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The Uncertainty of Embankment Dam Breach Parameter Predictions Based on Dam Failure Case Studies

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Introduction

Risk assessment studies considering the failure of embankment dams often make use of breach parameter prediction methods that have been developed from analysis of historic dam failures. Similarly, predictions of peak breach outflow can also be made using relations developed from case study data. This paper presents an analysis of the uncertainty of many of these breach parameter and peak flow prediction methods, making use of a previously compiled database (Wahl 1998) of 108 dam failures. Subsets of this database were used to develop many of the relations examined.

The paper begins with a brief discussion of breach parameters and prediction methods. The uncertainty analysis of the various methods is next presented, and finally, a case study is offered to illustrate the application of several breach parameter prediction methods and the uncertainty analysis to a risk assessment recently performed by the Bureau of Reclamation for Jamestown Dam, on the James River in east-central North Dakota.

Breach Parameters

Dam break flood routing models (e.g., DAMBRK, FLDWAV) simulate the outflow from a reservoir and through the downstream valley resulting from a developing breach in a dam. These models focus their computational effort on the routing of the breach outflow hydrograph. The development of the breach is not simulated in any physical sense, but rather is idealized as a parametric process, defined by the shape of the breach, its final size, and the time required for its development (often called the failure time). Breaches in embankment dams are usually assumed to be trapezoidal, so the shape and size of the breach are defined by a base width and side slope angle, or more simply by an average breach width.

The failure time is a critical parameter affecting the outflow hydrograph and the consequences of dam failure, especially when populations at risk are located close to a dam so that available warning and evacuation time dramatically affects predictions of loss of life. For the purpose of routing a dam-break flood wave, breach development begins when a breach has reached the point at which the volume of the reservoir is compromised and failure becomes imminent. During the breach development phase, outflow from the dam increases rapidly. The breach development time ends when the breach reaches its final size; in some cases this may also correspond to the time of peak outflow through the breach, but for relatively small reservoirs the peak outflow may occur before the breach is fully developed. This breach development time as described above is the parameter predicted by most failure time prediction equations.

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The breach development time does not include the potentially long preceding period described as the breach initiation phase (Wahl 1998), which can also be important when considering available warning and evacuation time. This is the first phase of an overtopping failure, during which flow overtops a dam and may erode the downstream face, but does not create a breach through the dam that compromises the reservoir volume; if the overtopping flow were quickly stopped during the breach initiation phase, the reservoir would not fail. In an overtopping failure, the length of the breach initiation phase is important, because breach initiation can potentially be observed and may thus trigger warning and evacuation. Unfortunately, there are few tools available for predicting the length of the breach initiation phase.

During a seepage-erosion (piping) failure the delineation between breach initiation and breach development phases is less apparent. In some cases, seepage-erosion failures can take a great deal of time to develop. In contrast to the overtopping case, the loading that causes a seepage-erosion failure cannot normally be removed quickly, and the process does not take place in full view, except that the outflow from a developing pipe can be observed and measured. One useful way to view seepage-erosion failures is to consider three possible conditions:

(1) normal seepage outflow, with clear water and low flow rates;

(2) initiation of a seepage-erosion failure with cloudy seepage water that indicates a developing pipe, but flow rates are still low and not rapidly increasing. Corrective actions might still be possible that would heal the developing pipe and prevent failure.

(3) active development phase of a seepage-erosion failure in which erosion is dramatic and flow rates are rapidly increasing. Failure can no longer be prevented.

Only the length of the last phase is important when determining the breach hydrograph from a dam, but both the breach initiation and breach development phases may be important when considering warning and evacuation time. Again, as with the overtopping failure, there are few tools available for estimating the length of the breach initiation phase.

Predicting Breach Parameters

To carry out a dam break routing simulation, breach parameters must be estimated and provided as inputs to the dam-break and flood-routing simulation model. Several methods are available for estimating breach parameters; a summary of the available methods was provided by Wahl (1998). The simplest methods (Johnson and Illes 1976; Singh and Snorrason 1984; Reclamation 1988) predict the average breach width as a linear function of either the height of the dam or the depth of water stored behind the dam at the time of failure. Slightly more sophisticated methods predict more specific breach parameters, such as breach base width, side slope angles, and failure time, as functions of one or more dam and reservoir parameters, such as storage volume, depth of water at failure, depth of breach, etc. All of these methods are based on regression analyses of data collected from actual dam failures. The database of dam failures used to develop these relations is relatively lacking in data from failures of large dams, with about 75 percent of the cases having a height less than 15 meters, or 50 ft (Wahl 1998).

Physically-based simulation models are available to aid in the prediction of breach parameters. Although none are widely used, the most notable is the National Weather Service BREACH model (Fread 1988). These models simulate the hydraulic and erosion processes associated with flow over an overtopping dam or through a developing piping channel. Through such a simulation, an estimate of the breach parameters may be developed for use in a dam-break flood routing model, or the outflow hydrograph at the dam can be predicted directly. The primary weakness of the NWS-BREACH model and other similar models is the fact that they do not adequately model the headcut-type erosion processes that dominate the breaching of cohesive-soil embankments (e.g., Hahn et al. 2000). Recent work by the Agricultural Research Service (e.g., Temple and Moore 1994) on headcut erosion in earth spillways has shown that headcut erosion is best modeled with methods based on energy dissipation.

Predicting Peak Outflow

In addition to prediction of breach parameters, many investigators have proposed simplified methods for predicting peak outflow from a breached dam. These methods are valuable for reconnaissance-level work and for checking the reasonability of dam-break outflow hydrographs developed from estimated breach parameters. This paper considers the relations by:

- Kirkpatrick (1977)
- SCS (1981)
- Hagen (1982)
- Reclamation (1982)
- Singh and Snorrason (1984)
- MacDonald and Langridge-Monopolis (1984)
- Costa (1985)
- Evans (1986)
- Froehlich (1995a)
- Walder and O'Connor (1997)

All of these methods except Walder and O'Connor are straightforward regression relations that predict peak outflow as a function of various dam and/or reservoir parameters, with the relations developed from analyses of case study data from real dam failures. In contrast, Walder and O'Connor's method is based upon an analysis of numerical simulations of idealized cases spanning a range of dam and reservoir configurations and erosion scenarios. An important parameter in their method is an assumed vertical erosion rate of the breach; for reconnaissance-level estimating purposes they suggest that a range of reasonable values is 10 to 100 m/hr, based on analysis of case study data. The method makes a distinction between so-called large-reservoir/fast-erosion and small-reservoir/slow-erosion cases. In large-reservoir cases the peak outflow occurs when the breach reaches its maximum depth, before there has been any significant drawdown of the reservoir. The peak outflow in this case in insensitive to the erosion rate. In the small-reservoir case there is significant drawdown of the reservoir as the breach develops, and thus the peak outflow occurs before the breach erodes to its maximum depth. Peak outflows for small-reservoir cases are dependent on the vertical erosion rate and can be dramatically smaller than for large-reservoir cases. The determination of whether a specific situation is a large-reservoir or small-reservoir case is based on a dimensionless parameter incorporating the embankment erosion rate, reservoir size, and change in reservoir level during the failure. Thus, so-called large-reservoir/fast-erosion cases can occur even with what might be considered "small" reservoirs and vice versa. This refinement is not present in any of the other peak flow prediction methods.

Developing Uncertainty Estimates

In a typical risk assessment study, a variety of loading and failure scenarios are analyzed. This allows the study to incorporate variability in antecedent conditions and the probabilities associated with different loading conditions and failure scenarios. The uncertainty of key parameters (e.g., material properties) is sometimes considered by creating scenarios in which analyses are carried out with different parameter values and a probability of occurrence assigned to each value of the parameter. Although the uncertainty of breach parameter predictions is often very large, there have previously been no quantitative assessments of this uncertainty, and thus breach parameter uncertainty has not been incorporated into most risk assessment studies. In some studies, variations in thresholds of failure (e.g., overtopping depth to initiate breach) have been incorporated, usually through a voting process in which study team members and technical experts use engineering judgment to assign probabilities to different failure thresholds.

It is worthwhile to consider breach parameter prediction uncertainty in the risk assessment process because the uncertainty of breach parameter predictions is likely to be significantly greater than all other factors, and could thus dramatically influence the outcome. For example Wahl (1998) used many of the available relations to predict breach parameters for 108 documented case studies and plot the predictions against the observed values. Prediction errors of ±75% were not uncommon for breach width, and prediction errors for failure time often exceeded 1 order of magnitude. Most relations used to predict failure time are conservatively designed to underpredict the reported time more often than they overpredict, but overprediction errors of more than one-half order of magnitude did occur several times.

The first question that must be addressed in an uncertainty analysis of breach parameter predictions is how to express the results. The case study datasets used to develop most breach parameter prediction equations include data from a wide range of dam sizes, and thus, regressions in log-log space have been commonly used. Figure 1 shows the observed and predicted breach widths as computed by Wahl (1998) in both arithmetically-scaled and log-log plots. In the arithmetic plots, it would be difficult to draw in upper and lower bound lines to define an uncertainty band. In the log-log plots data are scattered approximately evenly above and below the lines of perfect prediction, suggesting that uncertainties would best be expressed as a number of log cycles on either side of the predicted value. This is the approach taken in the analysis that follows.

The other notable feature of the plots in Figure 1 is the presence of a few significant outliers. The source of these outliers is believed to be the variable quality of the case study observations, the potential for misapplication of some of the prediction equations due to lack of detailed knowledge of each case study, and inherent variability in the data due to the variety of factors that influence dam breach mechanics. Thus, before determining uncertainties, an outlier-exclusion algorithm was applied (Rousseeuw 1998). The algorithm has the advantage that it is, itself, insensitive to the effects of outliers.

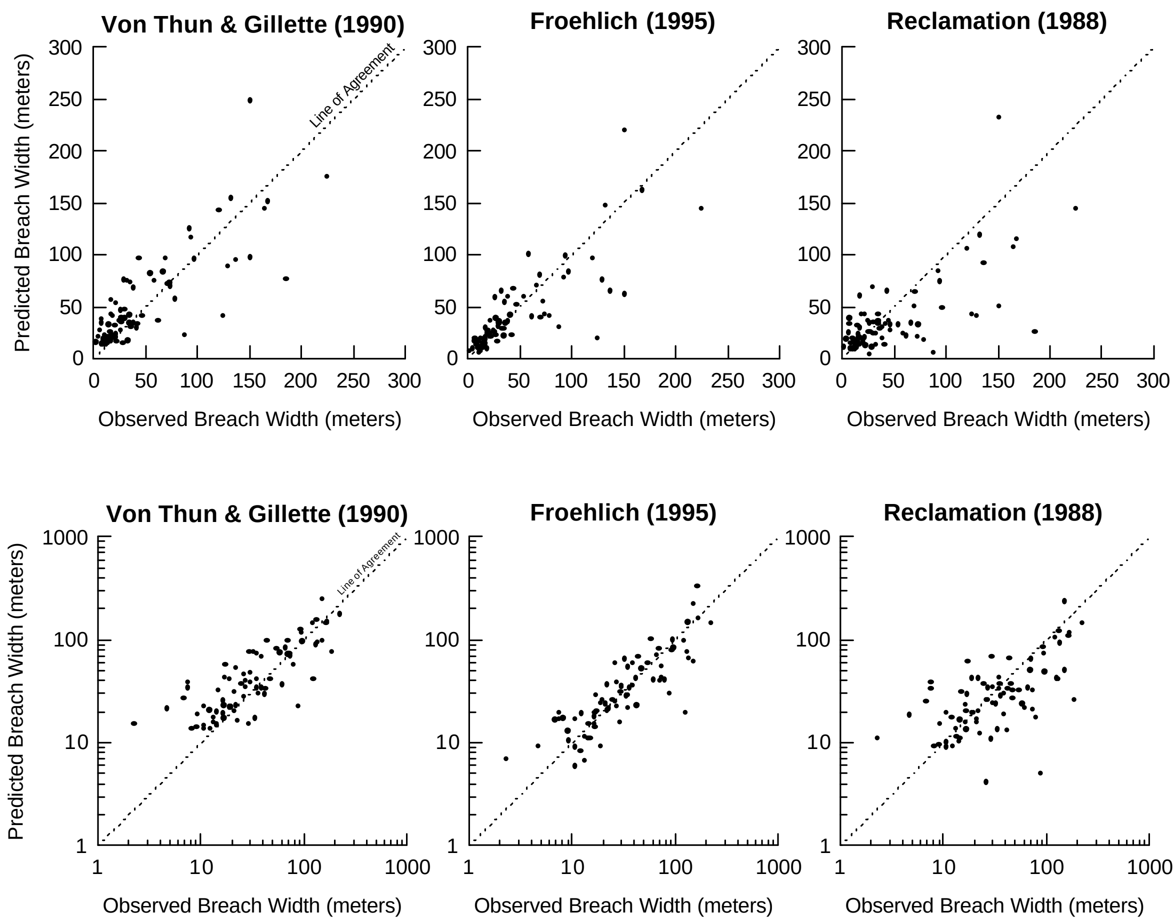

Figure 1. — Predicted and observed breach widths (Wahl 1998), plotted arithmetically (top) and on log-log scales (bottom).

The uncertainty analysis was performed using the database presented in Wahl (1998), with data on 108 case studies of actual embankment dam failures, collected from numerous sources in the literature. The majority of the available breach parameter and peak flow prediction equations were applied to this database of dam failures, and the predicted values were compared to the observed values. Computation of breach parameters or peak flows was straightforward in most cases. A notable exception was the peak flow prediction method of Walder and O'Connor (1997), which requires that the reservoir be classified as a large- or small-reservoir case. In addition, in the case of the small-reservoir situation, an average vertical erosion rate of the breach must be estimated. The Walder and O'Connor method was applied only to those dams that could be clearly identified as large-reservoir (in which case peak outflow is insensitive to the vertical erosion rate) or small-reservoir with an associated estimate of the vertical erosion rate obtained from observed breach heights and failure times. Two other facts should be noted:

- No prediction equation could be applied to all 108 dam failure cases, due to lack of required input data for the specific equation or the lack of an observed value of the parameter of interest. Most of the breach width equations could be tested against about 70 to 80 cases, the failure time equations were tested against 30 to 40 cases, and the peak flow prediction equations were generally tested against about 30 to 40 cases.

- The testing made use of the same data used to originally develop the equations, but each equation was also tested against additional cases. This should provide a fair indication of the ability of each equation to predict breach parameters for future dam failures.

A step-by-step description of the uncertainty analysis method follows:

(1) Plot predicted vs. observed values on log-log scales.

(2) Compute individual prediction errors in terms of the number of log cycles separating the predicted and observed value, $e_i = \log(\hat{x}) - \log(x) = \log(\hat{x}/x)$, where e_i is the prediction error, $\hat{x}$ is the predicted value and x is the observed value.

(3) Apply the outlier-exclusion algorithm to the series of prediction errors computed in step (2). The algorithm is described by Rousseeuw (1998).

(a) Determine T, the median of the e_i values. T is the estimator of location.

(b) Compute the absolute values of the deviations from the median, and determine the median of these absolute deviations (MAD).

(c) Compute an estimator of scale, S=1.483*(MAD). The 1.483 factor makes S comparable to the standard deviation, which is the usual scale parameter of a normal distribution.

(d) Use S and T to compute a Z-score for each observation, $Z_i=(e_i-T)/S$, where the e_i's are the observed prediction errors, expressed as a number of log cycles.

(e) Reject any observations for which $|Z_i|>2.5$

This method rejects at the 98.7% probability level if the samples are from a perfect normal distribution.

(4) Compute the mean, $\bar{e}$, and the standard deviation, S_e, of the remaining prediction errors. If the mean value is negative, it indicates that the prediction equation underestimated the observed values, and if positive the equation overestimated the observed values. Significant over or underestimation should be expected, since many of the breach parameter prediction equations are intended to be conservative or provide envelope estimates, e.g., maximum reasonable breach width, fastest possible failure time, etc.

(5) Using the values of $\bar{e}$ and S_e, one can express a confidence band around the predicted value of a parameter as $\{\hat{x}\cdot 10^{-\bar{e}-2S_e}, \hat{x}\cdot 10^{-\bar{e}+2S_e}\}$, where $\hat{x}$ is the predicted value. The use of $\pm 2S_e$ gives approximately a 95 percent confidence band.

Table 1 summarizes the results. The first column identifies the particular method being analyzed, the next two columns show the number of case studies used to test the method, and the next two columns give the prediction error and the width of the uncertainty band. The rightmost column shows the range of the prediction interval around a hypothetical predicted value of 1.0. The values in this column can be used as multipliers to obtain the prediction interval for a specific case.

Table 1. – Uncertainty estimates of breach parameter and peak flow prediction equations. All equations use metric units (meters, m^3, m^3/s). Failure times are computed in hours.

Equation	Number of Case Studies: Before outlier exclusion	Number of Case Studies: After outlier exclusion	Mean Prediction Error, $\bar{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
BREACH WIDTH EQUATIONS					
USBR (1988) $\overline{B} = 3(h_w)$	80	70	-0.09	±0.43	0.45 — 3.3
MacDonald and Langridge-Monopolis (1984) $V_{er} = 0.0261(V_w \cdot h_w)^{0.769}$ *earthfill* $V_{er} = 0.00348(V_w \cdot h_w)^{0.852}$ *non-earthfill (e.g., rockfill)*	60	58	-0.01	±0.82	0.15 — 6.8
Von Thun and Gillette (1990) $\overline{B} = 2.5h_w + C_b$ *where C_b is a function of reservoir size*	78	70	+0.09	±0.35	0.37 — 1.8
Froehlich (1995b) $\overline{B} = 0.1803K_o V_w^{0.32} h_b^{0.19}$ *where K_o = 1.4 for overtopping, 1.0 for piping*	77	75	+0.01	±0.39	0.40 — 2.4
FAILURE TIME EQUATIONS					
MacDonald and Langridge-Monopolis (1984) $t_f = 0.0179(V_{er})^{0.364}$	37	35	-0.21	±0.83	0.24 — 11.
Von Thun and Gillette (1990) $t_f = 0.015(h_w)$ *highly erodible* $t_f = 0.020(h_w) + 0.25$ *erosion resistant*	36	34	-0.64	±0.95	0.49 — 40.
Von Thun and Gillette (1990) $t_f = \overline{B}/(4h_w + 61)$ *highly erodible* $t_f = \overline{B}/(4h_w)$ *erosion resistant*	36	35	-0.38	±0.84	0.35 — 17.
Froehlich (1995b) $t_f = 0.00254(V_w)^{0.53} h_b^{-0.9}$	34	33	-0.22	±0.64	0.38 — 7.3
USBR (1988) $t_f = 0.011(\overline{B})$	40	39	-0.40	±1.02	0.24 — 27.
PEAK FLOW EQUATIONS					
Kirkpatrick (1977) $Q_p = 1.268(h_w + 0.3)^{2.5}$	38	34	-0.14	±0.69	0.28 — 6.8
SCS (1981) $Q_p = 16.6(h_w)^{1.85}$	38	32	+0.13	±0.50	0.23 — 2.4
Hagen (1982) $Q_p = 0.54(S \cdot h_d)^{0.5}$	31	30	+0.43	±0.75	0.07 — 2.1
Reclamation (1982) $Q_p = 19.1(h_w)^{1.85}$ *envelope equation*	38	32	+0.19	±0.50	0.20 — 2.1

Equation	Number of Case Studies: Before outlier exclusion	Number of Case Studies: After outlier exclusion	Mean Prediction Error, $\bar{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
PEAK FLOW EQUATIONS *(continued)*					
Singh and Snorrason (1984)					
$Q_p = 13.4(h_d)^{1.89}$	38	28	+0.19	±0.46	0.23 — 1.9
$Q_p = 1.776(S)^{0.47}$	35	34	+0.17	±0.90	0.08 — 5.4
MacDonald and Langridge-Monopolis (1984)					
$Q_p = 1.154(V_w \cdot h_w)^{0.412}$	37	36	+0.13	±0.70	0.15 — 3.7
$Q_p = 3.85(V_w \cdot h_w)^{0.411}$ *envelope equation*	37	36	+0.64	±0.70	0.05 — 1.1
Costa (1985)					
$Q_p = 1.122(S)^{0.57}$ *envelope equation*	35	35	+0.69	±1.02	0.02 — 2.1
$Q_p = 0.981(S \cdot h_d)^{0.42}$	31	30	+0.05	±0.72	0.17 — 4.7
$Q_p = 2.634(S \cdot h_d)^{0.44}$ *envelope equation*	31	30	+0.64	±0.72	0.04 — 1.22
Evans (1986)					
$Q_p = 0.72(V_w)^{0.53}$	39	39	+0.29	±0.93	0.06 — 4.4
Froehlich (1995a)					
$Q_p = 0.607(V_w^{0.295} h_w^{1.24})$	32	31	-0.04	±0.32	0.53 — 2.3
Walder and O'Connor (1997)					
Q_p estimated using method based on relative erodibility of dam and size of reservoir	22	21	+0.13	±0.68	0.16 — 3.6

Notes: Where multiple equations are shown for application to different types of dams (e.g., highly erodible vs. erosion resistant), a single prediction uncertainty was analyzed, with the *system* of equations viewed as a single algorithm. The only exception is the pair of peak flow prediction equations offered by Singh and Snorrason (1984), which are alternative and independent methods for predicting peak outflow.

Definitions of Symbols for Equations Shown in Column 1.

$\bar{B}$ = average breach width, meters

C_b = offset factor in the Von Thun and Gillette breach width equation, varies from 6.1 m to 54.9 m as a function of reservoir storage

h_b = height of breach, m

h_d = height of dam, m

h_w = depth of water above breach invert at time of failure, meters

K_o = overtopping multiplier for Froehlich breach width equation, 1.4 for overtopping, 1.0 for piping

Q_p = peak breach outflow, m^3/s

S = reservoir storage, m^3

t_f = failure time, hours

V_{er} = volume of embankment material eroded, m^3

V_w = volume of water stored above breach invert at time of failure, m^3

Summary of Uncertainty Analysis Results

The four methods for predicting breach width all had absolute mean prediction errors less than one-tenth of an order of magnitude, indicating that on average their predictions are on-target. The uncertainty bands were similar (±0.3 to ±0.4 log cycles) for all of the equations except the MacDonald and Langridge-Monopolis equation, which had an uncertainty of ±0.82 log cycles.

The five methods for predicting failure time all underpredict the failure time on average, by amounts ranging from about one-fifth to two-thirds of an order of magnitude. This is consistent with the previous observation that these equations are designed to conservatively predict fast breaches, which will cause large peak outflows. The uncertainty bands on all of the failure time equations are very large, ranging from about ±0.6 to ±1 order of magnitude, with the Froehlich (1995b) equation having the smallest uncertainty.

Most of the peak flow prediction equations tend to overpredict observed peak flows, with most of the "envelope" equations overpredicting by about two-thirds to three-quarters of an order of magnitude. The uncertainty bands on the peak flow prediction equations are about ±0.5 to ±1 order of magnitude, except the Froehlich (1995a) relation which has an uncertainty of ±0.32 orders of magnitude. In fact, the Froehlich equation has both the best prediction error and uncertainty of all the peak flow prediction equations.

Application to Jamestown Dam

To illustrate the application of the uncertainty analysis results, a case study is presented. In January 2001 the Bureau of Reclamation conducted a risk assessment study for Jamestown Dam (Figure 2), a feature of the Pick-Sloan Missouri Basin Program, located on the James River immediately upstream from Jamestown, North Dakota. For this risk assessment, two potential static failure modes were considered:

- Seepage erosion and piping of foundation materials
- Seepage erosion and piping of embankment materials

No distinction between these two failure modes was made in the breach parameter analysis, since most methods used to predict breach parameters lack the refinement needed to consider the differences in breach morphology for these two failure modes.

Figure 2. — Jamestown Dam and reservoir.

The potential for failure and the downstream consequences from failure increase significantly at higher reservoir levels, although the likelihood of occurrence of high reservoir levels is low. The reservoir rarely exceeds its top-of-joint-use elevation, and has never exceeded elevation 1445.9 ft. Four potential reservoir water surface elevations at failure were considered in the study:

- Top of joint use, elev. 1432.67 ft, reservoir capacity of about 37,000 ac-ft
- Elev. 1440.0 ft, reservoir capacity of about 85,000 ac-ft
- Top of flood space, elev. 1454 ft, reservoir capacity of about 221,000 ac-ft
- Maximum design water surface, elev. 1464.3 ft, storage of about 380,000 ac-ft

Breach parameters were predicted using most of the methods discussed earlier in this paper, and also by modeling with the National Weather Service BREACH model (NWS-BREACH).

Dam Description

Jamestown Dam is located on the James River about 1.5 miles upstream from the city of Jamestown, North Dakota. It was constructed by the Bureau of Reclamation from 1952 to 1954. The facilities are operated by Reclamation to provide flood control, municipal water supply, fish and wildlife benefits and recreation.

The dam is a zoned-earthfill structure with a structural height of 111 ft and a height of 81 ft above the original streambed. The crest length is 1,418 ft at elevation 1471 ft and the crest width is 30 ft. The design includes a central compacted zone 1 impervious material, and upstream and downstream zone 2 of sand and gravel, shown in Figure 3. The upstream slope is protected with riprap and bedding above elevation 1430 ft. A toe drain consisting of sewer pipe laid with open joints is located in the downstream zone 2 along most of the embankment.

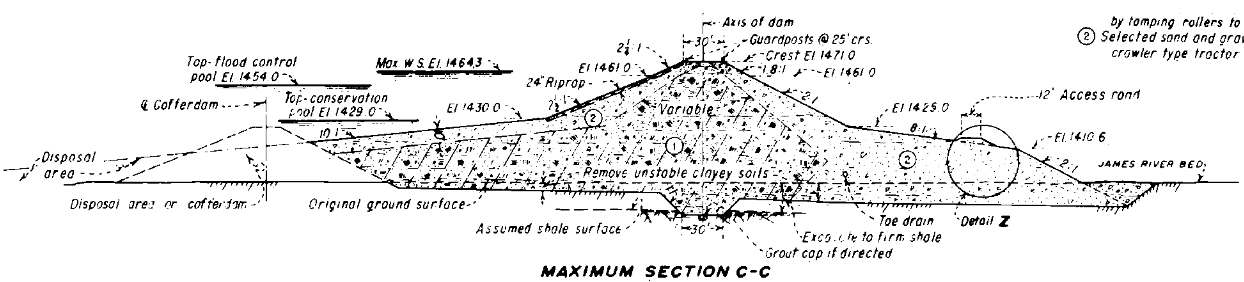

Figure 3. — Cross-section through Jamestown Dam.

The abutments are composed of Pierre Shale capped with glacial till. The main portion of the dam is founded on a thick section of alluvial deposits. The spillway and outlet works are founded on Pierre Shale. Beneath the dam a cutoff trench was excavated to the shale on both abutments, however, between the abutments, foundation excavation extended to a maximum depth of 25 ft, and did not provide a positive cutoff of the thick alluvium. The alluvium beneath the dam is more than 120 ft thick in the channel area.

There is a toe drain within the downstream embankment near the foundation level, and a fairly wide embankment section to help control seepage beneath the dam, since a positive cutoff was not constructed. The original design recognized that additional work might be required to control seepage and uplift pressures, depending on performance of the dam during first filling. In general, performance of the dam has been adequate, but, reservoir water surface elevations have never exceeded 1445.9 ft, well below the spillway crest. Based on observations of increasing pressures in the foundation during high reservoir elevations and significant boil activity downstream from the dam, eight relief wells were installed along the downstream toe in 1995 and 1996. To increase the seepage protection, a filter blanket was constructed in low areas downstream from the dam in 1998.

Results — Breach Parameter Estimates

Breach parameter predictions were computed for the four reservoir conditions listed previously: top of joint use; elev. 1440.0; top of flood space; and maximum design water surface elevation. Predictions were made for average breach width, volume of eroded material, and failure time. Side slope angles were not predicted because equations for predicting breach side slope angles are rare in the literature; Froehlich (1987) offered an equation, but in his later paper (1995b) he suggested simply assuming side slopes of 0.9:1 (horizontal:vertical) for piping failures. Von Thun and Gillette (1990) suggested using side slopes of 1:1, except for cases of dams with very thick zones of cohesive materials where side slopes of 0.5:1 or 0.33:1 might be more appropriate.

After computing breach parameters using the several available equations, the results were reviewed and engineering judgment applied to develop a single predicted value and an uncertainty band to be provided to the risk assessment study team. These recommended values are shown at the bottom of each column in the tables that follow.

Breach Width

Predictions of average breach width are summarized in Table 2. The table also lists the predictions of the volume of eroded embankment material made using the MacDonald and Langridge-Monopolis equation, and the corresponding estimate of average breach width.

Table 2. — Predictions of average breach width for Jamestown Dam.

BREACH WIDTHS ***B*****, feet**	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
Reclamation, 1988	128	58 — 422	150	68 — 495	192	86 — 634	223	100 — 736
Von Thun and Gillette, 1990	287	106 — 516	305	113 — 549	340	126 — 612	366	135 — 659
Froehlich, 1995b	307	123 — 737	401	160 — 962	544	218 — 1307	648	259 — 1554^
MacDonald and Langridge-Monopolis, 1984 (Volume of erosion, yd^3)	*191,000*	*29,000 — 1,296,000*	*408,000*	*61,000 — 2,775,000*	*1,029,000*	*154,000 — 6,995,000*	*1,751,000*	*263,000 — 11,904,000*
(Equivalent breach width, ft)	281	42 — 1,908^	601	90 — 4,090^	1,515^	227 — 10,300^	2,578^	387 — 17,528^
Recommended values	290	110 — 600	400	150 — 1000	540	200 — 1300	650	250 — 1418

* Recommend breach side slopes for all scenarios are 0.9 horizontal to 1.0 vertical.
^ Exceeds actual embankment length.

The uncertainty analysis described earlier showed that the Reclamation equation tends to underestimate the observed breach width, so it is not surprising that it yielded the smallest values. The Von Thun and Gillette equation and the Froehlich equation produced comparable results for the top-of-joint-use scenario, in which reservoir storage is relatively small. For the two scenarios with greater reservoir storage, the Froehlich equation predicts significantly larger breach widths. This is not surprising, since the Froehlich equation relates breach width to an exponential function of both the reservoir storage and reservoir depth. The Von Thun and Gillette equation accounts for reservoir storage only through the C_b offset parameter, but C_b is a constant for all reservoirs larger than 10,000 ac-ft, as was the case for all four of these scenarios.

Using the MacDonald and Langridge-Monopolis equation, the estimate of eroded embankment volume and associated breach width for the top-of-joint-use scenario is also comparable to the other equations. However, for the two large-volume scenarios, the predictions are much larger than any of the other equations, and in fact are unreasonable because they exceed the dimensions of the dam (1,418 ft long; volume of 763,000 yd^3).

The prediction intervals developed through the uncertainty analysis are sobering, as the ranges vary from small notches through the dam to complete washout of the embankment. Even for the top-of-joint-use case, the upper bound for the Froehlich and Von Thun/Gillette equations is equivalent to about half the length of the embankment.

Failure Time

Failure time predictions are summarized in Table 3. All of the equations indicate increasing failure times as the reservoir storage increases, except the second Von Thun and Gillette relation, which predicts a slight decrease in failure time for the large-storage scenarios. For both Von Thun and Gillette relations, the dam was assumed to be in the erosion resistant category.

Table 3. — Failure time predictions for Jamestown Dam.

FAILURE TIMES **t_f, hours**	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
MacDonald and Langridge-Monopolis, 1984	1.36	0.33 — 14.9	1.79	0.43 — 19.7	2.45*	0.59 — 26.9	2.45*	0.59 — 26.9
Von Thun and Gillette, 1990 $t_f = f(h_w)$ ...erosion resistant	0.51	0.25 — 20.4	0.55	0.27 — 22.2	0.64	0.31 — 25.6	0.70	0.34 — 28.1
Von Thun and Gillette, 1990 $t_f = f(B, h_w)$ ...erosion resistant	1.68	0.59 — 28.6	1.53	0.53 — 25.9	1.33	0.47 — 22.6	1.23	0.43 — 20.9
Froehlich, 1995b	1.63	0.62 — 11.9	2.53	0.96 — 18.4	4.19	1.59 — 30.6	5.59	2.12 — 40.8
Reclamation, 1988	0.43	0.10 — 11.6	0.50	0.12 — 13.6	0.64	0.15 — 17.4	0.75	0.18 — 20.2
Recommended values	1.5	0.25 — 12	1.75	0.25 — 14	3.0	0.3 — 17	4.0	0.33 — 20

* The MacDonald and Langridge-Monopolis equation is based on the prediction of eroded volume, shown previously in Table 2. Because the predicted volumes exceeded the total embankment volume in the two large-storage scenarios, the total embankment volume was used in the failure time equation. Thus, the results are identical to the top-of-joint-use case.

The predicted failure times exhibit wide variation, and the recommended values shown at the bottom of the table are based on much judgment. The uncertainty analysis showed that all of the failure time equations tend to conservatively underestimate actual failure times, especially the Von Thun and Gillette and Reclamation equations. Thus, the recommended values are generally a compromise between the results obtained from the MacDonald and Langridge-Monopolis and Froehlich relations. Despite this fact, some very fast failures are documented in the literature, and this possibility is reflected in the prediction intervals determined from the uncertainty analysis.

Results — Peak Outflow Estimates

Peak outflow estimates are shown in Table 4, sorted in order of increasing peak outflow for the top-of-joint-use scenario. The lowest peak flow predictions come from those equations that are based solely on dam height or depth of water in the reservoir. The highest peak flows are predicted by those equations that incorporate a significant dependence on reservoir storage. Some of the predicted peak flows and the upper bounds of the prediction limits would be the largest dam-break outflows ever recorded, exceeding the 2.3 million ft^3/s peak outflow from the Teton Dam failure. (Storage in Teton Dam was 289,000 ac-ft at failure). The length of Jamestown Reservoir (about 30 miles) may help to attenuate some of the large peak outflows predicted by the storage-sensitive equations, since there will be an appreciable routing effect in the reservoir itself that is probably not accounted for in the peak flow prediction equations.

Table 4. — Predictions of peak breach outflow for Jamestown Dam.

PEAK OUTFLOWS Q_p, ft^3/s	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
Kirkpatrick, 1977	28,900	8,100 — 196,600	42,600	11,900 — 289,900	78,200	21,900 — 531,700	112,900	31,600 — 768,000
SCS, 1981	67,500	15,500 — 162,000	90,500	20,800 — 217,200	142,900	32,900 — 342,900	188,300	43,300 — 451,900
Reclamation, 1982, envelope	77,700	15,500 — 163,100	104,100	20,800 — 218,600	164,400	32,900 — 345,200	216,600	43,300 — 455,000
Froehlich, 1995a	**93,800**	**49,700 — 215,700**	**145,900**	**77,300 — 335,600**	**262,700**	**139,200 — 604,200**	**370,900**	**196,600 — 853,100**
MacDonald and Langridge-Monopolis, 1984	167,800	25,200 — 620,900	252,400	37,900 — 933,700	414,100	62,100 — 1,532,000	550,600	82,600 — 2,037,000
Singh/Snorrason, 1984 $Q_p = f(h_d)$	202,700	46,600 — 385,200	202,700	46,600 — 385,200	202,700	46,600 — 385,200	202,700	46,600 — 385,200
Walder and O'Connor, 1997	**211,700**	**33,900 — 755,600**	**279,300**	**44,700 — 997,200**	**430,200**	**68,800 — 1,536,000**	**558,600**	**89,400 — 1,994,000**
Costa, 1985 $Q_p = f(S*h_d)$	219,500	37,300 — 1,032,000	311,200	52,900 — 1,463,000	464,900	79,000 — 2,185,000	583,800	99,200 — 2,744,000
Singh/Snorrason, 1984 $Q_p = f(S)$	249,600	20,000 — 1,348,000	369,000	29,500 — 1,993,000	578,200	46,300 — 3,122,000	746,000	59,700 — 4,028,000
Evans, 1986	291,600	17,500 — 1,283,000	453,100	27,200 — 1,994,000	751,800	45,100 — 3,308,000	1,002,000	60,100 — 4,409,000
MacDonald and Langridge-Monopolis, 1984 (envelope equation)	548,700	27,400 — 603,500	824,300	41,200 — 906,700	1,351,000	67,600 — 1,486,000	1,795,000	89,800 — 1,975,000
Hagen, 1982	640,100	44,800 — 1,344,000	970,000	67,900 — 2,038,000	1,564,000	109,500 — 3,285,000	2,051,000	143,600 — 4,308,000
Costa, 1985 $Q_p = f(S*h_d)$ (envelope)	894,100	35,800 — 1,091,000	1,289,000	51,600 — 1,573,000	1,963,000	78,500 — 2,395,000	2,492,000	99,700 — 3,040,000
Costa, 1985 $Q_p = f(S)$	920,000	18,400 — 1,932,000	1,478,000	29,600 — 3,104,000	2,548,000	51,000 — 5,351,000	3,470,000	69,400 — 7,288,000

The equation offered by Froehlich (1995a) clearly had the best prediction performance in the uncertainty analysis, and is thus highlighted in the table. This equation had the smallest mean prediction error and narrowest prediction interval by a significant margin.

The results for the Walder and O'Connor method are also highlighted. As discussed earlier, this is the only method that considers the differences between the so-called large-reservoir/fast-erosion and small-reservoir/slow-erosion cases. Jamestown Dam proves to be a large-reservoir/fast-erosion case when analyzed by this method (regardless of the assumed vertical erosion rate of the breach—within reasonable limits), so the peak outflow will occur when the breach reaches its maximum size, before significant drawdown of the reservoir has occurred. Despite the refinement of considering large- vs. small-reservoir behavior, the Walder and O'Connor method was found to have uncertainty similar to most of the other peak flow prediction methods (about ±0.75 log cycles). However, amongst the 22 case studies that the method could be applied to, only four proved to be large-reservoir/fast-erosion cases. Of these, the method overpredicted the peak outflow in three cases, and dramatically underpredicted in one case (Goose Creek Dam, South Carolina, failed 1916 by overtopping). Closer examination showed some contradictions in the data reported in the literature for this case. On balance, it appears that the Walder and O'Connor method may provide reasonable estimates of the upper limit on peak outflow for large-reservoir/fast-erosion cases.

For the Jamestown Dam case, results from the Froehlich method can be considered the best estimate of peak breach outflow, and the results from the Walder and O'Connor method provide an upper bound estimate.

NWS-BREACH Simulations

Several simulations runs were made using the National Weather Service BREACH model (Fread 1988). The model requires input data related to reservoir bathymetry, dam geometry, the tailwater channel, embankment materials, and initial conditions for the simulated piping failure. Detailed information on embankment material properties was not available at the time that the simulations were run, so material properties were assumed to be similar to those of Teton Dam. A Teton Dam input data file is distributed with the model.

The results of the simulations are very sensitive to the elevation at which the piping failure is assumed to develop. In all cases analyzed, the maximum outflow occurred just prior to the crest of the dam collapsing into the pipe; after the collapse of the crest, a large volume of material partially blocks the pipe and the outflow becomes weir-controlled until the material can be removed. Thus, the largest peak outflows and largest breach sizes are obtained if the failure is initiated at the base of the dam, assumed to be elev. 1390.0 ft. This produces the maximum amount of head on the developing pipe, and allows it to grow to the largest possible size before the collapse occurs. Table 5 shows summary results of the simulations. For each of the four initial reservoir elevations a simulation was run with the pipe initiating at elev. 1390.0 ft, and a second simulation was run with the pipe initiating about midway up the height of the dam.

Table 5. — Results of NWS-BREACH simulations of seepage-erosion failures of Jamestown Dam.

	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
Initial elev. of piping failure, ft →	1390.0	1411.0	1390.0	1415.0	1390.0	1420.0	1390.0	1430.0
Peak outflow, ft^3/s	80,400	16,400	131,800	24,050	242,100	52,400	284,200	54,100
t_p, Time-to-peak outflow, hrs (from first significant increased flow through the breach)	3.9	2.1	4.0	1.8	4.0	1.4	3.6	1.1
Breach width at t_p, ft	51.6	21.4	63.2	24.8	81.0	33.7	81.0	34.2

There is obviously wide variation in the results depending on the assumed initial conditions for the elevation of the seepage failure. The peak outflows and breach widths tend toward the low end of the range of predictions made using the regression equations based on case study data. The predicted failure times are within the range of the previous predictions, and significantly longer than the very short (0.5 to 0.75 hr) failure times predicted by the Reclamation (1988) equation and the first Von Thun and Gillette equation.

Refinement of the material properties and other input data provided to the NWS-BREACH model might significantly change these results.

Conclusions

This paper has presented a quantitative analysis of the uncertainty of various regression-based methods for predicting embankment dam breach parameters and peak breach outflows. The uncertainties of predictions of breach width, failure time, and peak outflow are large for all methods, and thus it may be worthwhile to incorporate uncertainty analysis results into future risk assessment studies when predicting breach parameters using these methods. Predictions of breach width generally have an uncertainty of about ±1/3 order of magnitude, predictions of failure time have uncertainties approaching ±1 order of magnitude, and predictions of peak flow have uncertainties of about ±0.5 to ±1 order of magnitude, except the Froehlich peak flow equation, which has an uncertainty of about ±1/3 order of magnitude.

The uncertainty analysis made use of a database of information on the failure of 108 dams compiled from numerous sources in the literature (Wahl 1998). For those wishing to make use of this database, it is available in electronic form (Lotus 1-2-3, Microsoft Excel, and Microsoft Access) on the Internet at http://www.usbr.gov/wrrl/twahl/damfailuredatabase.zip.

The case study presented for Jamestown Dam showed that significant engineering judgment must be exercised in the interpretation of predictions obtained from the regression-based methods. The results from use of the physically-based NWS-BREACH model were reassuring because they fell within the range of values obtained from the regression-based methods, but at the same time they also helped to show that even physically-based methods can be highly sensitive to the analysts assumptions regarding breach morphology and the location of initial breach development. The NWS-BREACH simulations revealed the possibility for limiting failure mechanics that were not considered in the regression-based methods.

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