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2D Modeling of Big Bay Dam Failure in Mississippi: Comparison with Field Data and 1D Model Results

M. S. Altinakar, M. Z. McGrath, V. P. Ramalingam & H. Omari
National Center for Computational Hydroscience and Engineering, University of Mississippi, Oxford MS, USA

ABSTRACT: The failure of 17.4m-high Big Bay Dam in southern Mississippi, USA, on March 12, 2004, resulted in a catastrophic flood that damaged 104 homes or businesses, of which 48 were destroyed completely, 37 experienced major damage and the remaining 19 had only minor damage. Shortly after the failure, the USGS and USDA-NRCS jointly surveyed the high water marks and provided maximum elevation data at 42 locations along the entire length of the downstream valley. In addition, data is available on the types of damages sustained by various structures and roads in the inundated area. This data can, thus, be used for validation of 2D numerical models not only based on flow depth and flood arrival time, but also with regard to the failure criteria of structures; which is extremely important for consequence analysis in a risk-based study frame. This paper describes the simulation of the Big Bay Dam failure case using a 2D numerical model developed at the National Center for Computational Hydroscience and Engineering, the University of Mississippi. The numerical model solves full dynamic shallow water equations over a regular Cartesian mesh (such as a DEM) using a shock capturing 1st order upwind finite volume scheme. The simulation results were with the field data on flow depths and arrival times. Simulated flow depths and velocities were also verified based on the damage sustained by individual structures and the available damage criteria. Finally, the 2D simulation results were also compared with an extensive 1D model study carried out by Yochum et al. (2008).

Keywords: Dam break, Numerical modeling, Field observation, Validation, Damage functions

1 INTRODUCTION

1.1 *Description of the Dam-Break Event*

Big Bay Dam was a privately owned earthen dam located on Bay Creek in Lamar County, Mississippi, USA. The principal data for the dam are listed in Table 1. About 1.6km south of the dam, Bay Creek discharges into Lower Little Creek, which flows westward into Marion county and then into the Pearl River.

On March 12, 2004, at about 12:35, the 13 year-old Big Bay Dam failed due to piping. The breaching took place in the area surrounding the spillway structure. Figure 1 shows the satellite images of the dam before and after the failure.

The failure occurred at approximately normal pool level and released 17,500,000m^3 of water to the downstream, creating a catastrophic flood. No human lives were lost; however, the flood waters damaged 104 homes or businesses over a distance of 27km from the dam down to confluence of Lower Little Creek with the Pearl River (Yochum et al., 2008). Of these, 48 were completely destroyed, 37 experienced major damage and the remaining 19 had only minor damage.

The report by NWS (2004) describes the level of damage as “incredible” and compares it to the damage that would be produced by an F3 to F4 tornado. According to the same report, the worst damage occurred within the first 8 km. Water flowing out of the breach flattened all the trees in the wooded area immediately downstream of the dam all the way down to Columbia-Purvis Road. The scar seen in Figure 1 is about 550m long and 280m wide. Further downstream, about 70 m of Tatum-Salt Dome Road was washed out and several wooden structure homes and cars were swept and lodged against a line of trees. NWS (2004) estimates a flow depth of 4.5 to 6 m in this area. Areas along Robbins Road, which parallels Lower Little Creek, were severely affected by flood waters. Every home that was not attached to a concrete slab was moved off its foundation and many homes were damaged or destroyed. A section of the road at the intersection of Robbins Road with Caney Church Road was washed out. Downstream, a section of Luther-Saucier Road was washed off. Houses along the McGraw Road to the west of Luther-Saucier Road were inundated by a water depth of 1.5 m and three structures were moved off their foundation.

Table 1. Principal data on Big Bay Dam

Longitude/Latitude	89º34’19.2”W ; 31º10’57.0”N
Dam Type	Earth fill
Purpose	Recreational
Year Completed	1991
Dam Length	609.6 m (2000 ft)
Dam Height	17.4 m (57.0 ft)
Maximum Discharge	$3m^3/s$ (107.0 cfs)
Maximum Storage	26,365,674 m3 (21375.0 acre-feet)
Normal Storage	13,876,670.7 m3 (11250.0 acre-ft)
Surface Area	3,642,171 (900.0 acres)
Drainage Area	25.3 km^2 (9.7688 square miles)
Hazard Classification	High-hazard

About 10 km downstream of the dam, somewhat slowed flood waters entered Marion County. Nevertheless, several homes were flooded up to a depth 0.9 to 1.5 m. Flow depth over Highway 13 was about 0.5m. The flood washed off the crown of the Pine Burr Road over a large section. The pieces of asphalt from the road were deposited in the gardens of the nearby houses.

Shortly after the incident, the U.S. Geological Survey (USGS) surveyed 42 high water marks along the length of the inundated area in collaboration with the Natural Resources Conservation Service (NRCS) of the U.S. Department of Agriculture (USDA). The locations of these high water marks are shown in Figure 2.

1.2 *1D Numerical Modeling*

Yochum et al. (2008) constructed a one-dimensional (1D) numerical model based on the river cross sections and surveyed bridge sections. The cross sections were developed from a 10-m digital elevation model (DEM) using HEC-GeoRAS 4.0 software (Ackerman, 2005). They were verified based on 7.5-min quadrangle topography maps and aerial photography. The geometry of the main channel was added by estimating it from the elevation contours crossing the stream bed. A total of 105 cross sections were developed and 61 additional cross sections were interpolated. The model included eight bridges based on the cross sections surveyed by USGS in 2005.

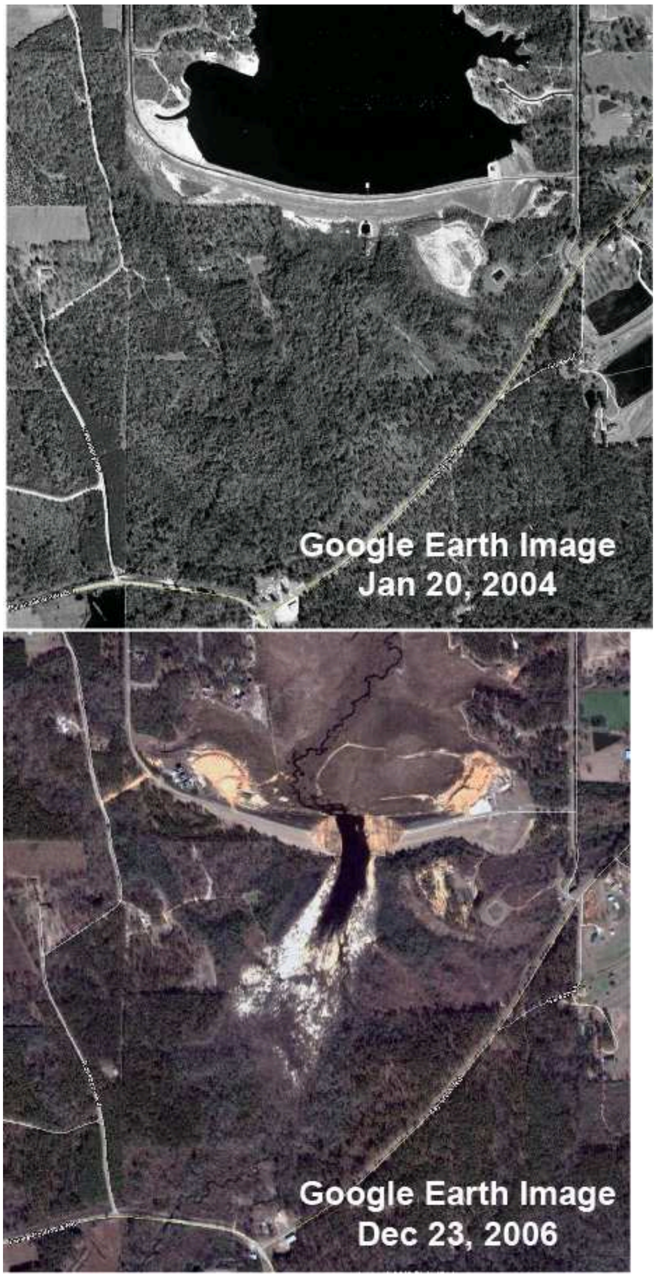

Figure 1. Satellite images of the Big Bay Dam before and after the failure. The scar left by the flood waters is clearly visible in the image below.

The Manning’s roughness coefficients were assigned as follows: 0.05 $m^{-1/3}s$ for main channel; 0.15 $m^{-1/3}s$ for densely vegetated areas; 0.10 $m^{-1/3}s$ for less densely vegetated areas; 0.07 $m^{-1/3}s$ for areas with patchy trees and shrubs, and 0.03 $m^{-1/3}s$ for roadways. The normal depth boundary condition was assumed at the downstream end with the energy slope equal to the valley slope. The computations were carried out using unsteady option in HEC-RAS (Brunner, 2002a and b) and a time step of 10s.

The actual final breach geometry measured on an aerial photo of summer of 2004 indicated a breach width of 70 m at the bottom and 96 at the crest. The local scour hole that formed at the breach location down to the soil-bentonite cutoff wall was ignored and the breach was assumed to stop at the original bed level of 71.3 m a.s.l.

The breach formation time was estimated to be 55 minutes based on the information provided by Burge (2004), who was on site at the time of the failure. The detailed account of the piping initiation and the breaching can be found in Yochum et al. (2008). The volume of the hydrograph developed by HEC-RAS was found to be 17,5000,000 m^3, which agreed with the estimated storage available at the time of the dam-break.

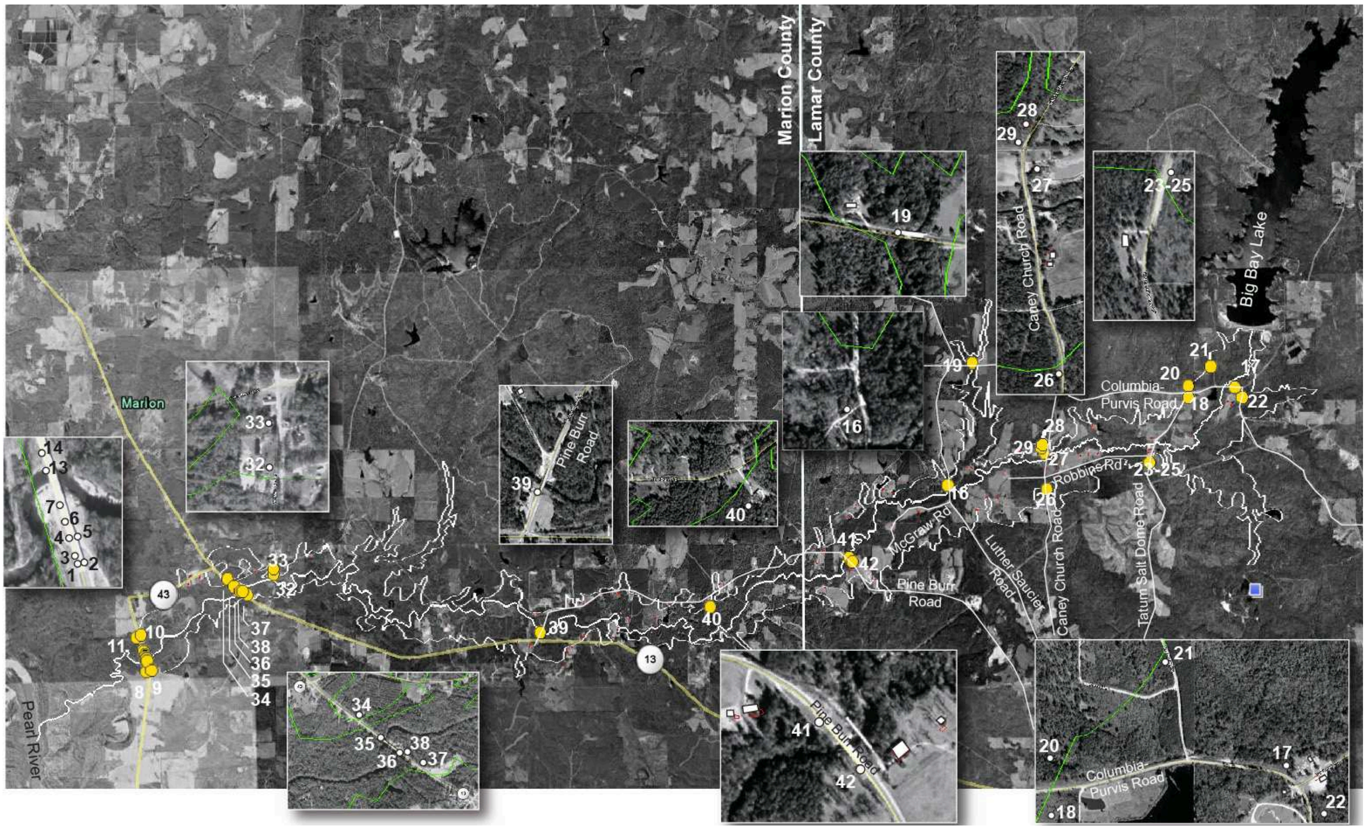

Figure 2. Locations of 42 high water mark observations along the inundated area. The inserts show enlarged pictures of the area around various groups of observation locations. The extent of the inundated area as calculated by Yochum et al. (2008) is also superimposed on the Google Earth image taken on Jan 20, 2004, i.e., about 2 months before the dam break incident.

The inundation area calculated by Yochum et al. (2008) is superposed on the Google Earth image in Figure 2. The differences between water surface elevations computed by HEC-RAS and the observed high-water marks were found to range from -0.02 m to -0.90m and +0.01 to +0.62m. Computed flow depths associated with high-water marks were in the range 5.7m to 9.3m. The peak flow average channel velocities were found to be 1.2m/s to 5.4m/s whereas the average floodplain velocities ranged from 0.2m/s to 1.5m/s. The rise time of the flow hydrographs was found to vary from 24 min at 1.8 km downstream of the dam up to 218 min at the confluence of Lower Little Creek with the Pearl River. Froude numbers in the main channel and the flood plain were less than one, indicating subcritical flow conditions everywhere.

1.3 *2D Numerical Modeling*

In the present study, the Big Bay Dam break was modeled using a two-dimensional (2D) numerical model called CCHE2D-FLOOD, which was developed at the National Center for Computational Hydroscience and Engineering, the University of Mississippi. The salient characteristics of the numerical model are briefly described and the results of the simulation of Big Bay Dam break flood are presented. The computed 2D results are compared with observed high-water marks and the 1D simulations by Yochum et al. (2008). USACE (1985) and RESCDAM (2000) define the collapse criteria for various types of structures subject to flood related loadings. Based on the computed values of flood depths and velocities, the collapse criteria were calculated at various locations where structural damage was observed. These values were compared with the limiting values given by USACE (1985) and RESCDAM (2000).

2 SIMULATION OF BIG BAY DAM BREAK USING CCHE2D-FLOOD

Two-dimensional (2D) simulation of the Big Bay Dam break incident and the resulting flood was modeled using CCHE2D-FLOOD.

2.1 *Description of CCHE2D-FLOOD*

CCHE2D-FLOOD numerically solves shallow water equations over complex topography using a 2D conservative upwinding finite volume scheme. The 2D shallow water equations governing the propagation flood waters are written as:

$$\frac{\partial \mathbf{U}}{\partial t}+\frac{\partial \mathbf{F}}{\partial x}+\frac{\partial \mathbf{G}}{\partial y}=\mathbf{S} \tag{1}$$

The vectors of conserved variables $\mathbf{U}$, fluxes in *x*, $\mathbf{F(U)}$, and *y*, $\mathbf{G(U)}$, directions, and sources, $\mathbf{S}$, are defined, respectively, as:

$$\mathbf{U}=\begin{bmatrix} h \\ Q_x \\ Q_y \end{bmatrix} \mathbf{F}=\begin{bmatrix} Q_x \\ Q_x^2/h \\ Q_xQ_y/h \end{bmatrix} \mathbf{G}=\begin{bmatrix} Q_y \\ Q_yQ_x/h \\ Q_y^2/h \end{bmatrix}$$
$$\mathbf{S}=\begin{bmatrix} 0 \\ -gh(\partial Z/\partial x)-g\left(u\sqrt{u^2+v^2}/C^2\right) \\ -gh(\partial Z/\partial y)-g\left(v\sqrt{u^2+v^2}/C^2\right) \end{bmatrix} \tag{2}$$

where h is water depth, u and v are fluid velocities in x and y directions, Q_x and Q_y are specific discharges in x and y directions, g acceleration of gravity, Z water surface elevation, and C represents Chezy's coefficient of resistance.

Finite volume discretization over a regular mesh results in the following explicit scheme:

$$\mathbf{U}_{ij}^{n+1}=\mathbf{U}_{ij}^{n}-\frac{\Delta t}{\Delta x_i}\left(\mathbf{F}_{i+1/2,j}-\mathbf{F}_{i-1/2,j}\right)$$
$$-\frac{\Delta t}{\Delta y_j}\left(\mathbf{G}_{i,j+1/2}-\mathbf{G}_{i,j-1/2}\right)+\Delta t\mathbf{S}_{ij} \tag{3}$$

Where $\mathbf{F}_{i+1/2,j}$, $\mathbf{F}_{i-1/2,j}$, $\mathbf{G}_{i,j+1/2}$, and $\mathbf{G}_{i,j-1/2}$ are the fluxes at a cell's right, left, top, and bottom interfaces, respectively, $\mathbf{S}_{ij}$ represent source/sink terms, $\Delta x=\Delta y$ the cell size and Δt the time step value. First order upwinding is used to compute the intercell fluxes:

$$\mathbf{F}_{i+1/2j}=\begin{bmatrix} Q_x \\ Q_x^2/h \\ Q_xQ_y/h \end{bmatrix}_{i+k} \quad \mathbf{G}_{ij+1/2}=\begin{bmatrix} Q_y \\ Q_yQ_x/h \\ Q_y^2/h \end{bmatrix}_{j+m} \tag{4}$$

where

$$k=\begin{cases} 0 & Q_x\geq 0 \\ 1 & Q_x\leq 0 \end{cases} \quad m=\begin{cases} 0 & Q_y\geq 0 \\ 1 & Q_y\leq 0 \end{cases} \tag{5}$$

The time step is variable and based on the CFL condition expressed by:

$$N_{CFL}=Max\left[\frac{\Delta t(|u|+\sqrt{gh})}{\Delta x},\frac{\Delta t(|v|+\sqrt{gh})}{\Delta y}\right]\leq 1 \tag{6}$$

A very small water depth threshold (10^{-8}m) is assumed in dry cells. If the computed water depth in a cell is less than some minimal threshold, the discharge components in both directions are set to zero. The numerical code is stable, oscillation-free near discontinuities, robust, and rigorously conserves mass and momentum.

The model has several additional capabilities. Linear terrain features, such as roads and railroad embankments that may affect the propagation of flood waters are taken into account by projecting them onto the computational grid as immersed boundaries that can be overtopped. The cells whose computational stencils are affected by the immersed boundaries are computed using ghost fluid technique (Miglio et al., 2008; Altinakar et al., 2009b). Immersed boundaries are also used to implement 1D-2D coupling capability. The details can be found in Altinakar et al. (2009a and c). The numerical model results can be directly imported into a GIS (Geographical Information System) software for mapping and consequence analysis. CCHE2D-FLOOD is also integrated with a set of GIS-based decision support tools running as an extension of ArcGIS. These tools allow the user to evaluate loss-of-life potential, and urban and agricultural damage, etc. The detailed description of the integrated software can be found in Altinakar et al. (2009a).

2.2 *Modeling of Big Bay Dam Break*

The 10m and 30m resolution DEMs (Digital Elevation Model) for the region of interest were available at the Mississippi Automated Resource Information System (MARIS), which provides access to Mississippi's statewide geographic information system (http://www.maris.state.ms.us/). To reduce the computational time, a 20m resolution DEM was prepared by resampling the 10m DEM.

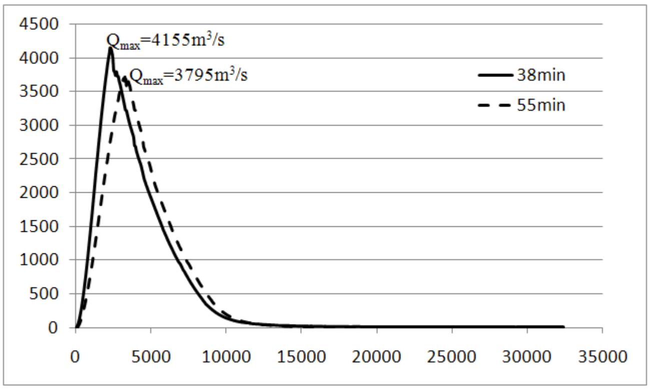

Figure 3. Hydrographs and peak discharges obtained with 55min and 38min breaching times.

The ground level in the vicinity of the failed section is 71.337m a.s.l. Based on a dam height of 17.4m, the dam crest would be at 88.7m a.s.l. Burge (2004) reports that, at the time of the failure, the storage was slightly above the normal pool level of 84.73m a.s.l. Following Yochum et al. (2008), the water surface level in the dam was assumed to be at 84.89m a.s.l. The final breach geometry was assumed to be the same as obtained by Yochum et al. (2008) based on a 2004 aerial picture. The breaching started immediately at the beginning. The final trapezoidal shape had a bottom width of 70.1m at the ground level (71.3m a.s.l.) and a top width of 96m at the elevation of 84.89m a.s.l. The side slopes were 1.30:1 (H:V) on the left side and 0.61:1 (H:V) on the right side.

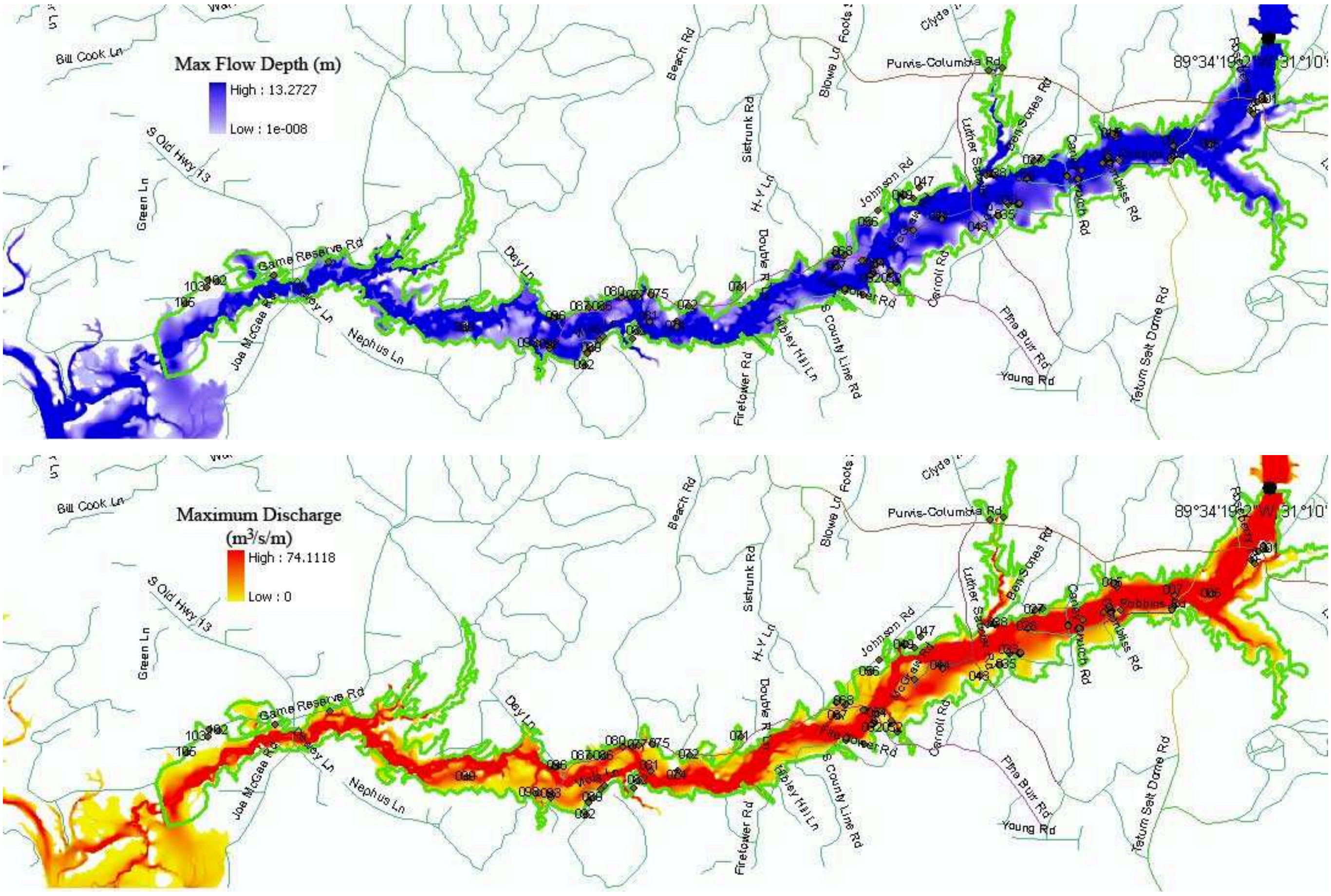

Figure 4. Maps of maximum flow depth (top) and maximum discharge (bottom) computed using CCHE2D-FLOOD. The extent of the inundated area as calculated by Yochum et al. (2008) is also superimposed on the Google Earth image taken on Jan 20, 2004, i.e., about 2 months before the dam break incident.

A 20-m resolution DEM of 1390 cells (27,800m) along North-South and 626 cells (12,520m) along East-West was chosen as computational grid. The river bathymetry was not available. The elevations from the DEM were directly used as the ground elevation. Bridges on Lower Little Creek were not modeled in this study. An average value of n= $0.05m^{-1/3}s$ was used as the Manning's roughness coefficient for the entire computational domain. Simulations were carried out for 32,400s (9 hrs), which lasted about 75,000s (~20.8 hrs) in real-time on a desktop computer with a dual-core AMD microprocessor running at 3.01GHz.

With breaching duration of 55min as reported by Burge at al. (2004) and assumed by Yochum et al. (2008), the resulting hydrograph had a peak discharge of $3795m^3/s$. By trial and error, it was found that a breaching duration to 38min provides a peak discharge of $4155m^3/s$, which is closer to the peak discharge of $4160m^3/s$ reported by Yochum et al. (2008). The simulation results with the breaching duration of 38min are presented in this paper.

3 DAM BREAK SIMULATION RESULTS

The maximum flood depth and maximum flood discharge per unit length computed with CCHE2D-FLOOD for a breach time of 38min are presented in Figure 4 together with the inundation area computed by Yochum et al. (2008).

3.1 *Flood Depths*

As it can be seen, although they are very similar, the inundation area computed by CCHE2D-FLOOD has a somewhat narrower footprint than that obtained from HEC-RAS 1D modeling. It is to be noted that HEC-RAS model ends where the river reaches the flat land around Pearl River, whereas CCHE2D-FLOOD simulation continues all the way down to the confluence with Pearl

River. This is probably due to the fact that, in the relatively flat Pearl River floodplain. river cross sections are no longer well defined and the 1D flow hypothesis does not hold.

In Table 1, observed high-water mark elevations are compared with water surface elevations and depths computed using CCHE2D-FLOOD.

Table 1. Comparison of water surface elevations and depths computed by CCHE2D-FLOOD with observed high-water marks.

High Water Mark ID	Latitude (Degrees)	Longitude (Degrees)	Bed. Elev. 20m-DEM (m a.s.l.)	CCHE2D-FLOOD		Obs. Elev (m a.s.l.)	Bed Elev using 20m DEM (m)	Elevation Differ-ence (m)
				Comp. Elev. (m a.s.l.)	Comp. Depth (m)			
HWML01	31.131389	-89.77494	37.535	38.507	0.971	38.71	1.175	-0.203
HWML02	31.131417	-89.77483	37.509	38.592	1.083	38.74	1.231	-0.148
HWML03	31.131500	-89.77500	37.935	38.646	0.711	38.71	0.775	-0.064
HWML04	31.131778	-89.77508	37.493	38.683	1.190	38.74	1.247	-0.057
HWML05	31.131806	-89.77494	38.010	38.772	0.763	38.74	0.730	0.032
HWML06	31.132028	-89.77517	36.992	38.824	1.832	38.77	1.778	0.054
HWML07	31.132278	-89.77525	36.589	38.888	2.299	38.80	2.211	0.088
HWML08	31.129750	-89.77500	36.576	38.421	1.845	38.68	2.104	-0.259
HWML09	31.129972	-89.77400	37.079	38.461	1.383	38.92	1.841	-0.459
HWML10	31.135389	-89.77594	38.864	39.242	0.378	38.62	-0.244	0.622
HWML11	31.134972	-89.77681	39.355	39.355	0.000	38.47	-0.885	0.885
HWML13	31.132806	-89.77547	35.397	38.919	3.522	38.89	3.493	0.029
HWML14	31.133083	-89.77556	36.585	38.944	2.358	38.89	2.305	0.054
HWML16	31.158333	-89.62703	60.340	61.373	1.033	62.36	2.020	-0.987
HWML17	31.173222	-89.57408	72.206	72.871	0.665	74.92	2.714	-2.049
HWML18	31.171694	-89.58267	71.682	71.706	0.024	73.03	1.348	-1.324
HWML19	31.177056	-89.62247	62.469	62.469	0.000	63.09	0.621	-0.621
HWML20	31.173444	-89.58269	72.523	72.523	0.000	73.49	0.967	-0.967
HWML21	31.176444	-89.57850	74.986	74.986	0.000	75.07	0.084	-0.084
HWML22	31.171694	-89.57272	71.062	71.062	0.000	72.33	1.268	-1.268
HWML23&25	31.161778	-89.58986	69.803	69.803	0.000	69.19	-0.613	0.613
HWML26	31.157694	-89.60878	66.222	66.222	0.000	65.96	-0.262	0.262
HWML27	31.163278	-89.60931	64.280	65.391	1.110	66.45	2.170	-1.059
HWML28	31.164472	-89.60956	64.501	64.584	0.083	65.75	1.249	-1.166
HWML29	31.164000	-89.60975	64.677	64.924	0.247	65.75	1.073	-0.826
HWML32	31.145528	-89.75144	43.969	43.969	0.000	43.13	-0.839	0.839
HWML33	31.144722	-89.75144	42.663	42.841	0.177	43.07	0.407	-0.229
HWML34	31.143972	-89.75989	42.390	42.390	0.000	42.28	-0.110	0.110
HWML35	31.142833	-89.75872	39.984	42.142	2.158	42.12	2.136	0.022
HWML36	31.142083	-89.75761	41.855	42.023	0.168	42.06	0.205	-0.037
HWML37	31.141556	-89.75625	41.416	42.545	1.129	42.49	1.074	0.055
HWML38	31.142083	-89.75717	41.892	42.474	0.582	42.43	0.538	0.044
HWML39	31.135833	-89.70219	46.021	50.884	4.863	50.81	4.789	0.074
HWML40	31.139722	-89.67083	54.907	55.548	0.641	55.66	0.753	-0.112
HWML41	31.147278	-89.64525	54.872	58.109	3.237	59.07	4.198	-0.961
HWML42	31.146639	-89.64461	55.023	58.417	3.394	59.13	4.107	-0.713

In Table 1, observed high-water mark elevations are compared with water surface elevations and depths computed using CCHE2D-FLOOD. The last column shows the difference between computed and observed values. The highlighted differences are greater than 0.70m. The largest difference of -2.049m is observed at HWML17, which remains dry in 2D simulation. In fact, the largest differences are occurring close to the dam (HWML17) and near the bridges (for example HWML41 & 42). Uncertainties in bed elevations, the use of a single roughness value for the entire computational domain, and neglecting the modeling of the bridges may all have contributed to the discrepancies between computed and observed elevations at certain observation points. The bed elevations at high-water mark locations are not known. The second column from the right shows the "observed water depth" calculated by subtracting the bed elevation in 20m DEM from the observed high water mark elevation. It is interesting to note that for six locations negative depths are obtained, indicating that the uncertainties in DEM may have played an important role. In fact, if HWML17 is displaced to the west by about 20m, one obtains a water surface elevation which differs from the observed one by only a few centimeters.

Yochum et al. (2008) do not provide a table of computed water depths at high-water mark locations. It is only mentioned that the differences in elevation range from −0.02m to −0.90m and from 0.01m to 0.62 m. A direct comparison for each high-water mark is, therefore, not possible. For CCHE2D-FLOOD simulation the differences in elevation range from −0.057m to −2.049m and from 0.022m to 0.622 m.

Table 2. Comparison of simulation results from CCHE2D-FLOOD with those simulated by HEC-RAS (Yochum et al., 2008).

Station	HEC-RAS (Yochum et al., 2008)			CCHE2D-FLOOD (present study)			
	River (km)	Peak Discharge (m^3/s)	Rise Time (s)	Peak Discharge (m^3/s)	Front Arrival Time (s)	Rise Time (s)	Peak Discharge Arrival Time (s)
Big Bay Dam Breach	0.0	4,160	3,300	4,155	0	2,280	6,540
Columbia-Purvis Roadway Bridge	1.4	4,000	1,680	4,084	982	NA	NA
Salt Dome Roadway Bridge	3.4	3,010	1,740	3,083	1,889	NA	NA
Chaney Church Roadway Bridge	5.9	2,550	2,400	2,738	3,015	3,525	6,540
Luther Saucier Roadway Bridge	8.6	1,970	3,180	2,341	4,108	3,092	7,200
Pinebur Roadway Bridge (upper)	11.7	1,470	4,560	2,223	5,834	3,166	9,000
Pinebur Roadway Bridge (lower)	19.6	964	7,800	1,702	10,558	4,652	15,210
MS-13 Roadway Bridge	28.6	781	11,820	1,113	17,020	4,700	21,720
MS-43 Roadway Bridge	30.9	762	13,080	653	18,796	4,920	23,716

Table 3. USACE (1985) criteria for structural damage.

Description	1-Story	2-Story
Masonry or concrete bearing walls	$V > 1.92\ m/s$ and $V^2h > 12.80\ m^3/s$	$V > 2.29\ m/s$ and $V^2h > 38.80\ m^3/s$
Wood studs in bearing walls with wood frame	$V > 3.05\ m/s$ and $V^2h > 7.51\ m^3/s$	$V > 4.57\ m/s$ and $V^2h > 7.51\ m^3/s$
Steel studs in bearing walls with steel frame	$V > 5.40\ m/s$ and $V^2h > 10.13\ m^3/s$	$V > 5.40\ m/s$ and $V^2h > 20.00\ m^3/s$

Table 4. RESCDAM (2000) criteria for structural damage.

Structure Type		Partial Damage	Total Damage
Wood framed	Unanchored	$q = Vh \geq 2\ m^2/s$	$q = Vh \geq 3\ m^2/s$
	Anchored	$q = Vh \geq 3\ m^2/s$	$q = Vh \geq 7\ m^2/s$
Masonry, concrete and Brick		$V \geq 2\ m/s$ and $q = Vh \geq 3\ m^2/s$	$V \geq 2\ m/s$ and $q = Vh \geq 7\ m^2/s$

3.2 *Peak Discharges*

Table 2 compares the cross section averaged peak discharges from HEC-RAS simulation with those from CCHE2D-FLOOD simulation. The CCHE2D-FLOOD peak discharges were obtained by integrating the area under maximum discharge versus cross section width. In a 2D simulation the maximum discharge does not necessarily arrive at the same time at all points across the width. The integration of maximum discharges at a cross section is expected to overestimate the peak discharge. Despite this fact, the maximum cross section discharges seem to agree quite well. As it would be expected, the values given by CCHE2D-FLOOD are slightly higher than those of HEC-RAS, especially as the inundation area enlarges in the downstream direction.

4 DAMAGES TO STRUCTURES

4.1 *Flood Related Loadings on Structures*

Structures located in inundation areas are subjected to the following forces: 1) hydrostatic force on the walls; 2) hydrostatic force of saturated soil on underground foundations; 3) buoyancy force; 4) hydrodynamic force; 5) surge impact force; and 6) debris impact force (FEMA/FIA and FEMA, 2000). Tables 3 and 4 present the damage criteria recommended by USACE (1985) and RESCDAM (2000) regarding the flood damage to structures.

4.2 *Evaluation of Damage Criteria for Big Bay Dam Failure Case*

Maximum flood depth, velocity and discharge values simulated with CCHE2D-FLOOD were post processed to compute the damage parameters, Vh and V^2h at the locations where damages to structures and residences were observed. In Table 5, these computed values are compared with the field observations. As it can be seen, the values of the criteria computed from 2D simulation are generally above the limiting values for partial and/or total damage values, indicating an agreement with field observations.

Table 5. Evaluation of RESCDAM (2000) criteria for structural damage at locations where flood damage was observed.

Damage location	$q = Vh\ (m^2/s)$	$V\ (m/s)$
Trees flattened in the wooded area from the dam to Columbia-Purvis Road	$5 \le q \le 50$	$2 \le V \le 12$
70m of Tatum-Salt Dome Road is washed out. Wooden houses and cars were swept against trees.	$6 \le q \le 12$	$2 \le V \le 3.5$
Along Robbins Road unanchored houses were moved, and many were damaged or destroyed.	$2 \le q \le 3.6$	$1.2 \le V \le 2$
A section of the road was washed out at the intersection of Robbins Road with Caney Church Road.	$1.4 \le q \le 2.8$	$1.5 \le V \le 2.1$
Section of Luther-Saucier Road was washed off	$2 \le q \le 12$	$1.4 \le V \le 2.8$
Along the McGraw Road houses were inundated by 1.5 m of water. Three structures were moved away.	$2.3 \le q \le 4.2$	$1 \le V \le 1.9$
	$2.5 \le h\ (m) \le 3.3$	
Crown of the Pine Burr Road was washed off over a large section.	$2 \le q \le 14$	$1.3 \le V \le 3.7$

5 CONCLUSIONS

Two-dimensional simulation of the Big Bay Dam break, including both the dam and the downstream valley, was carried out using the CCHE2D-FLOOD. The 2D results using a 20m-DEM and a constant value of n = 0.05m$^{-1/3}$s are compared with 1D HEC-RAS simulation results obtained by Yochum et al. (2008) and the field measurements. Despite oversimplifications, the results are overall in good agreement. It is believed that the discrepancies between 2D simulation and field measurements can be attributed to uncertainties in the DEM and to the use of an average n-value. More detailed simulations are being carried out currently to evaluate the sensitivity of the simulations to various modeling parameters and to the DEM. Additional field data regarding the damage to individual structures is being compiled. These findings will be reported in future papers.

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