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Risk assessment of revetments by Monte Carlo simulation

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In risk assessment, the second-order reliability method (SORM) and the conditional expectation Monte Carlo (CEMC) simulation were inter-related in order to analyse the safety of Brean and Newton revetments on the south and north coasts of the Inner Bristol Channel, South Wales, UK. In the CEMC simulation, storm surge and wave height were generated as correlated random variables with probability distributions describing the phenomena. Then, damage probability was predicted by approximating the limit state surface by a quadratic one having the identical curvature at the design point. It was found that revetment reliability was sensitive to tidal range, storm surge and wave data. Brean revetment was found to be safe, whereas maintenance works were required during the service life of Newton revetment. The model introduced a new technique into the reliability assessment of revetments subject to large tidal ranges and its predictions satisfactorily reflected the safety/damage conditions of the structures at the site

NOTATION

a, b, c	location, shape and scale parameters of probability distributions
D_{n50}	nominal diameter of armour rock
DL	damage level in Hudson equation
FTT–1	Fisher–Tippett Type 1 probability distribution
G(Z)	limit state (performance) function
H_s	significant wave height
CEMC	conditional expectation Monte Carlo simulation
K_D	dimensionless stability coefficient
P_e	exceedance probability of a certain damage level
R_p	return period
S	damage level defined as the number of cubic stones with a side of nominal stone diameter, eroded above and below the water level within a width of one nominal stone
SORM	second-order reliability method
Z	primary variable vector in the normalised space
W_{50}	median weight of the armour unit
α	angle of structure slope
β	first-order reliability index
κ_i	positive curvatures of the concave limit state function
λ_i	direction cosines
Δ_ρ	relative density of armour unit
δ	coefficient of variation
μ	mean value
σ	standard deviation value
ϕ	standard normal probability density function
Φ	standard normal distribution function.

1. INTRODUCTION

Risk analysis techniques provide a rational basis for hazard management decision-making on a national scale, as well as regionally and locally. For example, Hall *et al.* developed a methodology for a national scale flood risk assessment applied in England and Wales,[1] and Vrijling introduced a probabilistic design methodology for the water defence system in the Netherlands.[2] Such techniques are now being adopted in coastal and river engineering projects from high-level planning based on reliability analysis to detailed designs using high-resolution simulation techniques.[3]

For the structural risk assessment of revetments, the partial coefficient system introduced by the Permanent International Association of Navigation Congresses (PIANC) has been widely utilised[4] In this system, the first-order mean value approach (FMA) has been applied in the design by using the Hudson performance (limit state) function.[5] In FMA, the limit state surface was converted into a line around the expected value. The Hasofer–Lind (HL) second-order reliability index is the commonly used level II first-order method to compare risk levels of coastal structures.[6–8] Goda and Tagaki introduced a reliability design method by using Monte Carlo simulation where an expected distance has been utilised for the sliding failure mode of caisson breakwaters.[9]

The Brean and Newton revetments selected on the south and north coasts of the Bristol Channel (Fig. 1) were designed by deterministic methods using the Hudson equation and constructed in 1997 by the Environmental Agency of the UK. In this study, the safety of these revetments is examined by the application of a new reliability method as an improved risk assessment technique developed in this paper. The technique involves the second-order reliability method (SORM) inter-related with conditional expectation Monte Carlo (CEMC) simulation. In this technique, uncertainties that affect most of the variables in the design were incorporated as risk factors throughout the lifetime of structures.

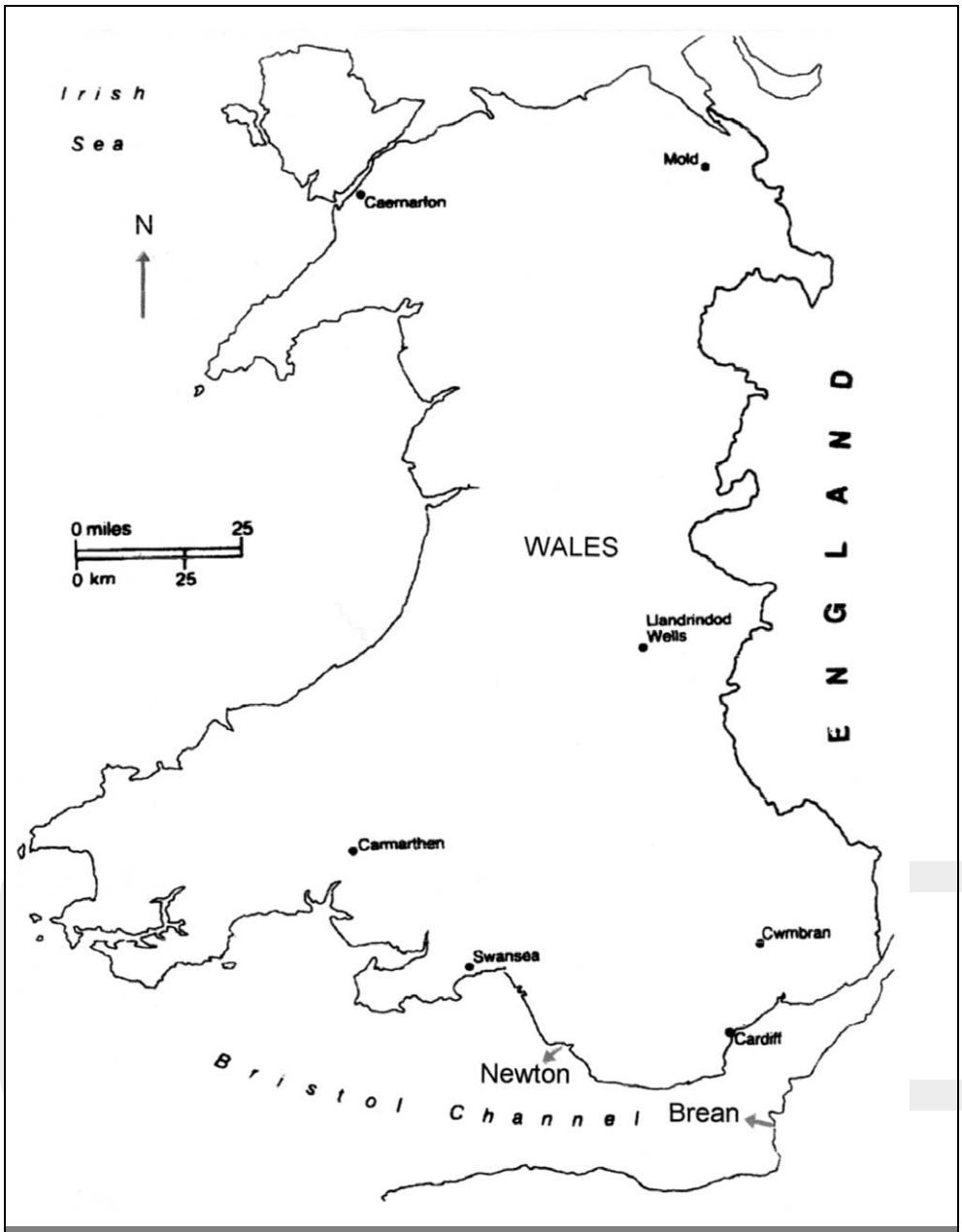


Fig. 1. Brean and Newton revetments (→ designates the location of revetments)

2. RELIABILITY–RISK ASSESSMENT

In structural safety, the limit state function consists of random load and strength variables designated by the primary variable vector Z in the normalised space. The functional form of the basic variables at the limit state is the performance function denoted by G(Z).[10] The structural safety can be assured by designating an admissible probability of achieving the limit state defined by G (Z) = 0.[11] Hence, the acceptable risk level for the design of rubble-mound structures is given in terms of exceedance probability of the limit state.[12] The serviceability (performance) limit-state was implemented for safety evaluations, since the exceeding of a specified damage level may not result in complete breakdown of the structure, but may cause an interruption in the achievement of its functions.[13] Exceedance probabilities of damage levels of rubble-mound revetments were evaluated by using the limit state equation generated from an expression obtained from the Hudson equation by taking into account the damage level[14]

1	$G = Y\Delta_\rho D_{n50}(K_D \cot\theta)^{1/3} S^k - H_s$

where Y is the variable signifying the uncertainty inherent in equation and regression; k is the power regression coefficient; H_s is the significant wave height; θ is the structure front face slope angle; S is the damage level of the structure, $S = A_e/D_{n50}{}^2$; A_e is the erosion area in a cross-section; K_D is the dimensionless stability coefficient; D_{n50} is the nominal diameter of armour rock, $D_{n50} = (M_{50}/\rho_r) - 1$; ρ_r is the mass density of armour stone; M_{50} is the 50% value of the mass distribution curve; Δ_ρ is the relative density of stone defined by $\Delta_p = (\rho_r/\rho_w) - 1$; ρ_w is the mass density of water.

The Hudson equation was rewritten for the damage-level parameter of Van der Meer by using a power regression analysis, in which the mean of uncertainty variable and the value of the regression coefficient were obtained as $\mu_y = 0{\cdot}7$

and $k = 0{\cdot}15$, respectively.[15] The (mean) damage level S in equation (1) was defined as the number of cubic stones with a side of nominal stone diameter, eroded above and below the water level within a width of one nominal stone diameter. The no-damage criterion of $S = 2$ was equivalent to DL = 2·5% damage and the damage level of S = 4 corresponded to DL = 5% damage of the Shore Protection Manual.[16]

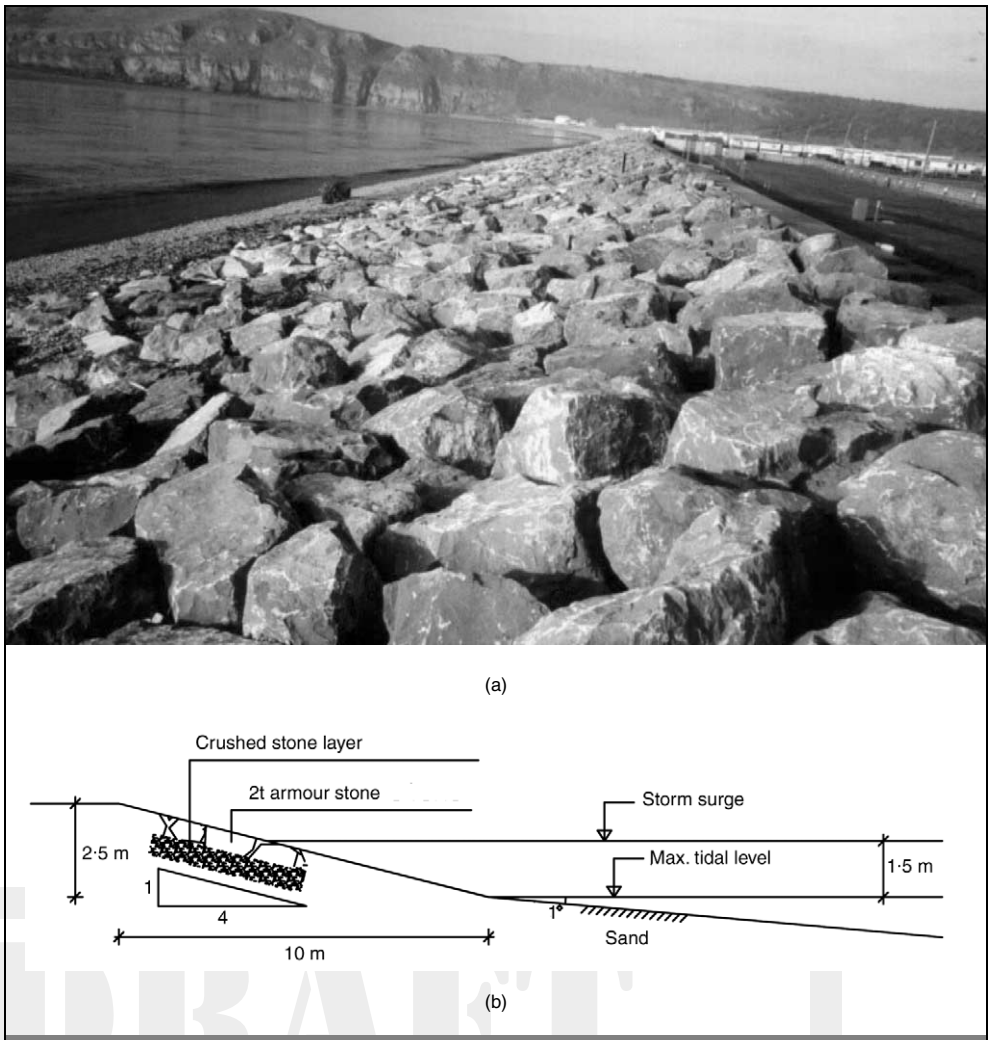


Fig. 2. Brean revetment: (a) general view (b) cross-section

A new reliability approach in which the second-order reliability method (SORM) and the conditional expectation Monte Carlo (CEMC) simulation[17] are inter-related was applied for safety evaluations. The tidal range of the site, storm surge and wave height were generated numerous times in CEMC simulation as correlated random variables with probability distributions describing the phenomena. When the generated superposed sea level exceeded the toe elevation of the structure in simulation, the reliability of the structure was investigated by SORM. The probability of exceeding the limit state was predicted in SORM by approximating the limit state surface by a quadratic one having identical curvature at the design point. Then, exceedance probability of limit state (P_e) was calculated by equation (2), as described by Zhao and Ono[18]

$$P_e \approx \frac{\varphi(-\beta)}{\prod_{i=1}^{n-1}\sqrt{1+2\frac{\varphi(\beta)\lambda_i\beta}{\Phi(-\beta)}}}\frac{1}{n-1} \times \sum_{i=1}^{n-1}\frac{\Phi\left(-\beta-\frac{(n-1)\lambda_i}{1+2\frac{\varphi(\beta)\lambda_i}{\Phi(-\beta)}}\right)}{\Phi(-\beta)}\exp\left(\frac{(n-1)\frac{\varphi(\beta)\lambda_i}{\Phi(-\beta)}}{1+2\frac{\varphi(\beta)\lambda_i}{\Phi(-\beta)}}\right) \quad (2)$$

where Φ is the standard normal distribution function, ϕ is the standard normal probability density function, β is the first-order reliability index, λ_i are the direction cosines defined as $\lambda_i = -0{\cdot}5\kappa_i$ and κ_i are the positive curvatures of the concave limit state function. For each generated load combination in the computer, the exceedance probability of the damage level was evaluated at the limit state in order to reflect the structural performance under the wave action correlated with variations in water level.

For life cycle damage estimation in the model, a computational approach was performed. In this approach, the annual damage that occurred in the revetment was stored in the computer by considering the return period of the generated significant wave height. In other words, the damage was accumulated in the computer for the simulation of damage progression in time as a function of the occurrence probability of loading in the life cycle, and the temporal variations in the mean damage was evaluated in each epoch by using the approach recommended by Melby and Kobayashi.[19]

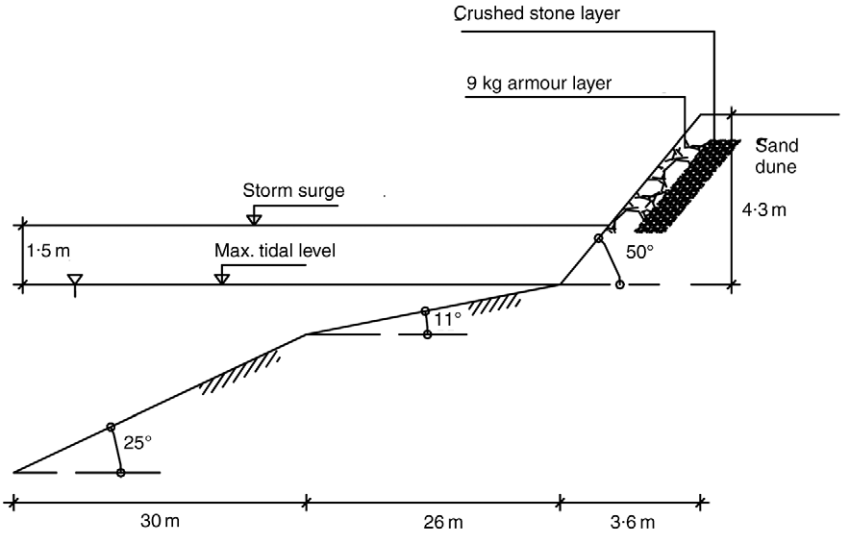
SORM and the CEMC simulation were applied together to determine the safety of Brean and Newton revetments on the south and north coasts of Inner Bristol Channel, South Wales, UK (Figs 2 and 3). Computer simulations repetitively reproduced revetment performance at the limit state condition until a specified standard mean error of convergence (ε) was satisfied.

3. RISK ASSESSMENT OF REVETMENTS

3.1. Brean revetment

The shore area in Brean is covered by up to 20 m of Holocene sediments (mainly marine and estuarine) covering Lias mudstone sediments which abut against a Carboniferous limestone headland. These sediments form the low-lying northwestern

(a)



(b)

Fig. 3. Newton revetment: (a) general view (b) cross-section

extremity of the Somerset levels. Sand dunes cover the coastal strip and can reach a height of greater than 10 m and wide (approximately 3 km) tidal flats front the dunes. Dunes are eroding usually due to sliding and toppling of blocks of sand as a result of wave action.

The UK Environment Agency has built approximately 1·2 km of rock armour frontage, protecting caravan sites/private properties/golf course/agricultural land, and adjacent to the headland, a wave return wall has been built (200 m of reinforced concrete walls plus steel sheet piling and rock armour). It is estimated that the rock revetment at Brean (Fig. 2), which was constructed in 1997, has a 200-year standard of protection against overflow with a reduced standard (50 years) against overtopping due to the constraint on defence height.[20] The revetment was designed and constructed by the UK Environment Agency by using deterministic methods in which the Hudson equation was utilised. The damage level of (0–5)% (Hudson no-damage criterion) was accepted in the deterministic design for the armour layer that is made up of randomly placed, single-unit rocks of approximately $W_{50} = 2{\cdot}0$ tonnes. The revetment has an armour layer slope of 1 : 4. The offshore wave climate of the site was predicted by Carmarthenshire County Council[21] and the design significant wave height used in deterministic design was $H_s = 2{\cdot}2$ m considering the effects of refraction, shoaling and breaking. The site has two (diurnal) tidal cycles. The spring tides have a greater vertical tidal range than neaps and produce higher tidal flow rates or currents. The tidal range is 11·2 m at Brean and flood-dominated tidal currents reach an average of 0·7 m/s. Therefore, tidal range is a dominant factor in the design with the addition of the 1 in 50 year storm surge that can increase sea level by more than 1·5 m.[21] Deterministic design values utilised during the construction stage of the revetment are listed in Table 1. The structural and loading characteristics listed in Table 1 were obtained from the shoreline and sea defence management studies of the Environment Agency.[22,23]

In the present study, the safety of the revetments is examined by the application of a new reliability method. In CEMC–SORM simulations (level III), the probability distributions of random variables (Table 2) were obtained from the statistical analysis of wind, wave, tide and storm surge data obtained by the Hydrographer of the Navy,[24] oceanographic

Design parameter	Brean revetment	Newton revetment
Nominal diameter of stone D_{n50}: m	0·91	0·15
Weight of stone W_{50}: tonnes	2	0·009
Wave height H_s : m	2·2	1·8
Tidal range R_T: m	7·1	8·1
Storm surge S_S: m	1·5	1·5
Relative density of stone Δ_ρ	1·64	1·62
Height of structure: m	2·5	4·3
Structure slope, cot θ	4	0·84

Table 1. Resistance and loading characteristics of revetments in deterministic design

Random variable (Z_i)	Probability distribution	Brean revetment	Newton revetment
Y	Beta	$a = 3$; $b = 2$; $c = 1{\cdot}5$	$a = 3$; $b = 2$; $c = 1{\cdot}5$
D_{n50}: m	Beta	$a = 1{\cdot}5$; $b = 1{\cdot}1$; $c = 1{\cdot}5$	$a = 3$; $b = 2{\cdot}8$; $c = 0{\cdot}25$
H_s: m	Fisher–Tippett Type I	$a = 1{\cdot}8$; $c = 1{\cdot}0$	$a = 1{\cdot}8$; $c = 0{\cdot}28$
Tidal range, R_T: m	Triangular	mode = 3·54	mode = 4·05
Δ_ρ	Normal	$\mu_{xi} = 1{\cdot}64$; $\sigma_{xi} = 0{\cdot}15$	$\mu_{xi} = 1{\cdot}62$; $\sigma_{xi} = 0{\cdot}18$
Cot θ	Beta	$a = 5$; $b = 2$; $c = 5$	$a = 3$; $b = 1{\cdot}8$; $c = 1{\cdot}2$

Table 2. Distribution parameters of basic variables used in risk assessment of revetments

institutions, such as: the Institute of Oceanographic Sciences[25] and Proudman Oceanographic Laboratory;[26] studies on shoreline management (e.g. Shoreline Management Partnership and HR Wallingford[27]); environmental assessment studies by the British Maritime Technology[28] and HR Wallingford;[29] wave prediction studies;[30–32] and analysis of wave data recorded by Scarweather wave-rider buoy (51°27′ N, 03°55′ W)[33] and Skomer wave buoy (51°48′20″ N, 05°20′00″ W).[34] The bathymetric data for the sites were obtained from the Hydrographical Office in Cardiff. In the CEMC simulation, storm surge and wave height were generated as correlated random variables with their probability distributions, and the exceedance probability of limit state was simulated by using equation (2) for each generated set of random values considering their correlation. Resistance variables were generated from beta distributions, since with the determination of statistical parameters the distribution could fit a wide variety of frequency shapes.

The cumulative exceedance probabilities of various damage levels at the Brean revetment are shown in Fig. 4(a). The exceedance probabilities of no-damage (0–5%) and serviceability limit-state (40–50%) levels in $L = 50$ years of lifetime were determined as $P_e = 52{\cdot}3\%$ and $P_e = 19{\cdot}4\%$, respectively (Fig. 4(a)). The SORM–CEMC simulation of the structural performance at the limit-state gave the variation in environmental loading conditions as the dispersion of performance function values. The variation of the uncertainties as a trend chart of annual probability and as a function of limit state are given in Figs 5(a) and 6(a), respectively. Simulation of the limit state was repeated for 30 000 trials performed for an average central processing unit (CPU) time of 2 min 13 s, with a standard mean error of $\varepsilon = 1\%$ by using a portable computer having an AMD K6-2$^+$ (3-D) processor. The CPU time is relatively short for the large number of simulations performed on a portable computer. This enables the efficient utilisation of the methodology in the design of coastal structures subject to large tidal variations.

The rank correlation coefficients (r_{ij}) of random variables[35] were determined from the simulated values of performance function and the averages of all cases are tabulated in Table 3. Uncertainties involved in the design, such as the reliability of wave and/or wind data, prediction model, near-shore calculation, statistical methods used, deviation between design and construction, all influenced the random generation range of correlated distributions in the simulations. The frequency distribution of the limit state function was fitted to a Weibull distribution[36] shown in Fig. 6(a). Location, shape and scale parameters of the distribution were calculated as $a = -2{\cdot}49$; $b = 3{\cdot}05$ and $c = 9{\cdot}29$, respectively (Table 4). In the simulation, the annual exceedance probability for the (0–5)% damage level was calculated as $P_e = 1{\cdot}47\%$. The scatter range of the randomly generated values was between $G_{min} = -2{\cdot}14$ m and $G_{max} = 16{\cdot}41$ m, signifying the effect of uncertainties on the limit-state having a mean value of $\mu_G = 5{\cdot}84$ m for the damage level of (0–5)% (Table 4).

The sensitivity study carried out by using the contribution to variance, showed that tidal level mainly influenced the simulation (46·7%; Fig. 7(a)). Although the reliability of revetments is generally regarded as a function of wave conditions, this sensitivity study showed that, it is mainly a function of water level due to large tidal variations. In the simulations, when the water level increased, the structure was exposed to larger waves, because the water depth governed the

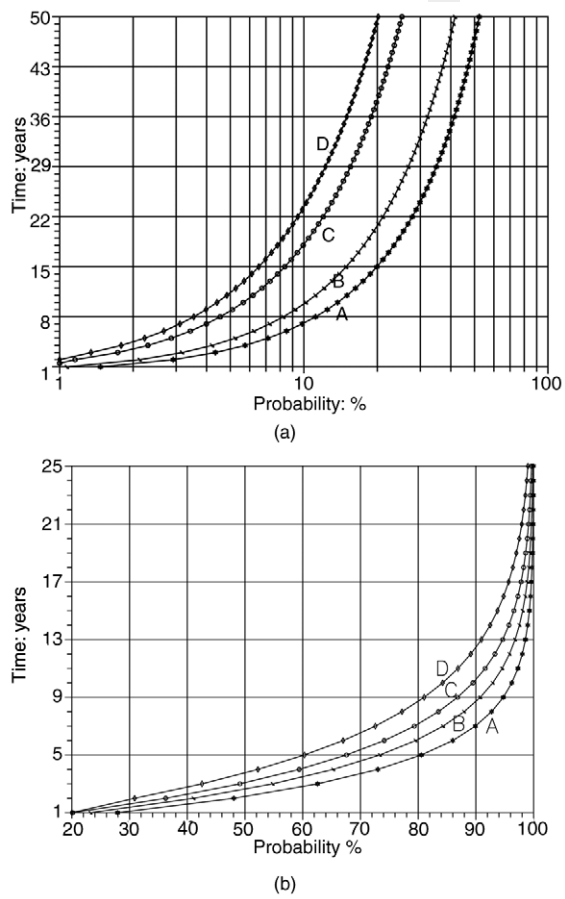


Fig. 4. Cumulative exceedance probabilities of damage levels (DL) of (a) Brean and (b) Newton revetments: A, DL = (0–5)%; B, DL = (10–15)%; C, DL = (20–30)%; D, DL = (40–50)%

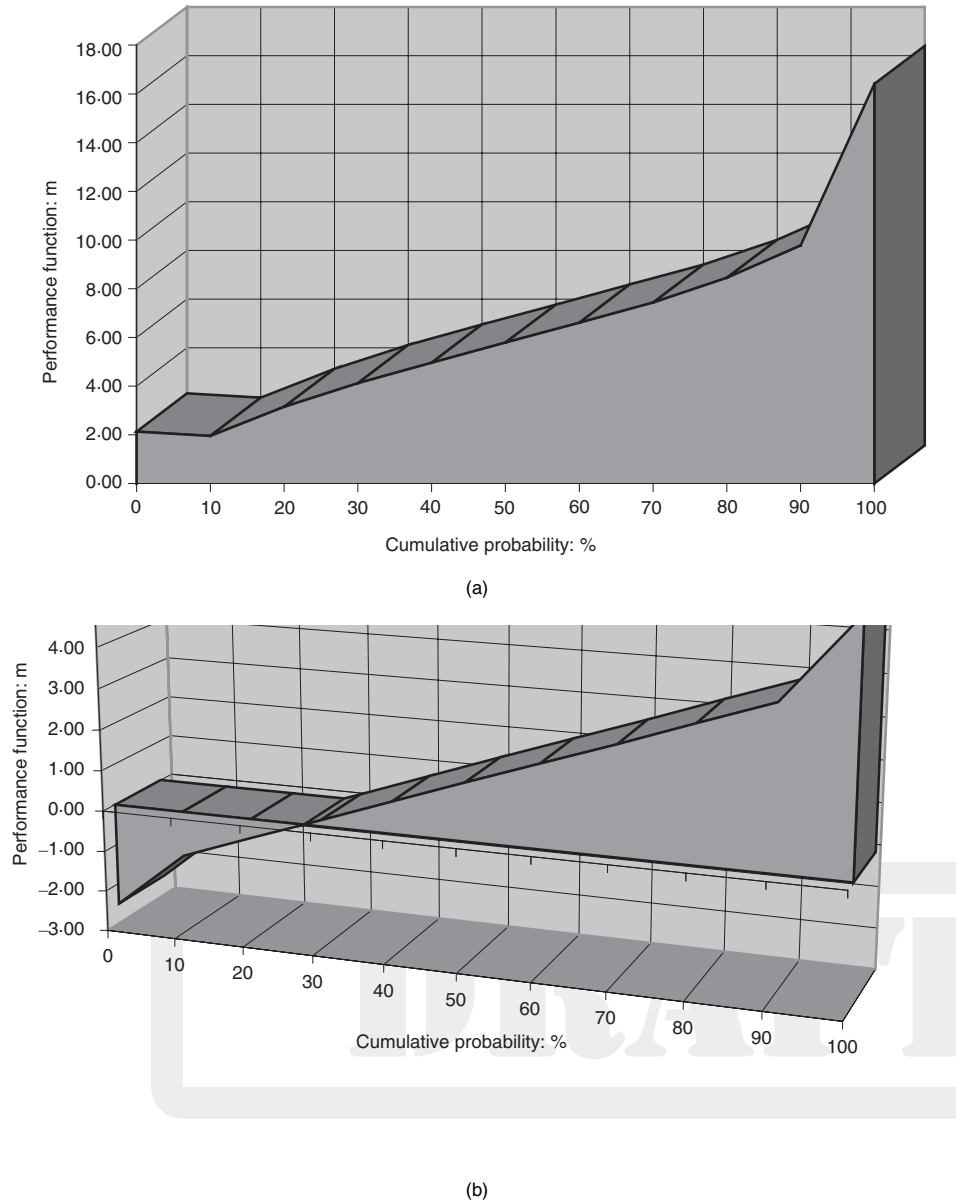


Fig. 5. Annual cumulative probability as the trend chart of reliability for (a) Brean and (b) Newton

breaking phenomena in the surf zone. On the other hand, when the water level dropped below the toe level of the structure, the structure was not exposed to waves during the simulation, as in nature. The uncertainty variable (18·2%) and wave height (15·5%) also affected the limit state function. Other variables, namely stone size (8·2%), slope (2·1%) and relative density (3·5%), had less effect on the limit-state.

3.2. Newton revetment

Newton is located on the eastern edge of Porthcawl, an important residential, coastal town in South Wales, which has a substantial tourism capacity due to its beaches, sand dunes, fishery harbour, amusement park and one of the largest caravan sites in Europe. The main shore protection structures and the harbour are located at the south-east part of the town. The harbour was constructed for the coal export trade and at present the inner harbour has been abandoned and used as a car park for tourists. The sand dune system that backs the beach is the relict of a much larger system that swept around the South Wales coastline. This system represents the eastern edge of soft sediments before outcrops of hard rock cliffs form the major portion of the Glamorgan Heritage Coast. Sand was blown into the system in the storm period, which encompassed the period between 1450 and 1660.[30] Up to the mid-1950s, much sand extraction occurred for building purposes, but this has now ceased. The system is well vegetated and stable, but *Hippophae rhamnoides* (sea buckthorn) is excessively spreading in the dunes at a rate of some 2 hectares per annum.

The major dune field of the region that extends from the east of the town to the River Ogmore is defended by three rock groins, which are spaced some 50 m apart, have a length of 30 m and a height of approximately 1·5 m above the beach level. In order to prevent flooding of the lower stretches of the village, the UK Environment Agency has designed and constructed a rubble-mound revetment that extends for some 180 m (Fig. 3). Deterministic design values utilised during the planning stage of this revetment are listed in Table 1, as obtained from the shoreline and sea defence management studies of the Environment Agency.[22,23]

The revetment in Newton encountered damage of approximately 6% during a relatively short period after construction. Therefore, the safety of the revetment was examined by the application of inter-related SORM and CEMC simulations. Probability distributions of random variables (Table 2) in the simulation were obtained from the statistical analysis of wind, wave, tide and storm surge data of the site. The dominant wave direction for the northern coast of Bristol Channel is the SW–W quadrant. A narrow band in this sector is exposed to swell waves approaching from the Atlantic Ocean having a maximum fetch distance of 6500 km. Generally; waves from the sector of 180–0 WCB have a fetch distance in excess of 100 km.[25] The extreme value significant wave height of the Fisher–Tippett Type 1 probability distribution[37] and zero crossing periods were determined for a return period of $R_p = 50$ years as $H_s = 6{\cdot}1$ m and $T_z = 8{\cdot}6$ s, irrespective of direction for an average storm duration of 12 h.[27] Several near-shore sand banks including

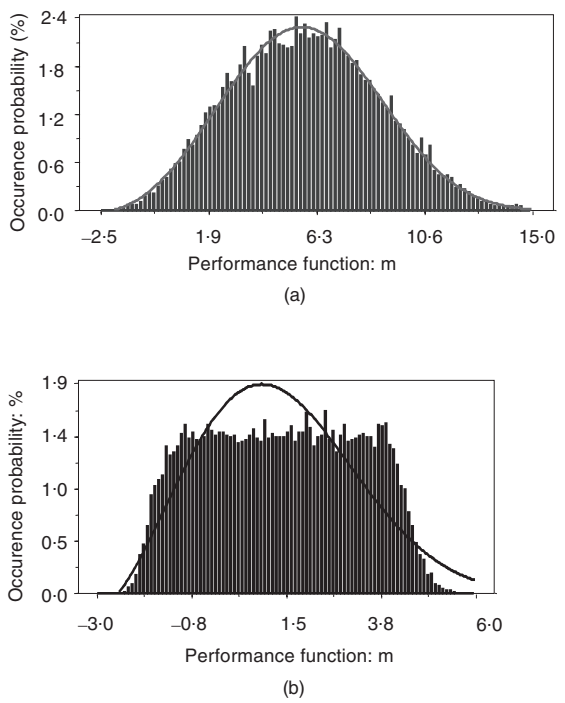


Fig. 6. Frequency simulation and the fitted theoretical distribution of Hudson performance function for: (a) Brean and (b) Newton revetments

Scarweather, Hugo and Nash provide some shelter for Newton. The expected value of significant wave height was determined as $H_s = 1{\cdot}8$ m from the wave transformation study which includes shoaling, refraction and breaking, and maximum tide level was measured at the toe elevation of the structure. The tidal flow reaches locally up to 3 m/s in front of the Porthcawl harbour breakwater where neap and spring tidal ranges are 4·6 m AOD (above ordnance datum) and 8·1 m, respectively.[24] Tidal level is enhanced by storm surges in excess of $S_s = 1{\cdot}5$ m.[29]

Utilising SORM–CEMC simulation, the structural performance of the revetment and the dispersion of the randomly evaluated limit state function were simulated. Cumulative exceedance probabilities of various damage levels at the Newton revetment are shown in Fig. 4(b). The exceedance probability of no-damage (0–5%) level in the lifetime of the structure was determined as $P_e = 99{\cdot}24\%$, signifying the requirement for maintenance works. The annual cumulative probability distribution is given in Fig. 5(b), as the trend chart of reliability. The effect of uncertainties in environmental loading conditions can be observed by the frequency distribution of the limit state function given in Fig. 6(b). The simulated values of limit state function in this figure were fitted to a triangular distribution with a mode of 0·20. Structural safety was sensitive to the environmental loading conditions, namely the variation in tidal range, storm surge and wave data. The sensitivity (77·2%) of structural reliability to tidal level is illustrated in Fig. 7(b).

4. DISCUSSIONS

The UK Environmental Agency carried out the design and construction of Brean and Newton revetments on the south and north coasts of the Inner Bristol Channel, South Wales, UK, by using a deterministic approach in which the Hudson equation was utilised. In this study, the safety of these revetments was assessed by the application of the new reliability model developed. This new risk assessment model reflected the effect of water-level changes on the reliability of revetments by simulating environmental loading conditions described by Kamphuis.[38] It was found that, the safety of revetments was sensitive to variations in these conditions, namely tidal range, storm surge and wave data. Hence, it is suggested that the reliability of revetments should be determined by using Monte Carlo simulations inter-related with the second-order reliability method (SORM), in which the limit state surface at the design point was approximated by a parabolic surface having its axis along the direction normal to the design point vector.

The revetment in Brean was considered to be safe, as the probability of exceeding the damage level of 5% was evaluated as $P_e = 52{\cdot}3\%$ from the model and the actual

Y	1					
D_{n50}	0·13	1				
H_s	0	−0·53	1			
S_S	0	0	0·76	1		
Δ_ρ	0·05	0·16	0	0	1	
Cot θ	0·07	0·19	−0·21	0	0	1
Variables	Y	D_{n50}	H_s	S_S	Δ_ρ	Cot θ

Table 3. Average rank correlation coefficient matrix (r_{ij}) of variables in performance function

Revetments	Brean	Newton
Fitted distribution of G	Weibull	Triangular
Distribution parameters	$a = -2{\cdot}5$; $b = 3{\cdot}1$; $c = 9{\cdot}3$	mode = 0·2
Average of G (μ_G): m	5·84	1·42
Variation coefficient: δ%	51	135
Annual P_e (%) of DL = (0–5)%	1·47	27·93
Minimum value of range	−2·14	−3·52
Maximum value of range	16·41	6·21

Table 4. Simulation characteristics of limit state (performance) function (G) with 30 000 trials

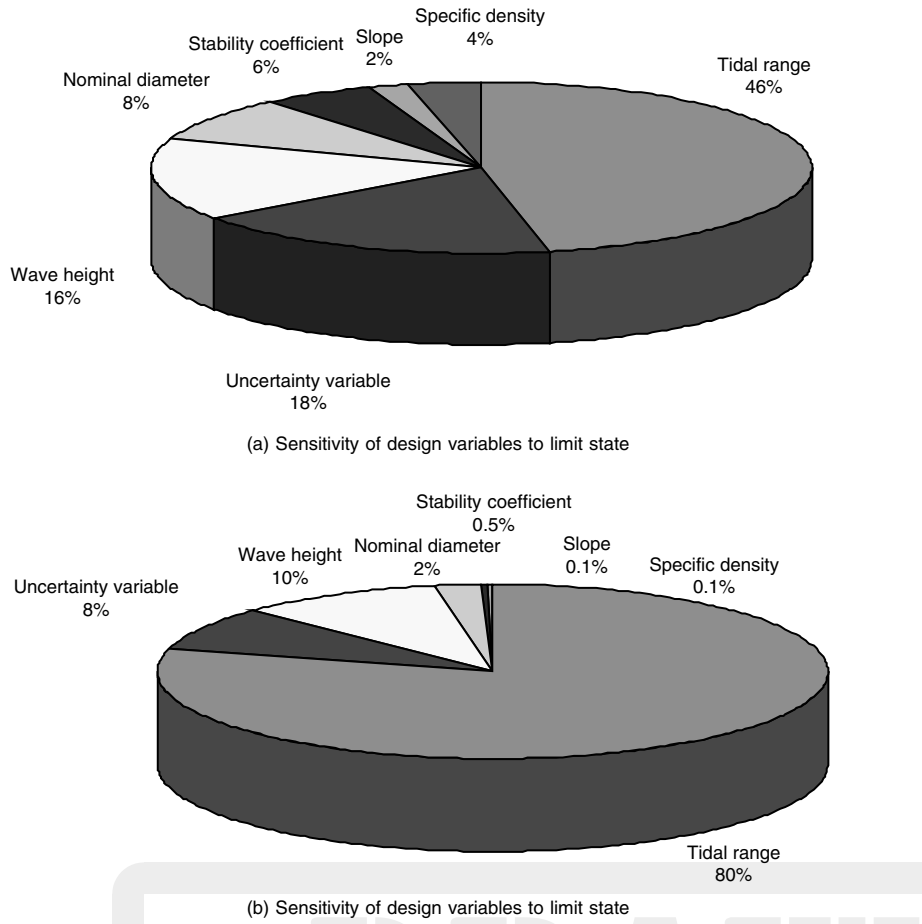


Fig. 7. The contribution (%) of design variables to the variance of limit state: (a) Brean and (b) Newton

damage observed in the revetment was less than 5% in various sections since its construction (in a period of 6 years). Hence, damage actually suffered by the revetment is in good agreement with the model predictions.[22] There was no damage encountered in the access road of the beach and the revetment showed a safe performance after its construction. The revetment in Newton required maintenance, as the exceedance probability of 5% damage in the lifetime of structure was determined as $P_e = 99{\cdot}24\%$. Notable damage of approximately 6% was already visible in the revetment and the Environmental Agency consequently carried out some repair work,[22] which supported the damage level predictions of the new reliability model introduced. Alternatives to such hard engineering structures in a Heritage Coast can be pebble beaches that abound in the area and these are probably the best protection that can be given both to nature and structures.

5. CONCLUSIONS

Reliability assessment has important advantages when compared with the conventional methods, since the random behaviour of load and resistance parameters throughout the lifetime of the structure can be estimated at the preliminary design stage.[39] This enables the designer to perform a cost-optimisation study, in which the total cost composed of initial and maintenance costs spent over the economical lifetime can be minimised, and produces an economical design associated with low damage risk.[40,41]

Structural safety is sensitive to the environmental loading conditions, namely variation in tidal range, storm surge and wave data. Therefore prior to any coastal structure design, it is recommended that the design parameters are modelled by representative probability distributions at their limit state. When the uncertainties inherent in design parameters are extensive, to obtain the safety of coastal structures it is suggested that Monte Carlo simulation inter-related with reliability methods is applied.

In this study, the safety of two revetments constructed on the south and north coasts of the Inner Bristol Channel, South Wales, UK was investigated by utilizing the risk assessment model developed, in which the CEMC simulation[42] generated the environmental loading conditions and the SORM determined the structural safety at the limit state. Brean revetment was considered to be safe, but the revetment in Newton required maintenance. Site observations for case studies showed that, the hybrid reliability model introduced in this paper satisfactorily predicted the safety levels of revetments under the effect of wave loading and water-level changes. The model application of CEMC–SORM simulations (level III), which is generally performed within a few minutes of CPU time in fast computers, has the advantage of robustness when compared with the second-order methods such as the Hasofer–Lind method, provided that the probability distribution of random variables and their correlation are described with an acceptable accuracy, which can be obtained from the data accumulated over a sufficient number of years.

6. ACKNOWLEDGMENT

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REFERENCES

1. Hall J. W., Dawson R. J., Sayers P. B., Rosu C., Chatterton J. B. and Deakin R. A methodology for national-scale flood risk assessment. *Proceedings of the Institution of Civil Engineers, Water And Maritime Engineering*, 2003, **156**, No. 3, 235–247.

2. Vrijling J. K. Probabilistic design of water defence systems in The Netherlands. *Reliability Engineering and System Safety*, 2001, **74**, No. 3, 337–344.
3. Sayers P. B., Hall J. W. and Meadowcroft I. C. Towards risk-based flood hazard management in the UK. *Proceedings of the Institution of Civil Engineers, Civil Engineering*, 2002, Special Issue, 150, 36–42.
4. Burcharth H. F. and Sorensen J. D. The PIANC safety factor system for breakwaters. In *Coastal Structures'99*, (Losada I. J. (ed.)). Balkema, Spain, 1999, pp. 1125–1144.
5. Burcharth H. F. and Sorensen J. D. Design of vertical wall caisson breakwater using partial safety factors. *Proceedings of 20th International Conference on Coastal Engineering*, Copenhagen, 1998, pp. 2138–2151.
6. Christiani E., Burcharth H. F. and Sorensen J. D. Reliability-based optimal design of vertical breakwaters modelled as a series system of failure. *Proceedings of International Conference of Coastal Engineering (ICCE)*, Orlando, Florida, 1996, **2** (124), 589–1602.
7. Oumeraci H. and Kortenhaus A. *Probabilistic Design Tools for Vertical Breakwaters (PROVERBS)*. Commission of the European Communities: MAST Days and EUROMAR Project Reports, Lisbon, 1998, pp. 1–15.
8. Burcharth H. F. Introduction of partial coefficients in the design of rubble mound breakwaters. *Proceedings of the Conference on Coastal Structures and Breakwaters*, Institution of Civil Engineers, London, 1991, pp. 43–81.
9. Goda Y. and Tagaki H. A reliability design method of caisson breakwaters with optimal wave heights. *Coastal Engineering Journal,* 2000, **42**, No. 1, 57–86.
10. Madsen H. O., Krenk S. and Lind N. C. *Methods of Structural Safety,* Prentice Hall International Series in Civil Engineering and Engineering Mechanics. Prentice Hall, New Jersey, 1986.
11. Williams A. T., Davies P., Ergin A. and Balas C. E. Coastal recession and the reliability of planned responses: Colhuw Beach, The Glamorgan Heritage Coast, Wales, UK. *Journal of Coastal Research*, 1998, Special Issue **26**, 72–79.
12. Vrijling J. K., van Hengel W. and Houben R. J. Acceptable risk as a basis for design. *Reliability Engineering and System Safety*, 1998, 59 (1): 141–150.
13. Balas C. E. and Ergin A. Reliability-Based Risk Assessment in Coastal Projects: Case Study in Turkey. *Journal of Waterway, Port, Coastal and Ocean Engineering,* 2002, **128**, No. 2, 52–61.
14. Burcharth H. F. Reliability evaluation of a structure at sea, design and reliability of coastal structures. *Proceedings of the short course on design and reliability of coastal structures, structural integrity*, 23rd International Conference on Coastal Engineering, Venice, 1992, pp. 511–545.
15. Van der Meer J. W. *Rock Slopes and Gravel Beaches under Wave Attack,* Delft Hydraulics Publications, No: 396. Delft Hydraulics Publications Delft, 1988, pp. 49–95.
16. *Shore Protection Manual*. Department of the Army, Waterways Experiment Station, Corps of Engineers, Coastal Engineering Research Centre, US Government Printing Office, Washington DC, 1984.
17. Melchers R. E. *Structural Reliability Analysis and Prediction,* 2nd edn. John Wiley and Sons Ltd, New York, 1999.
18. Zhao Y. and Ono T. A general procedure for first/second order reliability method (FORM/SORM). *Structural Safety,* 1999, **21**, 95–112.
19. Melby J. A. and Kobayashi N. Progression and variability of damage on rubble mound breakwaters. *Journal of Waterway, Port, Coastal and Ocean Engineering*, 1999, **124**, No. 6, 286–294.
20. Halcrow W. *Bridgewater Bay to Bideford Shoreline Management Plan, Volume 2—Studies and Results*. North Devon and Somerset Coastal Group, Sir William Halcrow and Partners, UK, 1998, pp. 21–28.
21. Carmarthenshire County Council. Carmarthen Bay Shoreline Management Plan. Shoreline Management Partnership, Environment Department, UK, 1999, pp.10–25.
22. Environment Agency. *Survey of Sea Defences in Wales owned by Local Authorities and Private or Public organizations,* Final Report Phases of II and III (JC148 W2SD). Environment Agency, Cardiff, Wales, UK, 1991, pp. F105.1–F105.17.
23. Environment Agency. *Shoreline Inspection Reports of Rest Bay to Irongate Point,* Final Report Vol. III. Environment Agency, Cardiff, Wales, UK, 1993, pp. 5–23.
24. Hydrographer of the Navy. *Admiralty Tide Tables*. Vol. 1 European Waters. UK, 1999, pp. 12–23.
25. Institute of Oceanographic Sciences. Wave Data and Computed Wave Climates, Report No: 5. UK, 1980, pp. 23–34.
26. Proudman Oceanographic Laboratory. *Spatial Analysis for the UK Coast*. Proudman Oceanographic Laboratory, Liverpool, UK, 1997, pp.1–122.
27. Shoreline Management Partnership and HR Wallingford. *Swansea Bay Shoreline Management Plan, Worms Head to Lavernock Point,* Stage 1, Vol. 3, 1999.
28. British Maritime Technology. *Nash bank environmental assessment*. ARC Marine Limited, United Marine Dredging Limited and British Dredging Aggregates Limited, Cardiff, UK, 1996, pp. 23–33.
29. HR Wallingford. *Dredging Application on the Nash Bank, Bristol Channel, Dispersion of dredged Material*, Report No: EX 3312. Wallingford, UK, 1995, pp. 7–17.
30. Lamb H. H. *Historic storms of the North Sea, British Isles and Northwest Europe*. Cambridge University Press, Cambridge, UK, 1991.
31. HR Wallingford. *Wave Data around the Coast of England and Wales—a Review of Instrumentally Recorded Information*, Report No: SR113, Wallingford, UK, 1987, pp. 18–26.
32. HR Wallingford. *Cardiff Barrage Design Study—Joint Probability of Extreme Sea Levels and Severe Wave Action*, Report No: EX 2175. Wallingford, UK, 1991, pp. 5–13.
33. Institute of Oceanographic Sciences. Waves Recorded at Scarweather Bank in the Bristol Channel, Report No: 79. Institute of Oceanographic Sciences, Somerset, UK, 1979, pp. 2–33.
34. British Maritime Technology. *Carmarthen Bay Coastal Study Stage III, Wind-wave Climate Data and Skomer Wave Buoy Data*. Ceemaid Limited, Carmarthen, Wales, UK, 1989, pp. 13–49.
35. Kottegoda N. T. and Rosso R. *Statistics, Probability and Reliability for Civil and Environmental Engineers*. The McGraw-Hill Companies, Singapore, 1998.
36. Pandey M. D., Van Gelder P. and Vrijling J. K. The estimation of extreme quintiles of wind velocity using L-moments in the peaks-over-threshold approach. *Structural Safety*, 2001, **23**, No. 2, 179–192.
37. De Haan L. Extreme value statistics. In *Extreme Value*

Theory and Applications, (Galambos J., Lechner J. and Simin E. (eds)) Vol. 1, 1994, pp. 93–122.
38. Kamphuis J. W. *Introduction to Coastal Engineering and Management,* Advance Series on Ocean Engineering Volume 16. World Scientific Publishing, Singapore, 2000.
39. Balas C. E. and Balas L. Risk assessment of some revetments in Southwest Wales, United Kingdom. *Journal of Waterway, Port, Coastal and Ocean Engineering, 2002*, **128**, No. 5, 216–223.
40. Balas C. E. and Ergin A. A sensitivity study for the second order reliability-based design model of rubble mound breakwaters. *Coastal Engineering Journal,* 2000, **42**, No. 1, 57–86.
41. Balas C. E. and Koç L. Risk assessment of vertical breakwaters—a case study in Turkey. *China Ocean Engineering*, 2002, **16**, No. 1, 123–134.
42. Balas C. E., Williams A. T., Simmons S. L. and Ergin A. A statistical riverine litter propagation model. *Marine Pollution Bulletin*, 2001, **42**, No. 11, 1169–1176.

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