PAP 867

Breach Parameters for Risk Assessment of Jamestown Dam

by

Tony L. Wahl

December 2000

DATE	PEER REVIEWER(S)	CODE
1/26/01	[signature] Signature / John A. Wilson Printed Name	D-8313
	Signature / Printed Name	
Author Initials	PEER REVIEW NOT REQUIRED	

Breach Parameters for Risk Assessment of Jamestown Dam

Tony L. Wahl
Water Resources Research Laboratory, D-8560
December 2000

This report describes studies performed to estimate parameters of potential embankment breaches of Jamestown Dam, a feature of the Pick-Sloan Missouri Basin Program, located on the James River immediately upstream from Jamestown, North Dakota. These studies were performed to support the risk assessment of Jamestown Dam to be conducted during January 2001.

The focus of these studies is on risks associated with static loading conditions. The significant potential static failure modes are:

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

No distinction between these two failure modes is made in the analyses described in this report, since most methods used to predict breach parameters lack the refinement needed to consider the differences in breach morphology for these two failure modes. Some methods do make a distinction between breaches caused by overtopping and those caused by generic piping.

Jamestown Dam is a multipurpose facility operated for flood control, municipal water supply, fish and wildlife benefits, and recreation. 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 are considered in this 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

Dam Description

Jamestown Dam is located on the James River about 1.5 miles upstream from the city of Jamestown, North Dakota. The reservoir impounds an active storage capacity of 28,910 ac-ft at reservoir elevation 1429.8 ft and has a total capacity of about 221,000 ac-ft at the top of exclusive flood control space, elevation 1454 ft. The dam and appurtenant structures were constructed during 1952-1954. 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 dam has a crest length of 1,418 ft at elevation 1471 ft and a crest width of 30 ft. The design includes a central compacted zone 1 impervious material, and upstream and downstream zone 2 of sand and gravel. 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.

The outlet works is located on the left side of the dam, and consists of an intake structure, a 9-ft 6-in. diameter upstream cut and cover conduit, a gate chamber, a downstream 13-ft 6-in. horseshoe shaped cut and cover conduit, a concrete-lined rectangular outlet channel, stilling basin, and discharge channel. The gate chamber contains two 5- by 6-ft high-pressure slide gates for regulation and two emergency gates of the same size. A 7-ft by 5-ft access adit parallels the horseshoe conduit from the downstream control house to the gate chamber. The outlet works capacity is 2,990 ft^3/s with the reservoir at elevation 1464.13 ft.

The spillway, located on the right abutment, is a concrete uncontrolled glory-hole with a 24.33 ft-diameter crest at elevation 1454 ft, and a 9.5-ft-diameter cut and cover conduit and a concrete lined channel and stilling basin. The design capacity of the spillway is 2,800 ft^3/s with the reservoir at elevation 1459 ft.

The safe downstream channel capacity as reported in the Performance Parameters Technical Memorandum is 900 ft^3/s, however, operations personnel indicate that the capacity is 1,250 ft^3/s, and up to about 1,800 ft^3/s with the storm sewer outlets through Jamestown blocked. The community of Jamestown is located immediately downstream from the dam along the James River. Extensive development exists in the downstream areas within the flood plain of the river. The maximum reservoir water surface to date, elevation 1445.9 ft, occurred in May 1997. The maximum discharge to date is about 1,200 ft^3/s.

The abutments of Jamestown Dam 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.

The design and construction for the dam included a toe drain within the downstream embankment near the foundation level, and a fairly wide embankment section to help control seepage anticipated 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, however, 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 1995-1996. To increase the seepage protection, a filter blanket was constructed in low areas downstream from the dam in 1998.

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. Most such models expend the bulk of 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 close to a dam, and assumptions about available warning and evacuation time will dramatically affect predictions of loss of life. For the purpose of modeling the development of a breach, the outflow from it, and the routing of the resulting flood wave, breach development begins when a breach has developed to the point that 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.

The breach development time does not include the potentially long preceding period sometimes described as the breach initiation phase (Wahl, 1998), which can also be important when considering available warning and evacuation time. This is the phase in 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 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. However, an overtopping failure is not the focus of this study.

During a piping failure the delineation between breach initiation and breach development phases is less apparent. In some cases, piping failures can take a great deal of time to develop. In contrast to the overtopping case, the loading that causes a piping failure cannot normally be removed quickly, and the process does not take place in full view, except that the outflow (seepage) from a developing pipe can be observed and measured. One useful way to make the distinction is to consider three possible conditions: (1) normal seepage outflow, with clear water and low flow rates, (2) initiation of a seepage/piping failure with cloudy seepage water that indicates a developing pipe, but flow rates are still low and not rapidly increasing, and (3) active development phase of a seepage/piping failure in which erosion is dramatic and flow rates are rapidly increasing. 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 are 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 dam heights less than 15 meters, or 50 ft (Wahl, 1998). Jamestown Dam has a structural height of 110 ft, but the depth of water stored behind the dam has thus far not exceeded about 47 ft.

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 gross breach parameters may be developed for use in a dam-break flood routing model. The primary weakness of the NWS-BREACH model and other similar models is the reliance on tractive-stress erosion models that do not adequately model the headcut-type erosion processes that dominate the breaching of cohesive-soil embankments. 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, rather than tractive stress. Research is presently underway to adapt headcut erosion models to dam breach scenarios, but there are no such tools at this time that could be applied to Jamestown Dam.

The uncertainty is very large in all available methods for predicting breach parameters. Most investigators have not specifically quantified the prediction uncertainty of their regression relations. Wahl (1998) used many of the available relations to predict breach parameters for 108 documented case studies. The predicted values were then plotted vs. 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.

An indication of the uncertainty in the breach parameter predictions was obtained in this study by using several different prediction methods, and by developing prediction uncertainty estimates for the individual methods using a procedure described in the Appendix. The uncertainty estimates are expressed as a number of log cycles, and are thus applied as multipliers on the predicted values. This differs from the more traditional method of computing an error band and simply adding or subtracting the uncertainty to establish upper and lower bounds of the predicted value. This approach was used because most of the breach parameter prediction equations are power-curve type equations in which the relations and their errors are logarithmic in nature. Thus, for example, if a prediction equation was deemed to have an uncertainty of ±0.3 log cycles, we would compute the upper prediction limit by multiplying the predicted value by $10^{0.3}=2$; the lower prediction limit would be obtained by dividing the predicted value by the same factor. Details of the development of the uncertainty estimates are given in the Appendix.

The methods used to predict breach parameters were:

- MacDonald and Langridge-Monopolis (1984) – Regression relations that predict the volume of eroded material and the failure time

- Reclamation (1988) – Regression relations for average breach width (3 times the depth of water behind the dam at failure) and time of failure

- Von Thun and Gillette (1990) and Dewey and Gillette (1993) – Regression relations that predict average breach width and failure time

- Froehlich (1995b) – Regression relations that predict average breach width and failure time

- BREACH model – simulations of breach development and breach outflow hydrograph

Specific equations for each of the breach parameter prediction methods (except the NWS-BREACH model) are included in the Appendix, Table A1.

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. For this latter purpose, estimates of peak outflow from Jamestown Dam were made using the following relations:

- 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 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. A key parameter in their method is an assumed vertical erosion rate of the breach, and they present a range of reasonable values developed from an analysis of case study data. The Walder and O'Connor 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. 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 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 breach hydrograph prediction methods.

Results — Breach Parameter Estimates

The breach parameter prediction methods outlined above were applied to Jamestown Dam 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.

Breach Width

Predictions of average breach width are summarized in Table 1. 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 1. — Predictions of average breach width.

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 in the Appendix showed that the Reclamation equation tends to underestimate the observed breach widths when tested against the 108 dam failure case study collection in Wahl (1998), 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 described in Appendix A 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. It would be valuable to consider these ranges in the risk assessment process.

Failure Time

Failure time predictions are summarized in Table 2. 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 2. — Failure time predictions.

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 1. 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 in Appendix A 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 3, 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 3. — Predictions of peak breach outflow.

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 described in Appendix A, and is thus highlighted in the table. This equation had the smallest mean prediction error and smallest median deviation 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; see Appendix A). 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.

Results — NWS-BREACH Simulations

Several simulations runs were made using the National Weather Service BREACH model (Fread, 1988), for the top-of-joint-use, top-of-flood-control, and maximum-design-water-surface scenarios. 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 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 4 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 4. — Results of NWS-BREACH simulations.

	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 could still significantly change these results.

Conclusions

Tables 1 and 2 provide recommended values of breach width and time of failure for use in analyzing consequences of a potential seepage-erosion failure of Jamestown Dam at four different reservoir levels:

- 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

In addition to the recommended values, upper and lower bound values of the parameters for each scenario are also provided. These prediction intervals were developed through an uncertainty analysis of predicted and observed breach parameters for 108 documented dam failures. Given the sensitivity of dam failure consequences to variations in the breach parameters, it may be valuable to consider incorporating these upper and lower bound values into the risk assessment process. This could be accomplished by performing the consequence analysis with a range of values of the most critical parameters, and assigning a probability of occurrence to each case.

Predictions of peak breach outflow were made by several methods (Table 3 and Table 4), for use in checking the reasonability of peak outflow estimates obtained using the DAMBRK model and the predicted breach parameters. The peak outflow predictions provided by the Froehlich (1995a) method can be considered a best estimate, and those provided by the Walder and O'Connor (1997) method can be considered a maximum reasonable value for this large-reservoir case. Peak outflows obtained from NWS-BREACH simulations were similar to the Froehlich (1995a) results.

Appendix

Developing Uncertainty Estimates for Breach Parameter Prediction Equations

In this study, uncertainty bands were computed around the predicted values of breach parameters for Jamestown Dam. The procedures used to develop these uncertainty bands were newly developed for this study, using data first presented in Wahl (1998). This appendix describes the methods used.

The study by Wahl (1998) presented data from 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 in the literature. A notable exception was the relation of Walder and O'Connor (1997), which was not tested in this manner because it requires the estimation of the expected vertical erosion rate of the embankment, which was unknown before the dam failures in the database occurred. 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.
- Some of the testing made use of the same data used to originally develop the breach parameter prediction equations, but each equation was also tested against some additional cases. This should provide a fair indication of the ability of each equation to predict breach parameters for future dam failures.

Results of these comparisons were primarily presented in graphical form in Wahl (1998), in plots comparing predicted and observed values. Some of the data were plotted on logarithmic scales (e.g., failure time), while others were plotted arithmetically (e.g., breach width). Regardless of the parameter, the differences between predicted and observed values were often large, and the range of the data to be considered was also large, given that the database included very small to very large dams.

To make estimates of the uncertainty of predictions made with the various equations and provide some basis for comparing their uncertainties, the following method was used:

(1) All of the predicted vs. observed value figures were plotted on log-log scales, so that prediction errors for small dams could be compared on an equal basis with the prediction errors for large dams.

(2) The individual prediction errors were computed 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) Recognizing that there were a few significant outliers in most of the data sets, an outlier-exclusion algorithm was applied to the series of prediction errors computed in (2). The source of these outliers is believed to the questionable quality of some of the observed values, the potential for misapplication of some of the prediction equations due to lack of intimate knowledge of each case study, and inherent variability in the data due to the variety of factors that influence dam breach mechanics. The outlier-exclusion algorithm is described by Wadsworth (1990). The algorithm has the advantage that it is, itself, insensitive to the effects of outliers. The outlier-exclusion procedure is as follows:

(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.

(c) Compute an estimator of scale, S=1.483*(median of the absolute deviations). 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) After the outliers have been excluded, the mean, $\bar{e}$, and the standard deviation, S_e, of the remaining prediction errors are computed. 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 $\pm 2S_e$ makes this confidence band equivalent to about a 95 percent prediction interval. (The multiplying factor for S_e should vary with the number of data points, but this level of refinement was not warranted.)

Table A1 summarizes the results of applying this method. The rightmost column of the table shows the range of the prediction interval around a hypothetical predicted value of 1.0. These are multipliers than can be applied to obtain the prediction interval for a specific case.

Table A1. – Uncertainty estimates of breach parameter and peak-flow prediction equations. All equations use metric units (meters, m³, m³/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 PARAMETER EQUATIONS					
USBR (1988) $\bar{B} = 3(h_w)$	80	70	-0.09	±0.43	0.45 — 3.3
MacDonald and Langridge-Monopolis (1984) $V_{er} = 0.0261(V_w \cdot h_w)^{0.769}$ *earthfil* $V_{er} = 0.00348(V_w \cdot h_w)^{0.852}$ *non-earthfill (e.g., rockfill*	60	58	-0.01	±0.82	0.15 — 6.8
Von Thun and Gillette (1990) $\bar{B} = 2.5h_w + C_b$ *where C_b is a function of reservoir size*	78	70	+0.09	±0.35	0.37 — 1.8
Froehlich (1995b) $\bar{B} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ *where K_o = 1.4 for overtopping, 1.0 for piping*	77	75	+0.01	±0.39	0.40 — 2.4
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 erodibl* $t_f = 0.020(h_w) + 0.25$ *erosion resistan*	36	34	-0.64	±0.95	0.49 — 40.
Von Thun and Gillette (1990) $t_f = \bar{B}/(4h_w)$ *highly erodibl* $t_f = \bar{B}/(4h_w + 61)$ *erosion resistan*	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) $t_f = 0.011(\bar{B})$	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)	38	32	+0.19	±0.50	0.20 — 2.1

Equation		Number of Case Studies: Before outlier exclusion	After outlier exclusion	Mean Prediction Error, $\overline{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
$Q_p = 19.1(h_w)^{1.85}$	*envelope equatio*					
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 equatio*	37	36	+0.64	±0.70	0.05 — 1.1
Costa (1985)						
$Q_p = 1.122(S)^{0.57}$	*envelope equatio*	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 equatio*	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 Table A1.

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

C_b = offset factor used 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, equal to 1.4 for overtopping failures, or 1.0 for piping failures

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

References

Costa, John E., 1985, *Floods from Dam Failures*, U.S. Geological Survey Open-File Report 85-560, Denver, Colorado, 54 p.

Dewey, Robert L., and David R. Gillette, 1993, "Prediction of Embankment Dam Breaching for Hazard Assessment," *Proceedings*, ASCE Specialty Conference on Geotechnical Practice in Dam Rehabilitation, Raleigh, North Carolina, April 25-28, 1993.

Evans, Steven G., 1986, "The Maximum Discharge of Outburst Floods Caused by the Breaching of Man-Made and Natural Dams," *Canadian Geotechnical Journal*, vol. 23, August 1986.

Fread, D.L., 1984, *DAMBRK: The NWS Dam-Break Flood Forecasting Model*, National Weather Service, Office of Hydrology, Silver Spring, Maryland.

Fread, D.L., 1988 (revised 1991), *BREACH: An Erosion Model for Earthen Dam Failures*, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, Maryland.

Fread, D.L., 1993, "NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction," *Dam Safety '93*, Proceedings of the 10th Annual ASDSO Conference, Kansas City, Missouri, September 26-29, 1993, p. 177-184.

Froehlich, David C., 1987, "Embankment-Dam Breach Parameters," *Hydraulic Engineering*, Proceedings of the 1987 ASCE National Conference on Hydraulic Engineering, Williamsburg, Virginia, August 3-7, 1987, p. 570-575.

Froehlich, David C., 1995a, "Peak Outflow from Breached Embankment Dam," *Journal of Water Resources Planning and Management*, vol. 121, no. 1, p. 90-97.

Froehlich, David C., 1995b, "Embankment Dam Breach Parameters Revisited," *Water Resources Engineering*, Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, p. 887-891.

Hagen, Vernon K., 1982, "Re-evaluation of Design Floods and Dam Safety," *Proceedings*, 14th Congress of International Commission on Large Dams, Rio de Janeiro.

Johnson, F.A., and P. Illes, 1976, "A Classification of Dam Failures," *International Water Power and Dam Construction*, December 1976, p. 43-45.

Kirkpatrick, Gerald W., 1977, "Evaluation Guidelines for Spillway Adequacy," *The Evaluation of Dam Safety*, Engineering Foundation Conference, Pacific Grove, California, ASCE, p. 395-414.

MacDonald, Thomas C., and Jennifer Langridge-Monopolis, 1984, "Breaching Characteristics of Dam Failures," *Journal of Hydraulic Engineering*, vol. 110, no. 5, p. 567-586.

Singh, Krishan P., and Arni Snorrason, 1984, "Sensitivity of Outflow Peaks and Flood Stages to the Selection of Dam Breach Parameters and Simulation Models," *Journal of Hydrology*, vol. 68, p. 295-310.

Soil Conservation Service, 1981, *Simplified Dam-Breach Routing Procedure*, Technical Release No. 66 (Rev. 1), December 1981, 39 p.

Temple, Darrel M., and John S. Moore, 1994, "Headcut Advance Prediction for Earth Spillways," presented at the *1994 ASAE International Winter Meeting*, Paper No. 94-2540, Atlanta, Georgia, December 13-16, 1994, 19 p.

U.S. Bureau of Reclamation, 1982, *Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams*, Reclamation Planning Instruction No. 82-11, June 15, 1982.

U.S. Bureau of Reclamation, 1988, *Downstream Hazard Classification Guidelines*, ACER Technical Memorandum No. 11, Assistant Commissioner-Engineering and Research, Denver, Colorado, December 1988, 57 p.

Von Thun, J. Lawrence, and David R. Gillette, 1990, *Guidance on Breach Parameters*, unpublished internal document, U.S. Bureau of Reclamation, Denver, Colorado, March 13, 1990, 17 p.

Wadsworth, H.M. (1990). *Handbook of Statistical Methods for Engineers and Scientists*. New York: McGraw-Hill.

Wahl, Tony L., 1998, *Prediction of Embankment Dam Breach Parameters - A Literature Review and Needs Assessment*, U.S. Bureau of Reclamation Dam Safety Report DSO-98-004, July 1998.

Walder, Joseph S., and Jim E. O'Connor, 1997, "Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earth Dams," *Water Resources Research*, vol. 33, no. 10, October 1997, 12 p.