaries where water will flow in multiple directions due to tidal fluctuations and water flows into the bay/estuary at multiple locations and times.
- Flood depths and velocities are computed for every point rather than interpolated between cross-sections. This provides more accurate information which can have a significant impact on the consequence assessment.

2D model disadvantages are:

- Relatively long model run times – multiple hours to days to run a large simulation.
- Modeling extent and resolution (size of the model) are restricted by computer hardware limitations – the larger the model, the longer it takes to run a simulation, and it can be difficult to run long river reaches at a reasonable terrain resolution.

- Model simulation time is also restricted by computer hardware limitations - this is particularly true for hydrologic scenarios where spillway releases from a dam occur for a long period of time prior to the initiation of a dam breach.

Note that highly detailed inundation modeling may not be justified when the estimation of life loss consequences involves lightly populated areas. The typical model is often a combination of 1D and 2D to take advantage of increased 2D accuracy where it is needed, and improved 1D speed where it is not.

### C-1.5.2 Breach Modeling

Representation of the dam or levee breach can be an extremely important consideration for consequence assessments. Breach parameters can affect the peak breach discharge, flood depths, and the timing of the downstream flood arrival in a very significant way. Examples of breach parameters are below:

- Breach width
- Bottom elevation
- Location
- Side slopes
- Erosion rates
- Material erodibility
- Time of breach
- Failure mode, etc.

Each of these parameters can have impacts on the amount of warning time available for evacuation as well as the flood severity for those remaining in the area. Flood waters near the breach are highly sensitive to the breach parameters as depth and velocity vary greatly with the speed of breach formation. Further from the breach the impact is lessened, but may still be quite significant depending on topography and location of population centers.

The state of the practice for dam breach modeling using HEC - River Analysis System (RAS) (a very popular and freely available tool) is described in TD-39 (Brunner 2014). Unfortunately, levee breach modeling does not have such a document. The USACE modeling Mapping and Consequence Production Center has a standard operating procedure (SOP) report (Ubben 2016) in draft that lays out their method for levees using the simplified physical breach tool in HEC-RAS. These and other best practices are outlined below.

Many of the dams in the USACE and Reclamation inventories have been extensively studied, analyzed and monitored. Potential failure modes have been developed, multiple inspections have been performed, details of design and construction have been compiled and the mechanics of failure modes have been analyzed. As a result of all of this, much is known about the dam and senior engineers assigned to a given dam may have opinions on how the failure might occur. Consultation by an inundation modeling analyst with senior engineers assigned to a particular dam or levee is vital when developing site specific breach parameters, based on knowledge of the dam, its composition, performance and its response in regard to potential failure modes. This approach may include the analysis of empirical breach equations, but in many cases is just based on informed opinions about the specific dam in question. There are a lot of uncertainties in the prediction of breach parameters and this approach can be valuable in that it enables collaboration and helps to build consensus between the inundation/consequences analyst and other risk analysis team members.

#### *C-1.5.2.1 Breach Analysis and Definition*

A breach typically develops in three phases from a modeling perspective: initiation, rapid downcutting, and widening (figure C-1-25). The rapid downcutting and widening phases combine to make up what is often known as the breach formation time.

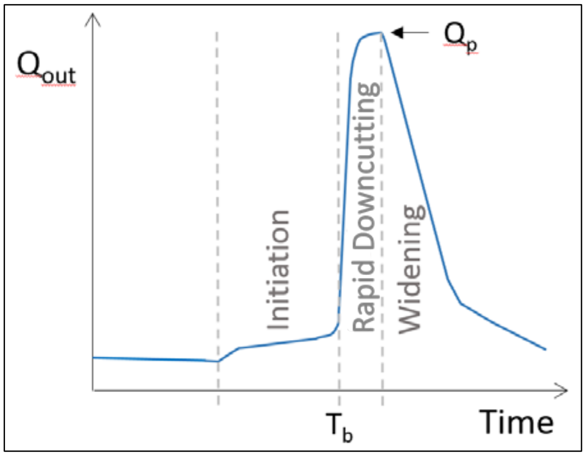


**Figure C-1-25.—Breach hydrograph showing phases of breach.**

The *breach initiation* phase begins with the first flow of water over or through a dam that produces observable erosion with the potential to progress and cause dam failure. During the breach initiation phase, the zone of active erosion is downstream from the point of hydraulic control of the flow, so outflow rate changes only in response to changes in the driving reservoir conditions, not as a result of erosion. As breach initiation proceeds, the zone of active erosion generally moves upstream (e.g., headcut or surface erosion during overtopping flow). The breach initiation phase ends when the active erosion front reaches the crest and upstream face of the dam, thereby producing a rapidly accelerating breach outflow and typically unstoppable failure of the dam (Reclamation 2017).

The breach formation phase begins when erosion and outflow begin to increase rapidly, often due to a headcut reaching the upstream side of the embankment or a piping roof collapsing. During this rapid downcutting phase, the breach widens and deepens quickly. It ends when flows begin to level off and the peak flow (Qp) has been released. In the final phase the breach has usually reached its full depth and continues to widen as long as a driving hydraulic head is available. Widening is accomplished through a combination of foundation and structure erosion and stability failures. The widening can be stopped either by exhaustion of the headwater, or raising of the tailwater that reduces breach inflow to non-erosive velocities. In some levee systems, widening can continue for days or months until flows subside or the breach is repaired. This is typically not important for consequence studies because the peak flood depth and maximum flood extent are realized much earlier.

Most hydraulic breach models use the breach formation time (rapid downcutting and widening phases of the breach). As part of the risk assessment process, the inundation analyst or risk assessment team should decide on the method of initiation and determine Tb (time of breach). These will be taken as input assumptions to the hydraulic model.

Often it is assumed that the time of breach coincides with the peak stage or inflow to the reservoir or peak flow in the river passing the levee breach site. This assumption may be refined based on the team's understanding of the failure mode to look at breaches before or after the peak inflow. This change can impact the ultimate breach size, outflow and warning time. Sensitivity of the results to this assumption should be demonstrated during the analysis. For overtopping scenarios, various flood frequency events are evaluated to select an inflow that produces the amount of overtopping required to cause a breach, but no more. This approach maximizes the annual failure probability by evaluating the most frequent event that might cause an overtopping breach.

#### *C-1.5.2.2 Types of Breaches*

As discussed elsewhere in this document, a structure may fail by many different failure modes. Different failure modes will likely require different user-defined breach parameters or different models to represent them correctly. For example, some breach models can simulate overtopping and breach of an embankment, but not piping-induced breaches. Breach parameters and models should be carefully selected to represent the breach type as accurately as possible. Currently, wave overwash breaching and riverbank erosion leading to levee collapse are not handled well by any widely used models. Conversely, models for piping, overtopping, and slope stability failures are common and widely accepted.

Breaches of embankment dams and levees are comparable in many ways, due to the similar physical processes acting on them, e.g. erosion. However, there are also some important differences. Possibly the most important difference is understanding the impact of breach location on the potential consequences. Given that dams are usually relatively short in crest length, and that water typically flows along the path of the river downstream from a breach, the potential consequences are rarely sensitive to the actual location of the breach within the dam structure. On the other hand, levees are typically much longer structures. Often there are communities located adjacent to the levee, sometimes multiple communities, miles apart. If a breach occurs right next to a community, the depths and velocities can be high enough to wash structures away. If a breach occurs downstream from a community, it may be possible that the community does not get flooded at all.

Another important difference is how the water flows through the impacted area. When a dam breaches, water generally moves downstream, giving the flood time to attenuate as it travels and reducing impact for communities far from the dam. The peak flow at a location determines the maximum inundation extent and depths. Levee breaches on the other hand, often fill a finite volume that may be thought of like a bathtub. The peak flow is only a concern for population near the breach. Maximum depth is rather a product of the interaction between basin volume, inflow volume and the elevation of containment (lowest levee crest). The tailwater of a levee breach can also have a dramatic effect on the breach outflow if begins to equalize with the river stage. These differences make dam and levee breaches sensitive to different parameters.

Another major difference is the quality and quantity of geotechnical information for more detailed studies. Many levees have only generalized material properties and sporadic or no material samples, while dams typically have records of sampling from borrow sources and placed materials. Locating a possible breach is also more challenging for levees than dams due to their extra length, adding complexity to selection of material properties. Because erosion is sensitive to properties like moisture content at time of placement, level of compaction, and amount of fines, knowledge of these properties impacts the model uncertainty. Hydraulic modelers should seek significant input from those familiar with the structure's material properties before attempting a more detailed breach model.

Some differences between dams and levees of lesser importance are: 1) Riverine levees have a sloping water surface instead of the assumed flatwater of a reservoir or ocean; 2) In steep rivers the flow direction in the river can impact the breach formation if the momentum is strong enough to direct erosive energy at the downstream end of the breach. That side may erode faster than the upstream side. In slower moving rivers the lateral acceleration into the breach opening will overshadow this effect; and 3) Breach outflow is usually set by a weir coefficient which can be much different for a dam than a levee given the difference in height and other geometries.

### *C-1.5.2.3 Selecting the Appropriate Breach Model*

Typically, consequence analysis will rely on some form of hydraulic data such as a 1D breach hydrograph or 2D depth and velocity grids. The level of detail in the hydraulic modeling should be commensurate with the decision being made and method of consequence analysis. The complexity of the breach modeling should also be scaled according to the detail/accuracy levels below and type of structure as shown in table C-1-6.

**Table C-1-6.—Breach Modeling Methods**

| Level of Detail/Accuracy | Dams | Levees |
|---|---|---|
| Lower | User defined breach parameters selected by regression analysis and/or expert judgment | Simplified physical breach model in HEC-RAS, or historical data and expert judgment |
| Higher | Physics based breach model coupled with hydraulic model | Physics based breach model coupled with hydraulic model |

A higher level of analysis is required if the breach parameters show significant uncertainty and the level of decision warrants further study. Some cases would not benefit from more analysis such as when downstream attenuation reduces the influence of the breach parameters, or if the uncertainty in material properties is larger than the hydraulic uncertainty and it cannot be reduced.

### *C-1.5.2.4 Lower Level Models for Dams*

The user-defined breach model is an option available in most hydraulic software packages, and has been a very common approach historically. In this method, the user relies on historical data in the form of regression equations for similar dams and expert knowledge of the dam and its materials to select the breach defining parameters. The parameters are input directly to the hydraulic model. The parameters define the rate and size of opening instead of allowing the model to calculate how the structure would breach.

For stability failures such as rapid settlement during an earthquake, many breach models (including those in HEC-RAS) have a “mass wasting” feature that quickly lowers the top of the structure to the desired elevation. That elevation may be determined by geotechnical modeling (such as FLAC analysis) or other team input.

Regression equations have been developed to help estimate breach parameters for dam, for both overtopping and internal erosion, but are these are not recommended for levees. Table C-1-7 shows several available, commonly-used regression options. Care should be taken to make sure the dam being modeled is contained within the limits of structures in the regression database based on its reservoir volume, height, length, material, etc.

**Table C-1-7.—Empirical Breach Formulations* (DSO-98-004 (1998) and HEC-RAS)**

| Reference | Number of Case Studies | Relations Proposed (S.I. units, meters, $m^3$/s, hours) |
|---|---|---|
| Johnson and Illes (1976) | | $0.5h_d \le B \le 3h_d$ for earthfill dams |
| Singh and Snorrason (1982, 1984) | 20 | $2h_d \le B \le 5h_d$<br>0.15 m $\le d_{ovtop} \le$ 0.61 m<br>0.25 hr $\le t_f \le$ 1.0 hr |
| MacDonald and Langridge-Monopolis (1984) | 42 | Earthfill dams:<br>$V_{er} = 0.0261(V_{out}*h_w)^{0.769}$ [best-fit]<br>$t_f = 0.0179(V_{er})^{0.364}$ [upper envelope]<br>Non-earthfill dams:<br>$V_{er} = 0.00348(V_{out}*h_w)^{0.852}$ [best fit] |
| FERC (1987) | | $B$ is normally 2-4 times $h_d$<br>$B$ can range from 1-5 times $h_d$<br>$Z$ = 0.25 to 1.0 [engineered, compacted dams]<br>$Z$ = 1 to 2 [non-engineered, slag or refuse dams]<br>$t_f$ = 0.1-1 hours [engineered, compacted earth dam]<br>$t_f$ = 0.1-0.5 hours [non-engineered, poorly compacted] |
| Froehlich (1987) | 43 | $\overline{B}^* = 0.47K_o\left(S^*\right)^{0.25}$<br>$K_o$ = 1.4 overtopping; 1.0 otherwise<br>$Z = 0.75K_c(h_w^*)^{1.57}(\overline{W}^*)^{0.73}$<br>$K_c$ = 0.6 with corewall; 1.0 without a corewall<br>$t_f^* = 79(S^*)^{0.47}$ |
| Reclamation (1988) | | $B = (3)h_w$<br>$t_f = (0.011)B$ |
| Singh and Scarlatos (1988) | 52 | Breach geometry and time of failure tendencies<br>$B_{top}/B_{bottom}$ averages 1.29 |
| Von Thun and Gillette (1990) | 57 | $B$, $Z$, $t_f$ guidance (see discussion) |
| Dewey and Gillette (1993) | 57 | Breach initiation model; $B$, $Z$, $t_f$ guidance |
| Froehlich (1995b) | 63 | $\overline{B} = 0.1803K_oV_w^{0.32}h_b^{0.19}$<br>$t_f = 0.00254V_w^{0.53}h_b^{(-0.90)}$<br>$K_o$= 1.4 for overtopping; 1.0 otherwise |
| Froehlich (2008) | 74 | $B_{ave} = 0.27K_0V_w^{0.32}h_w^{0.04}$<br>$K_0$ = 1.3 for overtopping, 1.0 for piping<br>$t_f = 63.2\sqrt{V_w/gh_b^2}$<br>Z = 1H:1V (overtopping), 0.7H:1V (piping/seepage) |
| Xu and Zhang (2009) | 182 | $B_{ave} = 0.787h_b(h_d/h_r)^{0.133}\left(V_w^{1/3}/h_w\right)^{0.652}e^{B_3}$<br>$B_t = 1.062h_b(h_d/h_r)^{0.092}\left(V_w^{1/3}/h_w\right)^{0.508}e^{B_2}$<br>$T_f = 0.304T_r(h_d/h_r)^{0.707}\left(V_w^{1/3}/h_w\right)^{1.228}e^{B_5}$ |

* Refer to source documentation for more information on input parameters.

Important note: Xu and Zhang (2009) data used in the development of the equation for breach development time includes more of the initial erosion and post erosion period than what is generally used in hydraulic models. In general, their equation will generate breach development times that are greater than the other widely used methods. Reclamation does not recommend the use of the Xu and Zhang (2009) breach equations as they may incorrectly estimate breach formation time and breach shape configurations for erosion resistant dams. Many of the cases used as data for the equation development were judged to have an 'unknown' basis for their erodibility classification, and the breach formation times used for many of the case studies were found to be inaccurate; often they were representative of the breach initiation time (which is often longer than the breach formation time) or the total failure time.

Dams may also benefit from the Simplified Physical method described in the following section for lower level analysis of levees, especially those that are relatively short and long compared to their height. The simplified physical method focusses more on the rate of breach widening than downcutting and may help build confidence in the selected breach parameters.

#### *C-1.5.2.5 Lower Level Models for Levees*

Attempts have been made to develop regression equations for levees, but a lack of information and other compounding factors have made those attempts fruitless. The equations above are not advised for levees.

Breach modelers have few options for a lower level levee breach modeling effort. If levees in nearby locations with similar geotechnical and geological attributes have breached in the past, the characteristics of those breaches can inform selection of parameters for a given study. More likely, available information will be inadequate, which will require the analyst to use a predictive model. One such predictive model is the recent addition to HEC-RAS that allows the user to define the erosion rate of the embankment material as it relates to the breach.

Velocity erosion rates versus velocities for different material types is shown in table C-1-8. Again. collaboration with geotechnical experts is advised. The method for levees is documented by the USACE Mapping and Consequence Production Center (Ubben 2016), utilizing erosion rate curves developed at Engineer Research and Development Center (Wibowo and Robbins 2016).

The levee material is selected as "very erodible" "erodible", or "moderately resistant." "Resistant" and "very resistant" curves are ruled out for levees, but are available in the original report and may be useful for dams. The determination of material erodibility gives the user a horizontal erosion rate curve which can be used directly in HEC-RAS. The vertical erosion rate is a factor of the horizontal such as 1:1 or 10:1. Because levee breaches are usually much wider than they are tall, the rate of downcutting is not usually a significant parameter.

**Table C-1-8.—Example Material Erodibility Rates**

| Velocity (ft/s) | Widening Erosion Rate (feet per hour) for 15-foot levee | | | | |
|---|---|---|---|---|---|
| | Very Erodible (Silt Material) | Erodible (Sand Material) | Moderately Resistant (Clay Material) | Resistant (Gravel Material) | Very Resistant (Cobble Material) |
| 0.4 | 82.2 | 0 | 0 | 0 | 0 |
| 1.0 | 86.0 | 1.2 | 0 | 0 | 0 |
| 1.5 | 90.0 | 3.2 | 0 | 0 | 0 |
| 2.0 | 97.5 | 5.9 | 0 | 0 | 0 |
| 3.0 | 118.0 | 13.1 | 1.0 | 0 | 0 |
| 4.0 | - | 23.6 | 3.3 | 0 | 0 |
| 6.0 | - | 52.7 | 10.4 | 0 | 0 |
| 8.0 | - | 94.5 | 19.7 | 0 | 0 |
| 10.0 | - | 146.0 | 31.7 | 0 | 0 |
| 15.0 | - | - | 72.7 | 0 | 0 |
| 20.0 | - | - | 126.3 | 10.1 | 0 |
| 25.0 | - | - | - | 23.5 | 0 |
| 30.0 | - | - | - | 38.8 | 0 |
| 50.0 | - | - | - | - | 0 |
| 75.0 | - | - | - | - | 48.5 |
| 100.0 | - | - | - | - | 141.4 |

Wibowo and Robbins (2016) have defined the erosion rates based on two factors: (1) material type and (2) levee height. They also included upper and lower confidence limits for each case to test sensitivity. Tables like the one above were created for upper-bound confidence, lower-bound confidence, and best estimate for each levee height in 5 foot increments from 5 to 30 feet. The confidence ranges are based on reasonable expected material properties ($k_d$, $\tau_{cr}$, $\gamma_{wet}$, c, $\varphi$). In addition, the report provides some guidance on selection of the material erodibility type based on its Unified Soil Classification and consistency (table C-1-9).

The curves were developed with consideration of excess shear stress erosion rates and geotechnical stability failures. The analysis assumes the breach is flowing with water up to the levee crest which is overly-aggressive. In most cases the flow in contact with the levee would be at a lower elevation due to contraction and the change in elevation from headwater to tailwater. The analysis assumes the breach has already reached the foundation elevation, neglecting any downcutting. It was validated against large scale breach testing done by the U.S. Department of Agriculture-Natural Resources Conservation Service (Hanson 2011).

**Table C-1-9.—Grouping Soil Materials into Erodibility Categories**

| | Unified Soil Classification | Additional Description | Consistency |
|---|---|---|---|
| Very erodible | SW, SP, SM | Loose | Very soft |
| Erodible | SC, SP-SM, SM-SC | Dense | Soft |
| Moderately resistant | GM, GP,GC-GM, SM-ML, ML, MH, ML-CL, CL | Low plasticity clay | Medium |
| Resistant | GW, GP, GW-GM,GP-GM, CH,CL-CH, CH-MH | High plasticity clay, gravel | Stiff |
| Very resistant | Cobble, CH | Coble | Very stiff, hard |

In HEC-RAS, a maximum breach width is also required, but should not limit the breach growth computation unless there are physical constraints such as a structure or the end of the levee.

### *C-1.5.2.6 Higher Level Models for Dams*

The higher-level analysis for dams uses a physics-based breach model to calculate the breach opening by considering the material properties which is then passed to a hydraulic model. This method also requires coordination between the hydraulic and geotechnical team members to determine all of the model input. The physics-based breach models rely heavily on geotechnical parameters discussed in "chapter D-1, Erosion of Soil and Rock." The method provides the most physically accurate representation of the breach erosion process for embankments, but it cannot be used for concrete dams. Unfortunately, there is still considerable uncertainty in this method that must be addressed, especially if the material properties are estimated instead of measured in-situ. Without field testing or good knowledge of the material properties this method may not provide much more reliable results than lower level methods. Model options are NWS-BREACH, HR-BREACH, WinDAM B/C, and DLBreach. Their modeled processes are summarized in table C-1-10.

NWS-BREACH is the oldest and least sophisticated of the models. It has been incorporated into some hydraulic software packages such as FLO-2D. The other three models have not yet been incorporated in to a hydraulic package so coupling requires expert judgement of hydraulics and erosion processes. Each of the models can route an inflow hydrograph and spillways or gates, making the coupling process a simple handoff of the breach outflow hydrograph from the breach model to the hydraulic model at the dam site. If the dam outflow may become impacted by the tailwater ($Depth_{TW} \cong 0.65\ Depth_{Breach}$) a method similar to levees should be followed. Tailwater can have a large impact on the outflow and erosion rates of a breach requiring feedback from upstream to downstream conditions and negating the simple hydrograph handoff.

**Table C-1-10.—Modeled Processes Available in Common Breach Models**

| Process | WinDAM B/C | DL Breach | HR BREACH | NWS BREACH |
|---|---|---|---|---|
| River Hydraulics | No | N | N | N |
| Breach Flow | Yes | Y | Y | Y |
| Internal Hydraulic Routing | N | N | Y | N |
| Tailwater Submergence | Y | Y | Y | Y |
| Piping Initiated | Y | Y | Y | Y |
| Overtopping Initiated | Y | Y | Y | Y |
| River Erosion and Stability Failure Initiated | N | N | N | N |
| Headcut | Y | Y | Y | N |
| Breach Widening | Y | Y | Y | Y |
| Breach Deepening | Y | Y | Y | Y |
| Foundation Scouring | N | Y | N | N |
| Mass Wasting (geotechnical failure) | Y | Y | Y | Y |
| Surface Erosion by Sediment Transport | N | Y | Y | Y |
| Sediment Volume | N | Y | Y | Y |
| Surface Protection Removal | Y | N | Y | Y |
| Composite Material Zones | N | Y | Y | Y |

Table C-1-11 describes the allowable hydraulic boundary conditions for each stand-alone breach model. The conditions allowed may complicate modeling or limit model selection.

### *C-1.5.2.7 Higher Level Models for Levees*

The method of coupling the breach and hydraulic models can have a significant impact on the results, especially for levees. Each of the models was first developed for use on dams, however application of the three newer models on levees is discussed in (Risher and Gibson 2016). The biggest consideration with coupling the models is how the breach erosion and headwater and tailwater levels impact each other. If the headwater and tailwater are minimally impacted by the breach (infinite water supply and infinite discharge capacity downstream) the coupling is easy. The breach parameters and progression can be taken from the

**Table C-1-11.—Allowable Hydraulic Boundary Conditions**

| Type | WinDAM | DL Breach | HR BREACH | NWS BREACH |
|---|---|---|---|---|
| US Inflow Hydrograph | Y | Y | Y | Y |
| US Reservoir | Y | Y | Y | Y |
| US Stage Hydrograph | N | Y | Y | N |
| US Channel Profile (Sloped Water Surface) | N | N | N | N |
| Spillway/Gate Outflow | Y | Y | Y | Y |
| DS Rating Curve | Y | N | Y | N |
| DS Reservoir | N | Y | Y | N |
| DS Stage Hydrograph | N | N | Y | N |
| DS Normal Depth Channel | N | Y | Y | Y |

US: Upstream, DS: Downstream

breach model and input directly to HEC-RAS similar to the lower level for dams method above. However, with most levee breaches the erosion rate can have big impacts on the headwater and tailwater elevations (e.g. the leveed area fills) reducing the rate of erosion. The breach models may or may not capture the hydraulic–erosion feedback loop of these situations. Each model has different capabilities for the hydraulic boundary conditions as seen in table C-1-11. There are a couple options to deal with this problem: iteration between breach and hydraulic models, or by extracting erosion rates from the breach model and using the simplified physical breach in HEC-RAS described above. An example of the latter method is described in Risher et al. 2017 for the other models.

#### *C-1.5.2.8 Sensitivity Analysis*

Each of the levels of breach analysis should include a sensitivity analysis on the most uncertain and/or sensitive breach parameters. Typically, those are D50 for non-cohesive materials, $k_d$ (erodibility coefficient) for embankments subject to headcutting, and handling of tailwater for levees. Other sensitive parameters are the location of the breach, failure mode, and timing of the failure which may be very sensitive with respect to the flood timing and consequences even if the breach is unchanged.

### C-1.5.3 Inundation Modeling, Terrain Data

Terrain data for inundation modeling is important. Current hydraulic models rely on GIS for pre- and -post processing. Digital elevation models (DEM) have become the most commonly used terrain data format. The DEM is a raster or grid based format, similar to a matrix of equal sided cells containing a single elevation value. Some hydraulic models make use of terrain data in a TIN surface format rather than a DEM format. The TIN consists of an aggregate of varying dimensioned triangles that form a surface. Certain 2D hydraulic models require the creation of what is called an unstructured mesh, which is a network of various sized triangles or trapezoids. Surveyed cross section data are sometimes used as well. In general, though, there are several common types of digital terrain data sources:

- **USGS DEMs.—**USGS produces DEMs which can be downloaded for free from the National Elevation Dataset (NED) website, ned.usgs.gov NED DEM data is usually available in 10 and 30-meter resolution. This is for the most part, lower resolution terrain. However, the NED DEM data is widely used and can be of adequate quality and resolution for modeling high discharge dam breach flows downstream of large storage reservoirs.

- **IFSAR Terrain Data.—**Interferometric Synthetic Aperture Radar (IFSAR) data is radar-based terrain data which is collected from the wing of an aircraft. IFSAR data is processed to remove vegetation, man-made structures and other features to produce a "bare earth surface." The currently available data is "medium quality resolution", significantly better than the USGS NED data. IFSAR data has 1-meter accuracy, both vertical and horizontal. Intermap Technologies, www.intermap.com, has collected IFSAR terrain data for the entire 48 U.S. mainland states. This data is readily available, and pricing of the data is very reasonable in comparison to higher resolution options. IFSAR data can be a good option when USGS NED data does not contain enough information to accurately portray downstream features. This would be particularly true when modeling lower discharge dam breach or levee breach flow and/or where flat terrain in downstream areas does not contain enough detail in the USGS NED data to have confidence in the modeling results.

- **Aerial Photogrammetry.—**Interpretation of aerial photography can produce digital terrain data with a variety of accuracy that depends on photo scale (flying height). This data can be very good quality, although it can be expensive and time consuming to acquire. Photogrammetric data may have cost advantages over Light detection and ranging data (LIDAR) when detailed data is desired within a small area.

- **LIDAR.—**LIDAR is laser-based data that is flown from an aircraft, much like the IFSAR data, but at a higher accuracy. LIDAR data typically has a vertical accuracy of +/- 15 cm (about 6-inches). The "bare earth surface" produced by LIDAR data is typically of very high accuracy. LIDAR is expensive and time consuming to acquire, but when the highest resolution data is needed, LIDAR may be the way to go. Note that ground-based

LIDAR systems also exist and may be of value to collect detailed data within a small area of interest. The USGS has long-term plans to obtain LiDAR for all of the U.S. Limited data has become available for free download from the NED website. This data is referred to on the NED site as 1/9 arc-second data, and its availability can be viewed through status graphics. Another source of LiDAR, and other high-resolution terrain data, is through the National Oceanic and Atmospheric Administration's United States Interagency Elevation Inventory Web site (https://coast.noaa.gov/inventory/). Much of the data referenced on the site can be obtained free of charge.

Using the power of a GIS, a wide variety of data formats can be utilized if available. For example, vector contour line data can be converted to DEM format, point data can be converted to TIN, TIN can be converted to DEM, etc. GIS technology allows the integration of a wide variety of potential data sources. Note that higher resolution data such as photogrammetric or LIDAR may have been acquired by local entities who may be willing to share the data at low or no cost. There is often value in contacting local county or city GIS offices to inquire about the existence of such data.

In working with different terrain types, it is important to keep a perspective on terrain accuracy versus terrain resolution. For example, changing the resolution (known as re-sampling) of a USGS NED 10-meter DEM from 10-meters to 3-meters does not make the data more accurate. However, re-sampling LIDAR data to a 10-meter resolution will provide more accurate data than the 10-meter NED DEM, since the vertical accuracy of the NED data is much lower than the LIDAR data. There are limits to this; re-sampling LIDAR data to a 1,000 or even 100-meter resolution, for the purpose of creating faster 2D model run times loses all the benefits of vertical accuracy that were gained with the LIDAR data.

The modeling of dam failure scenarios which include the operation and/or breaching of downstream dams may require the development of downstream reservoir bathymetry in order to properly represent the dam and reservoir in the model.

### C-1.5.4 Inundation Modeling Outputs

Inundation modeling outputs are used to develop a variety of information that is useful for estimating life loss. A standard inundation modeling output is the maximum inundation polygon. This is the flood boundary that is typically shown on an inundation map. The maximum inundation polygon depicts the widest and most severe extent of flooding that occurs in all of the downstream areas. In reality when upstream areas become inundated, the downstream areas have not yet been flooded. When these downstream areas reach maximum flooding, the upstream areas might start to dry out. The maximum inundation polygon is useful for viewing the maximum flooding that may occur at all flooded locations throughout the duration of the flooding event.

In additional to maximum inundation, typical inundation modeling output data includes flood depths, velocities, water surface elevations, maximum discharge, arrival time of leading edge and arrival time of maximum flooding.

1D models traditionally have presented output data at cross section locations. This data is typically portrayed in a tabular format. On an inundation map, it is common to depict cross sections, labeled by their location. A table on the map will include output information referenced to the cross sections.

2D models do not have cross sections and the presentation of 2D modeling results may make use of a variety of formats. 2D inundation polygons can be color-coded according to ranges of depth, velocity or DV. In addition to the maximum inundation polygon, it is easy to display “snapshots in time” which depict the entire flood configuration at a particular time of interest, for example - 3 hours after the initiation of the breach. The leading edge of flooding is irregular, and a poly-line data set can be digitized in the GIS to represent the front edge of the flood at various time steps. Maximum discharge and time to maximum flooding information can be obtained by extracting hydrographs from the 2D model output data at areas of interest. Interpolated results from 1D modeling output can be presented in a format similar to what is done with 2D modeling output. Care must be taken though when presenting 1D results in this manner, not to misrepresent the accuracy of the study in question.

## C-1.6 Estimation of Downstream Population at Risk

Life loss estimates are based on some assumption of the number of people that are present in the flood zone. There are different life loss estimation methods that take various approaches to how they develop fatality estimates, but one thing these methods all have in common is that they require an initial estimate of PAR. At a very basic level, the development of a PAR estimate can be as simple as visiting a site below a dam or levee and counting houses in the inundation zone. One of the online map services such as Google Maps, Google Earth, MapQuest, or Bing Maps can also be used to count inundated houses. Typically, PAR is estimated using the U.S. Census data. Often, PAR estimates are based on residential PAR. The most accurate data for residential PAR estimation is at the level of the census block. The flood inundation boundary can be overlaid with the census block data in a GIS, and the number of inundated PAR households can be calculated. Figure C-1-26 provides an example of an inundation boundary overlaid on a census block. Partially inundated census blocks must be treated

**Figure C-1-26.—Census block/inundation overlay.**

separately. If the residences are evenly distributed within the partially inundated block, such as occurs within an urban area, a percent inundated estimate can be applied to the total number of households within that block to estimate the partial total. If the distribution of residences within a partially inundated block is not uniform, as is often the case with larger blocks located in rural areas, then an approach would be to manually count the houses in the inundation zone. The counted houses are multiplied by a census block-specific household size multiplier to get the total PAR for the partially inundated block.

The use of residential PAR for life loss estimation is a simplifying assumption. If more detailed information is known about where people may be located during daytime hours, then this information can be used to develop daytime-specific life loss scenarios. Care must be taken though, not to double count PAR when looking at non-residential PAR distributions. A good example of this is a Reclamation Dam that has a mill operation located immediately downstream. The mill has maybe 400 employees present during daytime hours. The proximity of the dam to these employees puts them at the highest level of risk in the event of dam failure. It is unknown however, where the residences of these employees are located. Some may live in the flood zone at locations further downstream, and because of this they may be double counted. In this case though, the fatalities close to the dam can be assumed to be high and persons living downstream in the floodplain are assumed to have much more time to evacuate, so that the issue of potentially double counting is not considered to be introducing major errors. Double counting of PAR when considering non-residential situations should be evaluated on a case by case basis to avoid the possibility of overestimating fatalities.

Another type of PAR that is frequently estimated is recreational or transient PAR. This would include persons occupying campgrounds, fishing, boating or hiking along a river, etc. Recreational PAR estimates can be obtained through site visits and/or by consulting with land use and recreation management groups who oversee these areas. In some cases, visitation numbers data may be available, or in other cases, campground hosts or park rangers may have a general idea of user numbers. Typically, recreational PAR will vary by time of year and day of week, with greater numbers in the summer months and on weekends. Day use areas will of course have higher PAR during daytime hours, with low or no PAR present during the evening.

## C-1.7 Warning and Evacuation

In the most ideal situation, a dam breach in progress would be detected well in advance of the beginning of catastrophic outflows, clear evacuation orders would be issued to downstream PAR without delay, and all of the PAR would move safely out of the flood zone before flooding arrives in their area. Unfortunately, dam failure and flash flood case histories have shown that things do not always go that smoothly. The sequence of events that takes place is often a mix of physical and social phenomena, sometimes combined with a dose of luck or chance.

The issuance of warning and the decisions of downstream PAR are critical factors that impact the potential for life loss. Past dam failure flood instances show that, in general, the number of fatalities decreases as the distance downstream increases, but increasing distance by itself is not what decreases the life loss potential. Potential life loss decreases when the travel time begins to exceed the amount of time required to warn and evacuate the PAR. A combination of breach development rate and flood wave velocity determines the flood wave arrival time for a given distance. Then, the distance to a safe haven, the escape route capacity, and various human perceptions and choices determines who might be caught within inundation boundaries when the flood arrives.

Another attribute of increasing distance is the attenuation (reduction) in flow that occurs. However, flow depths and velocities can increase downstream if the flood plain transitions from a wider valley to a narrow canyon.

Warning time is broken into stages: detection of the threat, decision to issue warning, notification of the downstream PAR, and warning dissemination. Detection of a developing dam failure situation could be by automated instrumentation, by visual inspection by project personnel or by someone passing by the area such as a hiker or fisherman. After the unusual situation is noticed, some amount of time is required before project and emergency preparedness personnel assess the situation and decide that there is a reasonable chance it will develop into a condition that cannot be controlled. Then, the notification of those responsible for spreading the warning can take some additional time. Typically, a failure or failure in progress must be verified by an official or their representative, before the decision is made to order an evacuation. The actual warning to the PAR can be transmitted many ways, each with its own degree of effectiveness. The content and wording of the warning message is very important when it comes to how quickly people will take the necessary precautions, either giving people a strong perception of the danger or not. Informal warning can also spread by word-of-mouth through friends, family, neighbors, and concerned citizens. People who are at risk, but are not warned verbally, can still perceive danger by hearing an unusual sound or seeing a rapidly rising flow.

Estimation of the warning and evacuation process may include consideration of the following issues:

- Failure of the dam or its impending failure may need to be verified before warning is issued.

- The decision to order evacuations must be made. Often the decision makers will weigh the evidence at hand regarding the likelihood of catastrophic flooding versus perceived issues of public distrust when determining whether to issue a warning.

- After a warning is issued, it will spread through the targeted community. The speed at which is spreads is based on the types of warning systems/channels employed by the agency issuing the warning. There is no silver bullet when it comes to the best, most effective warning system. Research shows that using a wide range of traditional (sirens, door-to-door notification, emergency alert systems) and modern technology (auto-dial telephone, wireless emergency alert) provides the most efficient warning dissemination.

- People may receive warning or an order to evacuate, but may delay taking a protective action (e.g. evacuation) or may choose not to leave at all. The timeliness of taking the recommended protective action is heavily influenced based on the content of the warning message. Clear messages that contain information about the threat, the source of the warning, the potential consequences, specific instructions on when to leave and where to go are much more likely to lead to a quick response than those lacking information.

- Persons who do not attempt to evacuate or who attempt to evacuate at the last minute can be placed in critical situations where a number of factors may influence their survival. The flood depths, the intensity of flooding (often quantified as a function of depth and velocity), the strength of a shelter, and a person's physical condition will influence the survival chances of PAR exposed to flooding.

- Some people may not evacuate. Reasons for this include: warnings may not be taken seriously; elderly persons or disabled persons may have too much difficulty attempting to evacuate; people may not evacuate for fear of looting; people may not believe that the flood impacts will be severe enough to endanger them; people delay evacuation to protect personal property such as pets or livestock.

- Densely populated urbanized areas need more time to evacuate. These are special situations where traffic congestion may play a role in the ability to evacuate. Persons attempting to evacuate in advance of flooding may get stuck in traffic, resulting in exposure to flooding. In many situations, evacuating to a large, sturdy building, or staying in one's home may be safer than attempting to leave the area in a vehicle. Note that life loss simulation models such as LifeSim and Life Safety Model use transportation network models and attempt to address traffic congestion issues during flood events.

Case history data provides some examples of human behavior in relation to flood risk and evacuation:

- The failure of the Machhu II Dam in India in 1979 killed as many as 10,000 people. Once warned, some people did not leave because they lived above the highest flood levels that had occurred during their lifetime.

- Teton Dam failure in 1976 (11 fatalities) and Lawn Lake Dam failure in 1982 (3 fatalities) both contained fatality incidents where people who had safely evacuated re-entered the flood zone to retrieve possessions, thinking that they had more time before the arrival of flooding.

- The eruption of the Nevado del Ruis volcano and the deadly lahar mudflow flood at Armero, Columbia, in 1986 killed about 22,000 people. Most residents of Armero didn't evacuate because the severity of risk was downplayed by local officials.

- St. Francis Dam failed in 1928, killing more than 400 people. Some who heard the approaching flood waters could not conceive of a dam failure flood and thought the sounds to be due to a windstorm.

Experience indicates that there is sometimes a reluctance to issue dam failure warnings. The operating procedures or emergency actions plan that may be available for a dam or levee should provide some guidance regarding when a warning would be issued. There is no assurance, however, that a warning would be initiated as directed in a plan. A study investigating loss of life from dam failure can be used to highlight weaknesses in the dam failure warning process and provide some guidance on how improvements in the process would reduce the loss of life. Sensitivity analysis should be used to provide information on how significant warning issuance is to the uncertainty in a life-loss estimate.

For most breach mechanisms where the breach progression is observable prior to catastrophic failure of the dam or levee, the time when a warning is issued should be determined by first estimating the time when a major problem would be acknowledged relative to the time of dam failure. The major problem acknowledgment time for these failure modes is the time when a dam owner would determine that a failure is likely imminent, and they would decide that the dam breach warning and evacuation process should be initiated by notifying the responsible authorities. The time lag between major problem acknowledgement and when an evacuation order would pass from the dam owner to the responsible emergency agency (EMA) and then from the EMA to the public should be estimated based on available research, judgment of consequence specialists familiar with that research, dam operations personnel and emergency management personnel who have jurisdiction in the areas of each downstream community.

The amount of time it takes from when the evacuation warning is issued by the responsible agency (warning issuance) until the PAR receives that warning is dependent on the number and type of warning systems or processes that are used to disseminate that warning. A typical warning would be received by the population through various means. For example, the first group of people would typically receive warning through the primary warning process (e.g., Emergency Alert System), but then a secondary warning process would begin that includes emergency responders and the general population spreading that warning via word of mouth.

## C-1.8 Intensity of Flooding and Fatality Rate

Fatality rates represent the percentage of people exposed to flooding (typically known as threatened population) that lose their life. An important difference between RCEM and the simulation models used by USACE and Reclamation is that RCEM defines fatality rates as the percentage of pre-evacuation PAR that loses their life rather than a percentage of exposed population to the flooding (those remaining after evacuation has taken place). In either case, fatality rates are typically derived from empirical data.

The intensity of flooding can be correlated to the potential for fatalities. This intensity is often quantified in terms of depth multiplied by velocity, or DV. Mapping of DV can be produced from 1D or 2D modeling results and the DV maximum inundation boundary can be overlaid with census data in a GIS to assess zones of various levels of destructive intensity (also referred to as flood severity). Note that 2D hydraulic modeling can provide greater accuracy when assessing lateral variation of DV. Flooding depths are an important measure of flood intensity as well. Deeper water can make evacuation on foot impossible, submerge roads, float cars and mobile homes, and make structures uninhabitable. Fatality rates can be influenced by both flood depths and DV.

The potential for collapse of buildings within the flood zone can be some measure of the potential for fatalities, assuming people are present when the flood arrives. Most residential buildings would be vulnerable to major damage and/or collapse when flooding DV exceeds the range of 7 to 15 $m^2/s$ (75 to 161 $ft^2/s$).

Assumptions that affect fatality rates can be adjusted when justified by extenuating circumstances. If a particularly devastating earthquake is responsible for dam failure, it is possible the earthquake has also devastated infrastructure and communications in population centers in the vicinity. Every aspect of warning (i.e. detection, decision, notification, and dissemination) may be affected, and evacuation routes may be compromised. Emergency management personnel would be responding to several situations and will not be able to devote their entire attention on a developing situation at a dam. Using RCEM, it may be reasonable to increase the fatality rates for this case. For the simulation-based approaches, these considerations would be handled explicitly by adjusting the parameters in the warning and evacuation modeling.

Regardless of the life loss prediction method that is used, there is a great deal of uncertainty in all aspects of the life loss estimate. Therefore, communicating risk to decision-makers should be as a range, or better yet, as a graphical depiction of likelihood. The general shape of likelihood distribution graphs can be envisioned by thinking through many hypothetical scenarios. For example, many Reclamation dams have few people living within the dam break flood inundation boundaries, and many failure scenarios can be envisioned taking place very slowly with a long warning period. In these situations, it would make sense that there would be a significant likelihood of zero life loss. One could envision many dam break scenarios for the same dam and population, starting at different times of the day, breaching with different rates, and given various warning and evacuation scenarios. If many of these scenarios would end in life loss, there might be a range between zero and some number where the estimate is likely to fall. One could also envision some small chance that everything could go wrong, and that in these rare instances, a large number of lives would be lost. Figure C-1-27 shows what a fatality likelihood distribution could look like.

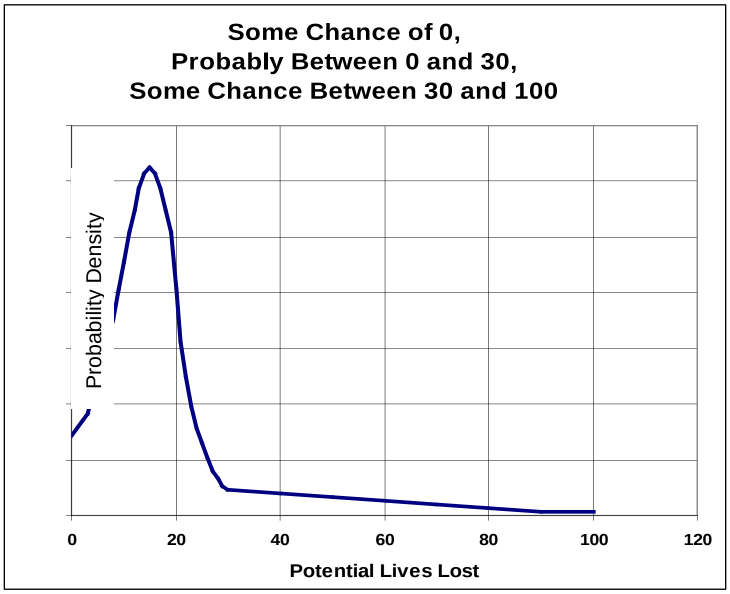


**Figure C-1-27.—Probability density function example fatality distribution for small PAR.**

Another example might occur when the PAR is much larger, and the dam is expected to fail much more quickly. In this case, it is much less likely that there would be zero life loss. But again, the expected life loss envisioning many different scenarios would likely fall in a range, with a tail of less-likely estimates to represent the extreme values. Figure C-1-28 show what this distribution would look like.

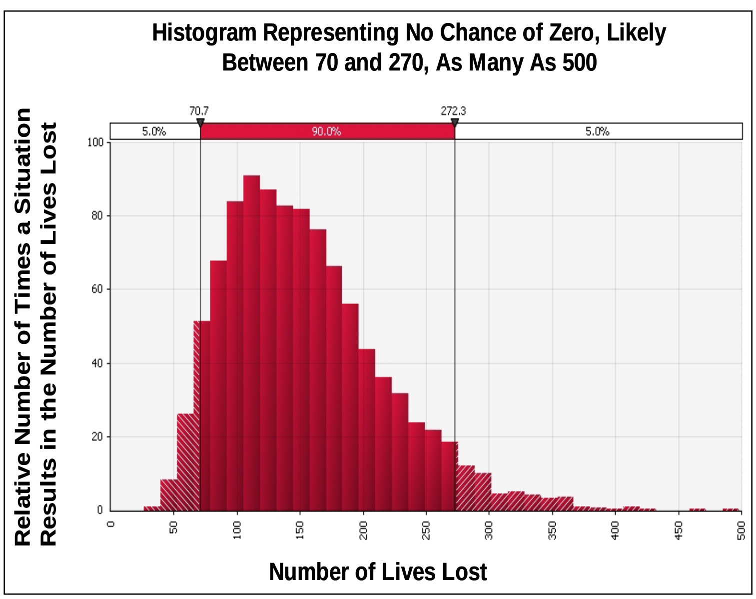


**Figure C-1-28.—Histogram example fatality distribution for rapid failure with large PAR.**

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