THE ROLE OF SENSITIVITY ANALYSIS IN SELECTING DAM BREACH PARAMETERS

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ABSTRACT

This paper focuses on conducting sensitivity analysis to guide the selection of breach parameters for both water retaining and tailings dams. Sensitivity analysis on breach parameters could be undertaken using multiple empirical equations or through probability analysis (e.g., Monte Carlo simulations), which requires that a range and a statistical distribution are defined for each parameter. The governing considerations for breach parameter selection are discussed. These include, but are not limited to, dam construction type and construction materials, erosion or downcutting rates, topographic considerations, materials impounded behind the dam, and review of relevant historical cases. New empirical equations for eroded dam volume and dam erosion rate are presented that were developed using multiple linear regressions with breach height and breach outflow volume as inputs. An additional consideration specific to tailings dams is estimating the volume of tailings that could mobilize in a breach. This depends on the size of the supernatant pond, on tailings characteristics and their susceptibility to liquefaction, as well as on rheological characteristics that define their flowability, all of which impact the shape and the peak discharge of the breach outflow hydrograph.

RÉSUMÉ

Cet article se concentre sur la réalisation de l'analyse de sensibilité pour guider la sélection des paramètres de brèche pour les retenues d'eau et les digues à résidus. L'analyse de sensibilité sur les paramètres de brèches peut être entreprise en utilisant plusieurs équations empiriques ou par une analyse de probabilité (p ex. des simulations Monte Carlo), ce qui nécessite qu'une plage et une distribution statistique soient définies pour chaque paramètre. Les considérations déterminantes pour leur sélection sont discutées. Ceux-ci comprennent mais sans s'y limiter, le type de construction de barrage et les matériaux de construction, les taux d'érosion ou d'abattage, les considérations topographiques, les matériaux retenus par le barrage et l'examen des cas historiques pertinentes. Une nouvelle équation empirique pour le volume du barrage érodé développé utilisant la régression linéaire multiple avec la hauteur de brèche et volume écoulé est présenté. Un considération supplémentaire spécifique aux digues à résidus est estimer le volume de résidus qui pourrait se mobiliser durant une rupture. Celui-ci dépend de la taille du bassin surnageant, des caractéristiques des résidus et de leur potentiel de liquéfaction, ainsi que des caractéristiques rhéologiques qui définissent leur fluidité, qui ont tous un impact sur la forme et le débit maximal du l'hydrogramme d'écoulement de brèche.

1 INTRODUCTION

The released volume, the discharge, and how the dam breach develops depend on numerous factors, including the dam construction type and geometry, the materials used to construct the dam, the materials impounded by the dam, and the storage basin and downstream topography (Brunner, 2014; Small et al., 2017; CDA, 2021). The breach development is primarily defined with breach parameters (geometry and formation time). The combination of breach parameters defines the peak discharge and the shape of the outflow hydrograph, which influence the estimated consequences in conjunction with downstream routing and inundation mapping. Breach development can largely be grouped into two types (CDA, 2021): erosional breaches (e.g., resulting from overtopping or seepage/piping events); and non-erosional and often nearly instantaneous breaches (e.g., resulting from toppling of entire slabs for concrete dams, or liquefaction for earth embankments).

Numerous empirical equations exist that can be used to evaluate the breach parameters and the peak discharge, as summarized in Wahl (1998, 2014), Brunner (2014), or West et al. (2018). Most empirical equations were developed based on erosional failures of water retaining dams, leading to additional uncertainty when applied to tailings dams. The estimates from various empirical equations can vary widely, signifying the uncertainty associated with determining the breach parameters (Martin and Akkerman, 2017). Furthermore, the application of empirical equations in situations dissimilar or fully outside the conditions of the original events used to develop the equations can result in irrelevant, non-conservative, or physically impossible outcomes.

The uncertainties in these types of analyses are numerous, and the methods are often based on approximations, which need to be understood and accounted for, often leading to results that are better represented with a range rather than a single value. For these reasons and to evaluate the uncertainty, sensitivity analysis is one of the key tools in dam breach analysis that should be undertaken to guide the selection of breach parameters and development of outflow hydrographs, as recommended in the CDA Technical Bulletin “Tailings Dam Breach Analysis” (CDA, 2021). It could be undertaken by evaluating multiple empirical equations, using physically based breach models with varying input parameters, or through probability analysis (e.g., Monte Carlo simulations).

This paper reflects on experiences and lessons learned during our practice, and focuses on the following topics: i) physical processes that can occur during a breach of water retaining or tailings dams; ii) development of a novel set of empirical equations to predict the volume of eroded dam material and the rate at which it is eroded; and iii) various considerations and applications relevant to conducting a sensitivity analysis. These discussion points highlight the relevant considerations for breach analysis and the selection of breach parameters through application of sensitivity analysis.

2 DAM BREACH PROCESSES

Strong understanding of the physical processes during the breach and subsequent flood wave propagation are key to successful breach studies (Rana et al., 2021a). For context, these processes and existing conventions are summarized below.

2.1 Breach and Outflow Processes

Breach processes and characteristics from internal erosion and overtopping failures of water retaining embankments are widely known, compiled, modelled, and studied (e.g., USBR, 1988; Wahl, 1998; Froehlich, 2008; Wahl, 2014; Walder et al., 2015; Walsh et al., 2021). These processes primarily involve erosion of the dam materials by water flowing over the crest or through the dam over a period of minutes to hours. Failures can also be caused by slope instability, foundation failure, structural failure, liquefaction or other mechanism that could remove the containment structure within seconds to minutes. These “non-erosional” breaches are typically more associated with concrete dam failures (e.g., the St. France Dam in the USA, 1928; the Malpasset Dam in France, 1959), but can occur for earth embankments as well (e.g., the Schaeffer Dam in the USA, 1921; the Edenville Dam in the USA, 2020). This paper focusses on embankments and does not explicitly address concrete dam structures, but general understanding of the purpose of sensitivity analysis is equally applicable to such structures.

Non-erosional breach processes are seemingly more common for tailings dams than for water retaining dams (e.g., the Stava dams in Italy, 1985; the Fundão and Feijão dams in Brazil, 2015 and 2019, respectively). The susceptibility of tailings dams to these types of failures may be associated with the longer duration of construction that typically continues throughout the operations period, variability of tailings materials impounded behind tailings dams and their potential to liquefy and flow, or the type of construction methods and materials used for tailings storage facilities (TSFs), as summarized by Rana et al., (2021a). Whether a dam is breached through erosional or non-erosional processes can impact the outflow volume, the breach geometry and formation time, and the peak flow and hydrograph shape, as discussed in this paper. Additional discussion of some of the differences between erosional and non-erosional breaches with respect to tailings dams is provided in Adria et al. (2021a) and Adria (2022).

The breach outflow volume for water reservoirs typically includes the entire volume above the bottom of the breach, while for tailings dam breaches it depends on whether there is a pond present in the TSF and whether the tailings are susceptible to liquefaction (Small et al, 2017, CDA 2021). The CDA guidelines (2021) define two processes to qualitatively describe the discharge mechanisms associated with the outflow of the supernatant pond and of the liquefied tailings runout from a TSF. Process I represents the discharge of the supernatant pond that carries eroded tailings and dam fill materials, which can sometimes be analysed using Newtonian flow characteristics. This process can be considered similar to breach outflows from water retaining dams. Process II represents the discharge of flowable tailings due to tailings liquefaction, or progressive slumping of unsupported tailings. The outflow from Process II would have a much lower water content compared to Process I, primarily containing tailings solids and interstitial water, and must be analyzed as non-Newtonian.

The discharge mechanisms are related to, but are different from, the processes during the breach development itself. Process I breach outflows have a relatively low solids content and are highly erosive, as was observed in the Mount Polley failure in 2014 with erosion of Hazeltine Creek (Morgenstern et al., 2015; Cuervo et al. 2017). In contrast, Process II outflows have a high solids content and typically result in limited erosion. For example, the liquefied tailings outflow from the Prestavèl TSF in Italy (Stava) had a high fluidity and an intensive destructive power, but the downstream creek channel itself did not suffer from much erosion or deposition (Berti et al. 1997). Takahashi (1991, 2014) stated that no erosion had occurred, because the solid fraction inside the water-sediment mixture of the Prestavèl tailings was so high (estimated to be about 48% by volume) that the flow could not become denser through additional erosion. This concept is also supported by Hungr et al. (2005), who stated that flows with lower volumetric sediment concentrations can be expected to be more erosive than flows with higher sediment concentrations. In this paper, the focus is on erosional processes through the embankment, which is considered separate from downstream processes of erosion and entrainment, which would include vegetation, structures, or other surface debris carried along with the flow.

2.2 Breach Geometry Conventions

The ultimate breach geometry is commonly represented with a trapezoid (Figure 1), which can be described by the breach height and any combination of two of the following parameters: average breach width, bottom breach width, top breach width, or breach side slopes. The ultimate breach geometry can be formed by a V-shape cut that progresses downwards until it encounters materials that are either non-erodible or substantially less erodible than the dam material. The breach then progresses laterally and widens until the outflow no longer has the capacity to enlarge the breach opening, commonly at the receding end of the outflow hydrograph. The period from when the V-shape begins cutting down the dam to when the ultimate breach geometry is reached is considered the breach formation time. The formation time excludes the comparatively minor headcutting during the initial overtopping or internal erosion development (Wahl, 1998) that occurs prior to the main breach progression.

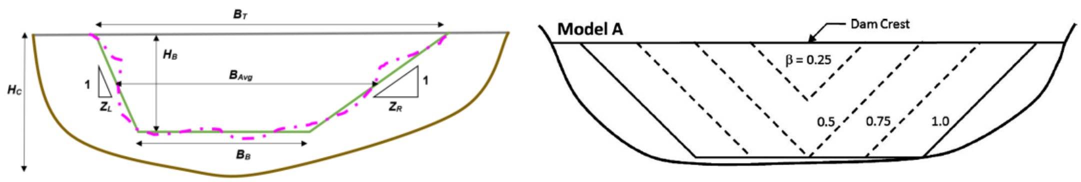

Figure 1: Trapezoid breach approximation a) Geometry and b) Breach formation (Froehlich, 2008)

3 DEVELOPMENT OF NOVEL BREACH EQUATIONS

Two new equations are proposed that estimate the eroded dam volume and the eroded dam volume rate. They are based on the multiple linear regression approach applied to water retaining and tailings dams databases that had adequate information.

3.1 Eroded Dam Volume

Our approach combines several concepts and data from work by other authors, but is primarily inspired by the approach of MacDonald and Langridge-Monopolis (1984). They developed a best fit empirical equation (Equation 1) based on 31 failures of water retaining earthfill dams to estimate the dam volume that would be eroded during a breach:

$$V_M = 0.0261\ (V_w\ H_w)^{0.769} \tag{1}$$

Where: V_M is the volume of eroded dam material [m^3] (denoted as V_{Er} in Wahl, 1998), V_W is the volume of water discharged through the breach (initial storage and inflow during failure) [m^3], and H_W is the hydraulic depth of water above the breach bottom [m].

Subsequent work assessed the average width through the hydraulic control of the breach, rather than the eroded dam volume (e.g., Von Thun & Gillette, 1990; and Froehlich, 1995 or 2008). Using the trapezoidal breach approximation and relevant geometric relations, the eroded dam volume can be converted to the bottom breach width, B_B, using Equation 2 (Washington Guidelines, 2008):

$$B_B = \frac{V_M - {H_B}^2(W_C Z_B + H_B Z_B (Z_U + Z_D)/3)}{H_B(W_C + H_B(Z_U + Z_D)/2)} \tag{2}$$

Where: H_B is the breach height [m], W_C is the width of the crest of the dam [m], Z_U is the slope for the upstream face of the dam [xV:1H], Z_D is the slope for the downstream face of the dam [xV:1H], and Z_B is the slope of the breach sides [xV:1H].

The disadvantage of the MacDonald and Langridge-Monopolis (1984) equation is that it does not include additional dam breach events that occurred after 1984 or were investigated and compiled by later authors. In addition, it does not take advantage of more sophisticated regression approaches. These limitations led to a development of a new equation that combines several concepts and existing databases. In our approach, the eroded dam volume is estimated using combined and verified databases for water retaining dam breaches from Wahl (2014) and for erosional tailings dam breaches from Adria (2022). The application of this equation to tailings dams is discussed in further detail later. The proposed relationship utilizes multiple linear regression (MLR) of dam height and outflow volume, similar to Froehlich (2008). MLR has better predictive quality than methods in which each input is used individually or combined (as in the height-volume product in MacDonald and Langridge-Monopolis, 1984). The new eroded dam volume relationship is shown in Equation 3:

$$\boldsymbol{V_{ED} = 1.26 H_W^{1.803} V_{Out}^{0.338}} \quad (3)$$

Where: V_{ED} is the volume of the eroded dam material [m^3], H_w is the height of water above the breach [m], and V_{Out} is the volume of breach outflow [m^3]. The variable V_{ED} is the same as V_M or V_{Er} used by MacDonald and Langridge-Monopolis (1984) or Wahl (1998), however the labelling is changed to prevent confusion with eroded tailings volumes or dam material volumes that discharge through non-erosional breach processes. The equation has an adjusted R^2 value of 0.891 and a standard error of 0.31 (in log space) based on 74 events in the database with adequate data availability.

Figure 2 compares V_{ED} values calculated using Equation 3 with observed values, with an inset showing a box and whisker plot of the predicted to observed ratios. The dashed lines represent half an order of magnitude above and below the 1:1 line, indicating the approximate range of the error. Ideally, the boxplot has a mean and median close to unity (indicating little error on average) and a narrow interquartile range (indicating lower variability in estimated values). The interquartile range for Equation 3 is 0.6 to 1.6. For comparison, the interquartile range for other breach parameter regressions can range from 0.4 to 2.4 and even up to 0.2 to 6.0 (Wahl, 2004), indicating Equation 3 has comparable or better performance to previous equations.

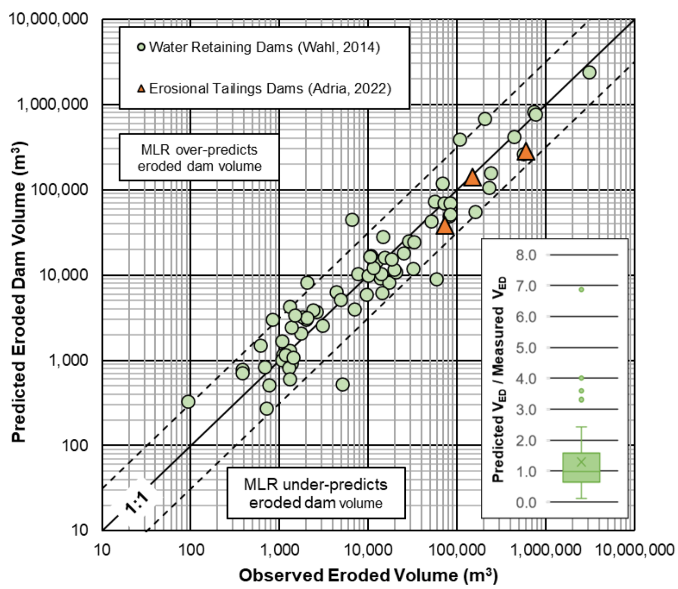

Figure 2: Comparison of observed and predicted eroded dam volume based on Equation 3.

For tailings dam breaches, the volume of tailings that are eroded with the supernatant pond during Process I outflows is included in V_{Out}, while the liquefied tailings volume discharged in Process II outflows is not included in consideration of the limited erosive potential of Process II flows. This equation should not be applied to non-erosional breach events, where the breach develops due to drivers other than erosion. Hypothetical example cases for application of Equation 3 are presented below.

3.1.1 Reservoirs of Equal Size with Different Dam Geometry

Example 1: The geometries of two hypothetical water retaining dams are summarized in Table 1. They are of equal height and reservoir volume, but the second dam has a much wider crest and shallower downstream slopes. The narrow dam is approximately representative of the average crest width and upstream and downstream slopes of dams in the databases of Wahl (1998 and 2014). The wider dam values were assigned to be larger than those for the narrow dam scenario.

Assuming a breach progresses to the natural ground (i.e., the breach height is equal to the dam height), one should intuitively expect a smaller average breach width for the wider dam because there is much more dam cross section that needs to be eroded. The empirical equations that directly predict the average breach width do not consider the dam geometry, so they may under-predict the average breach width for the narrower dam but over-predict it for the wider dam. The results from Equation 3 for two conceptual dam geometries are included in Table 1.

Table 1: Comparison of estimated breach sizes for different dam geometries

Parameter	Narrow Dam	Wide Dam
Total Outflow Volume (m^3)	50,000,000	50,000,000
Height of Water and Breach Height (m)	45 and 50	45 and 50
Crest Width (m)	5	15
Upstream and Downstream Slopes (xH:1V)	1.5 and 1.5	1.5 and 3.5
Breach Side Slopes (xH:1V)	1	1
Eroded Dam Volume (Equation 3) (m^3)	482,000	482,000
Breach Bottom Width (Equation 2) (m)	86	34

Physical modelling has been carried out recently to evaluate the effect of dam slopes on breach outflows (Walsh, 2019; Walsh et al., 2021). Flume experiments using scale dams with upstream slopes ranging from 10° to 30° (i.e., 5.88H:1V to 1.75H:1V) showed that the peak breach outflow was decreased for dams with shallower upstream slopes. The wide dam with a shallower slope and a narrower breach width in Example 1 would also produce a lower peak flow. This indicates the findings from Walsh et al. (2021) and the above example for Equation 3 are generally aligned; however, continued research is warranted to confirm if the findings in Walsh et al. (2021) are predominantly due to decreased breach width or slower formation times.

3.1.2 Application to Tailings Dams

The erosional tailings dam breach events from Adria (2022) include the 1998 Aznalcóllar event (Los Frailes) in Spain, the 2010 MAL Reservoir X (Ajka/Kolontár) event in Hungary, and the 2014 Mount Polley event in Canada. For the inclusion of tailings dam breach events in the regression equation, the estimated volumes of tailings eroded during the supernatant pond discharge were included in the analysis.

Example 2: The characteristics of two hypothetical TSFs are summarized in Table 2. They have the exact same dam geometries with a dam height of 50 m, a crest width of 20 m, upstream and downstream slopes of 1H:1V and 4H:1V, but the supernatant pond volumes are different. The tailings are considered non-liquefiable (i.e., they would represent the 1B classification case according to CDA, 2021), however, portion of the tailings would be eroded and mobilized during the breach development. Assuming a breach to the natural ground may be reasonable for the larger pond, but it may not be physically possible for the smaller pond. Equation 3 can be used to estimate the breach height given the outflow volume using a V-shaped breach geometry (i.e., 0 m bottom width).

Table 2: Comparison of estimated breach heights for different supernatant pond volumes for a tailings dam

Parameter	Smaller Pond	Larger Pond
Supernatant Pond Volume (m^3)	400,000	1,800,000
Eroded Tailings Volume (m^3)	600,000	2,700,000
Total Outflow Volume (m^3)	1,000,000	4,500,000
Dam Height (m)	50	50
Breach Side Slope (xH:1V)	1	1
Eroded Dam Volume (m^3)	62,000	258,000
Estimated Breach Height (m)	30	50

To demonstrate the relationship between the breach outflow volume and the possible eroded dam volume, the following approach can be utilized. First, the volume of the dam that could be eroded is calculated using Equation 2 with a breach side slope of 1H:1V, a bottom breach width of 0, and the geometry of the dam in question (i.e., crest width, upstream dam slope, and downstream dam slope). The calculation is performed at regular breach depth intervals, moving downwards from the crest elevation. This essentially represents estimating the eroded dam volume during the breach formation at each elevation interval, prior to the breach reaching the natural ground. Second, the eroded dam volume is calculated using Equation 3 for the same intervals using the outflow volume and varying the breach height. Third, the two eroded dam volume curves are plotted and the breach height where the two curves intersect represents the ultimate breach height. If the curves do not cross, it indicates that the breach could progress down to natural ground.

This process is shown on Figure 3, using the above hypothetical example with the two supernatant pond volumes from Table 2. The eroded tailings volume in Process I was calculated using a 30% volumetric solids concentration in the outflow and a void ratio of 1 for the tailings (Fontaine and Martin 2015; CDA, 2021). The smaller pond is unable to erode the dam to its foundation, while the larger pond is sufficient to just erode it to the foundation. A pond larger than 1,800,000 m^3 would be able to erode to foundation and proceed to lateral erosion, as described by Froehlich (2008).

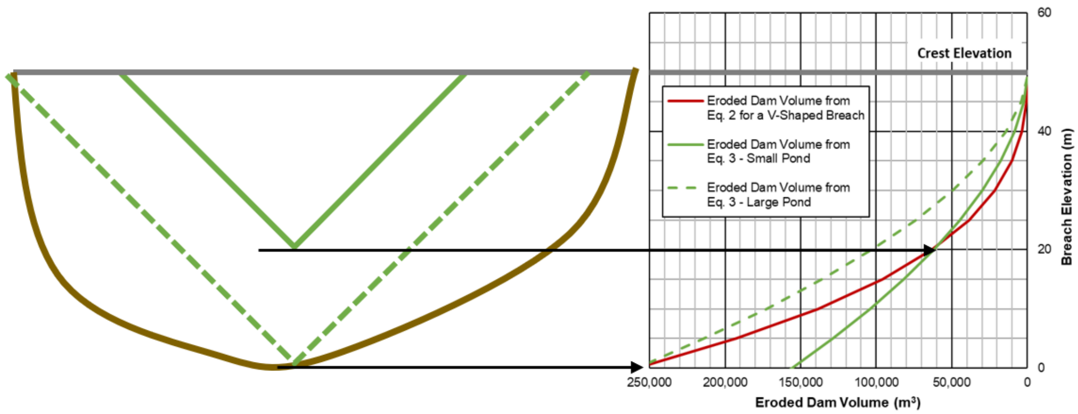

Figure 3: Comparison of the calculated eroded dam volume against the dam volume that would need to be eroded to reach a specific breach height with the V-shape breach

3.2 Mean Eroded Dam Volume Rate

Walder and O'Connor (1997) proposed a coefficient for the mean erosion rate, k, representing the breach height divided by the formation time, as a means to normalize the formation time across dams of different heights. They noted that dams rarely experienced mean erosion rates outside of 10 m/hr to 100 m/hr. This concept is adapted for another novel equation to estimate the mean eroded dam volume rate, M, or the eroded dam volume divided by the formation time (i.e., V_{ED}/T_f). Similar to Equation 3, the development methodology for Equation 4 utilizes the MLR approach:

$$M = 415\, H_W^{\,2.296} V_{Out}^{\,-0.098} \quad (4)$$

Where: M is the mean eroded dam volume rate [m^3/h], H_W is the height of water above the breach [m], and V_{Out} is the volume of breach outflow [m^3]. The equation has an adjusted R^2 value of 0.820 and a standard error of 0.34 (in log space), based on 25 events in the database with adequate data availability. Figure 4 compares the M values calculated using Equation 4 with observed values, with an inset showing a box and whisker plot of the predicted to observed ratios (interquartile range of 0.5 to 2.2).

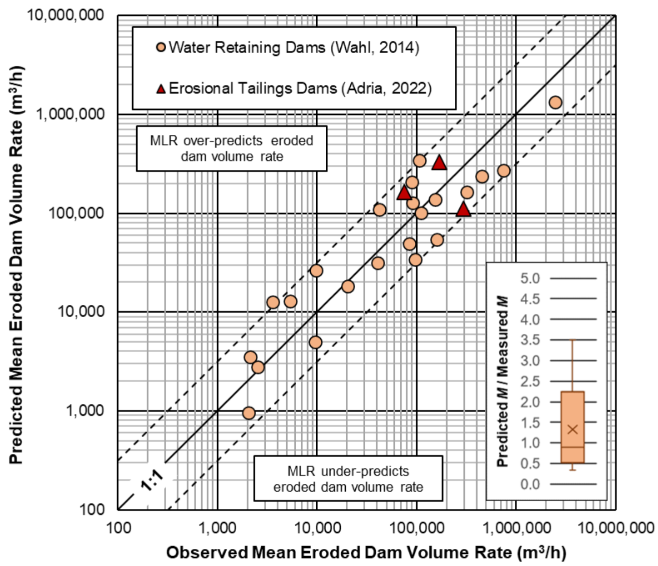

Figure 4: Comparison of observed and predicted mean eroded dam volume rate for Equation 4.

The eroded dam volume rate would vary during a breach, with higher rates during the peak outflow and lower rates during the rising and receding limbs of the breach hydrograph. The value predicted by Equation 4 represents the average or mean rate. The mean eroded dam volume rate itself cannot be used as an input for most dam breach hydrograph modelling approaches, but needs to be combined with an estimate of the eroded dam volume to inform the breach formation time. The eroded dam volume can be estimated using Equation 3, or other regression equations for breach parameters (e.g., Froehlich, 2008; Xu and Zhang, 2009) in conjunction with Equation 3. The primary advantage of estimating the mean eroded dam volume rate is to support the selection of both the breach geometry and the breach formation time.

3.2.1 Refining Formation Times for Sensitivity Scenarios

Example 3: The first case for using Equation 4 is to support sensitivity or uncertainty analysis. A larger breach, faster breach, or a combination of a larger and a faster breach could be assessed; however, these scenarios could result in physically impossible and extreme breach outflows. By using Equation 4 and the scatter in Figure 4, an upper bound on the mean eroded dam volume rate can be determined for the sensitivity scenarios. Using the same narrow dam from Section 3.1.1, a breach bottom width and breach formation time are estimated for the base case using Froehlich (2008), while the values selected for the sensitivity scenarios (wide or fast breach) are based on the results from Froehlich (2008) adjusted using the error guidance for these parameters from Wahl (2004). The selected breach side slope for this example is 1H:1V. The example results are shown in Table 3, where the V_{ED} and M values are determined based on the dam geometry and the breach width and breach formation time estimated using Froehlich (2008).

Table 3: Mean eroded dam volume rates for sensitivity scenarios for a narrow dam

Parameter	Base Case	Wide Breach	Fast Breach	Wide and Fast Breach
Average Breach Width (m)	100	200	100	200
Bottom Breach Width (m)	50	150	50	150
Breach Formation Time (h)	0.80	0.80	0.40	0.40
Eroded Dam Volume (m^3)	338,000	738,000	338,000	738,000
Mean Eroded Dam Volume Rate (m^3/h)	421,000	922,000	844,000	1,844,000

Equation 4 predicts a mean eroded dam volume rate of 456,000 m^3/h. The maximum error for Equation 4 is approximately 0.5 orders of magnitude, therefore the maximum and minimum expected *M* values would be 1,444,000 m^3/h and 144,000 m^3/h, respectively. The base case scenario has an *M* value close to the predicted value from Equation 4. The wide breach and fast breach scenarios have an *M* that is near the upper limit that may be expected for the example dam, while the wide and fast breach scenario has an *M* that is higher than may be physically possible, despite the relatively reasonable breach width and formation time used. A practitioner may ultimately choose more conservative inputs than what is estimated from any one regression equation, however, the combination of these conservative choices should remain within reason, and this form of comparison can support reasonable selections.

3.2.1 Refining Formation Times for Constrained Breach Geometries

For some conditions, the breach geometry might be physically constrained. This may occur for cross-valley embankments, for small saddle dams of large reservoirs, or where portions of the embankment include concrete structures that would not be expected to erode. If it is possible for the entire embankment volume to be eroded, the regression equation for eroded dam volume rate, *M*, can be used to support estimates of the formation time. The majority of the breach events used to develop the current, as well as previous, regression equations did not involve complete removal of dams. These equations then imply that the formation time is not interrupted or limited, and their application may overestimate the formation time for a constrained geometry. By estimating *M* and accounting for this constrained eroded dam volume, the breach formation time can be more appropriately selected for scenarios of this type.

Example 4: Table 4 shows an example of a constrained breach scenario, where the breach width was selected to correspond to a full removal of a relatively small saddle dam in consideration of a large outflow volume. The crest width and upstream and downstream dam slopes are the same as in the narrow dam in Example 1, but the outflow volume, heights, and breach width are adjusted to correspond to the hypothetical saddle dam scenario. The breach side slopes are 1H:1V and the breach bottom width is 1 m. The predicted formation time using the regression equation of Froehlich (2008) is nearly 1 h. In comparison, using the eroded dam volume and Equation 4 results in a formation time of 0.3 h, which could be expected for a comparatively large outflow volume eroding a small dam. Based on the general scatter represented with the dashed lines on Figure 4, the breach formation time could be as fast as 0.1 h or as slow as 1.0 h.

Table 4: Constrained eroded dam volume

Parameter	Small Dam
Total Outflow Volume (m^3)	3,000,000
Height of Water and Breach Height (m)	8 and 10
Crest Width (m)	5
Upstream and Downstream Slopes (xH:1V)	1.5 and 1.5
Breach Formation Time using Froehlich (2008) (h)	0.97
Constrained Eroded Dam Volume (m^3)	3,500
Mean Eroded Dam Volume Rate (Equation 4) (m^3/h)	11,400
Breach Formation Time estimated with V_{ED} / M (h)	0.31

4 SENSITIVITY ANALYSIS

Probabilistic breach modelling (e.g., Goodell et al., 2018) is a form of sensitivity analysis in which peak outflows are associated with a probability, given a breach occurs. A common approach for breach hydrograph modelling uses Monte Carlo analysis (MCA), which is available in HEC-HMS or the HEC-RAS add-on called McBreach (Goodell et al., 2018). The advantage of MCA is that the level of conservativeness of the combination of inputs is quantified through the probability of the resulting outflow. The disadvantages are that the dam breach professional needs to select a reasonable distribution and range for each parameter, and that the added information produced in the results requires more effort and thought to properly use the MCA. With inappropriate parameter ranges and distributions in MCA, the hazard and risk might be mischaracterized and hidden by the perceived “sophistication” of the MCA. The following considerations are broadly applicable to most forms of sensitivity analysis, however, specific considerations for Monte Carlo modelling are highlighted.

4.1 Determining Ranges for Inputs in MCA Sensitivity Analysis

The first consideration relates to using a range for breach parameters (e.g., width or formation time) that is based on the results from empirical equations to guide the range selection in the MCA. Direct results from regression equations can inform some of the ranges for sensitivity analysis, as shown in the examples discussed above, but the context of the regression equations needs to be understood for them to be used appropriately. Some equations represent the best fit and result in an average value for the output parameter given the input variables (e.g., Equations 3 and 4 presented in this paper). These equations would be best suited for the most likely value for MCA. Best fit equations can produce a wide range of different results (Martin and Akkerman, 2017), however, they still do not represent the full variability in observed dam breach parameters. Other equations are envelope equations that represent the worst case or upper value, recognizing that some of the older envelope equations were shown to not be conservative when applied to a larger database (Wahl, 1998), and therefore cannot be solely relied on for upper limits.

Example 5: Figure 5 shows the predicted versus observed formation times using MacDonald and Langridge-Monopolis (1984), Froehlich (2008), and Equations 3 and 4 from this paper (i.e., using the estimated eroded dam volume and the mean eroded dam volume rate to estimate the formation time), as well as the associated box plots for the ratios of predicted to observed values. Note that MacDonald and Langridge-Monopolis was originally developed as a lower envelope equation (to give fast formation times), but it is compared against the best-fit equations to demonstrate several of the comments in this paper. Furthermore, the formation time from both MacDonald and Langridge-Monopolis (1984) and Equations 3 and 4 use predicted values (i.e., the eroded dam volume) as inputs for the formation time and thus have a compounding error.

The observed and predicted formation times for the 2005 Taum Sauk pump storage reservoir failure in the USA are highlighted with a burgundy X on Figure 5 as an example. The observed formation time was 0.33 h (FERC, 2006), and the predicted values from the three approaches are 0.43 h (Froehlich, 2008), 0.65 h (Equations 3 and 4), and 0.95 h (MacDonald and Langridge-Monopolis, 1984), with an average formation time of 0.7 h between the three equations. The black dashed lines represent half an order of magnitude above and below the 1:1 line, while the vertical coloured dashed lines approximately represent the interquartile range from the box plots at the predicted value for each equation for Taum Sauk.

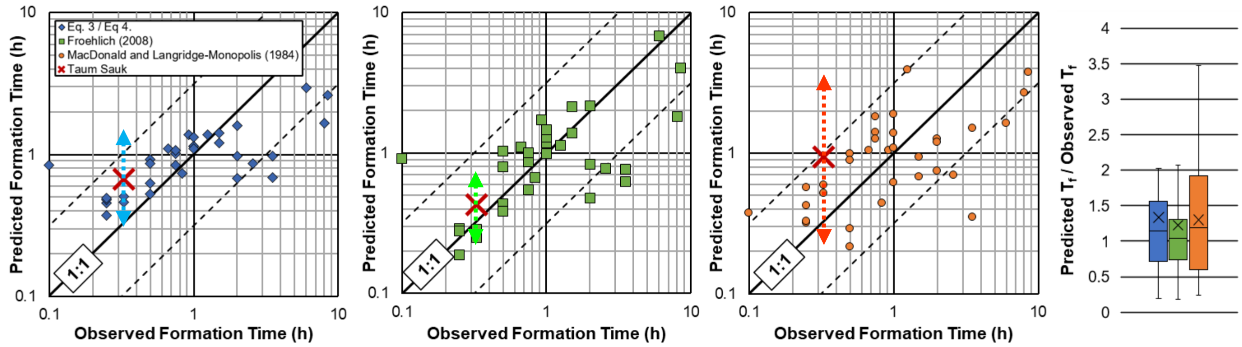

Figure 5: Comparison of predicted versus observed formation times for several equations

The following observations are drawn from Figure 5:

- The approaches with compounding error (i.e., Equations 3 and 4, and MacDonald and Langridge-Monopolis) have a wider interquartile range in the boxplot, compared to Froehlich.
- Equations 3 and 4 have a smaller interquartile range than MacDonald and Langridge-Monopolis, due to the MLR approach and the additional events used to develop these equations.
- Despite its origins as an envelope equation, MacDonald and Langridge-Monopolis predicts the slowest (least conservative) breach formation time.
- All three equations over-estimated the observed formation time of 0.33 h, indicating that the datasets used to develop those equations may not adequately represent the site-specific conditions at Taum Sauk.
- The range in the estimated formation time between the three equations is fairly narrow from 0.43 h to 0.95 h. It is noted, however, that this range only reveals the uncertainty in estimates between these equations, but it does not account for the error within each individual equation (i.e., the coloured dashed lines).

Accounting for the uncertainty when estimating the breach parameters requires consideration of the full range including the equation error, which is visually represented with coloured dashed lines on Figure 5. The full range of the potential formation times for Taum Sauk is perhaps better described as 0.25 h to 3.61 h when using these three equations (i.e., the full range of coloured dashed lines). This larger range should be used in the probabilistic analysis rather than a range developed based on the results of the three equations. It can potentially be refined, but only through careful consideration of site-specific conditions.

4.2 Site-Specific Sensitivity Analysis and Distributions for MCA

The second consideration relates to defining the statistical distribution for the parameters evaluated in the MCA. Methods to approximate a normal distribution are included in Goodell et al. (2018), and some distributions for various parameters are provided in da Silva and Eleutério (2023) based on compiled breach parameters for historic failures. Those recommendations can be treated as an initial estimate of the distribution, which can be refined based on site-specific conditions. The database in da Silva and Eleutério (2023) includes different types of water retaining earthfill dams of varying heights and reservoir volumes. Consequently, these distributions primarily relate to the question: given a hundred different dams and their singular failures, what is the probability distribution for the breach parameters? The question to ask, however, when selecting the distributions for MCA is: given a hundred failures of a singular dam, what is the probability distribution for the breach parameters? The difference is subtle, but important.

Taum Sauk dam experienced substantial overtopping and a failure of a parapet wall prior to the erosional breach (FERC, 2006). The sudden, sustained, and deep overtopping from this parapet failure explains why this dam may have failed faster than would be expected. The 2010 failure of the MAL Reservoir X TSF in Hungary (the Ajka event) involved both erosional and non-erosional breach processes (Adria, 2022). The downstream constructed embankment was built with fly ash on a deficient foundation (Bánvölgyi, 2018). Fly ash produces light bonding and responds in a brittle manner to deflection (Turi et al., 2013). The foundation settlement resulted in the release of the supernatant pond eroding through the dam (erosional breach process) combined with partial block collapse of the dam (non-erosional breach process) that only took 0.25 h to fail (M ≈ 300,000 m^3/h), which is relatively fast for its height and outflow volume.

These two events demonstrate that site-specific conditions (in these cases, minor to moderate aspects of non-erosional breach processes involved during the breach of the dam) strongly affect the breach parameters and their distributions. It is unlikely that in thousands of hypothetical parapet overtopping scenarios (i.e., a MCA simulation conducted specifically for Taum Sauk), the non-erosional breach process would not influence rapid erosion and breach development, and as such, the distribution should reflect this. An example distribution for this scenario could be a uniform distribution between 0.05 h and 0.55 h. The uniform distribution may not be as sophisticated as the other distributions, but is more appropriate in some cases. It is noted, however, that the knowledge about the parapet wall failure playing an important role in the breach development is a hindsight.

MCA analysis and dam breach studies, as a whole, are based on assuming occurrence of unlikely, but possible events. Consequently, it is not an inconceivable assumption that a structural feature like a parapet wall may fail, which is supported with case histories of such occurrences resulting in rapid erosion. Conservatively lowering the minimum value for the formation time range in MCA is justified, particularly if partial non-erosional breach processes can develop. Identifying appropriate non-erosional breach processes can be challenging, particularly for tailings dams where there could be uncertainties related to foundation conditions and tailings characteristics. Failure modes and effects analysis (FMEA), if available prior to the dam breach study, could be used to support the site-specific sensitivity analysis. If no failure conditions are identified that may affect the breach parameter distributions, then wide gamma or normal distributions could be adopted to represent the full extent of scenarios, noting that these distributions point to the most likely outcome based on their definition. An alternative is to use the uniform distribution, which indicates that any value within the range could represent an equally likely outcome.

4.3 Other Considerations

The parameter that produces the highest peak outflow at the breach location is not necessarily associated with the highest sensitivity in the downstream environment for the elements at risk. Adria et al. (2022) showed that the breach outflow hydrograph for the HEC-RAS back-analysis of the 1994 Merriespruit tailings dam failure event was more sensitive to the breach width than the outflow volume, but the impacts to the bird sanctuary 2 km downstream were more sensitive to the outflow volume (e.g., final depth, volume reaching, extent). Ghahramani et al. (2022) found similar results in a benchmarking exercise where two different tailings dam failures (the 1994 Merriespruit and the 1985 Stava events) were modelled using four different models (DAN3D, MADflow, FLO-2D, and FLOW-3D). These examples illustrate that sensitivity analysis needs to be tailored to the facility, while also considering the downstream elements at risk.

For many pumped storage hydro schemes, the upper reservoirs are ring-dyke style impoundments on top of ridges or hills. Likewise, the tailings from mines located in relatively flat regions are usually impounded in ring-dykes due to the lack of confining topography that could be used for cross-valley impoundments. Ring-dyke facilities often require assessment of several breach locations including the associated sensitivity analysis, as the dam height, downstream drainage patterns, and downstream elements at risk may differ substantially from one another.

Conventional MCA used for developing breach outflow hydrographs generally requires thousands of simulations to reach statistical convergence (Goodell et al., 2018). The required number of simulations to reach a statistical convergence can be evaluated by plotting the cumulative mean and standard deviation of the outflow. In our experience, few thousand simulations may be required when a full range of possible outcomes is used for the various breach parameters.

Downstream topographic constrictions located close to a breach location may choke the breach outflows and form additional hydraulic controls that govern the breach development. Intuitively, this would suggest an erosional breach should be slower and narrower than a hypothetical breach without a downstream constriction, all else being equal. Unfortunately, there is little in the case studies or other guidance to quantitatively support how much slower or smaller the breach should be. A physically based breach model (e.g., EMBREA, XBeach, or HEC-RAS with enabled 2D sediment transport modelling) coupled with the downstream flood wave model could be employed for this situation, which involves additional complexities.

Lastly, rheological parameters such as yield stress and viscosity for tailings flows were demonstrated to have substantial variability, both for measured values (Martin et al., 2022) and back-calculated values (Ghahramani et al., 2022; Adria, 2022). With regards to the outflow at the breach location, there may not be enough distance or time for the rheology values to materially affect the flow given the extreme conditions and shear stresses that develop in a violent breach event, as found in Ghahramani et al. (2022). Rheology values are recommended to be tested in sensitivity analysis for the downstream runout or flood wave modelling process (CDA, 2021); however, the impact of rheology on outflow volumes and breach formation may also need to be considered, depending on the modelling software and site conditions.

4.4 Comparisons to Case Histories

Considering the numerous uncertainties with each input parameter, it is beneficial to compare the final selection of breach parameters against case histories to assess whether the selections are reasonable and sufficiently conservative. This is applicable regardless of the breach modelling tools, or whether simple sensitivity analysis or MCA were used in the selection process. For tailings dams, the outflow volume can be compared to the impounded volume, as the volume of mobilized tailings represents a critical element in such studies. This comparison is shown on Figure 6, which is adapted from Rana et al. (2021a and 2021b).

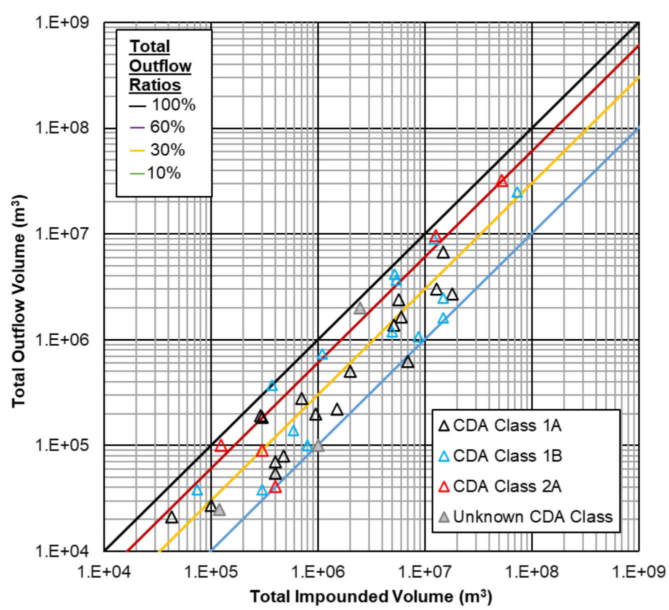

Figure 6: Total outflow ratios for past tailings dam failures (adapted from Rana et al., 2021a and 2021b)

Other types of comparisons shown on Figure 7 include scatter plots of breach heights, average breach widths, eroded dam volumes, and breach formation times from past failures, as compiled by Wahl (1998 and 2014), Rana et al., (2021b), and Adria (2022). Coloured lines that represent relevant ratios are included on these plots for reference.

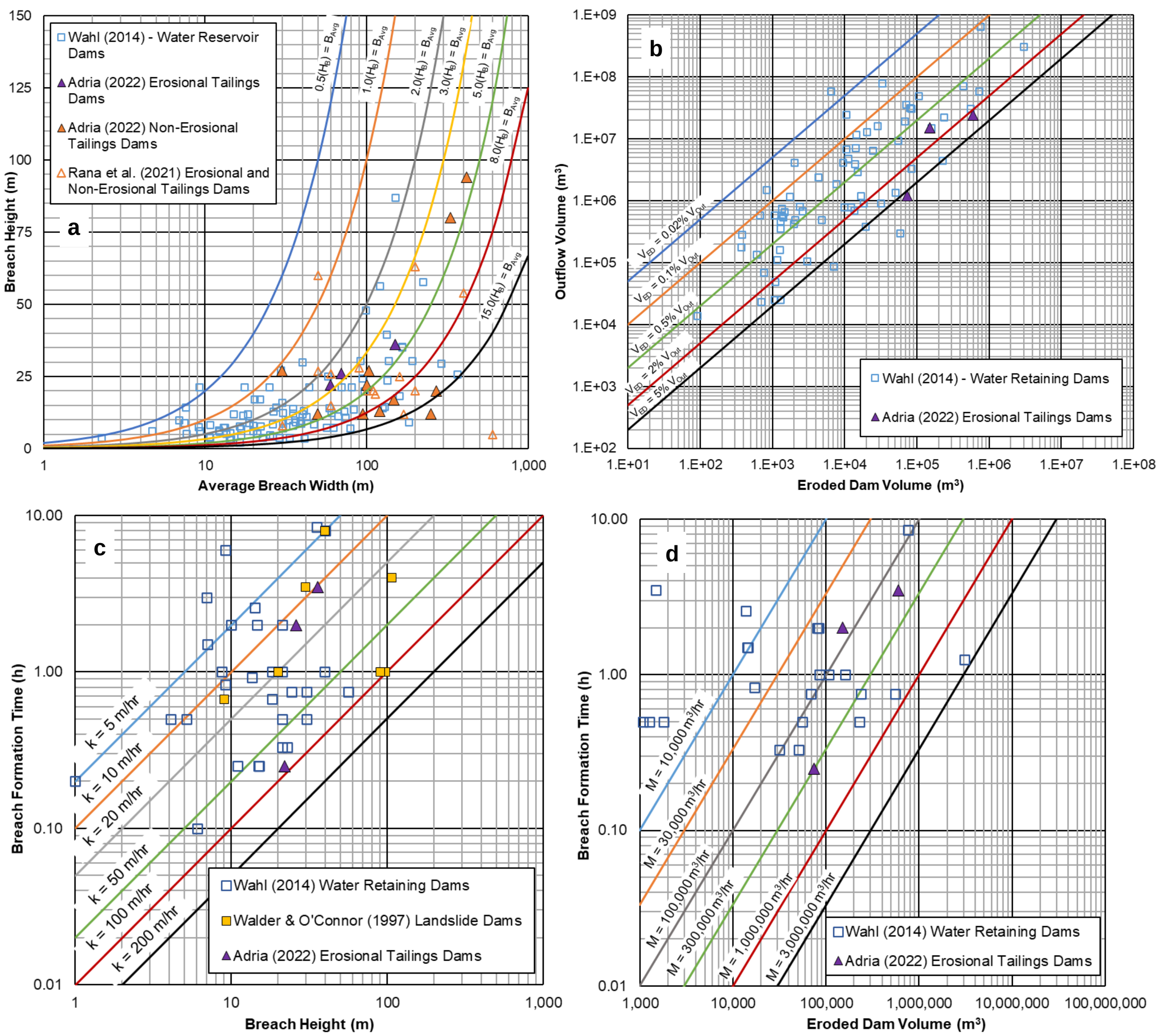

Figure 7: Data from past failures: a) Breach height to breach width ratios (adapted from Wahl, 1998); b) Outflow volume to eroded dam volume ratios; c) Mean erosion rates; and d) Mean eroded dam volume rates

In forward-analysis, if the conclusion after the breach analysis is that the dam could have a non-erosional failure mechanism, or if site-specific conditions indicate a breach may be worse than predicted using the typical outcomes from databases (e.g., the formation time example for Taum Sauk), the selected values should plot closer to the highest ratios shown with the black lines. Conversely, a slow breach scenario for a broad dam constructed of erosion-resistant materials should plot closer to the lower ratios shown with blue lines. When the selected values do not follow these trends, further examination and justification may be required.

5 CONCLUSIONS

This paper highlights the various considerations and data sources that inform the selection of breach parameters, including using sensitivity analysis to support development of conservative but realistic breach outflows for downstream runout or flood wave modelling. The processes that occur during a breach that need to be understood to prevent the application of case studies or empirical equations to non-analogous forward-analysis are discussed. A novel set of empirical equations is presented that estimates the eroded dam volume and the eroded dam volume rate, which can provide additional tools for breach parameter selection, particularly for dam geometries that are outside of the typical conditions in existing dam failure databases. Insights regarding Monte Carlo type probabilistic analyses and sensitivity analyses in general are shared, in consideration of empirical equations and the available dam failure databases. Lastly, a set of figures is provided that can be used by dam breach practitioners for a ”reality check” of estimated breach parameters in forward-analysis type studies.

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