Overtopping failure analysis of coastal flood defences affected by climate change

Mehrdad Bahari Mehrabani1, Hua-Peng Chen1* and Morris W Stevenson2
1 School of Engineering, University of Greenwich, Chatham Maritime, Kent, ME4 4TB, UK
2 Southern Testing, Keeble House, Stuart Way, East Grinstead, West Sussex RH19 4QA, UK

Abstract. Sea defence structures are expected to protect coasts for a long period, hence requiring reliable performance assessment strategies, in order to ensure their integrity and functionality. It has been demonstrated that rising sea level together with changing wave height can lead to increased risks of the failure to coastal defence structures. This paper presents a method for assessing the risk of wave overtopping failure, analysing the joint probability of sea water level and significant wave height under future hydraulic conditions due to climate change. Monte Carlo simulations are utilized to analyse the time-dependent overtopping failure probability of a seawall in the UK subjected to sea level rise. The numerical results for the flood defence example show that the seawall subjected to the sea level rise with high emission scenario could face a significant increase of the frequency and the rate of overtopping discharge in comparison with the present date conditions without consideration of seawall crest settlement.

1. Introduction
The change of future climate and performance deterioration of coastal flood defences will pose significant risks to human life, homes and infrastructure along the coastlines. Although research has been undertaken on flood protection schemes for controlling coastal flood risk in the future, the risk of overtopping over the crest of coastal defences still needs to be assessed accurately with consideration of uncertainties. From existing investigations, sea level rise and deterioration performance are considered as the major threat to coastal flood defences due to the effect of climate change. This will lead to the requirement of the time dependent reliability analysis of flood defence systems by using a probabilistic approach, especially for those existing structures which are have been designed without considering the impacts of environmental changes. Research show that environmental change leads to sea level rise [1], and a higher sea level probably induces a higher wave height. It is thus important to balance between the cost and reliability of structures.

Some studies have been carried recently in order to improve the assessment of integrity and reliability of coastal defence affected by future climate change especially sea level rise. Chini and Stansby [2] introduced a method for extreme values of coastal wave overtopping to estimate the projected failure due to the joint probability of sea level rise and wave height change. Chen and Alani [3] presented a reliability based approach to analyse and evaluate the failure probability of sea defences by

* Corresponding author, E-mail: h.chen@gre.ac.uk
considering future climate change and to provide cost-effective maintenance operation plan for the structures. Wave overtopping is still a main failure parameter due to sea level rise. Recently, UK Climate Projection 2009 (UKCP09) [4] updated hydrological sea conditions till 2095 under various scenarios. Rakonczai & Zemplénia [5] developed a joint probability method to evaluate bivariate or multivariate extreme values. The method is compatible with the nature of rise in sea level, which is useful to probabilistic approaches. Hawkes [6] introduced joint probability methods in flood and coastal defences by applying into several case studies. More studies on time dependant reliability analysis have undertaken to develop the integrity assessment methods [7-10]. Wave and water level are main factors to induce coastal flooding, lead to overtopping and damage at coastal locations. In the design of coastal structures, joint exceedance of significant wave height and still water level should be considered as the critical design factors.

This paper presents a method for overtopping failure analysis and a framework of joint probability estimation for still water level and wave height based on UKCP09 climate projection scenarios. Overtopping often occurs when extreme sea levels and high near-shore wave heights happen at the same time, however sometimes extreme wave height can lead to overtopping independently. Finally, a case study is adopted to demonstrate the proposed methodology to estimate the probability of wave overtopping failure in a coastal defence structure affected by hydrological change in environment.

2. Joint probabilistic approach
Still water level (SWL) and significant wave height (SWH) are considered as two main factors to estimate wave overtopping probability failure. These data are used to produce an estimated SWL-SWH joint probability in the period of 2001-2095 based on UKCP09 projection scenarios. Recorded recent local SWL and SWH are considered as datum. For example, data during 2001-2010 could be a useful datum to predict overtopping in the next 100 years.

2.1. Impact of sea level and near-shore wave height change
A summarised estimation for change in SWL and SWH is provided in UKCP09 second version briefing report. Sea level is projected to increase between +48 to +75 cm with 95\textsuperscript{th} percentile confidence level for various emission scenarios by 2095 [4]. Significant wave height is also estimated to have a change from -150 to +100 cm, as shown in Figure 1. Predictions of other emission scenarios for both SWL and SWH are also provided in the range with various percentile confidence levels such as 5th and 95\textsuperscript{th} in the briefing report.

![Figure 1.](image_url)

\textbf{Figure 1.} (a) Projected UK absolute sea level rise for three scenarios (low, medium and high carbon emissions) with 95\textsuperscript{th} percentile confidence interval; (b) Projected UK significant wave height annual changes for various locations from 1980–1999 to 2080–2095 in metre at the 95th percentile level.
These relative changes should be captured based on location and year (e.g. North Wales, UK, 2095, high emission scenario), and the absolute level in 1990. The local water depth $H_w(t)$ at time $(t)$ due the future sea level rise could be calculated as

$$H_w(t) = H_w(0) + \Delta H_w(t)$$  \hspace{1cm} (1)$$

where $H_w(0)$ represents initial local sea water level (1990), and $\Delta H_w(t)$ is the change in local water level from initial time. Similarly, the significant wave height $H_s(t)$ at time $(t)$ is expressed as

$$H_s(t) = H_s(0) + \Delta H_s(t)$$  \hspace{1cm} (2)$$

where $H_s(t)$ represents significant wave height at initial time (1990), $\Delta H_s(t)$ is the change in significant wave height from the initial time. Based on the extreme value analysis methods, e.g. generalised extreme value (GEV) and generalised pareto distribution (GPD), the parameters of the distributions, $\mu, \sigma$ and $\xi$, can be estimated with statistic methods provided in literatures. Chini and Stansby [2] considered changes in time on $\mu$ parameter, e.g. linear trend $\mu_t$ and initial value $\mu_0$, to evaluate new distribution parameter for the desired year, expressed as

$$\mu = \mu_0 + \mu_t t$$  \hspace{1cm} (3)$$

2.2. Overtopping analysis

For the overtopping rate of existing coastal defences subjected to sea level rise and crest settlement, the freeboard of a structure at specific time, $R_c(t)$, in the future equation can be expressed here as

$$R_c(t) = R_c(0) - \Delta h_w(t) - \sum \Delta H_d(\Delta t)$$  \hspace{1cm} (4)$$

where $R_c(0)$ is initial freeboard at present day, $\Delta h_w(t)$ is sea level rise, $\Delta H_d(\Delta t)$ is the time-dependant deterioration of dyke crest height due to settlement. Due to the decrees in freeboard, a significant increase will occur in wave run-up and wave overtopping discharge. An average overtopping discharge over time $q(t)$ is calculated from the methods in Pullen et al. [1], TAW [11], and CIRIA [12], expressed here as

$$\frac{q}{g H m^3/2} = \frac{A}{\tan \alpha^{1/2}} \gamma_f \epsilon_m^{-1,0} \exp \left( -B \frac{R_c}{H m^0 \epsilon_m^{-1,0} \gamma_f \gamma_y} \right)$$  \hspace{1cm} (5)$$

In TAW [11] overtopping is described by two formulae, one is for breaking waves ($\gamma_b \epsilon_m^{-1,0} \leq 2$) where wave overtopping increases for increasing breaker parameter, and another for non-breaking waves ($\gamma_b \epsilon_m^{-1,0} > 2$) where maximum overtopping is achieved. In a probabilistic analysis the values of A and B are taken as 0.067 and 4.75, respectively. The critical overtopping rate in Eq. (6) can be illustrated as the given line in Figure 2 for distribution analysis.

2.3. Joint probability

Joint probability estimation for various factors in coastal defences has been investigated in many studies [6, 13-16]. Plausible damage to a structure and maximum allowable overtopping rate are two main factors in performance of the structure, which can be assessed by using joint probability distributions. Hence, while a wave height is near to crest level, it is expected to have maximum damage to a structure [13].
Chini and Stansby [2] proposed the following equation in case of joint extreme value estimation for water level and significant wave height

\[
\hat{F} = 1 - F(x) - G(y) + H(x, y)
\]

where \( F \) and \( G \) are the cumulative distributions for each single variable, and \( H \) is the joint cumulative distribution based on the Gumble generator function. The marginal distributions should be determined by fitting a generalised extreme value (GEV) or a generalised pareto distribution (GPD). The cumulative and probability density functions of GPD distribution are given in [5], respectively, as

\[
F(x) = 1 - (1 + \xi x/\psi)^{-1/\xi}
\]

\[
f(x; \sigma, \xi) = \frac{1}{\sigma |\xi|} (1 + s \frac{x}{\sigma})^{-(1+\xi)/\xi}
\]

where \( \psi > 0 \) and \( \xi \) are scale and shape parameters. For \( \xi > 0 \) GPD is type I, if \( \xi = 0 \) it is the exponential distribution, and for \( \xi < 0 \) it becomes GPD type II, \( s = \text{sign} (\xi) \). There are several methods to estimate the distribution parameters such as moment based method, maximum likelihood, and likelihood-moment estimation in Jonathan, et al., [14] and Armagan, et al., [1].

2.4. Probabilistic analysis

From the sea dyke overtopping probability analysis with consideration of sea level rise discussed above, the summation of the probability density of all combinations of load and resistant can be computed [6] as

\[
\text{Pr}_o(t) = \int_{x<0} \int_{z<0} f(H_W(t), H_s(t)) dH_W dH_s
\]

where \( \text{Pr}_o(t) \) is dyke overtopping probability, and \( f(H_W(t), H_s(t)) \) is the joint GPD of two variables by assuming that they are conditionally dependent. Under the condition of statistical independence, the probability value of the exceedance sub-domain can be calculated as

\[
\text{Pr}(H_{\text{exceedance}}) = \text{Pr}(H_W > t_W, H_s > t_s) = (1 - p_w) (1 - p_s)
\]

where \( t_w \) and \( t_s \) are the threshold points on the two probability distributions, in this study for water level and significant wave height, respectively.
3. Numerical Example

A numerical example for a sea wall situated in North Wales is used in this paper for analysing the overtopping failure of a structure due to climate change. The data applied in numerical analysis are as follow: crest level = 14.5 m @Charted Datum, seaside slope = 1 : 4.5 m, roughness factor for the structure = 1 and crest width = 2 m. Joint data are generated based on case studies in [6]. In this paper, North Wales’s dataset in the UK including sea water level and significant wave height within 14 years period is chosen as indicated in Figure 3. In this study, mean wave period $T_m$ is used as an average value. The influence of sea level rise and significant wave height change in the calculations is based on the UK climate projection briefing report by [4,17], where the high emission scenario at 95th percentile interval confidence is considered.

**Figure 3.** Original data for North Wales, wave height and water level

Based on the data shown in Figure 1, the projected sea level rise of +0.76 m is estimated by the end of 21st century, and the significant wave height increases approximately +1.00 m at the same time in North Wales. From these data, Monte Carlo simulations are utilised to give the estimation for the cumulative probability and joint probability of extreme SWH and SWL. In order to estimate the joint extreme value, cumulative probability distributions fitted by GEV distribution are plotted in Figure 4, for sea water level denoted as $F(x)$ and for wave height denoted as $G(x)$, respectively. The fitted GPD cumulative distributions for the same input are shown in Figure 5. It is found that the results by using GPD distribution may be lower than those from the GEV distributions. For instance, sea level for 100 years joint return period in GPD and GEV fitted data are 13.68 m ODN and 14.12 m ODN.

**Figure 4.** Cumulative probability plots for significant wave height and sea water level, respectively, by considering high emission scenario, fitted by Generalised Extreme Distribution

The frequency analysis is often used to evaluate the hydraulic parameter values for various return periods. The return period here refers to the average period of time between occurrences of a particular high value of the variable. Recurrence interval plots for sea water level and wave height are estimated for 2, 5, 10, 25, 50, 100 and 200 years by considering sea level rise, as shown in Figure 6. By
assuming independency between sea water level and wave height, the joint probability counter lines can be estimated from the values of return period. For example, for 50 years return period values the joint return periods will be 15.3 m ODN and 6.9 m for sea water level and wave height, respectively.

\[ \text{Figure 5. Cumulative probability plots for significant wave height and sea water level, respectively, by considering high emission scenario, fitted by Generalised Pareto Distribution} \]

Sea water level and wave height are usually not independent. By applying dependency based on Eq. (6) and by using cumulative values, the counter lines of return periods of 5, 10, 50 and 100 years are given in Figure 7, by utilising GEV cumulative probability approach. Here, return period curves were estimated by 'counting back' through the ranked data in descending order above individual thresholds.

\[ \text{Figure 6. Recurrence interval plots by considering high emission scenario sea level} \]

The counter lines in Figure 7 provides data for extreme conditions for the combination of wave height and sea water level. For example, the exceedance probability in 50 years return period for sea level
and significant wave height are the same for case (14.5 m ODN, 6.8 m) and case (15.2 m ODN, 2.0 m). The linear trend $\mu_t$ is not significant for SWH at 2095 [2]. The dependency of water level and significant wave height is considered at good level ($0.12 < \rho < 0.36$). The return period estimated from joint return period shown in Figure 7 is lower than single return periods given in Figure 6.

Figure 8 shows the histograms of wave overtopping rates from Monte Carlo simulations. The rate and frequency of wave overtopping for the given structure are calculated over the period of almost 100 years from 2001 to 2095 in two cases, i.e. without and with consideration of Sea Level Rise (SLR), plotted in Figure 8(a) and Figure 8(b), respectively. In the case of SLR, the changes of sea water level and significant wave height are added to the original data yearly between +0.01 m to +0.76 m for sea water level and between +0.01 m to +1.00 m for significant wave height from 2002 to 2095. The results indicate environmental change has severe effects on the probability of failure in overtopping. Hence, the crest of the sea wall should be increased accordingly.

![Histograms of overtopping rate](image)

**Figure 8.** Histogram of overtopping rate (m$^3$/s/m) between 2001 and 2095: (a) without considering Sea Level Rise (SLR), (b) with considering Sea Level Rise (SLR)

### 4. Conclusion

This study presents an approach for assessing overtopping failure probability for coastal defences by considering joint probability failure affected by sea level rise. Different approaches of numerical calculations are adopted in this study, i.e. GPD and GEV distributions, in order to provide a better understanding of probability failure analysis. A method by using joint probability distribution model has been proposed to analyse the hydrological sea conditions to evaluate failure risk in the future. This method can be more accurate and flexible to allow a more specific investigation of various structural parameters such as overtopping, by considering joint of hydrological elements and their changes over the time.

From the numerical studies, rising of sea level and increasing significant wave height are important factors which generate significantly higher risks of the current coastal defences around the UK over the next 80 years. The frequency and the rate of overtopping discharge will be increased almost 3 times, without consideration of seawall crest settlement. Environmental change, particularly the increases in sea water level and significant wave height, has substantial effects on the future performance of the coastal defences. As a result, the frequency and rate of overtopping discharge will be escalated dramatically for the existing flood defence structures. Potential research for next steps includes investigation on the effect depth-limited significant wave height in the future climate change and the influence of performance deterioration on the structural reliability of coastal defences by using limit state analysis.
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