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# Time-dependent reliability analysis of coastal defences subjected to changing environments

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**Abstract.** This paper presents a method for assessing the risk of wave run-up and overtopping of existing coastal defences and for analysing the probability of failure of the structures under future hydraulic conditions. The recent UK climate projections are employed in the investigations of the influence of changing environments on the long-term performance of sea defences. In order to reduce the risk of wave run-up and overtopping caused by rising sea level and to maintain the present-day allowances for wave run-up height and overtopping discharge, the future necessary increase in crest level of existing structures is investigated. Various critical failure mechanisms are considered for reliability analysis, i.e., erosion of crest by wave overtopping, failure of seaside revetment, and internal erosions within earth sea dykes. The time-dependent reliability of sea dykes is analysed to give probability of failure with time. The results for an example earth dyke section show that the necessary increase in crest level is approximately double of sea level rise to maintain the current allowances. The probability of failure for various failure modes of the earth dyke has a significant increase with time under future hydraulic conditions.

**Keywords:** sea defences; lifetime performance assessment; climate change; time-dependent reliability

## 1. Introduction

Coastal structures, such as earth sea dykes, are essential components of sea defences against flooding and coastline erosion. It is of paramount importance that these structures are maintained at a high level of safety and serviceability. However, many earth sea dykes are currently under threat due to global warming induced sea level rises and increased sea storminess, which results in increased magnitude and frequency of hydrodynamic actions, larger overtopping flows, increased stresses within the structures and reduced resistance of the structure. Earth sea dykes can fail in different modes which, in general, can be summarized into two categories: the attainment of serviceability limit state, such as wave overtopping, and the attainment of ultimate limit state, such as erosion and piping. The identified failure modes are then used for time-dependent reliability analysis of existing sea defences. As a result of reliability analysis, weak spots in sea defences can be detected and the most dominant failure mechanism that may lead to system failure can be identified (Chen and Alani 2012a). Risk management measures based on rational decision-making therefore can be effectively proposed and implemented (DCLG 2006).

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Many investigations have been undertaken during last decade regarding the impact of climate change on the reliability and safety of existing coastal defences. Hall *et al*. (2006) introduced a method for coastal flood risk analysis to estimate the total expected annual damage due to climate change in England and Wales. Hawkes *et al.* (2003) presented practice guidance for appropriate use of climate change scenarios on the basis of the UKCIP (2002). The results indicated in the given example that climate change can have a dramatic impact on probability of failure: standard of service reducing by a factor of 50-100 after 80 years of projected climate change. Flood flows or overtopping rates, which are presently regarded as causing failure, could become several times more frequent after 80 years of projected climate change. Sutherland and Goulby (2003) investigated the vulnerability of coastal defence structures to wave and water level conditions that will be altered by climate change in the future. Wave overtopping was taken as an indicator of the vulnerability of the coastal defences and used for estimating the increase in crest elevation at five sites around the UK. CIRIA (2007) gave a guideline for design and safety assessment to account for the effect of sea level rise on flood defence structures. Recently, there is a growing interest in the use of reliability methods in structural condition assessment (Chen and Xiao 2015, Tee *et al.* 2013, Ghodoosi *et al.* 2014, Chen and Nepal 2015) and risk analysis of sea defences (Chen 2012, Chen 2014). A series of research projects related to integrated flood risk analysis and management methodologies have been undertaken under the European Community's Sixth Framework Programme (Van Gelder 2008, Buijs *et al*. 2006, Allsop *et al*. 2007). Whilst considerable research has been undertaken on the influence of climate change on wave overtopping, little research has been carried out within the framework of time-dependent reliability methods. Moreover, most predictions of sea dyke failures, even though probabilistically based, tend to be based on a single failure mode analysis.

This paper presents an approach for assessing the risk of wave run-up and overtopping of existing coastal defences and analysing the time-dependent probability of failure of the structures under future hydraulic conditions. The necessary increase in crest level of the existing coastal defence structure is given to counter rising sea level where the present-day allowances for wave run-up height and/or overtopping discharge are maintained. The effect of local water depth at the seaside slope toe on the necessary increase in crest level is studied. Four critical failure mechanisms occurring at various locations of the earth dykes are considered for reliability analysis, i.e., erosion of crest and landside slope by overtopping, failure of revetment on seaside slope during wave run-down, piping in the underlain water conductive layer and uplifting of impermeable layer at the landside slope toe. The time-dependent reliability analysis is undertaken for each of these failure modes due to change of hydraulic conditions associated with rising sea level. Overall failure probability is given based on the calculated results from individual failure modes. Finally, a worked example of an earth dyke section is employed to demonstrate the applicability of the proposed approach for estimating necessary increase in crest level and analysing failure probability of the structure due to sea level rise in the future.

## 2. Influence of sea level rise

Relative sea level change is the most useful measure for evaluating the impact of sea level change on risk of coastal flood as well as reliability of sea defences, such as an earth sea dyke shown in Fig. 1. Mean sea level rise with a rate of 1~2 mm per year over the last century has been reported on the basis of long-term measurements (CIRIA 2007). Recent research shows that global sea level will continue to rise, depending on greenhouse gas emissions and climate system sensitivity.

In the recently released UK Climate Projections Briefing report UKCP09 (Jenkins *et al*. 2009), projections of sea level rise for various regions in the UK due to climate change over the 21st century are recommended. The range with 5th to 95th percentile confidence intervals of absolute sea level rise around the UK before consideration of vertical land movements is projected to be between 12 and 76 cm for the period 1990–2095, depending on greenhouse gas emissions, i.e. high emissions, medium emissions and low emissions. The absolute sea level changes and vertical land movements are then combined into estimate of relative sea level rise. The relative sea level rise predicted by in London region is shown in Fig. 2.

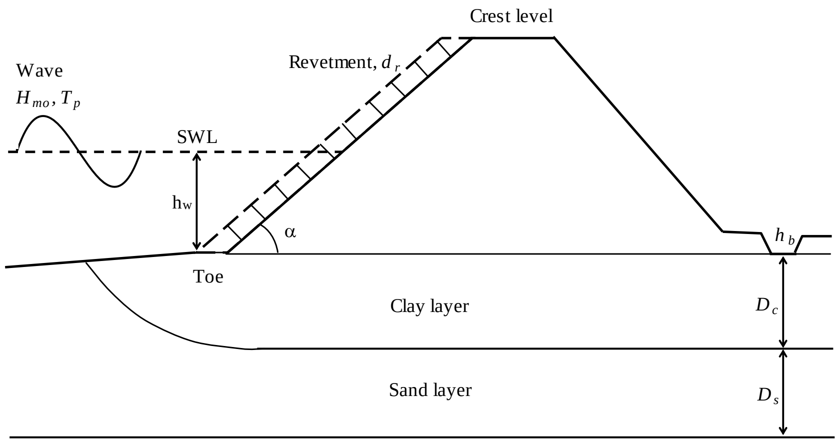


Fig. 1 Example of earth sea dyke section

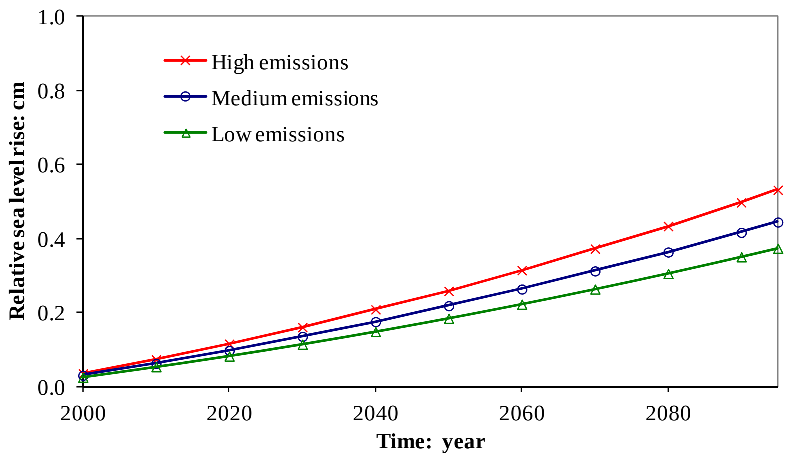


Fig. 2 Relative sea level rise since 1990 in London region (after UKCP09)

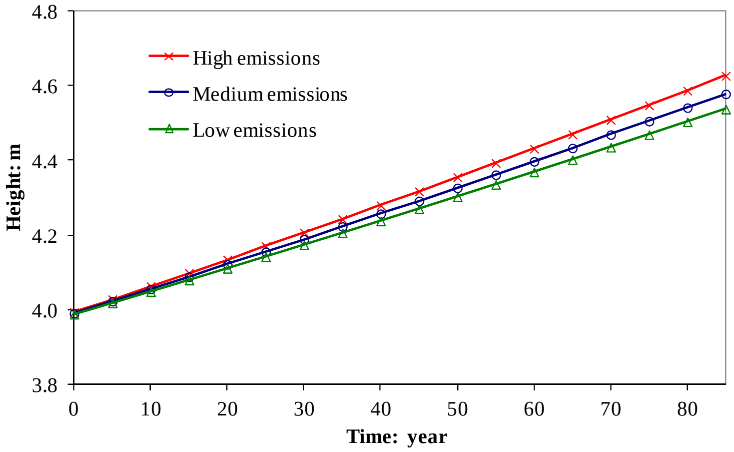


Fig. 3 Local depth-limited significant wave height due to sea level rise under various emissions scenarios

It is also reported that the projected long-term future trends in storm surge are physically small everywhere around the UK with an increase of less than 9 cm by year 2010, and in many places can be accounted for by natural variability. Changes in the winter mean wave height are projected to be between –35 and +5 cm, and changes in the annual maxima are projected to be between –1.5 and +1 m.

Sea level rise will increase the wave attack on sea defence structures, if the wave heights are depth-limited. When waves reach a relatively shallow foreshore at the toe of seaside slope, they may break due to the limited depth. The depth-limited spectral significant wave height $H_{mo}(t)$ depends on many factors, including the depth of still water level (SWL) at the structure toe. The local water depth $h_w(t)$ increases with time due to sea level rise $\Delta h_w(t)$, expressed as

$$h_w(t) = h_w(0) + \Delta h_w(t) \tag{1}$$

where $h_w(0)$ represents local water depth at present day (in year 2010 in this study) and is associated with the extreme water level with a certain return period. It is assumed here that the foreshore will not alter as a response to rising sea level and the same structure is utilised for present and future studies. Also, the crest level of existing sea defences keeps unchanged during their service life. The depth-limited significant wave height $H_{mo}(t)$ then can be estimated from the energy decay numerical model for uniform foreshore slope (Van der Meer 1990).

In this model, the prediction of local significant wave height is a function of deep-water wave steepness $s_{op}$, local relative depth $h_w(t)/L_{op}$ and foreshore slope. The deep-water wave steepness is defined as deep-water significant wave depth $H_{so}$ over peak wave length $L_{op}$

$$s_{op} = \frac{H_{so}}{L_{op}} \tag{2}$$

where the peak wave length $L_{op}$ is related to peak wave period $T_p$, defined as

$$L_{op} = \frac{gT_p^2}{2\pi} \tag{3}$$

The predicted results for depth-limited significant wave height associated with sea level rise with time are shown in Fig. 3, where wave parameters are taken as $H_{so} = 5.3m$, $T_p = 9.8s$, $h_w(0) = 4.3m$ and foreshore slope of 1:20. Results show that the local significant wave height increases about 14% in 85 years from year 2010 for the medium emissions scenario, and the rate of increase in local significant wave height is close to the rate of sea level rise.

## 3. Necessary increase in crest level

The influence of sea level rise with time is to increase the future local significant wave height $H_{mo}(t)$ at the structure toe and to reduce the future freeboard $R_c(t)$ of the existing structure. The increase in $H_{mo}(t)$ and decrease in $R_c(t)$ will lead to the increase in wave overtopping discharge as well as the increase in wave run-up height. In order to maintain the present-day allowances for wave run-up height and overtopping discharge, the crest elevation of the existing sea defence structures needs to be raised to match rising sea level.

### *3.1 Increase in crest level by wave run-up*

Wave run-up is defined vertically relative to the still water level, and the 2% wave run-up height $R_{u2\%}$ is sometimes used to design for the heights of sea defences. For a sloped sea defence, the relative wave run-up height $R_{u2\%}/H_{mo}$ is related to local breaker parameter $\xi_{m-1,o}$, expressed as (CIRIA 2007)

$$\frac{R_{u2\%}}{H_{mo}} = A_1 \gamma_b \gamma_f \gamma_\beta \xi_{m-1,o} \tag{4}$$

with a maximum for large values of $\xi_{m-1,o}$ of

$$\frac{R_{u2\%}}{H_{mo}} = \gamma_f \gamma_\beta \left( A_2 - \frac{A_3}{\sqrt{\xi_{m-1,o}}} \right) \tag{5}$$

where $A_1 = 1.75$, $A_2 = 4.30$ and $A_3 = 1.60$ are empirical coefficients for deterministic calculations, coefficients $\gamma_b$, $\gamma_f$, $\gamma_\beta$ are influence factors to account for the effects of berm, slope roughness and oblique wave attack, respectively, and local breaker parameter $\xi_{m-1,o}$ is related to the slope steepness $\tan\alpha$ and the mean wave steepness $s_{m-1,o}$, defined as

$$\xi_{m-1,o} = \frac{\tan\alpha}{\sqrt{s_{m-1,o}}} \tag{6}$$

where mean wave steepness $s_{m-1,o}$ can be calculated from Eqs. (2) and (3), in which local significant wave height $H_{mo}(t)$ and mean energy wave period $T_{m-1,o}$ should be adopted to replace $H_{so}$ and $T_p$, respectively. In order to counter sea level rise and maintain the present-day allowance for wave run-up height, the necessary increase in crest level associated with wave run-up $\Delta h_{cl}^{R}(t)$ is then determined from the sea level rise $\Delta h_w(t)$ and the increase in the 2% wave run-up height $\Delta R_{u2\%}(t)$, as

$$\Delta h_{cl}^{R}(t) = \Delta h_w(t) + \Delta R_{u2\%}(t) \tag{7}$$

where $\Delta R_{u2\%}(t)$ is the difference of wave run-up heights between the future and present-day

$$\Delta R_{u2\%}(t) = R_{u2\%}(t) - R_{u2\%}(0) \tag{8}$$

By using Eqs. (4) and (5) and considering the time-dependent $H_{mo}(t)$ and $\xi_{m-1,o}(t)$ due to rising sea level, Eq. (8) is rewritten as

$$\Delta R_{u2\%}(t) = A_1 \gamma_b \gamma_f \gamma_\beta [\xi_{m-1,o}(t) H_{mo}(t) - \xi_{m-1,o}(0) H_{mo}(0)] \tag{9}$$

with a maximum of

$$\Delta R_{u2\%}(t) = \gamma_f \gamma_\beta \left\{ A_2 \left[ H_{mo}(t) - H_{mo}(0) \right] - A_3 \left[ \frac{H_{mo}(t)}{\sqrt{\xi_{m-1,o}(t)}} - \frac{H_{mo}(0)}{\sqrt{\xi_{m-1,o}(0)}} \right] \right\} \tag{10}$$

From Eqs. (7), (9) and (10), it is found that the necessary increase in crest level associated with wave run-up for the existing sea defences is given by sea level rise together with further contribution from increasing wave run-up height due to increase in local significant wave height.

### *3.2 Future crest level determined by overtopping*

Overtopping is the critical response of sea defence structures. Overtopping performance can be predicted by simple empirical formulae (Allsop *et al*. 2005) or by probabilistic methods (Reis *et al*. 2006). In sea defence design, the crest elevation is often determined by the wave overtopping discharge. The overtopping rate mainly depends on freeboard $R_c$ defined as the height of the structure crest above still water level. The mean specific overtopping discharge $q$ is a function of the governing hydraulic and structural parameters (CIRIA 2007) and can be predicted for breaking waves by

$$\frac{q}{\sqrt{gH_{mo}^{3}}} = \frac{B_1}{\sqrt{\tan\alpha}} \gamma_b \xi_{m-1,o} \exp\left[ -B_2 \frac{1}{\gamma_b \gamma_f \gamma_\beta} \frac{R_c}{\xi_{m-1,o} H_{mo}} \right] \tag{11}$$

with a maximum for non-breaking waves of

$$\frac{q}{\sqrt{gH_{mo}^{3}}} = B_3 \exp\left[-B_4 \frac{1}{\gamma_f \gamma_\beta} \frac{R_c}{H_{mo}}\right] \tag{12}$$

where $B_1 = 0.067$, $B_2 = 4.30$, $B_3 = 0.20$ and $B_4 = 2.30$ are empirical coefficients for deterministic uses. In order to maintain the present-day allowance for overtopping discharge, from Eqs. (11) and (12) the increase in freeboard with time $\Delta R_c(t)$ is determined by the difference between the future and present-day freeboards, expressed for breaking waves as

$$\Delta R_c(t) = \left[\frac{\xi_{m-1,o}(t)H_{mo}(t)}{\xi_{m-1,o}(0)H_{mo}(0)} - 1\right] R_c(0) + \frac{1}{B_2}\gamma_b \gamma_f \gamma_\beta \ln\left[\frac{\xi_{m-1,o}(t)H_{mo}^{\frac{3}{2}}(t)}{\xi_{m-1,o}(0)H_{mo}^{\frac{3}{2}}(0)}\right] \xi_{m-1,o}(t)H_{mo}(t) \tag{13}$$

with a maximum for non-breaking waves of

$$\Delta R_c(t) = \left[\frac{H_{mo}(t)}{H_{mo}(0)} - 1\right] R_c(0) + \frac{1}{B_4}\gamma_f \gamma_\beta \ln\left[\frac{H_{mo}^{\frac{3}{2}}(t)}{H_{mo}^{\frac{3}{2}}(0)}\right] H_{mo}(t) \tag{14}$$

The necessary increase in crest level $\Delta h_{cl}^{q}(t)$ associated with wave overtopping is then determined by

$$\Delta h_{cl}^{q}(t) = \Delta h_w(t) + \Delta R_c(t) \tag{15}$$

Eq. (15) indicates that the necessary increase in freeboard requires further increase in crest level of the existing sea defences above the requirement for sea level rise to maintain the present-day overtopping allowance.

## 4. Reliability analysis

Due to the rise of sea level and increase in local significant wave height in the future, hydraulic loading conditions, such as wave overtopping and water level difference across the structure, will affect the probability of failure for existing sea defence structures. In this study, various important structural failure modes at different locations of the structure are considered for reliability analysis, wave overtopping, failure of seaside revetment, internal erosion within earth dykes. In time-dependent reliability theory, a limit state equation for each potential failure mechanism is needed to describe the strength with time $L(t)$ and loading with time $S(t)$ for the failure mode (Chen and Alani 2012b, Foster *et al*. 2000, Chen and Bicanic 2010, Allsop *et al*. 2007, Chen and Maung 2014). The limit state equation is in general expressed as

$$Z(t) = L(t) - S(t) \tag{16}$$

whereby the sea defence structure fails to deliver the required functions when the loading exceeds the strength, i.e., for $Z(t) \le 0$.

### 4.1 Landside slope failure due to overtopping

Excessive wave overtopping may lead to failure of the landside slope of the earth dykes by eroding the dyke crest and landside slope and by deteriorating soil strength due to saturation of the earth dyke. An overtopping discharge of $2\times10^{-3}\,m^3/s$ per meter length could cause damage of the crest and landside slope if they are not protected (CIRIA 2007). Here, failure due to wave overtopping is defined as the exceedance of a predefined mean overtopping discharge $q_{cr}$, and then a limit state equation for this failure mode is expressed as

$$Z_q(t) = q_{cr} - \chi_q q(t) \tag{17}$$

where $\chi_q$ is model uncertainty coefficient associated with overtopping and $q(t)$ is average overtopping discharge associated with the rise of sea level and the increase in local significant wave height with time. The occurring discharge $q(t)$ is calculated from Eqs. (11) and (12), where empirical coefficients are taken as $B_2$ =4.75, and $B_4$ = 2.60 for probabilistic calculations.

### 4.2 Failure of seaside revetment and slope

A block revetment placed on a permeable layer on the seaside slope, as shown in Fig. 1, may fail due to hydraulic uplifting during wave run-down. As a consequence of the uplifting of the revetment, the seaside slope may be exposed to wave attacks, causing further erosion of the earth dyke slope. On the basis of the stability analysis for the seaside revetment (Hussaarts *et al*. 1999), the limit state function associated with the local significant wave height is expressed as

$$Z_r(t) = h_{crr} - \chi_r \frac{H_{mo}(t)\xi_{m-1,o}(t)}{M\cos\alpha} \tag{18}$$

where $h_{crr}$ is critical pressure head for uplifting of revetment blocks, calculating from

$$h_{crr} = \frac{\gamma_r - \gamma_w}{\gamma_w} d_r \tag{19}$$

in which $\gamma_r$ is density of revetment block, $\gamma_w$ is density of water, $d_r$ is thickness of revetment block, $\chi_r$ is model uncertainty coefficient associated with the uplifting, and coefficient $M = 4.06$ is model parameter.

### 4.3 Internal erosion within earth dykes

Internal erosion occurs in earth dykes due to the erosive actions of seepage flow. The failure mechanisms of internal erosion can occur under two conditions, i.e., continuous transport of fine sand particles takes place (piping) and the clay layer under the dyke is ruptured (uplifting).

The structure fails as a consequence of piping if the water pressure head across the structure exceeds the critical head difference $h_{crp}$, expressed the limit state equation as

$$Z_p(t) = h_{crp} - \chi_p \Delta h(t) \tag{20}$$

where the critical pressure head is given by $h_{crp} = L / c_L$ in which $c_L$ is constant depending on soil types (Weijers and Sellmeijer 1992), $\chi_p$ is model uncertainty coefficient associated with piping, and $\Delta h(t)$ is the difference between sea side water level $h_w(t)$ and land side water level $h_b$ , defined as

$$\Delta h(t) = h_w(t) - h_b \tag{21}$$

Uplifting occurs when the upward hydraulic force in the permeable layer exceeds the weight of the impermeable layer. The limit state equation for this failure mode is expressed as

$$Z_u(t) = h_{cru} - \chi_u \Delta h(t) \tag{22}$$

where $\chi_u$ is model uncertainty coefficient associated with uplifting of the clay layer behind the dyke. The critical pressure head for uplifting $h_{cru}$ is calculated from (Allsop *et al*. 2007)

$$h_{cru} = \left( \frac{\gamma_c}{\gamma_w} - 1 \right) D_c \tag{23}$$

where $\gamma_c$ is density of saturated soils of the impermeable layer, $D_c$ is thickness of the impermeable layer.

### *4.4 Time-dependent reliability analysis*

Probabilistic analysis can be a useful tool for evaluating the probability of failure of the existing sea defence structures affected by sea level rise with time. In the time-dependent reliability approach, when the strength or acceptable limit in the limit state equation described in Eq. (16) is deterministic and the loading is a Gaussian process, the probability of failure of the structure may be estimated from (Melchers 1999)

$$p_f(t) = \int_0^t \nu(\tau) d\tau \tag{24}$$

where $\nu(\tau)$ is the mean outcrossing rate over time, determined from

$$\nu = \nu^+_{L=\det} = \frac{\sigma_{\dot{S}|S}}{\sigma_S} \phi\left( \frac{L - \mu_S}{\sigma_S} \right) \left\{ \phi\left( \frac{\dot{L} - \mu_{\dot{S}|S}}{\sigma_{\dot{S}|S}} \right) - \frac{\dot{L} - \mu_{\dot{S}|S}}{\sigma_{\dot{S}|S}} \Phi\left( - \frac{\dot{L} - \mu_{\dot{S}|S}}{\sigma_{\dot{S}|S}} \right) \right\} \tag{25}$$

where $\phi$ and $\Phi$ are standard normal density and distribution functions respectively, $\mu$ and $\sigma$ are the mean and standard deviation, and $\mu_{\dot{S}|S}$ and $\sigma_{\dot{S}|S}$ are given by

$$\mu_{\dot{S}|S} = \mu_{\dot{S}} + \rho \frac{\sigma_{\dot{S}}}{\sigma_S} \left( L - \mu_S \right) \tag{26a}$$

$$\sigma_{\dot{S}|S} = [\sigma_{\dot{S}}{}^{2}(1-\rho^{2})]^{1/2} \tag{26b}$$

For a given Gaussian stochastic process with mean function $\mu_S(t)$ and auto-covariance function $C_{SS}(t_i,t_j)$, the variables $\mu_{\dot{s}}$ and $\sigma_{\dot{s}}$ as well as $\rho$ could be determined by

$$\mu_{\dot{S}} = \frac{d\mu_S(t)}{dt} \tag{27a}$$

$$\sigma_{\dot{S}} = \left[ \left. \frac{\partial C_{SS}(t_i,t_j)}{\partial t_i \partial t_j} \right|_{i=j} \right]^{1/2} \tag{27b}$$

$$\rho = \frac{C_{S\dot{S}}(t_i,t_j)}{\left[C_{SS}(t_i,t_i)\cdot C_{\dot{S}\dot{S}}(t_j,t_j)\right]^{1/2}} \tag{27c}$$

For limit state problems associated with sea level rise discussed above, the loading $S(t)$ can be estimated from the prediction by the given model for each potential failure mode $S_m(t)$ multiplied by model uncertainty coefficient $\chi$, a random variable with a mean value of unity, associated with the corresponding failure mode

$$S(t) = \chi S_m(t) \tag{28}$$

The mean function $\mu_s(t)$ and auto-covariance function $C_{ss}(t_i,t_j)$ for the limit state problem could be given as

$$\mu_s(t) = E[(S(t)] = S_m(t) \tag{29a}$$

$$C_{ss}(t_i,t_j) = \rho S_m(t_i) S_m(t_j) \tag{29b}$$

where $\rho$ is the correlation coefficient for $S(t)$ between two time points $t_i$ and $t_j$. Once the probability of failure for each individual failure mode is obtained, the overall failure probability of the structure system can be calculated from

$$P_f(t) = 1 - \prod_{i=1}^{4} [1 - p_{f,i}(t)] \tag{30}$$

It is assumed here that the chosen four failure modes are fully independent from each other, since these failure mechanisms occur at different locations of the earth dyke. As a result of the calculated probability of failure, the associated reliability index can be estimated by assuming that its relation with probability of failure follows a standard normal distribution function. In this paper, the structural performance is directly assessed by the probability of failure to avoid the transformation.

## 5. Worked example

A worked example for the earth dyke section shown in Fig. 1 is utilised for investigating the necessary increase in crest level and for analysing the reliability of the structure due to rising sea level. The structure has a crest height of 11.0 m from the toe, a seaside slope of 1:4 and a landside slope of 1:3. A layer of revetment blocks 0.8 m thick is placed on the seaside slope to prevent the earth slope from wave attacks. The earth dyke rests on a layer of impermeable clay soils 7.0 m thick. Below the clay is 5.0 m of water conductive sand layer overlying impervious bedrock. The extreme height water level at present day, which is caused by a combination of high tidal elevation plus a positive surge, is assumed at a level of 4.3m above the structure toe with a return period of 100 years. Results of sea level rise plotted in Fig. 2 and local significant wave height plotted in Fig. 3 are also adopted in calculations. The present-day sea defence such as crest level and foreshores are assumed to be unchanged in the future. The central estimates for three greenhouse gas emissions senarios, i.e., high emissions, medium emissions and low emissions, are considered in the calculations.

In order to maintain the present-day allowance for freeboard, there is a need to increase the height of the earth dyke in the future. As indicated in Eqs. (7)-(10) for calculating the increase in crest level by wave run-up, the increase in wave run-up height is not related to the existing crest level. Fig. 4 gives results for the influence of water depth at the seaside slope toe on the increase in crest level under future hydraulic conditions. The necessary increase in crest level includes the increase in wave run-up height and the corresponding sea level rise over time. An increase in crest level of 1.056 m, 0.924 m and 0.816 m in 85 years is required for a central estimate of high emissions, medium emissions, and low emissions, respectively.

Fig. 5 shows the results for necessary increase in dyke crest level under future hydraulic and structural conditions due to sea level rise. As expected, the required increase in crest level depends on greenhouse gas emissions scenarios for the projected sea level rise. The increase in crest elevation over time to counter rising sea level and maintain the overtopping allowance is approximately double of the corresponding sea level rise, giving a required increase in crest level of 1.223 m, 1.085 m and 0.965 m in 85 years for a central estimate of high emissions, medium emissions, and low emissions, respectively. These results are approximately 15% higher than that determined by wave run-up.

A reliability analysis is carried out for each potential failure mode of the earth dyke section. An allowable wave overtopping discharge of $2\times10^{-3} m^3/s$ per meter length, suggested by CIRIA for protecting earth dyke crest and landside slope (CIRIA 2007), is adopted in the limit state equation for the failure mode. The probability of failure caused by wave overtopping is plotted in Fig. 6 against the time after year 2010. It is evident from Fig. 6 that the probability of failure due to overtopping affected by rising sea level increases significantly with time, reaching approximately $8\times10^5$ times the present-day failure probability in year 2070. The results indicate that the risk of failure at unprotected crest and landside slope of the earth dyke erosion by excessive overtopping increases rapidly with time.

The probability of failure of the revetment placed on seaside slope of the earth dyke is plotted against time in Fig. 7. The block revetment has a thickness of 0.8 m and a density of 27.5 $kN/m^3$. The failure of seaside revetment is related to the increasing local significant wave height due to rising sea level. The results given in Fig. 7 indicate that the probability of failure of the revetment in year 2070 is about 80 times the failure probability at the present day.

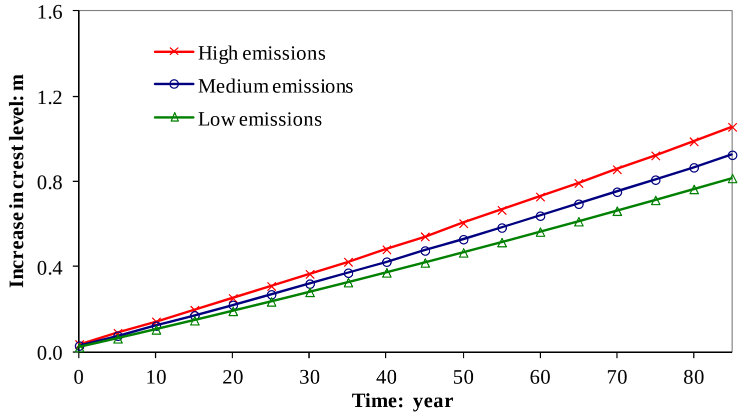


Fig. 4 Necessary increase in dyke crest level determined by future wave run-up to counter rising sea level for various emissions scenarios

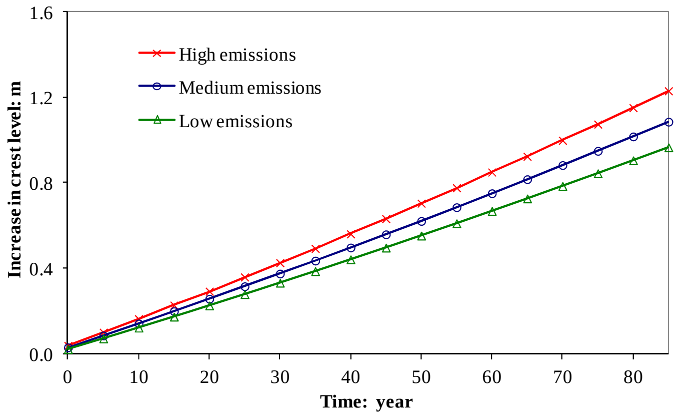


Fig. 5 Necessary increase in dyke crest level determined by future wave overtopping to counter rising sea level for various emissions scenarios

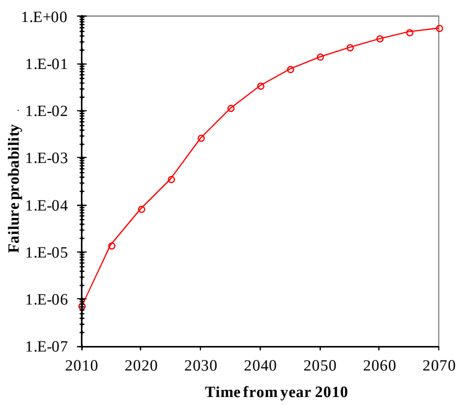


Fig. 6 Probability of failure caused by wave overtopping exceeding the allowable limit

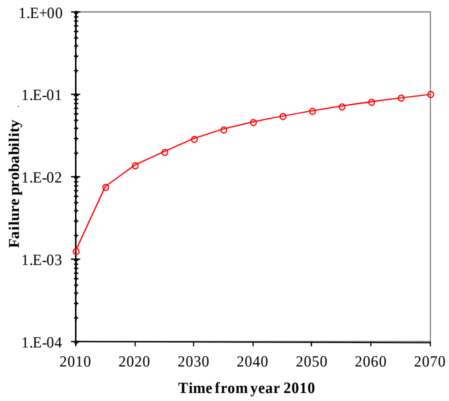


Fig. 7 Probability of failure caused by revetment failure on seaside slope

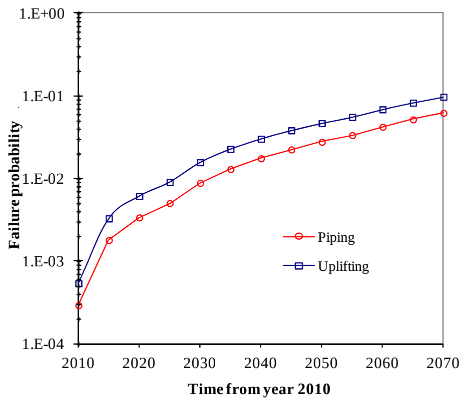


Fig. 8 Probability of failure caused by piping in sand layer and uplifting of clay layer of the structure

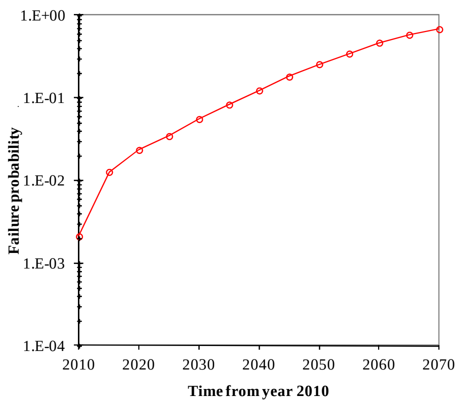


Fig. 9 Overall failure probability for earth dyke system due to rising sea level

Fig. 8 gives results for probability of failure due to internal erosions such as piping in the underlain water conductive sandy soils and uplifting of the impermeable layer behind the earth dyke, respectively. Both failure mechanisms are related to the increase in water pressure difference between the water levels on sea side and land side. The water level on sea side is taken as the rising sea level and the water level on land side is assumed at ground level. The permeable sand layer has a thickness of 5.0 m and seepage length of 76.5 m. The impermeable clay layer has a saturated density of $\gamma_c = 18kN/m^3$ and a thickness of 7.0 m at the toe of landside slope. The results in Fig. 8 show the probability of failure caused by piping in the sand layer in year 2070 is as many as about 214 times the present-day failure probability. In the meantime, the probability of failure due to uplifting of impermeable layer increases about 179 times during the same time period. It can be assumed therefore that the erosion process caused by piping and uplifting at the landward toe of the earth dyke becomes critical with time for the reliability of the dyke section concerned.

As discussed above, these four failure modes fail at different locations of the sea dyke section, i.e., at crest and landside slope by overtopping, at seaside revetment and slope during wave run-down, in underlain sand layer by piping and at clay layer by uplifting. The overall failure probability can then be obtained by combining the results for individual modes of failure, where the failure modes are assumed to be fully independent each other. The results in Fig. 9 show that the overall failure probability largely depends on the probability of failure due to wave overtopping. The overall failure probability reaches as many as about 10 times the present-day value in year 2025 and about 100 times in year 2050.

## 5. Conclusions

The effect of rising sea level due to climate change on the existing sea defences is critical to effective management of flood defence infrastructure. The local significant wave height will increase in the future at a similar rate to sea level rise. The wave run-up height and overtopping discharge of the existing sea defences will increase under future hydraulic conditions. In order to maintain the present-day allowances for the wave run-up height and overtopping rate in the future, the crest elevation of sea defences needs to be increased at an approximately double value of sea level rise, without consideration of the settlement of dyke crest level. The water depth at the toe of seaside slope has influence on the necessary increase in crest level. The shallower water depth at the toe requires greater increase in crest level. The predictions of increase in crest level by wave run-up and overtopping are close each other for the case considered, but the prediction by wave run-up does not require information on the existing crest level.

From the results obtained for the worked numerical example, overtopping is most critical for overall reliability of the given sea dyke system, and the probability of failure by overtopping increases significantly in order of magnitude of approximately 106 during the period from year 2010 to year 2070. Piping in the underlain water conductive sand layer and uplifting of the clay layer also make the earth dyke vulnerable due to sea level rise in the future, with failure probability in year 2070 of about 200 times the present-day value. The failure probability of revetment on seaside slope also increases quickly with time, causing progressive erosion at seaside slope in the future. The overall failure probability with time increases in order of magnitude of about 10 in 15 years and about 100 in 40 years, comparing with the failure probability at present day.

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