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Advances in Dam Breach Assessment

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Embankment Dam Breach Parameters and Their Uncertainties

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Abstract: Potential flood hazards that would be created by breached embankment dams need to be evaluated to select spillway design floods and to prepare emergency action plans. The breaches are often modeled simply, usually in the shape of a trapezoid that is defined by its final height, base width or average width, and side slopes, along with the time needed for the opening to form completely. Data collected from 74 embankment dam failures were used to develop mathematical expressions for the expected values of the final width and side slope of a trapezoidal breach along with its formation time. Information is provided that allows variances of the predicted quantities to be calculated as well. The findings of the statistical analysis were then applied in a Monte Carlo simulation to estimate the degree of uncertainty of predicted peak flows and water levels downstream from breached embankment dams.

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CE Database subject headings: Embankment; Dam failure; Parameters; Uncertainty principles; Floods.

Introduction

Because almost 80% of the major dams in the United States are formed by embankments constructed from natural erodible materials (U.S. Committee on Large Dams 1975), accurate assessments of the breaches created when they fail are needed to reduce exposure to flood hazards. The various ways in which breaches can form in embankment dams, and the large number of factors that influence the speed and extent of embankment erosion, are difficult to describe with rigorously precise mathematical formulas. Embankment breach formation by overtopping floodwaters has been simulated using complex two-dimensional depth-averaged flow models combined with soil erosion and slope failure algorithms by Froehlich (2004), Wang and Bowles (2006), and Faeh (2007). Models based on one-dimensional cross-section-averaged flow calculations combined with various sediment erosion and transport formulations have also been developed, including those by Cristofano (1965), Brown and Rogers (1977), Lou (1981), Ponce and Tsivoglou (1981), Nogueira (1984), Fread (1985), Al-Qaser (1991), Visser (1998), and Hanson et al. (2005).

Such physically based breach formation models, consisting of coupled simulations of the hydrodynamic and material aspects of embankment erosion, are being used more often to evaluate dam failures as the underlying physical processes are understood better, and as increased computational capabilities enable complicated mathematical calculations to be carried out in acceptably short amounts of time. However, dam failure algorithms of low levels of complexity are still needed when detailed simulations are not required or are not possible to apply easily or conveniently. For these reasons, a simple empirical model that considers a breach to form in a presupposed way, usually growing in the shape of a trapezoid (Fig. 1) is applied often in practice (see, e.g., Fread and Harbaugh 1973; Fread 1984; U.S. Army Corps of Engineers 1978; Brunner 2002).

Values of parameters used in such empirical breach-formation models can be estimated using relations developed based on data collected from historic failures (U.S. Bureau of Reclamation 1988; Froehlich 1995; MacDonald and Langridge-Monopolis 1984; Wahl 2004). The uncertainties of parameter estimates obtained in such a way can be large, as can their effects on planning actions developed to minimize flood hazards. Such uncertainties may be quantified so that reasonable bounds on parameter values can be estimated and used to establish the reliability of predicted outflow hydrographs at the dams, and the peak flood elevations and flow rates at downstream locations given by one-dimensional cross-section-averaged flow calculations.

Data collected from past dam failures are analyzed here using multivariate regression analysis to obtain expressions for the expected values of the average width of a trapezoidal breach, its side-slope ratio (horizontal to vertical), and the formation time (i.e., the needed time from initiation of a breach until it has reached its maximum size), along with their uncertainties. Prediction intervals of estimated parameters based on the levels of the regression variables can be found, and random variations about the expected values can be generated, using results of the analysis. The breach-parameter relations are then applied in a stochastic dam-breach flood model to determine the degree of uncertainty of predicted peak flow rates and water-surface elevations resulting from potential embankment dam failures.

Empirical Breach Formation Models

How a breach forms in an embankment dam depends on numerous factors including the embankment geometry, material composition, construction methods, type and degree of embankment crest and slope protective cover, reservoir dimensions, inflow to the reservoir during failure, and the mode of failure. An empirical

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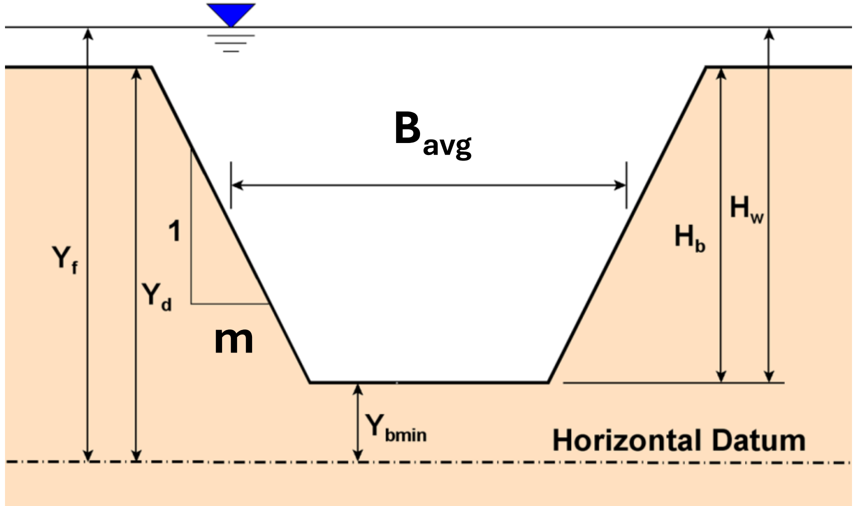

t_f = breach formation time

Breach of Duty (Not): Evaluating the Uncertainty of Dam-Breach Flood Predictions

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ABSTRACT

Dam-breach flood analyses rarely take into account the uncertain nature of numerical model parameters when assessing the flood hazards of potential failures. Solution uncertainty is evaluated here using a point-estimate method that provides a direct and efficient computational procedure to obtain moment estimates (specifically, the means and variances) of calculated peak water-surface elevations, peak discharges, and flood peak travel times. The method is applied to Big Bay Dam to define bounds on downstream flood hazards having specified pexceedance probabilities. Comparison to peak water-surface elevations that were produced by actual failure of the dam shows the approach to provide a reasonable estimate of downstream flood hazard uncertainty.

INTRODUCTION

When deciding if and how much care a dam owner has a duty to provide, the usual standard is how much care an ordinary prudent person in a similar circumstance would take. Where life and limb are at stake, the duty of care is usually the highest possible. An ordinary prudent person engaged in an enterprise that involves substantial risk would not only take every precaution to inform himself of the dangers of his enterprise before undertaking it, but also use unremitting diligence to maintain and inspect the enterprise (Thayer 1916).

Improvements in the sciences of hydraulics and hydrology enable dam owners to determine probable maximum floods that could occur in an area, and what would be the likely result of the failure of their dams. These recent advances have resulted in gradual refinement of the nature and extent of the duty of care a dam owner owes to his downstream neighbors. At least one court has used these developments to decide on the standard of care that a dam owner is obligated to apply (Thomas 2006, page 11).

In situations such as dam failures, many courts permit plaintiffs to invoke the doctrine of *res ipsa loquitur*, that is, the thing (in this case, failure of the dam) speaks for itself. Presumably, the failed structure shouts "negligence!" Certainly, shifting the burden of providing evidence to the defendant dam owner, who is required to explain how his structure failed despite due care, makes a plaintiff's case easier to present and improves his chances of being awarded compensation by a jury. Because

Dam Breach Flood Simulations

Valley cross-sections along Bay Creek and Lower Little Creek between the dam and the Pearl River, including representation of eight bridges, were developed by Yochum at al (2008) and used to prepare HEC-RAS input data files. These data files were obtained and used to carry out multiple simulations of hypothetical piping failures of the Big Bay Dam. The four parameters used by HEC-RAS to simulate a hypothetical dam breach caused by piping (B_{bot}, z, t_f, and C_d) are given in Table 2 for each of the $2^n = 2^4 = 16$ point estimates needed to apply the PEM.

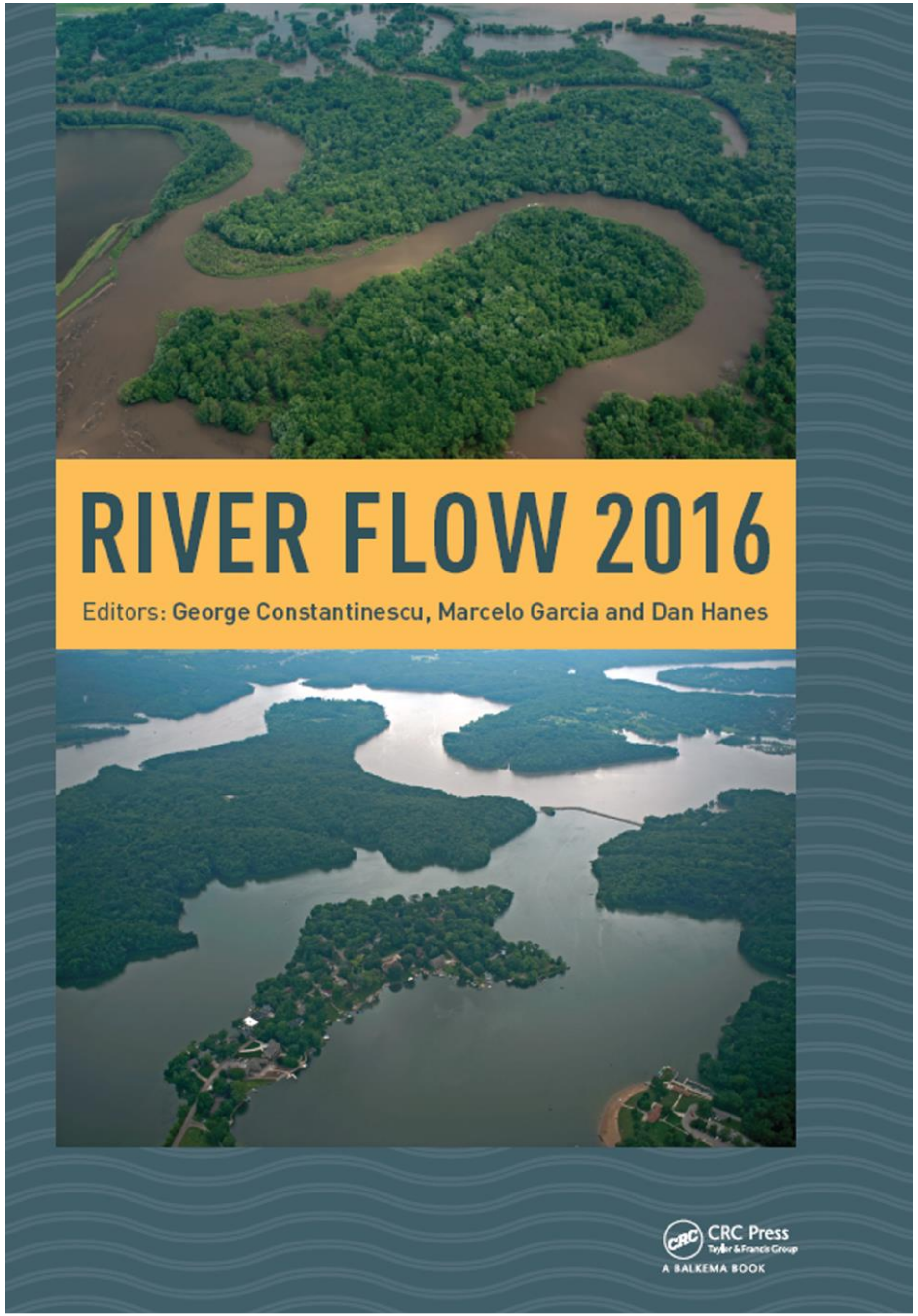
Table 2. Generalized Point Estimate Method HEC-RAS Breach Parameters for Hypothetical Big Bay Dam Failures.

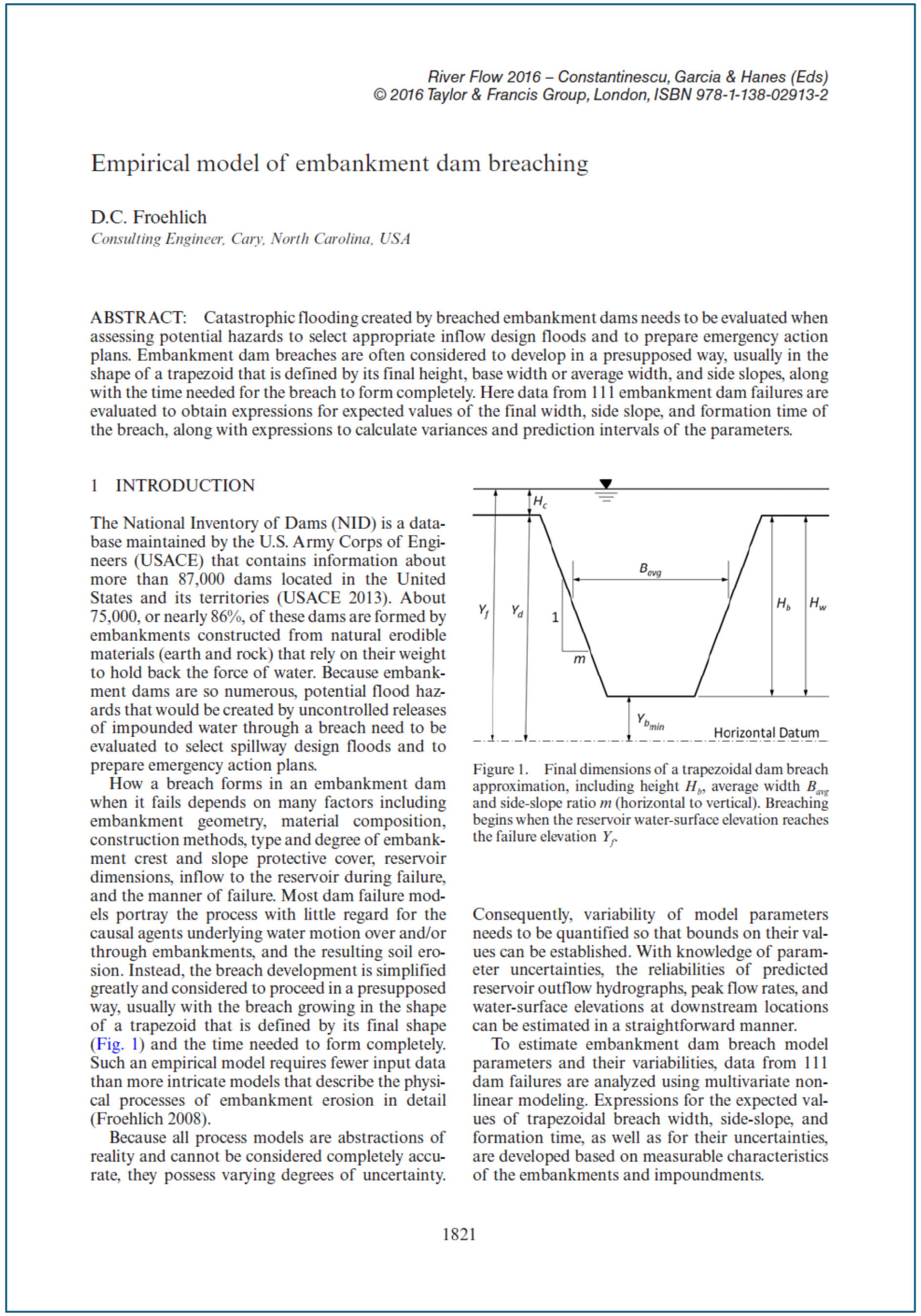
Point estimate number	Symbolic parameter permutation[a]	B_{bot} (m)	z (h:v)	t_f (h)	C_d ($m^{1/2}/s$)
1	+ + + +	73.6	1.52	1.96	1.45
2	+ + + -	73.6	1.52	1.96	1.35
3	+ + - +	73.6	1.52	1.06	1.45
4	+ + - -	73.6	1.52	1.06	1.35
5	+ - + +	93.0	0.286	1.96	1.45
6	+ - + -	93.0	0.286	1.96	1.35
7	+ - - +	93.0	0.286	1.06	1.45
8	+ - - -	93.0	0.286	1.06	1.35
9	- + + +	14.5	1.52	1.96	1.45
10	- + + -	14.5	1.52	1.96	1.35
11	- + - +	14.5	1.52	1.06	1.45
12	- + - -	14.5	1.52	1.06	1.35
13	- - + +	33.9	0.286	1.96	1.45
14	- - + -	33.9	0.286	1.96	1.35
15	- - - +	33.9	0.286	1.06	1.45
16	- - - -	33.9	0.286	1.06	1.35
	Average =	53.7	0.904	1.51	1.40

[a]Plus/minus symbols denote parameter variation about their mean values. Symbols refer to B_{bot}, z, t_f, and C_d, respectively.

Expected values and standard deviations of peak water-surface elevations, peak discharges, and flood peak arrival times at all cross-sections downstream from the dam were calculated from Eq. (3) based on the 16 solution values. Profiles of predicted 99%, 50%, and 1% exceedance probability peak water-surface elevations, along with 17 measured high water marks from the actual failure, are shown in Figure 3. Measured peak elevations along with predicted values and the water-surface elevation standard deviation for the 17 cross sections where high-water marks were obtained are presented in Table 3.

All but two measured high-water marks fall within the band formed by the 99% and 1% exceedance probability estimate profiles. Most of the measured high-water levels are above the 50% exceedance probability estimates (that is, the expected results of a failure). Differences between measured and calculated peak water-surface elevations can be attributed to several factors including the following: (1) uncertainty of high-water mark estimates; (2) one-dimensional flow approximations that consider water-surface elevation to be constant along cross sections; (3) imprecise estimates of flow resistance coefficients, expansion and contraction coefficients, and cross-section representations; and (4) the unaccounted for effect of debris blockages at bridges and other channel constrictions. Standard deviation of water-surface elevation decreases,





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Empirical model of embankment dam breaching

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ABSTRACT: Catastrophic flooding created by breached embankment dams needs to be evaluated when assessing potential hazards to select appropriate inflow design floods and to prepare emergency action plans. Embankment dam breaches are often considered to develop in a presupposed way, usually in the shape of a trapezoid that is defined by its final height, base width or average width, and side slopes, along with the time needed for the breach to form completely. Here data from 111 embankment dam failures are evaluated to obtain expressions for expected values of the final width, side slope, and formation time of the breach, along with expressions to calculate variances and prediction intervals of the parameters.

1 INTRODUCTION

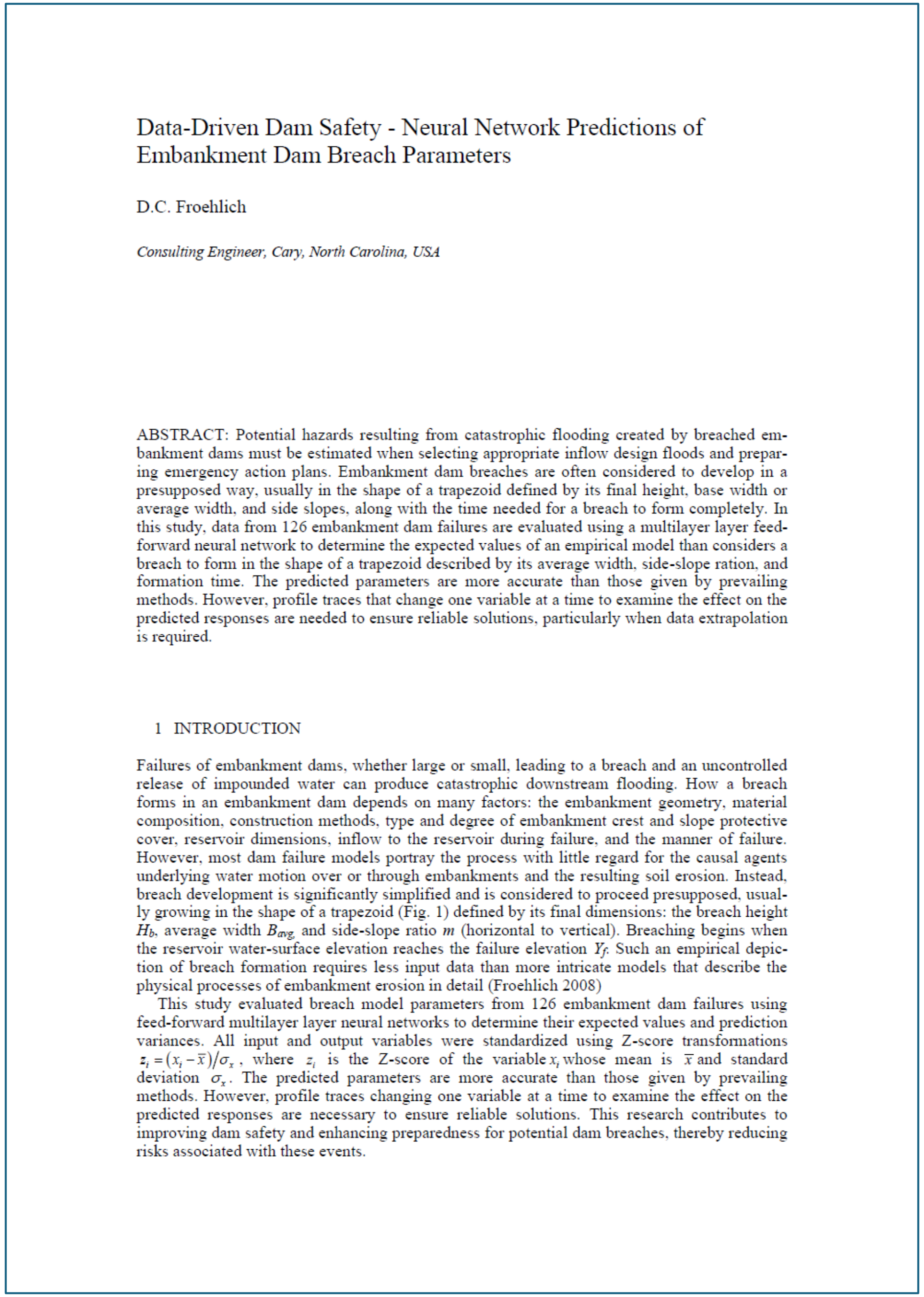
The National Inventory of Dams (NID) is a database maintained by the U.S. Army Corps of Engineers (USACE) that contains information about more than 87,000 dams located in the United States and its territories (USACE 2013). About 75,000, or nearly 86%, of these dams are formed by embankments constructed from natural erodible materials (earth and rock) that rely on their weight to hold back the force of water. Because embankment dams are so numerous, potential flood hazards that would be created by uncontrolled releases of impounded water through a breach need to be evaluated to select spillway design floods and to prepare emergency action plans.

How a breach forms in an embankment dam when it fails depends on many factors including embankment geometry, material composition, construction methods, type and degree of embankment crest and slope protective cover, reservoir dimensions, inflow to the reservoir during failure, and the manner of failure. Most dam failure models portray the process with little regard for the causal agents underlying water motion over and/or through embankments, and the resulting soil erosion. Instead, the breach development is simplified greatly and considered to proceed in a presupposed way, usually with the breach growing in the shape of a trapezoid that is defined by its final shape (Fig. 1) and the time needed to form completely. Such an empirical model requires fewer input data than more intricate models that describe the physical processes of embankment erosion in detail (Froehlich 2008).

Because all process models are abstractions of reality and cannot be considered completely accurate, they possess varying degrees of uncertainty. Consequently, variability of model parameters needs to be quantified so that bounds on their values can be established. With knowledge of parameter uncertainties, the reliabilities of predicted reservoir outflow hydrographs, peak flow rates, and water-surface elevations at downstream locations can be estimated in a straightforward manner.

Figure 1. Final dimensions of a trapezoidal dam breach approximation, including height H_b, average width B_{avg} and side-slope ratio m (horizontal to vertical). Breaching begins when the reservoir water-surface elevation reaches the failure elevation Y_f.

To estimate embankment dam breach model parameters and their variabilities, data from 111 dam failures are analyzed using multivariate nonlinear modeling. Expressions for the expected values of trapezoidal breach width, side-slope, and formation time, as well as for their uncertainties, are developed based on measurable characteristics of the embankments and impoundments.



Data-Driven Dam Safety - Neural Network Predictions of Embankment Dam Breach Parameters

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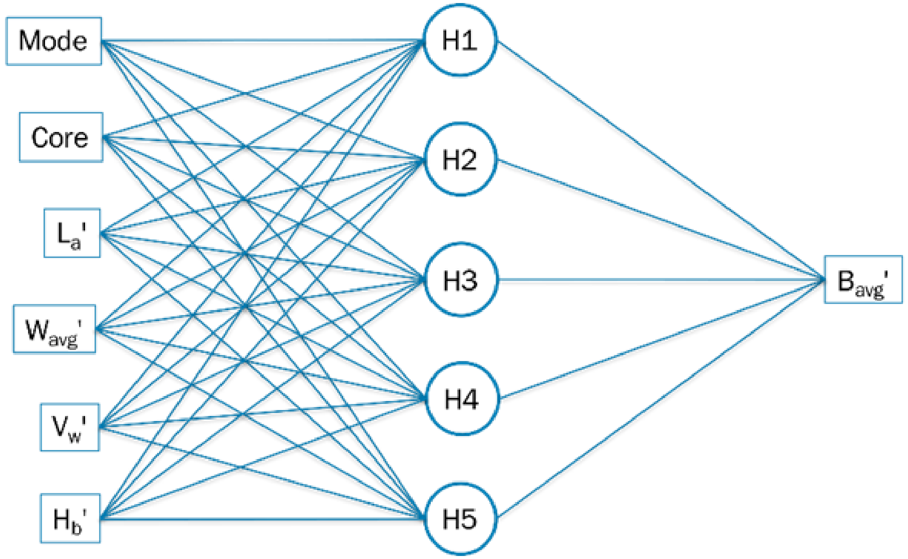
ABSTRACT: Potential hazards resulting from catastrophic flooding created by breached embankment dams must be estimated when selecting appropriate inflow design floods and preparing emergency action plans. Embankment dam breaches are often considered to develop in a presupposed way, usually in the shape of a trapezoid defined by its final height, base width or average width, and side slopes, along with the time needed for a breach to form completely. In this study, data from 126 embankment dam failures are evaluated using a multilayer layer feed-forward neural network to determine the expected values of an empirical model than considers a breach to form in the shape of a trapezoid described by its average width, side-slope ration, and formation time. The predicted parameters are more accurate than those given by prevailing methods. However, profile traces that change one variable at a time to examine the effect on the predicted responses are needed to ensure reliable solutions, particularly when data extrapolation is required.

1 INTRODUCTION

Failures of embankment dams, whether large or small, leading to a breach and an uncontrolled release of impounded water can produce catastrophic downstream flooding. How a breach forms in an embankment dam depends on many factors: the embankment geometry, material composition, construction methods, type and degree of embankment crest and slope protective cover, reservoir dimensions, inflow to the reservoir during failure, and the manner of failure. However, most dam failure models portray the process with little regard for the causal agents underlying water motion over or through embankments and the resulting soil erosion. Instead, breach development is significantly simplified and is considered to proceed presupposed, usually growing in the shape of a trapezoid (Fig. 1) defined by its final dimensions: the breach height H_b, average width $B_{avg,}$ and side-slope ratio m (horizontal to vertical). Breaching begins when the reservoir water-surface elevation reaches the failure elevation Y_f. Such an empirical depiction of breach formation requires less input data than more intricate models that describe the physical processes of embankment erosion in detail (Froehlich 2008)

This study evaluated breach model parameters from 126 embankment dam failures using feed-forward multilayer layer neural networks to determine their expected values and prediction variances. All input and output variables were standardized using Z-score transformations $z_i = (x_i - \bar{x})/\sigma_x$, where z_i is the Z-score of the variable x_i whose mean is $\bar{x}$ and standard deviation σ_x. The predicted parameters are more accurate than those given by prevailing methods. However, profile traces changing one variable at a time to examine the effect on the predicted responses are necessary to ensure reliable solutions. This research contributes to improving dam safety and enhancing preparedness for potential dam breaches, thereby reducing risks associated with these events.



$$H1 = \tanh\left(-0.985 + 0.506 \times Mode + 2.68 \times Core + 0.520 \times L_a' - 0.949 \times W_{avg}' + 0.142 \times V_w' + 0.135 \times H_b'\right)$$

$$H2 = \tanh\left(-5.64 + 0.738 \times Mode + 10..2 \times Core - 6.24 \times L_a' + 0.195 \times W_{avg}' + 2.48 \times V_w' + 1.12 \times H_b'\right)$$

$$H3 = \tanh\left(-0.992 + 0.963 \times Mode - 0.344 \times Core - 1.26 \times L_a' - 0.867 \times W_{avg}' + 2.43 \times V_w' + 2.44 \times H_b'\right)$$

$$H4 = \tanh\left(-0.913 + 0.521 \times Mode + 0.460 \times Core - 0.792 \times L_a' - 0.237 \times W_{avg}' + 0.721 \times V_w' + 1.12 \times H_b'\right)$$

$$H5 = \tanh\left(-1.12 + 0.355 \times Mode - 4.00 \times Core - 0.671 \times L_a' - 1.04 \times W_{avg}' + 0.477 \times V_w' - 0.611 \times H_b'\right)$$

$$B_{avg}' = 0.393 + 3.46 \times H1 - 1.06 \times H2 - 1.10 \times H3 + 2.01 \times H4 - 2.45 \times H5$$

$$B_{avg} = 49.7 + 56.3 \times B_{avg}'$$

Failure data for gravity, buttress, arch, and various other dams

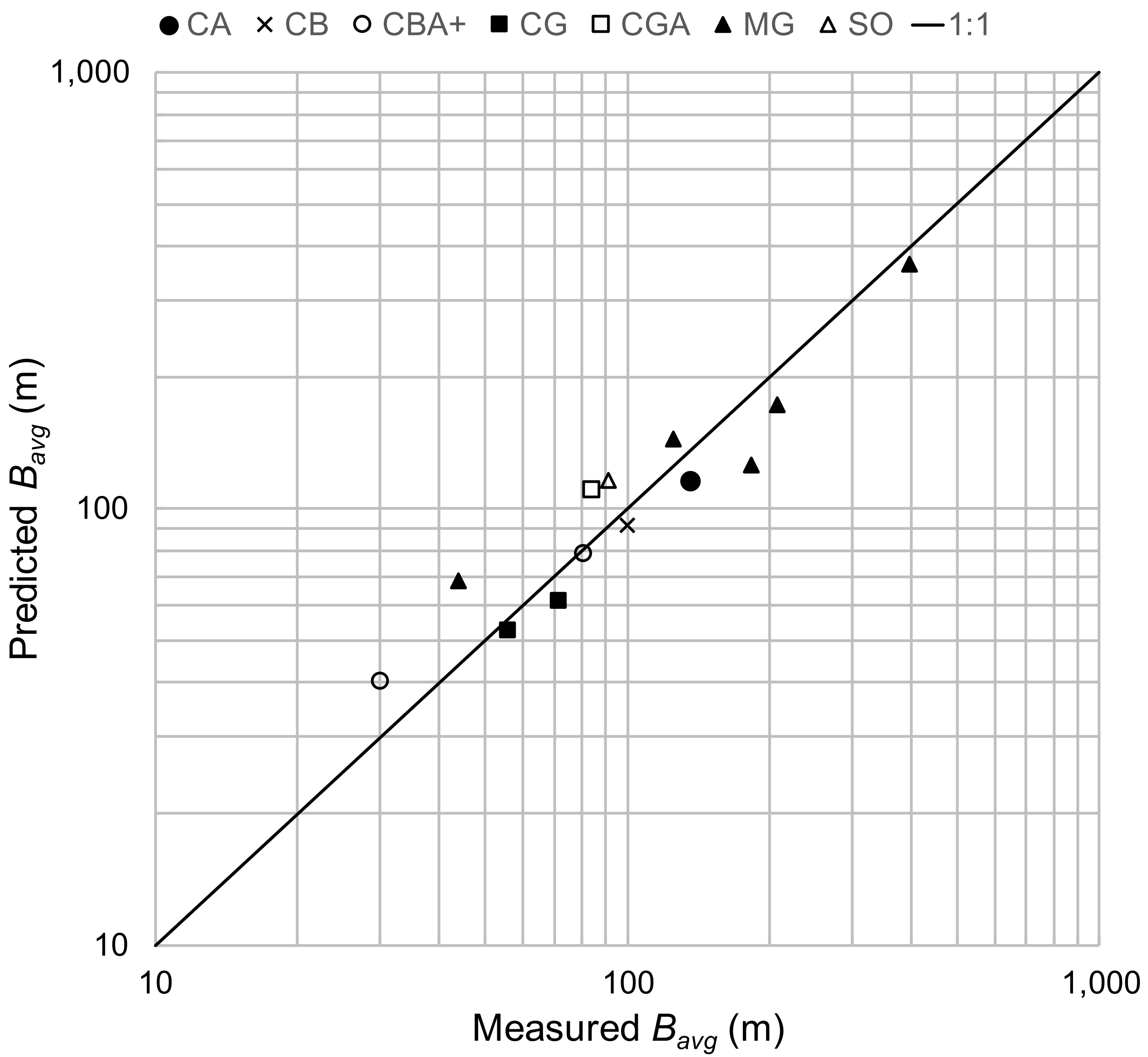
Dam name	Dam type[a]	Year completed	Year failed	V_w (Mm^3)	H_d (m)	H_b (m)	H_w (m)	L_a (m)	B_{avg} (m)
Malpasset, France	CA	1954	1959	55.0	66.5	44	43.5	190	136
Vega de Tera, Spain	CB	1958	1959	7.3	34	34	34	270	100
Gleno, Italy	CBA+	1923	1923	5.43	32.5	32.5	31.5	220	81
Rutte, Italy	CBA+	1952	1965	0.311	20	15	14	150	30
Bayless, USA	CG	1909	1911	1.05	15.9	15.9	14.9	170	56
St. Francis, Calif.	CG	1926	1928	47.1	57.3	57.3	56.5	210	84
Zerbino, Italy	CG	1924	1935	9.85	16.5	16.5	16.5	110	72
Tighra, India	MG	1917	1917	162	26.2	26.2	27.9	1520	396
Austin, USA	MG	1893	1900	102.7	20.1	20.1	23.5	400	208
Bouzey, France	MG	1881	1895	7.0	22.7	10.5	9.9	520	183
El Habra, Algeria	MG	1872	1881	30.0	35	35	36	450	125
Castlewood, USA	MG	1890	1933	6.17	21.3	21.3	21.6	160	44
Hauser, USA	SO	1908	1911	64.3	22.9	22.9	22.9	210	91

[a] CA= concrete arch, CB = concrete buttress, CBA+ = concrete buttress/multiple arch, CG = concrete gravity, MG = masonry gravity, SO = steel ossature.

$$B_{avg} = 0.4 \times V_w^{1/6} \times L_a^{1/2}$$

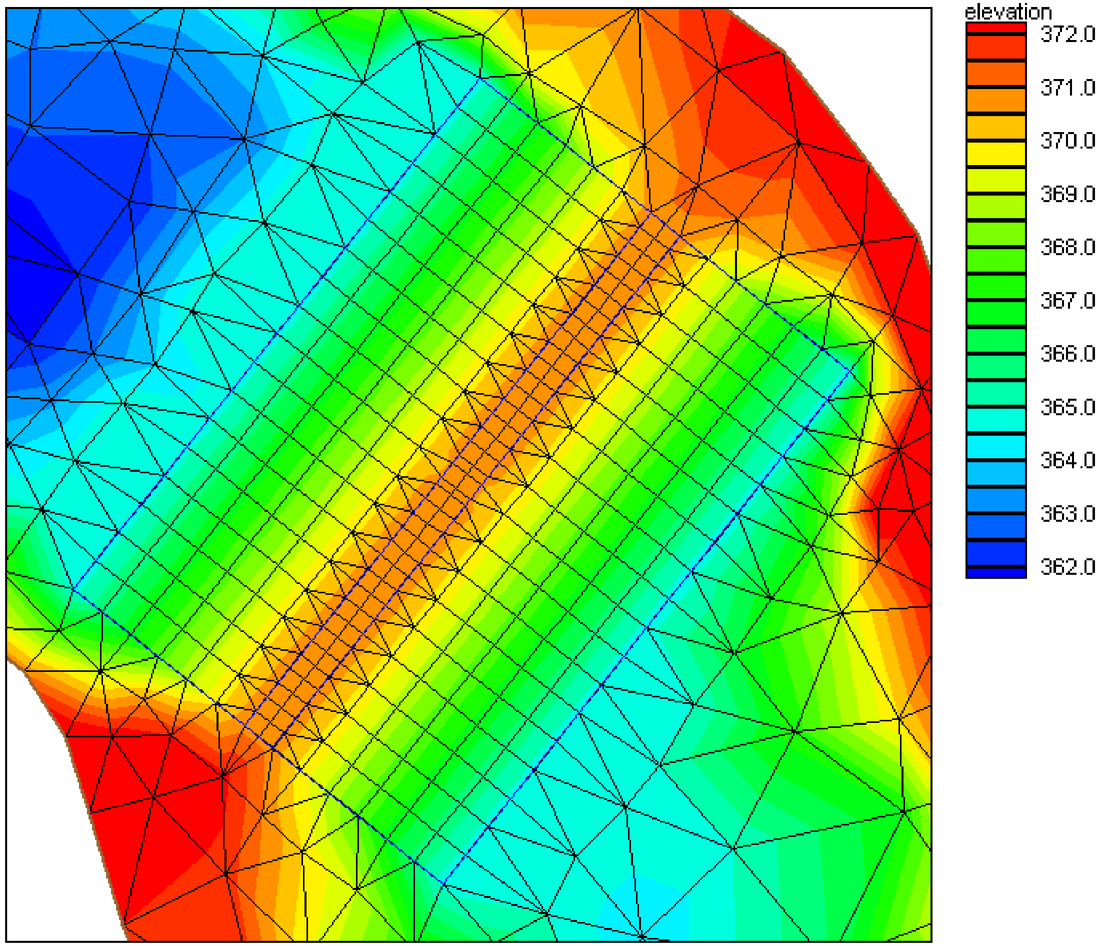
$$\mathrm{R}^2 = 0.88$$

$$s_{\ln B_{avg}} = 0.23$$



Field Test #1 Embankment

- Homogeneous (maximum cohesive) soils:
  - Clay = 25%
  - Silt = 60%
  - Sand = 15%
- Embankment:
  - Height = 6.0 m
  - Length = 36 m
  - Slope = 2:1 (horz:vert) upstream and down
- Crest:
  - Elevation = 370.81 m
  - Width = 2.0 m

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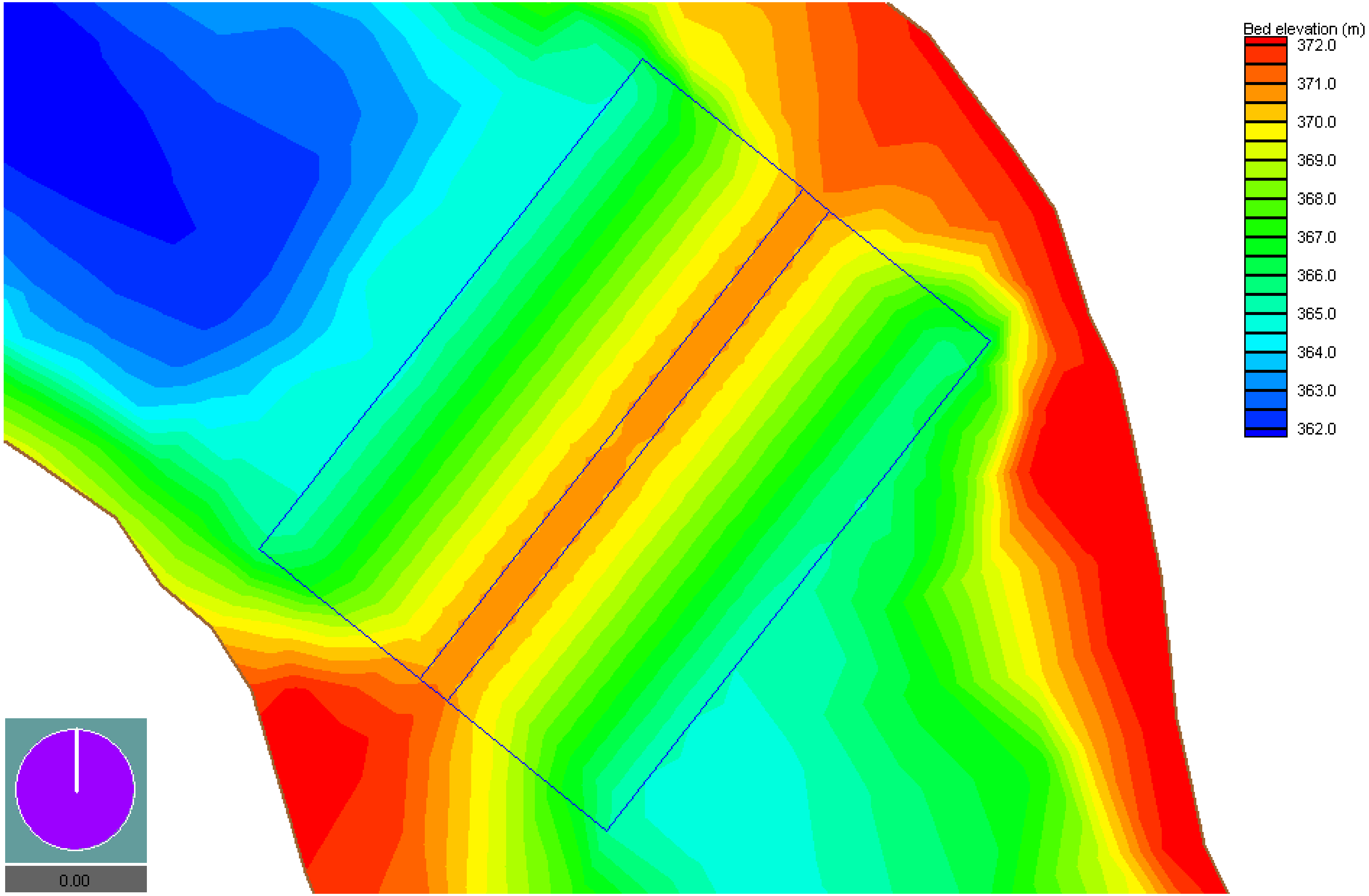
Bed elevation (m)
372.0
371.0
370.0
369.0
368.0
367.0
366.0
365.0
364.0
363.0
362.0
0.00

Field Test #1 Crest Profile