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ASSESSING THE VARIABILITY OF LONGSHORE TRANSPORT RATE COEFFICIENT ON A MIXED BEACH

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Abstract

This paper presents an analysis of field measurements to investigate the variability of empirical estimations of the longshore transport rate of a mixed beach, and is the first application of the impoundment technique to measure longshore transport rates on mixed beaches. The assessment is performed in terms of the coefficient "k" that is central to the CERC longshore transport equation. For this purpose, beach morphological changes were quantified applying an impoundment technique to act as a barrier to the sediments, while wave and tide data were also recorded during the same period. Beach profile data were analysed following two different methods. As a result, two distinct representative k coefficient values were obtained, one for the shingle profile and one for the mixed profile. Both values are in agreement with the published reduction of k for coarse grain sediments relative to sand. We also found a variation in k with wave conditions, in common with some earlier studies.

Key words: Longshore sediment transport rate, CERC Equation, mixed beach, impoundment, k coefficient

1. Introduction

In general, from an engineering perspective, beaches constitute the best natural buffer for the protection of coastlines against flooding and coastal erosion. In the U.K. mixed shingle-sand and gravel beaches are of particular significance to coastal engineering where they are important for marine aggregate extraction activities and beach nourishment (Van Wellen et al., 2000; The Crown Estate, 2015). Indeed these environments play an important role in softer approaches to coastal engineering, where viable (Tomasicchio et al., 2015). Since longshore sediment transport, (LST), is the main factor in the long-term development of drift-aligned beaches, (Rogers et al., 2010), further investigations on longshore drift should be focused upon providing guidelines for the best practice in coastal management schemes, especially for coarse sediment beaches, where there is a scarcity of suitable field data to evaluate longshore transport models, (Tomasicchio et al., 2015).

The direction and magnitude of the LST are important to evaluate their effect on the coast and assess potential beach erosion for the design of coastal protection structures or any other maritime engineering works. Thus there are some qualitative indicators that may provide evidence of existing sediment transport processes, and in consequence, some quantitative indicators that can be measured to provide estimations of the processes involved (CERC, 2002). Some evidence of the sediment transport direction are the accretion and erosion of sediments when significant structures as groynes act as a barrier for the littoral drift, geomorphologic changes observed as sediment depositions or displacement of the shoreline at headlands, inlets or beach alignment.

Therefore, one of the methods for estimating LST rates is that based on the measurements of the sediment depositions blocked by a structure normal to the shoreline and the resulting erosion at the

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adjacent side of the barrier (CERC, 2002). The LST can be quantified by assuming that the LST rate is 0 at the normal structure (CERC, 1984) as no sediment passes through the structure and below its crest. In order to estimate the LST rates and potential changes due to the presence of the structure, the beach profiles can be measured using a Global Positioning System adjacent to the structure (Van Wellen et al, 1998). Three applications of the short-term impoundment method in the field and for sandy beaches, have been found in the literature, those of Bodge (1986), Bodge et al. (1987) and Wang et al. (1999). The authors of those studies highlighted the efficiency of the technique to calculate LST rates among the existing field methods.

Looking at short term estimations of the longshore sediment transport, another quantitative method is the use of sediment tracers (e.g. Blackley, 1980; Kraus et al., 1982; Bray et al., 1996; Komar, 1998; Lee et al., 2007), sediment traps (e.g. Chadwick, 1987 and Bray et al., 1996); or optical devices such as the optical backscatter sensor, which can give estimations of the suspended sediment concentrations and turbidity.

The beach profile topographic measurements can be combined with estimations of the longshore energy wave power based on the Energy Flux method in order to estimate the transport rate. An existing equation to estimate longshore sediment transport rates is based on this method, and it has been developed specifically for coastal applications and for sandy beaches, (CERC, 2002). The Energy Flux method considers the immersed weight of the alongshore moving sediment proportional to the alongshore wave power per unit length of beach (Kamphuis et al. 1986), the proportionality coefficient relating the two parameters is known as the sediment transport coefficient, k. The value adopted for k, and consequently, the corresponding methods for its estimation, have being subject of discussion by many authors, (Bodge et al., 1987; Bodge et al, 1991; Bray et al., 1996; Stutz et al., 1999; Pilkey et al., 2002; Cooper et al., 2004), as it seems not a constant value but varies depending on the specific site conditions.

Different studies have determined that the range of variability of k values depended on whether the longshore wave energy was calculated using H_{rmsb}, root mean square wave height, or H_{sb}, significant wave height, in both situations at the breaking line. To compound the difficulties there are some cases where the type of wave height used is not specified in the literature, as reported by Martin-Grandes, (2014). The Engineering Manual (CERC, 2002) summarizes a comparison between different field data sets obtained applying different techniques for measuring LST rates for sand. Those data are related to the immersed weight transport rate (N/sec) and the longshore wave energy (N/sec) using $H_{rmsb.}$ The techniques followed consisted on measurements of sand deposition at jetties and breakwaters, sand tracers and sediment traps. While the CERC (1984) presented a k coefficient of 0.39 based on computations using $H_{sb,}$ the calibration of the Engineering Manual using the field data and $H_{rmsb.}$ presented a k coefficient of 0.92. Komar et al. (1970) documented a k coefficient of 0.77 using H_{rmsb}. Nowadays, a common assumption in coastal engineering practice is that the use of H_{rmsb} gives larger values of k than those using H_{sb}, by a factor of approximately two.

The CERC formula has been adapted by several authors to determine the transport rate coefficient k on gravel beaches, (e.g. Chadwick, 1989; Nichols et al., 1991; Bray et al., 1996; Voulgaris et al., 1999). A summary of k coefficient values for coarse sediment beaches found in the literature is presented in Martin-Grandes (2014). In general, it was proposed that a smaller value of k coefficient should be used for shingle beaches in comparison with those for sandy beaches (CERC, 2002). Thus, the use of a k coefficient for sand on shingle beaches may over-predict the longshore transport rate (Nicholls et al., 1991). Komar (1998) pointed out that the quality of the data is likely to have an effect on the correlation analysis between the k parameter and the sediment size in order to obtain the dependency trend between both.

Thus, the questions arise about whether the approach adopted by the CERC (1984) formula is realistic, or should being considered more qualitative rather than quantitative, or whether it is taking into consideration the appropriate physical parameters involved in the sediment transport processes. Therefore, the general applicability of this equation to predict LST rates for engineering management schemes is open to debate. Nevertheless, further research attempting to improve the capability of this method has been undertaken; this led to the development of new numerical formulations for the estimation of LST rates, (Bayram et al., 2007; Tomasicchio et al., 2013).

This paper presents an analysis of field measurements gathered during a short term impoundment experiment, to investigate the variability of empirical estimations of the longshore transport rate of a mixed beach composed of shingle and sand. This is performed in terms of the coefficient “k” that is central to the CERC longshore transport equation (1984).

2. Site of Study

The site of the study is the mixed beach of Milford-on-Sea, under Hordle Cliff, located in the county of Hampshire on the Southern coast of the UK. It is a managed mixed, shingle and sand, beach sited at the eastern side of Christchurch Bay, Figure 1.

The beach is subject to prevailing SSW waves; the tidal regime is semi-diurnal and mesotidal with a spring tidal range of 2m and a neap tidal range of 0.9m. The double tidal cycle in Christchurch Bay creates a marked double high water and is associated with significant tidal currents that enhance the potential for coarse sediment transport (SCOPAC, 2004).

Figure 1. Left: Location of the study site at Christchurch Bay, south UK; right: view of the beach from Hordle Cliff, near to Milford-on-Sea.

This length of coast is within an area of significant environmental value. The coastline from Barton on Sea to Hurst Spit is designated a Site of Special Scientific Interest (SSSI) and designated as a Jurassic Coast World Heritage Site; indeed, Milford-on-Sea is a Coastal Area of Outstanding Natural Beauty (AONB). In terms of coastal planning, Milford is subject to New Forest District Council and is covered by the Solent Strategic Guidance Plans and Local Authority Coastal Management Plans. In addition, the interest of this area lies on the following coastal elements: cliff eroding at the western side near Barton on Sea, managed mixed beach at Hordle Cliff and the western margin of Milford-on-Sea, and coastal defence structures comprising timber groynes and seawall between Milford-on-Sea and Hurst Spit.

The beach shows gravel size sediments above high watermark, mixed sediments in the inter-tidal beach, sand size below low water mark, a longshore sand bar exposed during spring tides and cusps formed under low energy and shore normal wave incidence.

The analysis of surface sediment samples collected at the study site showed that Milford-on-Sea presents a bimodal sediment distribution. The range of sizes, in terms of the mean grain size, varied between 33mm the coarsest and 0.3mm the finer, being a D50 of 10mm the representative mean sediment size considering the distribution by weight frequency, (Martin-Grandes, 2014).

2. *Methodology*

2.1. *Field experiment: Impoundment technique*

An empirical approach was adopted based on field measurements gathered during a short term impoundment experiment carried out at Milford-on-Sea mixed beach.

The impoundment technique consisted on the deployment of a temporary structure over approximately two months, from 24th September 2007 to 30th November 2007, Figure 2. The experiment took place between the eastern side of Hordle Cliff and the western margin of Milford-on-Sea.

A shore-normal temporary groyne, constructed of GeoTextile Bags filled with the native beach material, was constructed to function as a barrier for the sediment, and allowing the quantification of the morphological change. This is considered by several authors as a reliable and effective method to estimate longshore transport rates, (see for example, Bodge et al., 1987; Wang et al., 1999; Van Wellen et al., 2000; Tomasicchio et al., 2015). The structure was deployed for a two month period during which daily beach profile surveys at both sides of the groyne, defined in a survey grid, were conducted during one of the low tides. Wave and tide measurements, Argus Beach Monitoring System (ABMS) images, as well as beach sediment samples were acquired over the same period. The wave measurements were recorded by an acoustic wave and current profiler, (AWAC by Nortek), deployed at approximately 8m depth off the site of the study and logging every hour; and the tide elevation used in this study was measured by a tide gauge deployed at Becton Bunny logging every 15 minutes. Bathymetric surveys were also undertaken before and after the experiment, however the data were not incorporated to this analysis.

The groyne was approximately 40m length, (from survey location measurements), and it was designed and set out taking into account the spring range to ensure that the structure would cover the whole the length of the swash zone and all the shingle upper beach. The structure was built in two differentiated parts and with a pyramidal section. The upper part of the groyne was formed by three levels corresponding to three geobags; continuing the groyne construction seawards, the second part corresponds to two geobags. At any case the groyne did not reach to the bar and there was no influence of any other artificial structure over the stretch of beach considered.

Figure 2. Left and centre: groyne construction, filling the geobags and deployment of the lower section of the structure during low tide the 27th September 2007. Right: View of the temporary structure from the top of Hordle Cliff the 6th October 2007.

A straight stretch of beach of circa 300m was chosen and divided in two sections of circa 150m on each side of the groyne. This distance more than satisfies the extent recommended by the CERC (1984) that suggests a distance "on the order of two or three groyne length where this length is specified from the beach berm crest to the seaward end of the groyne.

2.2. *Beach profile analysis*

Spatial and temporal changes in LST along a coastline are inextricably linked to beach profile changes over both, the short and long term (Horikawa, 1988). For instance, measurements of beach profiles are used to estimate variability in relation to meteorological forcing and to monitor the changes of beach volume and the shoreline position (Reeve et al., 2004). The latter, "line of demarcation between the water and the exposed beach" (Komar, 1998), is variously defined depending on the data available to set a datum for the seaward extent (Farris et al., 2007).

Figure 3. Plan view of the survey grid lay out at Milford-on-Sea (aerial image courtesy of Plymouth Coastal Observatory, 2008).

A topographic survey grid was designed in order to conduct the beach profile measurements, this was set out to cover an extension of approximately 300 m alongshore, 150 m at each side of the temporary structure. The survey grid comprised 18 beach profile lines approximately 10m spacing, at each side of the structure, including the two cross-shore sections surveyed along the groyne. Points along the profile were set with a frequency of 1m, however, if any significant beach change or feature was observed in between it was also measured, Figure 3. The density of the profile lines was sufficient to provide an appropriate coverage of the beach.

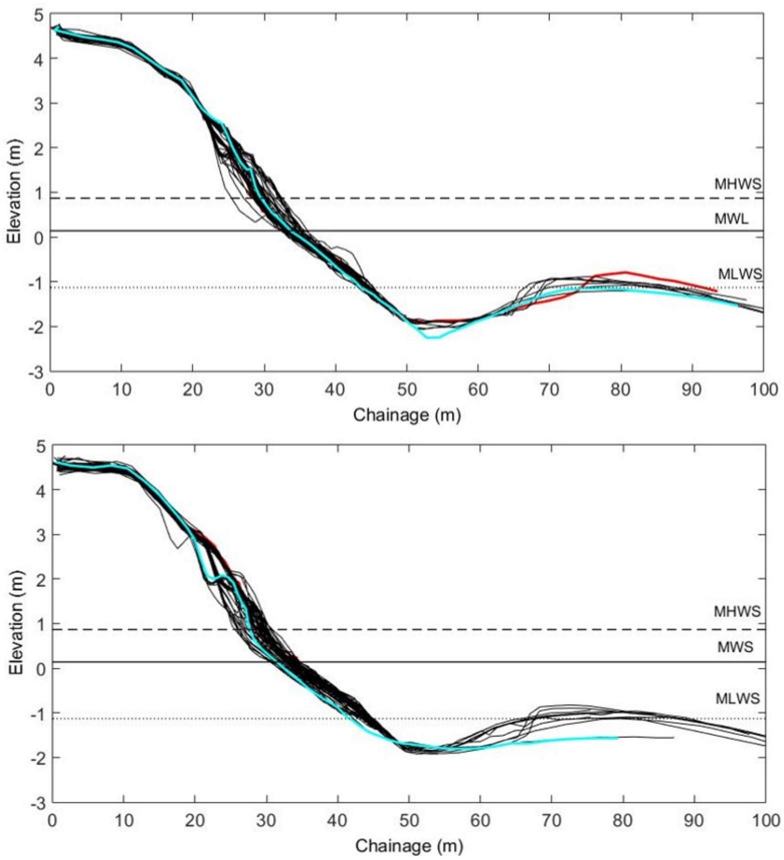

Figure 4. Top: Time series of the beach profile located updrift and approximately 10 m west from the temporary groyne (denoted as GW01); bottom: time series of the beach profile located downdrift and approximately 10 m east from the temporary groyne (denoted as GW01). In both figures the profile measured at day one corresponds to the red line, and the profile measured the final day of the experiment corresponds to the cyan line.

An example of the surveyed beach profile measurements during the experiment is presented in Figure 4**.** The profile data were used to assess the plan shape evolution of the beach. It was assumed that from changes on the beach profile it is possible obtain the changes in the plan shape due to longshore drift effects looking at particular contour elevation, i.e. the shoreline at specific water levels. In order to do that, the data were examined and analyzed considering two different methods. On one hand, only the extent of the profile for the shingle sediments was considered to understand the performance of the shingle fraction along the beach profile by defining the shingle-sand interface. On the other hand, the whole extent of the surveyed profile was considered being the seaward cut-off level the defined mean high water spring (MHWS, 0.89 m), mean water level (MWL, 0.14 m) and the mean low water spring (MLWS, -1.13 m).

Beach cusps were present for ~80% of the time of the experiment; the other 20% of the time they were not formed or they were not clearly defined. Therefore, it was assumed that cusps are common beach features at Milford-on-Sea and they were integrated in the analysis within the beach topographic surveys.

2.2.1. *Shingle profile analysis*

One of the management assumptions at Milford-on-Sea is that the coarse size sediments act as the principal mechanism for the coastal defence against the action of waves and tides. Accordingly, it is thought that the shingle fraction moves up and down along the beach face over a terrace of sand influenced by the hydrodynamic forces. Thus, there is a natural boundary established by the change in sediment grain size along the profile, i.e. the limit between the shingle fraction and the sand. In this study, this boundary is referred to as the interface shingle-sand. The position of this interface was defined and tracked using the sediment feature codes recorded during the topographic surveys.

To define the shingle beach profile data set, the beach profiles surveyed were examined individually, considering the sediment feature code and validating those with daily photographs to determine the position of the interface that in turn, was considered the seaward limit of the shingle profile. The criteria established was that the interface position as the last shingle point measured. A new data set was created containing the time series of the interface position for every profile line. Accordingly, the shingle section was also selected in the surveyed data to create the beach shingle profile data.

Once the point of the interface was defined for each surveyed profile line, the cross-shore position of that shingle- sand interface related to the beach elevation was assessed. Considering that the beach face is a function of the grain size, the degree of sediment sorting, the effect of the wave energy and the tidal cycle and stage (Komar, 1998), it was thought that the location of that interface should vary along the profile according to the variation of the water level. This assumption was confirmed by applying a bivariate analysis to the data, relating the cross-shore position of the interface to its elevation, Figure 5. Thus, by fitting a linear model to the data it is expected to provide evidence of the expected dependence between both variables and relate those to the tide cycle. The measure of the strength and the tendency of the linear relationship is described by the estimation of the Pearson's correlation coefficient.

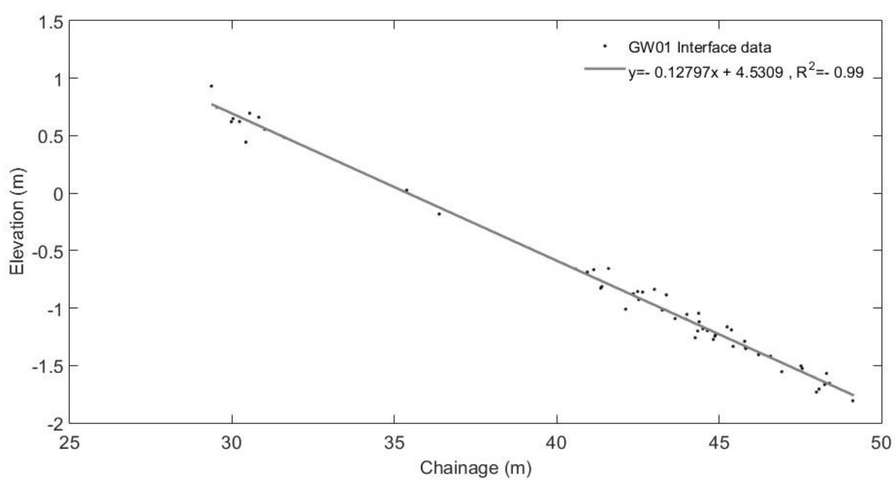

Figure 5. Example of linear relationship between the positions of the interface shingle- sand respect to the beach elevation for the profile lines located updrift approximately 10 m west to the temporary groyne (GW01).

The foreshore beach slope resulting from those linear relationships for the whole time series of the beach profiles, resulted to be within the range of 1:6 and 1:8 that are typical of mixed and gravel beaches (Van Wellen et al., 2000). The area of the profiles for the shingle fraction was calculated with respect to the profile line defined by the linear equation described above following the cut and fill method, Figure 6.

A representative daily position of the interface at each side of the structure was calculated as the alongshore average interface, resulting in one interface value per day for the west and for the east side of the groyne. Those mean values were related with the elevation of the tidal measurements in order to represent the effect of the tide into the beach profile analysis, Figure 7. The process when the tide elevation reached the elevation of the interface position was defined as 'active transport'; whereas the process when the tide did not reach the location of the interface was defined as 'no active transport'.

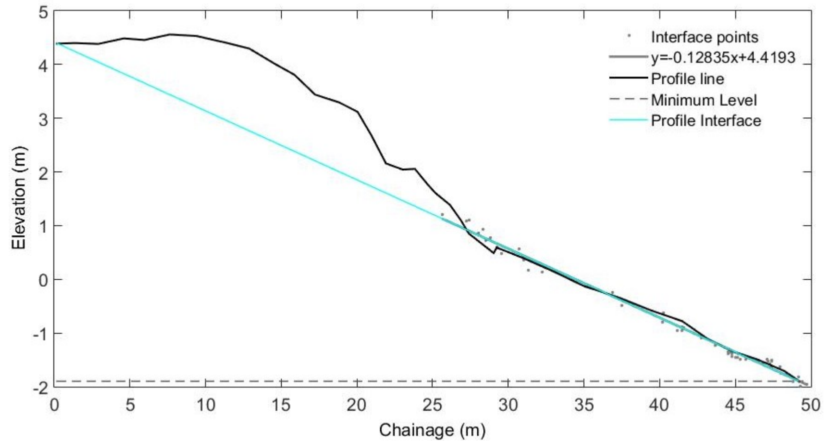

Figure 6. Example to show graphically the cut and fill method to calculate the area under the shingle profile with respect to the profile line defined by the linear equation relating the interface level respect to the beach elevation.

According to this definition it was observed that the tide did not reach the interface for some period of time during the neap tides in October.

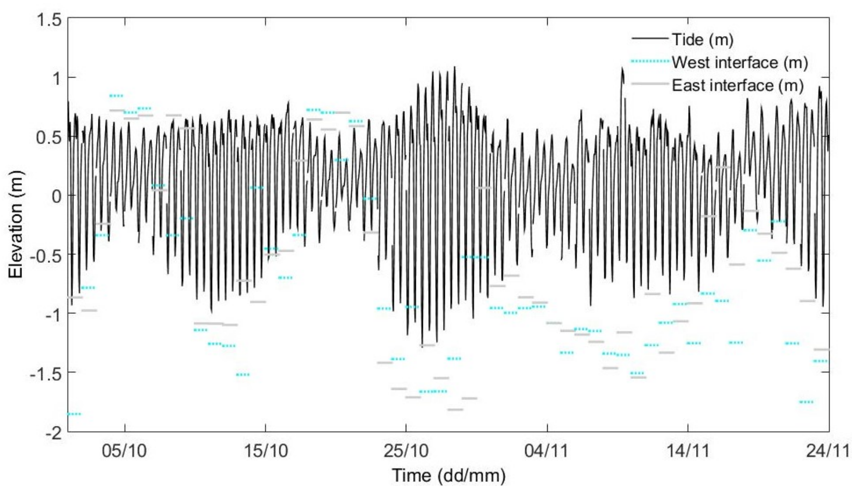

Figure 7. Tide elevation measured at Becton, and average alongshore interface elevation at each side of the groyne to identify the 'active' and 'no active' transport events.

The areas calculated for the shingle fraction following this analyses were used to estimate the correspon ding volumetric transport rates $Q_{shingle}$.

2.2.2. *Mixed shingle and sand analysis*
In order to assess the results obtained from the analysis of the shingle fraction of the beach profile as described above, a conventional beach profile analysis was also conducted for comparison. These are undertaken for conventional or traditional beach profile analysis established by standard practices and use concepts such as a master profile as baseline or the use of a known surface to close the upper and lower limits of the profile to allow comparisons to be made, (Rogers et al., 2010).

In this study, the tidal levels chosen for the analysis are the MHWS, the MWL and the MLWS as shown in Figure 4. The area under the profile was calculated respect to the surface defined by the intersection of the water level with the profile, considering the surveyed mixed profile, without any distinction between the shingle and the sand sediment fractions. Those areas were used to estimate the corresponding volumetric transport rates Q_{mixed}.

2.3. *CERC Equation*

The impoundment approach relied on the principle of mass conservation applied through the sediment continuity equation, assuming that the shore normal temporary structure functions successfully as a barrier for the sediment, Figure 8. In particular, for the shingle grain fraction, as it moves over or close to the beach surface, a barrier to the longshore transport such as a groyne is an effective method (Brampton et al., 1991). The continuity equation relates the rate at which a part of the beach considered retreats or advances to the changes in the quantity of littoral drift in the longshore direction (Komar, 1998). This method allowed the quantification of the morphological changes. The reliability of the technique lies on the mass balance as accretion on the up drift side and corresponding erosion on the down drift side, with the volumes being approximately the same if the structure works successfully, (Wang et al., 1999). The trapping efficiency of the groyne can be evaluated by the variability of the cross-shore distribution of the longshore drift influenced by the tidal elevation which is considered for the groyne design, (Brampton et al., 1991).

Thus, in this study it is assumed that the sediment balance between the deposition and loss of material in each cell is due to the longshore sediment transport under the influence of the variability of the wave conditions.

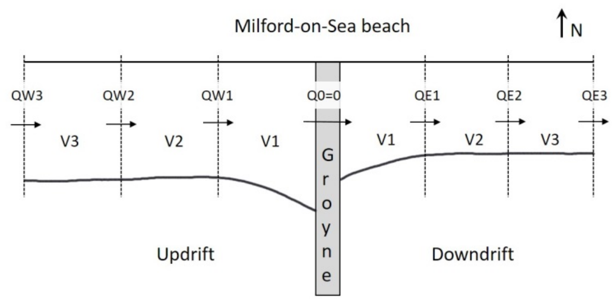

Figure 8. Schematic diagram of finite difference scheme for the volumetric survey data. The beach surveyed lines are represented by black dashed lines; notation 'QW' and 'QE' represents the longshore sediment transport rates at the west and east of the groyne respectively and 'V' are the volume.

The CERC Equation (1984), Equation 1, together with the field data, was used to calculate the longshore transport rate coefficient k. The in situ wave measurements were used to calculate the longshore wave power, P_l, using significant wave height H_b and wave direction at the breaking point. These parameters were estimated applying the known linear wave theory developed by Airy (1845), and the breaking criterion derived by Munk (1949) with the ratio at breaking (index b) $\gamma_b = H_b/h_b = 0.78$ to calculate the depth at breaking h_b (Komar, 1998). The volumetric transport rates, $Q_{shingle}$ and Q_{mixed}, derived from the beach profile measurements, were represented by the immersed longshore transport rate, I_l (N/s).

$$I_l = kP_l \tag{1}$$

The conversion of the volumetric transport rates to immersed longshore transport rates is given by Equation 2, where ρ_s and ρ are the density of the sediments (2700 Kg/m^3) and sea water (1025 Kg/m^3) respectively, g is the acceleration of gravity (9.8 m/s^2), a' is the relation of volume solids between the total volume (0.56), and Q is the empirical volumetric transport rates:

$$I_l = (\rho_s - \rho)ga'Q \tag{2}$$

The alongshore component of wave power in the breaking zone, P_l (N/s), was estimated from Equation 3, considering the wave group velocity C_{gb} to be its shallow water value. The wave angle direction is denoted by α_b:

$$P_l = \frac{1}{16} \rho g H_{sb}^2 C_{gb} \sin 2\alpha_b \tag{3}$$

Given that the wave measurements were recorded every hour, initially, the LST rate coefficients were calculated hourly for each beach volumetric cell, to be averaged over the 24 hours, resulting in a k value per day and per beach volumetric cell, (this is 54 days by 15 beach cells at each side of the groyne). The alongshore mean of those averaged k values was calculated to obtain a mean alongshore k value per day. Finally, in order to estimate a unique k coefficient at each side of the groyne as a result of the field experiment, the latter mean alongshore k values per day were averaged for the experimental period of 54 days.

3. Results

The analysis of the wave parameters allowed us to determine the wave climate conditions in the nearshore zone during the experiment period, Figure 9. The mean significant wave height was 0.44 m, being the maximum value recorded of 2.12 m the 18th November; and the mean peak period was 6.73 s with a maximum value recoded of 17.28 s the 19th October.

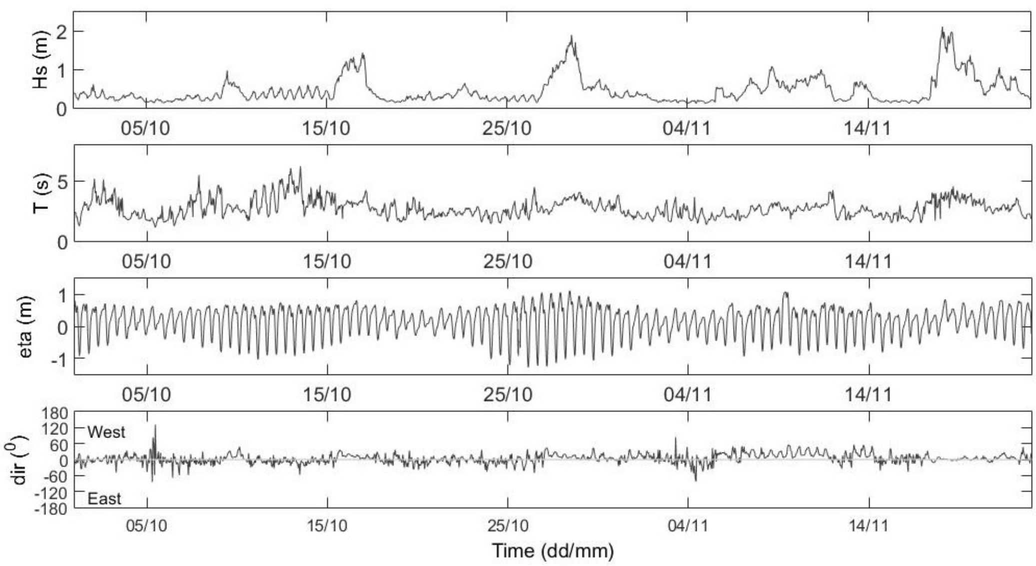

Figure 9. Significant wave height H_s, zero crossing wave period T and the wave direction, converted respect to the beach bearing, measured by the AWAC; and the tide elevation, eta, measured at Becton Bunny.

The type of breaking, given by the Iribarren Number, was calculated at Milford-on-Sea and it is characterized by plunging waves according to the criteria established by Batjes in 1974. The values obtained for the breaking index were between 0.4 and 2, characteristic of steeper beaches, and may be combined with flatter waves (Reeve et al., 2004). Only three isolated values were found just below 0.4, typical for spilling breaking waves. Bodge and Dean (1987), noted the tendency of the k coefficient to increase with the surf similarity parameter, meaning that for a given wave steepness, larger sediment sizes would present greater longshore transport rates.

Evidence of the efficiency of the impoundment technique was observed in the Argus –video system images, Figure 10, and by the representation of the beach contour maps of the surveyed profiles, Figure 11. In both figures it is observed the accretion on the updrift side of the groyne (west) respect to the downdrift side (east) where the shoreline moved landwards mainly in the area adjacent to the temporary structure. From the Argus – video images it is possible to identify the location of the longshore sandy bar indicated by the bright region of intense wave breaking.

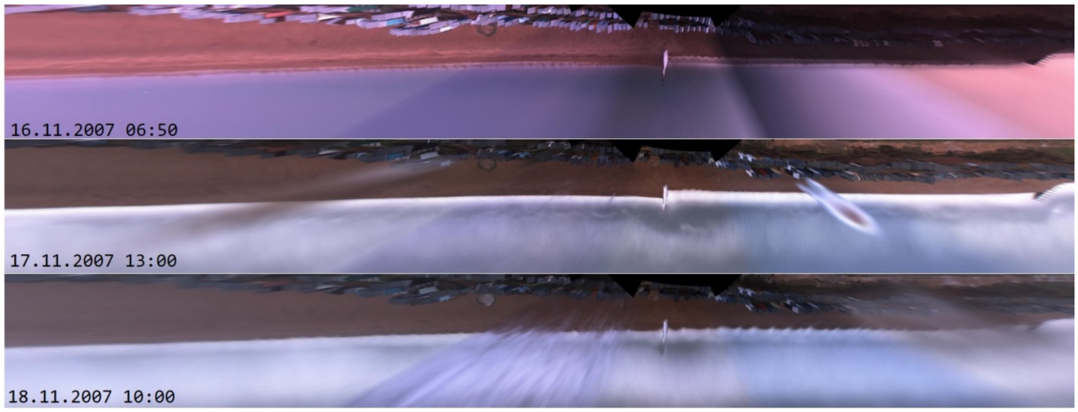

Figure 10. Plan view merged image from Argus - video system before and during the storm event during the 17th and 18th November 2007. The dates correspond to a neap tidal cycle, the low tide were approximately at 07:30, 08:30 and 09:40 respectively.

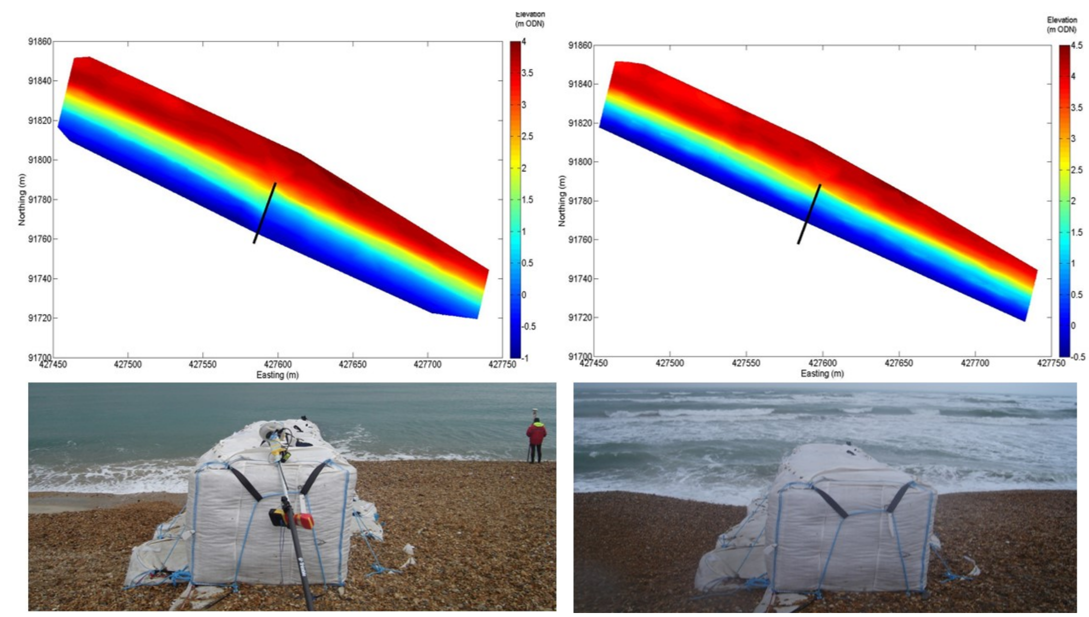

Figure 11. Left: contour maps of the shingle profiles and corresponding photograph from the temporary structure facing south the 17th November before the storm event; right: contour maps of the shingle profiles and corresponding photograph the 18th November during the storm event.

Two k coefficient values, $k_{shingle}$ and k_{mixed} were derived from this study and they are presented in Table 1. These results show two distinct values with a difference of one order of magnitude for the same wave climate conditions. This difference is also observed in the calculations of the volumetric transport rates. The coefficient $k_{shingle} = 0.22$ has same order of magnitude than those derived by Bray et al. (1996) at Shoreham Beach (southern England). For the high energy conditions event, the latter authors obtained a k value of 0.36 and 0.22 for the electronic and aluminium tracers respectively; and for the intermediate energy conditions the k values obtained were 0.30 and 0.11 for the electronic and 0.27 and 0.09 for the aluminium tracers. The k coefficient for the low energy conditions was about one order of magnitude lower than those for the higher wave energy measurements. This is the case for the coefficient $k_{mixed} = 0.052$, although it is in line with the value of k =0.054 used by Chadwick et al. (2005) at the gravel beach of Slapton Sands.

Table 1. Empirical longshore sediment transport rate coefficients k and comparison of the methodology applied.

Case	Profile Analysis	Cut-off level	k coefficient
Shingle	Interface – Active/No Active	Interface elevation	0.22
Mixed	Conventional	MWL	0.052

Nevertheless, both values are in agreement with the published reduction of k for coarse grain sediments relative to sand (Van Wellen et al., 2000); and with those results from tracer and trap experiments for shingle that vary between 0.002 and 0.36 depending on the method applied and the wave conditions.

4. Conclusions

The data measurements gathered during the field experiment demonstrated that the impoundment technique is an efficient method to evaluate morphological changes and to quantify LST rates related to specific hydrodynamic conditions.

Different values of the LST rate coefficient k were derived for a mixed beach using the CERC formula (1984) under the same wave climate conditions but applying different methods for the beach profile analysis. The k coefficient is a dimensionless constant of proportionality and it is interpreted as an indicator of the efficiency of the transport drift (CERC, 1984). Accordingly and in line with Bray et al. (1996), the k coefficient results from this study, suggest that the efficiency of the shingle transport might be higher than was assessed in previous studies which reported lower values of k, e.g. Nichols et al. (1991) and Chadwick (1989). The comparison between the beach profile analysis applied to the shingle fraction and that one for the mixed profile, suggests that the general application of conventional methods for the analysis of profiles for shingle or gravel beaches might underestimate the LST rate.

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