TAILINGS AND MINE WASTE 2015

PROCEEDINGS OF TAILINGS AND MINE WASTE 2015 CONFERENCE

Colorado State University

UBC NORMAN B. KEEVIL INSTITUTE OF MINING ENGINEERING THE UNIVERSITY OF BRITISH COLUMBIA

UNIVERSITY OF ALBERTA

Challenges with conducting tailings dam breach studies

Violeta Martin, Ph.D., P.Eng, Daniel Fontaine, P.Eng, and Jaime Cathcart, Ph.D., P.Eng
Knight Piésold Ltd., Vancouver, BC, Canada

ABSTRACT

Engineering practitioners currently use a combination of guidance documents, published papers, and professional experience to support tailings dam breach and inundation studies. There is considerable knowledge and information available in this relatively new field of practice, but standardized and comprehensive guidelines specific to tailings dams are not available. Practitioners face several key challenges: (a) the more established and mature guidance is typically focused on water storage dams; (b) the available literature may only address specific aspects of dam breach assessment and inundation modelling; (c) the available literature sometimes relies on historical data that may not be consistent with the proposed application; and (d) the results are highly sensitive to the selected modelling inputs. This paper reflects on the currently available literature on dam breach and inundation studies, compares how the major components for these studies are approached, and discusses how the resulting conclusions may vary. Key aspects of tailings dam breach and inundation assessment typically considered include: initial conditions and hydrology; boundary conditions; breach parameters; tailings mobility and outflow volume; and flood routing and inundation mapping. This paper quantitatively compares several of the key parameters proposed by various authors and comments on the factors that control their applicability.

Key words: tailings, dam breach, modelling, breach parameters

1 INTRODUCTION

Tailings dam breach studies are becoming generally expected and often required for operating and planned Tailings Storage Facilities (TSFs). The requirements to conduct these studies increased in British Columbia and Canada following the Mount Polley dam breach accident in August of 2014. The guidelines that are typically followed by practitioners around the world for tailings dam breach analyses were originally developed for water retaining dams, and as such, are not fully applicable to tailings dams. The recently issued Technical Bulletin *Application of Dam Safety Guidelines to Mining Dams* by the Canadian Dam Association (CDA) does not prescribe procedures for conducting tailings dam breach analyses, but is rather limited to identifying "some specific issues that should be considered during the design and safety evaluation of mining dams" (CDA 2014).

Tailings dam breach studies are being requested for various purposes listed below. The suitability of these studies for such purposes is not reflected on, or discussed in this paper.

1. Determine the dam hazard classification based on potential incremental consequences of failure to socio-economic and environmental values
2. Preparation of inundation maps in support of emergency preparedness and response planning
3. Part of the environmental studies for a proposed new project or expansion of an existing project
4. Part of alternatives assessments to evaluate various TSF locations or alternative tailings storage technologies

Dam breach studies include characterization of a hypothetical dam breach, flood wave routing, inundation mapping, and evaluation of impacts to socio-economic and environmental values. Standardized procedures and guidelines for conducting tailings dam breach studies do not exist at this time in Canada, the US, or other developed jurisdictions. Mining and dam safety practitioners are making efforts to standardize the procedure and are struggling with the practicalities of conducting meaningful assessments when every step of the process is inherently uncertain.

A range of methods may be applied in each step of the analysis depending on the extent of information available and the level of accuracy needed. CDA "Dam Safety Guidelines" (2007a, revised 2013) suggest that a preliminary assessment be done using simple and conservative procedures to obtain a first approximation of the level of consequences. These guidelines further recommend that complexity and accuracy be increased if a greater level of detail is needed to confirm the dam classification, support a detailed emergency response plan, or to inform risk management decisions. This level of assessment would typically involve modelling the breach process itself and modelling the flood wave propagation downstream. A wide range of options is left open to practitioners in terms of how to conduct the assessment. The most appropriate approach and the accuracy of the results that can be achieved will be driven by the availability, resolution and accuracy of the input data. Increased complexity does not always result in increased accuracy, and as such, complexity should not be an objective, but rather a tool to be used carefully. Considering all this, a preliminary high level assessment may very well provide a sufficient level of detail to meet some of the stated purposes of a dam breach study.

Tailings dam breach studies in Canada are generally required to follow the CDA guidelines (CDA 2007a), but there are several other guidelines and published literature that are also commonly used in these studies. The CDA Technical Bulletin *Inundation, Consequences and Classification for Dam Safety* (2007b) recommends that Chapter 2 of the Federal Energy Regulatory Commission (FERC) *Engineering Guidelines for the Evaluation of Hydropower Projects* (1993, revised 2015) be used for estimating the embankment dam breach parameters and the peak outflow discharge. These guidelines are based on the work of Fread (1981) and Macdonald & Langridge-Monopolis (1984), and are also summarized in the Dam Safety Guidelines of the Washington State Department of Ecology (Barker & Schaefer 2007). Works by Froehlich (1995a, b), Von Thun & Gillette (1990), Wahl (1998, 2004), Pierce (2010), Costa (1985), and Walder & O'Connor (1997)

are also often used to predict peak discharges or breach parameters; however, all these are based on water retaining dams, or landslide and other naturally formed dams (as in Costa 1985).

A frequently cited reference specific to tailings dam breaches is by Rico et al. (2007) that presents empirical relationships for outflow volume, runout distance and peak discharge based on past tailings dam failures. Even though there are over 200 known cases of tailings dam failures worldwide, the documentation of these cases is often incomplete. Relationships developed by Rico et al. (2007) are based on 28 historic tailings dam failures for which complete data on runout distances and outflow volumes were available. The peak discharge relationship presented in their paper is based on water retaining and naturally formed dams from Costa (1985), as peak discharge information was available only for three tailings dam cases.

The empirical data base of Rico et al. (2007) was further expanded through the work of a CDA technical committee for tailings dam breach analysis, which was presented in October 2015 during the 2nd CDA Workshop on Tailings Dam Breach Analysis – Advancing the State of Practice. Additional information specific to tailings dams is available in a paper by Clemente et al. (2013), which summarized the findings of the workshop organized by the Tailings Dams Committee of the U.S. Society on Dams (USSD) in 2011.

This paper is intended as a starting point for ongoing discussion about what approaches and methods are best suited to specific aspects of tailings dam breach and inundation studies. Some practical tools are proposed with the objective of providing practitioners with information that will allow them to use the available literature and avoid potential difficulties that have been identified through our experience. An example is reviewed relating to the predicted range of outflow volumes and peak discharges. The paper explores how the in situ tailings properties, combined with the hydrological conditions, can physically constrain the range of possible outflow volumes. Peak discharge depends on the outflow volume, physical characteristics of the breach (height, width, and side slopes), and an estimate of the time for breach formation. The calculation and analysis methods that are presented in the available literature can result in a broad range of hypothetical values, some of which are outside of the physical constraints for a given tailings dam.

2 MODES OF DAM FAILURE

A dam is defined as a barrier constructed for the retention of water, water containing other substances, fluid waste, or tailings (CDA 2007a). Furthermore, if a breach of the perimeter containment can trigger liquefaction and flow of the contents of the stack beyond the perimeter containment, then that perimeter containment must be considered as a dam, even if there is no pond with visible water on the surface of the stack (CDA 2014). It should be noted, however, that modelling a breach of a dam that retains both water and tailings is considerably different from modelling a breach of a facility with very little or no water stored. These different processes are described in more detail in this section.

There are three different potential dam failure modes based on how a component failure occurs to cause loss of the system function (CDA 2007a):

- *Overtopping* – Water flows over the crest of the dam contrary to the design intent.
- *Collapse* – Internal resistance to the applied forces is inadequate.
- *Contaminated seepage* – Contaminated fluid escapes to the natural environment.

Only failures due to *Overtopping* and *Collapse* that lead to catastrophic dam breach are considered in this paper.

CDA guidelines (2007a, b) recommend that dam breach evaluations consider two initial hydrologic conditions:

- *Sunny day failure* – "a sudden dam failure that occurs during normal operations", which may be "caused by internal erosion, piping, earthquakes, mis-operation leading to overtopping, or another event".
- *Flood induced or rainy day failure* – "a dam failure resulting from a natural flood of a magnitude that is greater than what the dam can safely pass".

These initial hydrologic conditions apply to both the impoundment levels and to the flow conditions at the facility and in the downstream drainage network that will be impacted by the passage of the breach flood wave. The impoundment levels for both scenarios and the flows in the downstream channels are defined by the CDA (2007a) for the sunny day scenario, but the choice of downstream flows for the flood induced scenario is left to the engineer conducting the study. In addition, a sunny day failure is typically modelled as occurring due to collapse of the dam rather than overtopping, while the flood induced failure is modelled to occur due to overtopping.

One of the main differences between a water retaining dam failure and a tailings dam failure is in the outflow volume and the solids content in that volume. A breach of a water retaining structure typically results in the discharge of the entire impounded volume of water above the breach, which has a relatively low solids content. A breach of a tailings retaining structure, in contrast, could result in the discharge of the entire supernatant pond volume, but does not have to result in the full discharge of the impounded tailings volume. Furthermore, for TSFs that have a supernatant pond, a tailings dam breach typically results in two discharge mechanisms: (a) an *initial flood wave*, and (b) slumping or *flow of liquefied tailings*. These mechanisms occur in sequence following a catastrophic failure of a TSF dam for all cases where there is a supernatant pond present, and are distinctly different in terms of the potential risk they pose to the downstream environment.

An *initial flood wave* would occur immediately following the failure of a TSF dam. The free water within the TSF supernatant pond would start to discharge and mobilize both tailings from the impoundment and construction materials from the TSF dam. The flood wave would propagate downstream causing significant erosion and inundating the downstream receiving environment (Fig. 1a). The extents of inundation are largely driven by the size of the impoundment, the rate of breach development, the peak discharge and the downstream topography, with consequences potentially extending downstream for tens or hundreds of kilometres. The flood wave would carry

the tailings solids and dam construction materials, as well as the material eroded along the way, resulting in high solids concentrations. Some of the coarser material carried with the flood wave could deposit along the way, while the fine sized tailings would be carried in suspension until the flow velocities are low enough for this material to settle (e.g. in a lake or ocean).

This discharge mechanism is the main focus of most dam breach analyses and inundation studies because it poses the greatest risk to life safety and has the greatest potential to cause physical damage to the downstream environment. The initial flood wave is often modelled as water, or a Newtonian fluid, which is a simplifying assumption that results in conservatively high flow velocities and discharge rates, and longer runout distances. Heavily sediment laden water is a non-Newtonian fluid, but modelling this type of fluid requires knowledge of the rheology (viscosity and yield stress), which is dynamic and impractical to predict. Furthermore, a non-Newtonian fluid option is not available for some of the flood routing software. It should be noted, however, that research conducted for a hypothetical breach using water and a non-Newtonian fluid has shown that the differences in peak discharge at various points downstream are within 5%, while the differences in depth are on the order of 10% (Bernedo et al. 2011). This finding was specific to the study and further work is needed in this area; however, considering that the shear stress imposed by the passage of the flood is orders of magnitude larger than the yield stress of the tailings slurry, this result is perhaps not so surprising. The errors associated with assuming that the released fluid flows like water is well within the modelling accuracy, considering the uncertainty inherent in all the assumptions and simplifications required in the modelling process, and potentially there may be little to be gained by using a non-Newtonian model; however, more research is needed in this area.

The *flow of liquefied tailings,* or the second discharge mechanism, would occur following the initial flood wave, as some portion of the tailings mass would be expected to undergo static liquefaction resulting from the loss of confinement and the local steepening of slopes created by the initial discharge. Tailings that are not mobilized in the initial flood wave may slump through the breach in a paste-like fashion until the tailings mass stabilizes downstream (Fig. 1b). Similarly, for facilities with little or no free water stored, the flow of liquefied tailings would be the only consequence of a loss of confinement.

Figure 1: (a) Erosion caused by the initial flood wave following the Mount Polley dam failure (MPMC, 2015); and (b) Cone of depresion of slumped liquefied tailings caused by the Kolontár dam failure (Oszvald, 2015)

Flow of liquefied tailings cannot be modelled as water. The viscosity and the yield stress of liquefied tailings play a significant role in the flow, which would behave like Bingham plastic fluid (Jeyapalan et al. 1983; Seddon 2010; Kulesza 2011). Some historic cases indicate that the cone of depression of the tailings mass remaining in the TSF forms at angles of 3.5˚ to 6˚ (Fig. 1b), while the liquefied saturated tailings stabilize downstream at slopes of 1˚ to 4˚ (Lucia et al. 1981, Blight & Fourie 2003), with the deposition of the slumped tailings largely depending on the tailings rheology, downstream topography and stream/valley slopes. The inundation extent from the flow of liquefied tailings would be considerably less than from the initial flood wave, but the slump would deposit far more solids in the first few kilometres downstream of the breach location.

3 CHALLENGES WITH TAILINGS DAM BREACH ASSESSMENTS

Tailings dam breach practitioners are faced with numerous uncertainties when making assumptions and simplifications necessary to conduct these analyses. Some specific challenges related to determining the initial hydrologic conditions and the breach characteristics are discussed in this section.

3.1 Determining Initial Hydrologic Conditions

3.1.1 Water Levels in the Impoundment

The water levels in the impoundment are defined to be (CDA 2007b):

- The normal operating pond level for a sunny day scenario, and
- A water level that would cause overtopping in a flood induced scenario, as the failure is due to a natural flood of a magnitude that is greater than what the dam can safely contain or pass.

The flood levels are not always simple to define for several reasons. TSFs are typically constructed and raised throughout operations during the life of a mine. The design basis for flood management is often to store the inflow design flood (IDF), unlike water dams where the IDF is safely passed through a spillway. Passing a flood through an active tailings facility during operations is not common practice due to the potential environmental and societal concerns. Therefore, a TSF may not have a spillway until closure.

TSFs are often designed with sufficient capacity above the maximum operating water level and can store the IDF in addition to freeboard for wave run-up. In many cases, especially in recent practice, the PMF has been used as the IDF. The PMF, which would be caused by the probable maximum precipitation event (PMP), is considered to be the theoretical maximum flood that could plausibly occur at a particular time of year in a design watershed (WMO 2009). However, there are often no clear guidelines as to what duration PMP event should be used for the PMF determination, with PMP durations ranging from 6 hours to 72 hours, but events as long as 30 days have been found in practice. The duration of the PMP can have a dramatic impact on the volume of the storm inflow and consequently on the magnitude of the dam breach outflow.

Breaching of a dam is typically assumed to start at the peak of the inflow hydrograph for the flood event that would cause overtopping. In some instances, the starting water level required to simulate overtopping in conjunction with a PMF needs to be specified above what can reasonably

occur in the facility. In facilities designed with very large storage capacities, the additional volume that needs to be incorporated into the model may require consecutive occurrence of more than one PMF, which is theoretically impossible. Consequently, facilities that have additional storage to prevent overtopping end up with unrealistically large outflow volumes in the flood induced overtopping scenario, resulting in unrealistically high incremental impacts. In such cases, overtopping of the facility may not represent a credible failure scenario. Unrealistic results hold no value to the mine owner or other stakeholders, and provide little insight into the actual risk posed by the facility.

3.1.2 Hydrologic Conditions in the Downstream Drainage Network

A sunny day failure scenario is modelled to be coincident with mean annual flow conditions in the downstream channels (CDA 2007b). A flood induced failure is modelled to be coincident with a flood of a magnitude that is greater than the IDF for the facility, where the IDF is defined based on the dam classification. If the IDF is equivalent to the PMF, then the failure of the facility must be modelled under the PMF conditions. In such cases, pre-breach flood flows in the channels immediately downstream of the facility must be simulated for PMF conditions and then prorated with a distance from the facility depending on the size of the model. For models that extend for tens and hundreds of kilometers and through several watersheds, it may not be a reasonable assumption that PMF flows occur in all the watersheds concurrently. It would perhaps be more reasonable to assume that the PMF is occurring in watersheds immediately downstream of the dam and that some other large event (but smaller than the PMF), such as the 200 year flood, is occurring concurrently in more distant watersheds. There are no specific guidelines defining the flood events to be considered for the downstream tributaries and specifying the geographic extent of the applicable floods.

A review of over 30 publically available dam breach studies conducted in British Columbia after the Mount Polley dam breach event revealed that practitioners use flood flows ranging from the 2-year event to the PMF as the baseline condition for downstream water courses. Assuming a small natural flood (e.g. 2-year event), as opposed to a large flood (e.g. 1000-year event) would result in larger incremental consequences but smaller overall inundation during a flood induced scenario. Conversely, a larger natural flood would result in smaller incremental but larger overall inundation.

Additional uncertainty is embedded in estimating an event as large as 1000-years, as confidence in the predicted estimate decreases as the extrapolation is made further beyond the number of years with reliable record. In general, statistical analysis of annual exceedance probabilities for a single station should not extend beyond 2 to 4 times the number of years of record (CDA 2007c). This means that in order to estimate a 1000-year peak flow with reasonable confidence using frequency analysis, the historical flood record would need to be 250 to 500 years long.

***3.2** Determining Breach Characteristics*

The quantitative assessment of the potential consequences of a flood from a TSF dam breach requires estimating the volume of water and tailings released in the breach, the peak outflow discharge, the physical characteristics of the breach (height, width, and side slopes), and an estimate of the time of breach formation. These characteristics are used to develop a dam breach

outflow hydrograph, which is subsequently routed through the downstream drainage network to simulate the inundation limits.

3.2.1 Breach Outflow Volume in the Initial Flood Wave

With water retaining dams, it is reasonable to assume that the entire volume of water above the lowest point of the breach would be discharged when the breach occurred. This is not the case with tailings dams, and a review of past failures reveals that discharge volumes ranging from a few percent to a 100% of the impounded volume are possible (Tailings.info 2015). The considerable range indicated by these data is likely due to various factors including the volume of stored water, the type and volume of stored tailings, the dam height and construction method, the type and size of failure, and the failure trigger.

Rico et al. (2007) developed an empirical relationship (Equation 1) which predicts that approximately 37% of the impounded volume (V_T in Mm^3) comprising supernatant water, tailings solids and interstitial water, is released in the breach outflow volume (V_{OUT} in Mm^3). This approach is quite commonly used by practitioners in Canada to estimate outflow volumes, but may at times result in unrealistic estimates, as discussed below.

$$V_{OUT} = 0.354 \times V_T^{1.01} \quad (1)$$

Another approach is to consider the available free water in the supernatant pond (including concurrent storm water if applicable for the given scenario) that has the potential to mobilize a certain mass of tailings and embankment construction material. This potential mass of solids can be estimated as a function of the free water volume assuming instantaneous mixing at 65% solids content by mass, for example. The assumption of 65% solids is based on the "flowability" of slurry and is considered a conservative upper limit to Newtonian-like fluid behavior. This assumption does not apply to all types of tailings and should be developed on a case by case basis. The tailings deposit at the time of a breach is described by the mass of solids in the deposit, the density of the solids, and the average dry density of the deposit. All of the tailings could discharge during the breach using this approach if the impounded volume of tailings is relatively small with a lot of free water available. Conversely, if the volume of tailings is large relative to the volume of free water, only a small portion of the tailings may be mobilized. This approach is further discussed in a companion paper presented in this conference (Fontaine & Martin (2015).

Table 1 compares the outflow volume estimates in the initial flood wave for a hypothetical TSF with 300 Mt of stored tailings solids using Rico et al. (2007) with values calculated using the available free water approach and assuming instantaneous mixing of 65% solids by mass, a solids density of 2.65 t/m^3, and a tailings dry density of 1.4 t/m^3. The volume of free water is the only variable in this example. It is illustrated that the two approaches in this example converge at a free water volume of 19 Mm^3, and then diverge again. The approach by Rico et al. (2007) when compared with a physically based mixing model appears to over-predict outflows at lower free water volumes and under-predict outflows at higher volumes.

Table 1: Breach Outflow Volume Calculation (values shown in $m^3 \times 10^6$)

Volume of Free Water	Volume of Stored Tailings[1]	Total Impounded Volume	Outflow Volume[1] (Rico et al. 2007)	Outflow Volume[1] (65% Solids)
1	214	215	80 (37%)	5 (2%)
5	214	219	82 (37%)	23 (10%)
10	214	224	84 (37%)	45 (20%)
19	214	233	87 (37%)	87 (37%)
40	214	254	95 (37%)	181 (71%)

1. Volume of stored tailings and outflow volume include interstitial water.

Estimates of the outflow volume were produced using Rico et al. (2007). These volumes were then used to back calculate the percent solids by mass in the predicted outflow. The results indicated that the outflow would contain 74%, 72%, 70%, 65% and 54% solids for a pond volume of 1 Mm^3, 5 Mm^3, 10 Mm^3, 19 Mm^3, and 40 Mm^3, respectively.

In the case of a 5 Mm^3 pond, the Rico et al. (2007) estimate of 82 Mm^3 of outflow is certainly more conservative than the 23 Mm^3 of outflow estimated using the approach of 65% solids mixing by mass. However, the outflow volume of 82 Mm^3 would require instantaneous mixing of 72% solids by mass, which would be more representative of a tailings deposit with a dry density of 1.3 t/m^3 and a moisture content of 39%. The outflow tailings characteristics would only be marginally different from the original tailings stored in the facility with a dry density of 1.4 t/m^3, solids content of 75% and a moisture content of 34%. This is not a realistic scenario, since the sudden mobilization of 5 Mm^3 of water caused by a dam failure would undoubtedly result in an initial flood wave, and would not be restricted to slumping of liquefied tailings. This example demonstrates the importance of considering the actual physical processes that would likely occur and of conducting a critical review of the estimated values.

3.2.2 Peak Discharge for the Initial Flood Wave

Determining the peak discharge during a dam breach is essential for downstream flood wave routing. Various equations have been proposed for estimating the peak discharge, many of which are based on reviews of past water retaining dam failures. However, estimating the peak discharge during the actual dam breach is challenging and there is uncertainty in the data on which the empirical relationships are based. Consequently, the calculated values that are based on these empirical relationships are also uncertain.

Peak discharge values estimated using several published relationships are compared in this paper. These relationships can be grouped into two categories: (1) peak outflow based on a dam factor represented by a product of the outflow volume and the height of water above the bottom of the breach; and (2) peak outflow based on multiple linear regression of these two parameters. The peak discharge values shown in Table 2 were calculated for a hypothetical dam breach with an outflow volume (V_{OUT}) of 23 Mm^3 (tailings solids and water combined) and a dam height above the bottom of the breach (H_W) of 60 m.

This example shows that the equations using the dam factor have a similar exponent and resulted in similar peak discharge values. The only difference is in the Macdonald & Langridge-Monopolis (1984) envelope equation, which resulted in considerably higher values, as expected for an

envelope approach. The equations based on multiple linear regression resulted in approximately 50% to over 100% higher values than the dam factor equations.

If the dam height is doubled to 120 m and the outflow volume is kept the same, the calculated peak discharge values estimated from the three methods start to deviate considerably, as shown in Table 3 (same equations used as shown in Table 2).

Table 2: Peak Outflow Estimates for a 60 m High Dam (values shown in m^3/s)

Methodology	Macdonald[1,2]	Froehlich[3]	Pierce[4,5]	Rico, Costa[6]
Dam Factor (V_w x H_w)	6,682	-	6,025	6,724
Multiple Regression	-	14,378	10,266	-
Envelope Equation	21,828	-	-	-

1. Macodnald & Langridge-Monopolis (1984) dam factor equation (Source: Wahl 2004): $Q_P = 1.154(V_{OUT} * H_w)^{0.412}$

2. Macodnald & Langridge-Monopolis (1984) envelope equation (Source: Wahl 2004): $Q_P = 3.85(V_{OUT} * H_w)^{0.411}$

3. Froehlich (1995a) multiple regression equation: $Q_P = 0.607(V_{OUT}^{0.295} * H_w^{1.24})$

4. Pierce et al. (2010) dam factor equation: $Q_P = 0.0176(V_{OUT} * H_w)^{0.606}$

5. Pierce et al.(2010) multiple regression equation: $Q_P = 0.038(V_{OUT}^{0.475} * H_w^{1.09})$

6. Rico et al. (2007) and Costa (1985) dam factor equation for constructed dams: $Q_{max} = 325[(V_{OUT}*10^{-6}) * H_w]^{0.42}$

Table 3: Peak Outflow Estimates for a 120 m High Dam (values shown in m^3/s)

Methodology	Macdonald[1,2]	Froehlich[3]	Pierce[4,5]	Rico, Costa[6]
Dam Factor (V_w x H_w)	8,890	-	9,170	8,996
Multiple Regression	-	33,960	21,853	-
Envelope Equation	29,022	-	-	-

The larger discrepancy between peak outflows compared to Table 2 is likely due to the range of dam heights, outflow volumes, and peak discharges in the data sets for which the relationships were developed. The highest dam for all five references used is the Teton Dam, USA, reported at 86.9 m above the breach. In all five data sets there are a few dams higher than 30 m, and only one or two higher than 50 m. The highest outflow volume for the water retaining dams is well below 50 Mm^3 in most data sets. There are two cases with outflow volumes larger than 300 Mm^3 (Teton Dam, USA, and Oros Dam, Brazil), but there are very few between 50 Mm^3 and 300 Mm^3 in each of the data sets. In comparison, the Rico et al. (2007) data set, which is based on tailings dam failures, has all outflow volumes below 10 Mm^3. Finally, the peak outflow discharge for all data sets is below 10,000 m^3/s, with only one to three higher peak discharges reported in each study. The highest peak discharge was estimated for the Teton Dam, USA, at about 65,000 m^3/s. Rico et al. (2007) used Costa's (1985) data set for peak discharges estimated for constructed and landslide generated dam failures. Three peak discharge estimates for tailings dam failures were added to this data set, which ranged between 200 m^3/s and 1,400 m^3/s.

Modelling of the breaching process of a tailings dam with a long, flat tailings beach seems to "confuse" the dam breach models. The process of down-cutting through the solids contained in the dam and the adjacent beach must be different than the breaching process for a water retaining dam, which is not yet captured in the existing tools or empirical equations. It is challenging to

select conservative, yet realistic, peak outflow values for downstream routing. Conducting a sensitivity analysis on a range of outflows is a reasonable approach to address some of the uncertainty. Determining what that range of outflows should be, and whether a three- to four-fold increase is realistic is open for discussion.

3.2.3 Breach Dimensions and Time of Failure

Similar to peak discharge empirical relationships, there are numerous relationships developed for the breach parameters (e.g. average width, eroded volume, side slopes) and the time for breach formation. These empirical relationships are based on water retaining dams. Such relationships do not exist for tailings dam breaches, but it is reasonable to expect that there would be differences and that the time for breach formation may be the most affected parameter considering the existence of tailings beaches.

The physical characteristics of the breach and the time for breach formation greatly impact the peak discharge and the outflow hydrograph. This is captured in physically based dam breach models that use hydraulics and sediment transport principles (e.g. BREACH). Peak discharges predicted by empirical relationships do not depend on the breach characteristics, except for the height above the breach. However, the connection between the peak discharge and the breach characteristics is made when developing the outflow hydrographs by matching the peak discharge while keeping within the predicted physical dimensions and breach formation time (e.g. in HEC-HMS or HydroCAD). The breach dimensions and time of failure are also needed as inputs to hydrodynamic models (e.g. HEC-RAS) when the breaching process is simulated that way.

Several proposed relationships are reviewed in this paper for the same hypothetical dam configuration that was used for the peak discharge calculations, i.e. a 120 m high dam and an outflow volume of 23 Mm^3. Additional assumptions include the downstream dam slope of 2H:1V and a dam crest width of 25 m. The results are shown in Table 4.

There is a wide range of values evident in the calculated parameters. The eroded volume varies thirteen-fold, the average breach width twenty two-fold, and the time to failure eight-fold. If the average breach width was 27 m with breach side slopes of 0.5H:1V (as per Macdonald & Langridge-Monopolis 1984), the breach would be of a triangular shape, 54 m high, and would not develop to the bottom of the dam, which is contrary to the common assumption of the breach cutting down to the dam foundation. The breach formation time of 0.27 hours using Froehlich (1995b), is equivalent to a down-cutting rate of greater than 400 m/h. This is much faster than the range of reasonable values of 10 m/h to 100 m/h based on case study data (Walder & O'Connor 1997). The 10 m/h to 100 m/h range of down-cutting rates is consistent with rates back-calculated from other studies (Froehlich 1995b, Macdonald & Langridge-Monopolis 1984).

Table 4: Dam Breach Characteristics for a 120 m High Dam

Methodology	Volume Eroded, V_{er} [1,7] (m^3)	Failure Formation Time, t_F [2,3,4] (hours)	Average Width, B_{avg} [5,6,8] (m)	Side Slope, Zb RATIO H:1V
Macdonald, 1984	469,024	2.08	*27*	0.5
Froehlich, 1995	*2,461,385*	0.27	141	0.9 - 1.4
Von Thun and Gillette, 1990	*6,177,000*	1.80	355	0.33 - 1.0
FERC Guidelines, 1993	-	0.1 - 1.0	120 - 600 (1 to 5 x H)	0.25 - 1.0

1. Macodnald & Langridge-Monopolis (1984) volume eroded equation: $V_{er} = 0.0261*(V_{OUT}*H_w)^{0.769}$
2. Macodnald & Langridge-Monopolis (1984) failure formation time equation: $t_F = 0.0179*V_{ER}^{0.364}$
3. Froehlich (1995b) failure formation time equation: $t_F = 0.00254*V_{OUT}^{0.53} * H_w^{-0.9}$
4. Von Thun& Gillette (1990) failure formation time equation: $t_F = 0.015*H_w$
5. Froehlich (1995b) average width equation (K_0 = 1.4 for overtopping and 0.9 for piping): $B_{avg} = 0.1803*K_0*V_{OUT}^{0.32} * H_b^{0.19}$
6. Von Thun& Gillette (1990) average width equation (C_b = 55): $B_{avg} = 2.5*H_w + C_b$
7. In the absence of an empirical calculation for volume eroded, the average width is used to calculate an approximate volume based on the given dam dimensions (shown in Italics).
8. In the absence of an empirical calculation for average width, the erosion volume is used to calculate an approximate width based on the given dam dimensions (shown in Italics).

Wahl (2004) conducted a quantitative analysis of the uncertainty of predicting the breach parameters and peak outflows of embankment dams using regression based methods. He concluded that the uncertainty is large for all methods, where the breach width is generally within ±1/3 order of magnitude, the failure time is ±1 order of magnitude, and the peak discharge is ±0.5 to 1 order of magnitude, with Froehlich equation (1995a) resulting in the lowest uncertainty of ±0.32. Barker & Schaefer (2007) acknowledge that peak discharge rates computed using the Froehlich equation (1995a) result in the lowest predicted error, but may also result in values substantially higher or lower than the values that may actually occur.

4 CONCLUSIONS

Tailings dam breach modelling is becoming an accepted part of the planning process for operating and proposed TSFs, with various objectives ranging from dam hazard classification to alternatives assessment. In order to conduct a dam breach analysis, a failure mode must be selected, the size of the breach must be approximated, the volumes of released tailings and water must be estimated, and the resulting runout pattern of the released material must be modelled. The considerable uncertainty involved in each step of the analysis, combined with the lack of standardized or mandated approach for completing such an analysis, results in substantial uncertainty in the modelled results. It is important that all dam breach modelling results be viewed in the context of this framework and its limitations, and that clients, regulators and the public be educated in this regard.

Modern tailings storage facilities are designed to contain mine tailings and associated water, often for all conceivable conditions; therefore, it is sometimes difficult to foresee a credible failure mode for properly engineered structures. Furthermore, physical constraints need to be considered and applied where appropriate throughout the process, as was discussed for some of the fundamental parameters including outflow volume and peak discharge. Simply applying an

empirical relationship can result in unrealistic or impossible results. Modelling unrealistically large breaches without due consideration of the physical constraints may produce unrealistically large consequences. Such unrealistic results provide no value to stakeholders and provide little insight into the actual risk and potential impacts posed by the facility.

Even though the mining and dam safety communities are making efforts to standardize the procedures for tailings dam breach studies, conducting meaningful assessments is challenging when there is such a high level of uncertainty throughout the assessment process. In this paper, we reviewed some of our recent experience and discussed challenges faced along the way. Practitioners must rely on good professional judgement and experience when carrying out and presenting their work. Streamlining and standardizing the process of conducting tailings dam breach studies would not necessarily remove the uncertainties and improve on the results significantly, but would help the engineering practitioners conduct these studies in a similar fashion and make the results more comparable. It is the authors' hope that discussions around the dam breach assessment process continue and are perhaps incorporated into future guidance documents.

5 REFERENCES

Barker, B. and M. Schaefer. 2007. Dam Safety Guidelines Technical Note 1: Dam Break Inundation Analysis and Downstream hazard Classification. For the Washington State, Department of Ecology. Olympia, Washington.

Bernedo C.E., P. Julien and A. Leon. 2011. Dam Breach Analysis in Tailings Storage Facilities (TMF). ASCE World Environmental and Water Resources Congress, Palm Springs, California, 2011.

Canadian Dam Association (CDA). 2007a, Revised 2013. Dam Safety Guidelines.

Canadian Dam Association (CDA). 2007b. Technical Bulletin: Inundation, Consequences, and Classification for Dam Safety.

Canadian Dam Association (CDA). 2007c. Technical Bulletin: Hydrotechnical Considerations for Dam Safety.

Canadian Dam Association (CDA). 2014. Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams.

Clemente, J.L.M, R.E. Snow, C. Bernedo, C.L. Strachan, and A. Fourie. 2013. "Dam Break Analysis Applied to Tailings Dams: USSD Workshop Summary and Perspectives". Proceedings of the 33rd Annual USSD Conference. Phoenix, Arizona, February 11-15, 2013.

Costa, J.E. 1985. "Floods from Dam Failures." U.S. Geological Survey Open-File Report No. 85-560," U.S. Geological Survey, Denver, Colorado.

Federal Energy Regulatory Commission (FERC). 1993. Engineering Guidelines for the Evaluation of Hydropower Projects. Accessed in October 2014. http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide.asp

Fell R., and G. Hunter. 2003. "Travel Distance Angle for "Rapid" Landslides in Constructed and Natural Soil Slopes." Canadian Geotechnical Journal, Vol.40: pp 1123-1141.

Fontaine D.D. and V. Martin. 2015. “Tailings Mobilization Estimates for Dam Breach Studies”. Proceedings of the Tailings and Mine Waste 2015 Conference. Vancouver, BC, October 26-28, 2015.

Fread, D. 1988, Revised 1991. “Breach: An Erosion Model for Earthen Dam Failure”. NWS Report, NOAA, Silver Spring, USA.

Froehlich, D.C. 1995a. “Peak Outflow from Breach Embankment Dam,” Journal of Resources Planning and Management, Vol. 121, no. 1, pp 90-97.

Froehlich, D.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, pp 887-891.

Jeyapalan, J.K., J.M. Duncan, and H.B. Seed. 1982. “Analyses of Flow Failures on Mine Tailings Dams.” Journal of Geotechnical Engineering, ASCE, Vol. 109, No. GT2, Feb., pp 150-171.

Kulesza, R. 2011. “Review of Technical Literature on Tailings Dam Failures involving Flow of Tailings.” USSD Workshop on Dam Break Analysis Applied to Tailings Dams. Denver, Colorado, August 24 - 26, 2011.

Lucia, P.C., J.M. Duncan, and H.B. Seed. 1981. “Summary of Research on Case Histories of Flow Failures of Mine Tailings Impoundments,” Mine Waste Disposal Technology, Proceedings of the Bureau of Mines Technology Transfer Workshop, Denver, Colorado, July 16, 1981, pp 46-53.

MacDonald, T.C., and J. Langridge-Monopolis. 1984. “Breaching Characteristics of Dam Failures,” Journal of Hydraulic Engineering, ASCE, Vol. 110, no. 5, pp 567-586.

Mount Polley Mining Corporation (MPMC). 2015. “Post Event Environmental Impact Assessment Report – Key Findings Report” by Golder Associates Ltd. Internet accessed July 14, 2015. (http://www.imperialmetals.com/i/mt-polley/2015-06-18-MPMC-KFR.pdf)

Oszvald, T., *Geologist*. Internet accessed 17 June, 2015. (http://oszvaldtamas.com/geologist.php)

Pierce M.W., C.I. Thornton, and S.R. Abt. 2010. “Predicting Peak Outflow from Breached Embankment Dams.” Journal of Hydrologic Engineering. Vol. 15, Issue 5. pp. 338-349

Rico M., G. Benito and A. Díez-Herrero. 2007. “Floods from Tailings Dam Failures.” Journal of Hazardous Materials. Vol. 154(1-3), pp 79-87

Seddon, K. D. 2010. “Approaches to Estimation of Runout Distances of Liquefied Tailings”, First Int. Seminar on the Reduction of Risk in the Management of Tailings and Mine Waste, Mine Waste 2010, Perth.

Tailings Info, 2014. Tailings Related Accidents – Failure, Breaches and Mudflows. Internet accessed on May 12, 2015. (www.tailings.info/knowledge/accidents.htm).

Von Thun, J.L. and D.R. Gillette. 1990. “Guidance on Breach Parameters,” Internal Memorandum Rep. Prepared for U.S. Dept. of the Interior, U.S. Bureau of Reclamation, Denver.

Wahl, T.L. 1998. “Prediction of the Embankment Dam Breach Parameters – A Literature Review and Needs Assessment.” Dam Safety Rep. No. DSO-98-004, U.S. Dept. of the Interior, Bureau of Reclamation, Denver.

Wahl, T.L. 2004. “Uncertainty of Predictions of Embankment Dam Breach Parameters.” Journal of Hydraulic Engineering, ASCE, Vol. 130, No. 5, pp 389-397.

Walder, J.S. and J.E. O’Connor. 1997. “Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earth Dams.” Water Resour. Res., Vol. 33(10), 12.

World Meteorological Organization (WMO), 2009. Manual on Estimation of Probable Maximum Precipitation (PMP). WMO-No. 1045. Geneva, Switzerland.