In this paper, the reliability analysis of sea flood defence systems and the probabilistic approach of the design of the sea flood defences are outlined. The application of probabilistic design methods offers the designer a way to unify the design of engineering structures, processes and management systems. For this reason, there is a growing interest in the use of these methods in the design and safety analysis of civil engineering systems including coastal flood defences. In this paper, these methods are critically reviewed. The so-called probabilistic safety assessment and reliability-based design models are developed and applied to a coastal flood defence system in Nam Dinh, Vietnam. An accurate safety assessment of the existing coastal defence system and a set of reliability-based solutions are, therefore, of large importance for rehabilitating coastal flood defences of the considered case study area. In accordance with present situation and analysis results, the coastal flood defences in Nam Dinh are not strong enough to withstand the actual sea boundary condition and ensure the safety for protected areas in views of present socio-economic development. As part of knowledge transfer, the analysis results could contribute to a fundamental base for long-term planning rehabilitation of the sea defences in Vietnam.

1. Introduction

Use of probabilistic design methods is increasing in civil engineering design in general and hydraulic/coastal engineering in particular. In order to undertake an effective design and assessment of the overall reliability of a coastal structure, it is essential to have a thorough knowledge and understanding of all possible failure modes in the design stage. Last few decades effort has been devoted to improving our knowledge of how the structure system fails and the probabilistic design concepts have been increasingly proposed and applied to the field of hydraulic/coastal engineering (see e.g. the concept, method and application in Bakker & Vrijling 1980; Vrijling et al. (1998); Voortman (2002); and Oumeraci et al. (2001)). However, gaps in knowledge still remain in widening applications of the probabilistic approach in design and safety assessment of coastal structures i.e. estimating the appropriate statistical distributions of the sea loads acting on the structures; investigating the appropriate characterization of failure mechanisms of coastal structures; description of the structure system as a whole and
quantifications of system failure probability; and reliability based design of the system and system components. This paper presents the recent development of the probabilistic design approach with an attention of the last two issues with an application to a case study of coastal flood defences in Vietnam.

Coastal flood defence systems can be represented by fault trees. An example is given in Figure 1 failure of the subsystems (dike, dune sluice, levee) of the system leads to flooding of the polder area. The subsystems all consist of elements. The dikes can for instance be divided in sections. Failure of any of the elements of the subsystem leads to flooding of the protected area. The most important failure modes of a flood defence have been addressed and modelled by Allsop et al. (2006) and Mai Van et al. (2007). A reliability tool has been developed in van Gelder et al. (2008), which is able to calculate the failure probability of the top event flooding with a Monte Carlo simulation approach, for any construction type, configuration of the fault tree, and for all probability density functions of load and resistance variables. This paper will show the applicability of the reliability tool and further develop a reliability based design models to a case study in Vietnam. This paper is outlined as follows. First an introduction is given to probabilistic design approach. The paper continues by addressing the tools for a probabilistic systems analysis and calculation methods. Subsequently, application of these methods for a case study of a coastal flood defence system in Nam Dinh, Vietnam is given. Finally, conclusions and recommendations are presented.

Probabilities of failure can be determined by analysing historical failure data and by probabilistic calculation of the limit states. For most cases there is not enough specific failure data available so we have to determine the failure probabilities by computation. A limit state function is a function of the strength and the load for a particular failure mode. In general the formulation of the limit state function is: \( Z=R-S \) in which \( R \) is the strength and \( S \) is the load. The failure mode will not occur as long as the limit state function is positive. The line \( Z=0 \) is a limit state. This line represents all the combinations of values...
of the strength an the loading for which the failure mode will just not occur. So it is a boundary between functioning and failure. In the limit state function the strength and load variables are assumed to be stochastic variables. A stochastic variable is a variable which is defined by a probability distribution and a probability density function. The probability distribution $F(x)$ returns the probability that the variable is less than $x$. The probability density function is the first derivative of the probability distribution.

If the distribution and the density of all the strength and load variables are known it is possible to estimate the probability that the load has a value $x$ and that the strength has a value less than $x$. The failure probability is the probability that $S=x$ and $R<x$ for every value of $x$. So we have to compute the sum of the probabilities for all possible values of $x$:

$$P_f = \int_{-\infty}^{\infty} f_S(x) F_R(x) \, dx$$

This method can be applied when the strength and the load are independent of each other. In case of dependence, the failure probability can by determined by summation of the probability density of all the combinations of strength and load in this area.

$$P_f = \int_{x<0} \int f_{RS}(r,s) \, dr \, ds$$

In a real case the strength and the load in the limit state function are nearly always functions of multiple variables. For instance the load can consist of the water level and the significant wave height. In this case the failure probability is less simple to evaluate. Nevertheless with numerical methods like numerical integration and Monte Carlo simulation it is possible to solve the integral:

$$P_f = \int_{x<0} \int \int f_{x_1,\ldots,x_{12},s_1,\ldots,s_{12}}(x_1,\ldots,x_{12},s_1,\ldots,s_{12}) \, dx_1 \ldots dx_{12} \, ds_1 \ldots ds_{12}$$

These methods which take into account the real distribution of the variables are called level III probabilistic methods. In the Monte Carlo simulation method a large sample of values of the basic variables is generated and the number of failures is counted. The number of failures equals:

$$N_f = \sum_{j=1}^{N} I(g(x_j))$$

In which $N$ is the total number of simulations. The probability of failure can be estimated by:

$$P_f \approx \frac{N_f}{N}$$

The coefficient of variation of the failure probability can be estimated by:

$$V_{P_f} \approx \frac{1}{\sqrt{NP_fN}}$$
In which $P_f$ denotes the estimated failure probability.

If the limit state function ($Z$) is a sum of a number of normal distributed variables then $Z$ is also a normal distributed variable. The mean value and the standard deviation can easily be computed with these equations:

$$Z = \sum_{i=1}^{n} a_i x_i , \quad \mu_Z = \sum_{i=1}^{n} a_i \mu_i , \quad \sigma_Z = \sqrt{\sum_{i=1}^{n} a_i \sigma_i} .$$

This is the base of the level II probabilistic calculation. The level II methods approximate the distributions of the variables with normal distributions and they estimate the limit state function with a linear first order Taylor polynomial, so that the $Z$-function is normal distributed.

If the distribution of the $Z$-function is normal and the mean value and the standard deviation are known it is rather easy to determine the failure probability.

3. Safety assessment and Reliability based design

On the above basis, if the geometry of CFDS is known and the joint probability distribution of load and strength variables is quantified, the probability of failure of the system of flood defences can be found. This can be applied for technical management purpose to determination of safety levels of an existing system and to find out the weak points of the system, see Table 1 for an example of result form.

<table>
<thead>
<tr>
<th>section</th>
<th>overtopping</th>
<th>piping</th>
<th>etc.</th>
<th>total</th>
</tr>
</thead>
<tbody>
<tr>
<td>dike 1.1</td>
<td>$P_{1.1}$(overtop.)</td>
<td>$p_{1.1}$(piping)</td>
<td>$p_{1.1}$(etc.)</td>
<td>$p_{1.1}$(all)</td>
</tr>
<tr>
<td>dike 1.2</td>
<td>$P_{1.2}$(overtop.)</td>
<td>$p_{1.2}$(piping)</td>
<td>$p_{1.2}$(etc.)</td>
<td>$p_{1.2}$(all)</td>
</tr>
<tr>
<td>etc.</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>dune</td>
<td>$p_{dune}$(overtop.)</td>
<td>$p_{dune}$(piping)</td>
<td>$p_{dune}$(etc.)</td>
<td>$p_{dune}$(all)</td>
</tr>
<tr>
<td>sluice</td>
<td>$p_{sluice}$(overtop.)</td>
<td>$p_{sluice}$(piping)</td>
<td>$p_{sluice}$(etc.)</td>
<td>$p_{sluice}$(all)</td>
</tr>
<tr>
<td>total</td>
<td>$p_{all}$(overtop.)</td>
<td>$p_{all}$(piping)</td>
<td>$p_{all}$(etc.)</td>
<td>$p_{all}$(all)</td>
</tr>
</tbody>
</table>

The last column of the table shows immediately which element or section has the largest contribution to the probability of flooding of the polder under study. Inspection of the related row reveals which mechanism will most likely be the cause. Thus a sequence of measures can be defined which at first will quickly improve the probability of flooding but later runs into diminishing returns.

On the other hand, for a pre-defined failure probability (e.g. given an acceptable level of probability of flooding for a certain location) vector of acceptable design geometrical alternatives can be given by equation (*). In order to find the solution a cost optimization can be done by equation (**), see also Voortman 2002 and Mai et al. 2006.

\[
\begin{align*}
D &= \{ z \mid P_f(z) \leq P_{f,max} \} \hspace{1cm} \text{..................................................(*)} \\
\text{Min.} \left[ I(z \mid P_f(z) \leq P_{f,max}) \right] + PV[R_f(z \mid P_f(z) \leq P_{f,max})] \hspace{1cm} \text{......(**)}
\end{align*}
\]
where $P_{f,max}$ denotes the maximum acceptable probability of system failure; $I$ is the initial investment cost and $R_T$ is the maintenance/repair cost during the total service lifetime of the structures, $T$. The investment cost can be estimated based on given geometry while the later term can be expressed through the failure probability of the each consider alternative.

4. Applications

4.1 Coastal flood defences in Vietnam and description of the case study

Vietnam lies in a tropical monsoon climate region that has a long coastline along the South China Sea that is regularly substantial suffering due to floods and typhoons. Typhoons arrive on average 4 to 6 times per year at the Vietnam coast. The deltaic coastal areas to a distance of about 20 km behind, which are protected by the 2000 km of sea dikes, is threatened by storm surges and high tides from the sea and high water level from the rivers. Thus, water defences are crucially important to protect the country from flooding.

The applied design safety levels of sea dikes are relatively low. This was also noticed in 1996 after the visit of Dutch expertise missions (DWW/RWS, 1996). Most designs of the sea dikes in Vietnam are based on the sea loads with return period 20 years. Besides this fact the Dutch mission marked that most Vietnamese dikes were in poor and disputable conditions (DWW/RWS, 1996). Consequently, the true probability of failure of the Vietnamese sea flood defense system exceeds by far the design frequency (Mai et al., 2006, 2007). Although designed to fail once in 20 years the sea defense system might well fail almost every year. The experiences in the past years support this statement.

Currently, the design guidelines for sea dikes, named 14TCN-130-2002, are used for all sea dike design in Vietnam. In this guidelines the design water level comprises of two components of i) sea water level of 5% exceedence frequency (1/20 year return period) of occurrence of tidal levels, and ii) storm surge heights cause by the design storm which corresponds to the wind speeds at the Beauford scale 9 to 10. The storm surge heights in dike design were fixed in the code by specific values. Arbitrary selection of 5% tidal level and artificial treating storm surge level and tidal level as two independent components does not reflect well the physic of total water level due to typhoon occurrence. In addition, selection of the design storms does not properly reflect statistical sense with no associating by any frequency of occurrence. Therefore, the safety level in the design guidelines only explicitly refers to a 1/20 year return period of the tidal level; it does not statistically count for extreme events i.e. typhoons.

Coastal flood defence system of Nam Dinh province in Vietnam is selected as a case for this study. Nam Dinh coastal zone is protected by 90 km of sea dikes. The dike system has been constructed on the basis of a sea load with 20 year return period. There are two type of sea dikes: one was constructed approximately 20 years ago and the other
was just constructed after the typhoon Damrey September 2005 as emergency responds for closure of more than 8 km of sea dike breaches during the typhoon. Both types are considered in the safety assessment of the Nam Dinh sea defence system.

4.2 Safety assessment

By the probabilistic safety and reliability assessment models, the safety of the Nam Dinh sea dike system is determined. All possible failure mechanisms of the sea dike sections are taken into account. The safety assessment of the present dike system and of different scenarios of dike improvement has been made. By using level III method with Monte Carlo simulation the failure probability of possible failure modes and of the whole dike system is analyzed. In addition contribution of failure modes to the total system failure probability can be determined by fault tree analysis. Fault tree of the system and analysis result are summarized in Figure 3 & 4. More details on application techniques and extensive calculation steps see also Mai Van et al. 2006.

Study found that the failure of the Nam Dinh sea dikes is mainly due to wave overtopping. The existing sea dike system has a total failure probability of 0.15 per year (1/7 year) when taking into account the length effects, although the dike system was designed to withstand a one in 20 year sea load. From Figure 4, in order to come up with the current existing standard of 1/20 year, the dikes should be heightened up to around 6.8m. Findings of this study are a good agreement with results of deterministic safety assessments and what have happened at the case study area during the last few decades (DWW/RWS 1996).
4.3 Reliability based optimal design

By applying the reliability-based model the design parameter can be determined. Maximum failure probability of the Nam Dinh sea dike system is 5% (1/20 year safety standard) as from the current safety standard (14TCN-130-2002). Taking into account the length effects the acceptable failure probability of a dike section approximates 0.0171. By implementing a fault tree analysis, based on the failure probability of the dike section the failure probability of every failure mode can be reallocated. Design values of the interested geometries for the dike section are obtained as in Table 2, column e.

Table 2. Failure probability vs. failure mode and the design values of interested parameters per mechanism

<table>
<thead>
<tr>
<th>Failure modes</th>
<th>Safety assessment</th>
<th>Reliability based design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pf [year⁻¹]</td>
<td>important</td>
</tr>
<tr>
<td>(a)</td>
<td>(b)</td>
<td>(c)</td>
</tr>
<tr>
<td>Overflowing</td>
<td>3.81E-03</td>
<td>12.80%</td>
</tr>
<tr>
<td>Excessive wave overtopping</td>
<td>1.30E-02</td>
<td>43.69%</td>
</tr>
<tr>
<td>Instability of armour unit</td>
<td>6.25E-03</td>
<td>21.00%</td>
</tr>
<tr>
<td>Instability of outer slope</td>
<td>3.10E-05</td>
<td>0.10%</td>
</tr>
<tr>
<td>Instability of inner slope</td>
<td>5.70E-03</td>
<td>19.15%</td>
</tr>
<tr>
<td>Instability of toe protect</td>
<td>1.10E-03</td>
<td>3.70%</td>
</tr>
<tr>
<td>Excessive toe erosion</td>
<td>1.89E-04</td>
<td>0.64%</td>
</tr>
<tr>
<td>Instab. of toe structure</td>
<td>1.00E-02</td>
<td>0.64%</td>
</tr>
<tr>
<td>Piping condition 1</td>
<td>6.57E-04</td>
<td>0.00%</td>
</tr>
<tr>
<td>Piping condition 2</td>
<td>9.00E-12</td>
<td>0.00%</td>
</tr>
<tr>
<td>System failure probability upper bound</td>
<td></td>
<td></td>
</tr>
<tr>
<td>System failure probability lower bound</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Length effects: Considered the dike system is uniform cross-sectional system which comprises number of independent sections. This can be described as a series system in probabilistic design. The maximum failure probability of this series system with n element may be calculated by: $P_{\text{system}}=1-(1-P_{\text{section}})^n$
5. Conclusions and recommendations

Probabilistic design approach is a powerful tool in reliability analysis of coastal flood defences. By introducing simple failure probability table (Table 1) the weakest point/link of the system can be easily identified as the first insight by the component which has the largest failure probability. More insight into how importance of failure mechanisms contributing to the total failure probabilities of every system component and to the overall failure probabilities of the whole system can be found by use of fault tree analysis with Monte-Carlo simulation. Influence of all related stochastic variables to the corresponding failure mechanisms has been archived by sensitivity analysis based on the influence factor $\alpha$ from FORM analysis.

Application of the methods for the case of Namdinh sea dikes in Vietnam gave us interesting results and allows the following remarks:

The existing old dike system is not safe according to the current design codes (overall system failure probability is 0.15 per year while the required failure probability is 0.05 per year).

Within ten possible failure mechanisms indicated above overtopping is the most likely to occur with 43.69 percentages of contribution. Instability of amour layer influences relatively high at 21%.

Reliability based design comes up with a set of dike geometry given a 0.05 per year system failure probability as the current safety standard. Re-checking the safety of proposed solution shows that the overall failure probability of top event, dike system failure, is nearly $5 \times 10^{-2}$ per year. This fulfils the current safety standard of Vietnam for sea defences.

References

5. DDMFC-Department of Dikes Management and Flood Control of Vietnam. 2007. Flood damage reports.