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***Abstract*—In the framework of a regulatory hazard assessment study applied to the coastal protection system of the Mont Saint-Michel Bay, a coupled TELEMAC-2D/TOMAWAC dynamic flood model was set-up and run in storm conditions. Besides considering hazard scenarios such as dyke breaching or culverts (due to e.g. electric failure of tidal gates), overtopping is also considered. To that end, a specific module dedicated to the computation of wave overtopping discharges over the coastal dykes and transfer to the rear of the structures was developed in TELEMAC-2D. The EurOtop 2018 formulations for overtopping discharges were implemented as a decision tree algorithm. The coastal protection system was segmented in a number of sections along which structural, geometric and incident hydrodynamic characterictics were deemed homogeneous. Such an implementation has the advantage of introducing overtopping discharges in an automated way along the simulation, being robust and computationally efficient.**

I. INTRODUCTION

Coastal and flood modelling is more and more required in a context of sea level rise and increasing frequency of erosion/submersion issues worldwide. A good integration of all physical processes occurring on the sea-land interface in models is crucial for a realistic simulation of water intrusion (overflow, overtopping, erosive processes, breaching etc.) through or over protection systems such as dykes, embankments, tidal gates, dams etc.

In France, regulatory hazard assessment studies – also known as "Etudes de dangers" – are required from communities in order to define the so-called 'protection level', which corresponds to the storm water level (tide + surge) the coastal protection system is able to withstand. In this frame, a set of regulatory scenarios at the protection water level or above (with/without dike, working failure of the hydraulic system, natural breaches etc.) is to be run in order to identify – and account for – the residual water intrusions, if occurring. The modelling steps usually are three-fold : 1/ "upstream" simulation for the determination of wave/level conditions at the toe of structures at each time step, 2/ calculation of overtopping discharge series – using e.g. the EurOtop formulations [1] – along the protection system, 3/ "downstream" simulation wherein discharges are introduced as hydraulic source terms at the rear of structures.

In the framework of a hazard assessment study for the *Syndicat Mixte du Littoral de la Baie du Mont Saint-Michel*, a maritime-terrestrial model was built with the open TELEMAC-MASCARET suite by coupling the TELEMAC-2D and TOMAWAC codes. The coupled model simulates various processes such as tidal oscillation, propagation of offshore sea states up to the coastline, related wave-current interactions and overtopping over structures, which is responsible for the flooding over the protected terrestrial area – if not for dyke breaching. Overtopping is represented as the addition of two specific phenomena: 1) overflow by local exceedance of the structure's crest by the static water level (including wave setup), and 2) overtopping discharge due to waves running up the face of the structure (green water) and/or wave breaking (splash). The first phenomenon is natively handled in TELEMAC-2D by the resolution of the Saint-Venant equations over high-resolution topo-bathymetric relief (Digital Elevation Model, DEM). Due to its dynamic nature, the second phenomenon is dealt with by implementing the EurOtop probabilistic formulae for overtopping discharges, reported as source terms at the rear of the coastal protection system, following previous works (see e.g. [2]). A specific methodology was adopted, which involved the input of structural parameters into the model as well as the implementation of a decision tree for the overtopping discharge formulae to be used in TELEMAC-2D as source term at each time step. Such an approach allows to merge the three steps evoked earlier into one simulation only, which saves a considerable amount of time while rationalising the calculation methodology.

This paper presents the TELEMAC-2D/TOMAWAC model (v7p3) specifically constructed for the hazard study of the Bay of Mont Saint-Michel (Sections II and III) and the implementation of the EurOtop formulations into TELEMAC-2D (Section IV). The promising results obtained for wave overtopping occurrences (among others) in one representative regulatory scenario are then illustrated and commented (Section V). Conclusions are finally drawn with further work avenues (Section VI).

II. STUDY AREA

The bay of Mont Saint-Michel is located in North-Western France, in the English Channel. It extends over about 30km (~500km^2) from Avranches in Normandy to the Grouin Head Point in Brittany. A large part of the bay is a tidal sand-mud flat covered by the sea at high tide and the tidal oscillation can reach 15m in spring tide – the largest one in continental Europe. A continuous protection against flooding made of successive dykes (with or/without road) is present all along the coastline, namely (from West to East, see Fig. 1) : the Duchess Ann dyke (starting South from Cancale), the Western/Eastern polders dykes (at either side of the Mont Saint-Michel) and the Guintre dyke (ending in

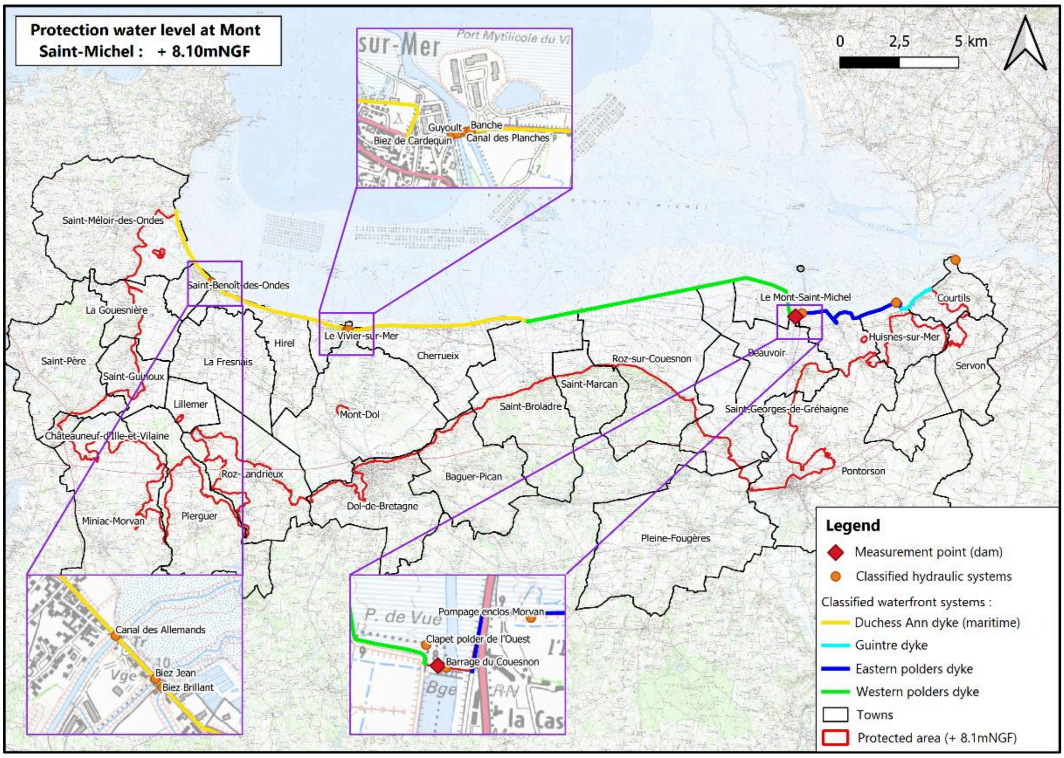


Figure 1 – Area protected from flooding in the Bay of the Mont Saint-Michel and description of the coastal protection system (dykes).

Courtils). In some places, the dykes are bordered by emerged, grassy zones (*schorres* or “herbus”), which can extend up to 2km seawards in the polders zones. Various rivers are flowing out in the bay, among which the Sée and the Sélune in the easternmost part of the bay, and the Couesnon, close to the Mont Saint-Michel rock, whose flow mixes with sea water and is controlled by a dam for sediment flushing purposes. Other hydraulic systems (mechanic/electric gates) are also present along the dykes in order to control the inland drainage. The maximum tidal currents reach 2 to 3m/s in the North of the Mont Saint-Michel so that the flow usually covers the bay at a speed far lower than that “of a galloping horse”, as a famous saying erroneously goes. Swells and storm waves (peak period $T_p > 8$s) mostly come from W-NW ([270°N;300°N]) while wind-sea systems ($T_p < 6$s) from NW to NE generally are observed when offshore wave systems are weak. The largest incoming sea states have a significant wave height H_{m0} of 4-4.5m with typical peak period of 10-11s. In storm conditions, water levels up to +8.50mNGF (+8.34mNGF during Xynthia in 2010) have been recorded at the toe of the Mont Saint-Michel.

III. Setup of the hydrodynamic model

A. Domain extent and meshing

The hydrodynamic model was built by combining two spatial domains – a maritime one and a terrestrial one – , which intersect along the 37 km-long coastal protection system (dykes) containing water works like dams, tide gates, sluices etc. The whole domain covers an area of 893km² (almost one fourth of which is land), including the whole Mont-Saint-Michel Bay from the Grouin Head Point to Granville, offshore, and to the southern coastal lowland, onshore (up to level +9.50mNGF).

The maritime mesh was built with refined resolution around the Grouin Head Point (and the scattered islands located close-by), the Mont-Saint-Michel and Tombelaine rocks. The resolution ranges from 5m in *schorres* and around coastal protection structures to 1 000m offshore.

The terrestrial domain contains a number of drainage channels, hydraulic connections between local water systems and coastal protection dykes as well as inland embankments. Only the main drainage channels (78 over a few thousands, with fine 3m wide / 10m long resolution) and the inland embankments (set back dykes, with 5m wide resolution and 5 parallel constraint lines) were included in the mesh. The lowland default mesh resolution was loosened to 100m for optimisation purposes. The final maritime+terrestrial mesh is composed of 369 112 nodes (Fig. 2).

B. Digital Elevation Model

The Digital Elevation Model (DEM, see Fig. 3) was created by merging two topo- and bathymetric datasets :

- the coastal topo-bathymetric DEM (20m-resolution) partly covering the Normand-Breton Gulf and produced by the SHOM in 2020 [3] ;
- the LIDAR-based DEM of the Bay of Mont Saint-Michel (1m-resolution) produced in the frame of a regulatory action plan against flooding in 2012.

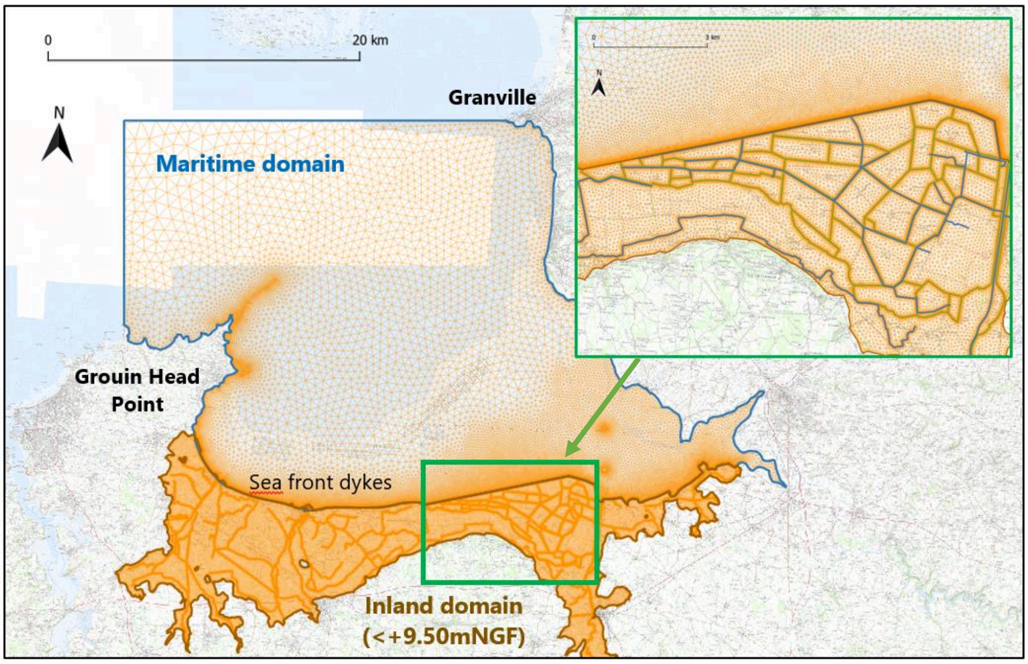


Figure 2 – Mesh of the flood model (maritime + terrestrial) in the Bay of Mont Saint-Michel (369 112 nodes).

The altimetry of the maritime area is entirely based on the 20m-resolution DEM (2020). The altimetry of the terrestrial and intermediate areas is based on the finest (1m) data (2012), although those be less recent.

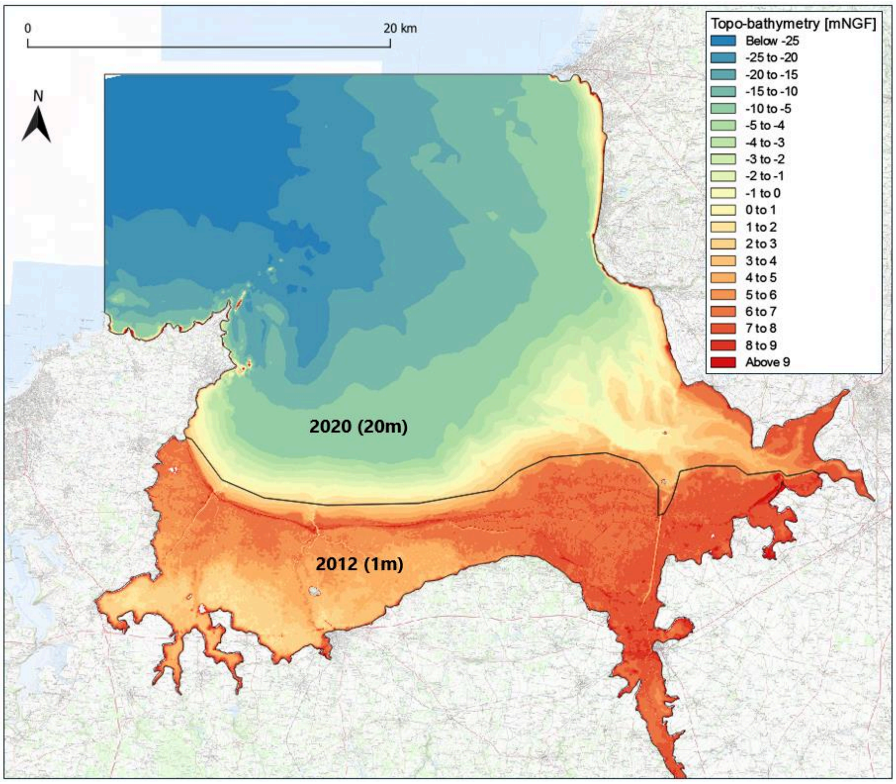


Figure 3 – DEM of the flood model in the Bay of Mont Saint-Michel : fusion of 1m topo-bathymetric data (2012) and 20m bathymetric data (2020).

C. Boundary conditions

The environmental conditions are defined as: 1/ a given water level (protection value Z_w = +8.10mNGF or above) and 2/ sea state parameters (H_{m0}, T_p, θ_p). An analysis based on existing extrapolated data on water level in the bay and HOMERE offshore sea state data [4] was conducted in order to produce water level/significant wave height couples with same return period following the "desk study" method described in [5]. This joint-probability analysis was used to determine adequate scenarios for the hazard study (see §V.A).

As tidal oscillation is required for model validation and running regulatory scenarios, the PREVIMER atlas covering the Western English channel area ("MANW", 250m-resolution) was used to prescribe heights and velocities along the open boundary. To this end, the amplitude and phase of 37 tidal constituents were interpolated on each boundary node.

Sea state conditions were imposed along a part of the liquid boundary only (the north-westernmost one), for sea states are predominantly coming from W-NW. The input offshore sea states were modelled as JONSWAP/$\cos^{2s}$ directional spectra with $\gamma = 1$ (Bretschneider-like shape) and s = 14 – these mean values were determined by inspection of the spectral shapes and mean spreading values provided in the HOMERE dataset. As stormy conditions are simulated only, the assumption of unimodal spectra is valid here.

The effect of wind on currents was not taken into account in the modelling because the potential wind setup possibly generated onshore does not affect the most critical part of the protection system, from which the overall protection level is determined. River outflows (Sée, Sélune) were not taken into account in the model boundaries either, as it was verified, beforehand, that their impact on water level was very limited.

Space-varying bottom friction conditions were defined according to the Corine Land Cover 2018 occupation database [6]. The Strickler coefficients set up over the domain were based on the recommendations of Paris *et al*. [7], as : K = 30m$^{1/3}$/s in salt marsh/grassy areas along the coast, K = 20m$^{1/3}$/s in marshes, fields and polders, K = 10m$^{1/3}$/s in urban areas and K = 40m$^{1/3}$/s on the sea bed (sand-mud, value obtained after calibration, see §III.D.2)).

D. Model calibration and validation

A calibration of two parameters in TELEMAC-2D and TOMAWAC, namely bottom friction coefficient K and breaking parameters γ_2[-], was conducted from available *in situ* datasets in the bay in order to adjust the simulated current velocities and significant wave height to measurements. As it is weakly sensitive to bottom friction, water level was directly validated against reference data.

1) Water level (tide): The simulated water level was compared to two water level datasets: one near Cancale and another one at the St Aubert's chapel, at the toe of the Mont-Saint-Michel. Figure 4 compares the modelled level to *in situ*-validated EPSHOM forecast levels near Cancale on 1-4 January 1991. A very good agreement (R^2 = 0.99) was found,

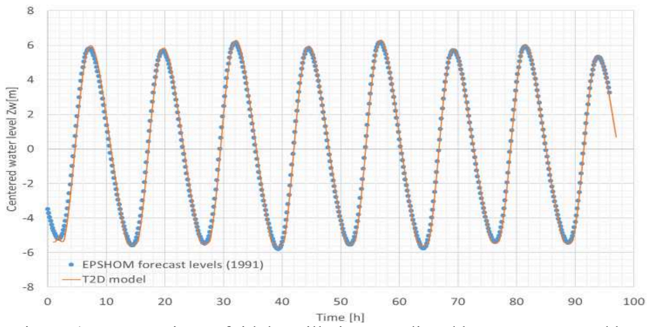


Figure 4 – Comparison of tidal oscillations predicted by EPSHOM and by the flood model near Cancale (1-4 January 1991).

which validated the water level modelling in the western part of the bay.

Figure 5 compares the model results with piezometer measurements at the St Aubert's chapel recorded on 5-12 April 2012 (near highest astronomical tide). The observed level offset at low tide and phase lag are most probably due to topo-bathymetric changes since 2012 (maritime DEM of 2020) and local water catchment. Indeed, sediment dynamics lead to permanent changing in the eastern bay around the Mont Saint-Michel, in particular since the construction of the new access bridge in 2014. The TELEMAC-2D model showed, however, very realistic high tide levels (R^2 = 0.81), which validated the modelling in the eastern part of the bay.

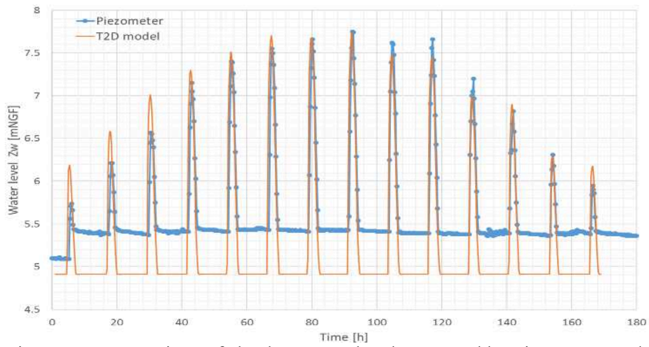


Figure 5 – Comparison of absolute water level measured by piezometer and predicted by the flood model at the St Aubert's chapel (5-12 April 2012).

1) Current velocity: The model was compared to available *in situ* current velocity data, provided by the University of Caen and collected using an electro-magnetic current/wave meter deployed 300m north from the Mont-Saint-Michel between December 2007 and February 2008 [8]. A sensitivity analysis to bottom friction Strickler parameter K was conducted (see Fig. 6). The simulated velocities were found of the same order of magnitude as the measured (depth-averaged) ones and the value K = 40m$^{1/3}$/s was adopted for bottom friction – although this parameter have a limited influence on velocity results in that location.

2) Waves: The same device recorded significant wave height, period and direction data. Because of significant topo-bathymetry changes between 2008 and 2020, and in order to perform relevant comparisons, a depth-based correction was applied to the measured data assuming a conservation of the ratio between significant wave height (H_{m0}) and depth (h).

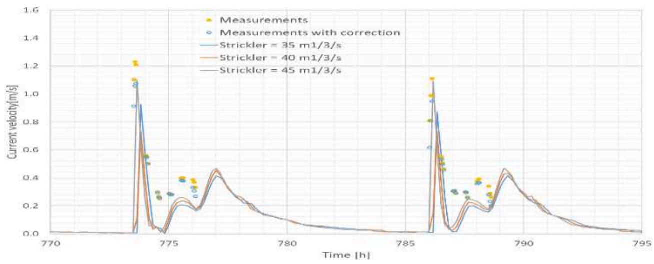


Figure 6 – Comparison of depth-averaged current velocity modulus measured by current meter and predicted by the flood model near the Mont Saint-Michel (~24h in January 2008, [8]).

Also, a sensitivity analysis to breaking parameter γ_2 in TOMAWAC was conducted. Figure 7 depicts the comparison between simulated and measured H_{m0} values around high tide: the obtained range of heights was found in agreement with measurements and the value $\gamma_2 = 0.7$ was eventually adopted. This calibration choice ensured conservative wave penetration and wave breaking modelling over the intertidal zone in view of assessing the performance of coastal protection as regards overtopping and flooding issues.

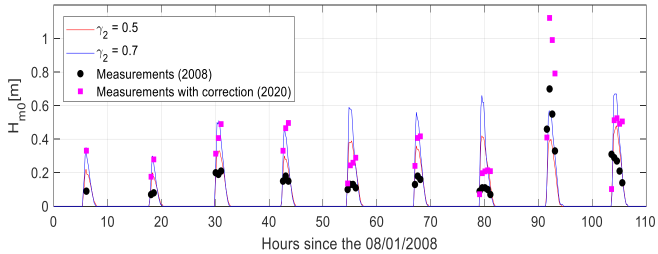


Figure 7 – Comparison of (depth-corrected) significant wave height estimated by wave meter and predicted by the flood model near the Mont Saint-Michel (8-13 January 2008, [8]).

II. Implementation of Overtopping Discharges in TELEMAC-2D

C. EurOtop definitions, formulae and hypotheses

The EurOtop manual [1] gives a set of empirical probabilistic formulations for the analytical calculation of overtopping-related quantities in given sea state and water level, among which discharge q (in m^3/s per linear meter of structure). This quantity includes the phenomena of green water (wave running up on the structure's seaward face and passing over the crest, see Fig. 8) and the splash effect due to wave breaking on the structure (droplets passing over due to momentum or wind). This wave-related overtopping quantity is different from overflow, schematically illustrated in Fig. 9. The input parameters in the EurOtop formulae therefore are related to hydrodynamics (Table 1) and geometry (Table 2) parameters and some influence factors related to slope roughness, wave attack, presence of a wall or promenade etc. (Table 3).

The basic formulation for the overtopping discharge q is :

$$\frac{q}{\sqrt{g\cdot H_{m0}^3}} = A\cdot exp\left[-\left(B\cdot\frac{R_c}{H_{m0}\cdot\Pi_j\gamma_j}\right)^C\right] \qquad (1)$$

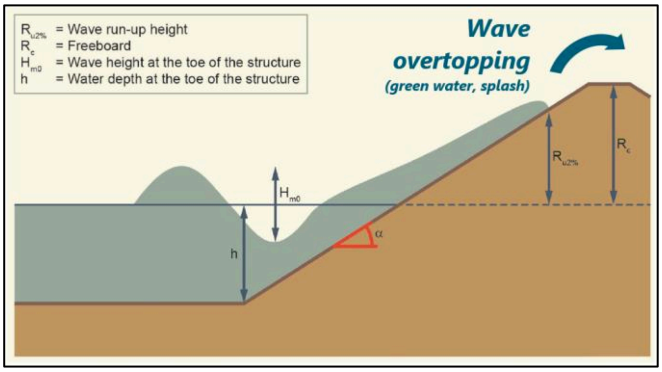


Figure 8 – Sketch of wave overtopping phenomenon [adapted from EurOtop 2018].

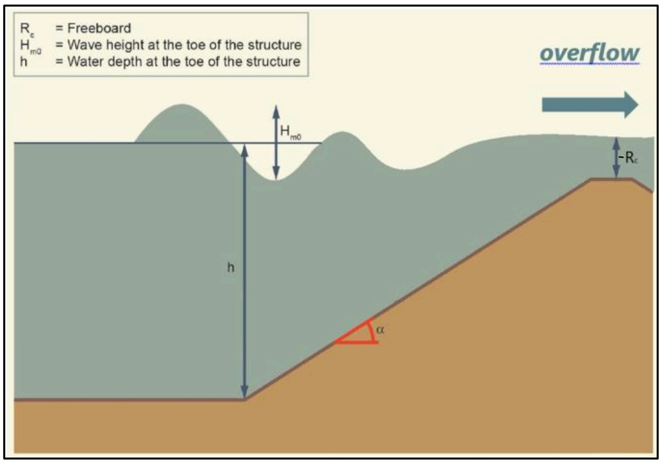


Figure 9 – Sketch of overflow phenomenon [adapted from EurOtop 2018]; Rc < 0 here.

where A, B, C are (empirical) constant values or functions of other input parameters, according to the case. Parameters γ_j stand for appropriate influence factors, where required. The parameterisation depends on the type of structure (coastal dykes and embankment seawalls, armoured rubble slopes and mounds, vertical and steep walls, respectively in chapters 5, 6 and 7 of the EurOtop manual), and roughness (γ_f), geometry (cot α, berm…) and hydrodynamic parameters (e.g., ξ_{-10}).

Table 1 – EurOtop hydrodynamic input parameters (at the toe of structure)

Symbol	Unit	Name/ Formula
H_{m0}	m	Spectral significant wave height ($\neq H_s$)
T_p	s	Spectral peak period
T_{-10}	s	Energy mean period: $T_{-10} = 1.1*T_p$, mean experimental value
T_m	s	Mean period : $T_m = T_p/1.2$, mean experimental value
Z_w	mNGF	Water level (in given vertical reference level)
ξ_{-10}	-	(Energy) Breaker parameter : $\xi_{-10} = \tan\alpha/(H_{m0}/L_{-10})^{0.5}$
L_{-10}	m	(Energy) Mean wavelength : $L_{-10} = g*T_{-10}^2/(2\pi) \sim 1.56*T_{-10}^2$
s_{m-10}	-	(Energy) Mean steepness : $s_{m-10} = H_{m0}/L_{-10} = 2\pi*H_{m0}/(g*T_{-10}^2)$

TABLE 2 – EUROTOP GEOMETRY INPUT PARAMETERS

Symbol	Unit	Name/ Formula
cot α = 1/tan α	-	Structure slope (cot α = 0 corresponds to a vertical wall)
Z_c	mNGF	Crest level (in given vertical reference level)
R_c	m	Crest freeboard of structure : $R_c = Z_c - Z_w$
H_{wall}	m	Height of storm wall on top of slope or at promenade
G_c	m	Promenade width
B	m	Berm width (horizontal)

TABLE 3 – EUROTOP INFLUENCE FACTORS

Symbol	Unit	Name/ Formula
γ_b	-	Influence factor for a berm
γ_f	-	Influence factor for the permeability and roughness of or on the slope
γ_β	-	Influence factor for oblique wave attack
γ_v, γ^*	-	Influence factors for a vertical storm wall on the slope and/or a promenade

When the static water level exceeds the structure's crest, i.e., when $R_c < 0$ (negative freeboard), the EurOtop discharges may still be valid by considering $R_c = 0$. Hence, the formulae do not depend on freeboard any longer. This assumption has a limit however, as suggested by Hughes and Nadal [9], given as:

$$R_c/H_{m0} < -0.3 \qquad (2)$$

meaning that as soon as the dimensionless freeboard (R_c/H_{m0}) is lower than -0.3, the residual wave overtopping may be seen as negligible.

In this work, the *mean value approach* formulations of the EurOtop manual have been implemented as the goal is to perform realistic simulations of hydraulic processes on the sea-land interface. However, in a conservative approach, the definition of influence factors was made simpler by considering no berm, storm wall or promenade ($\gamma_b = \gamma_v = 1$) and frontal wave attack ($\gamma_\beta = 1$). Only roughness/permeability factor γ_f was varying according to the structure (e.g., $\gamma_f = 1$ for concrete/grass slopes, $\gamma_f = 0.40$-0.60 for rocky layers etc.).

D. Methodology and implementation

The first task consists in subdividing the coastal protection system into a set of individual sections (length of e.g. some tens to some hundreds of meters) where local hydrodynamic, structural (roughness, berm…) and geometrical (crest width, slope…) characteristics are deemed homogeneous. An example along the Duchess Ann dyke is given in Fig. 10.

Then, the implementation of overtopping discharges in TELEMAC-2D simply boils down to introducing a set of culverts along the coastal protection system. The main difference with the default culvert module is the fact the discharges do vary in time and depend on TELEMAC-2D and TOMAWAC hydrodynamic variables calculated by the model at each time step (namely Z_w, H_{m0}, T_p…). For each section, these variables are extracted at a given "upstream" node (standing for "toe" conditions as required in the EurOtop manual) and used for calculating the corresponding wave overtopping discharge (in a specific routine), which is then conveniently prescribed as a source point in a given "downstream" node at the rear of the structure (see Fig. 11). The location of both the up- and downstream nodes is freely defined by the user. Here, it is taken approximately between 10 and 25m at both sides of the dykes' crestline depending on the topo-bathymetry and dyke slope.

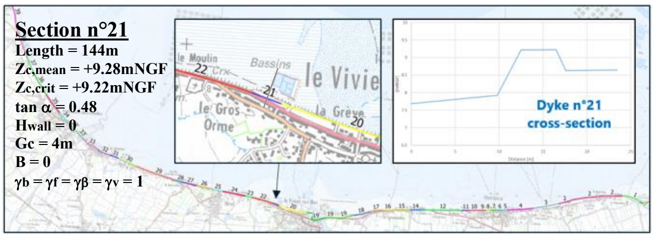


Figure 10 – Map of coastal sections of the Duchess Ann dyke and focus on section n°21 (cross-section and characteristics).

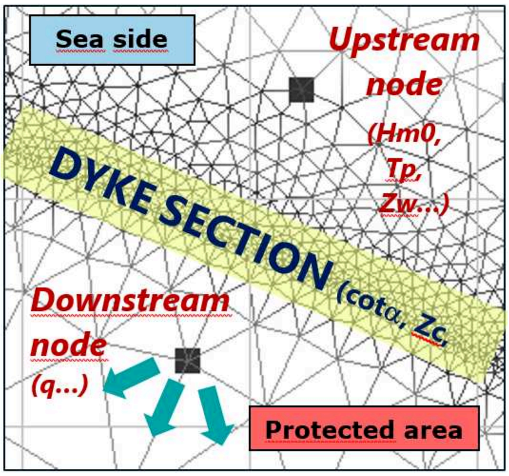


Figure 11 – Sketch of sinks/sources implemented in TELEMAC-2D for overtopping discharge calculation.

The main routine to be modified in TELEMAC-2D is PROSOU, wherein wave parameters have been made available (HM0_TEL, TPR5_TEL…). As the model is expected to be run in parallel mode, it must be ensured that the couple of upstream/downstream nodes defined for each section complies with the partitioning scheme, that is, both nodes belong to the same submesh. Otherwise, the simulation is forced to crush.

A specific output variable is created in order to allow for extracting time series of wave overtopping discharge over each section, which can also be useful for checking the order of magnitude of the computed values.

Such an implementation therefore is straightforward. The limitations may be, however, the fact the realism of the model rests on a sufficiently accurate dyke sampling (one source point per section only), which means a larger effort has to be made prior to running the simulation, so that a satisfying trade-off is found. Also, the location of both up- and downstream nodes might be sensitive. In any case, the upstream work devoted to the discretization of the coastal protection system is fundamental for a good accuracy of the model and should require due attention.

III. SIMULATIONS

A. Simulated scenario

Among the regulatory scenarios simulated in the frame of the risk study, scenario n°3 (SC3) is considered here for illustrative purposes. The scenario corresponds to a 50-year return period event (Z_w = +8.40mNGF in Mont-Saint-Michel, H_{m0} = 2.1m, T_p = 15s et θ_p = 290°N) and includes structural failures (breaches) throughout the coastal defence system. A 24-hour high spring tidal event was simulated and a constant surge component was added to the tidal level so that the extreme water level is reached in the Mont Saint-Michel approximately 5 hours after the simulation start. The surge then decreased linearly up to the second tidal peak where it is zero from there on. Such a definition allows a better understanding of the flood expansion over the lowland topography right after the storm peak (i.e., the first high tide in the simulation). The prescribed water level was adjusted so that the highest level simulated at the Mont Saint-Michel matched the target value (Z_w). Due to local transformation effects in the bay (flow, wave setup…), indeed, the water level is always higher on the coast and varies alongshore: high tide is e.g. delayed by approximately 30 min between the Mont Saint-Michel and the Nielles, in the western part of the bay.

SC3 is expected to exhibit three types of phenomena: 1/ direct wave overtopping discharges inducing (limited) flooding at the rear of the coastal defence, 2/ direct overflow and 3/ breaching occurrences due to large overtopping discharges. The breaching scenario is based on a preliminary analysis – a regulatory part of the risk study, – which aims to characterise the incident hydraulic conditions at each coastal section in the various storm events previously defined. This means each storm event – among which, that of SC3 – was simulated for this purpose (here, on the maritime domain only and over a shorter time span, up to the first high tide) and the overtopping module was used in order to produce the discharge curves related to the critical crest level of each coastal section. According to these results and to the EurOtop recommendations for maximum tolerable discharge regarding structural design (e.g. typical value of 5l/m/s over seawalls with grass covered crest and landward slope), the risk of potential breaching could be assessed. In SC3, 21 natural breaches have been set up in TELEMAC-2D among which 16 are assumed to be due to an excess of overtopping discharge. The other breaches defined in the scenario were due to other types of risks (e.g. excessive overflow, toe scouring, slipping or external/internal erosion, etc.). For all breaches, the embedded breaching module was used with trigger criterion based on water level in front of the structure (to the so-called "safety level"). Figure 12 shows the location of the breaches with highlight on those specifically initiated by overtopping (red line) and those due to other type of risk (orange line). Some of them are located in the westernmost part of the bay (Nielles, Le Vivier-sur-Mer) while others are located in the polders region. Along the rest of the coastal defence system (green line), no structural failure is expected but flooding due to overtopping discharge may occur, namely on those sections indicated by an arrow.

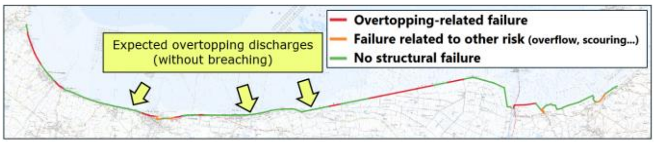


Figure 12 – Location of breaches along the coastal defence system in the bay of Mont St-Michel in SC3; location of expected wave overtopping discharges not causing breaches.

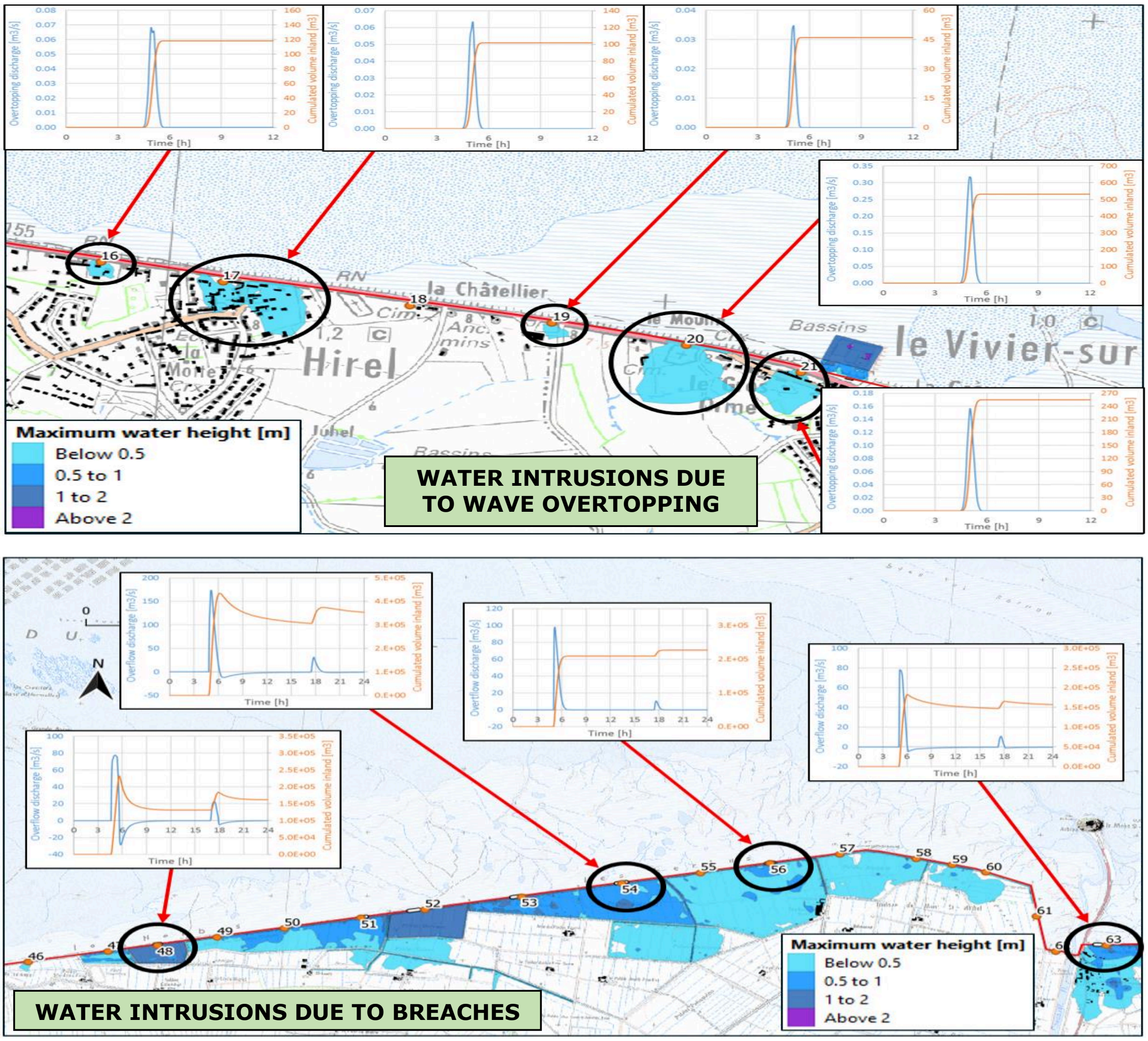
B. Results

1) Direct wave overtopping discharges: Figure 13a illustrates the maximum water height reached during the 24-hour simulation along a part of the Duchess Ann dyke, where inland expansion due to wave overtopping is observed, namely on sections 16, 17, 19, 20 and 21. The orange points denote the downstream node of each coastal section. As the tidal range is large, the overtopping duration is rather short – approximately 1.4h according to the discharge and cumulated volumes curves plotted in the figure. In every such section, only one discharge peak is obtained during the simulation, which occurs at the first high tide – around the 5th hour – when water level is highest. The largest discharge value – exceeding 0.30m^3/s along the section – is obtained in section 20, with a final transmitted volume of water of about 530m^3. The implemented overtopping module therefore looked to behave satisfactorily.

2) Breaches: Lower crested dykes and breaches let large volumes of water flow into the terrestrial domain as compared with overtopping discharges, as shown in Fig. 13b. The order of magnitude is 10^{5}m^3, which is 1 000 times as big as those due to overtopping discharges: as expected due to breaching assumptions, overflow is the dominant phenomenon causing flooding in the protected area of the bay of Mont St-Michel in this scenario. The curves exhibit an overflow discharge peak as soon as the breach is triggered. The discharge generally becomes negative after the first peak because a part of the water volume returns to the sea with ebb-tide. Section 56 is an exception as the water expansion is too wide (with the aid of the drainage channel) to let a significant volume of water flow back into the sea. A second, smaller overflow discharge peak is also observed at the second high tide due to the locally lowered altimetry.

IV. CONCLUSIONS

A coastal flood model including automatic overtopping discharge calculation following the EurOtop formulations has been constructed and successfully run in the frame of the hazard assessment study of the Bay of Mont Saint-Michel. To this end, the coastal protection system was discretised as a sequence of homogeneous sections whose characteristics were introduced in the model. The flooding occurrences due to overflow, wave overtopping discharges and overtopping-related breaches could be satisfactorily simulated and analysed in storm conditions. Based on these first encouraging results, the methodology and related implementations can be extended and improved by considering more coastal contexts and types of structure, involving more EurOtop parameters and quantities.



(a)

(b)

Figure 13 – Examples of water intrusions at the storm peak (maximum water height) due to wave overtopping (a) and wave-overtopping-related breaches (b) simulated by the flood model in SC3; instantaneous discharge (left axis) and cumulated volume inland (right axis) curves.

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