Analysis of Dam Failure in the Saluda River Valley

February 8, 2005

Abstract

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 liquification 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 utilize 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, the depth and celerity of a dam break wave resulting from the sudden flood release is approximated through sinusoidal perturbation of the dynamic wave model. Both models are calibrated using flow data measured from two river observation stations. Our results conclude that the flood levels will not reach the Capital Building but will intrude deeply into Rawls Creek.

Contents

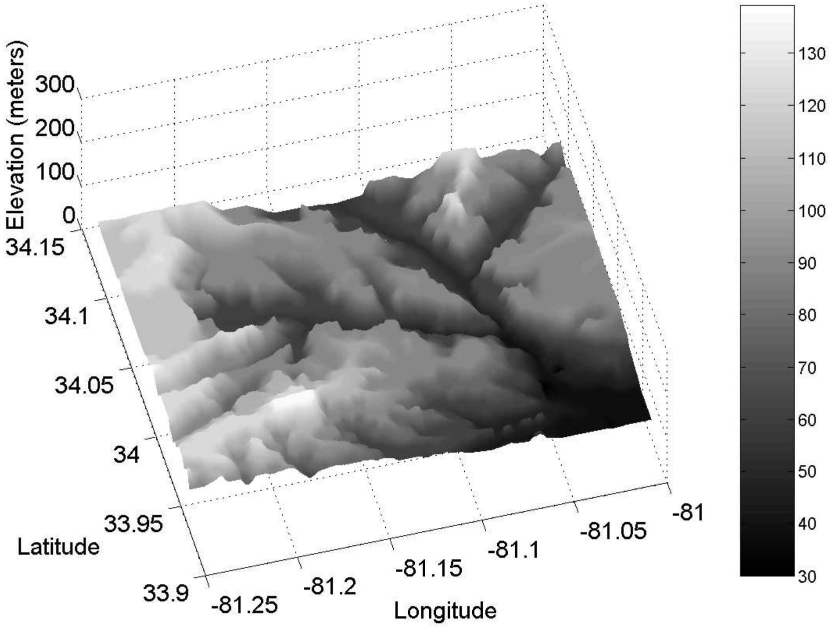

Figure 1: An overview of Lake Murray and the Saluda River Valley generated from the NCRS topographical data [8] we use in our model.

1 Introduction

The Saluda Dam, located 20 kilometers above Columbia, South Carolina, impounds the almost 3 billion cubic meter Lake Murray [5]. It is a large earthen dam of a type that has failed in earthquakes before [7]. In the event of such a failure, the water in Lake Murray would rush down the Saluda River Valley towards Columbia, its one hundred thousand residents, and the State Capitol Building. It is the goal of this paper to present a comprehensive mathematical description of the resulting flood. By describing the possible movements of flood waters, we will estimate their intrusion into Columbia City and the tributaries of the Saluda. See Figure 1 for an overview of the local topography.

A brief survey of earthquake related earthen dam incidents [7] 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 accurately describe both of these situations, we apply two different models.

Section 2 of this paper presents a model for flooding resulting from a breach in the dam. The relatively gradual rate at which water is introduced into the downstream valley suggests that the dispersion of the flood may me modeled using classical open-channel hydraulics. We divide the downstream river course into a number of basins or reaches and then use the Manning formula and the continuity equation to describe the movement of water between them. The flow into the first basin is determined using a model for the destruction of the dam due to breach erosion.

We create a three dimensional topographical model of each basin using three arcsecond resolution data from the NGDC Coastal Relief Model [8]. (Figure 1 was generated using these models.) This allows us to estimate the relationship between the volume contained in each basin and the cross-sectional area of their outflow channel. The Mannings Formula and the Continuity equation yield a system of four coupled first-order differential equations. We numerically integrate this system and calibrate it using data for normal flow in the Saluda River from the USGS river observation stations located just below the Saluda Dam and just above Columbia City [9].

Section 3 of this paper presents a model that describes the propogation of the flood wave resulting from the sudden total failure of the dam due to vibration induced liquification or other causes. The flood wave is described as a sinusoidal perturbation to the steady-state solution of the Saint-Venant equation. The dynamic wave model [2] is applied to determine the surge's propagation.

To simplify calculations, the Saluda River valley is represented as a prismatic channel of rectangular cross section. A small surge recorded by the USGS river observation station in the Saluda valley [9] is used to calibrate the frictional constant governing the rate of attenuation of the flood waves.

2 Gradual Failure

Our model for downstream flooding depends on the conservation of matter as described by the continuity equation. For any given reach of the river the continuity equation states that the change in volume is equal to the difference between the flow in and the flow out:

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

where V denotes the volume of the reach, t is time, and Q_{in}, Q_{out} represent the flow in and out respectively.

We divide the Saluda River Valley into four reaches. Since the amount of water involved in a dam failure flood will 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 will occur along the Saluda. For each reach we set Q_{in} equal to the value of Q_{out} corresponding to the reach above it, ignoring all tributaries. Equation (1) becomes:

$$\frac{dV_1}{dt} = Q_0 - Q_1 \qquad \frac{dV_2}{dt} = Q_1 - Q_2 \tag{2a}$$

$$\frac{dV_3}{dt} = Q_2 - Q_3 \qquad \frac{dV_4}{dt} = Q_3 - Q_4 \tag{2b}$$

where V_n denotes the volume in the nth reach as numbered downstream from the dam and Q_n denotes the flow out of the nth reach. There is no reach number zero; Q_0 denotes the flow into the 1st reach through the breach in the dam (See section 2.1).

Before we can evaluate 2 we must estimate several parameters and relations. In Section 2.1 we estimate the flow out of the reservoir resulting from a dam breach. In section 2.2 we describe our method for determining the flow through each reach and in Section 2.3 we describe the topographical profiles created of the reaches to make this possible. These factors allow us to solve 2 numerically, describing a flood in terms of time. In Section 2.4 we analyze data from USGS river observation stations and use it calibrate our model and in Section 2.5 we present the predictions of the calibrated model.

2.1 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 has depleted or until the dam resists further erosion. This subsection estimates the flow through the breach, Q_0.

We assume that the pre-breach flow into and out of the reservoir can be ignored as they are of opposite sign and of negligible magnitude in comparison to that of the flooding waters. Applying the continuity equation to the reservoir:

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

where H is the height of the reservoir surface above the bottom of the lake, $A_s(H)$ is the surface area of the reservoir at height H and Q_0 is the flow through the breach into the first reach, as above.

Experimental observations have shown that the flow of water through a breach can be simulated by the hydraulics of broad-crested weir flow [3], [4]

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

where Z is the breach bottom height measured from the bottom of the lake. For critical flow conditions, $\alpha = [(2/3)^3 g]^{1/2} = 1.7m/s$ and $\beta = 1/2$. [6]

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 errosion in the breach using the simpelest method that has been used to accurately model dam breaks in the past [6] 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 $\gamma = 0.01$. This value of γ has given good results for linear erosion in the past [6]. Equations (5) and (6) are two 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, trapazoids, or rectangles but rectangles are most often used as the resulting ODEs (5), (6) is solve relatively easily [6]. For simplicity, we model the breach as a rectangle with constant width b such that it erodes only in the vertical direction.

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

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

$$A_s \frac{dH}{dt} = \alpha b(H - Z)^{\frac{3}{2}} \tag{8a}$$

$$\frac{dZ}{dt} = -\gamma\alpha(H - Z)^{1/2} \tag{8b}$$

Equation (8) now permits the solution.

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

$$t(Z) = \frac{2\sqrt{q}\text{arctanh}\left(\sqrt{H(Z)/q}\right)}{\gamma\alpha} - D \tag{9b}$$

where $q = \frac{A_s\gamma}{b}$ and $C = H_0 - Z_0 - q$ where H_0, Z_0 are the initial values of H and Z respectively at $t = t_0$. D is a constant of integration that can also be determined from the intial condtions. Z(t) can be determined from (9b) and $H(Z)$. Then from (3) and (5) we can determine Q_0

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

When $Z(t) = 0$, the dam must stop eroding and from (5) we obtain

$$\frac{dH}{dt} = -pH(t)^{3/2} \tag{11}$$

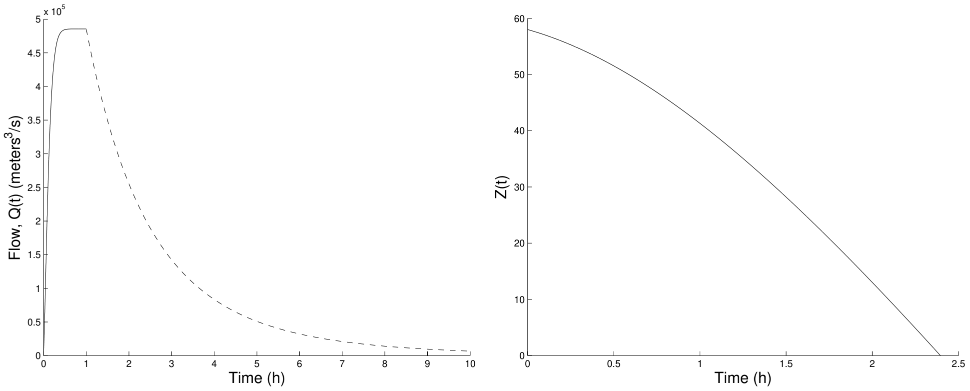

Figure 2: Flow through dam and height of breach bottom as breach enlarges.

resulting in

$$H(t) = \Big(\frac{1}{\sqrt{H_0}} + \frac{q}{2}(t - t_0)\Big)^{-2} \tag{12}$$

Figure 2 depict Q and Z verses time. Notice the discontinuity of the derivative at $t \approx 2$ is the transition between these two solutions.

2.2 Flow Between Reaches and the Manning Equation

We select our reaches so that the river valley is relatively prismatic and narrow (see section 2.3) at their junctions. Assuming that the flow in a flood will 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 flood plain is analyzed as the flow in a prismatic channel using the Manning Equation:

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

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 [10]. We apply it in our model because we may estimate n_m for the Saluda from data for other flood plains. The prismatic nature of each of our narrows indicates that we can apply the Manning equation without correcting for channel irregularities. Typical values of n_m are .5 for a brush coverd flood plain and 1.5 for a tree covered one [10]. We choose a moderate value of $n_m = .1$, assuming that our flood plain is somewhere between.

We set S equal to the slope of the valley floor. Assuming that depth and speed are constant with respect to flow direction in each narrows. Because of this our model will be most accurate in periods where radical changes in volume occur on a time scale greater than the time required to pass through the narrows. From the propogation rates observed in Section 2.4 the time required for the water to pass through the narrows is on the order of 0.1 hours. The flood we wish to consider (section 2.1) rises sharply for one half hour stays steady for 1.5 hours and then trails off gradually. (See Figure 2) Our model of the propagation of the sharpest part of the inital rise may be inaccurate but we will accurately describe the behavior of a large part of the flood.

We estimate the slope of the channel out of each reach from USGS topographical maps [11], [12], [13].

$$S_1 = \frac{1}{1200} \qquad S_2 = \frac{1}{800} \tag{14a}$$

$$S_3 = \frac{1}{600} \qquad S_4 = \frac{1}{800} \tag{14b}$$

We estimate S_4 conservatively so as to produce a worst case scenario for the residents of 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 (see section 2.3). For reach n, 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:

$$Q = uA \tag{15}$$

equation (13) can now be stated as a constraint on **??**:

$$Q_n = A_n u_n = \frac{A_n(\rho V(t-\zeta))}{n_m}\left(\frac{A_n(\rho V(t-\zeta))}{P_n(\rho V(t-\zeta))}\right)^{2/3}\sqrt{S_n} \tag{16}$$

where A_n and P_n are functions of ρV and V is a function of $t - \zeta$.

We introduce ρ and ζ to allow calibrate of the model. They are constants that will be determined from observational data in Section 2.4. ρ describes the storage of water by a reaches ground cover. ζ describes the amount of time it takes a volume of water to travel from the top end of a reach to the bottem. We assume ζ to be constant because of the constant length of our reaches.

2.3 Selection and Analysis of Reaches

We use three arcsecond topographical data from the National Geophysical Data Center Coastal Relief Model to construct the topographical model used in our calculations. To establish corespondences between, V_n the volume of water contained in each basin and the area, A_n, and wet perimiter, P_w of their outflow channels we in we intersect the topographical model of each basin by a plain representing the water level and numerically integrate over the apropriate regions. We construct a database of these values in terms of height to be used as we simulate (2). Figures 3 - 6 display the profiles we generate for volume and area next to topographical maps of the basins used to generate them.

Our accuracy is limited by the 0.2 meter height resilution of the NGDC data. This does not significantly effect the acuracy of our volume estimates but the area and wet perimiter estimates display step function like jumps for small volumes. (The oscilatory behavior seen in Figure 7 is caused by this.) Our model could be improved by conducting better surveys of the outflow chanel of each reach to correct this problem but as we are primarly interested in large volumes we may proced.

To sumerize the requirements our model places upon our selection of reaches:

- The inflow and outflow channel must be narrower then the rest of the reach.

- Said channels must be almost prismatic.
- Water must take the same amount of time to flow down each reach.

To satisfy these conditions we construct our reaches as follws:

- Reach 1: the 6km section from Saluda Dam to the narrows at Correly Island.
- Reach 2: the 6 km sektion from Correly Island to the narrows just below Interstate 20.
- Reach 3: the 6 km section from that point 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 6km strech of downstream channel.

Topographical maps of these reaches are included in Figures 3 - 6. Reaches 1 through 3 satisfy our requirements extremely well. The Congaree River valley widens rapidly into a flood plain below Columbia and there is 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.

2.4 Calibration and Sensitivity Analysis

The US geological survey has two river observation stations in the area of our study one located just below the dam at 34°03′03" North 81°12′35" West and one located just above Columbia at 34°00′50" North 81°05′17" West. Each of these station keeps a log of the last 31 days flow and makes it available over the Internet [9]. On January 6th the Station at Saluda Dam registered a surge of $30000m^3$ flow rates jumped from $27m^3/s$ to $700m^3/s$ and then receded over a five hour period. A similar surge was recorded by the Columbia observation station 1.5 hours later (see Fig. 7). We used this event to calibrate our model.

We first calibrated the model to produce a typical river flow at the dam to a typical value of $60m^3/s$ and systematically varying ρ. We found that our model displayed stable but oscillatory behavior for $\rho < .1$. The oscillations can be traced back to jumps in the flow rates between the breaches and we

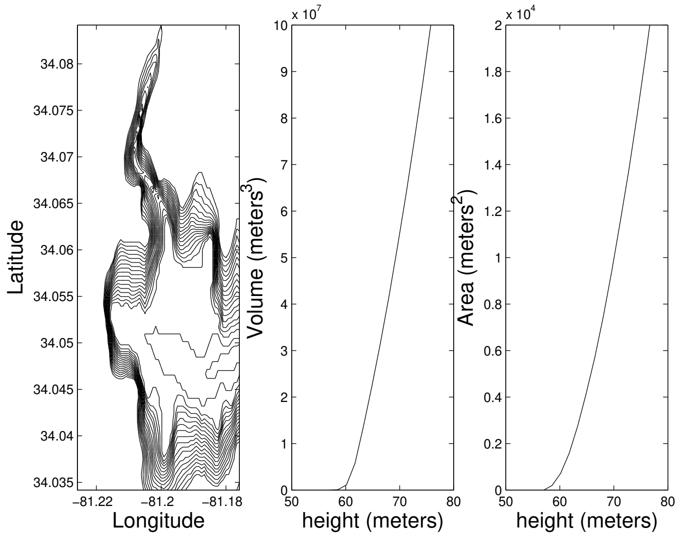

Figure 3: Topographical Map and volume and outlet channel area profiles for Reach 1.

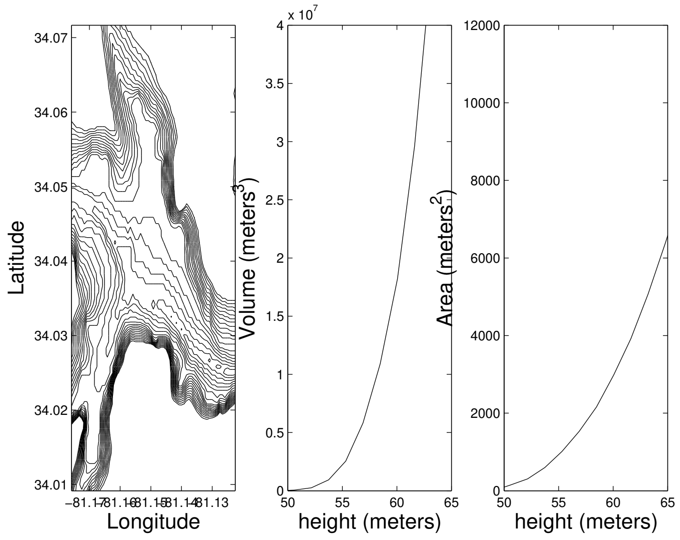

Figure 4: Topographical Map and volume and outlet channel area profiles for Reach 2.

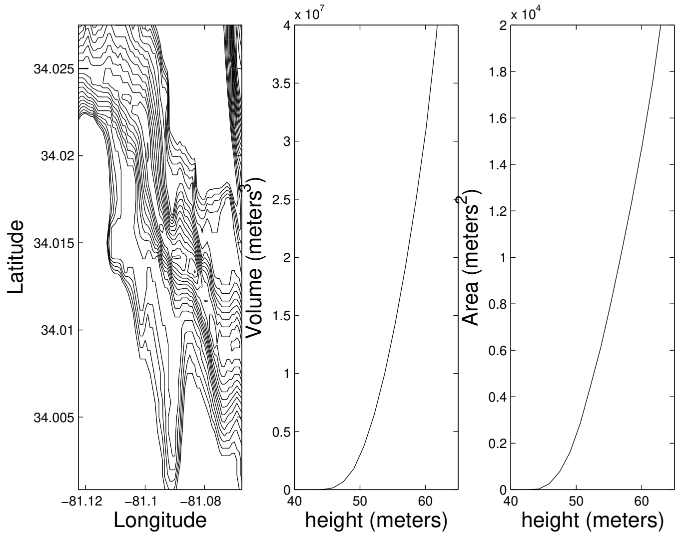

Figure 5: Topographical Map and volume and outlet channel area profiles for Reach 3.

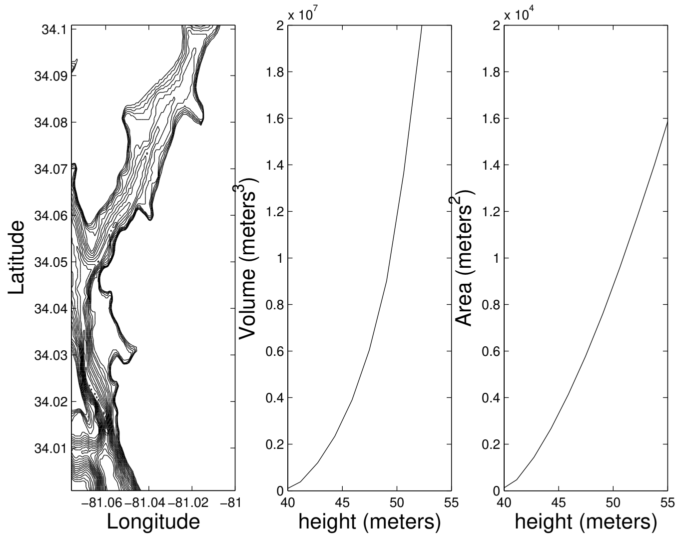

Figure 6: Topographical Map and volume and outlet channel area profiles for Reach 4.

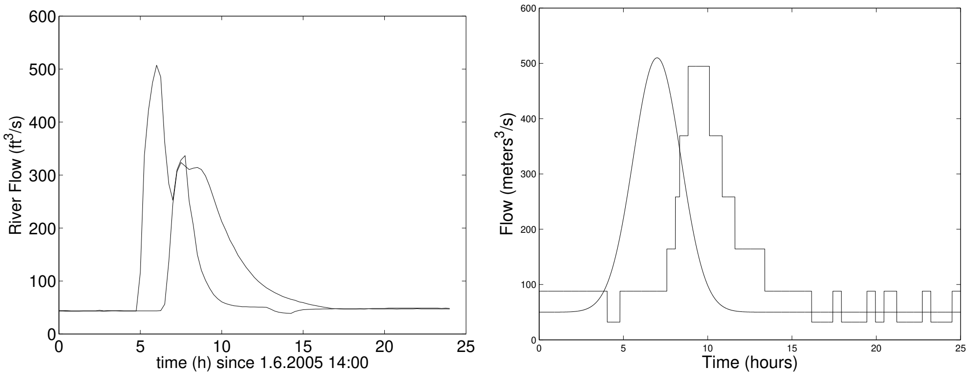

Figure 7: The left hand figure shows the profile of a surge traveling down the Saluda River on January 6, 2005 at 2:00. The right hand figure shows our simulation of the same surge.

attribute them to them to the inaccuracy of our channel profiles for small volumes as discussed in Section 2.3.

For $\rho = .1$, our system becomes unstable when large volumes are introduced. Since we are interested in a large flood of we set $\rho = .01$. This 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 magnitude of ρ is due to the fact that in our equation it scales volume.

Once our model is behaving stably for typical flow volumes we introduce a 'flood' in the form of a Gaussian bump in Q_0 of similar shape to the January 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 transversed. In calibrating our model for a large flood with a small one we have made the assumption that the effect of ζ is independent of flood size. A better calibration could be achieved by analysing observations of a larger flood but such data are not available from the observation stations [9].

2.5 Predictions of Our Model

Our model predicts that the flood waters will travel slowly down the Saluda River Valley produceing extremely high levlels of flooding in the upper reaches

Table 1: The maximum flood volumes in each basin and the corresponding elevation above sea level.

Basin	Max. Flood Vol. (m^3)	Max Flood Elv. (m)	Avg. River Elv. (m)
1	1.9e9	87	58
2	1.5e9	79	55
3	3.5e8	68	50
4	2.4e8	68	45

Table 2: Elevation of points of interest.

Point of Interest	Elevation (m)
Lake Murray	120
Saluda River (Just Below Dam)	60
Rawl Creek	55
Saluda River (Bottom of River)	45
Capitol Building	100

of the Saluda near Rawls Creek and near the Columbia. Our results are sumerized in Fig. 8 and in Tables 1 and 2. The State Capitol building will remain dry.

3 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 [10]. 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 9).

The propagation properties of the wave resulting from the intersection of the

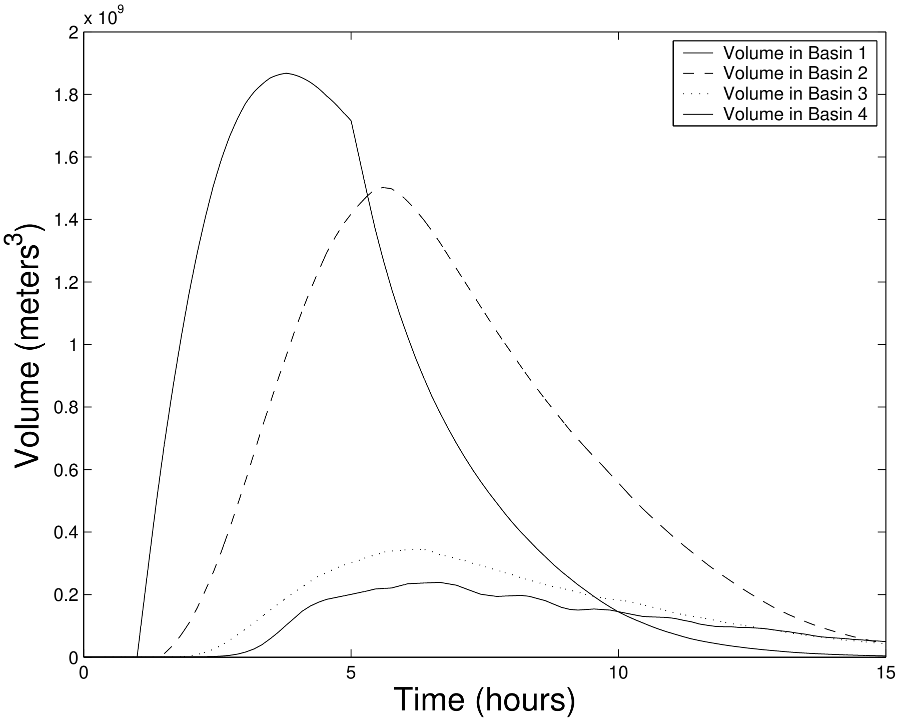

Figure 8: Volumes predicted in each Reach as a gradual flood propagates.

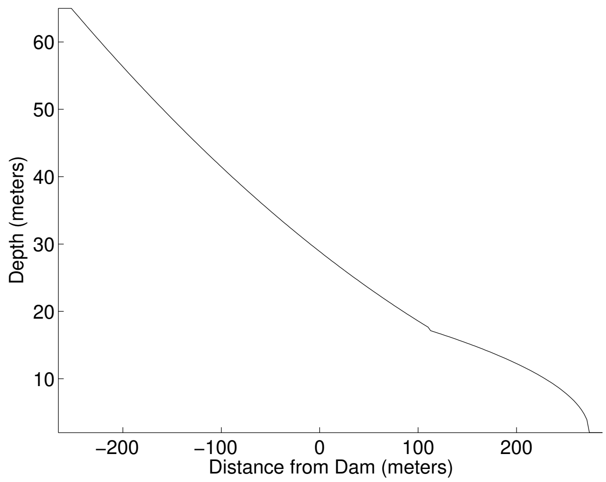

Figure 9: The shape of the wave just after the dam fails. The dam was located at $x = 0$. Note the discontinuity between the positive and negative surges at $x = 0.1$ km.

positive and negative surges can be described using the equations developed by Saint-Venant, known as the Saint-Venant equations. These equations form a coupled system of one-dimensional quasi-linear hyperbolic partial differential equations describing varied unsteady channel flow [14]. The Saint-Venant equations are comprised of two different equations, known respectively as the Equation of Motion and the Equation of Continuity.

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

$$\frac{\partial d}{\partial t} + \frac{\partial du}{\partial x} = \frac{\partial d}{\partial t} + d\frac{\partial u}{\partial x} + u\frac{\partial d}{\partial x} = 0 \tag{17b}$$

where u is the mean velocity of the wave, d is the flow depth, x is the direction of propogation, 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. Equation (17a) 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 in the equation 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 Equation of Continuity (17b) expresses the conservation of mass. The Saint-Venant Equations assume the following [10] [14]:

- The flow is one dimensional; Motion occurs only in the direciton of propogation.
- The vertical acceleration is negligable, 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 [1]. This allows for generalization of results calculated by our model to waves of similiar size.

In [1], a solution to (17) in the case of a dam failure, is derived through sinusoidal perturbation of the steady state solution. Using spectral analysis, it can be shown that the peak discharge at position X has magnitude

$$q_p = q_{p_0} \exp(-\alpha X/L_0) \tag{18}$$

where

$$\alpha = \frac{2\pi}{m^2}\Big(\frac{L_0 d_0 B}{V_w}\Big)\Big[\zeta - (\frac{C-A}{2})^{1/2}\Big] \tag{19}$$

For

$$A = \frac{1}{F_0^2} - \zeta^2 C = \Big[A^2 + \zeta^2\Big]^{(}1/2)\zeta = \frac{1}{\sigma F_0^2}\sigma = (\frac{2\pi}{L})L_0 F_0 = \frac{u_0}{\sqrt{g d_0}} \tag{20}$$

with Froude number F_0, steady equilibrium mean flow velocity u_0, perturbation wavelength L, reference channel length L_0, steady equilibrium flow depth d_0, average reach width B, reservoir volume V_w, and Manning friction coefficient m.

The equation for unit width discharge (speed $\times$ depth) is

$$q = \frac{N}{N+1} V_{max} d \tag{21}$$

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

We computed the wave celerity using the emperical data used in Section 2.4. From this we estimate the Darcy friction factor, f, for the Saluda River Valley. From (21) we also estimate the depth of the wave.

3.1 Results

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 were able to approximate the height/depth of a Dam Break wave front as a function of distance from the damsite. Figure 10 displays the results of such calculations. The height of the Dam Break Wave decreases exponentially from an initial value of 65 meters to a final value of approximately 4 meters at a distance of 20 km from the dam site. This distance roughly corresponds to the distance between the Saluda Dam and the Capital building.

Since the Capital building sits at an elevation of approximately 50 meters above the surface level of the Saluda River, the possibility of the wave reaching the Capital Building is extremely unlikely. This probability of the occurrence of this event is made further decreased by the simplistic Riverbed geometry used in the calculations. The geometry of the Riverbed was 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 will serve to further attenuate the flow depth of the propagating wave. This, in addition with the independence of final unit width peak discharge of initial amplitude, allows us to conclude that a wave resulting from the annihilation of the Saluda Dam will not reach the Capital Building.

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

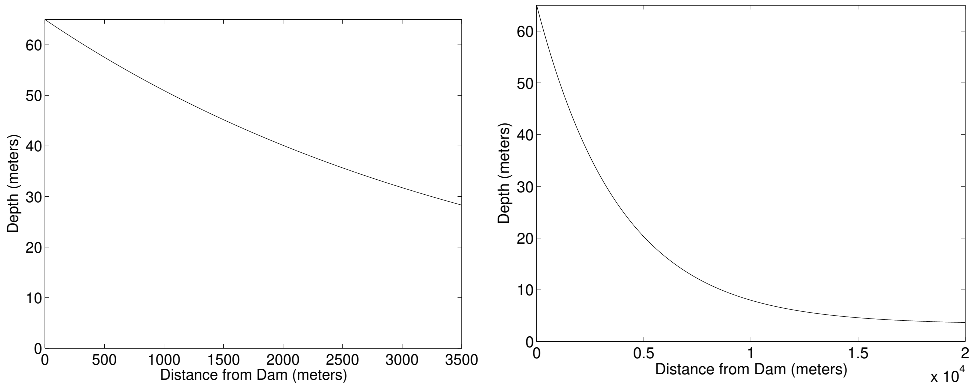

Figure 10: stuff

4 Conclusions

In either gradual failure or rapid failure of the Saluda dam the effect on the down stream areas will be severe. The waters near Rawls Creek could rise by as much as 40 meters protruding far into the Rawls Creek basin and other drainages. The waters will not be as high near Columbia and will not reach the Capitol Building. However damage to the low lying areas will be severe.

Our model has successfully described small events in the Saluda drainage however to be sure of their accuracy for a flood of the magnitude we are concerned with we should test them using larger events in the Saluda and, if possible, extremely large events in other river drainages. We should also construct a better profile of the outlet channels of each basin.

References

[1] Ponce, V. M; A. Taher-shamsi; A. V. Shetty. (2003). "Dam-Breach Flood Wave Propagation Using Dimensionless Parameters." J. Hydraul Div., ASCE 129 (10), 777-782

[2] Ponce, V. M; D.B. Simons. (1977). "Shallow Wave Propagation in Open Channel Flow." J. Hydraul. Div., Am. Soc. Civ. Eng., 103(12), 1461-1476.

[3] Chow, V. T. "Open-Channel Hydraulics." McGra-Hill, New York. 1959.

[4] Pugh, C. A and F. W. Gray Jr. (1984) Fuse Plug Embankments in Auxillary Spillways – Devloping Design Guidlines and Paramaters, Bureau of Reclamation Reports, U.S. Department of the interior, Denver, CO.

[5] *Lake Murray Shoreline Management Program,* South Carolina Electric & Gas Company. Columbia, SC. 1995.

[6] Singh, Vijay P. *Dam Breach Modeling Technology* Kluwer Academic Publishers. Dordecht. 1996.

[7] *Workshop on Analysis and Design of Earth and Rockfill Dams, Lecture Notes Volumes I and II.* New United Process, New Delhi. 1986.

[8] National Geophysical Data Center Coastal Relief Model. Geodas Grid Translator-Design-a-Grid. http://www.ngdc.noaa.gov/mgg/gdas/gd_designagrid.html

[9] USGS Real Time Water Resources Stations 02168504 (Saluda Dam) and 02169000 (Columbia City). NWISWeb Data for the Nation. http://waterdata.usgs.gov/nwis/

[10] Chanson, Herbert. *The Hydraulics of Open Channel Flow: an Introduction.* 2nd Edition. Elsevier, Amsterdam. 2004.

[11] Irmo Quadrangle South Carolina 7.5 Minute Series (Topographic). United States Department of the Interior Geological Survey. Denver, CO. 1971, photorevised 1990.

[12] Columbia North Quadrangle South Carolina 7.5 Minute Series (Topographic). United States Department of the Interior Geological Survey. Denver, CO. 1997.

[13] South West Columbia Quadrangle South Carolina 7.5 Minute Series (Topographic). United States Department of the Interior Geological Survey. Denver, CO. 1994.

[14] Fread, D. L. (1971). "Transient Hydraulic Simulation: Breached Earth Dams." Doctoral Dissertation, University of Missouri-Rolla. 66 pp.