Analysis of Dam Failure in the Saluda River Valley

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Summary

We identify and model two possible failure modes for the Saluda Dam: gradual failure due to an enlarging breach, and sudden catastrophic failure due to liquefaction of the dam.

For the first case, we describe the breach using a linear sediment-transport model to determine the flow from the dam. We construct a high-resolution digital model of the downstream river valley and apply the continuity equations and a modified Manning equation to model the flow downstream.

For the case of dam annihilation, we use a model based on the Saint-Venant equations for one-dimensional flood propagation in open-channel flow. Assuming shallow water conditions along the Saluda River, we approximate the depth and speed of a dam break wave, using a sinusoidal perturbation of the dynamic wave model.

We calibrate the models with flow data from two river observation stations.

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Introduction

The Saluda Dam, located 20 km above Columbia, South Carolina, impounds the almost 3-billion-cubic-meter Lake Murray [South Carolina Electric & Gas Company 1995]. It is a large earthen dam of a type that has failed in earthquakes before [Workshop 1986]. In such a failure, the water in Lake Murray would

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rush down the Saluda River Valley towards Columbia, its 100,000 residents, and the State Capitol.

We present a comprehensive mathematical description of the resulting flood, including its intrusion into Columbia and the tributaries of the Saluda. See **Figure 1** for an overview of the local topography.

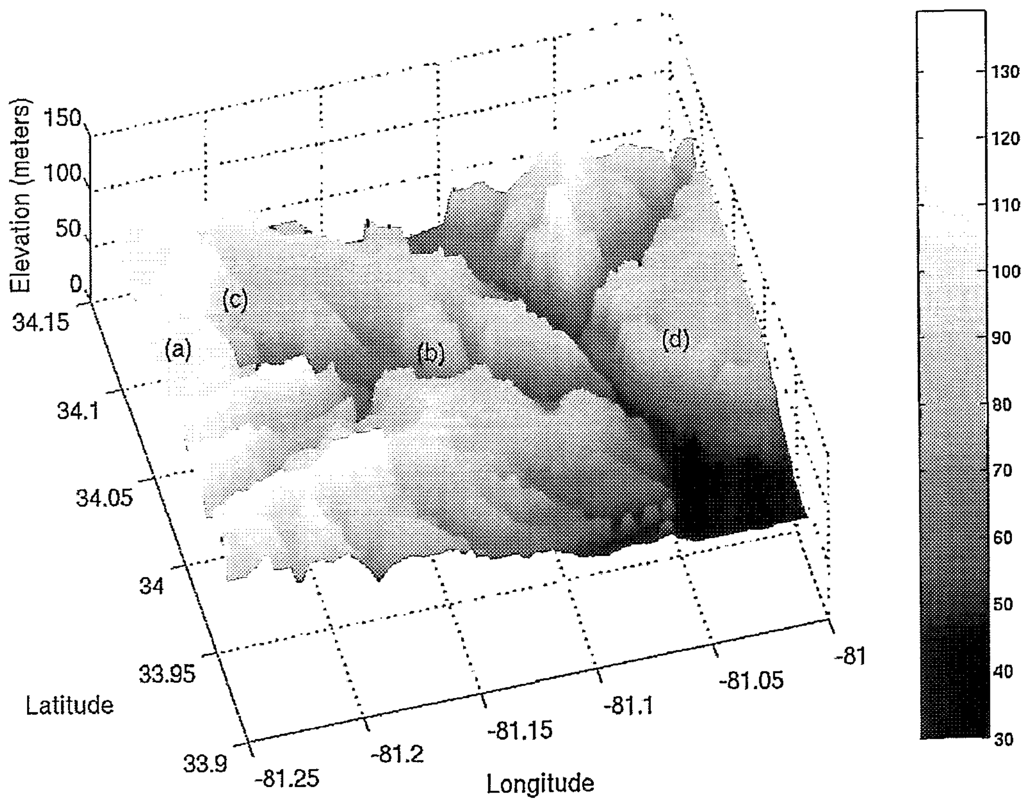


Figure 1. An overview of Lake Murray and the Saluda River Valley generated from the NCRS topographical data [National Geophysical Data Center 2005]. (a) Lake Murray. (b) Saluda River. (c) Rawls Creek. (d) State Capitol Building.

A brief survey of earthquake-related earthen dam incidents [Workshop 1986] reveals that failure can follow two distinct courses:

- A crack or breach forms in the dam, causing gradual failure due to erosion.
- The dam is completely annihilated, resulting in the formation of a surge.

To describe both of these situations accurately, we apply two different models.

Gradual Failure The relatively gradual rate at which water is introduced into the downstream valley suggests that the dispersion of the flood may be modeled using classical open-channel hydraulics. We divide the downstream river course into basins or reaches and then use the Manning formula and the continuity equation to describe the movement of water between them.

We determine the flow into the first basin using a model for the destruction of the dam due to breach erosion.

We create a three-dimensional topographical model of each basin using 3' resolution data from the NGDC Coastal Relief Model [National Geophysical Data Center 2005]. (**Figure 1** was generated using these models.) This lets us estimate the relationship between the volume in each basin and the cross-sectional area of its outflow channel. The Manning formula and the continuity equation yield a system of coupled first-order differential equations. We integrate this system numerically and calibrate it using data for normal flow in the Saluda River from river observation stations just below the Saluda Dam and just above Columbia City [USGS 2005].

Rapid Failure The flood wave is described as a sinusoidal perturbation to the steady-state solution of the Saint-Venant equation. We apply the dynamic wave model of Ponce et al. [1997] to determine the surge's propagation.

We represent the Saluda River Valley as a prismatic channel of rectangular cross section. We use a small surge recorded by the USGS river observation station in the Saluda Valley [USGS 2005] to calibrate the frictional constant governing the rate of attenuation of the flood waves.

Finally, we address the results of the two models and their consequences for Rawls Creek, the Capitol, and the residents of Columbia.

Gradual Failure

Our model for downstream flooding depends on the conservation of matter as described by the continuity equation, which states that for any given reach of the river, the change in volume equals the difference between flow in and flow out:

$$\frac{dV}{dt} = Q_{\text{in}} - Q_{\text{out}}, \tag{1}$$

where V is the volume of the reach, t is time, and Q_{in} and Q_{out} are the flows.

We divide the Saluda River Valley into four reaches. Since the amount of water involved in a dam failure flood would be significantly greater than that contributed by any other source, we simplify our model by assuming that all flow into and out of a reach would occur along the Saluda. For each reach, we set Q_{in} of each reach equal to Q_{out} of the reach above it, ignoring all tributaries. Eq. (1) becomes:

$$\frac{dV_n}{dt} = Q_{n-1} - Q_n, \qquad n = 1, \ldots 4, \tag{2}$$

where V_n is the volume in the nth reach (numbered downstream from the dam) and Q_n is the flow out of the nth reach; Q_0 is the flow into the reach 1 through the breach in the dam.

To evaluate (2), we must estimate several parameters and relations:

- the flow out of the reservoir (into the first reach) resulting from a dam breach,
- the flow through each reach, and
- the topographical profiles of the reaches.

Flow Through the Breach

Dam-breach erosion is an interaction between the flooding water and the material of the dam. Once a breach has formed, the discharging water further erodes the breach. Enlargement of the breach increases the rate of discharge, which in turn increases the rate of erosion. This interaction continues until the reservoir water is depleted or until the dam resists further erosion.

We assume that the pre-breach flow into and out of the reservoir can be ignored, since they are of opposite sign and of negligible magnitude compared to the flooding waters. The breach outflow discharge Q_0 equals the product of the rate at which the water is lowering and the surface area at that height, $A_s(H)$. Also, the breach outflow discharge is related to the mean water velocity u and the breach cross-sectional area A_b by the continuity equation:

$$A_s(H)\frac{dH}{dt} = -Q_0 = -uA_b. \tag{3}$$

Experimental observations show that the flow of water through a breach can be simulated by the hydraulics of broad-crested weir flow [Chow 1959; Pugh et al. 1984]:

$$u = \alpha(H - Z)^{\beta}, \tag{4}$$

where Z is the breach bottom height measured from the bottom of the lake. For critical flow conditions, $\alpha = [(2/3)^3 g]^{1/2} = 1.7$ m/s and $\beta = 1/2$ [Singh 1996].

We further assume that the surface area of the reservoir, A_s, is independent of the height (i.e., the reservoir is rectangular). Combining (3) and (4) yields

$$A_s\frac{dH}{dt} = -uA_b = -\alpha(H - Z)^{\frac{1}{2}}A_b. \tag{5}$$

We describe erosion in the breach using the simplest method that has been used to model dam breaks accurately in the past [Singh 1996] and assume that

$$\frac{dZ}{dt} = -\gamma u^{\phi} = -\gamma\alpha^{\phi}(H - Z)^{\phi\beta}, \tag{6}$$

where γ and ϕ are determined from experimental analysis of the dam material and u is given by (4). Because we do not have access to the dam, we assume that $\phi = 1$ (linear erosion) and approximate γ as 0.01. This value of γ has given good results for linear erosion in the past [Singh 1996]. Eqs. (5)–(6) are coupled first-order differential equations governing the elevation of the water surface and the elevation of the breach bottom as functions of time. To evaluate them, we must determine the shape of the breach.

Breaches in dams are typically modeled as triangles, trapezoids, or rectangles; but rectangles are used most often, since the resulting ODEs **(5)**–**(6)** are solved relatively easily [Singh 1996]. For simplicity, we model the breach as a rectangle with constant width b such that it erodes only in the vertical direction. Thus, the area of the breach is given by

$$A_b = b(H - Z). \tag{7}$$

Substituting **(7)** into **(5)** and rewriting **(6)** with $\phi = 1$ and $\beta = 1/2$ gives

$$\frac{dH}{dt} = -\frac{\alpha b}{A_s}(H - Z)^{\frac{3}{2}}, \qquad \frac{dZ}{dt} = -\gamma\alpha(H - Z)^{\frac{1}{2}}. \tag{8}$$

Equations **(8)** admit the solution

$$H(Z) = Z + q + C\exp\left(\frac{-(Z_0 - Z)}{q}\right), \tag{9}$$

$$t(Z) = \frac{2\sqrt{q}\,\operatorname{arctanh}\left(\sqrt{\frac{H(Z)-Z}{q}}\right)}{\gamma\alpha} - D,$$

where $q = A_s\gamma/b$ and $C = H_0 - Z_0 - q$, H_0 and Z_0 are the initial values of H and Z at $t = t_0$, and D is a constant of integration determined from the initial conditions. The quantity $Z(t)$ is defined implicitly by **(9)**, and $H(t)$ can then be recovered from **(9)**. Then the flow through the breach, Q_0 can be determined from **(3)** and **(5)**:

$$Q_0 = -\alpha\left[q + C\exp\left(\frac{-(Z_0 - Z)}{q}\right)\right]^{1/2} A_b. \tag{10}$$

When $Z(t) = 0$ at some time $\tilde{t}$, the dam must stop eroding and from **(8)** we obtain

$$\frac{dH}{dt} = -\frac{\alpha b}{A_s}H^{3/2}, \tag{11}$$

resulting in

$$H(t) = \left(\frac{1}{\sqrt{H_0}} + \frac{\alpha b}{2A_s}(t - t_0)\right)^{-2} \qquad \text{for } t \geq \tilde{t}. \tag{12}$$

Figure 2 graphs Q and Z vs. time. The discontinuity of the derivative at time $\tilde{t} \approx 2$ h is the transition between these two solutions.

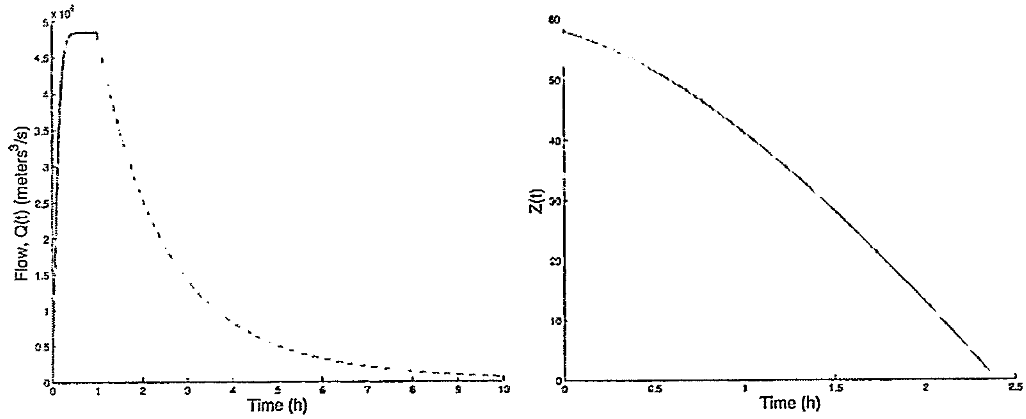


Figure 2. Flow through dam and height of breach bottom as breach enlarges.

Flow Between Reaches and the Manning Equation

We select reaches so that the river valley at their junctions is relatively prismatic and narrow. Assuming that the flow in a flood would be regulated by the rate at which water can flow through these narrows, we model the river as a series of pools, one flowing into the next.

Traditionally, flow in a floodplain is analyzed as the flow in a prismatic channel using the Manning equation

$$u = \frac{1}{n_m}\left(\frac{A}{P_w}\right)^{2/3}\sqrt{S}, \tag{13}$$

where

u is the mean flow velocity,

n_m is determined experimentally for each channel,

A is the cross-sectional area of the flow channel,

P_w is the wet perimeter of the channel cross section, and

S is the slope of the energy line.

There is no theoretical basis for the Manning equation; however, it has been extensively verified experimentally. Its primary advantage is the amount of information available on estimating Manning coefficients, n_m [Chanson 2004]. We apply it in our model because we can estimate n_m for the Saluda from data for other floodplains. The prismatic nature of the narrows means that we can apply the Manning equation without correcting for channel irregularities. Typical values of n_m are 0.5 for a brush-covered floodplain and 1.5 for a tree-covered one [Chanson 2004]. Assuming that our floodplain is somewhere between, we choose a moderate value of $n_m = 1$.

We set S, the slope of the energy line, equal to the slope of the valley floor. This is equivalent to assuming that the depth and speed of the water are constant with respect to flow direction in each narrows. Because of this, our model will be most accurate when radical changes in volume occur on a time scale greater than the time required to pass through the narrows. From the propagation rates observed, the time required for the water to pass through each of the narrows is on the order of 0.1 h. The flood that we wish to consider rises sharply for 0.5 h, stays steady for 1.5 h, and then trails off gradually (see **Figure 2**). Our model is least accurate for the steepest part of the initial rise but ultimately describes most of the flood well.

We estimate the slope of the channel out of each reach from USGS topographical maps [USGS 1971; 1994; 1997]:

$$S_1 = \frac{1}{1200}, \qquad S_2 = \frac{1}{800}, \qquad S_3 = \frac{1}{600}, \qquad S_4 = \frac{1}{800}. \tag{14}$$

We estimate S_4, the slope of the final outflow channel, conservatively so as to produce a worst-case scenario of the flooding of the basin that contains Columbia.

Our topographical models of the river basin allow us to establish one-to-one correspondences between the volume of water in each reach, the cross-sectional area and wet perimeter of its outlet, and the height of the water in the reach. These correspondences define the cross-sectional area and wet perimeter of the outflow narrows as functions of volume; we designate these functions as $A_n(V)$ and $P_n(V)$. Noting that for a given channel cross section, the flow Q satisfies

$$Q = uA_n, \tag{15}$$

where u is the mean water velocity, **(13)** can be stated as a constraint on **(2)**:

$$Q_n = A_n u_n = \frac{A_n}{n_m} \left(\frac{A_n}{P_n} \right)^{2/3} \sqrt{S_n}, \tag{16}$$

where $A_n = A_n(\rho V)$, $P_n = P_n(\rho V)$, and $V = V(t - \zeta)$.

We introduce parameters ρ and ζ to calibrate of the model; we determine them subsequently from observational data.

ρ describes how friction and surface features of the reach prevent the entire volume of water from flowing downstream.

ζ describes the amount of time that it takes water to pass through a reach. We assume ζ to be constant because of the constant length of our reaches.

Selection and Analysis of Reaches

We use 3' topographical data [National Geophysical Data Center 2005] to construct a topographical model. To establish correspondences between the volume V_n of water contained in each basin and the area A_n and wet perimeter P_w of their outflow channels, we intersect the topographical model of each basin with a plane representing the water level and integrate numerically over the appropriate regions. We construct a database of these values in terms of height, to be used as we simulate (2). **Figure 3** displays one such profile, for reach 4, with volume and area next to the topographical map of the basin.

Our accuracy is limited by the 0.2-m height resolution of the NGDC data. This does not significantly effect the accuracy of our volume estimates, but the area and wet perimeter estimates display noticeable discontinuities for small volumes. (The oscillatory behavior seen later in **Figure 4** is caused by this.) Our model could be improved by conducting better surveys of the outflow channel of each reach; since we are primarily interested in large volumes, we proceed.

To summarize, our model places the following requirements on the selection of the reaches:

- The inflow and outflow channels must be narrower than the rest of the reach.
- The channels must also be prismatic.
- Water should take approximately the same amount of time to flow down each reach.

To satisfy these conditions, we construct reaches as follows:

- Reach 1: The 6-km section from Saluda Dam to the narrows at Correly Island.
- Reach 2: The 6-km section from Correly Island to the narrows just below Interstate 20.
- Reach 3: The 6-km section from just below Interstate 20 to the narrows just above the Saluda's outlet into the Congaree river.
- Reach 4: A large section of the Congaree River Basin including the area around the Capitol and a 6-km stretch of downstream channel.

Reaches 1, 2, and 3 satisfy our requirements extremely well. The Congaree River valley widens rapidly into a floodplain below Columbia and there are no natural narrows. We end our basin at a point that is somewhat narrow and satisfies the requirement for water flow time. A large portion of the broad river valley is included to allow for upstream flooding.

Calibration and Sensitivity Analysis

The US Geological Survey (USGS) has river observation stations just below the dam (at 34°03′03" N 81°12′35" W) and just above Columbia (at 34°00′50" N 81°05′17" W). Each logs the last 31 days' flows [USGS 2005]. On 6 January 2005, the station at Saluda Dam registered a surge of 30,000 m^3. Flow rates jumped from 27 m^3/s to 700 m^3/s and then receded over a 5-h period. A similar surge

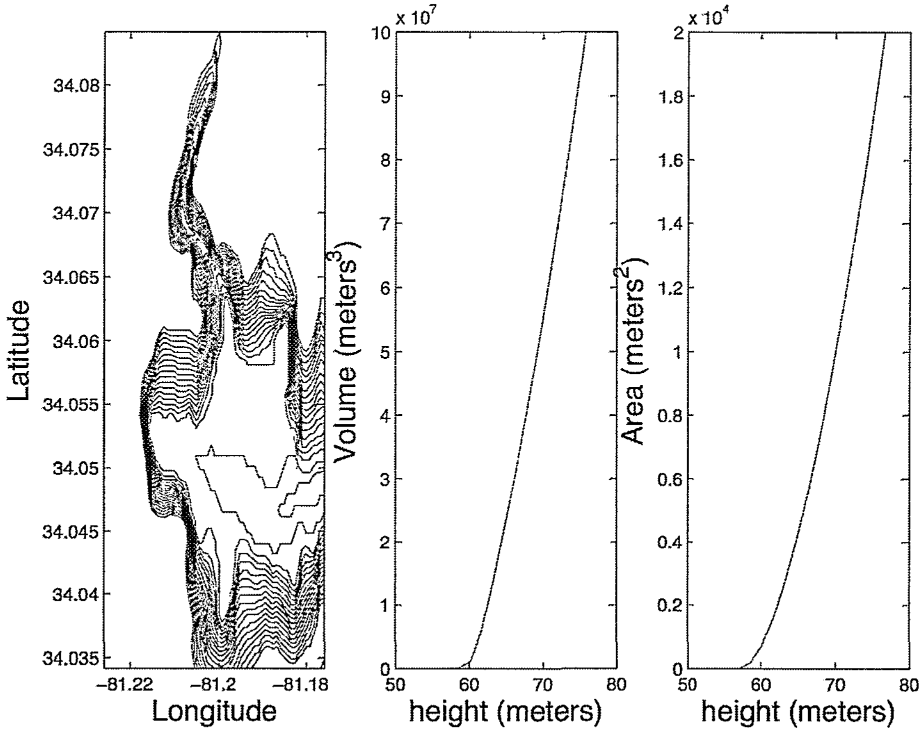
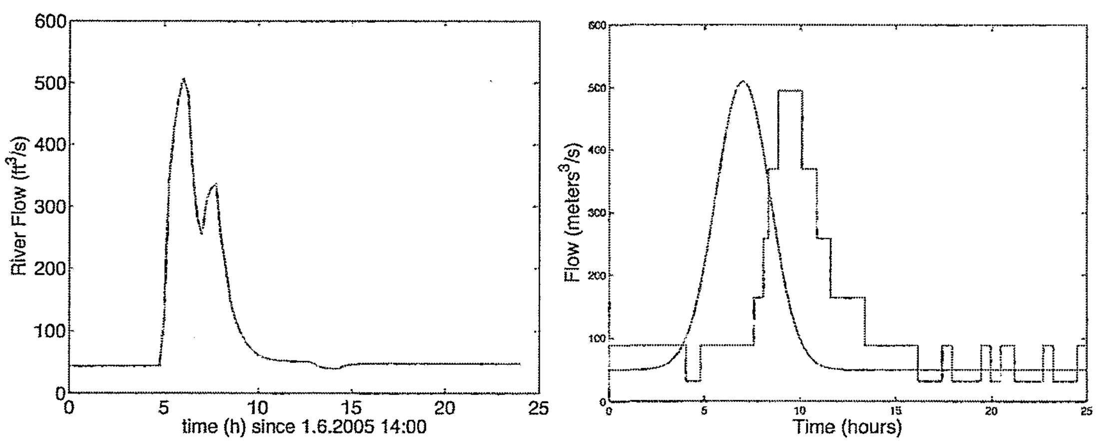


Figure 3. Topographical map and volume and outlet channel area profiles for Reach 4.



Surge down the Saluda River on 6 January 2005. The solid line is flow at the upstream station and the dotted line is flow at the downstream station.

Our simulation of a similar surge.

Figure 4. Actual surge and simulated surge.

was recorded by the Columbia observation station 1.5 h later (see **Figure 4**). We use this event to calibrate our model.

We first calibrate the model to produce a typical river flow at the dam to a value of 60 m^3/s and systematically vary ρ. We find that our model displays stable but oscillatory behavior for $\rho < 0.1$. The oscillations can be traced to jumps in the flow rates between the breaches, and we attribute them to inaccuracy of our channel profiles for small volumes.

For $\rho = 0.1$, our system becomes unstable when large volumes are introduced. Since we are indeed interested in a large flood, we set $\rho = 0.01$. This value is consistent with the idea that the ground cover density, and thus the amount of water stored in the ground cover, increases with distance from the main river channel. The small size of ρ corresponds to the fact that in our equation it scales volume.

Once our model is stable for typical flow volumes, we introduce a "flood" in the form of a Gaussian bump in Q_0 of similar shape to the Jan. 6th event. We adjust ζ until this event arrives at the bottom of reach 3 in 1.5 hours. This occurs when $\zeta = -0.5$, consistent with the three reaches that must be traversed. In calibrating our model for a large flood by using a small one, we assume that the effect of ζ is independent of flood size. A better calibration could be achieved by analyzing observations of a larger flood, but such data are not available from the observation stations [USGS 2005].

Predictions

Our model predicts that the flood waters would travel slowly down the Saluda River Valley, producing extremely high levels of flooding in the upper reaches of the Saluda near Rawls Creek (reach 1) and near Columbia (reach 4). Our results are summarized in **Figure 5** and in **Tables 1–2**. Our numerical simulations suggest that Rawls Creek would flood approximately 32 m but the State Capitol building would remain dry.

Table 1.
The maximum flood volumes in each reach and their corresponding elevations above sea level.

Reach	Max. Flood Vol. ($\times 10^8$ m^3)	Max Flood Elev. (m)	Avg. River Elev. (m)
1	19	87	58
2	15	79	55
3	3.5	68	50
4	2.4	68	45

Table 2.
Elevation above sea level of points of interest.

Point of Interest	Elevation (m)
Lake Murray	120
Saluda River (Just Below Dam)	60
Rawls Creek (Reach 1)	55
Saluda River (Bottom of River)	45
Capitol (Reach 4)	100

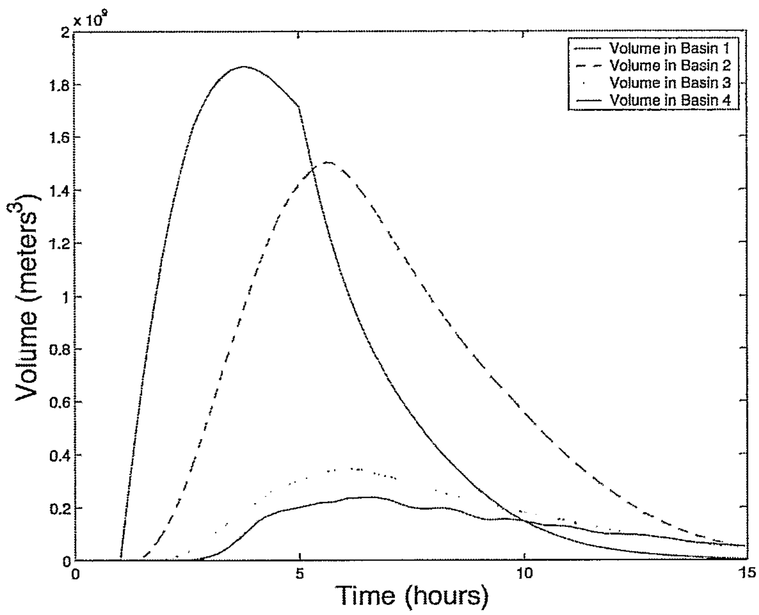


Figure 5. Volumes predicted in each reach as a gradual flood propagates.

Rapid Failure: Dam Break Wave

The complete annihilation of a dam results in a highly turbulent, unsteady flow that is commonly known as a *dam break wave*. The removal of the dam results in the creation of a retreating (negative surge) wave front in response to the sudden reduction in flow depth [Chanson 2004]. In the case of a dam separating two bodies of water, the intersection of the resulting negative surge with a relatively slow moving body of water results in a discontinuity of velocity. Since momentum must be preserved, these two bodies of water cannot intersect without the creation of a second wave moving in a direction opposite to that of the first wave; this second wave is a positive surge (see **Figure 6**).

The propagation properties of the wave resulting from the intersection of the positive and negative surges can be described using equations developed by Saint-Venant. These equations form a coupled system of one-dimensional quasi-linear hyperbolic partial differential equations describing varied unsteady

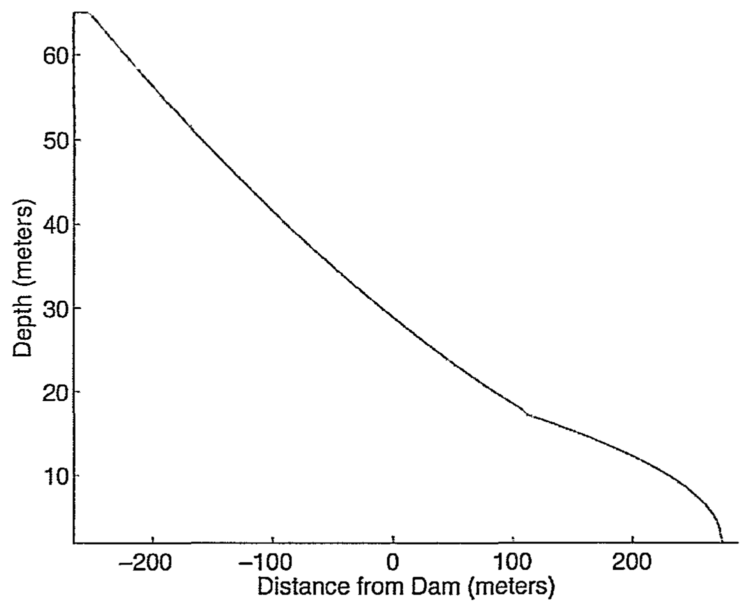


Figure 6. The shape of the wave just after the dam fails. The dam is located at $x = 0$. Note the discontinuity between the positive and negative surges at $x = 100$ m.

channel flow [Freed 1971]:

$$\frac{1}{g}\frac{\partial u}{\partial t} + \frac{u}{g}\frac{\partial u}{\partial x} + \frac{\partial d}{\partial x} + (S_f - S_0) = 0, \tag{17}$$
$$\frac{\partial d}{\partial t} + \frac{\partial du}{\partial x} = \frac{\partial d}{\partial t} + d\frac{\partial u}{\partial x} + u\frac{\partial d}{\partial x} = 0,$$

where u is the mean velocity of the wave, d is the flow depth, x is the direction of propagation, t is time, and g is the acceleration due to gravity. S_f is the friction slope and S_0 is the slope of the canal.

The first equation is known as the *equation of motion* and describes the contribution of various forces to wave propagation, each of which is represented by a separate term:

- the first term describes the local inertia of the wave,
- the second term describes the convective inertia of the wave,
- the third term describes the pressure differential, and
- the fourth term describes the friction and bed slope.

The second equation, known as the *equation of continuity*, expresses conservation of mass.

The Saint-Venant equations assume the following [Chanson 2004; Freed 1971]:

- The flow is one dimensional; motion occurs only in the direction of propagation.

- Vertical acceleration is negligible, resulting in a hydrostatic pressure distribution.
- Water is incompressible.
- Flow resistance is the same as for uniform flow, $S_f = S_0$.

We are interested in describing the flood wave attenuation. In our model, we assume that the total volume of water impounded by the Saluda Dam is released as a single giant surge. The final value to which the peak discharge is attenuated is independent of the magnitude of the initial peak discharge [Ponce et al. 2003]. This allows for generalization of results calculated by our model to waves of arbitrary size.

Solutions to the Saint-Venant Equations

Ponce et al. [2003] derives a solution to **(17)** in the case of a dam failure through sinusoidal perturbation of the steady-state solution. Using spectral analysis, it can be shown that the peak discharge at position X has magnitude

$$q_p = q_{p_0} \exp\left(\frac{-\alpha X}{L_0}\right), \qquad \textbf{(18)}$$

where

$$\alpha = \frac{2\pi}{m^2}\left(\frac{L_0 d_0 B}{V_w}\right)\left[\zeta - \left(\frac{C-A}{2}\right)^{1/2}\right], \quad A = \frac{1}{F_0^2} - \zeta^2, \quad C = \left[A^2 + \zeta^2\right]^{\frac{1}{2}},$$

$$\zeta = \frac{1}{\sigma F_0^2}, \qquad \sigma = \left(\frac{2\pi}{L}\right)L_0, \quad F_0 = \frac{u_0}{\sqrt{g d_0}},$$

with

F_0 the Froude number F_0,

u_0 the steady equilibrium mean flow velocity,

L the perturbation wavelength,

L_0 the reference channel length,

d_0 the steady equilibrium flow depth,

B the average reach width,

V_w the reservoir volume,

g the acceleration due to gravity, and

m the Manning friction coefficient.

The equation for unit width discharge (speed × depth) is

$$q = \frac{N}{N+1} V_{\max} d \tag{19}$$

where $N = 0.4\sqrt{8/f}$, f is the Darcy friction factor, $V_{\max}$ is the maximum reservoir volume, and d is the flow depth.

We compute the wave speed using the empirical data in **Figure** 4 and from it estimate the Darcy friction factor f for the Saluda River Valley. From **(19)**, we also estimate the depth of the wave.

Predictions

Using estimated values of the depth of the water impounded by the dam, the depth of the Saluda River in close proximity to the dam, and the volume of the Saluda River Basin, we approximate the depth of a dam break wavefront as a function of distance from the damsite. **Figure 7** displays the results. The depth of the dam break wave decreases exponentially from an initial value of 65 m to a final value of approximately 4 m at a distance of 20 km from the dam site. This distance roughly corresponds to the distance between the Saluda Dam and the Capitol Building.

Since the Capitol Building sits approximately 50 m above the Saluda River, the possibility of the wave reaching the Capitol Building is extremely small. The probability is further decreased by the simplistic geometry of our model, which approximates the river bed approximated as rectangular and of uniform width and texture. In reality, the river exhibits numerous contractions and expansions and is far from uniform in texture. These qualities would further attenuate the flow depth of the propagating wave.

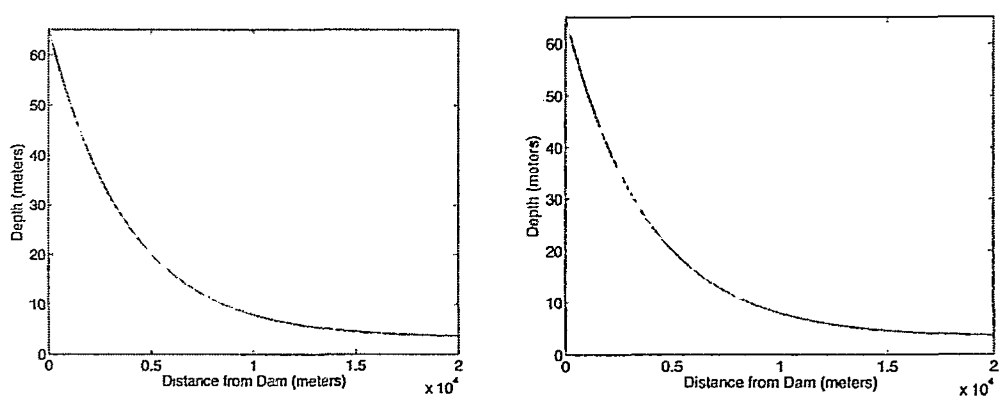


Figure 7. Predicted maximum depth of the floodwave for the upper Saluda River (left) and the entire Saluda River (right).

Our model predicts a wave 40 m high in the vicinity of Rawls Creek. A rapid dam failure would cause significant intrusion of flood waters into the Rawls Creek basin.

Conclusions

In either gradual or rapid failure of the Saluda dam, the effect on the downstream areas would be severe. Our models predict that the waters near Rawls Creek could rise by as much as 40 m (rapid dam failure) or 32 m (gradual dam failure), protruding far into the Rawls Creek basin and other drainages. The waters would not be as high near Columbia and would not reach the Capitol Building. However damage to low-lying areas would be severe, since the water might rise as much as 23 m.

Several improvements could be made to the models:

Gradual Failure Model

- This model successfully describes small surges in the Saluda River. However, extrapolating small events to larger events is inherently problematic; so for a flood of the magnitude that we are considering, we should test the model against larger events in the Saluda and/or large events in comparable rivers.
- Estimates in our erosion model could be strengthened with better information about the material from which the dam is constructed.
- Better profiles of the outlet channel of each reach would allow us to apply the Manning Equation more accurately.

Rapid Failure Model

- We calibrate this model too from a small surge in the Saluda River. A more comprehensive study of waves from other breached dams would provide better data for calibration of this model for large events.
- Access to the river site would provide better estimates for friction factors of the floodplain.
- This model is intended to place an upper bound on the magnitude of the flood wave. Further consideration of factors such as turns in the flood course would increase the accuracy of this model.

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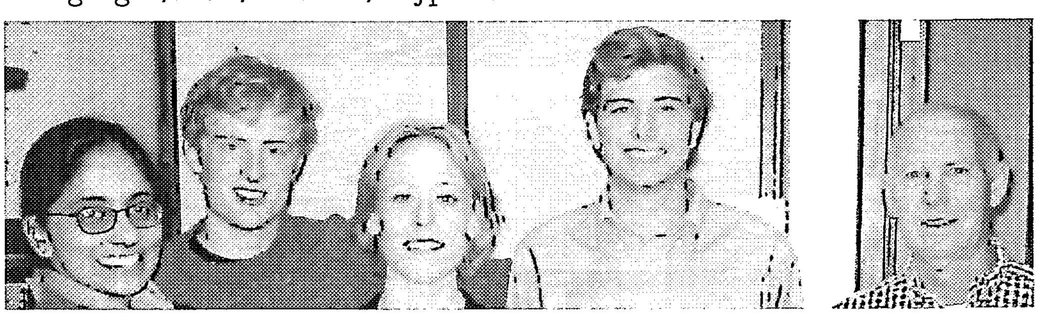

Prof. Rekha Thomas, Ryan Bressler, Christina Polwarth, Braxton Osting, and Prof. Jim Morrow. Prof. Morrow coached the other MCM team from the University of Washington.