

## FORESHORE EVOLUTION AND HYDRAULIC PERFORMANCE OF A BEACH PROTECTED BY A NEARLY VERTICAL SEAWALL

Riccardo Briganti (1), Rosaria Ester Musumeci (2), Jentsje van der Meer (3), Alessandro Romano (4), Laura Maria Stancanelli (5), Matthias Kudella (6), Rizki Akbar (7), Ryard Mukhdia (8), Corrado Altomare (9), Tomohiro Suzuki (10), Paolo De Girolamo (11), Giulia Mancini (12), Giovanni Besio (13), Nicholas Dodd (14), Stefan Schimmels (15)

(1) Department of Civil Engineering, University of Nottingham, United Kingdom  
[riccardo.briganti@nottingham.ac.uk](mailto:riccardo.briganti@nottingham.ac.uk)

(2) Dept. of Civil Engineering and Architecture, University of Catania, Italy, [rmusume@unict.it](mailto:rmusume@unict.it)

(3) IHE Delft Institute for Water Education, The Netherlands

(4) Dept. of Civil, Building and Environmental Engineering (DICEA), "Sapienza" University of Rome, Italy

(5) Dept. of Civil Engineering and Architecture, University of Catania, Italy

(6) Forschungszentrum Küste (FZK), Hannover, Germany

(7) IHE Delft Institute for Water Education, The Netherlands

(8) IHE Delft Institute for Water Education, The Netherlands

(9) Universitat Politècnica de Catalunya, Barcelona, Spain

(10) Flanders Hydraulics Research, Antwerpen, Belgium

(11) Dept. of Civil, Building and Environmental Engineering (DICEA), "Sapienza" University of Rome, Italy

(12) Department of Civil Engineering, University of Nottingham, United Kingdom

(13) Department of Civil, Chemical and Environmental Engineering (DICC), University of Genoa, Italy

(14) Department of Civil Engineering, University of Nottingham, United Kingdom

(15) Forschungszentrum Küste (FZK), Hannover, Germany

This work presents the results of an experimental investigation on the effects of a sequence of storms on wave overtopping at a nearly vertical battered seawall at the back of a sandy foreshore. The experiments were carried out in the Large Wave Flume (GWK) at Leibniz Universität Hannover (Germany), as part of the research project ICODEP (Impact of Changing foreshore on flood DEfence Performance), within the European Union programme Hydralab+.

The layout consisted of a 10/1 battered seawall and a natural sandy foreshore with an initial 1:15 slope. The beach sand had a nominal diameter of 0.30 mm. Three sequences of individual storms were simulated. Each storm was divided into six steps in which the wave conditions and still water level were varied to represent the peak of an actual storm. Two storm profiles were considered, the first one with a lower level of energy and the second one with a higher one. These were combined in the three different sequences. All the tested wave conditions were designed to be erosive for the beach, with no recovery in between. Each sequence started from a plain beach configuration and the beach was not restored in between storms. The measurements included: the profile of the beach after each sea state tested, waves, wave pressure and forces at the sea wall, sediment concentrations, flow velocity, and wave overtopping.

### 1. INTRODUCTION

Coastal flood defence structures are often found at the back of a natural beach, and an interaction between the beach and the structure itself determines the joint performance of this system. With an increased emphasis on natural and mixed hard/soft coastal management approaches, the prediction of the performance of protected beaches received more attention by the coastal research community.

One important aspect that needs to be investigated is the influence of sequences of storms on the general behaviour of the system. Sequences of storms separated by intervals that are shorter than recovery time of the beach are often referred to as clusters. Existing investigations on this subject analyse natural beaches and very little research is available on the behaviour of engineered ones. The focus of the existing research is to understand if the cumulative impact of clusters is higher than the sum of the average impact of the individual events, as identified in the review by Senechal et al (2017), which presents a comprehensive overview of the literature on this topic.

The enhancement of the impact of storms of moderate intensity found for natural beach opens to the possibility that the same can happen for coastal defences. This occurrence has never been

investigated in depth. The project ICODEP (Influence of foreshore evolution on COastal DEfence Performance) aims at filling this gap in research.

Wave overtopping at a composite flood defence during a sequence of storms was investigated with a large-scale laboratory test in the Large Wave Flume (GWK) at Leibniz Universität Hannover (Germany).

## 2. EXPERIMENTAL LAYOUT

The experimental layout (Fig. 1) consists of a 10/1 sloped seawall and a sandy foreshore with an initial 1/15 slope. The natural sand of the beach has a nominal diameter ( $D_{50}$ ) of 0.30 mm. The reference frame used in this work has the origin of  $x$  at the neutral position of the wave paddle and the origin of  $z$  is set at the bottom of the flume at the toe of the beach, which is at  $x=161.9$  m. The  $y$  coordinate origin is located on the lateral instrumented wall of the flume, i.e. on the hydraulic right side of the wall (see Fig. 2). The coordinates of crest of the seawall are  $x=240.93$  m and  $z=5.5$  m (see Fig. 2).

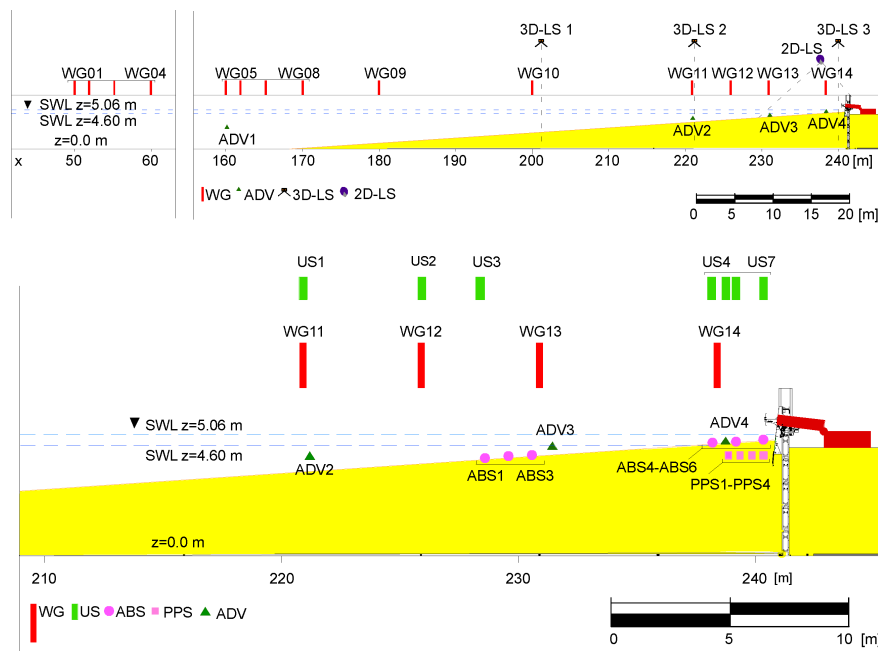


Figure 1: Layout of the ICODEP engineered beach and instruments. Upper panel: overview of the flume. Lower panel: detail of the structure

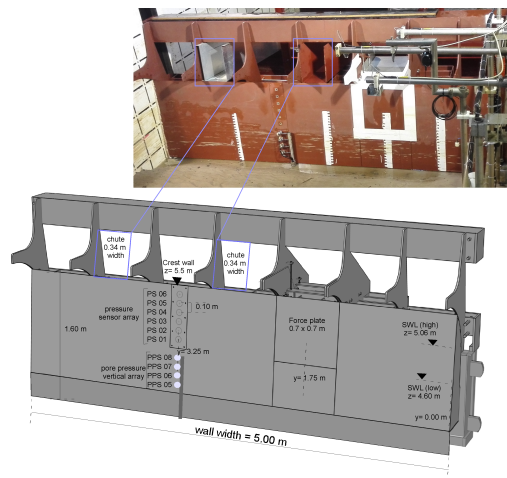


Figure 2: 3D view of the tested seawall with the position of the instruments installed.

The tested wave conditions have been chosen to be representative of those of typical storms in macro-tidal Northern European seas. However, in large scale laboratories, it is not feasible to

replicate a sequence of complete storms, hence the focus was on the peak of each storm and the simultaneous peak of the sea levels.

Each storm profile was divided into six sea states, referred to as sea states T1 to T6. They were generated using the JONSWAP spectrum with a shape factor  $\gamma=3.3$ , consistent with the capability of the wave generator at the GWK. The duration of each sea state is  $D_s=32$  minutes. Fig. 3 shows the target significant wave height ( $H_{m0}$ ), peak period  $T_p$  and Still Water Level (SWL) in all trunks in S1 and S2. The sequence of individual waves for each sea state is kept the same throughout the tests, in order to remove the variability due to the particular sequence of waves at the boundary (Romano et al., 2015).

Two SWL conditions were applied during each storm; these were the same for storm S1 and S2. Sea states T3 and T4 were always tested with the SWL at 5.06 m from the bottom of the flume (referred as high SWL conditions hereinafter), and the rest of the storm was tested with the SWL at 4.60 m (low SWL hereinafter). As shown in Fig. 3 in both S1 and S2,  $H_{m0}$  increases from the first to the second segment of the storm (T1 and T2) at low SWL conditions, it reaches a maximum at T3 in high SWL conditions and decays afterwards from T4 to T6. The coincidence of maximum  $H_{m0}$  and maximum SWL was chosen because maximum overtopping is expected to occur when high tide and the maximum significant wave height occur at the same time. This was the case of the storm on the east coast of England (U.K.) that occurred from 5<sup>th</sup> to 6<sup>th</sup> December 2013, as discussed in Dissanayake et al. (2015), when the peak storm wave height coincided with high-water during spring-tide.

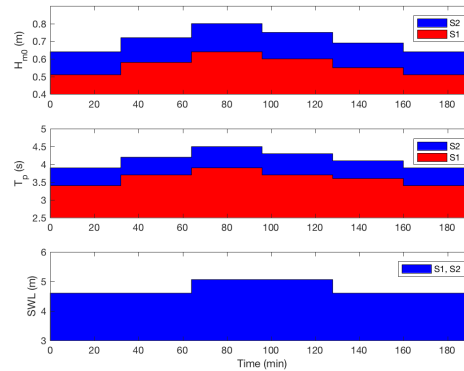


Figure 3: Characteristics of the simulated storms.  
Upper panel:  $H_{m0}$ ; middle panel:  $T_p$ ; lower panel: SWL

S1 and S2 have been combined into three sequences (see Table 1). Each sequence started from a 1/15 plain slope, while the beach profile after each storm was the initial one of the following storm. In C1 the repetition of the most energetic storm was tested, in order to measure the effects of previous bed evolution on the same storm. Conversely, C2 and C3 were meant to test the effects of storms of different energy levels alternating in a sequence.

Sequence	Position	Storm Profile	Initial beach profile
C1	1	S2	Slope 1/15
	2	S2	As evolved due to previous S2
	3	S2	As evolved due to previous S2
C2	1	S2	Slope 1/15
	2	S1	As evolved due to previous S2
	3	S2	As evolved due to previous S1
C3	1	S1	Slope 1/15
	2	S1	As evolved due to previous S1
	3	S1	As evolved due to previous S1

Table 1: Storms and initial bed conditions for the three sequences.

In this paper each storm sea state will be referred using the two character codes in a sequence, separated by "-", in which first the sequence is indicated, then a number from 1 to 3 is used to identify the position of the storm in the sequence, followed by the codes for the storm and that of the sea state. For example, C3-2-S2-T4 indicates sea state T4 of the storm S2 that was tested as second in sequence C3.

### **3. HYDRODYNAMICS MEASUREMENTS**

The measurement of the free surface was carried out using 14 resistive wave gauges (WG) and 7 ultrasonic wave gauges (US). The positions of these instruments are shown in Fig. 1.

An array of four WGs (WGs 01 to 04) was located at 50 m from the wavemaker and it was used to compute the actual incident wave conditions using the Mansard and Funke (1980) method at the paddle. A second array of 4 WGs (WGs 05 to 08) was located at the toe of the beach starting from  $x=160$  m, WGs 05 to 07 were used to compute the actual incident wave conditions at the toe of the beach. Along the beach, WGs 08 to 14 were located in the shoaling and surf zone, while all seven USs are close to the structure. Note that US 1 and US 2 were co-located with WGs 10 and 11 respectively. It should be noted that WG 14 did not function properly during the experiments and it will not be included in the analyses.

An additional source of data for the free surface comes from one of the eight pore pressure sensors (PPSs) installed in the beach close to the toe of the structure (See lower panel of Fig.1). Four of these sensors (PPS 01 to 04) were installed along a horizontal array, always buried in the beach, while PPS 05 to 08 were located on a quasi-vertical array on the structure (see Fig. 2). These latter four sensors were exposed when erosion developed. In particular, it was observed in a series of preliminary tests that PPS 08 was exposed after a few waves when erosion started.

Four Acoustic Doppler Velocimeters (ADV) were used to measure local flow velocity, these were located along the whole active part of the beach.

### **4. BEACH PROFILE AND SEDIMENT TRANSPORT MEASUREMENT**

The beach profiles were obtained by a combination of data gathered by a mechanical wheel profiler, a Sick LMS291-S14 Two-dimensional time of flight laser scanner (2D-LS hereinafter), and a Faro Focus 20/120 three-dimensional laser scanner (3D-LS hereinafter).

The mechanical beach profiler was run to measure the initial planar sloping beach and the profile after each sea state condition, from  $x=100$ m up to about  $x=240.3$ m, with a spatial resolution of  $\Delta x=0.25$ m.

Point clouds of the bottom have been obtained by means of the 3D-LS, which was moved at three measuring stations along the flume (see upper panel of Fig. 1) and calibrated using the proprietary Faro software and the known coordinates of several markers in the lab. The resolution of the point clouds was  $O(0.01 \text{ mm})$ .

3D reconstructions of the entire sandy bottom ( $x>190$  m) were obtained both at the beginning and at the end of each storm sequence, when the flume was drained before beach reshaping. Additional 3D-LS measurements of the region close to the wall,  $231.5\text{m}<x<241.0\text{m}$ , were obtained in dry conditions in the upper part of the beach, i.e. at the end of every even sea state, except for T3, when a dry condition was not reached. In particular, the beach profile was extracted from the 3D point cloud by span-wise averaging the points gathered within a central stripe,  $2.49\text{m}<y<2.51\text{m}$ .

2D-LS measurements running continuously during each test provided information on the foreshore evolution in proximity of the wall as well as a further measurement of the moving free surface. These measurements were calibrated using three metallic poles located at the center of the measurement region. The coordinates of the poles were known with topographic precision. The 2D-LS had an angle resolution of  $0.1667^\circ$  and a measuring angle of about  $115^\circ$ . The measuring range was  $228.5\text{m}<x<243.5\text{m}$ .

Due to physical constraints, the mechanical wheel profiler could not reach the toe of the wall, located at about  $x=240.9$ m. The data on the near-wall bed profile obtained by the 3D scanner, for the initial condition and at the end of sea states T2, T4 and T6 were used to complete the beach profile for the last 60 cm. For sea states T1 and T5 the data of the 2D scanner were used for the same purpose.

Due to the presence of water, it was not possible to have data on the bottom morphology at the toe of the wall at the end of all the sea states T3, in high SWL conditions, and for some of the T5 sea states, as the scour was flooded notwithstanding the fact that the SWL was lowered.

Analysis of the data revealed that the profiles recovered by the wheel profiler were systematically lower, O(2 cm), than those recovered by the calibrated 2D and 3D laser scanners. On the other hand, the latter ones matched fairly well with each other. The mismatch is probably due to the burial of the wheel of the profiler in the sandy bottom. Therefore a correction was made of each bed profile measured mechanically.

The complete beach profiles were then obtained by joining the offshore part of the wheel profiler data and onshore part of the 3D/2D scanners data.

Finally, the scour evolution was also monitored by the same video camera used for wave overtopping. To this end, four rulers are attached to the sloping wall, in order to have a visual reference of the scour depth.

Sediment transport was measured by two Acoustic Backscattering Sensors (ABS), one located in correspondence of the surf-zone bar and the second close to the structure.

## 5. OVERTOPPING MEASUREMENTS

A gravimetric overtopping tank with a volume of approximately  $1.40 \text{ m}^3$  was used to collect the volume of water due to overtopping. The mechanism consisted of two tanks (inner and outer) and a connection to the crest of the wall by means of one or two chutes, depending on the expected overtopping discharge.

The inner tank was placed on four force transducers (the positions of which are indicated in the additionally provided spreadsheet) that measured its weight during the tests. The outer tank provided a dry area around the inner tank to put the sensors. Also, a pumping system was used to pump the collected water during the experiment, which was activated manually, with the trigger signal of the pump being recorded to identify the intervals of usage.

The chute (0.34 m wide) was placed at the inner edge of the crest of the middle of the wall, connecting the structure and the inner tank. The number of chutes used depended on the water level. During low SWL sea states, two chutes were used, as opposed to high SWL conditions, in which only one was used (with the only exception of storm C2-1-S2-T3 in which two chutes were used).

A pressure transducer (PS), referred as PS 07, was installed at the crest of the sloping wall, in the central chute, in order to make an estimation of the number of wave overtopping events. Furthermore, two synchronised video cameras recorded the overtopping events. The first camera was located at  $x=225 \text{ m}$ , at the level of the aisle along the flume, and the second one located close to the roof, at  $x=238 \text{ m}$ . Finally, in order to avoid problems due to the presence of the top beam of the structure, a plywood panel was installed on top of the beam that directed the entire overtopping volume within the chute.

## 6. FORCES AND PRESSURE MEASUREMENTS

In order to measure pressures on the wall, an array of six PSs (1 to 6) was installed on the wall (see Fig. 2). The PSs are spaced about 0.1 m from each other. Forces were measured on the wall using a plate at which 5 load cells were connected (see Fig. 2).

## 7. DYNAMICS OF THE PROTECTED BEACH

The foreshore evolution within a storm is studied by analysing the bed change after each sea state with respect to the initial bed profile  $\Delta z_b = z_b(t_e, s) - z_b(t_0)$ , where  $z_b$  is the bed level from the bottom of the flume,  $(t_e, s)$  is the end time of each sea state ( $s=1, 2, \dots, 6$ ) and  $t_0$  is the start time of each sequence, when the beach had its 1/15 slope. The intra-storm evolution of the bed is qualitatively similar in all cases. Fig. 4 shows for  $\Delta z_b$  the first storm of C1 (i.e., C1-1-S2) as an example. At the start of the sequence, during T1, a bar started developing and grew in the subsequent sea states. At the same time erosion developed in the region between the bar and the wall, approximately between  $x=230 \text{ m}$  and  $x=240.86 \text{ m}$ . The evolution close to the structure is more complex as accretion occurred during the first storm of the sequence. During T2 the bar grew and migrated offshore, while the erosion of the region between the bar and the structure increased.

In the two high SWL sea states significant erosion developed near the toe of the structure. Also, it appears that partial backfilling occurred and  $\Delta z_b$  reached almost zero around  $x=236$  m during T4. The backfilling in this region reversed to erosion in the subsequent T5, while partial filling of the scour at the toe of the seawall and continued in T6.

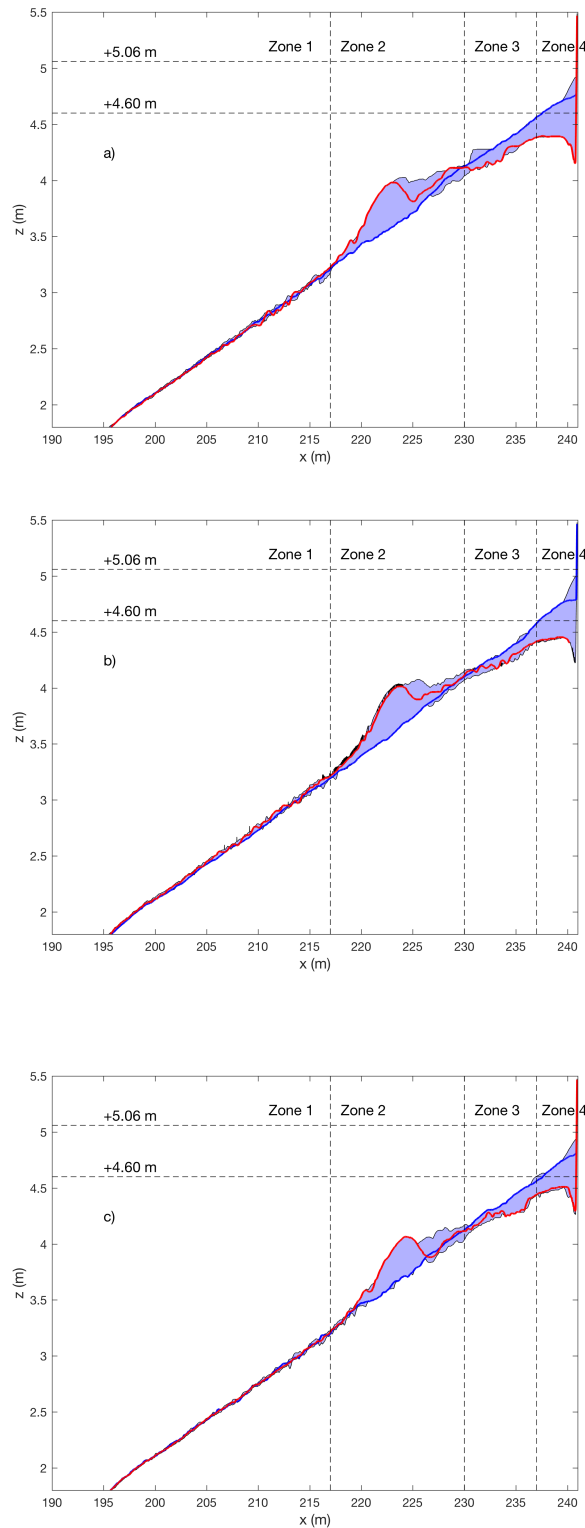


Figure 4: Zones of the beach evolution for sequence. Blue line: initial profile, red line: end profile. The blue shaded area is the range of the bed level at each position. a) C1 b) C2, c) C3.



The second storm of the sequence (i.e. C1-2-S2) started from a barred beach configuration. The bar continued to grow and migrates offshore, as shown in Fig. 4.  $\Delta z_b$  is negative overall close to the structure, however, during high SWL, the morphodynamics of the region between the bar and the seawall is more complex, with deposition occurring just offshore the scour area. Fig. 4 shows the initial and final profile for the three sequences together with the range of  $z_b$  reached. Results are consistent for all three sequences and the evolution of the beach can be described by dividing it into four zones. Zone 1 is the ripple region extending approximately from  $x=200$  m and  $x=217$  m. Zone 2 is the bar region extending approximately from  $x=217$  m and  $x=230$  m in C1 and C3; here the bar formed and migrated offshore during each sequence. The boundary between Zone 2 and 3 is a pivot around which the beach rotated. Zone 3 is characterised by overall erosion with respect to the initial profile, as the figure shows. However, in Zone 3 backfilling occurred at high SWL conditions as shown in Fig. 4; this could be large enough to establish a secondary bar above the original profile as the results for C1 (panel a of Fig. 4) show. The slope of Zone 3 was milder than the original 1/15 and appears to be almost zero in the upper part of this region in all sequences. This "terrace" formed at the position of the initial shoreline at the lower SWL. This is considered as the boundary with Zone 4. Here both swash motion and scour have an important role in modelling the beach.

## 7. DISCUSSION AND CONCLUSIONS

The ICODEP experiments investigated the evolution of an engineered beach during sequences of storms of different energy content. The analysis of the data reveals that the order of the storms has an important role in determining the beach profile. Further results (not shown here) show that, after the first storm of the cluster in which the bar rapidly forms, the bar offshore movement slowed down and its position only slightly changed during the rest of the sequence. The most active region, and the one in which the effect of the cluster is greater, is the one between the bar and the seawall itself. This region affects, in turn, the wave transformation after the passage of the bar, determining different levels of overtopping in different sequences.

This dynamics challenges existing predictive tools because of the need of knowing the initial beach profile and how this evolves during a storm. Therefore, an analysis of existing empirical and numerical prediction tools is a necessary step to support the improvement of natural/engineered coastal defence design.

## ACKNOWLEDGEMENT

We thank the staff of the GWK at Leibniz Universität Hannover (Germany) for their invaluable contribution to the experiments.

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No 654110, HYDRALAB+.

## REFERENCES

- Dissanayake, P., Brown J., Wisse, P., and Karunaratna, H. (2015). Effects of storm clustering on beach/ dune evolution. *Marine Geology*, 370:63–75, 2015.
- Mansard, E.P.D. and Funke, E.R. (1980). The measurement of incident and reflected spectra using a least squares method. *Proceedings of the 17th Coastal Engineering Conference*, Vol. 1, pp. 154-172.
- Romano, A., Bellotti, G., Briganti, R., and Franco, L. (2015) Uncertainties in the physical modelling of the wave overtopping over a rubble mound breakwater: The role of the seeding number and of the test duration. *Coastal Engineering*, 103:15–21.
- Sénéchal, N., Castelle B., and Bryan K. R. (2017). Storm Clustering and Beach Response, chapter 8, pages 151–174. John Wiley & Sons, Ltd.