Case Study of the Big Bay Dam Failure: Accuracy and Comparison of Breach Predictions

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Abstract: The Big Bay Dam embankment failure occurred on March 12, 2004, releasing 17,500,000 m^3 (14,200 acre-ft) of water. In all, 104 structures were documented as being damaged or destroyed as a result of this failure. No human lives were lost. This paper documents data gathered and analyses performed on the hydraulics of the failure. High water levels from the failure were marked and measured. A HEC-RAS unsteady flow model was developed. Using observed breach geometry, HEC-RAS provided results that agreed with the measured high water marks from −0.02 to −0.90 m and 0.01 to 0.62 m with associated modeled flow depths ranging from 9.3 to 5.7 m (from 30 to 19 ft). A peak breach flow of 4,160 m^3/s (147,000 ft^3/s) was predicted at the embankment. Breach peak flow prediction equations were found to substantially underpredict the peak flow indicated by HEC-RAS for this failure. HEC-RAS modeling utilizing predicted breach geometry and formation time also underpredicted the peak flow, but by a lesser amount. The National Resources Conservation Service models WinTR-20 and TR 66 were also assessed. WinTR-20 results compared reasonably well with the high water marks for this failure. TR-66 results did not compare well.

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CE Database subject headings: Dam failure; Hydraulic models; Dams, embankment; Dynamic models; Flood routing; One-dimensional flow.

Introduction

On March 12, 2004 the Big Bay Dam embankment, of Lamar County, Miss. failed in the vicinity of the principal spillway 12 years after construction. The Big Bay embankment is approximately 576 m (1,890 ft) long and 15.6 m (51.3 ft) high. With the failure occurring at approximately normal pool, 17,500,000 m^3 (14,200 acre-ft) of water was released, inundating 23 km (14.3 mi) of valley to depths of up to 10.0 m (33 ft) from the dam to the Pearl River. Woody material was stripped from the stream valley for a length of 700 m (2,300 ft) immediately below the dam, after which velocities decreased to such an extent that little vegetation was uprooted. Fig. 1 provides aerial photography of the failed structure, with the upper 220 m (720 ft) of stripped vegetation and sediment deposition also shown.

The inundation impacted Bay Creek and Lower Little Creek in Lamar and Marion Counties. A damage assessment indicated that, within Lamar County, 26 homes were destroyed, 8 homes had major damage, 8 homes had minor damage, 25 mobile homes were destroyed and 1 mobile home had minor damage (MEMA 2004). In Marion County, 1 home was destroyed, 13 homes had major damage, 12 homes had minor damage, 1 mobile home was destroyed, 3 mobile homes had major damage, 3 mobile homes had minor damage, Pine Burr Church suffered major damage, Hub Chapel Church had minor damage, and the Pinebur Volunteer Fire Department suffered major damage (MEMA 2004). In all, 104 structures were documented as damaged or destroyed. No human lives were lost.

To assess the potential accuracy of dam breach inundation predictions for hazard classifications and emergency action plans, high water marks were measured and modeling was performed. Results relate to the Big Bay failure in particular, but shed light on the performance of these tools in general. Additionally, breach peak flow, formation time, and geometry are provided.

High Water Marks

The U.S. Geological Survey (USGS), in cooperation with the Natural Resources Conservation Service (NRCS), surveyed high water marks from the failure throughout the length of the inundated valley. The indicators were marked shortly after the failure occurred, from debris on trees, bridges and buildings. The coordinates of the high water marks, with surveyed elevations, are provided in Table 1. The high water marks are spatially indicated in Fig. 2.

Hydraulic Modeling

Using the geometry of the embankment breach, timing estimated from engineering notes of the failure recorded during the event,

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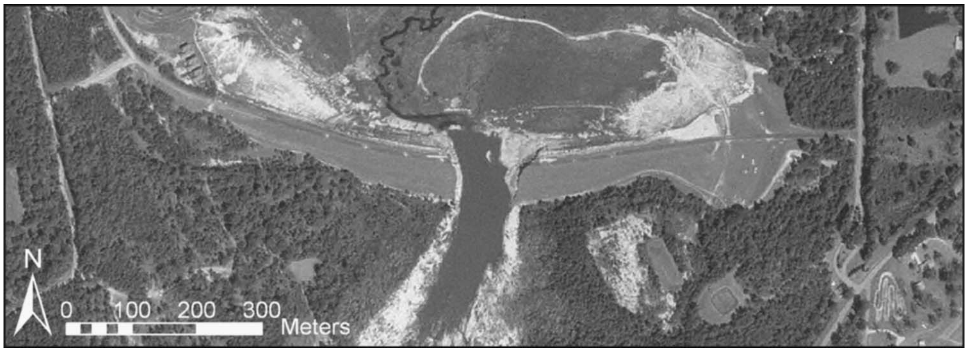

Fig. 1. Big Bay Reservoir embankment

geometry of the cross sections and measured bridge sections, and Manning's n values from a site visit and aerial photography, a one-dimensional hydraulic model of the failure was constructed and verified using the measured high water marks.

Table 1. Measured High Water Marks

Identification	Latitude	Longitude	Elevation (m)
1	31.131389	−89.774944	38.71
2	31.131417	−89.774833	38.74
3	31.131500	−89.775000	38.71
4	31.131778	−89.775083	38.74
5	31.131806	−89.774944	38.74
6	31.132028	−89.775167	38.77
7	31.132278	−89.775250	38.80
8	31.129750	−89.775000	38.68
9	31.129972	−89.774000	38.92
10	31.135389	−89.775944	38.62
11	31.134972	−89.776806	38.47
13	31.132806	−89.775472	38.89
14	31.133083	−89.775556	38.89
16	31.158333	−89.627028	62.36
17	31.173222	−89.574083	74.92
18	31.171694	−89.582667	73.03
19	31.177056	−89.622472	63.09
20	31.173444	−89.582694	73.49
21	31.176444	−89.578500	75.07
22	31.171694	−89.572722	72.33
23 and 25	31.161778	−89.589861	69.19
26	31.157694	−89.608778	65.96
27	31.163278	−89.609306	66.45
28	31.164472	−89.609556	65.75
29	31.164000	−89.609750	65.75
32	31.145528	−89.751444	43.13
33	31.144722	−89.751444	43.07
34	31.143972	−89.759889	42.28
35	31.142833	−89.758722	42.12
36	31.142083	−89.757611	42.06
37	31.141556	−89.756250	42.49
38	31.142083	−89.757167	42.43
39	31.135833	−89.702194	50.81
40	31.139722	−89.670833	55.66
41	31.147278	−89.645250	59.07
42	31.146639	−89.644611	59.13

Modeling of the Big Bay failure was performed using the unsteady flow option of HEC-RAS 3.1.3 (Brunner 2002a,b), which was adapted from the work of Barkau (1982, 1985, 1997) and influenced by the work of many researchers and practitioners, including Fread (1974, 1976), Smith (1978), Liggett and Cunge (1975), Amein and Fang (1970), and Chen (1973). The capabilities of HEC-RAS 3.1 for dam break modeling, in comparison to FLDWAV (Fread and Lewis 1998), are discussed in Holler (2003). Uncertainties in dam breach analysis tools and parameters are discussed in Folmar and Miller (2003) and Wahl (2004).

Breach Hydrograph Development

Timothy Burge of Timothy R. Burge, P.A., Inc. Consulting Engineers (Hattiesburg, Miss.) was on site at the time of failure and recorded the failure with notes. According to Burge (2004), the embankment failed with the reservoir level about 0.15–0.20 m (6–8 in.) above the normal pool elevation of 84.73 m (278.0 ft). A pool elevation of 84.89 m (278.5 ft) was used in this analysis, which corresponds to storage of 17,500,000 m^3 (14,200 acre-ft). The breach hydrograph was created using the dam breach option within HEC-RAS, with breach geometry measured primarily from aerial photography (Fig. 1) and breach formation time developed from Burge (2004).

Summer 2004 aerial photography indicated a bottom breach width of 70.1 m (230 ft), with a top width of 96.0 m (315 ft). The photography indicated a right-hand side slope of 0.61 and a left-hand side slope of 1.3 (horizontal/vertical). Scour was modeled to the original ground elevation of 71.3 m (234 ft). This choice ignores the effects on peak flow of a scour hole that formed approximately to the depth of the soil–bentonite cutoff wall (Burge 2004). An orifice coefficient of 0.8 was used in this analysis, with an initial piping elevation of 72.6 m (238 ft), just above the toe slope elevation and similar in elevation to the bottom of the discharge box. Breach progression was assumed to follow a sine wave.

Breach formation time, defined as the time from breach initiation until the breach has reached its full (geometric) size, was estimated from Burge (2004). According to Burge (2004), increased discharge from an existing seep was first noticed by maintenance man Jim Daughdrill, Jr. on Thursday, March 11, 2004. The seep gradually increased its discharge, with the flow carrying material by the next morning. At midmorning on March 12 the seep was inspected and was noticed that it had about a 0.01 m (0.3 in.) of head height. By 1215 hrs water "shot up out of the hole." Shortly after this the seep was observed to be "spouting

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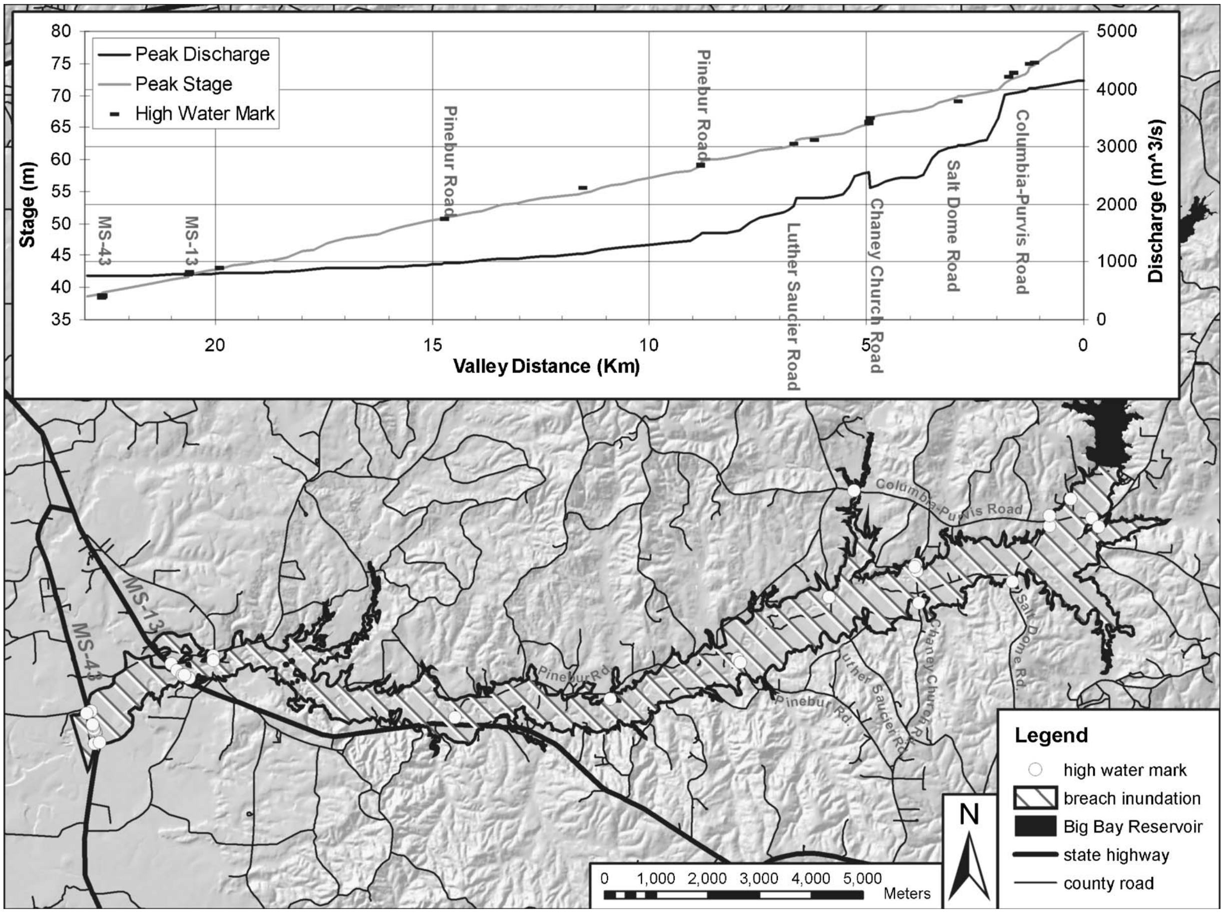

Fig. 2. Big Bay breach flood inundation, plan and profile

approximately 2–3 ft in height, with a diameter of about 18 in." The area around the boil then collapsed and the embankment began to rapidly erode. This is the point where the breach is assumed to start in the analysis, at about 1220 hrs. From the timeline and dimensions recorded by Burge (2004), the final breach dimensions occurred from about 1310 hrs (when "breach widens to ±200 ft") to 1315 hrs (when the flood flow downstream of the embankment reached its maximum extent). Full formation is assumed to occur at 1315 hrs—the breach formation time is estimated to be 55 min. Volume of the HEC-RAS developed breach hydrograph was $17{,}500{,}000\ m^3$ ($14{,}200$ acre-ft), matching the estimated storage available at the time of failure with an initial water surface elevation of 84.89 m (278.5 ft).

Cross-Sectional Development

Nonbridge cross sections were developed from a 10-m digital elevation model (DEM) using HEC-GeoRAS 4.0 (Ackerman 2005), an extension for ArcGIS 8.3. The 10-m DEM was derived from the 7.5-min USGS quadrangle maps. The cross sections were compared to the 7.5-min quadrangle topography maps and color aerial photography to verify their accuracy. The cross sections were altered to incorporate a channel section and thalweg that was estimated from linear interpolation of contours crossing the stream. Ineffective flow areas were noted and coded into the model. Below the embankment, 105 sections were developed, with additional interpolated sections added at a spacing of 61 m (200 ft).

Eight bridges were modeled in this analysis, namely (from upstream to downstream) Columbia-Purvis Rd., Salt Dome Rd., Chaney Church Rd., Luther Saucier Rd., Pinebur Rd. (upper), Pinebur Rd. (lower), MS-13, and MS-43. The geometry of the bridge sections was surveyed by the USGS in 2005. These survey data, combined with the construction plans, were used to model the bridges.

With the event occurring on March 12, vegetation had not yet leafed out—winter leaf-off roughness estimates were used. Manning's n was selected using the standard visual inspection method, with guidance from Chow (1959), Arcement and Schneider (1989), and Brunner (2002a). The horizontal variation in n option available in HEC-RAS (Brunner 2002b) was used, with guidance from aerial photography and a site visit. Stream channel n was estimated to be 0.050 (channel with weeds, stones, and pools). Floodplain n in densely vegetated areas was estimated to be 0.15. Areas of less dense trees (as seen in color-aerial photography), with shrubbery and brush, were given an n of 0.10. Floodplain areas composed of patchy trees and open areas were given an n of 0.075. Open areas were given an n of 0.050 (light brush and trees, in winter). Roadways and shoulders were given an n of 0.030 (short grass, no brush). A roughness polygon was developed in HEC-GeoRAS (Ackerman 2005) using these selections, and exported along with the cross-sectional geometry and downstream reach lengths to HEC-RAS.

A single model was constructed for the analysis. A normal depth downstream boundary condition was used—the energy slope assumed equal to the valley slope. A 10-s computation step was used in the analyses, with a water surface calculation tolerance of 0.031 m (0.10 ft), maximum number of iterations of 40, and a theta implicit weighting factor of 1.0. The cross-sectional HTab parameter was set at the 100-point maximum for all sections.

Modeling Results

The use of actual dam breach geometry combined with standard approaches for model development, including visual inspection for Manning's n selection, produced a relatively accurate model with respect to the measured high water marks. Modeling results are provided in Table 2. Departures in water–surface estimates from measured high water marks ranged from −0.02 to −0.90 m (from −0.07 to −3.0 ft) and 0.01 to 0.62 m (0.03 to 2.0 ft), with an absolute average error of 0.34 m (1.12 ft). Associated modeled flow depths, for high water mark sections, ranged from 9.3 to 5.7 m (from 30 to 19 ft). A comparison of the modeled flow depth with the closest high water mark to the breached embankment indicates close agreement with reality, with a difference of 0.01 m (0.03 ft).

The model indicates that peak breach flow attenuates from 4,160 m^3/s (147,000 ft^3/s) to 761 m^3/s (27,800 ft^3/s) within the 23.0 km (14.3 mi) stream valley. Fig. 2 provides a map of the inundated area with a plot of modeled peak discharge profile and peak stage profile, compared to measured high water marks.

The modeling indicates peak flow average channel velocities ranged from 1.2 to 5.4 m/s (from 3.9 to 17.7 ft/s), average floodplain velocities ranged from 0.2 to 1.5 m/s (from 0.7 to 4.9 ft/s), maximum depths ranged from 5.3 to 10.0 m (from 17.4 to 32.8 ft), bridges and approach roadways overtopped from 0 to 4.2 m (13.7 ft), and a rise time of the floodwave ranged from 24 min 1.8 km (1.1 mi) downstream of the embankment to 218 min just above the Pearl River confluence. All Froude numbers, for both the floodplain and channel, were computed to be less than 1.0, indicating a subcritical flow regime.

Breach Flow Comparison

A comparison was made between two peak breach flow prediction methodologies and the peak flow indicated by the verified HEC-RAS model. Dam breach flow developed from predicted breach geometry was also compared. These are just a few of a number of methods available to predict breach peak flow and geometry. For a more comprehensive list of available tools, see Wahl (1998). Additionally, two other hydraulic models for routing the breach hydrographs through the Lower Little Creek valley were developed and compared to the verified HEC-RAS model.

Peak Flow Prediction

Froehlich (1995b) developed a peak breach flow prediction equation from 22 embankment dam failures that occurred from 1889 to 1989. As documented in Froehlich (1995b), failure modes for these structures included 15 piping, 6 overtopping, and 1 piping or foundation failure. The height of the breaches ranged from 3.7 to 86.9 m (from 12.1 to 285 ft). Peak flows computed to have been released by these breaches ranged from 71 to 65,120 m^3/s (from 2,500 to 2,300,000 ft^3/s). Teton Dam was the largest failure included in Froehlich's analysis.

The prediction equation developed by Froehlich (1995b) is

$$Q_p = 0.607 V_w^{0.295} H_w^{1.24} \tag{1}$$

where Q_p=peak flow (m^3/s); V_w=reservoir volume at the time of failure (m^3), and H_w=height of the water in the reservoir at the time of failure (m). Applying this equation to the Big Bay failure, with a reservoir volume at the time of failure of 17,500,000 m^3 (14,200 acre-ft) and a height of water of 13.5 m (44.3 ft), provides a peak flow estimate of 2,097 m^3/s (74,100 ft^3/s). The Froehlich equation substantially underestimates the 4,160 m^3/s (147,000 ft^3/s) peak flow estimated to have been released from Big Bay Reservoir.

The Natural Resources Conservation Service [formerly known as the Soil Conservation Service (SCS)] has developed peak breach flow prediction equations for application to hazard classification and emergency action plans. These equations are presented in *Technical Release No. 60* (TR-60) (NRCS 2005). Documentation on the development of these equations has been provided in SCS (1986). The following equations, were developed from 31 structures, with two of those being concrete and one being a mine tailings dam. The height of the breaches ranged from 4.3 to 83.8 m (from 14 to 275 ft). Peak flows ranged from 2.1 to 65,100 m^3/s (from 75 ft^3/s to 2,300,000 ft^3/s). Eleven of the structures are the same as in those used by Froehlich (1995b). The next set of equations were developed from 39 breach failures, where unavailable cross-sectional area was not required.

The prediction equations developed by NRCS (SCS 1986) and applicable to Big Bay Reservoir, for $H_w < 31.4$ m (103 ft), are

$$Q_p = 0.000421 B_r^{1.35} \tag{2}$$

where

$$B_r = \frac{V_w H_w}{A} \tag{3}$$

and A=cross-sectional area of the embankment (m^2) at the breach water surface elevation. Additionally, Q_p is neither less than

$$Q_{p,\min} = 1.77 H_w^{2.5} \tag{4}$$

nor greater than:

$$Q_{p,\max} = 16.6 H_w^{1.85} \tag{5}$$

From these equations, the NRCS method recommends a peak flow prediction from 1,180 to 2,040 m^3/s (from 41,800 ft^3/s to 72,200 ft^3/s). The NRCS equations underestimate the peak flow estimated to have been released from Big Bay Reservoir.

Breach Geometry Prediction

Froehlich (1995a) and MacDonald and Langridge-Monopolis (1984) have developed dam breach geometry prediction methodologies. These methods predict average breach width and breach formation time. Additionally, Froehlich (1995a) provided rough recommendations for breach side slope ratios. These values can be inputted into such hydraulic models as NWS FLDWAV (Fread and Lewis) and HEC-RAS (Brunner 2002b) to develop dam failure hydrographs through the use of the broad crested weir and orifice equations.

Data from 63 embankment failures were used to develop the Froehlich (1995a) regression equations. Average breach width (m) is predicted by

Table 2. Selected Modeling Results, HEC-RAS Analysis

River station	Downstream distance: River (km)	Downstream distance: Valley (km)	Peak discharge (m^2/s)	Time of peak (hrs)	Time of rise (min)	Peak water elevation (m)	Measured HWM (m)	Model prediction departure (m)	Maximum water depth (m)	Energy grade slope (m/m)	Average channel velocity (m/s)	Top width (m)	Channel Froude number
500000	0.0	0.0	4,160	1309	49	79.70	—	—	8.3	0.0003	1.5	448	0.20
498022	0.6	0.6	4,080	—	—	77.17	—	—	7.8	0.0052	5.4	592	0.64
496453	1.1	1.0	4,030	1316	30	75.61	—	—	7.0	0.0054	5.2	762	0.64
496048[a]	—	—	4,030	—	—	75.08	75.07	0.01	7.1	0.0051	5.1	678.8	0.62
495418	1.4	1.2	4,020	1317	28	74.33	74.92	−0.59	7.2	0.0037	4.3	640	0.52
495360	1.4	1.2					Columbia-Purvis Roadway Bridge						
495304	1.4	1.3	4,000	1317	28	73.77	—	—	6.7	0.0058	5.1	613	0.64
494615	1.6	1.5	3,950	1319	28	72.99	—	—	6.9	0.0022	3.2	1091	0.41
494416[a]	—	—	3,940	—	—	72.91	73.49	−0.58	6.8	0.0019	2.4	1108	0.31
493821	1.9	1.7	3,930	1321	26	72.60	—	—	6.5	0.0030	1.2	1068	0.16
493621[a]	—	—	3,930	—	—	72.43	73.03	−0.60	6.5	0.0034	2.3	1137	0.30
493222	2.1	1.8	3,900	1322	24	71.90	—	—	6.4	0.0062	5.1	1179	0.67
490312	3.0	2.6	3,060	1336	28	70.19	—	—	7.3	0.0010	2.3	861	0.28
489003	3.4	2.9	3,020	1341	29	69.81	69.19	0.62	9.0	0.0009	2.4	845	0.26
488950	3.4	2.9					Salt Dome Roadway Bridge						
488893	3.4	2.9	3,010	1341	29	69.75	—	—	8.9	0.0009	2.4	841	0.27
487483	3.8	3.3	2,920	1347	32	68.81	—	—	8.2	0.0031	4.4	723	0.50
485728	4.4	3.7	2,510	1350	31	67.86	—	—	7.8	0.0022	3.7	705	0.42
482760	5.3	4.4	2,440	1359	33	67.27	—	—	8.4	0.0006	2.0	912	0.23
481305	5.7	4.7	2,340	1413	43	66.88	—	—	8.7	0.0019	3.5	1003	0.39
480714	5.9	4.9	2,290	1411	40	66.52	66.45	0.07	9.3	0.0015	3.2	846	0.35
480665	5.9	4.9					Chaney Church Roadway Bridge						
480601	5.9	4.9	2,550	1411	40	65.54	65.86[b]	−0.32	8.4	0.0045	5.2	657	0.60
479214	6.3	5.3	2,480	1417	42	64.81	—	—	7.8	0.0036	4.6	893	0.54
475828	7.4	6.0	2,110	1424	41	63.61	—	—	7.8	0.0016	3.1	785	0.36
474299	7.8	6.2	2,100	1440	53	63.46	63.09	0.37	8.0	0.0007	2.1	1300	0.23
472007	8.5	6.6	2,100	1447	53	63.14	—	—	9.8	0.0022	1.3	943	0.14
471950	8.5	6.6					Luther Saucier Roadway Bridge						
471891	8.6	6.7	1,970	1447	53	62.34	62.36	−0.02	9.0	0.0048	1.8	876	0.20
471001	8.8	6.9	1,870	1450	53	61.90	—	—	8.3	0.0019	3.1	824	0.37
466767	10.1	7.9	1,560	1511	61	60.54	—	—	8.1	0.0011	2.5	961	0.29
464073	11.0	8.4	1,500	1535	76	60.05	—	—	8.5	0.0003	1.3	1448	0.14
461687	11.7	8.8	1,500	1543	76	59.72	—	—	10.0	0.0027	1.4	1172	0.15
461620	11.7	8.8					Pinebur Roadway Bridge (upper)						
461552	11.7	8.8	1,470	1543	76	59.12	59.04	0.08	9.3	0.0077	2.3	908	0.25
456875	13.1	10.0	1,300	1613	86	57.25	—	—	8.7	0.0013	3.0	812	0.33
452428	14.5	11.1	1,210	1638	94	55.54	—	—	8.3	0.0017	3.2	804	0.37
450426	15.1	11.5	1,150	1646	95	54.76	55.66	−0.90	8.0	0.0018	3.3	378	0.38
445031	16.8	12.5	1,090	1712	104	53.93	—	—	8.4	0.0007	2.2	628	0.24
441029	18.0	13.5	1,050	1738	114	52.66	—	—	8.5	0.0031	4.4	568	0.50
436955	19.2	14.5	987	1802	123	51.20	—	—	8.5	0.0008	2.2	918	0.25
435769	19.6	14.7	978	1811	129	50.93	50.81	0.12	9.2	0.0014	3.2	828	0.34
435695	19.6	14.7					Pinebur Roadway Bridge (lower)						
435623	19.6	14.8	964	1818	130	50.66	—	—	8.9	0.0021	3.8	790	0.41
432385	20.6	15.7	924	1841	140	49.49	—	—	8.2	0.0017	3.1	770	0.36
427972	22.0	16.7	898	1910	160	47.92	—	—	7.5	0.0014	2.5	626	0.32
422149	23.7	17.7	867	1934	161	45.95	—	—	6.9	0.0026	3.3	647	0.44
413387	26.4	19.0	812	2018	175	44.07	—	—	7.6	0.0008	2.1	586	0.25
408806	27.8	19.9	797	2042	181	42.95	43.10[b]	−0.15	7.9	0.0029	4.0	361	0.47
407910	28.1	20.1	789	2048	184	42.57	—	—	7.8	0.0012	2.6	811	0.30
406278	28.6	20.6	784	2108	197	42.05	42.39[b]	−0.34	7.7	0.0016	2.7	909	0.33
406278	28.6	20.6	784	2108	197	42.05	42.43	−0.38	7.7	0.0016	2.7	909	0.33
406200	28.6	20.6					MS-13 Roadway Bridge						

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Table 2. (*Continued.*)

River station	Downstream distance River (km)	Downstream distance Valley (km)	Peak discharge (m^2/s)	Time of peak (hrs)	Time of rise (min)	Peak water elevation (m)	Measured HWM (m)	Model prediction departure (m)	Maximum water depth (m)	Energy grade slope (m/m)	Average channel velocity (m/s)	Top width (m)	Channel Froude number
406117	28.6	20.6	781	2108	197	41.76	42.09[b]	−0.33	7.4	0.0019	2.9	735	0.36
402865	29.6	21.4	770	2130	206	40.65	—	—	6.7	0.0017	2.8	955	0.36
398757	30.9	22.6	762	2202	218	39.32	38.92	0.40	5.9	0.0013	2.0	809	0.29
398675	30.9	22.6	MS-43 Roadway Bridge										
398594	30.9	22.6	762	2202	218	39.07	38.80	0.27	5.7	0.0019	2.3	781	0.34
395699	31.8	23.0	761	2211	211	38.61	—	—	5.2	0.0008	1.4	1023	0.22

[a]Interpolated cross section.

[b]Average of section HWMs.

$$\bar{B} = 15k_0V_m^{0.32}H_w^{0.19} \tag{6}$$

where V_m=reservoir volume at the time of failure, in millions of cubic meters; and k_0=1.4 for an overtopping failure and 1.0 for other failures. The breach formation time (h), which is defined by the time from the beginning of rapid growth of the embankment breach to the point of maximum breach width (Froehlich 1995a,b), was found to be predicted by

$$t_f = 3.84V_m^{0.53}H_w^{-0.90} \tag{7}$$

Additionally, Froehlich (1995a) provides rough estimates of side slope ratios (1.4 for overtopping failures and 0.9 for other breach events).

Applying these equations to the Big Bay embankment yields an average breach width of 61.5 m (202 ft) and a breach formation time of 1.7 h. The Big Bay embankment had an average breach width of 83.2 m (273 ft), with a 0.92-h breach formation time and side slope ratios of 0.6 and 1.3.

Data from 42 dam failures were used to develop the MacDonald and Langridge-Monopolis (1984) method. This method is graphical, plotting breach formation factor (defined as outflow volume of water multiplied by the breach height) against the volume of material removed, and breach development time against the volume of material removed. This method assumes a breach side slope of 2 vertical to 1 horizontal.

Applying this method to the Big Bay embankment yields a predicted volume of material removed as 61,000 m^3 (80,000 yd^3), which corresponds to an average breach width of 59.6 m (195 ft). The breach formation time is predicted to be 1.0 h.

With the breach geometry and formation time predictions from Froehlich (1995a) and MacDonald and Langridge-Monopolis (1984), two HEC-RAS models were constructed and compared to the measured high water marks and the actual breach geometry model. The result of the modeling indicate that the predicted geometry underpredicts breach flow to a lesser extent than the peak flow prediction equations, with a peak flow of 2,700 m^3/s (95,400 ft^3/s) from Froehlich and 3,130 m^3/s (110,000 ft^3/s) from MacDonald and Langridge-Monopolis (1984).

Using Froehlich (1995a) geometry and timing, the stage was consistently underpredicted compared to the high water marks, with a few exceptions. An absolute error of 0.52 m (1.7 ft) was found to result from using this geometry and timing prediction. The stage was underpredicted in this model initially by as much as 1.3 m (4.4 ft) below Columbia-Purvis Rd. with respect to high water marks, but the error decreased in the downstream direction. It was found that predicted stage approached the high water marks within 0.30 m (1.0 ft) about 6 valley km (3.7 mi), roughly 25% of the 23 km (14.3 mi) model length. The stage predicted using Froehlich geometry agreed with the actual breach geometry model within 0.30 m (1.0 ft) at 5.3 km (3.3 mi), or 23% of the model length. Flow predictions also converged on the actual breach model results in the downstream direction. Discharge varied from being 42% different at the embankment, 16% at 6.0 km, under 10% at 7.9 km (4.9 mi) and less than 1 percent for the lowest quarter of the model.

Using the geometry and timing of MacDonald and Langridge-Monopolis (1984), similar results to the Froehlich geometry and timing model were obtained, with the errors being a bit less, reflecting the shorter breach formation time and higher initial peak breach flow. An absolute error of 0.46 m (1.5 ft) was found to result from using this geometry and timing prediction.

Modeling results and breach inundation mapping for Big Bay reservoir from a model based upon either Froehlich (1995a) or MacDonald and Langridge-Monopolis (1984) predictions, with an assumption of failure at normal pool, would have been less accurate in upstream areas but would have then converged on the actual breach inundation as the model proceeded down the flooded stream valley.

Other Hydraulic Model Performance

Two additional hydraulic routing programs, specifically the NRCS TR-66 and WinTR-20 models, were evaluated for predicting high water levels for this embankment failure. The geometry used in these additional analyses was based upon the HEC-RAS geometry. The breach flow hydrograph predicted by HEC-RAS at the embankment was used as the upstream boundary condition for these models.

The TR-66 model is a simplified dam breach routing program (SCS 1985) created to provide relatively rapid estimates of breach flow downstream of a failed structure for relatively short distances [often less than 8 km (5 mi)]. A simplified Att-Kin routing methodology is implemented in this model. The application of the TR-66 model to the Big Bay Dam failure was only found to be somewhat appropriate for the upper 3.7 km (2.3 mi) valley length of the model. Below this point, excessive attenuation was predicted by the model and the results were not valid. Valid results of the TR-66 modeling are provided in Table 3.

WinTR-20 (NRCS 2004) performs flood hydrograph routing by means of the Muskingum-Cunge procedure. Muskingum-Cunge routing (Merkel 2004) is a coefficient-based procedure, where the coefficients are based on the hydraulic properties of the

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Table 3. Hydraulic Modeling Comparison

River station	Peak stage (m)				Peak discharge (m^3/s)			Time of peak (hrs)	
	TR-20	TR-66	HEC-RAS	HWM[a]	TR-20	TR-66	HEC-RAS	TR-20	HEC-RAS
500000	—	79.71	79.70	—	4,200	4,160	4,160	—	1309
498022	77.02	76.84	77.17	—	3,940	3,570	4,080	—	—
495846	74.83	74.62	74.80	—	3,920	3,200	4,020	1,319	1317
495418	74.37	—	74.33	74.92	—	—	4,020	—	1317
493821	72.73	72.21	72.60	—	3,800	2,860	3,930	—	1321
490312	70.65	69.59	70.19	—	3,340	2,200	3,060	1,329	1336
489003	70.16	—	69.81	69.19	—	—	3,020	—	1341
485728	68.37	67.27	67.87	—	3,090	1,840	2,510	—	1350
481305	67.06	—	66.88	—	2,750	—	2,340	1359	1413
480714	66.51	—	66.52	66.45	—	—	2,290	—	1411
471891	62.91	—	62.34	62.36	2,330	—	1,970	1429	1447
461687	59.98	—	59.72	—	1,870	—	1,500	1509	1543
461552	59.92	—	59.12	59.07	—	—	1,470	—	1543
435769	51.57	—	50.93	50.81	1,330	—	978	1659	1811
407910	43.31	—	42.57	—	1,050	—	789	1909	2048
406278	42.55	—	42.05	42.43	—	—	784	—	2108
401231	40.51	—	40.09	—	1,020	—	765	1939	2144
398757	39.75	—	39.32	38.92	—	—	762	—	2202
395699	38.77	—	38.61	—	996	—	761	—	2211

[a]Measured high water mark.

stream cross sections. WinTR-20 (version 1.00) was used to route the breach hydrograph downstream from the Big Bay Dam failure.

Cross-sectional ratings for WinTR-20 were developed from a steady-flow run of the HEC-RAS model. The cross sections, selected from the HEC-RAS model, were non-interpolated, at intervals of approximately 610 m (2,000 ft). In all, 41 reaches, each represented by a cross-sectional rating, were modeled in WinTR-20 downstream from the Big Bay Dam.

Seventeen measured high water marks from those provided in Table 1 were selected for comparison. Several of the high water marks which occurred at the same river section were averaged into one representative elevation. The WinTR-20 routing generally over predicted high water level by an average of 0.37 m (1.2 ft) for all 17 high water marks. Five of the high water marks were underpredicted. Compared to HEC-RAS, WinTR-20 averaged 0.34 m (1.1 ft) greater elevation at the locations shown in Table 3.

Closer to the dam, WinTR-20 peak flows were lower than HEC-RAS peaks. At about 2.2 km (1.4 mil) downstream from the dam, WinTR-20 peak flows became greater than HEC-RAS peak flows. WinTR-20 peak flows averaged 10 percent greater than HEC-RAS for the Table 3 locations. WinTR-20 peak times occurred sooner than HEC-RAS times, up to 2 h earlier at the downstream end of the study. Table 3 shows peak flow, peak elevations, and time comparisons between TR-66, TR-20, and HEC-RAS, at selected locations.

Discussion

Standard, readily available, and relatively simple hydraulic models and breach flow prediction techniques have been assessed with respect to the Big Bay embankment failure using readily available data. This analysis was performed to assess the ability of hydrologists and engineers to predict the effects of an embankment dam failure. Such predictions are commonly applied to hazard classification and emergency action plans.

Using breach formation time in conjunction with the final breach geometry and reservoir storage at the time of failure, a breach hydrograph was produced and routed downstream using surveyed bridge geometry and a 10-m DEM. The use of HEC-GeoRAS and the DEM was an efficient method for inputting cross-sectional geometry. Geometry is a key source of potential error in any hydraulic model. The DEM was previously created using 3.1 m (10 ft) contours from 7.5-min USGS quadrangles.

The Manning's n selections were another fundamental source of uncertainty in the hydraulic models. The horizontal variation in n option in HEC-RAS was used. Standard n selection techniques, through visual inspection using on the ground fieldwork and aerial photography, were applied in a consistent manner. HEC-GeoRAS was an efficient method for inputting n into the model.

It was found that the use of observed breach geometry and breach formation time produced a HEC-RAS model that performed relatively well in predicting peak flood levels. The most upstream high water mark agreed with the prediction to within 0.01 m (0.03 ft). The average absolute error of the water surface measurements was 0.34 m (1.12 ft), with the model alternating between overpredicting and underpredicting high water marks. The more substantial random error and small systematic error in the prediction of high water marks suggests that the sources of error are localized. As n values affect the whole model, it is likely that channel geometry errors are dominant over n-value errors. As a result, Manning's n values were not calibrated in the modeling. Higher resolution data, such as from an extensive ground survey or from remotely sensed elevation data such as LiDAR (light detection and ranging) would have been preferable to minimize error.

Breach flow predictions were developed using two common direct regression equation-based methods and compared to the predicted breach hydrograph. The predictions were found to substantially underestimate the modeled peak breach flow for the Big

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Bay failure. Breach geometry and formation time prediction equations were also used to develop inputs for additional HEC-RAS analyses. Use of these predictions produced initial peak breach flows that were more accurate than the peak predictions, though still underestimated. Water surface predictions from these models converged downstream upon the results from the actual breach geometry model and the measured high water marks. This indicates that such a prediction model may have been sufficiently accurate, especially in downstream reaches, for application in an emergency action plan for this structure.

The convergence of results from several models that used differing breach parameters, but the same breach volume, shows the usefulness of a sensitivity analysis in the selection breach parameters, to identify where results are known with lesser or greater confidence. Such a sensitivity analysis is recommended with the use of HEC-RAS in breach studies (Brunner 2002b).

Two additional hydraulic models were constructed to assess their accuracy in routing this specific flood wave. The short stretch where the TR-66 analyses was computationally valid, combined with the overattenuation even where it was considered valid, indicates that this simplified method is not appropriate for application to Big Bay Reservoir. The limited model length that TR-66 was designed for (SCS 1985) supports an assumption that this model should only be applied to very small structures, where almost all attenuation would occur within just a few kilometers. In contrast to TR-66, the routing methodology of WinTR-20 was found to perform relatively accurately for this specific application, in comparison to high water marks. On average, WinTR-20 overpredicted the water surface elevations compared to the high water marks and to the HEC-RAS unsteady flow model. Additionally, WinTR-20 routed the breach wave more rapidly than HEC-RAS. As the peak timing of this event is not known, the performance of both models in predicting the timing of the breach inundation is unknown.

Conclusions

On March 12, 2004 the Big Bay Dam embankment of Lamar County, Miss. failed. High water marks were measured shortly after the failure and have been provided in this report. A hydraulic model was constructed of the failure to assess the capabilities of one-dimensional dam breach hydraulic modeling, specifically HEC-RAS unsteady, to predict the movement of a dam breach floodwave through the downstream valley. It was found that HEC-RAS performed well, with relatively accurate agreement of predicted high water levels with measured high water marks. However, random error in the model's projected peak water surface elevations, in comparison to the high water marks, indicates that the geometry extracted from the 10-m DEM introduced measurable error to the analysis. Breach inundation studies should consider gathering higher resolution data, using tools such as LiDAR.

The HEC-RAS modeling indicates that a peak breach flow of 4,160 m^3/s (147,000 ft^3/s) resulted from the failure of the embankment. This modeled value is considered the best estimate of the peak breach flow from this failure. This peak estimate is substantially higher than the peak flow directly predicted through the use of equations provided by Froehlich and NRCS. The breach flow developed from the breach geometry and formation time prediction methodologies of Froehlich and MacDonald and Langridge-Monopolis were also found to underestimate the Big Bay breach flow, though less so than the direct peak flow equations.

The use of differing breach parameters in the modeling provided water surface elevations and flow estimates that converged as the breach waves were routed downstream. This illustrates the power of a sensitivity study of breach parameters to determine where model results are known with greater certainty.

The accuracy of two other hydraulic models in predicting breach attenuation, specifically WinTR-20 and TR-66, both NRCS programs, were assessed. It was found that WinTR-20 results compared reasonably well with the high water marks for this failure. TR-66 results did not compare well, only providing a solution for a short and insufficient distance downstream and overpredicting attenuation.

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