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# In-situ measurements of energetic depth-limited wave loading

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## Abstract

An extensive database of in situ measurements of wave impact pressure on the wall of a composite breakwater and associated explanatory variables (i.e., waves, wind, and water level) was collected in a particularly high-energy wave environment. Due to the bottom profile, which includes a wide mound of concrete blocks with a seaward edge that rises to the Lowest Astronomical Tide level over a large distance, most waves break before reaching the monitored barrier, making the data set representative of depth-limited wave loading. Maximum pressure is consistently found at the sensor position closest to the mean free surface, and statistically, a decrease of maximum pressure with the altitude is observed. Nevertheless, the database also shows a wide variety of vertical profiles of maximum pressure. A detailed analysis of the pressure signal shows that there are two impact classes associated with large pressure values. The first is mostly observed during stormy conditions with relatively slow pressure variations over time and a fairly uniform spatial distribution. The second class exhibits very limited pressure peaks in time and space and is most often observed during moderate sea states and high water levels. The pressure signals for each class agree well with the prediction of the PROVERB [35] impact classification based on breakwater dimensions,

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water level, and wave parameters. Our dataset also shows the existence of impacts with simultaneous large pressure and rise time, which extends the range of the observed values of the pressure impulse already reported in-situ [10] from 31000Pa.s to 100000Pa.s. For strong impacts that show regular upward propagation, the peak pressure propagation velocity was estimated and most values remain below 20 m/s. Finally, we performed a statistical analysis to relate the maximum pressure to a number of variables, including offshore wave parameters, water level, and wind parameters. The multiple linear regression model created confirms the dominant influence of wave height and the negligible influence of the wind on pressure maxima. The other wave parameters are still significant but of secondary importance. Finally, the upper bound of the 99% prediction interval predicts the maximum pressure given a set of wave and water level conditions with a good confidence level which is an encouraging result.

*Keywords:* wave, impact, breaking wave, loading, breakwater, field measurement, pressure impulse, multiple linear regression, wind, water level

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## 1 Introduction

Understanding wave loading on coastal structures, in the broad sense (i.e., either artificial like breakwaters or natural like cliffs), is a necessary step to predict their stability. It remains a challenge owing to the complexity of the phenomenon which involves wave impact at the wave scale in a full three-dimensional environment.

Wave impact has been extensively studied since the beginning of the 20th century. Sainflou [41] and then Rouville et al. [40] first identified the existence of different types of impact. This typology mainly depends on the position of the breaking point with respect to the obstacle [25]. When the fictitious breaking point is located after the obstacle, the wave interacts with it as a sloshing flow, more and more violent as the breaking point gets closer to the obstacle. The critical case, in which the breaking point and obstacle share the same location, is called the flip-through impact [15]. The latter generates a very high-pressure peak in a short time. Sloshing and flip-through impacts mainly involve incompressible flows for which the pressure is well predicted by the theory developed in Wagner [49]. Compressible processes start to play a role as soon as air is involved in the phenomenon (i.e., when breaking starts just before the obstacle) either as small bubbles advected in the wave or as

20 a formed air pocket entrapped between the wave and the obstacle. Bullock  
21 et al. [12] categorized low and high aeration impacts. The first one occurs  
22 when the impacting wave contains relatively little air (typically less than  
23 5%) and produces a high and short-duration pressure spike. In the second  
24 one, a well-defined cavity is entrapped, and the impact produces a short  
25 duration pressure peak followed by specific damped oscillations attributed to  
26 the variation of the air pocket volume. Bagnold [3] first proposed a model  
27 to account for this particular process. These phenomena have been studied  
28 extensively either numerically or experimentally in the last decades (e.g.,  
29 [34, 36, 8, 43] among others).

30 This extensive knowledge is difficult to transpose to real situations ow-  
31 ing to the complexity of the structures usually considered as well as of the  
32 surrounding environment itself. Seemingly, there are also differences in the  
33 wave impact process itself between laboratory and field conditions illustrated  
34 by maximum pressure value differences observed in both environments (i.e.,  
35 5600 kPa in controlled environment [8] and a little less than 800 kPa at full  
36 scale [11]). Peregrine [36] identified a few factors prone to damp maximum  
37 impact pressure in natural conditions compared to calibrate impacts in wave  
38 flume or numerical models. In particular, the three-dimensional effects in-  
39 herent to real waves as well as the breakwater block armor units are likely to  
40 diminish impact pressure. Aeration processes, different in fresh and salted  
41 waters, and scaling in general make it also difficult to compare flume and field  
42 measurements [9, 7, 6, 37]. On the other hand, Peregrine [36] also mentioned  
43 that bathymetric wave focusing could conversely increase impact pressure in  
44 some particular cases.

45 The analysis of field data could help reduce the gap existing between the  
46 controlled and the natural environments [35]. Unfortunately, the amount  
47 of available data appears quite limited. One of the reasons for that is the  
48 difficulty to perform accurate and long-term measurements in such harsh con-  
49 ditions. Surprisingly, one of the most comprehensive data was obtained quite  
50 early in Rouville et al. [40] more than seven decades ago. Three piezo-resistive  
51 sensors, distributed vertically 1m apart, were installed on the slightly inclined  
52 wall of the Dieppe breakwater (northwest of France) around the mean water  
53 level (MWL) with a tide range of around 10 m. The bathymetry profile was  
54 composed of a pebble beach followed by masonry of slope 1:5 and finally the  
55 equipped wall. Rouville et al. [40] describes variable wave conditions includ-  
56 ing waves breaking directly on the wall trapping air. The maximal pressures  
57 recorded range from 180 to 690 kPa with breaking wave heights estimated

58 from 2 to 3.5m. Blackmore and Hewson [5] conducted in-situ wave impact  
59 measurements at the Ilfracombe seawall using seven pressure sensors over  
60 the vertical (from about 2 to 6 m ). The waves were estimated visually at  
61 around 1m during the campaign which resulted in 112 impacts of less than  
62 48 kPa. Although it is not indicated, according to the beach profile, the  
63 wave impacts measured should have been produced by broken waves. The  
64 few data obtained show linearly decreasing dynamic pressure with heights.  
65 More recently, Bullock et al. [10] carried out extensive wave impact measure-  
66 ments at the Alderney breakwater. They used a specifically designed sensor  
67 to record both pressure and aeration [4]. In that study, the maximal recorded  
68 pressure was 337 kPa at 6.2 m (Chart Datum) with weaker values at higher  
69 levels. Significant wave heights up to 3.1 m were reported. Bullock et al. [11]  
70 later obtained a maximal pressure of 745 kPa at the same site. The wave  
71 conditions which generated this value are not indicated. This paper also  
72 points out the lack of accessible field data for extreme conditions. Recent  
73 field measurements of wave impact have brought their interest to new site  
74 types. Cliff ground motion due to wave action was for instance conducted  
75 in Thompson et al. [48]. They showed that the dominant control on peak  
76 ground motion magnitude was wave impact type rather than the incident  
77 significant wave height, the larger ground motion being produced by wave  
78 breaking directly onto the cliff face compared to broken or unbroken wave  
79 impacts. Measurements of wave, current, and structure accelerations were  
80 conducted on La Jument offshore lighthouse under extreme wave conditions  
81 (a 19m high wave) in Filipot et al. [19].

82 While worth investigating, the link between field wave impact pressures  
83 and the expected explanatory variables, i.e. meteo-marine forcing conditions  
84 such as wave features and water level, appears significantly understudied in  
85 the literature. This is surprising as better knowledge in this direction could  
86 help improve the accuracy of the current empirical design formulas. Hence,  
87 for instance, Goda [20] and Takahashi [47] used to calculate the design load,  
88 a "design wave", assumed to be the largest wave in the design sea-state  
89 at the toe of the structure [21, 20, 35]. More specifically, these formulas  
90 require the design wave height, length and incident angle as input parameters  
91 among other ones (such as the geometrical parameters, etc.). In the field, the  
92 situation is more complex than in the laboratory, numerous interacting modes  
93 between the wave field, water level and the structure specificity can result  
94 in different pressure distributions on the structure. Regarding this aspect,  
95 the PROVERB project [35, 30], again, proposed a classification of impact

96 types depending on the breakwater dimensions, wave height and water-level  
97 illustrating this strong link. The water level influence on the breaking wave  
98 impact on vertical or quasi vertical walls is also stressed in Kirkgöz [27]. Wind  
99 may play a role in the phenomenon for instance by slightly shaping the wave  
100 [46] which may finally influence the local pressure field. Finally, Salvadori  
101 et al. [42] insists on the importance of the coupled influence of parameters  
102 on the critical load requiring a multivariate analysis to be relevant.

103 There are actually very few studies of broken wave loading as this con-  
104 dition is generally dismissed as not being very important. Nevertheless, al-  
105 though maximum pressures are known to be generated by waves breaking  
106 directly onto the structure, and especially in the case of low aeration im-  
107 pacts, broken wave loading should not be neglected, as they seem to produce  
108 a longer loading by increasing both the rise time and duration of the impact  
109 [12]. This feature may increase both the force and impulse on the structure  
110 [12].

111 In this paper, we propose high-frequency wave impact pressure measurements  
112 conducted at the Artha breakwater in Saint Jean de Luz Bay, south-west of  
113 France (See Larroque et al. [32] for a preliminary presentation of the exper-  
114 iment). Pressure data was collected in a wide range of forcing conditions,  
115 including very energetic wave conditions with significant wave heights up to  
116 7.5 m. The data is now public and can be found in Abadie et al. [1]. Due to  
117 the particular bottom profile involving a wide mound of concrete blocks with  
118 a seaward crest that rises to the Lowest Astronomical Tide level over a large  
119 distance, most of the waves break before reaching the monitored obstacle  
120 and therefore the database is representative of depth-limited wave loading.  
121 In the present paper, input data such as significant wave height, significant  
122 period, mean swell direction, water level, wind speed and wind direction were  
123 simultaneously collected and added to the pressure data to form an extensive  
124 dataset suitable for statistical analysis. The objective of this statistical study  
125 is to better understand the role of the environmental variables on wave im-  
126 pact pressure in natural conditions. To our knowledge, this is the first time  
127 that this relation is investigated in the field.

128 In the first part of the article, the method employed for the measure-  
129 ments and the statistical models will be described. Then the results will be  
130 presented and finally, the discussion addresses several aspects of the problem.

131 **2. Method**

132 *2.1. Presentation of the experiment*

133 *2.1.1. Configuration of the field site*

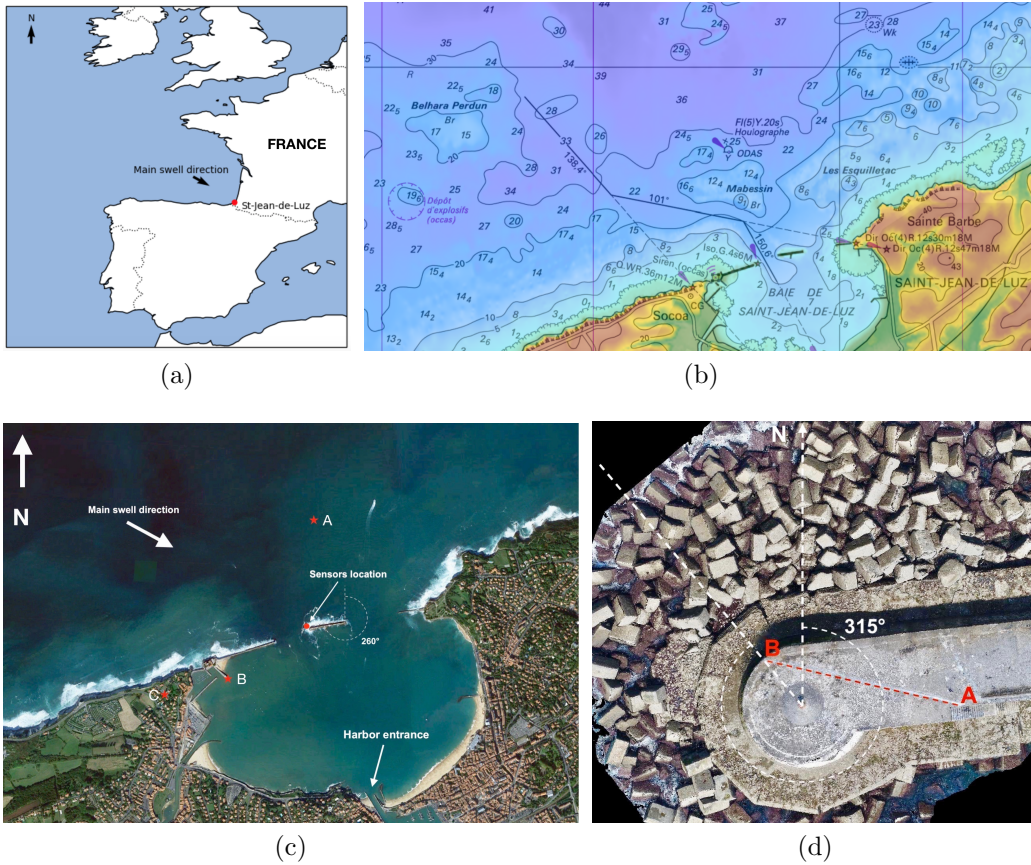


Figure 1: Presentation of the study site with : a) Geographical position (red dot), b) Aerial view of the bay of St-Jean de Luz. The location of the sensors on the breakwater is indicated by a red dot. The red stars correspond to (A): The directional wave buoy, (B): Socoa tide gauge, (C): Socoa Semaphore. Source: Google earth V 7.3.2.5776. (2019/03/05). St-Jean-De-Luz, France.  $43^{\circ}23' 53''\text{N}$ ,  $1^{\circ}40' 27''\text{W}$ , Eye alt 4 km. Maxar Technologies 2020, c) Bathymetry in the area of the Artha breakwater (depths in m - source SHOM website), d) Ortho-photograph of the Artha breakwater western roundhead. (A) : Acquisition station (B) Sensor location. The trench connecting the sensor to the acquisition station is visible along the red dashed line. The azimuth of the sensors to roundhead center axis is indicated. source: LP GEO 3D - Lycée Cantau / C. Bagieu.

134 The site considered for this study is the Artha breakwater in St-Jean  
135 de Luz, french basque coast, France (Figure 1 (a)). The Artha breakwater  
136 was built between 1872 and 1895 to protect the towns of St-Jean de Luz  
137 and Ciboure from flooding and erosion. Numerous concrete blocks were first  
138 immersed on an existing reef to create a high and stable bedrock. A large  
139 central caisson, made up of a mix of concrete and stones, surrounded by a  
140 concrete berm was then built on that foundation. The armour, made of more  
141 than 20000, 50 tonnes, cuboid,  $4 \times 2.5 \times 2 \text{ m}^3$  concrete blocks, defends the  
142 structure. The recurring displacement of blocks by the waves requires adding  
143 about 30 concrete elements every year. In addition to these maintenance  
144 operations, repairs of the masonry are also frequently required.

145 A directional wave buoy (Figure 1 (b) and (c)), part of the national CAN-  
146 DHIS network <http://candhis.cetmef.developpement-durable.gouv.fr/>,  
147 provides swell measurements 1 km offshore of the breakwater, in 20 m depth.  
148 The Socoa tide gauge is located within the bay of Saint Jean de Luz, less  
149 than 1 km from the pressure sensors. Socoa semaphore is 1km away and  
150 provides wind data.

151 The pressure sensors are located on the offshore facing side of the caisson,  
152 at the western roundhead (Figures 1 (c) and (d), and Figure 3) on the most  
153 exposed location. This position was determined based on observations of  
154 damages and climatology of swell parameters measured at the buoy.

155 The depth in front of the western round head is of 15m, but the bathymetry  
156 in the area is fairly complex with shallower areas on the eastern part and  
157 in front of Socoa breakwater (Figure 1 (b)). Several reefs and troughs can  
158 be identified which likely affect wave transformation from the offshore do-  
159 main to the toe of the breakwater making refraction and shoaling processes  
160 complex in this area. Additionally, wave breaking more than 1km offshore is  
161 regularly observed during the strongest winter storms. In the bay, the depth  
162 is globally lower than 10 m.

### 163 *2.1.2. Wave climate and tides*

164 Kergadallan et al. [26] provides an overview of the statistical character-  
165 istics of the wave climate based on 7 years of measurements. The average  
166 climate is characterized by a percentile 50% of 1.26 m and 80% of 2.27 m  
167 for the spectral significant wave height  $H_{m0}$  (respectively 1.76 m and 2.93 m  
168 in winter) with an associated peak period  $T_p$  between 10 s and 12 s (respec-  
169 tively 11 s and 13 s). The 5-year return level of the significant wave height is  
170 7.43 m (respectively 7.66 m and 7.85 m for the 10-year and 20-year values)



171 with 70% confidence interval between 7.09 m and 7.59 m based on a GPD  
 172 fitting.

173 The peak direction of storm waves is predominantly from 315 °, with some  
 174 events also reported from north-north-west. The tidal regime is mesotidal  
 175 with 4.5 m spring tidal range around a mean water level of 2.64 m (C.D.).

176 *2.1.3. Load expected at the Artha breakwater*

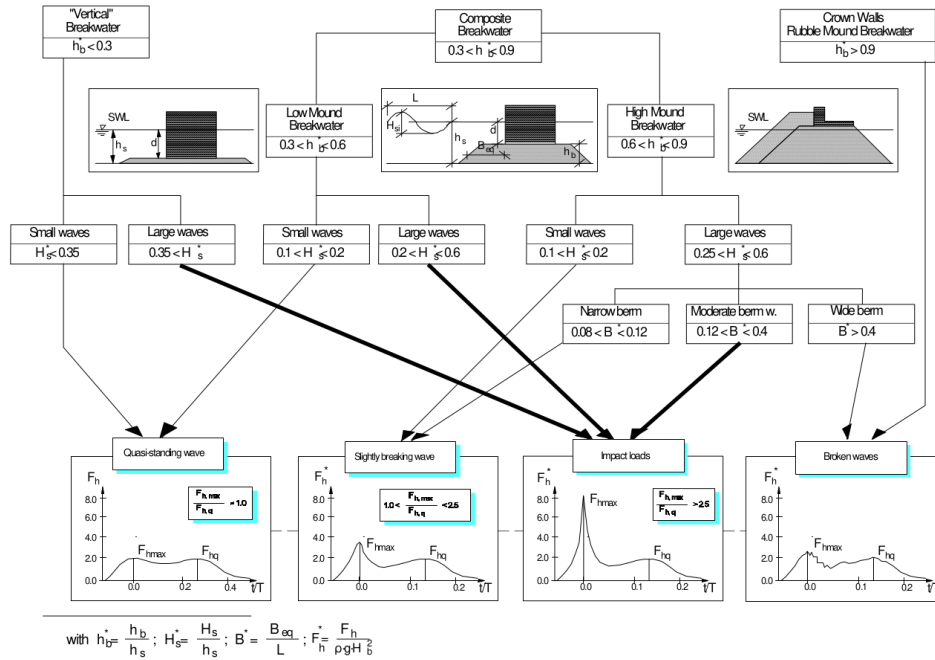


Figure 2: Typology of impacts in function of the breakwater type and geometry and waves and water level conditions. Source : PROVERB project chapter 2.2 [35]

177 The PROVERB classification (Figure 2) provides the shape of the load  
 178 expected for a specific breakwater depending on three parameters :  $h_b^* =$   
 179  $h_b/h_s$ ,  $H_s^* = H_{si}/h_s$ ,  $B^* = B_{eq}/L_{pi}$ , with  $h_s$  the depth at the toe of the  
 180 breakwater,  $h_b$  the vertical distance from the toe to the berm,  $B_b$  the width  
 181 of the berm,  $H_{si}$  and  $L_{pi}$  the wave height and length at depth  $h_s$  and  $B_{eq} =$   
 182  $B_b + h_b/2tan(\alpha)$ .

183 To determine the category in the PROVERB classification to which the  
 184 Artha breakwater belongs, the geometrical characteristics of the structure as  
 185 well as the water level matter. Figure 3 shows two cross-sections of the Artha

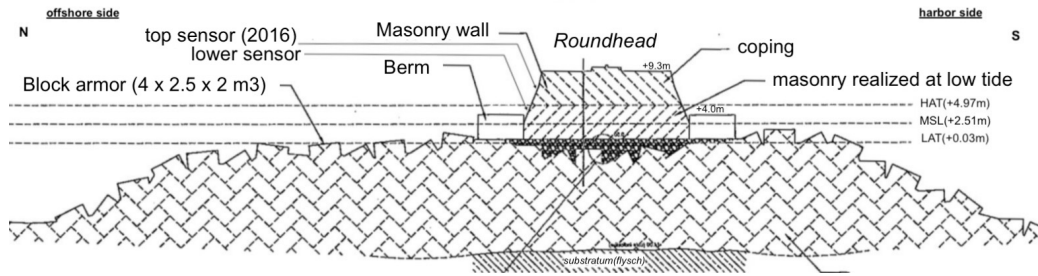
186 breakwater. According to this, we may consider the averaged berm elevation  
 187 as  $h_b = 16.5$  m for the calculation. Then, considering the tide amplitude,  
 188  $h_s$  ranges approximately between 14.5 m and 19.5 m (Figure 3 (b)) leading  
 189 to  $h_b^*$  varying from  $\approx 0.85$  for very high water levels to  $\approx 1.13$  for very  
 190 low water levels. Therefore, for low to intermediate water levels, the Artha  
 191 breakwater is considered as a crown wall rubble mound breakwater i.e., all  
 192 the waves break before reaching the wall. In this situation, the load is likely  
 193 to resemble the one depicted on the broken waves panel in Figure 2. At very  
 194 high water levels, the breakwater should work as a composite breakwater and  
 195 the load consequently depends on several other parameters.  $h_b^*$  being larger  
 196 than 0.6, the breakwater is considered as a high mound breakwater. Small  
 197 waves in the PROVERB classification (i.e.,  $H_s^* < 0.2$ ), which represent a  
 198 maximum  $H_s$  of  $\approx 3 - 4$  m, are supposed to generate slosh impact or slightly  
 199 breaking wave impact as in the second panel of Figure 2. For larger waves,  
 200 (say,  $H_s > 5$  m) the relative berm  $B^*$  has to be calculated. Periods associated  
 201 to such wave heights are expected to range between 15 and 17 s [? ]. The  
 202 calculation shows that the breakwater falls in the moderate berm category  
 203 for which impulsive (impact loads in Figure 2) loads are likely to occur.

#### 204 2.1.4. Sensors and acquisition station

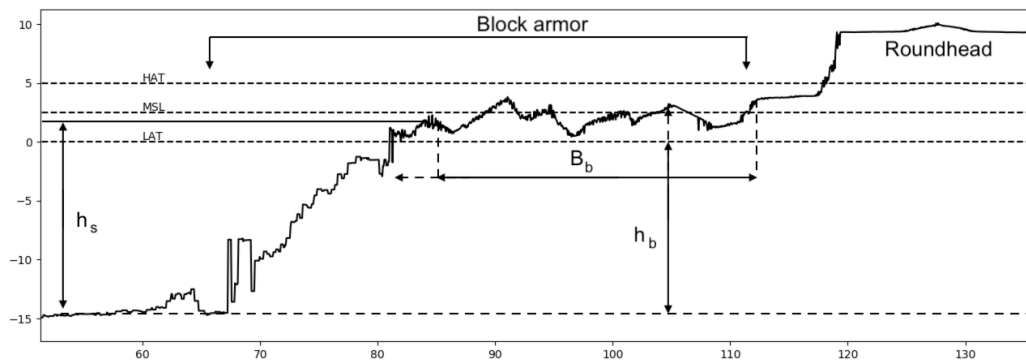
205 To achieve high-frequency measurement in marine grade conditions, piezo-  
 206 resistive sensors with a response frequency of 1 kHz and IP68 protection  
 207 rating were selected.

208 A data acquisition station is installed on the protected side of the break-  
 209 water, in a dedicated chamber [32]. It is connected to the sensors by a trench  
 210 of more than 30 m, on the top of the roundhead, as seen in Figure 1 (d). The  
 211 isolated nature of the site implies using a flexible and remotely programmable  
 212 data acquisition system. To that end, a CompactRIO 9076 from National  
 213 Instruments© was deployed with four NI-9203 modules (20  $\mu$ A, 8 canals,  
 214 16 bits). With a 16 bit encoding, the digitization precision is suited to the  
 215 accuracy of the sensors. The acquisition frequency of 10 kHz was selected  
 216 with respect to the Shannon–Hartley theorem Shannon [44]. To download  
 217 the data and modify the recording parameters if needed, a wireless system  
 218 is used to communicate with the data acquisition station. To ensure the  
 219 durability of the measurements, the installation is powered by a photovoltaic  
 220 panel and a high-capacity battery with a solar charge controller. This setup  
 221 allows measurements of wave impact over extended periods.

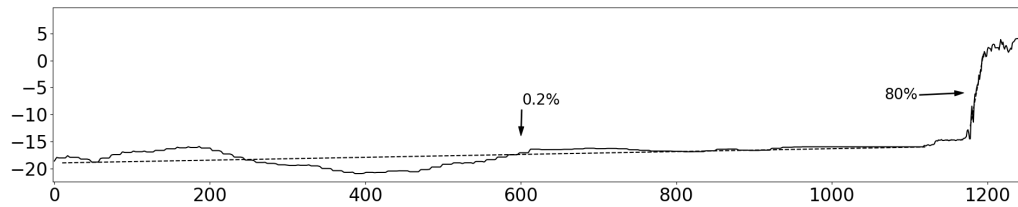
222 A preliminary campaign with two sensors vertically distant of 1.85 m was



(a)



(b)



(c)

Figure 3: (a) Schematic cross section of the Artha breakwater roundhead perpendicular to the structure trunk. The main structural elements are mentioned as well as the position of the sensors during the 2016 campaign (b) Bottom level measured in a cross section of the Artha breakwater western roundhead along the axis passing by the lower sensor and the center of the roundhead. The equivalent PROVERB parameter are indicated. Altimetry is in chart-datum.(c) Bathymetric profile from the directional wave buoy to the Artha western roundhead wall. The two dotted lines are linear regressions of the two obvious distinct areas, namely, from the buoy to the structure toe and the block armor slope. The respective slopes of these regression lines are also indicated.

223 first performed in 2015-2016 [32]. In 2018, the installation was upgraded with  
 224 15 new sensors while the upper sensor of 2016 was removed. This provides  
 225 a set of 16 sensors including the lower sensor of 2016. The two set-ups are  
 226 presented in Figure 4. The highest water levels reach an area just above the  
 227 sensor #0 and it is well known that the maximum impact pressure is more  
 228 likely to occur around the water level. Therefore, a better placement of this  
 229 new set-up would have been between the two older sensors. Unfortunately,  
 230 this option was not accepted by the breakwater maintenance service who  
 231 feared the embrittlement of the wall in this area (Figure 4 (b)). Because of  
 232 this, the new set-up is not ideally placed vertically, and this is undoubtedly  
 233 a limitation of our study.

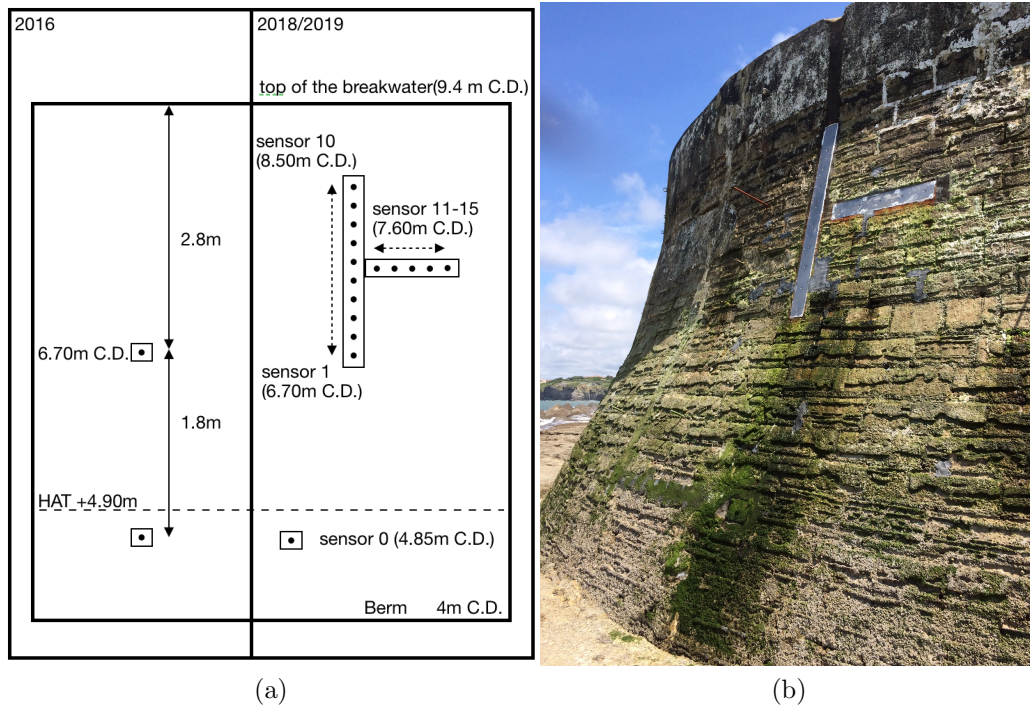


Figure 4: (a) Disposition of the sensors for both campaigns with the level of the berm, the level of the Highest Astronomical Tide (b) photograph of the installation (taken by the author)

234 *2.2. Data overview*

235 *2.2.1. Environmental variables*

236 The environmental parameters, whose measurement location is indicated  
237 in Figure 1 (b) are collected simultaneously with the impact pressures :

- 238 • 28 variables describing the sea state determined from 30 minutes ob-  
239 servations, every hour, at the buoy A,
- 240 •  $w_l$  : water level as measured at B at the time of the burst, hourly,
- 241 •  $w_s$  and  $w_d$  : respectively mean wind speed and direction over 10 minutes  
242 as measured at C every hour,

243 *2.2.2. Wave impact pressure data*

244 The procedure of pressure acquisition and post-processing is described in  
245 detail in [18]. The acquisition station records wave impact pressure hourly  
246 in 10 minutes burst, with a sampling frequency  $Fe = 10000$  Hz. An embed-  
247 ded threshold, just higher than the atmospheric pressure, allows saving only  
248 significant signals. With this threshold, approximately 300 to 400 bursts of  
249 10 minutes are obtained every month. For each sensor, the maximal pres-  
250 sure observed over the 10 minutes burst provides an observation of  $P_{max}$ , our  
251 variable of interest or response variable.  $P_{max}$  is given after removing the  
252 offset due to atmospheric pressure.

253 The preliminary campaign carried out in 2015-2016 winter lasted four  
254 months, yielding 461 observations. In 2018-2019, the first successful tests  
255 were conducted in June and the main campaign extended from October 2018  
256 to April 2019. A malfunction shut the station down between the 2018/12/07  
257 and 2019/01/17, period during which, the waves conditions were mostly mod-  
258 erate. Overall, this second campaign resulted in 1390 observations.

259 The network of 16 sensors used during the campaign of 2018-2019 aims at  
260 giving an insight on the pressure spatial distribution. The spatial distribution  
261 is also used to evaluate the force by linear meter exerted on the wall. The  
262 calculation is made by integrating the instantaneous vertical distribution  
263 of pressure using a trapezoidal integration rule assuming pressure linearity  
264 between sensors. The linear profile is extended up to the top and down to the  
265 bottom to obtain the full force. Note that with the trapezoidal integration  
266 rule, sensor 0 has a major weight in the determination of the force as  $z_1 - z_0 =$   
267 1.9 m while the next  $\Delta z = 0.2$  m, with  $z_i$  the altitude of the sensors. The  
268 bursts have been automatically processed by d'Amico et al. [18], 152999  
269 impacts were identified overall in 2018-2019.

270 *2.2.3. Rise time, pressure impulse, pressure peak propagation celerity and*  
271 *force*

272 Rise times and the impulse pressures statistics computed from the data  
273 of the 2018/2019 measurements campaign are also briefly analyzed in this  
274 paper. The rise time  $T_r$  is defined as the time between the beginning of the  
275 impact and the pressure peak for the highest pressure recorded in the burst.  
276 Pressure impulse, which corresponds to the time integral of the impulsive  
277 pressure peak is here approximated by  $T_r \times P_{max}$ . The pressure peak vertical  
278 celerity has been determined for the 2018-2019 dataset. The computation is  
279 limited to sensors 1-10, regularly spaced at 0.2 m intervals (sensor 0 was  
280 not considered due to its shifted location compared to the other sensors).  
281 Between two sensors, the celerity is calculated considering the peak pressure  
282 dates at each location and the distance between the two sensors.

283 Regarding the determination of the force from pressure measurements,  
284 Lamberti et al. [31] showed that the result is strongly dependent on the pres-  
285 sure sampling frequency and the distance between sensors. In order to obtain  
286 physical results, the sampling frequency has to be lower than the frequency  
287 corresponding to the inverse of the propagation time of the pressure peak  
288 between two sensors. Again, as sensor 0 is both horizontally and vertically,  
289 very distant from the rest of the array, the estimation of the force was per-  
290 formed only for sensors 1 to 10. Note that this computation can only be  
291 performed for impacts with a clear upward propagation of the pressure peak.

292 *2.3. Statistical model*

293 The relationship between the explanatory environmental variables and  
294 the response variable  $P_{max}$  is examined with multiple linear regression. At  
295 each timestamp  $i$ , the observation of  $P_{max}$  is expressed as a function of the  $i^{th}$   
296 explanatory variable observation plus a residual  $\epsilon_i$ . The linear coefficients  $\beta_j$   
297 associated with the explanatory variables are estimated using a least-squares  
298 method. By averaging the variable importance across all ordered combina-  
299 tions of variables, the so-called LMG metric [33, 22] determines the propor-  
300 tion of variability attributable to each variable. In parallel, the colinearity  
301 between variables has to be checked to ensure that it is low enough to be  
302 acceptable. For that purpose, the Variance Inflation Factor [39] is employed.  
303 A VIF below a threshold of 3 is usually accepted (see [13, 29, 23]).

### 304 **3. Results**

#### 305 *3.1. Data analysis*

##### 306 *3.1.1. Environmental variables*

307 The VIF analysis yielded the following 13 variables as noncolinear: the  
308 maximum period, the lowest measured free surface height, the maximum  
309 steepness, the period of the steepest wave, the skewness, the kurtosis, the  
310 correlation coefficient between successive wave heights, the spectral narrow-  
311 ness, the peak direction, the directional dispersion, the water level, the wind  
312 speed and direction. A multiple linear regression on this set of variables  
313 showed that only 4 variables among the 13 are significant, namely: the low-  
314 est free surface height, the maximum period, the peak direction, and the  
315 water level. One of the first results is therefore the negligible influence of  
316 the wind on the response variable (i.e., the maximum pressure over a time  
317 window). The 3 sea state variables can actually be replaced by  $H_{1/3}$ ,  $T_{H_{1/3}}$ ,  
318 and  $\theta_m$ , more classically used for scaling purposes, without noticeably de-  
319 grading the statistical model (adjusted  $R^2$  of 54% instead of 58%), as they  
320 are strongly related to their counterpart.

321 Figure 5 presents the histogram of the latter explanatory variables for  
322 different thresholds of the response variable. The first column plots corre-  
323 spond to the distribution of the explanatory variables for all the impacts  
324 recorded by the station. As stated in [18], the embedded pressure threshold  
325 which triggers the data acquisition, is only slightly larger than the atmo-  
326 spheric pressure. These plots show how explanatory variables are almost  
327 normally distributed. Note that as the campaign duration is limited and due  
328 to the embedded threshold triggering the acquisition station, the data are  
329 therefore slightly filtered as evidenced by the water level plot. The second  
330 column shows the plots for a threshold of 10 kPa on the response variable;  
331 hence, the values of the explanatory variable favor moderate and strong im-  
332 pact pressures. Consequently, the smallest wave heights are less present.  
333 The wave incidence is in average slightly shifted to the north. The lowest  
334 periods disappear as well as the low water levels. Finally, in the last column  
335 plots, restricting the dataset to wave impact pressures larger than 100 kPa,  
336 wave heights are in majority in the range 3 -6 m, wave incidence is almost  
337 restricted to  $0 -20^\circ$  with a peak in  $10^\circ$ , the period is comprised between 12  
338 and 15 s, with a few events around 17 s or lower than 10 s and higher water  
339 levels are more frequent.

340 Regarding extreme events, the significant wave height measured at the  
 341 buoy reached 7.5 m at 3 a.m. on 2016/02/09, and a swell period of 19.5 s was  
 342 measured on 2018/11/18 at 3 am. All  $H_s > 6$  m in this dataset are from the  
 343 same storm event, which occurred on 2016/02/09. In terms of water level,  
 344 the peak 5.01 m was reached at 5 am on 2019/01/23.

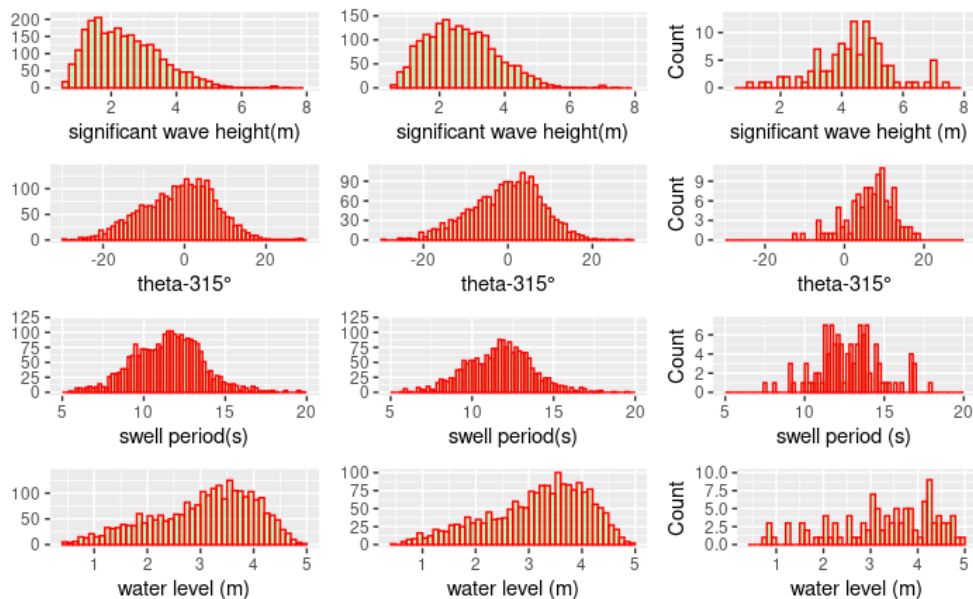


Figure 5: Histogram of the four explanatory variables ( $H_{1/3}$ ,  $T_{H_{1/3}}$ ,  $\theta_m$ ,  $wl$ ) with different thresholds for the response variable : 0, 0.1 and 1 bar.

### 345 3.1.2. Maximal pressure and pressure impulse

346 The distribution of  $P_{max}$  at sensor #0 is presented in Figure 6 (a). The  
 347 majority of impacts is below 100 kPa. Only 7 impacts are in the range 200  
 348 to 300 kPa. Among those 7 impacts, 4 happened during the same storm,  
 349 in the night of the 2016/02/09. Only one of these  $P_{max}$  was recorded at the  
 350 upper sensor #1 (see Figure 4 left), the 2016/02/09 at 4 am. The 2019/01/21  
 351 at 4 am an impact of 447 kPa was recorded at sensor #0 in moderate wave  
 352 and wind conditions (i.e.,  $H_{1/3} = 1.5$  m,  $T_{H_{1/3}} = 8.2$  s,  $ws = 4.6$  m/s,  
 353  $wl = 4.39$  m). This is our maximum value so far.

354 The vertical distribution of  $P_{max}$ , restricted to large pressure values, is  
 355 shown in Figure 7. Overall, there is a tendency of decreasing pressure values  
 356 when moving up the wall. This is true for all the components of the boxplot,



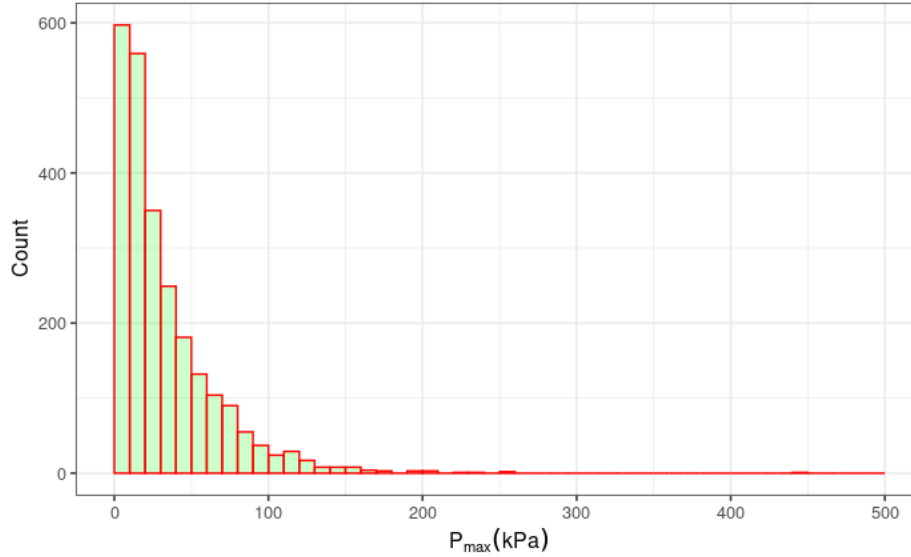


Figure 6: Histogram of  $P_{max}$  based on the observations over the whole measurement duration at sensor #0

357 namely the median value, the first and third quartiles, as well as the outliers.  
 358 As a matter of consequence, extreme pressure values, larger than 200 kPa,  
 359 only appear at the lowest sensor position.

360 Examples of pressure time signals are presented in Figure 8. The 2016/02/09  
 361 (Figure 8 (a,c,e)) was a particularly energetic storm with significant wave  
 362 heights up to 7.5 m, water level up to 4.6 m and wind speed up to 21 m/s. On  
 363 the other hand, the 2016/03/11 (Figure 8 (b,d,f)), a significant wave heights  
 364 of 2.1 m, a water-level of 4.8 m and wind speed of 1.6 m/s were recorded.  
 365 These cases illustrate two recurrent signal patterns observed in this experi-  
 366 ment. In (c), many strong impacts can be observed in groups, while in (d),  
 367 only one major impact is recorded. In the zoom windows, Figures 8 (e) and  
 368 (f), two types of impact can be distinguished. The five impacts in (e) have  
 369 a similar pressure signature, with a relatively rapid rise followed by a slower  
 370 decrease, a stabilization at a relatively high value before going back to zero.  
 371 In (f), a very sharp peak with a very rapid rise and fall is followed by a low  
 372 steady pressure value before going back to zero.

373 To compare the different impacts, the ratio of pressure peak to quasi-  
 374 steady level is used. This ratio was determined for 81 impacts of more than  
 375 100 kPa from the 2016 database. Most impacts are similar to the one pre-

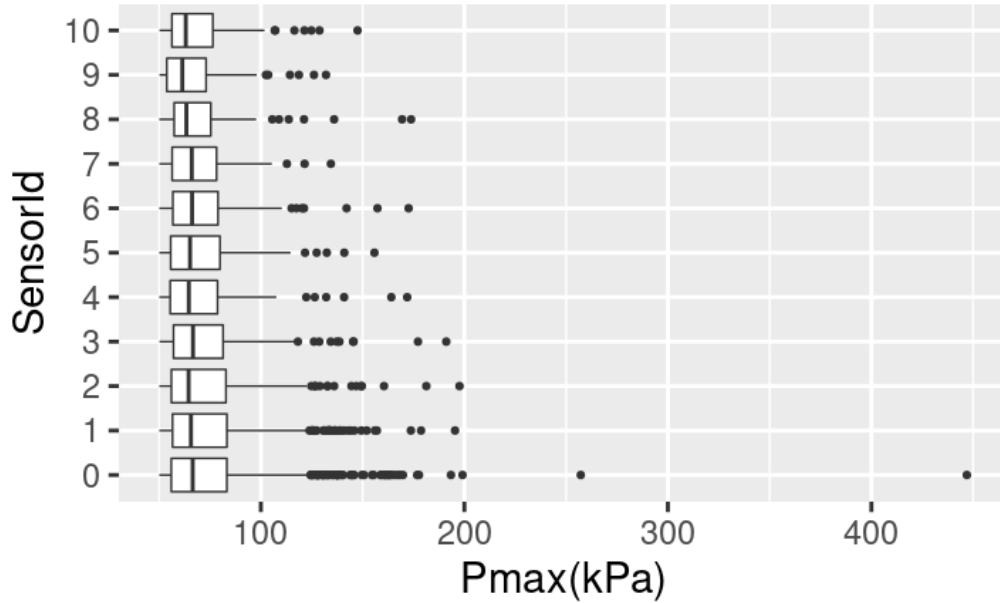


Figure 7: Vertical distribution of  $P_{max}$  represented as boxplots. Only impacts larger than 50 kPa are considered in this plot. Dataset restricted to the 2018-2019 campaign.

376 sented in Figure 8(c), and the mean ratio is 1.66 with a standard deviation  
 377 of 26%. Among these 81 impacts, 8 presented a shape similar to (d). For  
 378 these impacts, the mean ratio is 10 with a 3.5% standard deviation. They  
 379 all present oscillations following the impulsive pressure peak with a main  
 380 frequency around 15 to 20 Hz.

381 The vertical distribution of pressure impulse is shown in Figure 9. As  
 382 for the maximum pressure, a decrease tendency of the magnitude with the  
 383 sensor altitude is observed for the quartiles and the outliers.

### 384 3.1.3. Notable outliers

385 Among all the recorded impacts, the impact shown in Figure 10 is particu-  
 386 larly interesting because of its magnitude and the presence of very well-  
 387 defined, damped oscillations. This type of pressure signal is typical of a  
 388 highly aerated impact, the oscillations likely being due to the compression  
 389 and dilatation of a trapped air pocket. This impact, which occurred under  
 390 moderate wave conditions (i.e.,  $H_{1/3} = 1.5$  m,  $T_{H_{1/3}} = 8.2$  s), generated a  
 391 very large impulsive peak of  $\frac{p}{\rho_w g H_{1/3}} \approx 30$ . In the power density spectrum,

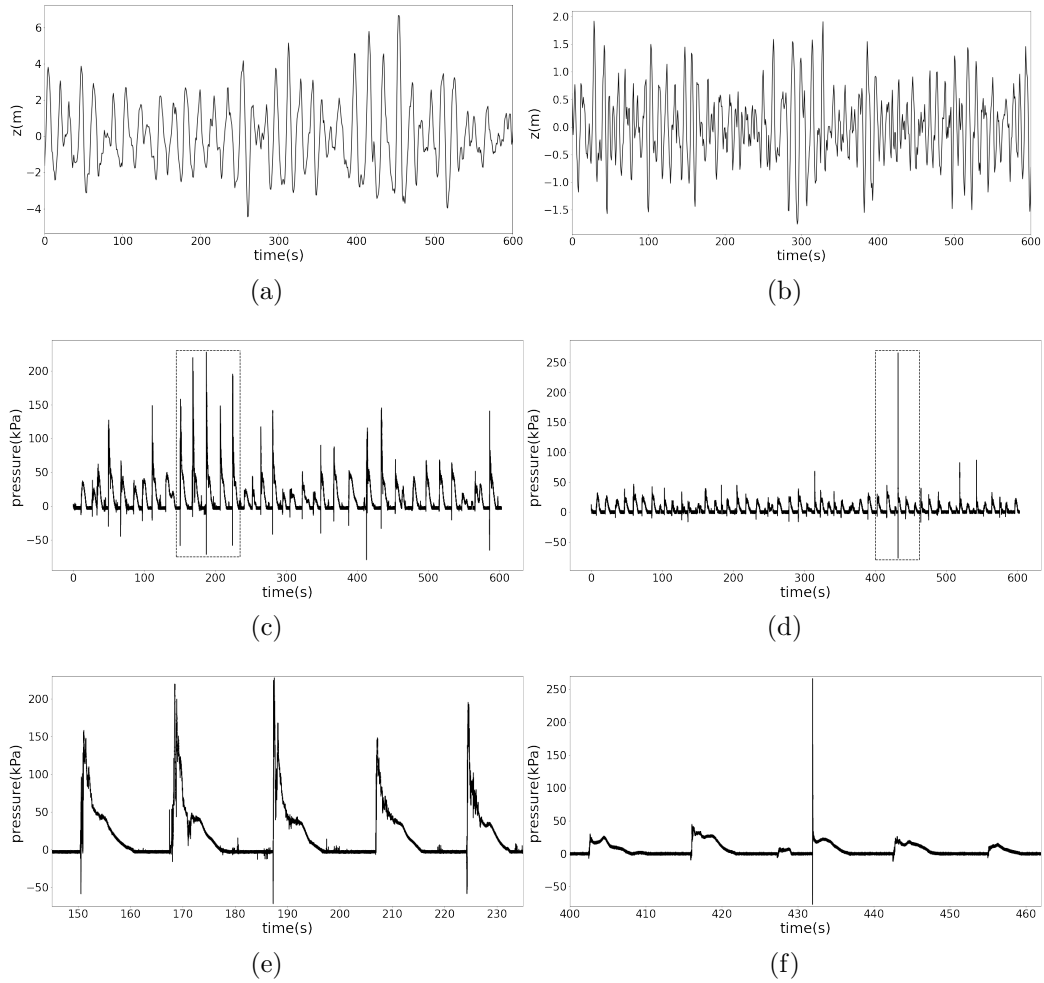


Figure 8: Examples of typical strong impact pressure time signals recorded at the Artha breakwater with the associated simultaneous free surface signal recorded at the buoy. Left column : 2016/02/09 at 4am, right column : 2016/03/11 at 5am. (a) and (b) free surface time signal recorded at the buoy location, (c) and (d) 10 minutes pressure burst at sensor #0, (e) and (f) Detailed signal within the dashed rectangles

392 shown Figure 10 lower panel, a well-defined peak at 29 Hz can be seen, so a  
 393 little more than for the other impacts also involving pressure oscillations (i.e.  
 394 between 15 and 20 Hz). In order to secure the interpretation of these oscillations,  
 395 the plate holding the sensors was tested with an impact hammer and  
 396 an accelerometer. The objective was to determine the natural frequencies of

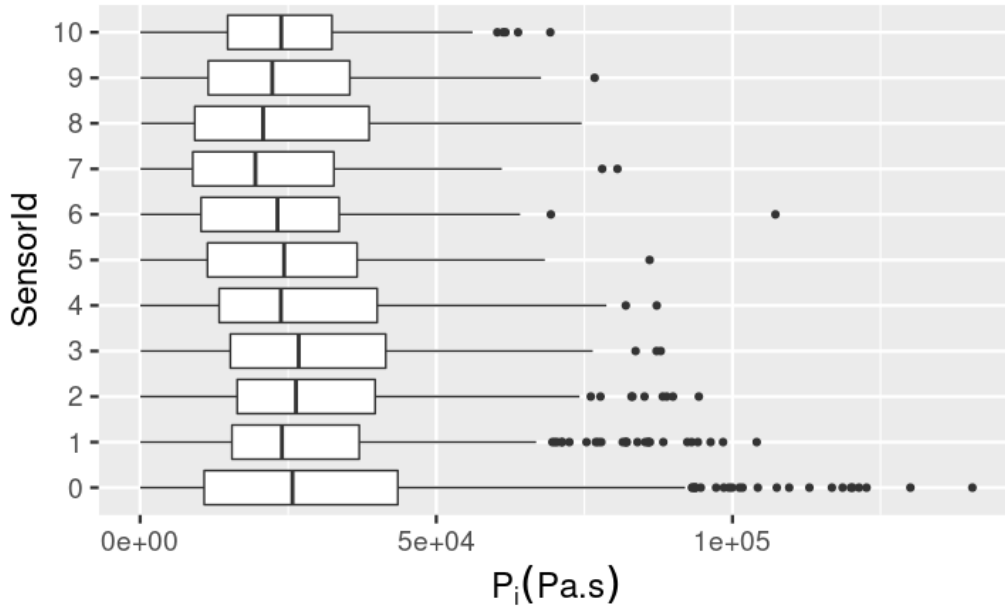


Figure 9: Vertical distribution of pressure impulse represented as boxplots. Only impacts larger than 50 kPa are considered in this plot. Dataset restricted to the 2018-2019 campaign.

397 the measuring system and compare them to the frequencies observed in these  
 398 particular pressure signals. This test gave two non-negligible resonances of  
 399 the measuring device at 470 Hz and 800 Hz, so at much higher frequencies  
 400 than the one observed in those impacts. Therefore, the vibration of the plate  
 401 is likely not responsible for the oscillation observed in the impact pressure.

#### 402 3.1.4. Pressure peak vertical celerity and rise times

403 Impact measurements in-situ meet very different conditions leading to  
 404 high variability. Regarding the pressure peak celerity along the wall, we  
 405 tried to illustrate this variability by calculating the Spearman coefficients of  
 406 the rank correlation between pressure peak dates and sensor elevations. The  
 407 histogram presented in Figure 11 (a), for impacts greater than 50 kPa, shows  
 408 the distribution of this Spearman coefficient. The perfect upward propaga-  
 409 tion is the highest bin, nevertheless, overall, the majority of the large impacts  
 410 do not follow this expected upward propagation. A few rare events with an  
 411 overall downward propagation are identified (negative Spearman value). It  
 412 is suspected that many values in this histogram are representative of broken

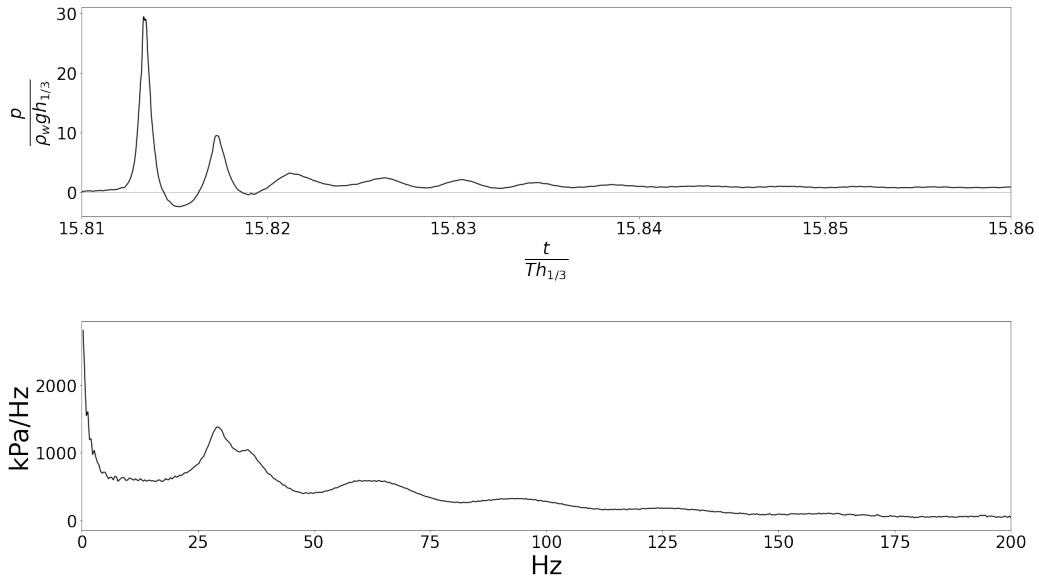


Figure 10: Top panel : dimensionless pressure history during the maximal impact of the 2019/01/21 4 am and associated power spectral density (bottom panel).

413 wave front hitting all sensors quasi-simultaneously, or random water projec-  
 414 tions from the block armor, rather than a well-defined upward propagation  
 415 of a pressure spike.

416 Following this first analysis, the database was next restricted to impacts  
 417 where the maximum index of sensors 1-5-6-8 (i.e., the most consistent of  
 418 the array) was naturally sorted. The average velocity was then determined  
 419 between sensors 1-6, since many impacts do not reach the threshold at the  
 420 top of the array. This limited the database to 389 observations (38% of  
 421 the database), most of which, involving a peak celerity smaller than 20 m/s  
 422 (Figure 11 (b)). For example, the propagation velocity for the very impulsive  
 423 impact presented (Figure 10) is  $\approx 12$  m/s.

424 In the lineage of Blackmore and Hewson [5], Bullock et al. [12], Cuomo  
 425 et al. [16], Ariyaratne et al. [2], Kisacik et al. [28], Song et al. [45], Chuang  
 426 et al. [14], the relationship between maximum pressure and rise time is shown  
 427 in Figure 12. As expected, impacts with long rise time exhibit weaker pres-  
 428 sure peaks while, on the contrary, impacts with a high value of  $P_{max}$  have a  
 429 shorter rise time. In our observations, approximately 92.7% of the impacts  
 430 are located under the contour  $P_{max} = 31000/risetime$  which was the upper  
 431 bound in [10]. A new curve,  $P_{max} = 100000/risetime$ , is introduced to better

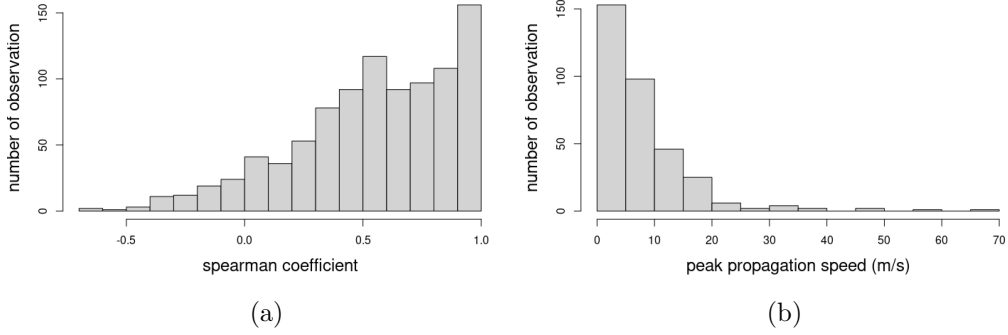


Figure 11: (a) Histogram of Spearman coefficients illustrating rank correlation between pressure peak dates and sensor elevations  
 (b) Histogram of pressure peak propagation vertical speed for impacts of more than 50 kPa calculated between sensors 1 and 6 with upward propagation only.

432 bound our data. This curve leads to a 0.1% exceedance rate. The dimensionless values are also presented in panel (b) using the same scaling as for  
 433 instance, Kisacik et al. [28]  
 434

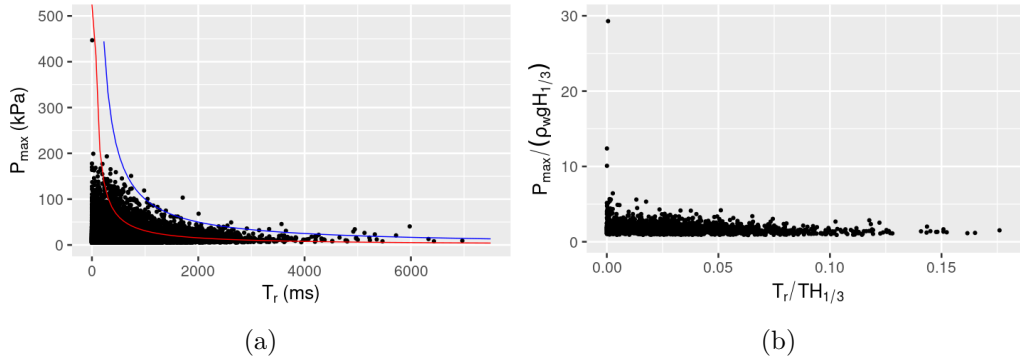


Figure 12: Relationship between maximal pressure and rise time for the 2018-2019 data.  
 (a) In kPa and ms with the non exceedance curve of Bullock et al. [10] in red and the present 0.1% exceedance curve in blue, (b) Same as (a) but scaled.

435 *3.1.5. Estimation of the force*

436 An estimation of the force is presented in Figure 13 for the maximal  
 437 impact of the 2019/ 01/23 at 06 am. As explained previously, to calculate

438 the force, the signal had to be downsampled to respect the condition given  
439 in [31], function of the pressure peak celerity. The pressure peak vertical  
440 propagation celerity is calculated as the slope of the regression line of the  
441 maximum pressure instant at each sensor in the  $t$ - $z$  space. For this impact,  
442 the celerity is estimated at 8.3 m/s. This gives a quite low sampling frequency  
443 criterion of 40 Hz.

444 During this record, the significant wave height was 4.02 m and the water  
445 level 4.7 m. For this latter value, the water depth over the berm is only  
446 about 80 cm (Figure 2 (b)). Considering that the upper part of the block  
447 armor is almost at the same level as the berm over 30 m upstream, it is  
448 very likely that this impact was generated by a broken wave or a breaking  
449 wave at the latest stage. This curve therefore illustrates the pressure signal  
450 to be expected in this type of conditions. Compared to the classification  
451 presented Figure 2.1.3, it may be seen as an impulsive wave impact or an  
452 intermediate state between the impulsive impact and the broken wave impact.  
453 In our record, if the time scale appears coherent with Oumeraci et al. [35],  
454 the transition between the peak and the quasi-hydrostatic stage is smoother.  
455 The measured force maximum value (i.e.,  $\approx 0.7$ ) should be interpreted with  
456 caution. Indeed, the pressure was only integrated over about one third of the  
457 wall. Multiplying by three this value would lead to something comparable to  
458 the values reported by Oumeraci et al. [35] for broken waves (i.e., around 2).

### 459 3.2. Statistical prediction of $P_{max}$

460 The preliminary analysis reported in section 3.1.1 already allowed to re-  
461 duce the number of predictor variables to four. The database was also filtered  
462 to broken wave impacts in order to reduce the variability of the response  
463 variable. This is achieved by only considering water levels of less than 3.5 m  
464 or significant wave heights larger than 6 m. These conditions ensure that  
465 waves break before the wall and allow to remove the few events correspond-  
466 ing to waves breaking directly on the wall, which appear as outliers in the  
467 database. The database is then split into a training sample and a test sample  
468 (respectively, 80% and 20%). The multivariable linear model fitted to this  
469 sample yields an adjusted  $R^2$  of 0.65.  $H_{1/3}$  is the most important variable in  
470 the model and accounts for 73% of the explained variance, followed by  $\theta_m$ ,  
471  $T_{H_{1/3}}$ , and water level  $w_l$ . The main interest of such a model is to provide  
472 a prediction of the maximum pressure expected for a given condition (i.e.,  
473 an estimate of the maximum value among all the burst maximum pressures  
474 recorded for similar conditions). This is achieved with the 99% prediction

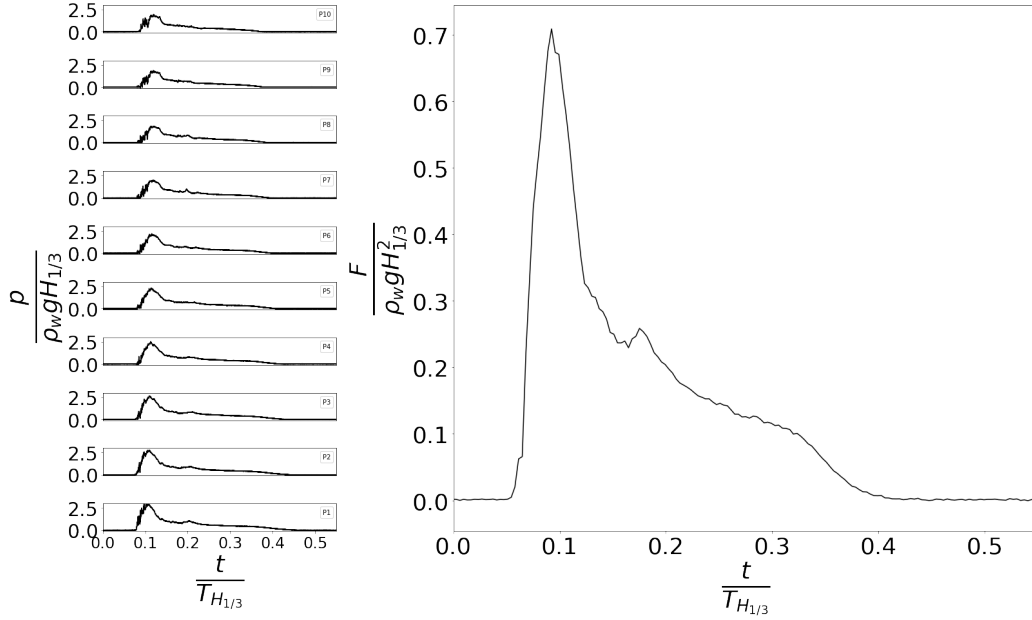


Figure 13: Pressure time history during the maximal impact of the 2019/01/23 6 am at each sensor on the vertical (left panel) and associated force signal sampled at 40 Hz following Lamberti et al. [31]

475 interval of the multiple linear regression model as illustrated in Figure 14.  
 476 This figure illustrates the performance of the model in the plane  $H_{1/3}$ - $P_{max}$ .  
 477 For this test sample, the prediction is, overall, only exceeded by 3% of the  
 478 subset observations.

479 The equation for the 99% prediction interval of the multiple linear regres-  
 480 sion model is given by :

$$P_{max_{99\%}} = \hat{P}_{max}(x_0) + t_{\alpha/2, n-p} \cdot s_e \sqrt{1 + x_0^\top (X^\top X)^{-1} x_0} \quad (1)$$

481 With  $\hat{P}_{max}$  the (mean)  $P_{max}$  predicted by the multiple linear regression,  
 482  $x_0$  the vector of the predictors,  $X$  the design matrix,  $s_e$  the residual standard  
 483 error and  $t_{\alpha/2, n-p}$  the quantile of order  $\alpha/2$  of a student distribution with  
 484  $n - p$  degree of freedom ( $n$  number of observations,  $p$  number of predictor  
 485 variables plus one).



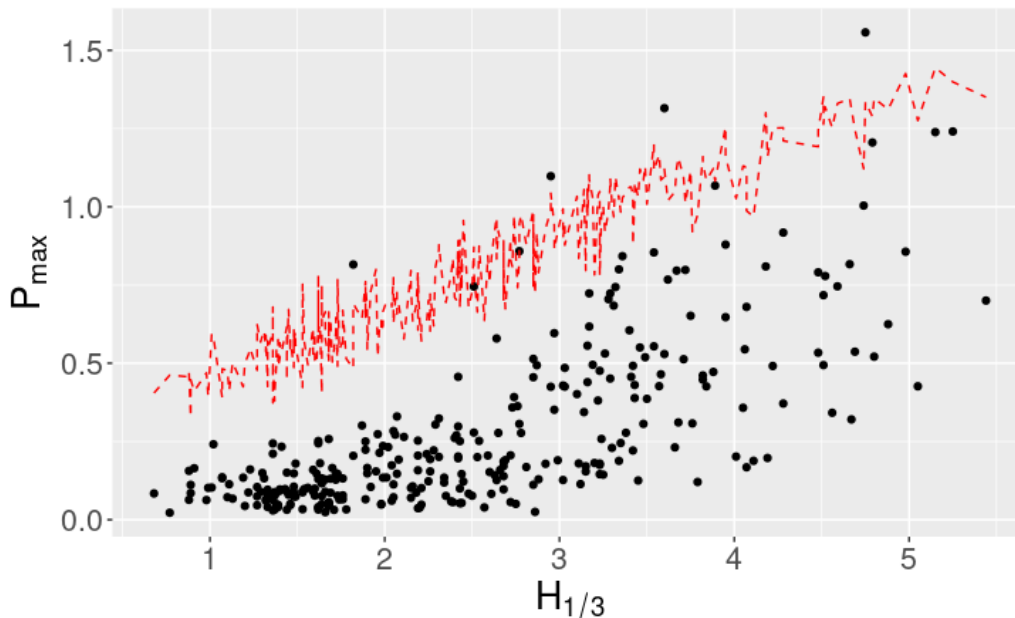


Figure 14: Observation from the test sample and 99% prediction interval of the multiple linear regression model (dashed red line) in the  $H_{1/3}$ - $P_{max}$  plane.

#### 486 4. Discussion

487 All the studies reporting in-situ measurements of waves impacts have  
 488 highlighted the difficulty associated with measurements in this harsh envi-  
 489 ronment. Continuous very high-frequency measurements over a long period  
 490 are challenging and therefore not frequent. As a result, most in-situ mea-  
 491 surements were restricted to one or two years [5]. Overall very few in-situ  
 492 data linking offshore wave conditions and impact pressure are available.

493 In the present experiment, two years of data have been collected so far but  
 494 the particularly exposed nature of the site and the efficiency of the remote,  
 495 independent station, allowing a quasi-continuous recording, has already pro-  
 496 vided a greater number of impacts than previous in-situ campaigns. Hence,  
 497 1851 10-minutes bursts at 10 kHz, reaching pressure of more than 10 kPa  
 498 over atmospheric pressure have been collected, among which 122 included  
 499 pressures of more than 100 kPa. During the 2018-2019 campaign alone, this  
 500 makes more than 400 impacts of more than 100 kPa. This unique dataset  
 501 offers the opportunity to provide wave impact statistics based on in-situ

502 measurements with a large number of observations. Moreover, these impacts  
503 were generated in wave conditions ranging from  $H_s = 0.5$  m to 7.50 m so  
504 covering a wider range than in previous studies. The close proximity of a  
505 directional wave buoy, a tide gauge, and a semaphore additionally provide  
506 a relevant set of synchronised external variables to build a statistical model  
507 that relates the maximum impact pressure to the corresponding environ-  
508 mental conditions. To our knowledge, this is the first time such a model has  
509 been proposed based on field data. It should be noted that according to the  
510 bathymetric profile upstream of the breakwater and our observations and  
511 simulations [38], most of the waves impact the structure as broken waves,  
512 so this data set focuses particularly on this type of loading, which is rarely  
513 addressed.

514 The strongest impacts ( $P_{max} \geq 200$  kPa ) have been mainly observed in  
515 two types of conditions: (i) storm events with large significant wave heights or  
516 (ii) moderate wave heights and water levels of more than 4 m. In the second  
517 case(ii), the impacts present distinctive oscillations and a high impulsive peak  
518 to steady level ratio. The two classes are consistent with the classification  
519 proposed in Oumeraci et al. [35]. This classification predicts a behavior  
520 change of the Artha breakwater depending on the water level and the wave  
521 height.

522 Most of the impacts are generated by high waves. Figures 15 (a) and (b)  
523 shows a wave impact in this kind of conditions. As illustrated by this figure,  
524 it is reasonable to consider that they generally correspond to broken waves or  
525 waves at the latest stage of breaking as previously mentioned. Additionally,  
526 broken wave impacts are also predicted by the PROVERB classification [35].

527 According to [35], the impulsive impact type is likely to appear for very  
528 large water levels and moderate wave energy. An example of the condition  
529 favoring the occurrence of this type of impact is given in Figure 15 (c) and  
530 (d). Impacts belonging to this second class are not frequent and are likely  
531 to involve waves breaking directly on the wall. This type of impact is also  
532 characterized by very localized high magnitude short pressure peaks (Figure  
533 13 (bottom left panel)) which may be the cause of the localized damages  
534 recurrently observed on the breakwater masonry. Our measurements show  
535 that these impacts are also associated with characteristic pressure oscillations  
536 which may be the signature of air entrapment [3, 12]. The observed frequen-  
537 cies around 20 – 30 Hz are lower than those observed in, for example, Hattori  
538 et al. [24] with thin and thick entrapped air pocket, but closer to what was  
539 observed by Hofland et al. [25]. In Bagnold [3], the following expression is



(a)

(b)



(c)

(d)

Figure 15: Photographs of wave impacts at the sensor location with (a) and (b) : during a storm event on December 9 2019 around 4pm.  $H_s = 5.9$  m,  $T_p = 15$  s,  $wl = 2.9$  m, courtesy of Bernard Bayle, and (c) and (d): in moderate wave condition and high water level (11/05/2018 1h45pm,  $H_s = 2.70$  m,  $T_s = 13.5$  s,  $wl = 4.213$  m).

540 proposed to estimate the frequency of oscillation of the air pocket :

$$T_n = 2\pi \sqrt{\frac{\rho_w k_w D}{\gamma p_0}} \quad (2)$$

541 With  $D$  the thickness of the pocket,  $\gamma = 1.4$  the ratio of the air specific  
542 heat at constant pressure to that at constant volume,  $k_w$  the thickness of  
543 the water mass effectively contributing to the impact and  $p_0$ , atmospheric  
544 pressure. Cuomo et al. [17] proposed the following assumptions to solve  
545 this equation in a realistic case:  $D = \pi/12 \times H_{m0}$  and  $k_w = 0.2(18\pi/12)H_{m0}$ .  
546 Under these assumptions, the estimated period of oscillation of the air pocket  
547 would be  $T_n = 6.4$  Hz for the impact of Figure 10. Therefore, the reduction in  
548 the oscillation frequency is probably mainly due to the larger scale involved  
549 in the present study.

550 As seen in Figure 8, sub-atmospheric pressure have been measured for  
551 numerous impacts, in two-part of the pressure signal. Either during the  
552 oscillation following the impulsive peak or just before the rise of pressure at  
553 the very beginning of the impact. The latter is also reported in Hattori et al.  
554 [24] who invokes the very fast upward motion of water at impact as a possible  
555 explanation.

556 Thanks to the pressure sensor array, wave impact pressure distribution is  
557 known and its statistical distribution, as well as specific impacts distribution,  
558 was studied. Overall the lower sensor is always the closest to water level and  
559 the box plots presented in Figure 6 shows an overall decreasing pressures from  
560 the bottom sensor to the top. This is in agreement with previous studies,  
561 including Goda [20], Cuomo et al. [16], showing that maximal pressure is  
562 located around the mean water level.

563 As previously indicated, the estimated force applied on the structure is  
564 consistent with the value proposed by the PROVERB classification with the  
565 scaled  $F_{hmax} \approx 2$ . The quasi-static part is weaker than the one predicted  
566 Figure 2.1.3. This might be due to an integration spatial domain of the force  
567 located significantly above the water level.

568 It is likely that large and long-lasting forces are required to cause poten-  
569 tially large displacements of structural elements. Here we show that these  
570 conditions correspond to those of a storm with large waves.

571 Figure 12 also shows that we have already recorded the largest pressure  
572 impulses in-situ reported in the literature. These events are represented by  
573 the points beyond the 31000/ $rt$  line and required the introduction of a new

574 0.1% exceedance curve defined by  $100000/rt$ . A good agreement is found  
575 with the laboratory experiments of Chuang et al. [14] and Kisacik et al. [28]  
576 using the same scaling as the latter reference.

577 For the events studied in this paper, involving mainly broken or wave be-  
578 ing breaking (i.e., in a late stage of breaking), the multiple linear model built  
579 on the data show that the pressure maximum is mostly explained by the sig-  
580 nificant wave height. The other parameters of importance are, by decreasing  
581 order : direction, wave period and water level. Note that this order depends  
582 on the model used as a non linear GAM model fitted on the same data find  
583 the water level as the second parameter, behind the wave height [38]. Local  
584 wind speed and direction appeared not statistically significant . This is new  
585 result of the present paper, the wind effect being largely ignored in previous  
586 studies. In the preliminary statistical tests, very impulsive impacts appeared  
587 as outliers and were therefore removed to focus on broken waves. These im-  
588 pulsive impacts needs more observations to be better understood and may  
589 also require higher-level models such as machine learning algorithms to reach  
590 a possible prediction. Finally, for broken waves, we show that it is possible  
591 to predict with a reasonable accuracy the maximum pressure expected given  
592 a set of wave parameters and a water level based on the equation of the up-  
593 per bound of the 99% prediction interval. Of course, the coefficients which  
594 appear in this equation are expected to depend on the particular structure  
595 studied, limiting thereby its application. Nevertheless, the fact that the max-  
596 imum pressure is predictable in function of in-situ conditions is already an  
597 encouraging result in itself.

598 Finally, there are a few limitations in this study. First, the position of  
599 the sensors, fixed by technical constraints, is a limitation in this study. The  
600 lowest sensor is always the one the closest to the mean water level and it  
601 indeed regularly detected the highest pressures which is coherent with the  
602 present knowledge about wave impact. Nevertheless, the strong gap between  
603 sensors #0 and #1 prevented from getting information on the pressure profile  
604 in this crucial area.

605 The remote nature of the installation makes it complicated to access the  
606 station in case of failures, in particular during the winter season. Some failure  
607 occurred and generated some discontinuities in the acquisition. Progressive  
608 improvements of the station are ongoing to avoid such failures. It is therefore  
609 expected that a much larger and more comprehensive dataset will be available  
610 in a near future and provide a unique opportunity to better understand this  
611 complex process in nature.

## 612 5. Conclusions

613 High-frequency wave impact pressure measurements have been carried  
614 out at the Artha breakwater in Saint Jean de Luz France in particularly  
615 energetic wave conditions ( $H_s$  up to 7.5 m). The maximal pressure over 10  
616 minutes and its dependency on off-shore wave parameters, local wind speed  
617 and direction and water level was studied. The following conclusions can be  
618 drawn from this work which mostly concerns depth-limited wave loading:

- 619 • the largest recorded pressure values are found at the sensor position the  
620 closest to the mean free surface. Overall, statistically, the maximum  
621 pressure decrease with the altitude.
- 622 • the strong impacts involve mainly two classes of pressure signals, in  
623 accordance with the PROVERB classification:
  - 624 – the first one corresponding to storm broken waves exhibits mod-  
625 erate peak pressure and relatively large quasi-hydrostatic loading.  
626 Those waves generate large forces over variable duration,
  - 627 – the second one (estimated to 10% of the large impacts), occurring  
628 during mild wave conditions and large water levels, are impul-  
629 sive impacts with very large localized peak pressure followed by  
630 characteristic oscillations and moderate quasi-hydrostatic loading
- 631 • The dataset shows the existence of impacts with simultaneous large  
632 pressure and rise time (i.e., large pressure impulse) which extends the  
633 range of the observed values already reported to 100000 Pas.
- 634 • Pressure peak propagation celerity has been evaluated and shows a  
635 wide range of values and even directions (i.e., downward). In case  
636 of upward propagation for impacts of more than 50 kPa, most values  
637 remain below 20 m/s.
- 638 • The statistical study relating the maximal pressure to the environmen-  
639 tal conditions, for broken waves, shows that the significant wave height  
640 has a dominant influence overall. The wind influence seems negligible.  
641 Finally, given a set of wave and water level conditions, the maximal  
642 pressure expected can be predicted with a good accuracy (3% error)  
643 using the multiple linear upper bound.

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