ASSESSMENT OF THE RISK OF FAILURE OF EMBANKMENT DAMS AND FLUVIAL LEVEES BY OVERFLOWING: THE EDF PRACTICE AND RESEARCH NEEDS

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Abstract. According to the French regulation, EDF (Electricité de France) must perform dam safety risk analysis for all dams which height is above 10 meters. Dam safety risk analysis includes a potential failure mode analysis, where overflowing erosion must be taken into account most of the time. In such cases, the dam owner must determine if the overflow will erode or not the dam or levee and form a breach and, if a breach forms, what is the breach outflow hydrograph. The criteria and methodologies used by EDF to answer these questions are presented. Research needs to fill the gaps and improve the reliability of potential failure mode analysis are discussed.

1 INTRODUCTION

EDF (Electricité de France) is the largest French utility. Its hydro-retail (EDF Hydro) operates 427 hydro-power plants with an installed capacity of 21 GW, including about 100 embankment dams and 500 km of canal embankments and fluvial levees. According to French regulations, the dam owner must perform every 10 years a dam safety risk analysis for dams rated A (dam height (H) ≥ 20 metres) and every 15 years for dams rated B (dam height (H) ≥ 10 metres and $H^2 \times V^{1/2} \geq 200$, V being the reservoir capacity in hm^3).

The dam safety risk analysis is based upon, among other studies, a potential failure mode analysis, including the analysis of failure by overflowing erosion for a large number of EDF embankment dams, canal embankments, and fluvial levees. When the hydraulic loading conditions lead to consider overflowing above the dam/levee crest, the dam owner needs to answer the two following questions:

1) Will the overflow erode the dam or levee and then form a breach?
2) If a breach forms, what is the breach flow hydrograph?

The criteria and methodologies used by EDF to answer these questions are presented hereafter. Research needs to fill the gaps and improve the reliability of potential failure mode analysis are finally discussed.

2 CRITERIA USED TO DETERMINE IF THE OVERFLOWED DAM OR LEVEE WILL BREACH OR NOT

In most cases, the present state-of-the-art does not allow to answer deterministically the question of whether an overflowed dam or levee will breach or not. Except in the case of a homogeneous dam that retains a reservoir, constituted with fine cohesive soils, where a methodology has been validated (see. 3.3), no numerical approach is currently validated to estimate the evolution of embankment erosion versus time. However, like many other dam or levee owners, EDF often has to answer this question in the frame of potential failure modes.

For dams rated A or B, the breach is considered highly probable to certain when the elevation of the water reaches the dam crest, whatever the erosion resistance of the soil material constituting the type of dam body and the dam design type.

For levees for flood protection, the criteria used to consider if the levee will breach or not depend on two parameters (see. Fig. 1):

- The overflow depth (E_S, in metres), defined as the difference between the total energy head upstream from the elevation of the levee and the levee crest.
- ΔE (in metres), which is the difference between the total energy head along the levee in the river and the total energy head along the side of the levee on the protected area.

E_{riv} is the depth of the water in the river (in metres), E_{val} is the water depth downstream of the levee.

$$\Delta E = E_{val} - E_{riv} \text{ if } E_{riv} < E_{val} \quad (1)$$

$$\Delta E = E_{riv} - E_{val} \text{ if } E_{riv} \geq E_{val} \quad (2)$$

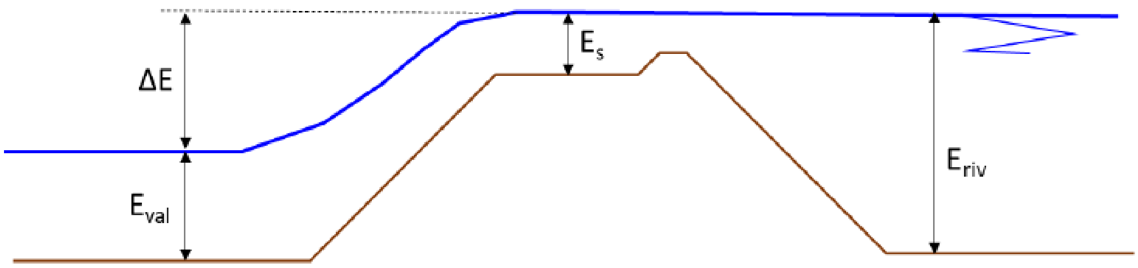

Figure 1: Representation of parameters governing the breach criteria for levees.

The reference breach criterion is as follows: a breach occurs if $E_s > 0$ m and $\Delta E > 1$ m. This criterion was deduced from the feedback of levee failure case histories in France (although this feedback has not been formalized up to now) and it is more adapted to levees constituted with silty sand soils, which are the dominant soils constituting the French levees.

Breaching is evaluated to be nearly impossible when $E_s < -0.25$ m. Breaching is assessed nearly certain when $E_s \geq 0.20$ m.

3 METHODS USED TO DETERMINE THE BREACH FLOW HYDROGRAPH

3.1 Definition and context

When a breach occurs, the outflow of the breach is defined as the function of the flow through the breach versus time. Breath outflow is the upstream boundary condition of the dam/levee-break flood wave propagation modelling.

In some cases of old levees that have experienced breaches in the past, it may be possible to estimate the breach hydrograph from historic data. However, historical data are often not sufficient to help define a breach hydrograph, especially in the cases of dams or, more generally for recent structures. In these cases, the breach hydrograph can be determined with a simplified approach. In a limited category of embankment dams, the breach hydrograph can be determined with a physically-based numerical modelling, using numerical tools which have been validated against large-scale experiments and real case studies.

3.2 Estimation of the breach outflow hydrograph with a simplified approach

The breach outflow hydrograph can be determined with a simplified approach for the following types of structures:

- Dams retaining a reservoir other than homogeneous dams constituted with fine cohesive soils,
- Canal embankments,
- Levees for flood protection.

For dams retaining a reservoir and canal embankments, the simplified approach consists in determining a triangular-shape breach outflow hydrograph defined by the three following points (Figure 2):

- The point (t = T0, Q = 0) shows the initial time of formation of the breach,
- The point (t = T1, Q = Qmax) shows the peak of the breach outflow,
- The point (t = T2, Q = 0) shows the end of the breach outflow.

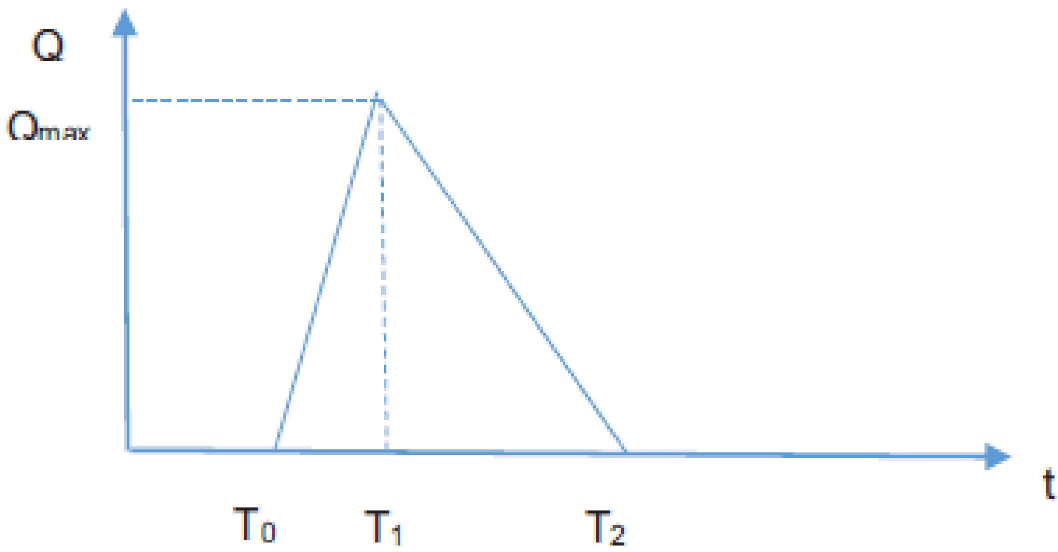

Figure 2: Breach outflow hydrograph determined with a simplified approach.

It is generally considered that T1 corresponds to the time where the breach reaches its maximum length.

Determining the initial time of the formation (T_0) has no influence on the breach outflow hydrograph. From this point of view, the point ($t=T_0$, $Q=0$) is arbitrary. However, the methods used to determine the two other points that define the breach outflow hydrograph depend on the type of structure.

3.2.1 Case of dams that retain a reservoir

For dams retaining a reservoir, the approach consists first in determining the peak outflow, then estimating the duration [$T_1 – T_0$] and finally calculating time T_2.

For embankment dams that retain a reservoir other than homogeneous dams constituted with fine cohesive soils, the breach peak outflow is estimated with empirical formula, which has been all fitted by regression relationships based on data from real case studies [1].

The three empirical formulas currently used in EDF are presented in Table 1 below.

Table 1: Recommended empirical formula to estimate the peak outflow.

Authors	Formula	
Froehlich (1995) [1]	$Q_P = 0.607\ (V_r)^{0,295}\ (H_W)^{1.24}$	(3)
CLF (2020) [2]	$Q_P = c\ H_d\ (V_r)^{0.5}$	(4)
Xu & Zhang simple (2009) [3]	$Q_P = 0.133\ (g\ (V)^{5/3})^{0.5}\ (V^{1/3} / H_w)^{-1.276}\ e^{FM+ER}$	(5)

Q_P is the peak outflow, in m^3/s. H_w is the depth of the water depth above the breach invert, in metres (H_w being generally estimated as the difference between the reservoir elevation and the river bed). H_d is the dam height, in metres (the dam height being defined here as the difference between the crest elevation and the river bed). V_r is the reservoir capacity, in m^3. V is the volume of water stored above the invert in the breach, in m^3.

The Froehlich formula includes two parameters: the volume of water stored in the reservoir and the water depth above the breach invert. This formula has been fitted with about twenty real case studies of dam failures. It does not allow to take into account, even empirically, the erosion resistance of the soil material constituting the dam body. This is the two-parameter formula which provides the best predictions.

The CLF 2020 and Xu&Zhang simple formula take empirically into account the erosion resistance of the soil material constituting the dam body, in addition to the other parameters: water depth or dam height and reservoir capacity.

In the CLF 2020 formula, the empirical coefficient 'c' takes into account the erosion resistance of the soil material constituting the dam body. This parameter c can have the values shown in Table 2 below. These values were determined by regression using data from 56 real case studies of embankment dam failures [1].

Table 2: Values of the 'c' coefficient in the CLF 2020 formula.

Erodibility Class	Embankment Dam Type	Breach Initiation process: Overflowing	Breach Initiation Process: Internal Erosion
L (Low)	Zoned, well compacted clay core	c = 0.035	c = 0.015
M (Medium)	CFRD, or zoned silty core	c = 0.06	c = 0.04
H (High)	Homogeneous few or not compacted	c = 0.1	c = 0.08

The Xu&Zhang "simple" formula also takes into account the erosion resistance of the soil material constituting the dam body through the empirical coefficient ER. For highly erodible dams, ER = -0.089. For low-erodible dams, ER = -1.433. The parameter FM allows for differentiating the breach initiation process: FM = -0.788 for failures initiated by overflowing and FM = -1.232 for failures initiated by internal erosion. This formula was fitted by regression using data from 41 real case studies of dam failures.

The engineer who uses these formulas must be aware of the high uncertainty of the results they provide. Although these three formulas provide the best results according to the present state-of-the-art, the engineer is never sure to get a conservative result when using them.

Furthermore, a verification of the coherence of the breach peak outflows provided by these formulas versus the maximum possible breach dimensions must be performed. This verification of coherence relies on the principle that the peak outflows provided by the formulas cannot exceed the flow obtained with a flow weir relation, considering a rectangular breach shape, with a breach length equal to the total dam length and a breach height equal to the maximum dam height. The flow weir relation to be used for this verification of coherence is the following:

$$Q_{max} = C_d \times L_{max} \times (2g)^{1/2} \times (H_{max})^{3/2} \quad (6)$$

where Q_{max} is an envelope of the breach peak outflow, in m^3/s, L_{max} is the total length of the dam crest, in metres, H_{max} is the maximum dam height above the downstream ground level, in metres and C_d is the flow weir relation coefficient, in this case taken equal to 0.385.

Finally, it is first recommended to systematically use all three formulas ((3), (4), (5)), then to perform the verification of coherence presented above and finally, from the three values of breach peak outflow obtained, derive a range of likely values of breach peak outflows. The upper and lower bounds of this range of probable values can be adapted according to the safety issues (the hazard level of the dam will be taken into account for instance).

Furthermore, in addition to the warning on the uncertainties associated with these formulas mentioned above, the breach peak outflows must be written with significant numbers which are coherent with those uncertainties: numbers of units, dozens, and sometimes hundreds of m^3/s are generally meaningless.

Once the breach peak outflow was estimated, the duration $[T_1 - T_0]$ (see. Fig. 1), which represents the breach outflow increase stage, is estimated through an expert judgement, within a range from 30 minutes to a few hours, depending essentially on the dam height, the dam cross-section geometry, and the erosion resistance of the soil materials constituting the dam body.

Finally, time T_2, which corresponds to the end of the breach outflow, is calculated in order the integral of the breach outflow equals the reservoir capacity.

3.2.2 Case of canal embankments

Canal embankments differ significantly from dams that retain a reservoir in terms of hydraulic operation, which has important consequences on the breach-widening process in case of failure.

When a breach develops in a canal embankment, the breach outflow is controlled by two critical sections located upstream and downstream the breach. Consequently, the peak outflow of canal embankments is dependent not only on the embankment height, the volume of water stored in the reach and the erosion resistance of the material constituting the embankment, but also depends on the canal cross section upstream and downstream the breach and the critical water height in those sections.

This has the effect of limiting the maximum length and the breach peak outflow of the canal embankments compared to the large crest length dams retaining a reservoir where these critical section processes do not exist.

The recommended method to determine canal embankment breach peak outflows consists in using a weir formula consistent with breach processes considering previously an estimate of the final breach length.

The final length of the breach is assessed from an expert judgment. When a canal embankment fails, the final width is generally in the same order of magnitude as the width of the canal. A range between 1 and 2 canal widths can be considered a good framework of the final breach width.

The recommended weir formula for calculating the peak outflow is:

$$Q_P = C_d \times L \times (2g)^{1/2} \times (H)^{3/2} \quad (7)$$

where Q_P is the maximum breach outflow, in m^3/s, L is the final breach width, in meters, H is the water depth between the water levels upstream and downstream of the breach, in metres and C_d is the coefficient of the weir formula, generally between 0.3 and 0.385.

The duration of the stage of increase in the flow of the breach hydrograph $[T_1 – T_0]$ is then determined from an estimate (order of magnitude) of the widening celerity. Four ranges of breach widening celerities are proposed below, corresponding to four categories of embankments. These four categories were defined from case studies of dam failures.

- Old embankments constituted of highly erodible materials (dams built without modern compactors): breach widening celerity from 10 to 15 m/min.
- Embankments built with modern compactors but constituted with erodible to highly erodible materials: breach widening celerity from 0.5 to 3 m/min.
- Embankments built with modern compactors with a homogeneous body or with an impervious core moderately resistant to erosion: breach widening celerity from 0.1 to 0.5 m/min.
- Embankments built with modern compactors and made of materials highly resistant to erosion: breach widening celerity from 0.01 to 0.1 m/min.

Once the duration $[T_1 – T_0]$ determined, time T_2 is calculated in order the integral of the breach outflow equals the volume of water stored in the canal reach where the breach happened.

3.2.3 Case of levees for flood protection

The breach formation process for levees for flood protection is even more complex than that of the one of dams. In addition to erosion of the embankment itself, it often comprises a significant scour in the foundation. This scour can be located below the levee embankment, upstream to it, or downstream to it. This process of scouring in the foundation is highly dependent on the geological characteristics (for instance, the level of the bedrock roof) and the resistance to erosion of the alluvial materials located between the embankment and the bedrock.

When historical data are complete enough or when the levee is comparable to a well-documented case study, it is possible to define the final geometry and the breach outflow hydrograph by analogy.

When this is not possible, a simplified breach outflow hydrograph can be estimated considering a maximum breach length, a water depth through the breach, and a weir formula.
It is recommended to use the following formula:

$$Q(t) = C_d \times L(t) \times (2g)^{1/2} \times H(t)^{3/2} \quad (8)$$

where Q(t) is the outflow (in m^3/s), L(t) is the breach length versus time, in meters, H (t) is the water depth through the breach versus time, in meters, and C_d is the weir formula coefficient, generally between 0.3 and 0.385.

$$L(t) = L_0 + dW/dt \times t \quad (9)$$

where L_0 is an initial breach length, which can be taken arbitrarily equal to 1m, dW/dt is the breach widening celerity, which can be considered constant or variable with time depending on the available input data. L(t) is bounded by L_{max}, the final breach length.

$$H(t) = Z_{\text{Upstream water elevation}}(t) – Z_{\text{Bottom}} \quad (10)$$

where $Z_{Upstream\ water\ elevation}(t)$ is the water elevation in the river versus time, generally calculated with a 1D or 2D-hydraulic numerical model. Z_{Bottom} is the breach invert elevation, depending on the potential scour in the foundation. If the ground level (Z_{Ground}) is equal to the river bottom, $H_{Upstream\ breach}(t)$ is the water depth in the river versus time (11) and $H_{Foundation\ scour}$ is the foundation scour depth (12).

$$H_{Upstream\ breach}(t) = Z_{Upstream\ water\ elevation}(t) - Z_{Ground} \tag{11}$$

$$H_{Foundation\ scour} = Z_{Ground} - Z_{Bottom} \tag{12}$$

This simplified approach does not take into account several physical processes, in particular the evolution of foundation scour depth with time and the effects of the breach widening dissymmetry on the breach widening celerity.

Orders of magnitude of the parameters L_{max} (final breach length), dW/dt (breach widening celerity) and $H_{Foundation\ scour}$ (foundation scour depth) are proposed hereafter. These orders of magnitude are just intended to provide some guidance. However, the engineer has to consider the geological, geotechnical, and hydraulic specific conditions of the levee which is studied.

- **Final breach length (L_{max}) in metres:**
  In a preliminary approach, the final length of the breach can be estimated in a range of 0.5 to 1.5 times the width of the dyked riverbed.
- **Breach-widening celerity (dW/dt) in metres/minute:**
  Several publications show that the breach width is not constant versus time for levees [4], with a fast-widening stage and at second a slower widening stage. Nevertheless, a constant mean breach widening celerity is considered in a preliminary approach over the widening stage.
  For levees made of sandy gravel materials, a range of breach widening celerity between 1.5 m/min and 3 m/min can be considered.
  For levees made of fine cohesive materials (soil materials mainly silty and/or clayey), a range of widening celerity between 0.1 m/min and 0.3 m / min can be considered.
- **Foundation scour depth ($H_{Foundation\ scour}$) in meters:**
  The maximum foundation scour depth can be estimated preliminarily in a range of 1 to 2 times the levee height. Knowledge of the geological conditions below the levee is essential to estimate this parameter. If the bedrock roof were located at a shallow depth below the embankment, it would obviously limit the potential scour depth.

3.3 Estimation of the breach flow hydrograph using a physically-based simplified numerical approach

A numerical approach representing embankment dams overflowing has been developed in the US from the 90’s by the USDA/ARS HERU laboratory in Stillwater, OK. The research work carried out by this laboratory consisted first of large-scale experimental modelling and the description of overflowing erosion processes for homogeneous embankments constituted with fine cohesive soils. This work led to the characterization of the Head Cut Migration process and the description of the breach initiation, formation, and widening stages. This laboratory developed the Jet Erosion Test [5] to allow quantifying the erosion resistance parameters and the WinDAM numerical model [6] which computes overflowing erosion of a homogeneous embankment dam.

Both WinDAM and EMBREA codes, the latter having been developed by HR Wallingford with the same physical modelling implemented [7] have been tested and validated against large-scale field tests and real dam failure case studies by an international team within which EDF participated, in the framework of the CEATI DSIG Erosion project [8].

These two numerical models use a 3D conceptual physical modelling of the « head cut migration » erosion process. They are based on a 2D vertical representation of the embankment geometry, on the geotechnical properties of the materials of the embankment, and on the upstream and downstream hydraulic conditions. They also take into account the resistance to overtopping due to the presence of grass or rip-rap on the downstream face of the dam. They provide the outflow hydrograph (whether there is formation of a breach or not) as well as the geometry of the eroded cross section.

The use of these two models is limited to the following assumptions.

- Dams retaining a reservoir (not canal embankments nor levees) with a homogeneous dam body constituted with fine cohesive materials.
- Constant slopes upstream and downstream.
- Foundation erosion not taken into account.

When the characteristics of the dam to be studied are compatible with these assumptions, it is recommended to use one of these two models, having previously estimated the erosion resistance of the soil constituting the embankment with JET tests. Without the results of the JET test, it is possible to get a preliminary estimate of the erosion resistance parameters (erosion kinetics coefficient K_D and critical shear stress τ_c) using abacus presented by USDA [9].

4 RESEARCH NEEDS

The criteria and methodologies presented above to determine whether a dam or levee will breach or not when submitted to overflowing conditions and determine the breach hydrograph are still relying on expert judgements and empirical relations with many uncertainties. Moreover, these approaches do not ensure the engineer to always provide conservative results. Despite the issues of potential failure mode analysis, very few research effort was deployed by dam owners and research institutes have made very little research effort over the last decades to improve this state-of-the-art.

A similar research effort to the one performed by USDA/HERU in the 90’ and 2000’ on overflowing erosion of embankment dams constituted with fine cohesive soils needs to be carried out on overflowing erosion of embankment dams constituted with gap-graded soils, using a similar methodology, based on multiscale experimental tests, including large-scale field tests and the definition of a simplified numerical modelling. Then, research efforts will need to address the influence of the different types of sealing of embankment dams, other than the homogeneous-type ones, like sealing upstream faces or internal cores.

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