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OTE2C^{ISL}: an overtopping breach model developed within the Polder2C's project

Cédrine ALLEON [1], Moez JELOULLI [1], Myron VAN DAMME [2]

1. ISL Ingénierie, 51 rue de l'Alcazar, 59800 Lille, France ; Head office: 75 Bd MacDonald, 75019 Paris, France.
   alleon@isl.fr
2. Rijkswaterstaat WVL, 8224 AD Lelystad, The Netherlands.
   myron.van.damme@rws.nl

Abstract:
Several breach models, such as WinDAM C or EMBREA, are already available for predicting headcut erosion development induced by overflow on homogeneous embankments. These codes were designed to quickly predict breach growth and breach hydrographs for earthen dams using simplistic calculations. However, these models don't consider headcut erosion by wave overtopping, whereas the hydrodynamics and the loads exerted on the landside slope differ significantly from those during overflow.
Within the Polder2C's project, ISL developed a model inspired by the philosophy of existing breach models but tailored for sea or estuary dikes. This new model introduces the overtopping phenomenon and the initiation of erosion due to wave overtopping as a starting point. This project has received funding from the Interreg 2 Seas program 2014-2020 cofounded by the European Regional Development Fund under subsidy contract No [2S07023].

Keywords:
Coastal engineering, Overtopping, Overflowing, Breach model, Nimble tool, Python, Coastal and estuarine environment.

1. Introduction

1.1 <u>Interreg2seas Polder2C's project</u>

The Interreg2seas Polder2C's is an international research project within the framework of the updated Sigmaplan for the river Scheldt. The former Hedwige-Prosperpolder, surrounded by nearly 8-meter-high dikes, was transformed back into tidal nature. Depoldering of this area offered a testing ground, where current and innovative techniques, processes, methods, and products could be tested on existing levees for practical validation. Together, fourteen project partners, led by the Dutch Foundation of Applied Water Research (STOWA) and the Flemish Department of Mobility and Public

Works, conducted numerous destructive field and lab tests as well as numerical modellings to validate flood defence practices.

1.2 Objectives

Based on the gathered literature and the data produced within the project, the partnership aimed to develop a nimble breach model for assessing dike safety against overtopping. Why focus on overtopping? External erosion is a leading cause of failure for earthen dams and dikes globally. While international recommendations or assessment methods have more extensively addressed external erosion due to overflowing – typically related to negative freeboard – wave overtopping, related to a positive freeboard, presents a unique challenge. Currently, there appears to be no model available to industry specifically addressing external erosion caused by wave overtopping. Why aim for a nimble model? Given the complexity of the process, involving multiple phenomena, simplification was deemed necessary. This approach enables users to quickly run test cases, easily adjust and refine numerical parameters, and perform rapid multiple runs across ranges of properties for statistical analysis.

2. Methodology

2.1 Generalities

A model in Python has been developed to provide an analytical assessment of the breaching process of an earthen vegetated dike subject to wave overtopping. A user-friendly interface is also provided to ensure the model can be easily used by water professionals and students. Within the framework of the Polder2C's project, this model was named "OTE2CISL", which stands for "OverTopping Erosion to See".
Drawing on the philosophy of existing earthen dams breach models under overflow, such as WinDAM C or EMBREA, we utilized the head-time boundary conditions to integrate the mechanism of erosion under overtopping. The dynamic loading on the dike due to wave overtopping varies in time and space and is determined by various factors such as the sea state, wind direction, dike shape, and the topography behind the dike. Our goal was to develop a model that is straightforward to configure and capable of rapid computations to facilitate its use in statistical analyses for flood risk assessments.
The calculation steps in the OTE2C model include four distinct stages:

- *The first stage* is dedicated to defining the hydrodynamic conditions that lead to wave overtopping at the crest of the dike. This involves using parameters related to the characteristics of the sea, specifically the characteristic wave period and the peak overtopping discharge.
- *The second stage* converts the characteristic wave period and peak discharge into a measurable maximum flow depth and maximum velocity of waves overtopping the crest of the dike, which can affect the downstream face of the dike.

- *The third stage* calculates the impact location and changes in maximum flow velocity on the landside slope of the dike, employing a time-based projectile equation.
- *The last stage* computes the erosion depth based on the velocities and locations.

These stages, along with their assumptions, are detailed in the paper.

2.2 Computing overtopping flow (stage 1)

The hydrodynamic conditions are partly defined by the model user, as described below. The equations utilized in the OTE2C model are derived from the EurOtop Manual (2018), a comprehensive guide on wave overtopping of sea defences and related structures. It allows for the estimation of a representative characteristic flow over the crest based on varying sea conditions. The formulas presented in the following sections are applied under specific hypotheses that mirror the conditions found at the Polder2C's project site, ensuring their relevance and applicability to the real-world scenarios encountered there. These conditions form the basis for accurately modelling the impact of wave overtopping on sea defences, crucial for the development and validation of the OTE2C model.

- Dike with smooth upstream slope, without berm or sea wall.
- Upstream slope similar to downstream slope: vegetated slope, between 1.5 and 3H/1V (formula "gentle slope").
- Swell orientation perpendicular to the dike axis (frontal impact).

The EurOtop formulas integrated into the OTE2C code are:

- Relatively gentle slopes (EurOtop eq. (5.10) and (5.11))
- Shallow and very shallow foreshores (EurOtop eq. (5.15))
- Gentle slopes / Deep water conditions and non-breaking (EurOtop eq. (5.12)).

This last is reiterated here as it supports a demonstrative example:

$$q = \sqrt{g.H_{m0}^3} \cdot \frac{0.026}{\sqrt{\tan(\alpha)}} \cdot \zeta_{m-1,0} \cdot \exp\left[-\left(2.5 \cdot \frac{R_c}{\zeta_{m-1,0} \cdot H_{m0}}\right)^{1.3}\right] \qquad (1)$$

with: q, the overtopping mean discharge [l/s/ml], H_{m0} the significant swell height at bottom of the dike [m], $tan(\alpha)$ the downstream slope of the dike [m/m], Rc the freeboard relative to the crest [m] = Dike crest level [m AD] - Mean sea level (SWL) [m AD] and $\zeta_{m-1,0}$ the Iribarren number = $\tan(\alpha)/\sqrt{(S_{m0})}$ [m/m].

The overtopping discharge during a wave overtopping event varies with time and is characterized by a rapid increase in discharge at the beginning of the overtopping and a slower decrease after the peak discharge. To describe the most unfavourable hydraulic conditions, the mean discharge calculated with the EurOtop formulas (q or q_{car}) were transformed into a hydrograph for single overtopping event, based on a peak discharge (q_{max}), to be estimated, spread over a characteristic overtopping time T1.

$$q_{max} = q_{car}/K \qquad (2)$$

Based on waves hydrodynamics, the K coefficient that depends on the relative crest freeboard R_c/H_{m0} was defined as:

$$K = 0{,}25 * \exp\left[-\left(1.5 * \frac{R_c}{\zeta_{m-1,0} * H_{m0}}\right)^{1.3}\right] \tag{3}$$

Based on the EurOtop formulas, formula (3) contains a coefficient of 1.5, which, however, was calibrated and adapted for each specific context.
Furthermore, when the crest freeboard equals zero, the characteristic overtopping time amounts to a quarter of the peak period (K = 0,25). As the crest freeboard increases, this ratio decreases, leading to overtopping concentrated over a shorter duration.
Figure 1 illustrates a case with gentle slopes, deep water conditions and non-breaking waves (formula (4)), with Tp =10s / H_{m0} = 2,0 m / tan(α)=1/3. q_{car} have been calculated based on formula (4) and q_{max} on formula (1).

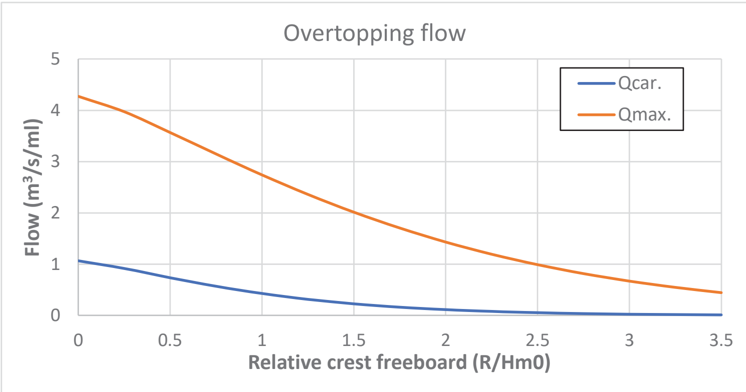

Figure 1. Overtopping flows as a function of relative crest freeboard.

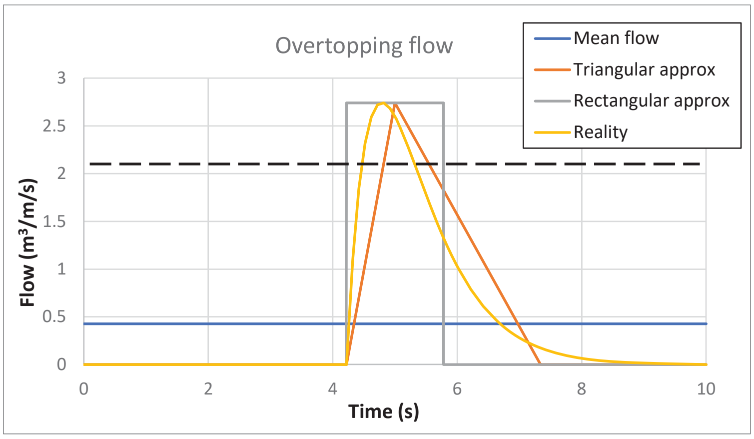

Figure 2. Overtopping flows over time for one single wave (example with R/H_{m0} = 1).

Furthermore, the graph (see Figure 2) shows the shape of the estimated hydrograph in yellow, based on a calculated q_{car} in blue, and a realistic shape based on previous lab

experiments (see section 2.3) and CFD calculations (ANSYS CFX) of realistic wave overtopping. To make calculations easier and faster, two levels of simplification can be made in the model: a) triangular approximation (orange) and b) rectangular approximation (grey). For all the curves, including the blue one, the total discharge remains constant. The triangular approximation aligns most closely with reality and is the approach adopted in the model. Initially, the rectangular approximation was employed for simplicity, followed by the introduction of a correction when calculating the erosion effect due to overtopping.
The underlying principle is that erosion occurs only above a certain threshold, which can be velocity-based (critical velocity) or stress-based (critical shear stress). This threshold can be quantified as a critical discharge, q_{crit}.
On the other hand, erosion generally scales with the discharge raised to the power of a coefficient, N. For instance, according to the Manning-Strickler formula (CARLIER, 1987), shear stress is related to $Q^{0.6}$. Therefore, erosion is proportional to the integral of $\max(0, (q^N - q_{crit}^N))$, representing the area above the threshold in the illustrative Figure 3 provided below.

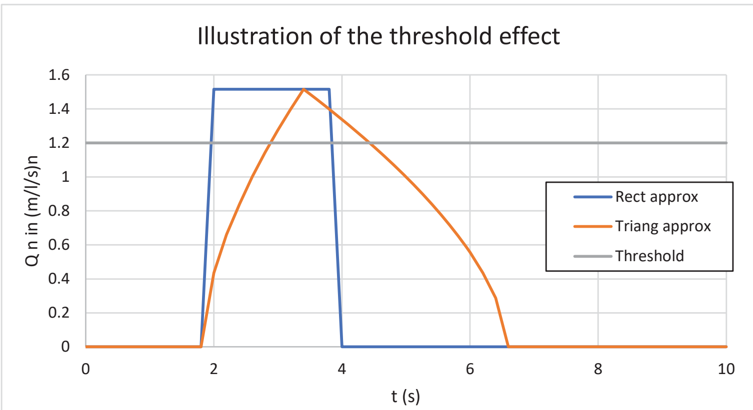

Figure 3. Overtopping flows over time for one single wave (example with R/Hm0 = 1).

When the critical discharge is near the peak flow, the rectangular approximation tends to overestimate the erosion velocity. An analytical expression for a correction coefficient was derived to reconcile for the difference in the triangular and rectangular approximations. This correction coefficient is denoted here as K_{corr}:

$$K_{corr} = 2\frac{(1-(N+1).\gamma^N + N.\gamma^{N+1})}{(N+1).(1-\gamma^N)} \tag{4}$$

with $\gamma = q_{crit}/q_{max}$ between [0,1].

2.3. <u>Computing water velocity and level over the crest (stage 2)</u>
For various situations, with the equations previously discussed, both the peak overtopping discharge and the corresponding overtopping period were calculated based

on the existing sea state and dike geometry. HUGHES *et al*., (2012) offered empirical relations that connect wave parameters with the volumes of individual wave overtopping event. HUGHES *et al.,* (2012) presented time series of flow thickness and flow velocity on the crest measured during small-scale experiments conducted by HUGHES & NADAL (2009) to characterize parameters of individual wave volumes.
The ensuing graph displays scatter plots from one experiment involving 226 individual overtopping wave volumes, (see Figure 4).

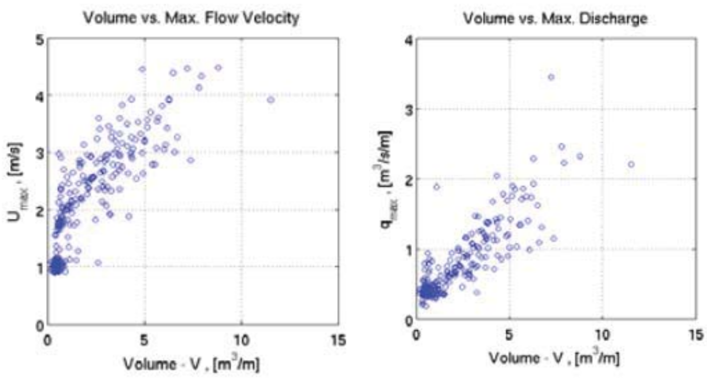

Figure 4. Individual wave volume parameters from an experiment with 226 waves (HUGHES et al., 2012).

From the right graph on Figure 6, the maximum overtopping discharge was approximated as a function of the overtopping volume as follows:

$$V = \frac{10}{3} q_{max} \tag{5}$$

Then, from the left graph on Figure 6, the maximum flow velocity as a function of the volume was approximated as follows:

$$U_{max} = 1{,}42\, V^{0.5} \tag{6}$$

So, the maximum flow velocity U_{max} [m/s] is function of the maximum overtopping flow q_{max} [m^3/s] have been established based on the small-scaled experiments hereinabove mentioned:

$$\boldsymbol{U_{max} = 2,6\; q_{max}^{0,5}} \tag{7}$$

The following graphs (Figure 5) presents parameters for one of the individual waves from the experiment of HUGHES & NADAL (2009).

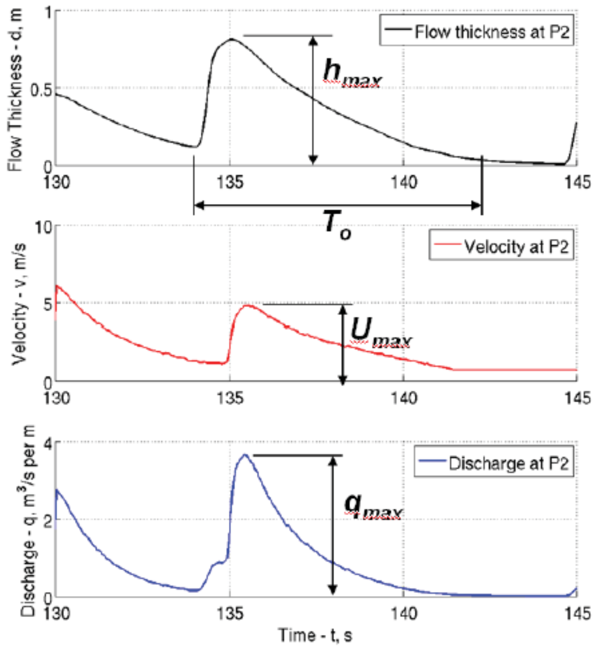

Figure 5. Example of an individual wave from HUGHES & NADAL (2009) experiment.

The formula of the maximum flow velocity defined previously is consistent with the graph in Figure 7 because it gives a velocity of 5 m/s or a maximum overtopping flow of 3,75 m^3/s.

2.4. Computing impact point and flow velocity (stage 3)

Using the flow velocity predicted at the dike crest from the previous stage, the impact point on the downstream landside slope can be determined by employing a time-based projectile equation. This calculation considers:

- U_{max} the maximum overtopping velocity over the crest dike.
- α the angle of the downstream landside slope with the horizontal.
- β the angle made by the flow with the downstream landside slope.

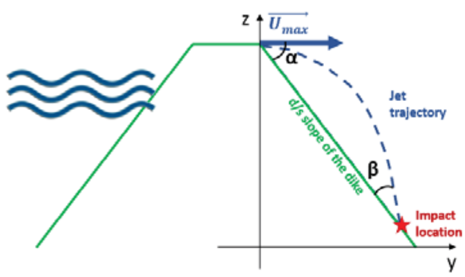

Figure 6: Wave impact trajectory.

The expressions derived for determining the location of the impact point are as follows:

$$x = 2\frac{U_{max}^2}{g}tan(\alpha) \tag{8}$$

$$y = 2\frac{U_{max}^2}{g}tan^2(\alpha) \tag{9}$$

The impact angle β is expressed as follows:

$$\beta = \operatorname{atan}\big(2\tan(\alpha)\big) - \alpha \tag{10}$$

The flow velocity at the impact location as follows:

$$U_{\max\_impact} = U_{max}\sqrt{1 + 4\tan^2(\alpha)} \tag{11}$$

As mentioned in the previous section, it's important to recognize that the impact location, as calculated using the projectile equation, accurately describes the wave trajectory only under specific conditions. This method is applicable when, on the inner side of the dike, the significant wave height is sufficiently high, and the freeboard is low enough. Otherwise, the flow pattern of the overtopping discharge resembles the hydrodynamics of overflow on the landside slope. For this reason, the use of the projectile trajectory to initiate the erosion somewhere else than on the crest should be limited within input boundary conditions. The trajectory is calculated based on the overtopping event mean wave and the maximum overtopping discharge estimated on stage 1. To account for the irregularity of the waves, a Rayleigh distribution coefficient has been applied for considering 5 different levels of wave heights. The highest energy is then applied for the largest waves.

2.5 <u>Computing downstream erosion (stage 4)</u>

2.5.1 *Grass erosion*

Prior to external headcut erosion, the Dutch Cumulative Overload Method describes the erosion initiation on the landside slope of the dike due to wave overtopping. The impact or the maximum flow velocity of the overtopping wave is more important than the duration of the flow (VAN DER MEER *et al*., 2011). The “cumulative overload” function is an erosional index in which the overtopping duration is not considered, but sums the (square of the) velocity above a critical velocity: the overload above a certain threshold:

$$D = \sum_{i=1}^{N} \max\left(0;\ \alpha_m(\alpha_a U_i)^2 - \alpha_s U_c^2\right) \tag{12}$$

where:

- α_m is a correction factor for the load when a transition between levee covers is present,
- α_a is an acceleration factor between U_{max} and U_{max_impact}, $\approx \sqrt{1 + 4\tan^2(\alpha)}$,

- α_s is a correction factor for the critical velocity as the consequence of a transition,
- U_c is the critical velocity of the grass cover,
- D is the damage number.

A damage number of 7000 indicates that the grass cover has locally failed over a depth of approximately 15 cm (~ depth of the grass roots system), (VAN DER MEER *et al.*, 2011). At that point in time, it can be expected that the headcut erosion of the levee has initiated at the location where the grass cover has failed. Please note that the cumulative overload method, given above does not predict the location of failure, but could be used to predict the time at which the grass cover fails.

2.5.2 *Headcut erosion*

TEMPLE *et al.*, (2005) developed a simplified headcut erosion model (SIMBA) with empirical equations for the erosion rate and a 1D flow description, based on observations of large-scale breach experiments of cohesive clay embankments. This modelling approach is also adopted in the EMBREA, WinDamC, and AREBA breach models to simulate headcut erosion. Under overflow, the headcut is assumed to start where the crest meets the landside slope since the growth and progression of this headcut determines the change in invert level when it reaches the waterside slope. The effects of the formation and progression of other headcuts are therefore neglected, leading to a faster model. The EMBREA model does allow for the headcut to initiate at other locations. TEMPLE *et al.*, (2005) identified the following four stages in the headcut breach process:

- A: Headcut formation on the downward slope.
- B: Headcut advance through the embankment crest.
- C: Breach formation as the headcut enters the reservoir.
- D: Breach expansion during reservoir drawdown.

These stages are visible on the hereinafter Figure 7.

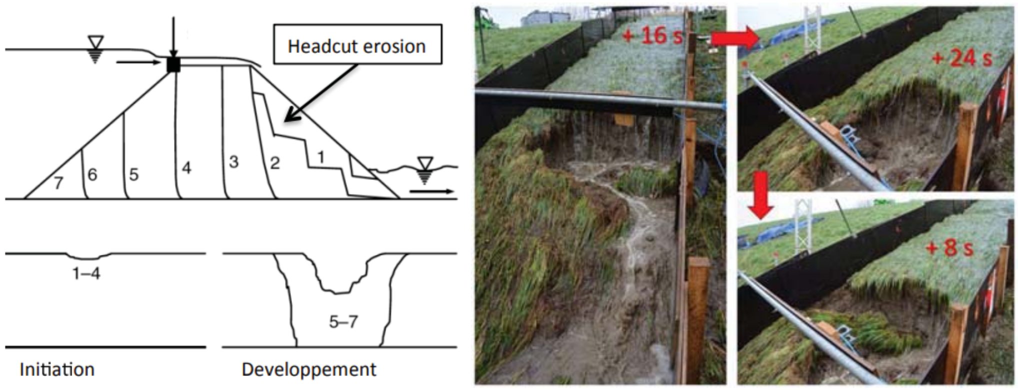

Figure 7. Headcut erosion process for cohesive soils – theoritical process scheme on the left (COURIVAUD et al., 2019), similar to the one observed at Polder2C's site on the right (KOELEWIJN et al., 2022).

The erosion process as described by TEMPLE *et al*., (2005) has also been integrated in the OTE2C model. The formulas and underlying assumptions of the breaching process have not been integrated into the present paper.

2.6 OTE2C interface

The left-hand side of the interface (see Figure 8) is dedicated to settings and can be configured either directly in the provided boxes or within the Python program, accessible via the second main tab. The right-hand side displays the results after the simulation completed. The primary view shows a 2D dike profile with the state of grass cover and erosion progress at each timestep. Additionally, several tabs present results in curve format. See interface screenshots in Figure 8.

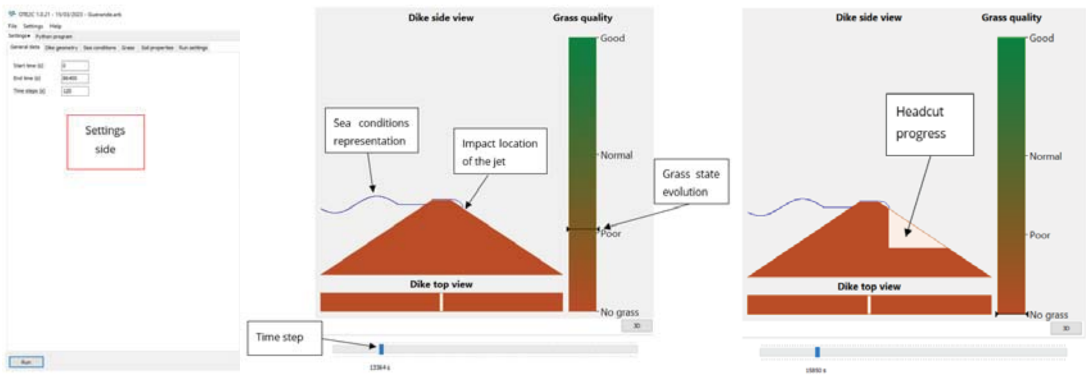

Figure 8. OTE2C^{ISL} setting side of the interface (left) and results extracts (middle and right).

3. OTE2C probabilistic module

3.1. Objectives

One of the objectives of the tool is to assess the probability of failure of the dike regarding: *(i)* the uncertainty and variability on sea conditions: high tide levels (HTL) and significant wave heights (SWH); *(ii)* the uncertainties on material properties: critical velocity (for grass), critical shear stress and erodibility for soil properties.

In the model, there are 5 stochastic input parameters. The main simplification hypotheses are:

- The stochastic parameters are independent,
- The failure is corresponding to a complete erosion of the crest after 1 simulation period (1 day corresponding to 2 tide cycles in general).
- All the calculations are made for one period (1 day), and then transferred to a yearly probability of failure: during the simulation period, one can assume that tide

levels and wave heights do not vary a lot. At the other hand, storms and exceptional events don't usually last more than 1 day.
In addition to the required data for a deterministic OTE2C simulation, the user must provide the cumulative distribution function (CDF) of the 5 varying parameters. Files containing the CDF for SWH and HTL should be expressed in terms of daily distribution. They can be calculated with theoretical Weibull or generalized Pareto (GPD) laws, or empirically derived from measurements.
It is highly recommended to provide extreme values of SWL and HTL. The model will, in any case, extrapolate the data until the 1 value (1 is interpreted as the maximal possible value for the parameter). For now, the OTE2C probabilistic module can only be run in command line. The number of simulations is an input parameter, the only advice is to choose it carefully in function of the target computing time.

3.2. <u>Provided results</u>

The results provided by the model are:

- *Probability of failure and scatter maps*: Each point corresponds to a simulation that leads to a failure of the dike. The aim of the output map is to show the zones where we have failures and the probability of having these failures. Several simulations are needed to have a representative scatter map.
- *Histograms*: The outputs integrate 5 histograms. It represents the distribution of failures for each varying parameter in percentage of total simulated failures. An example of output histograms is shown in Figure 9.
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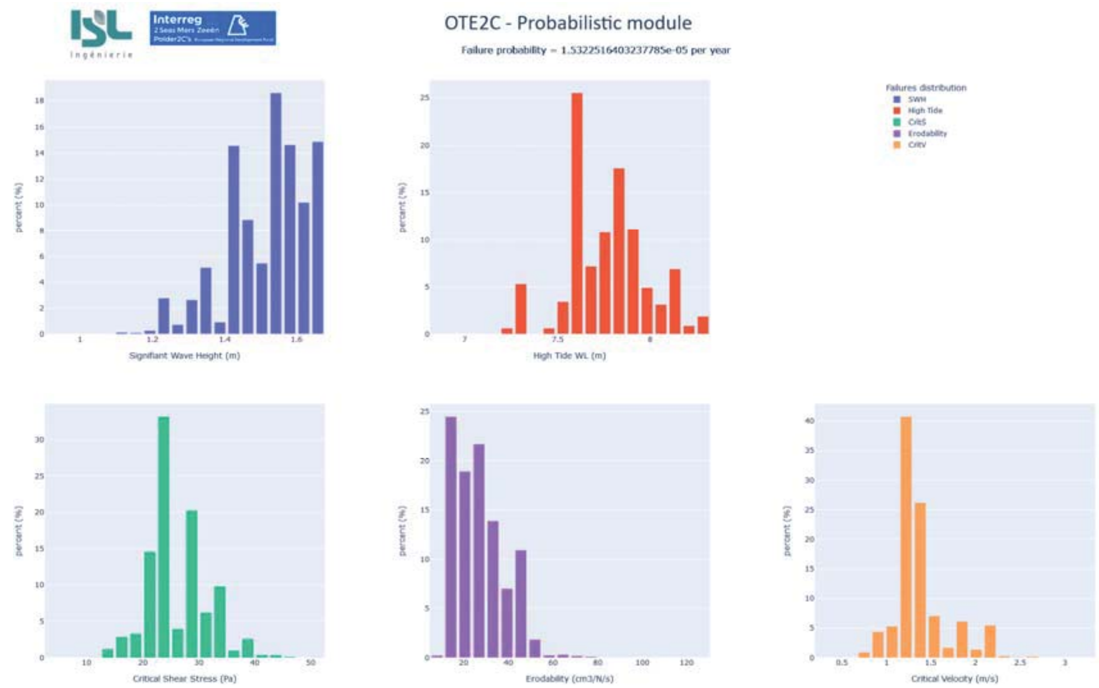

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Figure 9. Example of OTE2C output histograms.

4. Conclusions

- The OTE2C model has been developed for industry use for performing flood risk assessments on levees subject to wave overtopping erosion.
- The OTE2C model facilitates deterministic runs via a Graphical User Interface, and probabilistic runs via a command line.
- The OTE2C model simulates headcut erosion in clay due to wave overtopping and predicts the onset of failure of grass covers.
- Further development of the model is planed.

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