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# Wave runup on composite beaches and dynamic cobble berm revetments

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## Abstract

The effects of climate change and sea level rise, combined with overpopulation are leading to ever-increasing stress on coastal regions throughout the world. As a result, there is increased interest in sustainable and adaptable methods of coastal protection. Dynamic cobble berm revetments consist of a gravel berm installed close to the high tide shoreline on a sand beach and are designed to mimic naturally occurring composite beaches (dissipative sandy beaches with a gravel berm around the high tide shoreline). Existing approaches to predict wave runup on sand or pure gravel beaches have very poor skill for composite beaches and this restricts the ability of coastal engineers to assess flood risks at existing sites or design new protection structures. This paper presents high-resolution measurements of wave runup from five field and large-scale laboratory experiments investigating composite beaches and dynamic cobble berm revetments. These data demonstrated that as the swash zone transitions from the fronting sand beach to the gravel berm, the short-wave component of significant swash height rapidly increases and can dominate over the infragravity component. When the berm toe is submerged at high tide, it was found that wave runup is strongly controlled by the water depth at the toe of the gravel berm. This is due to the decoupling of the significant wave height at the berm toe from the offshore wave conditions due to the dissipative nature of the fronting sand beach. This insight, combined with new methods to predict wave setup and infragravity wave dissipation on composite beaches is used to develop the first composite beach/dynamic revetment-specific methodologies for predicting wave runup.

## Keywords

Wave runup; Swash; Dynamic cobble berm revetment; Dynamic revetment; Composite beach; Wave reflection

## 43 1. Introduction

44 Significant progress towards a better understanding of morpho- and hydrodynamic processes on  
45 sandy, and to a lesser extent gravel beaches has been made over the past few decades through a  
46 variety of laboratory, field and numerical investigations. Knowledge of mixed beaches which contain  
47 both sand and gravel is less well advanced however, due largely to the greatly increased complexity  
48 introduced by bi- or multimodal sediment size distributions. Despite the presence of sand, mixed  
49 beaches are commonly considered to be a type of gravel beach, which were divided into three sub-  
50 categories by Jennings and Schulmeister (2002): (1) pure gravel beaches which are not strictly  
51 “mixed” and are dominated by gravel throughout their profile, (2) mixed sand and gravel beaches  
52 which are composed of fully mixed gravel and sand, and (3) composite gravel beaches which have a  
53 two-part profile consisting of a steep upper foreshore or backshore gravel ridge fronted by a lower-  
54 gradient sand slope. Note that following the Wentworth (1922) grain size classification, the term  
55 gravel is used to describe coarse sediments in the size range  $2 \text{ mm} < D_{50} < 256 \text{ mm}$  which is  
56 subdivided into granules ( $2 \text{ mm} < D_{50} < 4 \text{ mm}$ ), pebbles ( $4 \text{ mm} < D_{50} < 64 \text{ mm}$ ) and cobbles ( $64 \text{ mm} <$   
57  $D_{50} < 256 \text{ mm}$ ).

58 This study focusses specifically on a subset of composite beaches (type 3; Jennings and Schulmeister,  
59 2002) that exhibit a permanent ridge of pebbles and/or cobbles situated around mean high water  
60 (MHW) with a clear delineation between the sand and gravel. Such beaches were first defined in the  
61 literature by Carter and Orford (1993) and are commonly found in the UK, USA (West Coast), New  
62 Zealand and Ireland. Note that composite beaches with seasonally transient backshore cobble berms  
63 and with isolated patches of cobbles over the foreshore (e.g. Matsumoto et al., 2020) have also been  
64 identified in the literature. Also, gravel beaches that are sandy below mean low water (MLW) are  
65 common on the south coast of England (e.g. Karunaratna et al., 2012; Watt et al., 2008). While such  
66 beaches are expected to have similarities with composite beaches as defined here, they are not  
67 directly considered in this work.

68 Komar (2005) highlights that composite beaches differ from mixed sand and gravel beaches (e.g.  
69 type 2; Jennings and Schulmeister, 2002) primarily due to the bimodal nature of the sediment size  
70 distribution and a much higher proportion of sand relative to coarse material. Cross-shore sorting by  
71 waves leads to a highly dissipative lower foreshore of sand ( $\tan\beta_{sand} \approx 0.01$  to  $0.05$ ) that is exposed  
72 except at high tide, backed by a steep, permeable and reflective backshore ridge of cobbles ( $\tan\beta_{berm}$   
73  $\approx 0.1$  to  $0.25$ ). The differences in the slope, roughness, permeability, and groundwater characteristics  
74 of the sand and gravel components of composite beaches means that both the hydro and  
75 morphodynamics vary with water level and they behave differently to both pure sand and pure  
76 gravel beaches. Effectively, composite beaches represent a combination of the two most stable  
77 states at either end of the morphodynamic classification of Wright and Short (1984), making them  
78 relatively stable in response to changing hydrodynamic conditions. The dissipative sand beach  
79 component exhibits lower mobility because the low gradient promotes a wide surf zone, within  
80 which wave energy is gradually dissipated leading to smaller short-wave heights at the shoreline.  
81 The steep cobble ridge derives its stability primarily from its porous nature which promotes swash  
82 asymmetry (Masselink and Li, 2001). By contrast, mixed sand and gravel beach morphologies tend to  
83 be more variable as the more uniform grain size distribution means smaller grains fill the voids  
84 between the larger clasts, reducing the porosity of the beach and hence swash asymmetry.

85 Improved knowledge of composite beach processes is valuable because such beaches are relatively  
86 common and are known to be resilient to extreme waves and water levels, but are under-  
87 researched. Furthermore, the inherent stability of composite beaches under large storm waves and

88 the overtopping protection provided by the gravel ridge to the hinterland has inspired coastal  
89 engineers to nourish beaches using gravel or cobbles. The structure created is commonly called a  
90 dynamic revetment, although this term has been used to describe a wide range of structures from  
91 gravel beaches (e.g. Tomasicchio et al., 2010) to cobble berms (e.g. Everts et al., 2002). A summary  
92 of these structures is provided by Bayle et al. (2020), however in line with the definition of  
93 composite beaches considered here, this paper will consider the sub-category termed dynamic  
94 cobble berm revetments by Bayle et al. (2020). These structures are effectively cobble berms  
95 constructed around or above the shoreline at mean high water (MHW) to create an artificial  
96 composite beach. A small number of such structures currently exist, primarily on the west coast of  
97 the USA (e.g. Komar and Allan, 2010; Allan et al., 2005, 2012; Allan and Gabel, 2016; Kaminsky et al.,  
98 2020; Everts et al., 2002), however guidance to design them is limited and relatively little monitoring  
99 of their performance has been undertaken. Some more recent studies have undertaken large scale  
100 laboratory (Bayle et al., 2020) and field experiments (Allan et al., 2006; Bayle et al., 2021) designed  
101 to investigate revetment morphodynamics, wave overtopping, wave-by-wave sediment fluxes and  
102 cobble transport using tracked cobbles.

103 For resilient design of dynamic cobble berm revetments, robust predictions of wave runup are  
104 essential, but as detailed in Section 2 below, are not currently available. This paper will investigate  
105 the different processes which contribute to wave runup including short wave and infragravity swash  
106 as well as wave setup using multiple field and large-scale laboratory datasets. The insight gained  
107 from this analysis is then used to develop new approaches for estimating wave runup on composite  
108 beaches and dynamic cobble berm revetments. The manuscript is organised as follows. Section 2  
109 summarises existing studies of wave runup that are relevant to composite beaches. Section 3  
110 introduces the different field and laboratory experiments from which the data analysed in this paper  
111 were obtained and outlines the analysis methods. Section 4 presents new measurements of swash  
112 motions and infragravity waves at the shoreline on composite beaches. The discussion in Section 5  
113 develops three new approaches of varying complexity to estimate wave runup on composite  
114 beaches based on the results presented in Section 4 and conclusions are provided in Section 6.

## 115 **2. Wave Runup on Composite Beaches**

116 Swash processes are the primary driver of morphology change on composite beaches and large wave  
117 runup events can overtop or overwash the gravel ridge at high tide, which in extreme cases can lead  
118 to barrier breaching. While many empirical equations to predict wave runup on sandy beaches are  
119 available, no composite beach-specific studies of runup have been undertaken.

120 A variety of investigators have examined wave runup on gravel and mixed beaches. Kirk (1975)  
121 presented measurements of wave runup on a mixed sand and gravel beach and found breaker  
122 height to be the primary controlling factor. Powell (1990) developed a predictor for runup on gravel  
123 beaches based on laboratory and field data. Bayle et al. (2020) measured wave runup on a dynamic  
124 cobble berm revetment in a large-scale laboratory experiment and demonstrated that extreme  
125 runup represented by  $R_{2\%}$  - the elevation exceeded by 2% of wave runup events - was reduced  
126 compared to an equivalent sand beach. Billson et al. (2019) measured the infragravity component of  
127 swash on 5 beaches, including two wind-wave dominated composite beaches. They found that  
128 shoreline infragravity energy was low at these sites and existing predictive equations based on  
129 offshore wave conditions had mixed skill with reasonable performance at one beach (Minsmere) and  
130 no skill at the other (Camber). Poate et al. (2016) used extensive field measurements from gravel  
131 beaches in the UK along with synthetic runup data generated using the XBeach-G numerical model  
132 to develop new runup predictors for pure gravel beaches with the form:

133 
$$R_{2\%} = C_p H_0 \tan \beta_{gravel}^{0.5} T \quad (1)$$

134 where  $\tan \beta_{gravel}$  is the gravel beach slope,  $H_0$  is the deepwater significant wave height and  $T$  is either  
 135 the mean zero-crossing ( $T_z$ ) or peak ( $T_p$ ) wave period for which different values of the constant  $C_p$   
 136 were presented ( $C_p = 0.49$  or  $0.33$  respectively). It is noted that the field data from two composite  
 137 beach sites: Westward Ho! and Seascale were eliminated from the analysis because the recorded  
 138 runup values were notably smaller than those recorded on the pure gravel beaches due to the  
 139 dissipative nature of the sandy lower profile, thus Eq. 1 is not considered appropriate for composite  
 140 beaches.

141 If we consider composite beach ridges or dynamic revetments as coastal structures, a range of wave  
 142 runup equations for sloping coastal defence structures exist and typically estimate wave runup  
 143 based on the wave height at the structure toe (e.g. EurOtop, 2018). The EurOtop manual provides a  
 144 general equation to predict wave runup on coastal structures:

145 
$$\frac{R_{2\%}}{H_{m0}} = 1.65 \gamma_f \xi_{m-1,0} \quad (2)$$

146 where  $H_{m0}$  is the spectral significant wave height at the toe of the structure,  $\gamma_f$  is a reduction factor  
 147 to account for the slope roughness and  $\xi_{m-1,0}$  is the Irribarren number based on the mean spectral  
 148 wave period  $T_{m-1,0}$  to calculate wavelength ( $\xi_{m-1,0} = \frac{\tan \beta_{struct}}{2\pi H_{m0}/(gT_{m-1,0}^2)}$ ). Little work has been done to  
 149 determine suitable values of  $\gamma_f$  for gravel or cobble slopes, although recent numerical work by  
 150 Zaalberg (2019) suggested values between 0.62 and 0.75 for a dynamic cobble berm revetment.

151 An earlier version of the EurOtop manual (EurOtop, 2007) also includes an expression for wave  
 152 runup on gravel beaches where it is assumed that the crest elevation of a gravel beach  $h_c$  is  
 153 approximately equal to the elevation of the largest wave runup events and could approximate  $R_{2\%}$ :

154 
$$R_{2\%} \approx h_c = C_e H_{m0} \sqrt{s_{om}} \quad (3)$$

155 where  $H_{m0}$  is the spectral significant wave height at the toe of the gravel,  $s_{om}$  is the wave steepness  
 156 calculated using the mean wave period  $T_z$  and  $C_e$  is a constant which was given the value  $C_e = 0.3$ .  
 157 Polidoro et al. (2014) applied Eq. 3 at a gravel beach at Worthing, UK and found it to overpredict  
 158 wave runup, however they used spectral significant wave height in deep water, potentially leading to  
 159 the overprediction.

160 No method to predict wave runup on composite beach ridges and dynamic revetments currently  
 161 exists. The summary above details existing approaches to predict wave runup on beaches and  
 162 structures with similarities to composite beaches and dynamic revetments. These methods indicate  
 163 a likely dependence of wave runup on wave height at the cobble berm/dynamic revetment toe but  
 164 any attempts to apply them to composite beaches have proven unsuccessful. The primary goal of  
 165 this study is to use new understanding of wave runup processes based on field and laboratory  
 166 measurements to develop empirical equations capable of predicting wave runup on composite  
 167 beaches. The ability to estimate wave runup is important for the design of dynamic cobble berm  
 168 revetments as it gives insight into the crest height and revetment volume required to limit  
 169 overtopping during high water levels.

### 170 **3. Methodology**

171 This study utilises inner surf and swash data obtained at 2 composite beach field sites in the UK, a  
 172 dynamic cobble berm revetment in the USA and 2 large-scale laboratory experiments designed to  
 173 investigate dynamic cobble berm revetments as summarised below. Hereafter, the term “berm”

174 refers to either the gravel ridge of a composite beach or a dynamic cobble berm revetment. A list of  
175 symbols is provided in Table A1.

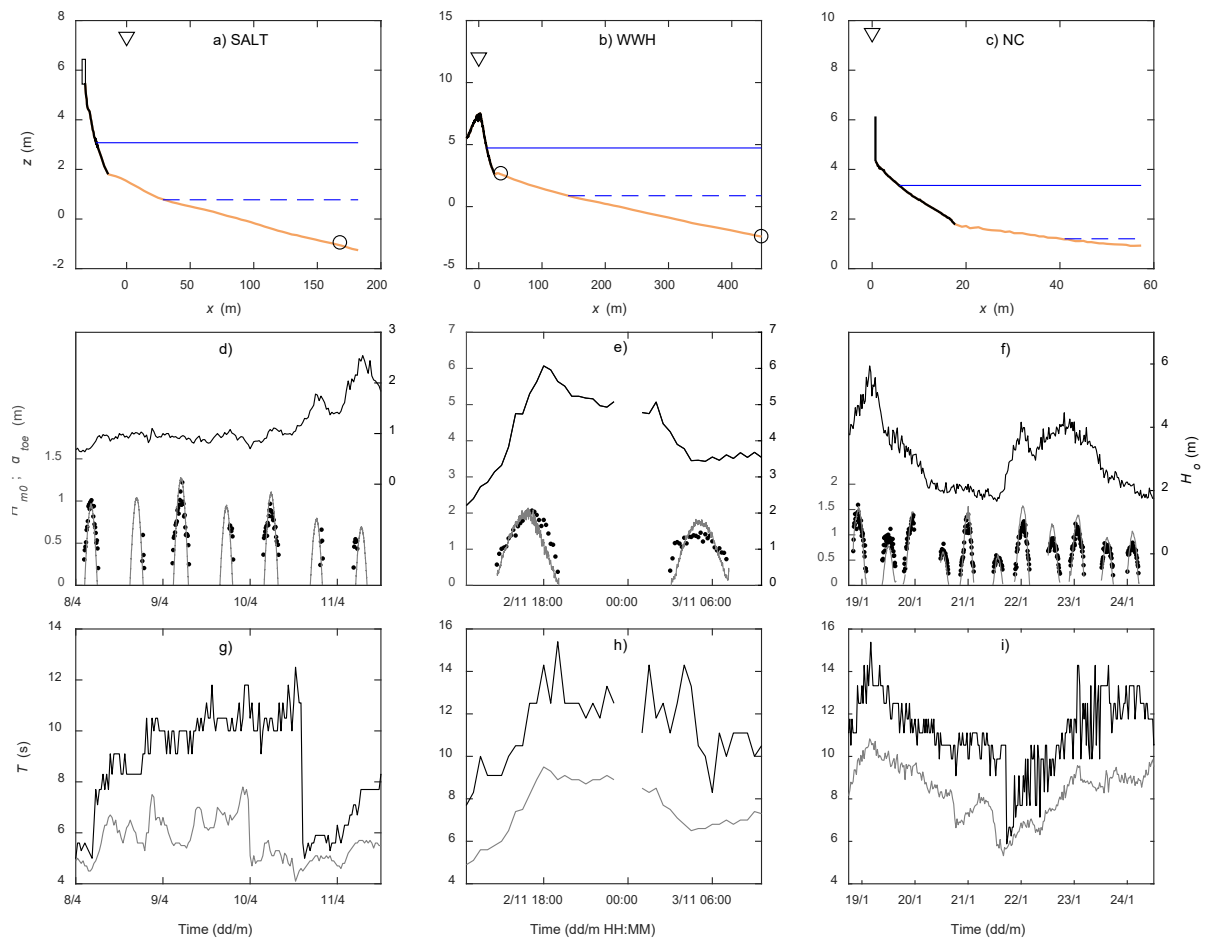
### 176 **3.1 Field Sites**

177 Experiments were conducted at 3 field sites: composite beaches at Saltburn-by-the-Sea and  
178 Westward Ho! in England and a dynamic cobble berm revetment at North Cove, USA. Figure 1  
179 presents details regarding the beach morphology, instrument locations, wave conditions and water  
180 levels and key parameters for each experiment are given in Table 1. At all sites, wave runup was  
181 captured using an elevated 2D scanning Lidar located close to the high tide shoreline (see Fig 1a-c).  
182 Note that at all sites, the berm toe is submerged at high tide but exposed at lower tidal elevations.

183 The Saltburn-by-the-Sea experiment is described in detail by Martins et al. (2017a). Saltburn-by-the-  
184 Sea (SALT) is a northeast-facing composite beach in NE England on the North Sea coast. The  
185 dissipative sandy foreshore is backed by a rounded pebble berm and a vertical seawall. The toe of  
186 the berm is located at approximately neap mean high water (MHWN) and during high tides under  
187 non-storm conditions the swash zone falls entirely on the berm without reaching the vertical  
188 seawall. The analysis in this paper focusses on 8/4 to 11/4/2016 when there was substantial wave  
189 runup on the berm. Wave conditions at the toe of the berm were extracted from the Lidar data.

190 Wave runup measurements during storm wave conditions were completed at Westward Ho! on  
191 2/11 and 3/11/2013 (Almeida et al., 2017). Westward Ho! (WWH) is a west-facing composite beach  
192 that lies within Bideford Bay in SW England. The beach has a wide sandy foreshore, backed by a 6.5  
193 m high berm made of rounded pebbles, cobbles and small boulders. The toe of the berm is located  
194 at approximately MHWN and during high tides incoming bores collapse directly onto the berm and  
195 the swash zone is entirely confined to the berm. In addition to the Lidar, three pressure transducers  
196 (PTs) were deployed on the sand beach, including one at the toe of the berm which was used to  
197 extract wave data at this location.

198 An experiment designed to investigate the performance and behaviour of the dynamic cobble berm  
199 revetment during storm conditions is described in detail by Bayle et al. (2021). North Cove (NC) is a  
200 naturally sandy, west-facing macro-tidal beach in Washington, USA where a dynamic cobble berm  
201 revetment has been installed using angular pebbles, cobbles and small boulders to protect the  
202 hinterland. The toe of the revetment is located at approximately MHWN and it is backed by a  
203 vertical scarp (Fig. 1c). This paper focusses on the period 19/1 to 24/1/2019 when wave runup was  
204 recorded on the berm. Wave conditions at the toe of the berm were extracted from the Lidar data.



205

206 **Figure 1:** Site geometry (top), wave height and water level data (middle) and wave period data (bottom) for each  
 207 experiment: Left column shows Saltburn-by-the-Sea (SALT); Middle column shows Westward Ho! (WWH); Right column  
 208 shows North Cove (NC). **a-c)** Representative beach profiles (orange = sand; black = berm), Lidar positions ( $\nabla$ ), PT locations  
 209 (o), maximum (solid blue line) and mean tidal elevation (dashed blue line) measured during the experiments are shown.  
 210 Elevations are defined relative to the local survey datums: SALT and WWH = Ordnance Datum (OD) NC = North American  
 211 Vertical Datum of 1988 (NAVD 88). **d-f)** Primary y-axis: Significant wave height at the berm toe  $H_{m0}$  (black dots), water  
 212 depth at the berm toe  $d_{toe}$  (grey solid line). Secondary y-axis: Offshore significant wave height  $H_o$  (black solid line). **g-i)** Peak  
 213 wave period (black line) and mean wave period (grey line).

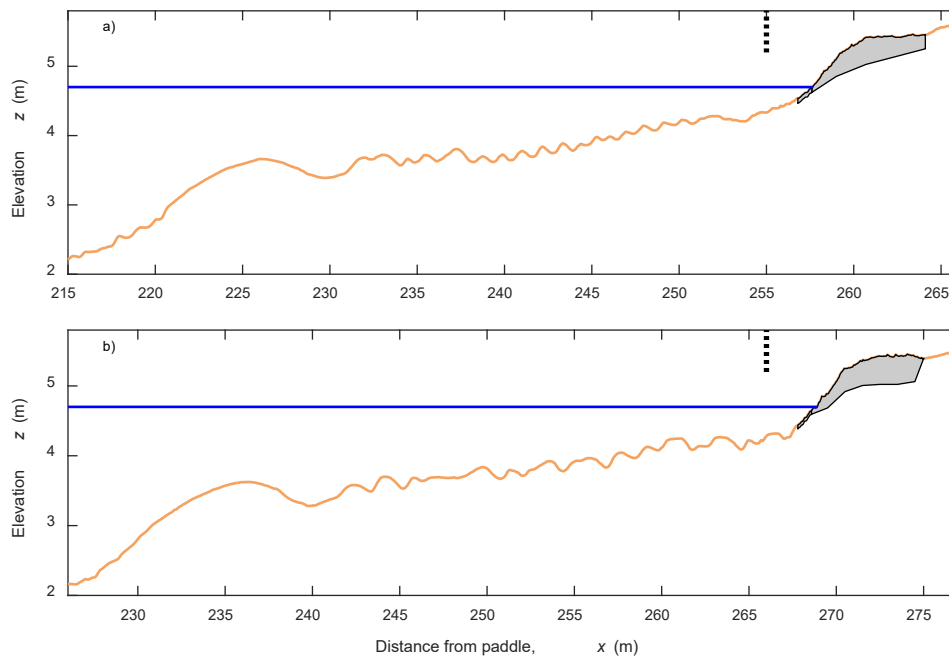
### 214 3.2 Laboratory Experiments

215 Two experiments (DynaRev1 and 2) designed to investigate the performance of a dynamic cobble  
 216 berm revetment under a rising water level and varying wave conditions were undertaken in the  
 217 Large Wave Flume (Großer Wellenkanal, GWK) in Hannover, Germany. Key details of the  
 218 morphology, sediment and wave parameters during these experiments are summarised in Table 1.  
 219 In both experiments, wave runup and wave conditions at the revetment toe were measured using a  
 220 SICK LMS511 2D scanning Lidar located 6.3 m above the revetment toe ( $z = 11.8$  m) at a sample  
 221 frequency of 25 Hz.

222 DynaRev1 (DR) is described in detail by Blenkinsopp et al. (2021) and Bayle et al. (2020). A dynamic  
 223 cobble berm revetment with an initial slope  $\tan\beta_{berm} = 0.167$  and volume of  $9.375$  m<sup>3</sup> was  
 224 constructed on a sand beach with gradient  $\tan\beta_{sand} = 0.067$  using rounded, well-sorted cobbles ( $D_{50} =$   
 225  $63$  mm,  $D_{85}/D_{15} = 1.32$ ) (see Fig. 2a).

226 DynaRev2 (2DR) used the same experimental methodology and initial revetment design, however  
 227 the revetment was constructed using angular, poorly-sorted cobbles ( $D_{50} = 44$  mm,  $D_{85}/D_{15} = 3.79$ )  
 228 (see Fig. 2b).

229 The results presented in this paper for both experiments are confined to a single wave condition ( $H_s$   
 230 = 0.8 m,  $T_p = 6.0$  s) and water level ( $z_{WL} = 4.7$  m) above the flume bottom. During these tests the  
 231 wave runup was confined to the seaward-facing slope of the revetment which had steepened from  
 232 its as-built condition to  $\tan\beta_{berm} = 0.24$  (DR) and 0.3 (2DR).



233  
 234 **Figure 2:** Sandy beach and revetment geometry for (a) DynaRev1 and (b) DynaRev2 at the start of testing at  $z_{WL} = 4.7$  m.  
 235 The grey shaded area indicates the revetment volume and the orange line the sand beach profile. The flume bottom  
 236 provides the vertical datum,  $z = 0$  m. The short vertical dotted lines indicate the centreline of the Lidar field of view.

237 **Table 1:** Summary of key morphology, sediment and wave parameters for the field and laboratory (grey shading)  
 238 experiments.

Experiment	$\tan\beta_{sand}$ ( $\tan\beta_{berm}$ )	Tidal Range (m)	$D_{50,sand}$ ( $D_{50,berm}$ ) (mm)	$H_0$ (m)	$T_p$ (s)	$H_0/L_0$
SALT	0.015 (0.18)	4.6	0.25 (56)	0.65 - 2.54	5 - 12.5	0.004 - 0.033
WWH	0.0125 (0.20)	7.7	0.19 (170)	2.16 - 6.1	7.7 - 15.4	0.002 - 0.042
NC	0.0225 (0.12)	4.3	0.18 (150)	1.8 - 6.0	5.9 - 15.4	0.003 - 0.05
DR	0.067 (0.24)	N/A	0.33 (63)	0.8	6	0.016
2DR	0.067 (0.3)	N/A	0.33 (44)	0.8	6	0.016

239  
 240 **3.3 Extraction of wave runup and nearshore wave data**

241 At all sites, wave runup during high-tide periods when swash motions were confined to the berm  
 242 was measured using elevated 2D Lidar scanners (Fig. 1a-c). As detailed by (Blenkinsopp et al., 2010;  
 243 Brodie et al., 2015), this approach enables the elevation of the time-varying water surface and  
 244 intermittently exposed bed to be measured at high spatio-temporal resolution along a cross-shore  
 245 transect. Each Lidar dataset was converted from polar to cartesian coordinates and interpolated  
 246 onto a horizontal grid ( $\Delta x = 0.1$  m). This data was then post-processed using the method of Almeida



247 et al. (2015) to generate separate timeseries of bed elevation and swash depths. The horizontal  
 248 shoreline position  $X_s(t)$  was taken as the most landward “swash” point at every timestep. The  
 249 resulting timeseries of horizontal shoreline positions was despiked to remove any spikes larger than  
 250 0.5 m that lasted less than 1 s and then filtered using a zero-phase moving average filter. The  
 251 shoreline positions were then projected onto the berm topography to obtain a timeseries of  
 252 shoreline elevations  $Z_s(t)$ , accounting for wave-by-wave changes in bed elevation. Previous authors  
 253 have used a similar Lidar-based approach to investigate wave runup statistics on gravel beaches  
 254 (Almeida et al., 2015), sandy beaches (Brodie et al., 2012; Almar et al., 2017), dikes (Hofland et al.,  
 255 2015) and dynamic cobble berm revetments (Bayle et al., 2020).

256 For SALT, NC and both DynaRev experiments, spectral significant wave height and time-averaged  
 257 water depth at the toe of the berm ( $H_{m0}$  and  $d_{toe}$ ) were obtained using Lidar measurements of the  
 258 time-varying free surface. At WWH the Lidar field of view did not quite extend to the berm toe, as  
 259 such a pressure transducer with a sampling frequency of 4 Hz was used to measure water depths  
 260 and wave statistics after applying the non-linear weakly dispersive reconstruction of Bonneton et al.  
 261 (2018) to account for non-hydrostatic effects (see Fig. 1b for instrument location).

### 262 3.4 Wave runup analysis

263 A schematic detailing key composite beach parameters used in this study is shown in Figure 3. The  
 264 investigation of wave runup presented below uses an approach similar to that of Stockdon et al.  
 265 (2006) in their comprehensive analysis of wave runup on sandy beaches. They proposed a general  
 266 relationship for the elevation of extreme (2% exceedance) runup relative to the still water level  
 267 (SWL):

$$268 \quad R_{2\%} = \bar{\eta} + \frac{S}{2} \quad (4)$$

269 where  $\bar{\eta}$  is the wave setup at the shoreline ( $\bar{\eta} = \overline{Z_s} - SWL$ ) and  $S$  represents the significant swash  
 270 height:

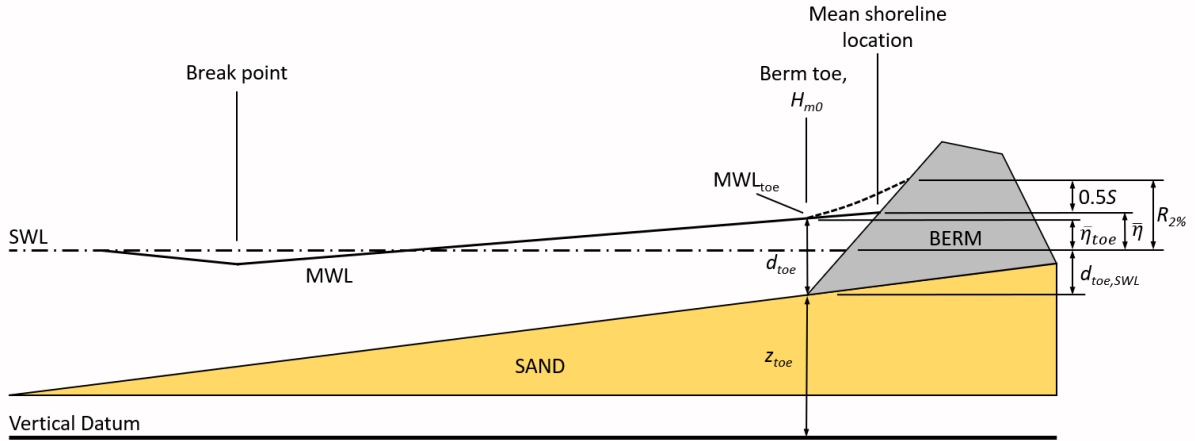
$$271 \quad S = 4\sqrt{m_0} \quad (5)$$

272 where  $m_0$  is the zeroth moment of the variance density spectrum  $S_{\zeta\zeta}(f)$  computed from the linearly  
 273 detrended shoreline elevation timeseries  $\zeta(t)$ . Thus,  $S$  is equivalent to four times the standard  
 274 deviation of  $\zeta(t)$ .

275 Stockdon et al. (2006) further decomposed the significant swash height into a short-wave ( $S_{sw}$ ) and  
 276 infragravity component ( $S_{ig}$ ):

$$277 \quad S = \sqrt{S_{sw}^2 + S_{ig}^2} \quad (6)$$

278 where  $S_{sw}$  and  $S_{ig}$  are calculated by applying Eq. 5 only over the appropriate frequencies. For this  
 279 study a frequency of  $0.6f_p$  was used to separate the short-wave and infragravity components of both  
 280 significant swash ( $S_{sw}$  and  $S_{ig}$ ) and wave height at the berm toe ( $H_{m0,sw}$  and  $H_{m0,ig}$ ), where  $f_p$  is the  
 281 discrete peak wave frequency recorded at the offshore waverider buoy at each site. Hamm and  
 282 Peronnard (1997) and Oh et al. (2020) demonstrated the importance of using a separation frequency  
 283 which varies with the wave period in the shoaling and surf zones to ensure that short-wave energy is  
 284 not included in the infragravity band. The separation frequency of  $0.6f_p$  used here was chosen based  
 285 on inspection of the surface elevation spectra at the berm toe at each site.



286

287 **Figure 3:** Schematic detailing key composite beach parameters.

288 Due to the macrotidal nature of the three field sites, wave runup, water depths and wave conditions  
 289 were evaluated over relatively short 10-minute time windows, to approximate tidal stationarity. This  
 290 window length was chosen based on a sensitivity analysis and for consistency, 10-minute windows  
 291 were also used for the DynaRev data. The swash gradient ( $\tan\beta_{swash}$ ) within each time window was  
 292 taken as the linear gradient within limits defined by  $\pm 2\sigma$  of the shoreline position timeseries  $X_s(t)$   
 293 around the mean. The mean water level at the berm toe ( $MWL_{toe}$ ) is defined here as the 10-minute  
 294 time-averaged water surface elevation at the location of the toe. The still water level (SWL) is  
 295 approximated as the time-averaged water surface elevation measured using pressure transducers  
 296 located seaward of the surf zone at SALT and WWH and wave gauges in the deep-water section of  
 297 the tank for DR and 2DR (SWL measurements were not possible for NC).

298 Regression model performance was quantified through the use of the following statistical  
 299 parameters: root mean square error (RMSE), coefficient of determination ( $r^2$ ) and normalised mean  
 300 square error (NMSE) which was calculated as:

301 
$$NMSE = \frac{\frac{1}{N} \sum (y_{pred,i} - y_{obs,i})^2}{\sigma^2} \quad (7)$$

302 where  $y_{pred}$  and  $y_{obs}$  are the predicted and observed values of a particular parameter,  $N$  is the number  
 303 of observations and  $\sigma^2$  is the variance of the observed values.

304 **Swash motions on composite beaches and dynamic revetments**

305 The following section investigates the components of the significant swash height ( $S$ ,  $S_{sw}$  and  $S_{ig}$ ) on  
 306 composite beaches as a function of the water depth and total, short wave and infragravity spectral  
 307 significant wave height at the berm toe ( $H_{m0}$ ,  $H_{m0,sw}$ ,  $H_{m0,ig}$ ). These observations are then used in  
 308 Section 5.1 to develop new approaches to predict wave runup on composite beaches and dynamic  
 309 revetments using the framework presented in Section 3.4.

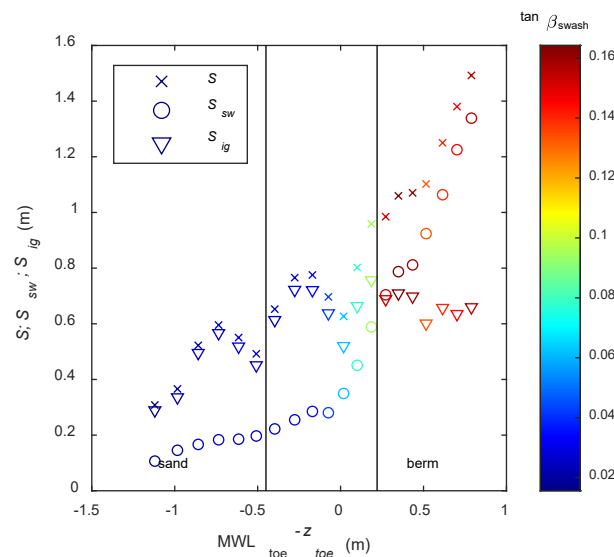
310 **4.1 Variability of swash motions with swash zone position**

311 One of the complexities of runup on a composite beach is that the swash zone transitions from a  
 312 gently sloping sand beach to the steeper and more porous berm. Previous authors have suggested  
 313 that the porous nature of pebble/cobble slopes will attenuate wave runup due to high friction and  
 314 flow volume loss during wave runup (Mason and Coates, 2001). Furthermore, a dependence of  
 315 runup height on swash slope is a common feature of wave runup equations (e.g. Hunt, 1959;  
 316 Ruggerio et al., 2001; Stockdon et al., 2006; Poate et al., 2016). This has been primarily linked to the

317 level of energy dissipation in the surf zone, i.e. dissipation will be larger for milder beach slopes  
 318 typical of dissipative sand beaches (Brodie et al., 2012; Guedes et al., 2012).

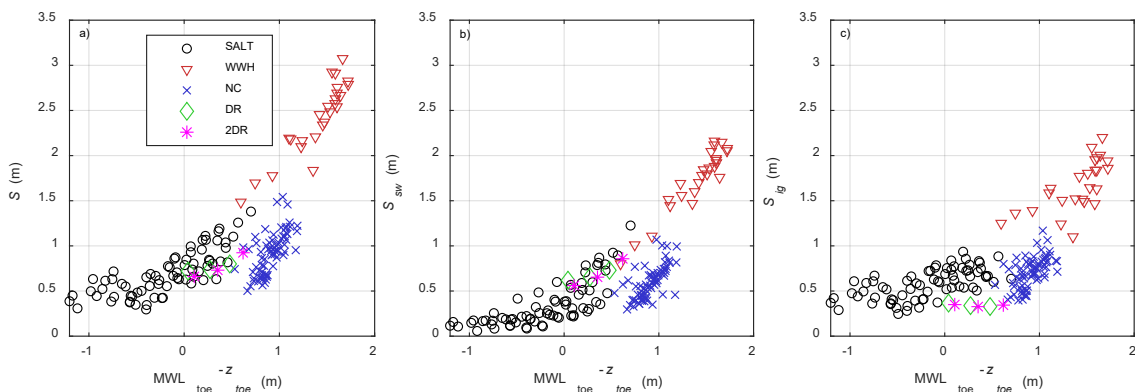
319 Fig. 4 shows  $S$ ,  $S_{sw}$  and  $S_{ig}$  as a function of mean water level relative to the berm toe ( $MWL_{toe} - z_{toe}$ ,  
 320 where  $z_{toe}$  is the elevation of the berm toe relative to the local datum) at SALT as the swash zone  
 321 transitioned from the sand beach to the berm during the afternoon rising tide on 9/4/2016. This  
 322 example is used because SALT is the only field site where swash was measured on both the sand and  
 323 berm.

324  $MWL_{toe} - z_{toe}$  is used primarily to indicate the location of the swash zone relative to the berm. When  
 325 the swash zone is entirely on the sand beach ( $MWL_{toe} - z_{toe} < -0.45$  m), the total significant swash  
 326 height is dominated by the infragravity component which tends to increase as the water level rises  
 327 for this tide (Fig. 4). Dominance of infragravity swash on dissipative beaches has been observed by  
 328 multiple researchers (Guza and Thornton, 1982; Raubenheimer et al., 1996; Ruggiero et al., 2001).  
 329 The short-wave component of significant swash height  $S_{sw}$  is relatively small but increases gradually  
 330 with  $MWL_{toe}$  on the sand beach as the sand gradient, and hence swash slope, increases slightly  
 331 towards the berm toe. As swashes initially reach the berm toe ( $-0.45$  m  $< MWL_{toe} - z_{toe} < 0$  m) there is  
 332 no immediate change in the rate of change of  $S_{sw}$  with water depth. However, once the mean water  
 333 depth above the toe surpasses 0 m, the majority of the swash zone is on the berm leading to an  
 334 increase in swash slope from around 0.02 to 0.06 and  $S_{sw}$  becomes strongly controlled by the water  
 335 depth at the berm toe (see Section 4.2). By contrast, the infragravity significant swash component  $S_{ig}$   
 336 remains approximately constant as the water depth increases and the swash zone passes onto the  
 337 berm. As a result, the total significant swash height  $S$  becomes dominated by  $S_{sw}$ . The trend of  
 338 increasing  $S_{sw}/S_{ig}$  after the swash zone reaches the berm is common to all tides at SALT. The  
 339 observation that the infragravity component of significant swash height dominates on the sand  
 340 beach and the short-wave component dominates on the berm is not consistent for all tides, but is  
 341 the most common behaviour.



342  
 343 **Figure 4:**  $S$ ,  $S_{sw}$  and  $S_{ig}$  as a function of the elevation difference between the berm toe and  $MWL_{toe}$  at SALT as the swash  
 344 zone transitions from the sand beach to the berm during the afternoon rising tide on 9/4/2016. On the left of the plot, the  
 345 swash zone is located entirely on sand and on the right, it is entirely on the berm. Between these limits, indicated with the  
 346 two vertical black lines, swash events pass over both sand and gravel. The colorbar indicates the gradient of the swash  
 347 zone ( $\tan\beta_{swash}$ ) within each 10-minute time window.

348 Fig. 5 shows the variation of the significant swash height within the different frequency bands with  
 349  $MWL_{toe} - Z_{toe}$  for all experiments. There is a clear trend that once swash events interact with the berm  
 350 ( $MWL_{toe} - Z_{toe} > -0.45$ ),  $S_{sw}$  increases with mean water level relative to the berm toe for all  
 351 experiments (Fig. 5b) and this is explained in Section 4.2. Due to the similar berm slopes at SALT  
 352 ( $\tan\beta_{berm} = 0.18$ ) and WWH ( $\tan\beta_{berm} = 0.2$ ) the results fall on approximately the same line, however  
 353 at the lower gradient NC ( $\tan\beta_{berm} = 0.133$ )  $S_{sw}$  is smaller for the same value of  $MWL_{toe} - Z_{toe}$ .  
 354 Infragravity significant swash heights have a similar magnitude to the short-wave values and a trend  
 355 of increasing  $S_{ig}$  with  $MWL_{toe} - Z_{toe}$  within each field dataset is evident in Fig. 5c for  $MWL_{toe} - Z_{toe} > -$   
 356 0.45. This may be a function of the reducing surf zone width as the water level rises above the berm  
 357 toe, leading to lesser dissipation of infragravity energy by short-waves (see Section 4.3). However, it  
 358 is acknowledged that this observation may be biased by the fact that the WWH data, which includes  
 359 the largest water depths, was collected during a major storm with offshore significant wave heights  
 360 up to 6.1 m that would be expected to cause very large infragravity swash (e.g. Guza and Thornton,  
 361 1982). The total significant swash height  $S$  increases approximately linearly with water depth above  
 362 the toe for  $MWL_{toe} - Z_{toe} > -0.45$  at all field sites.



363  
 364 **Figure 5: a) Total, (b) short-wave, and (c) infragravity significant swash height as a function of the elevation difference**  
 365 **between the berm toe and  $MWL_{toe}$  for all field and laboratory datasets.**

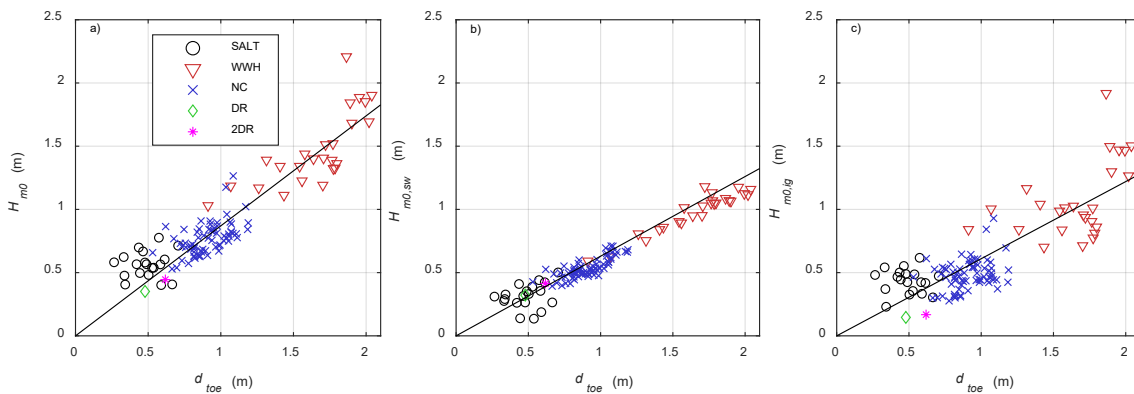
#### 366 4.2 Variability of swash with wave height at the berm toe

367 The correlation between significant swash height and water depth above the berm toe (Fig. 5) is a  
 368 result of the fact that the sand slope of composite beaches is typically highly dissipative. The surf  
 369 zone is saturated at short-wave frequencies which effectively decouples the short-wave height in  
 370 shallow water from the conditions offshore. Consequently, the height of short-waves, which are a  
 371 primary driver of swash motions becomes a function of water depth. This is evident in Fig. 1d-f,  
 372 which shows that the spectral significant wave height measured at the berm toe  $H_{m0}$  is  
 373 approximately equal to the water depth above the berm toe ( $d_{toe} = MWL_{toe} - Z_{toe}$  for  $MWL_{toe} - Z_{toe} > 0$ )  
 374 throughout all three field experiments and is independent of the offshore wave conditions, even  
 375 when significant wave height in deep water  $H_o$  is very large.

376 To explore this observation, Fig. 6 presents the total, short-wave and infragravity spectral significant  
 377 wave height at the berm toe ( $H_{m0}$ ,  $H_{m0,sw}$ ,  $H_{m0,ig}$ ) as a function of water depth at the berm toe  $d_{toe}$ .  
 378 Note that in Fig. 6 and the remainder of this paper, only the 10-minute windows where the swash  
 379 zone fell entirely on the berm are shown. Thus, there are no values for  $d_{toe} < 0.26$  m.

380 Fig. 6a and b demonstrate a linear relationship between  $H_{m0}$  and  $H_{m0,sw}$  with  $d_{toe}$ . An apparent  
 381 relationship between significant infragravity wave height  $H_{m0,ig}$  and water depth above the berm toe  
 382 is also evident in Fig. 6c which is investigated further in Section 4.3. The wave height to water depth

383 ratio  $\gamma_s = H_{m0}/d_{toe}$  is remarkably consistent for all sites ( $\gamma_{s,SALT} = 0.88$ ;  $\gamma_{s,WWH} = 0.90$ ;  $\gamma_{s,NC} = 0.85$ ) with a  
 384 mean value of  $\gamma_s = 0.87$  (RMSE = 0.15 m; NMSE = 0.18). These values of  $\gamma_s$  are relatively high  
 385 compared to previously reported field values and there is no obvious dependence on the gradient of  
 386 the sand slope seaward of the berm (e.g. Sallenger and Holman, 1985). It is expected that the higher  
 387 values are caused by proximity of the measurement location to the shoreline and possibly the  
 388 contribution of reflections from the steep berm. A similar effect was observed by Martins et al.  
 389 (2017b) for a reflective sand beach in a large-scale laboratory wave flume. Mean values of  $\gamma_s$  for the  
 390 short-wave and infragravity components of significant wave height are  $\gamma_{s,sw} = 0.59$  (RMSE = 0.06 m;  
 391 NMSE = 0.07) and  $\gamma_{s,ig} = 0.61$  (RMSE = 0.18 m; NMSE = 0.36). Note that all statistics are significant at  
 392 the 95% confidence interval or better.



393  
 394 **Figure 6:** a) Total, b) short-wave, and c) infragravity spectral significant wave height as a function of the water depth  
 395 above the berm toe. The solid black lines in each plot present the best fit wave height to water depth ratios ( $\gamma_s$ ) for each  
 396 panel ( $\gamma_s = 0.87$ ,  $\gamma_{s,sw} = 0.59$  and  $\gamma_{s,ig} = 0.60$ ).

397 A relationship between significant swash height  $S$  and  $H_{m0} \tan \beta_{swash}$  is observed within all  
 398 frequency bands in Fig. 7. **Similar variation of  $S$ ,  $S_{sw}$  and  $S_{ig}$  with offshore wave height  $H_o$  has been**  
 399 **presented previously on a dissipative sand beach by Senechal et al. (2011), however here the wave**  
 400 **height at the berm toe is used and a slope term consistent with previous authors (Hunt, 1959;**  
 401 **Battjes, 1974; Holman, 1986) has been included. This causes the data from the lower gradient NC**  
 402 **experiment to collapse onto that for SALT and WWH. The relationship between  $S_{sw}$  and**  
 403  **$H_{m0,sw} \tan \beta_{swash}$  is approximately linear.** However, there is evidence that  $S_{ig}$  (and hence  $S$ ) increases  
 404 more slowly with increasing  $H_{m0,ig}$  for  $H_{m0,ig} \tan \beta_{swash} > 0.3$  m at WWH, potentially indicating  
 405 infragravity swash saturation. Comparable results were presented by Senechal et al. (2011) who  
 406 suggested that significant infragravity swash height may reach a maximum value for large offshore  
 407 wave heights ( $H_o > 4$  m).

408 For the current data, a linear regression analysis leads to the following expressions:

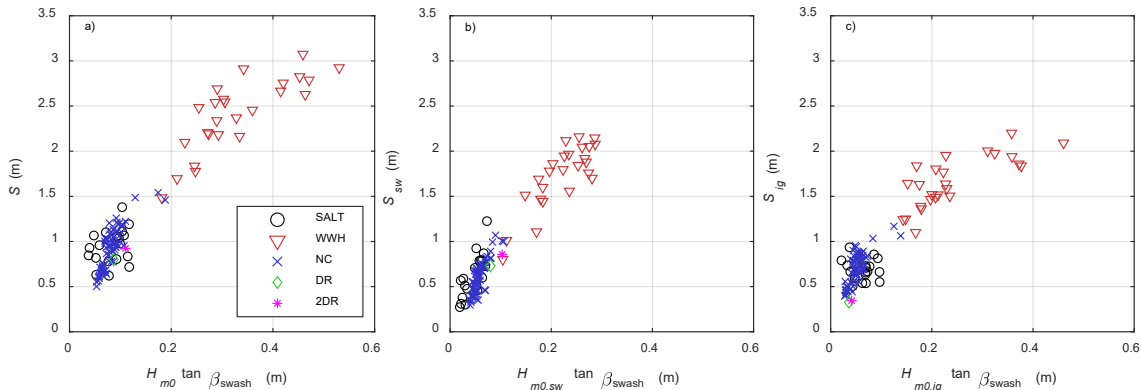
409 
$$S = 0.48 + 5.65 H_{m0} \tan \beta_{swash} \quad (r^2 = 0.91, \text{RMSE} = 0.20 \text{ m; NMSE} = 0.10) \quad (8)$$

410 
$$S_{sw} = 0.23 + 6.79 H_{m0,sw} \tan \beta_{swash} \quad (r^2 = 0.92, \text{RMSE} = 0.15 \text{ m; NMSE} = 0.08) \quad (9)$$

411 
$$S_{ig} = 0.48 + 4.59 H_{m0,ig} \tan \beta_{swash} \quad (r^2 = 0.85, \text{RMSE} = 0.17 \text{ m; NMSE} = 0.17) \quad (10)$$

412 Equations 8-10 include a non-zero intercept which is a common feature of existing wave runup  
 413 equations for sand beaches (e.g. Guza and Thornton, 1982; Holman, 1986; Ruggerio et al., 2001). On  
 414 composite beaches, this non-zero intercept may be physical meaningful because when the mean  
 415 shoreline position is seaward of the berm and the significant wave height at the berm toe is zero,  
 416 wave runup can still be initiated on the sandy part of the beach and propagate onto the berm.

417 No dependence of significant swash height on wave period was found within the current dataset in  
 418 contrast to many previous studies of wave runup (e.g. Guza and Thornton, 1982; Holman, 1986;  
 419 Ruggerio et al., 2001). For composite beaches it is suggested that this lack of dependence is because  
 420 significant swash height is controlled by the wave height at the berm toe as shown in Fig 7. For the  
 421 composite beaches studied here, wave height at the berm toe is depth-limited and previous studies  
 422 have suggested that wave height to water depth ratio in the inner surf zone is not strongly  
 423 dependent on wave period on dissipative slopes (e.g. Thornton & Guza, 1982).



424

425 **Figure 7:** a) Total, b) short-wave, and c) infragravity components of significant swash height as a function of  
 426  $H_{m0} \tan \beta_{swash}$  where  $H_{m0}$  is evaluated within the relevant frequency band ( $H_{m0}$ ,  $H_{m0,sw}$ ,  $H_{m0,ig}$ ).

427

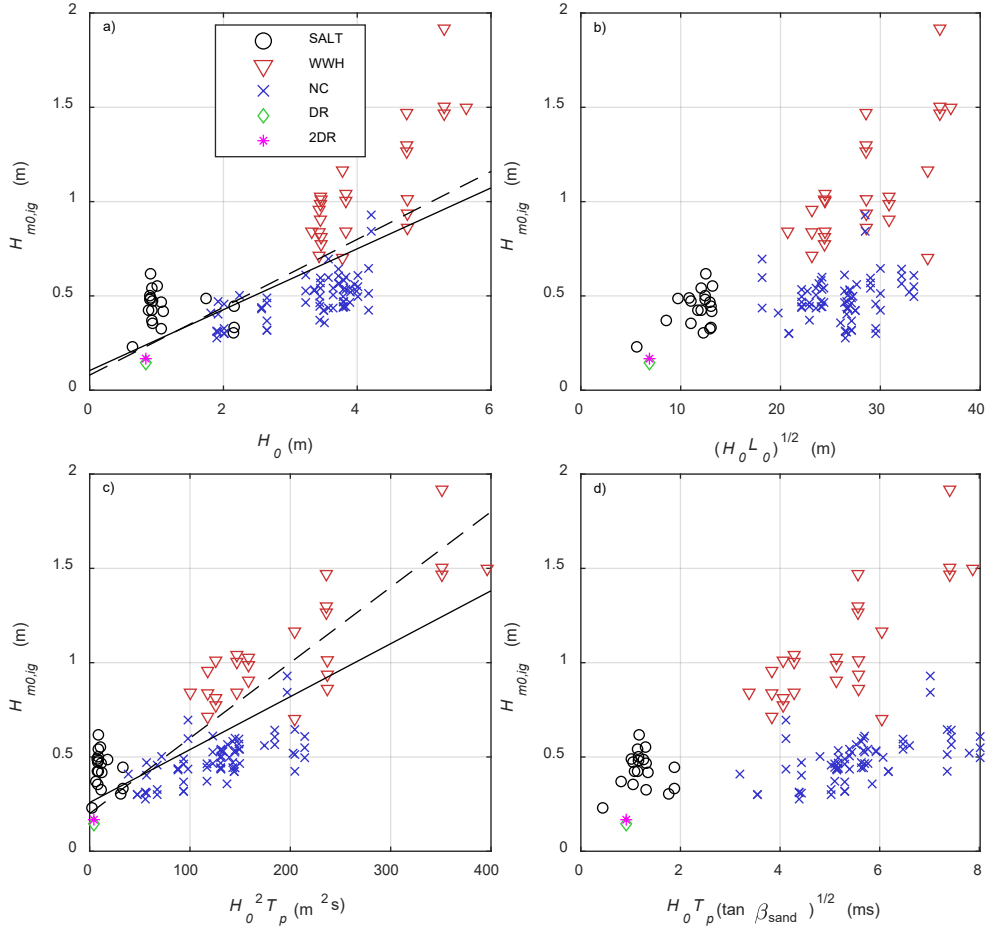
### 4.3 Infragravity wave height at the berm toe

428 Fig. 7 demonstrates a strong relationship between  $S$  and  $H_{m0} \tan \beta_{swash}$  in the infragravity and short-  
 429 wave frequency bands. However, while the spectral significant short-wave height  $H_{m0,sw}$  is  
 430 independent of that in deep water when the surf zone is saturated, infragravity wave height  $H_{m0,ig}$  is  
 431 dependent not only on depth at the berm toe but is also influenced by the offshore wave conditions.  
 432 Previous authors have shown  $S_{ig}$  or  $H_{m0,ig}$  to be dependent on offshore wave height  $H_0$  (Guza and  
 433 Thornton, 1982), wave power (Inch et al. 2017),  $\sqrt{H_0 L_0}$  (Stockdon et al., 2006; Fiedler et al., 2015)  
 434 and  $H_0 T_p \sqrt{\tan \beta}$  (Billson et al., 2019). The measured significant infragravity wave height at the berm  
 435 toe as a function of these parameters are presented for the current data in Fig. 8. All panels in Fig. 8  
 436 display scatter, but a dependency of significant infragravity wave height at the berm toe,  $H_{m0,ig}$  on  $H_0$   
 437 is evident in Fig. 8a. Inch et al. (2017) found good correlation between  $H_{m0,ig}$  and both wave height  
 438 and wave power and provided best-fit linear regressions:

$$439 \quad H_{m0,ig} = 0.18H_0 + 0.08 \quad (11)$$

$$440 \quad H_{m0,ig} = 0.004H_0^2 T_p + 0.20 \quad (12)$$

441 These parameterisations have some predictive ability for the current dataset (Eq. 11:  $r^2 = 0.40$ , RMSE  
 442 = 0.20 m, NMSE = 0.43; Eq. 12:  $r^2 = 0.65$ , RMSE = 0.17 m, NMSE = 0.44), but they do not improve on  
 443 the linear relationship with depth above the berm toe observed in Fig. 6c and discussed in section  
 444 4.2 ( $\gamma_{s,ig} = 0.61$ ). Note that equivalent plots of  $S_{ig}$  as a function of these parameters provide a very  
 445 similar result due to the relationship between  $H_{m0,ig}$  and  $S_{ig}$  observed in Fig. 7c.



446

447 **Figure 8:** Spectral significant infragravity wave height at the berm toe as a function of previously reported predictors: **(a)**  
 448 significant offshore wave height (Guza and Thornton, 1982), **(b)**  $\sqrt{H_o L_o}$  (Stockdon et al., 2006), **(c)**  $H_o^2 T_p$  (Inch et al. 2017),  
 449 and **(d)**  $H_o T_p \sqrt{\tan \beta}$  (Billson et al., 2019). The dashed lines in panels a and c show the fits obtained by Inch et al. (2017) and  
 450 the black lines show linear regressions for the current dataset.

451 Previous authors have suggested that while transfer of energy to infragravity frequencies is greater  
 452 on low gradient beaches (de Bakker et al., 2015), the rate of infragravity energy dissipation typically  
 453 increases with reducing depth (Inch et al., 2017). As such,  $H_{m0,ig}$  at the shoreline is expected to  
 454 reduce with increasing surf zone width for a constant value of  $H_o$ . Due to the two-slope nature  
 455 composite beaches with a low gradient foreshore and steep berm, the surf zone width will reduce  
 456 not only with smaller  $H_o$ , but also with increasing water depth over the berm toe for a constant  
 457 deepwater wave condition. Fig. 9a presents the significant infragravity wave height at the berm as a  
 458 function of a geometric proxy for composite beach surf zone width,  $l_{sz}$ :

459 
$$l_{sz} = \frac{5/3H_o - d_{toe}}{\tan \beta_{sand}} + \frac{d_{toe}}{\tan \beta_{berm}} \quad (13)$$

460 where  $5/3H_o$  is used to estimate the breaker depth in a saturated surf zone based on Thornton and  
 461 Guza (1982). It is acknowledged that this approach is simplistic as it does not account for wave  
 462 shoaling, however it appears to provide reasonable estimates of surf zone width at the three field  
 463 sites (based on additional Lidar measurements at SALT and visual estimates at WWH and NC) and  
 464 further refinement to approximate the shoaling process did not improve the results. The line of best  
 465 fit to the data presented in Fig. 9a is:

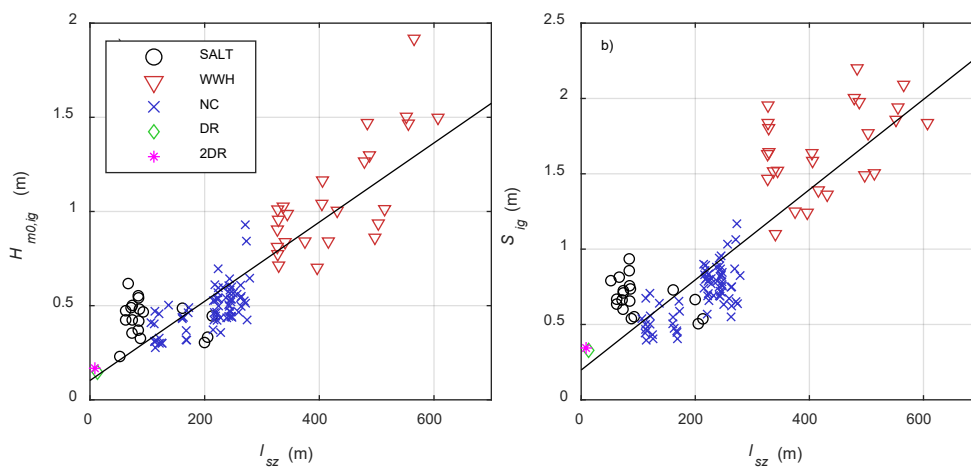
466 
$$H_{m0,ig} = 0.0021l_{sz} + 0.10 \quad (r^2 = 0.74, \text{RMSE} = 0.16 \text{ m}, \text{NMSE} = 0.26) \quad (14)$$



467 Note that estimating  $d_{toe}$  using the SWL elevation ( $d_{toe,SWL} = SWL - z_{toe}$ ) from measurements obtained  
 468 seaward of the surf zone rather than  $MWL_{toe}$  (hence ignoring wave setup) causes only a small  
 469 increase in  $l_{sz}$  (<5%).

470 The improved performance of Eq. 14 compared to Eq. 11 or 12 suggests that it is worthy of further  
 471 investigation over a wider range of conditions because it retains a dependence on offshore wave  
 472 height (e.g. Fig. 7a: Guza and Thornton, 1982) and explicitly includes beach gradient and surf zone  
 473 width which are known to influence infragravity wave dissipation. Inch et al. (2017) found that  
 474 infragravity wave height is better predicted if wave period is accounted for. This was not observed  
 475 for the data presented here, but should be considered in further investigations with a wider range of  
 476 sites and wave conditions. A direct relationship between significant infragravity swash height and  $l_{sz}$   
 477 is also observed in Fig. 9b:

$$478 \quad S_{ig} = 0.0030l_{sz} + 0.20 \quad (r^2 = 0.70, \text{RMSE} = 0.24 \text{ m}, \text{NMSE} = 0.34) \quad (15)$$



479  
 480 **Figure 9: a)** Spectral significant infragravity wave height at the berm toe and **(b)** significant infragravity swash height as a  
 481 function of  $l_{sz}$ . The black lines show linear fits to the data.

#### 482 4. Estimating wave runup on composite beaches

483 For coastal engineers, the runup parameter of most importance is  $R_{2\%}$ , the elevation relative to the  
 484 SWL exceeded by 2% of swash events. This parameter enables an assessment of overtopping risk  
 485 and is key for assessing the design crest elevation of dynamic cobble berm revetment structures,  
 486 however to date, no runup equation exists for composite beaches. The following section uses the  
 487 results presented in Section 4 to develop three new methodologies of varying complexity to predict  
 488 wave runup on composite beaches and dynamic cobble berm revetments.

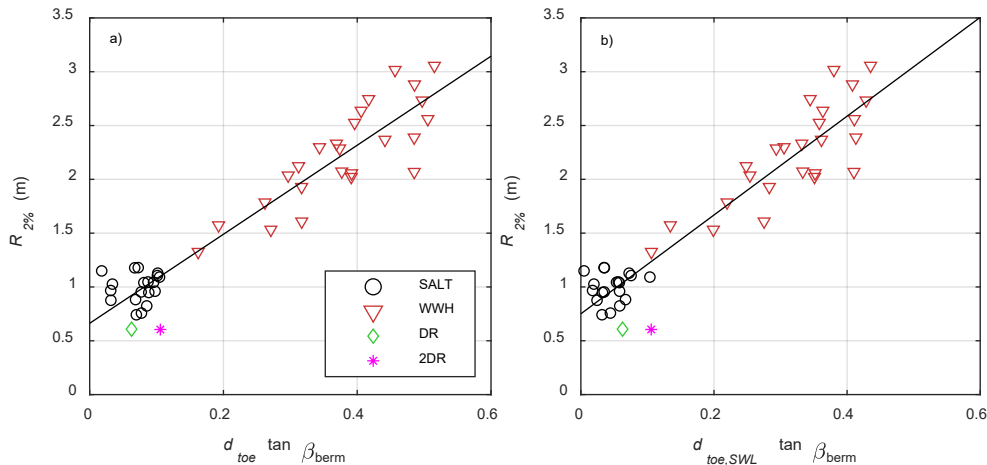
489 For the sites investigated here, there is a strong linear relationship between  $R_{2\%}$  and  $d_{toe} \tan \beta_{berm}$   
 490 when the swash zone occurs entirely on the berm as would be expected during design storm  
 491 conditions at high tide (Fig. 10a).

$$492 \quad R_{2\%} = 4.14d_{toe} \tan \beta_{berm} + 0.66 \quad (16)$$

493 It is acknowledged that estimating water depth at the berm toe is not trivial due to wave setup and  
 494 this is addressed below, however for the data presented here, comparable predictive ability can be  
 495 achieved using the depth of the berm toe below SWL,  $d_{toe,SWL}$  (Fig. 10b) which can be easily  
 496 estimated.



$$R_{2\%} = 4.59d_{toe,SWL}\tan\beta_{berm} + 0.75 \tag{17}$$



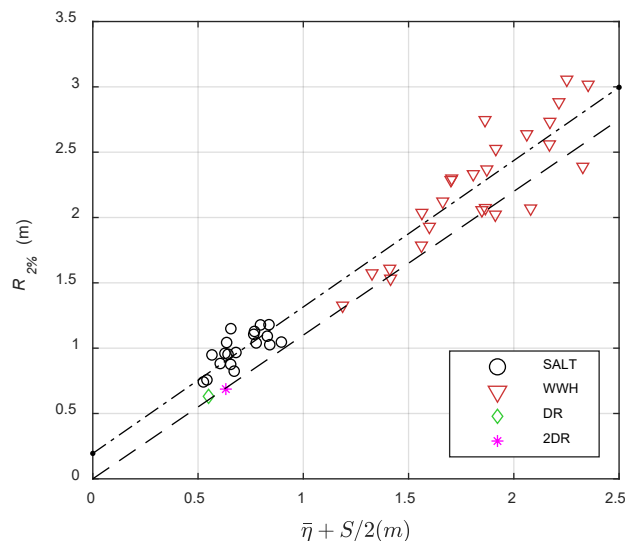
499 **Figure 10:**  $R_{2\%}$  as a function of (a)  $d_{toe}\tan\beta_{berm}$ , and (b)  $d_{toe,SWL}\tan\beta_{berm}$ . Note that the NC field data are excluded  
 500 because no measurement of SWL from seaward of the surf zone was available to enable an estimate of  $R_{2\%}$ .

501 While Eq. 16 and 17 provide good predictions for the current dataset (Eq. 16:  $r^2 = 0.90$ , RMSE = 0.23  
 502 m, NMSE = 0.10; Eq. 17:  $r^2 = 0.89$ , RMSE = 0.25 m, NMSE = 0.11), their validity over a wider variety of  
 503 beach geometry and offshore wave conditions may be more limited. Consequently, a more general  
 504 approach is outlined below.

505 As noted in section 3.4, total runoff  $R$  is composed of a setup  $\bar{\eta}$  and swash component  $S$  (Eq. 4) and  
 506 Stockdon et al. (2006) related the statistical runoff parameter  $R_{2\%}$  to these parameters using field  
 507 data:

$$R_{2\%} = 1.1 \left[ \bar{\eta} + \frac{S}{2} \right] \tag{18}$$

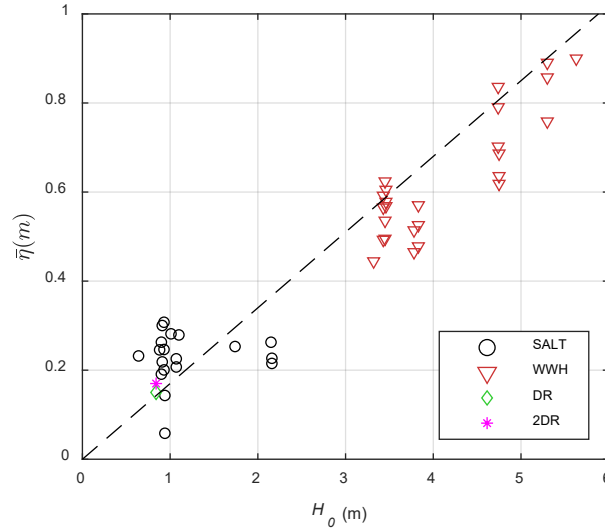
509 Fig. 11 demonstrates that the current data has the same gradient as Eq. 17, but with a non-zero  
 510 intercept ( $c = 0.19$  m). A non-zero intercept was also found by Stockdon et al. (2006;  $c = 0.10$  m) but  
 511 they removed it to make Eq. 18 physically consistent. If Eq. 18 is applied to the current dataset, the  
 512 error statistics (RMSE = 0.28 m, NMSE = 0.16) are only slightly poorer than for the optimum  
 513 regression line (RMSE = 0.18 m, NMSE = 0.06) and as such it will be used in the following analysis.



515 **Figure 11:**  $R_{2\%}$  as a function of the sum of setup and  $S/2$ . The dashed line represents Eq. 18 (Stockdon et al., 2006) while the  
 516 dash-dot line is the best fit to the current data with a non-zero intercept ( $c = 0.19$  m).

517 The wave setup component of runup was shown by Guza and Thornton (1981) to be related to  
 518 offshore wave height,  $H_0$  and their parameterisation provides a good fit to the current data ( $r^2 =$   
 519  $0.87$ ,  $RMSE = 0.10$  m,  $NMSE = 0.19$ ):

$$520 \quad \bar{\eta} = 0.17H_0 \quad (19)$$



521  
 522 **Figure 12:** Setup as a function of offshore wave height, where the dashed line represents Eq.18 (Guza and Thornton, 1981).

523 Combining Eq. 18 and 19 with the previously defined relationship for  $S$  as a function of the  
 524 significant wave height at the berm toe,  $H_{m0}$  and  $\tan\beta_{swash}$  (Eq. 8), where here we assume that the  
 525 swash zone is entirely on the berm and so  $\tan\beta_{swash} \approx \tan\beta_{berm}$ :

$$526 \quad S = 0.48 + 5.65H_{m0}\tan\beta_{berm} \quad (20)$$

527 we can obtain an expression to estimate  $R_{2\%}$ :

$$528 \quad R_{2\%} = 0.19H_0 + 3.11H_{m0}\tan\beta_{berm} + 0.26 \quad (21)$$

529 In common with many predictors of wave runup on coastal defence structures (e.g. EurOtop, 2018),  
 530 a value for the significant wave height at the berm toe  $H_{m0}$  is required. It is proposed here that  
 531 because the sand slope on composite beaches is typically highly dissipative, surf zone waves can  
 532 reasonably be assumed to be depth-limited so that  $H_{m0}$  at the berm toe can be estimated using an  
 533 appropriate value of wave height to water depth ratio,  $\gamma_s$ . For the current study, the mean measured  
 534 value of  $\gamma_s = 0.87$  (see Section 4.2) and this allows  $H_{m0}$  to be estimated based on the water depth at  
 535 the berm toe,  $d_{toe}$ . However, it is also noted that the superelevation of the mean water level at the  
 536 berm toe  $\bar{\eta}_{toe}$  is not equal to the setup at the shoreline  $\bar{\eta}$  and is required to obtain a suitable value  
 537 of  $d_{toe}$ . For the present data  $\bar{\eta}_{toe}$  is best estimated as:

$$538 \quad \bar{\eta}_{toe} = 3.33 \times 10^{-4}l_{sz} + 0.12 \quad (22)$$

539 where  $l_{sz}$  is obtained using Eq. 13, but using  $d_{toe,SWL}$  to approximate  $d_{toe}$ . This approach enables  $d_{toe}$  to  
 540 be estimated with an RMSE of just 0.04 m ( $NMSE = 0.32$ ) where  $d_{toe} = d_{toe,SWL} + \bar{\eta}_{toe}$ . A flow chart  
 541 which outlines the suggested methodology to estimate  $R_{2\%}$  using Eq. 21 is provided in Figure A1.

542 A further, more complex approach for estimating  $R_{2\%}$  which more explicitly accounts for the short-  
 543 wave and infragravity components of significant swash height and may be more widely applicable  
 544 can be defined by decomposing  $S$  as shown in Eq. 6 and thus:

$$545 \quad R_{2\%} = 1.1 \left[ \bar{\eta} + \frac{\sqrt{S_{sw}^2 + S_{ig}^2}}{2} \right] \quad (23)$$

546 To apply Eq. 23,  $\bar{\eta}$  and  $S_{sw}$  can be estimated using Eq. 19 and 8 respectively using  $\gamma_{s,sw} = 0.59$  to  
 547 obtain  $H_{m0,sw}$ . At present, the most general approach to estimate  $S_{ig}$  is not clear but a range of  
 548 options exist. For the current dataset, Eq. 10 enables  $S_{ig}$  to be predicted with good skill as a function  
 549 of  $H_{m0,ig}$ , however as discussed below, there are uncertainties in the estimation of infragravity wave  
 550 height at the berm toe. As such, Eq. 15 which directly relates  $S_{ig}$  to  $l_{sz}$  is used to test Eq. 23 (see  
 551 Figure A2 for a suggested step-by-step approach to apply Eq. 23).

552 A range of different runup predictors were tested against the measured data (SALT, WWH and DR)  
 553 and the results are presented in Table 2. As shown by Poate et al (2016), their gravel beach predictor  
 554 (Eq. 1) greatly overestimated wave runup on a composite beach if the berm slope was used because  
 555 wave energy dissipation in the surf zone is much lower on a pure gravel beach than on the  
 556 dissipative sand slope of a composite beach. Much better performance was achieved by setting the  
 557 slope term in Eq. 1 equal to the sand gradient  $\beta_{sand}$  and the equation is simple to apply, requiring  
 558 only deepwater data. Nonetheless Eq. 1 underperformed all other methods because it does not  
 559 account for wave transformation across the dissipative sand slope. The general runup equation  
 560 presented in EurOtop (2018), Eq. 2 was found to perform poorly for all appropriate values of the  
 561 roughness factor  $\gamma_f$ , with the best results obtained using  $\gamma_f = 0.62$  which is at the bottom of the range  
 562 suggested for dynamic cobble berm revetments by Zaalberg (2019). Despite not being designed for  
 563 composite beaches, Eq. 3 (EurOtop, 2007) performed well if the empirical constant  $C_e$  was reduced  
 564 from 0.3 to 0.2. It is noted that Eq. 3 does not require a value for the berm gradient which could be  
 565 advantageous for practising engineers where the slope of an evolving composite beach ridge or  
 566 dynamic cobble berm revetment may not be known. **Caution is suggested however, because the**  
 567 **range of berm gradients within the SALT and WWH datasets which were used primarily to test the**  
 568 **equations is small ( $\tan\beta_{berm} = 0.15$  to  $0.25$ ). Unfortunately, no values of  $R_{2\%}$  are available for NC where**  
 569 **the berm slope is lower ( $\tan\beta_{berm} = 0.08$  to  $0.13$ ) than at the other field sites, however we can obtain**  
 570 **further insight into the applicability of Eq. 3 (with  $C_e = 0.2$ ) for composite beaches with lower gradient**  
 571 **berms by analysing the NC dataset.** If Eq. 18 is rearranged to make  $S$  the subject, we obtain:

$$572 \quad S = 2 \left( \frac{R_{2\%}}{1.1} - 0.17H_0 \right) \quad (24)$$

573 If Eq. 24 is then applied to estimate  $S$  at NC using values of  $R_{2\%}$  calculated using Eq. 3 (with  $C_e = 0.2$ ),  
 574 it has no predictive ability ( $r^2 = 0.11$ , RMSE = 1.83 m, NMSE = 72.8). Consequently Eq. 3 cannot be  
 575 recommended for use on composite beaches based on the current dataset.

576 The runup equations developed in this paper (eqs. 17, 21 and 23) were all found to have smaller  
 577 RMS errors than those observed for runup models on sand beaches (e.g. Stockdon et al., 2006;  
 578 Power et al., 2019) and NMSE values smaller than 0.08. Additionally, the new methods performed  
 579 better than the previously reported approaches (without modification of the coefficients), though  
 580 this is partly because they were developed and tested against the same dataset. As expected, Eq. 17  
 581 which simply relates  $R_{2\%}$  to  $d_{toe,swl} \tan\beta_{berm}$  performed the best because it was fitted directly to the  
 582 current dataset (excluding NC), however it is unclear whether this will be valid outside of the range  
 583 of the data presented here. Equations 21 and 23 break down the wave runup into separate setup

584 and swash components with the aim of making the approach more robust and generally applicable.  
 585 Both predictors tend to underestimate  $R_{2\%}$  because the value of  $\gamma_s = 0.87$  and equations 8, 9 and 15  
 586 were developed using the entire dataset including the NC data, but no valid measurements of  $R_{2\%}$   
 587 are available for NC for equation testing. The NC dataset has a tendency for slightly lower values of  
 588  $S_{ig}$  (Fig. 9b) and  $\gamma_s$  ( $\gamma_{s,NC} = 0.85$ ) which leads to underestimates of  $H_{m0}$  used in Eq. 21 and 23 and  $S_{ig}$   
 589 which is used in Eq. 23.

590 Due to the uncertainties associated with the prediction of  $H_{m0,ig}$ , and the contribution of infragravity  
 591 wave components to both significant wave height and significant swash height, it is presently  
 592 unclear whether Eq. 21 or 23 is the most general approach for predicting wave runup on composite  
 593 beaches. Part of this uncertainty arises due to the application of a cutoff frequency to separate, and  
 594 hence decouple, wave motions at short-wave and infragravity frequencies. Besides inevitably leading  
 595 to some short-wave energy being assigned to the infragravity band or vice-versa, this separation is  
 596 fundamentally questionable close to the shoreline, especially during energetic conditions.  
 597 Additionally, the behaviour of infragravity waves in the nearshore is complex, and parametric  
 598 models for predicting bulk parameters are far less accurate than in the short-wave frequency band.  
 599 Infragravity wave heights at the shoreline are strongly influenced by processes such as the  
 600 transformation of short-waves groups in both shoaling and surf zone regions (Thomson et al., 2006;  
 601 de Bakker et al., 2015; Mendes et al., 2018) and frequency-dependent reflection observed at low  
 602 frequencies in the nearshore (Elgar et al., 1994; Sheremet et al., 2002; Bertin et al., 2020). Through  
 603 their influence on  $H_{m0,ig}$ , such processes will directly affect the predictive skill of Eq. 21 and 23. From  
 604 an engineering point of view, Eq. 15 which can be used to directly estimate  $S_{ig}$  as a function of surf  
 605 zone width  $l_{sz}$  is potentially valuable when using Eq. 23 since the prediction becomes independent  
 606 from our limited capacity to predict the wave energy at infragravity frequencies. However, until  
 607 further work is completed to make progress on these issues, the authors recommend using Eq. 21 to  
 608 predict wave runup on composite beaches and dynamic cobble berm revetments due to the greater  
 609 simplicity of this approach.

610 **Table 2:** Summary of wave runup equation performance detailing the primary equation used and the secondary equations  
 611 applied to resolve the input parameters for the primary equation. See Figure A1 and A2 for a step-by-step approach to  
 612 apply equations 20 and 22.

Source	Primary Eq.	Secondary Eqs.	RMSE (m)	NMSE	Bias (m)
Poate et al. (2016)	1	$\beta = \beta_{berm}$	2.61	12.37	1.24
Poate et al. (2016)	1	$\beta = \beta_{sand}$	0.54	0.53	-0.42
EurOtop (2018)	2	$\gamma_s = 0.87,$ $\gamma_f = 0.62$ (Zaalberg, 2019)	0.56	1.59	0.31
EurOtop (2007)	3	22, $\gamma_s = 0.87, C_e = 0.3$	0.87	1.47	0.79
EurOtop (2007)	3	22, $\gamma_s = 0.87, C_e = 0.2$	0.23	0.10	-0.04
This paper	17		0.23	0.09	-0.04
This paper	21	22, $\gamma_s = 0.87$	0.27	0.14	-0.15
This paper	23	9, 15, 19, 22, $\gamma_{s,sw} = 0.59$	0.33	0.22	-0.19

613

614 The three new methods to predict wave runup on composite beaches (Eq. 17, 21 and 23) were  
 615 developed and tested using the same dataset. This is a common approach for the establishment of  
 616 wave runup equations and the dataset covered a range of beach geometries, water levels and wave  
 617 conditions. Nonetheless, it is valuable to investigate the robustness of the fits provided, and the  
 618 variability of the regression coefficients by re-fitting key equations using a subset of the complete  
 619 dataset. In the analysis below, the complete dataset was split at random into separate training and  
 620 validation datasets containing 75% and 25% of the data respectively. This process was repeated to  
 621 generate 100 different training and validation subsets. For each training dataset, linear regressions

622 were performed to re-fit runup equations 17 and 21, as well as equations 9 and 15 which are the key  
 623 parameterisations needed to apply Eq. 23. The performance of these equations was then evaluated  
 624 using the validation dataset.

625 It is evident from the summary of results presented in Table 3 that the mean values of gradient and  
 626 intercept obtained from this process are almost identical to those in the original equations and the  
 627 relative standard deviation is small (< 5%) except for the intercept of Eq. 15. For the intercept of Eq.  
 628 15, the standard deviation is small in absolute terms (0.03 m) and the apparently large relative  
 629 standard deviation is due to the small value of the mean. RMSE values are within 10% of those  
 630 obtained for the original equations. **The RMSE values for both the training and validation datasets  
 631 are comparable and in line with the values for the entire dataset, indicating that the fits presented  
 632 above are reasonably robust and not highly sensitive to the exact data used for the fitting process.**

633 Table 3: Summary of re-fitting analysis completed for key equations. Mean and relative standard deviation values [in  
 634 square brackets] are shown for the gradient and intercept of each equation. The mean and relative standard deviation of  
 635 the RMSE for both the training (RMSE<sub>train</sub>) and validation (RMSE<sub>val</sub>) datasets are also given.

Eq.	Gradient, m [RSD]	Intercept, c [RSD] (m)	RMSE <sub>train</sub> [RSD] (m)	RMSE <sub>val</sub> [RSD] (m)
9	6.80 [2.1%]	0.23 [4.4%]	0.15 [6.7%]	0.13 [7.7%]
15	0.003 [4.2%]	0.19 [16%]	0.25 [3.8%]	0.20 [7.0%]
17	4.60 [2.8%]	0.74 [4.1%]	0.25 [8.0%]	0.24 [20.8%]
21	3.11 [2.6%]	0.26 [3.9%]	0.31 [4.9%]	0.31 [15.0%]

636

637 A further evaluation of the potential performance of the new methods for data from outside of the  
 638 parameter space used to develop the equations was also undertaken. The data corresponding to the  
 639 highest energy conditions (WWH, 2<sup>nd</sup> November) were removed as a validation dataset, with the  
 640 remainder used for training. The results of this process are shown in Table 4. It is evident that the re-  
 641 fitted equations are able to predict wave runup for the whole  $R_{2\%}$  dataset (SALT, WWH, 1DR and  
 642 2DR) with a comparable precision to the original forms of equations 17, 21 and 23. Furthermore,  
 643 when tested against the high-energy validation data subset, only a slightly increased RMSE and  
 644 marginally greater negative bias is observed. NMSE values are mostly smaller due to the reduced  
 645 range of values covered by the validation dataset. While it is recommended that the methods  
 646 developed in this study should be thoroughly tested against new datasets as they become available,  
 647 this result gives some confidence that all three are reasonably robust and capture the main  
 648 processes causing wave runup on composite beaches.

649 Table 4: Summary of re-fitted wave runup equation performance. Statistical parameters are given for the entire  $R_{2\%}$   
 650 dataset and the high-energy validation subset (WWH, 2<sup>nd</sup> November) [square brackets].

Re-fitted Eq.	RMSE (m)	NMSE	Bias (m)
17	0.24 [0.30]	0.11 [0.12]	-0.09 [-0.20]
21	0.24 [0.28]	0.11 [0.04]	-0.12 [-0.14]
23	0.31 [0.38]	0.18 [0.07]	-0.17 [-0.29]

651

## 652 5. Conclusions

653 This paper describes new measurements of swash using 2D scanning Lidar on composite beaches  
 654 and dynamic revetments during a series of field and large-scale laboratory experiments.

655 It was demonstrated that the significant swash height  $S$  increases substantially as the swash zone  
 656 moves from the dissipative sand beach to the reflective gravel berm during a rising tide. When the  
 657 swash zone is on the sand beach, the total significant swash height is dominated by swash motions

658 at infragravity frequencies ( $S_{ig}$ ), however as the berm toe becomes submerged, the significant short-  
 659 wave swash height  $S_{sw}$  increases rapidly, leading to the observed increase in total significant swash  
 660 height.

661 The wave height at the toe of composite beach berms and dynamic revetments is decoupled from  
 662 the offshore wave conditions and primarily controlled by the water depth at the berm toe during  
 663 high tides when inundation risk is greatest due to the dissipative nature of the fronting sand beach.  
 664 As a result, significant swash height was found to be highly correlated to water depth, particularly at  
 665 short-wave frequencies. Infragravity wave height at the berm toe, and hence significant infragravity  
 666 swash was also found to vary with water depth. There is evidence that this is due to the changing  
 667 surf zone width with water level on the two-slope composite beach geometry.

668 Based on the new insight gained from the experimental results, three new methods with differing  
 669 complexity were developed to predict wave runup on composite beaches and dynamic revetments  
 670 incorporating wave setup, short-wave and infragravity swash components. These new composite  
 671 beach-specific methods were found to provide good predictions of extreme wave runup and will  
 672 enable more robust design of dynamic cobble berm revetements and assessment of coastal hazards  
 673 at composite beaches. It was noted however that the proposed methods are partly restricted by  
 674 uncertainties related to infragravity swash motions and further research in this area will be valuable.

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## 692 Appendices

693 **Table A1:** List of symbols.

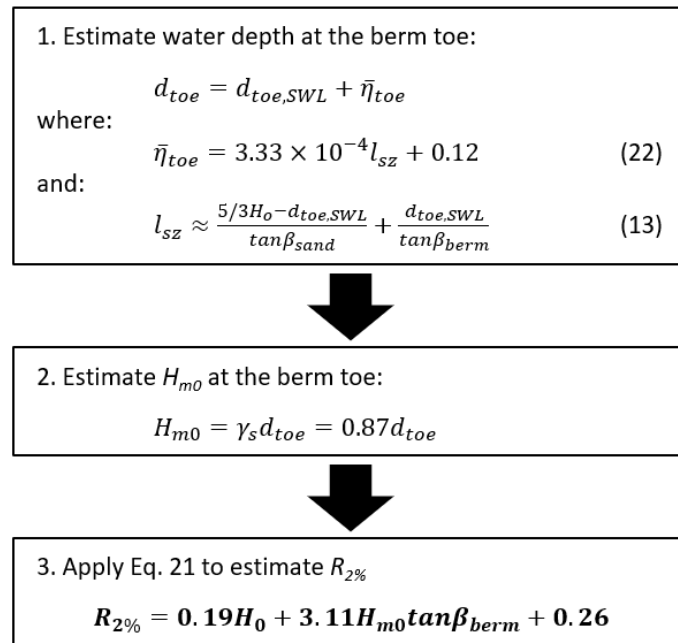
Symbol	Definition
$d_{toe}$	Water depth above berm toe (m)
$d_{toe,SWL}$	Vertical elevation difference between berm toe and SWL (m)
$D_n$	Sediment diameter exceeded by $n\%$ of particles
$H_o$	Significant wave height measured offshore (m)
$H_{m0}$	Spectral significant wave height at berm or structure toe (m)
$H_{m0,sw}$	Short wave spectral significant wave height at berm or structure toe (m)
$H_{m0,ig}$	Infragravity spectral significant wave height at berm or structure toe (m)

*K*

$L_o$	Deep water wavelength (m)
$l_{sz}$	Composite beach surf zone width (m)
$MWL_{toe}$	Mean water surface elevation at the berm toe within 10-minute time windows (m)
$R_{2\%}$	Runup elevation exceeded by 2% of incident waves (m)
$S$	Total significant swash height (m)
$S_{sw}$	Significant short wave swash height (m)
$S_{ig}$	Significant infragravity swash height (m)
$S_{om}$	Wave steepness calculated using $T_z$
SWL	Mean water surface elevation seaward of the surf zone within 10-minute time windows (m)
$T_z$	Mean zero-crossing wave period (s)
$T_p$	Spectral peak wave period (s)
$T_{m-1,0}$	Mean spectral wave period (s)
$X_s$	Horizontal shoreline position (m)
$Z_s$	Vertical shoreline position (m)
$z$	Vertical elevation above datum (m)
$z_{toe}$	Elevation of berm toe above datum (m)
$z_{WL}$	Water depth above flume bed at DR and 2DR (m)
$\beta_{sand}$	Angle between the mean sand beach slope and horizontal
$\beta_{berm}$	Angle between the mean gravel berm slope and horizontal
$\beta_{struct}$	Angle between coastal structure slope and horizontal
$\beta_{gravel}$	Angle between mean pure gravel beach slope and horizontal
$\beta_{swash}$	Angle between the mean swash slope and horizontal within a 10-minute time window
$\bar{\eta}$	Wave setup at the shoreline (m)
$\bar{\eta}_{toe}$	Superelevation of the mean water level at the berm toe due to wave setup (m)
$\gamma_f$	Reduction factor to account for the slope roughness in Eq. 2
$\gamma_s$	Wave height to water depth ratio at the berm toe
$\gamma_{s,sw}$	Short wave height to water depth ratio at the berm toe
$\gamma_{s,ig}$	Infragravity wave height to water depth ratio at the berm toe
$\zeta$	Linearly detrended shoreline elevation

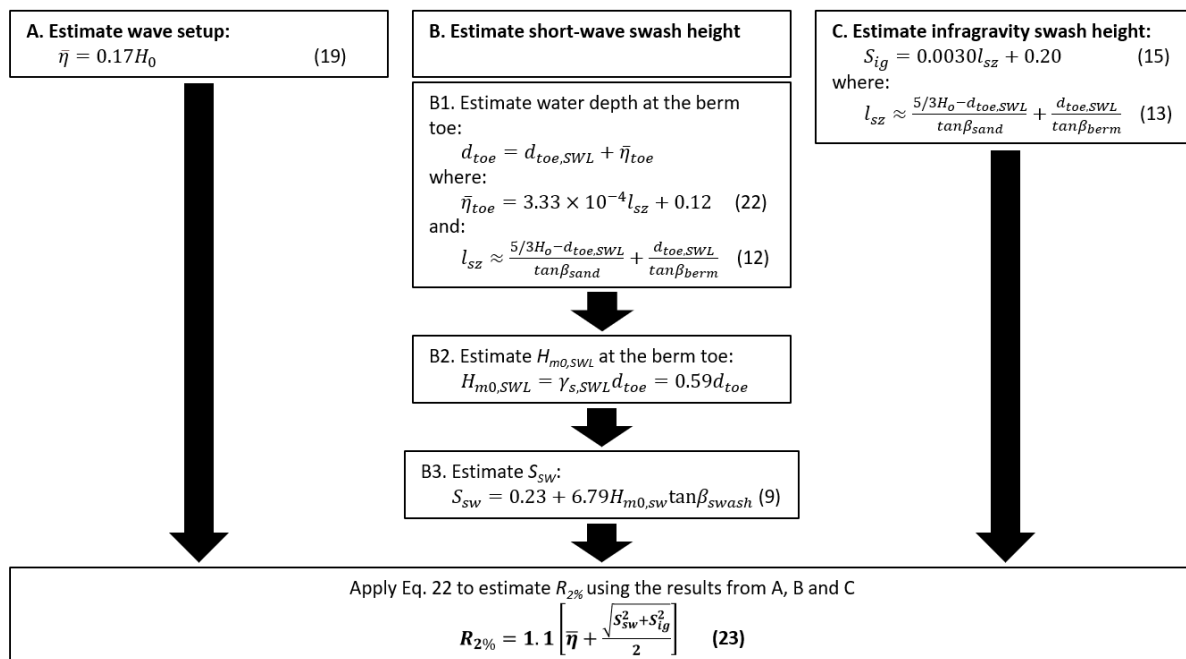
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696

697 **Figure A1:** Suggested step-by-step procedure for application of Eq. 21.



698

699 **Figure A2:** Suggested step-by-step procedure for application of Eq. 23.

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