Safety of coastal defences and flood risk analysis

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ABSTRACT: This paper is aimed at safety assessment of coastal flood defences and investigation of flood risk for coastal regions. Firstly, a general safety assessment procedure of coastal defences is performed by applying reliability analysis, which includes steps of identification of failure modes by various possible failure mechanisms, statistic description of hydraulic loads and resistance of the coastal defences and calculation of the failure probability. Secondly, overview of risk analysis of coastal defence system is followed, in which due to possible failures of the system, loss of life, economic, environmental, cultural losses and further intangibles can occur. It is necessary to determine the question if safe is safe enough and figure out acceptable risk levels. Acceptable risk is strongly related with the acceptable probability of failure and the acceptable amount of damages and losses. The state of the art in risk analysis will be reviewed and discussed. Application of the reliability and risk analysis to case studies of Vietnam coastal flood defences is investigated.

1 INTRODUCTION

1.1 Background

Sea dikes are the most important coastal structures along the coastlines of Vietnam and many other coasts around the world which have their low-lying areas behind. The main function of sea dikes is to protect low-lying coastal areas which are highly vulnerable to coastal flooding. In general the design of these coastal structures is based on purely deterministic or quasi-deterministic approaches. The design water level of such structures is normally based on the maximum wave run-up or on acceptable overtopping rates under extreme storm surge conditions. Due to the fact that the coastal structures are usually under impacts of many natural phenomena and processes which happens randomly both by time, space, their intensity and amplitude, therefore the deterministic approaches same to not bring closer proper outcomes. In addition, geotechnical related failure modes of sea dikes are often disregarded and/or not properly accounted for in the design process.

Probabilistic approach with reliability and risk based design concepts have been increasingly proposed and applied in the last few years (see e.g. the concept, method and application in Bakker & Vrijling 1980; Vrijling et al., 1998; Voortman 2002; and Oumeraci et al., 2001). The probabilistic method allows designers take into account the uncertainties of the input parameters and treat them as the random variables. Moreover, probabilistic models describe possible failure modes for various types of hydraulic and coastal structures. Even though, application of these methods is still limited to the simple cases or just to a couple of failure mechanisms. Especially in developing countries as Vietnam these methods is just introduced as a new modern design approach which has few applications. Implementation of probabilistic design and risk analysis to Vietnamese situation of flood defence and coastal protection is therefore necessary. The implementation methods can also be applied for the cases of other developing countries.

Nevertheless, before go into detailed analyses and applications of probabilistic approach in the field of coastal defence structures, it is strongly necessary to have an overview on understanding of the methods. Study on physical processes of loading and the physical failure mechanisms of the coastal structures are also large important to have a proper view before establishing a probabilistic models for these issues. These need a broad research effort in near futures. This paper just presents some initial results of applying reliability and risk based approach for Vietnamese coastal defence situation.

1.2 Fluvial flood and coastal defences in Vietnam

Vietnam is affected regularly by substantial suffering due to floods. The most severe floods occur during high river discharges and during, and shortly after, typhoons. The typhoons are accompanied by torrential rains causing flash floods which regularly submerge
Although designed to fail only once in 20 years, this system may exceed the design frequency. Since 1996, Vietnam was affected by several flood disasters, each of those responsible for the loss of hundreds of lives and considerable damage to infrastructure, crops, rice paddies, fishing boats and trawlers, houses, schools, hospitals, etc. The total material damage of the flood disasters in these years exceeded USD 500 million, which damage was accompanied by the loss of almost over 1000 lives. The flood disasters occurred in North Vietnam (1996 and 2005), in South Vietnam (1997) as well as in the Central provinces (1998, 2005). Most floods were initiated by typhoons and occurred in the coastal zone.

The relative low safety level of the Vietnamese dikes was noticed in 1996 during two visits of Dutch missions to Vietnam. Most designs of the Vietnamese sea and river dikes are based on loads with return period 20 years or even shorter periods. Compared to the Dutch standard (return periods 3000 to 10000 year) these return periods are very small. Besides this fact the Dutch mission marked most Vietnamese dike designs as poor and disputable (Vrijling & Hauer 2000). Designs are not always based on the right formulae, hydraulic boundary conditions are not always based on proper statistics (for example design water level and wind set up are not always treated as two independent phenomena), failure mechanisms which differ from overtopping are often neglected, no attention is given to length effects, monitoring and timely repair of small damages is often at a poor level, etc. As a result the true probability of failure (probability that the load is larger than or equaled to the resistance of the flood defence system) of the Vietnamese water defence system may exceed the design frequency. Although designed to fail only once in 20 years this water defense system might well fail almost every year. The experiences in last 10 years support this assertion.

1.3 Damrey typhoon and its consequences

The Damrey typhoon (named in Vietnam as Storm No. 7) is considered the most vigorous in the last 50 years. With wind force near to the storm center at Beaufort scale 12 (118 to 133 km per hour) and the wind gusting above Beaufort Scale 12, the typhoon was forecasted to affect all coastal provinces of Vietnam from Quang Ninh to Da Nang. The typhoon was forecasted to be very dangerous, particularly to the sea dike system, serves to protect hundreds thousands of people, rice fields and aquaculture production areas along the coast of Vietnam. By 5.00 AM on 27 September the wind force rose from Beaufort scale 7-8 to scale 12 (133 km per hour) hitting provinces from Hai Phong to Thanh Hoa as the typhoon moved in a west to northwest direction. The typhoon caused high storm surges, which coincided with high tide so they were amplified.

The wave run up was as high as 3–4 meters, high storm surge in combination with high tide led to too much overtopping of sea water at sea dikes in almost all affected provinces. It broke certain sea dike sections in Hai Hau district in Nam Dinh province and Hau Loc district in Thanh Hoa. The total damage was enormous: 25 km of sea dikes were broken and nearly totally destroyed. In Nam Dinh a stretch of 800 m sea dikes was completely washed out.

The seawater penetrating the inland by 3–4 km in coastal provinces and the following flash floods in upland areas have destroyed at least 1,194 houses and damaged another 11,576. More than 130,000 ha of rice fields have been submerged and damaged, most of which could not been harvested before the typhoon. Severe damage occurred to transport and electricity infrastructure and particularly the irrigation systems. According to the rapid assessment of Disaster Management Working Group the direct damage estimate as of 28 September is USD 430 million.

2 OVERVIEW OF PROBABILISTIC APPROACH IN FLOOD DEFENCES

Since the last few decades the awareness grew that the probability of exceedance of the design water level, the design frequency or the return period is not a good predictor of the true probability of flooding. Normally the dike crest exceeds the design water level by some measure, thus the probability of overtopping is smaller than the design frequency. But some parts of the dike may already be critically loaded before the design water level is reached. There are more failure mechanisms that can lead to flooding of the polder than overtopping. Another danger is that water discharge structures (e.g. sluices, gates, ship logs) are not functioned well in time before the high water moment. Moreover the length of a dike ring has a considerable influence. A chain is as strong as the weakest link. So a single weak spot determines the actual safety of an entire dike ring. Due to this so called length effect the probability of failure of a dike ring depends on the length of this
Figure 2. Flood defence system and its elements presented in a fault tree (CUR, 1988).

Figure 3. A dike section as a series system of failure modes.

ring as well. In a practical case the probability of failure of an entire dike ring may exceed the probability of failure of a single section by for example a factor 10 or higher. More extensive description can be found in Vrijling et al., 1993.

The probabilistic approach aims to determine the true probability of flooding of a polder and to judge its acceptability in view of the consequences. As a start the entire water defence system of the polder is studied. Typically this system contains sea dikes, dunes, river levees, sluices, pumping stations, high hills, etc. (Figure 2). In principle the failure and breach of any of these elements leads to flooding of the polder. The probability of flooding results from the probabilities of failure of all these elements.

Within a longer element e.g. a dike of some kilometers length, several independent sections can be divided. Each section may fail due to various possible failure mechanisms like overtopping, sliding of outer or inner slope, piping, erosion of the protected outer slope, ship collision, dike’s toe instability, etc. The relation between the failure mechanisms in a section and the unwanted consequence flooding can be schematised with a fault-tree (Figure 3).

The failure probabilities of the mechanisms are calculated using the methods of the modern reliability theory, like Level III or Level II methods.

In the reliability calculations all uncertainties are taken into account. Three classes of uncertainties are distinguished: The intrinsic uncertainty is characteristic for natural phenomena; Model uncertainty describes the imperfection of the engineering models in predicting the behaviour of river courses, dikes and structures; Statistical uncertainty is caused by the lack of data. More extensive discussion see also van Gelder 1999.

Because all uncertainties are included in the calculations of the failure probability the latter is not singly a property of the physical reality but also of the human knowledge of the system. The consequence is that the safety of a dike system as expressed by the calculated probability of flooding can be improved by increasing our knowledge as well, apart from strengthening the weakest elements in this system. All calculated probabilities of failure can be presented in a form of Table 1.

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.

2.1 Reliability analysis of a multi-elements system

Quantification of the probability of system failure starts with the definition of reliability functions for all failure modes in the lowest level of the fault tree. As from literatures, the general form of reliability function of component ith of the system can be simply written by Bakker & Vrijling (1980):

$$Z_i = R_i - S_i$$  (1)

In which $R_i$ and $S_i$ are multivariate functions which stand for strength and load, respectively and both concerned with stochastic variables. The probability density function of $Z_i$ given by $f_{Z_i}(X)$. Then the probability of failure of the element ith which has the limit state function $Z_i$ is:

$$P(Z_i < 0) = \int_{-\infty}^{0} f_{Z_i}(X)dx$$  (2)

In reality $Z_i$ may be a function of number of stochastic variables $X_1, X_2, \ldots, X_n$, as both the “load”, $S_i$, and
the “strength”, $R_i$, and may depend on more than one variable. In order to perform a level II calculation, the first order reliability methods (FORM) is used. In the FORM analysis, the failure surface $Z(X) = 0$ at the design point $X^*_j$ is approximated by the hyperplane normal to the vector $X^*_j$ (Rackwitk, 1977). By using Taylor expansion the failure function is linearised and after the linearisation it can be stated as:

$$Z_m = Z(X_1, X_2, ..., X_n) + \sum_{j=1}^{n} (X_j - X^*_j) \left( \frac{\partial Z}{\partial X_j} \right)_{X_j=X^*_j} = 0$$

$Z_{lin}$: Linearized reliability function of $Z_i$ in $\{X^*_j\}; (\frac{\partial Z}{\partial X_j})_{X_j=X^*_j}$ gradient vector at the design point $X^*_j$, determined by partial derivative of $Z_j$ with respect to $X_j$ evaluate in $X_j = X^*_j$.

The mean value and standard deviation of $Z_{lin}$ are:

$$\mu(Z_{lin}) = Z(X_1, X_2, ..., X_n) + \sum_{j=1}^{n} (\mu_{X_j} - X^*_j) \left( \frac{\partial Z}{\partial X_j} \right)_{X_j=X^*_j}$$

$$\sigma^2_Z = \sum_{j=1}^{n} \sigma_{X_j}^2 \left( \frac{\partial Z}{\partial X_j} \right)^2_{X_j=X^*_j}$$

If mean values $X_1^* = \mu(X_1), ..., X_n^* = \mu(X_n)$ are situated, a so called mean value approximation of the probability of failure is obtained. If the failure boundary is nonlinear, a better approximation can be achieved by linearization of the reliability function at a design point. The design point is defined as the point on the failure boundary in which the joint probability density is maxima. Therefore the design point can be obtained by:

$$X_j^* = \mu_{X_j} - \alpha_j \beta \sigma_{X_j}$$

where, reliability index and influence factor of variable number $j$th to failure probability of element $i$th can be determined by:

$$\beta = \frac{\mu(Z_{lin}) - \alpha_j \sigma_{X_j}}{\sigma(Z_{lin})}$$

$$\alpha_j = \frac{\sigma(X_j)}{\sigma(Z_{lin})} \left( \frac{\partial Z}{\partial X_j} \right)_{X_j=X^*_j}$$

A more general form of a reliability function that covers a large number of cases is given by:

$$g(z,X) = R(z,X) - S(z,X)$$

where $R$ is the resistance of the component, $S$ is the loading on the component, $z$ is a vector of design variables describing among others the structural geometry of the component and $X$ is a vector of load of random variables.

The occurrence of the failure mode described by equation (9) is indicated by negative values of the reliability function. If $f_j(X)$ denotes the joint probability distribution of random input boundaries. The probability of occurrence of every failure mode is given by:

$$P_j(z,X) = P(g(z,X) < 0) = \int_{g(z,X)<0} f_j(X) dX$$

The overall failure probability of a dike section number $i$th is given by:

$$P_{failure,sec; i} = P(Z_1 < 0; Z_2 < 0; ... Z_i < 0; ... Z_m < 0)$$

where $(Z_1 < 0; Z_2 < 0; ... Z_i < 0; ... Z_m < 0)$ denotes at least one of $m$ failure mechanisms occurs.

The fundamental lower and upper bounds in this level II approach are approximated by:

$$\max \left\{ P(Z_i < 0) \right\} \leq P_{failure,sec; i} \leq \sum_{i=1}^{m} P_i \left( Z_i < 0 \right)$$

For more narrower bounds with better approximation it is suggested to use Ditlevens bound (see Ditlevens 1979).

2.2 Discussions

Following the above background, there are two essential probabilistic approach based models can be applied and developed conceptually:

1) Probabilistic assessment of safety level: If the geometry of every component 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 the existing system and to find out the weakest point of the system.

2) Reliability-based design model: very often the question is, however, reversed for a design purpose. For a pre-defined failure probability (e.g. given an acceptable level of probability of flooding for a certain location) a geometry of the structure needs to be found in order to fulfill the design requirement in combination with minimisation of the construction cost of the system. For a fixed value of the probability of failure, the set of acceptable design alternatives is conceptually given by (12a). In order to find the unique solution the cost optimisation has to be accounted for as shown in (12b). The optimal design geometry can be found mathematically by the following equation system (see Voortman, 2002):

$$D = \{ z \mid P_j(z) \leq P_{j,max} \}$$

$$\text{Min} I(z \mid P_j(z) \leq P_{j,max})$$

where $P_{j,max}$ denotes the maximum acceptable probability of system failure; $I$ denotes the direct cost of the system.

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The estimation of the consequences of a flood constitutes a central element in the modern approach. Most probably society will look to the total damage caused by the occurrence of a flood. This comprises a number of casualties, material and economic damage as well as the loss of or harm to immaterial values like works of art and amenity. Even the loss of trust in the water defense system is a serious, but difficult to quantify the effect.

However for practical reasons the notion of risk in a societal context is often reduced to total number of casualties using a definition as “the relation between frequency and the number of people suffering from a specified level of harm in a given population from the realisation of specified hazards”. If the specified level of harm is limited to loss of life, the societal risk may be modelled by the frequency of exceedence curve of the number of deaths, also called the FN-curve (see more detail in Vrijling et al., 1998). When the probabilities of flooding of any polder are calculated the question of if the polder safe enough may arise. The answer for this question can only come from a broad judgement of the cost of improving the defences against the reduction of the probability of flooding related to the scale of the damage. This judgement is political in essence, because the costs as well as the benefits contain many aspects that may be valued quite differently by different parties. Dike improvement costs money, land area, cultural heritage and amenity.

Obviously, if dike improvement is relatively expensive a higher probability of flooding will be accepted. On the other hand if the consequence of flooding is relatively substantial one will aim for a smaller probability. Moreover, the environmental consequences of flooding and the potential effects on nature and cultural heritage should also play an increasing role in assessing the required scale of flood protection. The image of the country as a safe place to live, works, and invest is finally at stake. This justifies a fundamental reassessment of the acceptability of the flood risks in view of the costs of improvement.

From literatures, the acceptance of risk should be studied from the three different points of view in relation to the estimation of the consequences of flooding. The first point of view is the assessment by the individual. Attempts to model this are not feasible, therefore it is proposed to look to the preferences revealed in the accident statistics. The fact, that the actual personal risk levels connected to various activities show statistical stability over the years and are approximately equal for the Western countries, indicate a consistent pattern of preferences. The probability of losing one’s life in normal daily activities such as driving a car or working in a factory appears to be one or two orders of magnitude lower than the overall probability of dying. Only a purely voluntary activity such as mountaineering entails a higher risk (see Vrijling et al., 1998).
Apart from a slightly downward trend of the death risks presented (due to technical progress), it seems permissible to use them as a basis for decisions with regard to the personally acceptable probability of failure, by means of the equation below:

\[ P_{\beta} = \frac{\beta_i \times 10^{-7}}{P_{d,\beta}} \]  \hspace{1cm} (14)

where \( P_{d,\beta} \) denotes the probability of being killed in the accident event. In this expression the policy factor \( \beta_i \) varies with the degree of voluntariness (see Vrijling et al, 1998). For the safety of dikes a \( \beta_i \)-value of 1.0 to 0.1 is thought to be applicable.

Another point of view concerns the assessment by society. The judgment of the societal risk due to a certain activity should be made on a national level. The determination of the socially acceptable level of risk assumes also that the accident statistics reflect the result of a social process of risk appraisal and that a standard can be derived from them. In addition to that the formula should account for risk aversion in a society. Relatively frequent small accidents are more easily accepted than one single rare accident with large consequences like a flood, although the expected number of casualties is equal for both cases. The standard deviation of the number of casualties reflects this difference. Risk aversion can be represented mathematically by adding the desired multiple \( k \) of the standard deviation to the mathematical expectation of the total number of deaths, \( E(N_{d,i}) \) before the situation is tested against the norm of \( \beta_i \times 100 \) casualties for the Netherlands:

\[ E(N_{d,i}) + k \times \sigma(N_{d,i}) < \beta_i \times 100 \]  \hspace{1cm} (15)

where: \( k \) = risk aversion index.

To determine the mathematical expectation and the standard deviation of the total number of deaths occurring annually in the context of activity \( i \), it is necessary to take into account the number of independent places \( N_{Ai} \) where the activity under consideration is carried out.

The translation of the nationally acceptable level of risk to a risk criterion for one single installation or polder where an activity takes place depends on the distribution type of the number of casualties for accidents of the activity under consideration. In order to relate the new local risk criterion to the common shape of a FN-curve, the following type is preferred:

\[ I - F_{N_{d,i}}(x) < \frac{C_i}{x^2} \text{ for all } x \geq 10 \]  \hspace{1cm} (16)

where \( x \) is the number of casualties in a year, \( F_{N_{d,i}}(x) \) is the distribution function of the number of casualties (probability of less than \( x \) casualties in a year) and \( C_i \)

Figure 5. CBA in risk-based design (Vrijling et al., 1993). is a constant, which depends on the activity \( i \) under consideration.

The problem of the acceptable level of risk can be also formulated in a way of economic optimal based on cost benefit analysis (CBA). The expenditure \( I \) for a safer system is equated with the gain made by the decreasing present value of the risk. The concept scheme is shown in Figure 5. The optimal level of safety indicated by \( P_{f_{\text{opt}}} \) corresponds to the point of minimal cost:

\[ \min(Q) = \min \left[ I(P_f) + PV(P_f \times S) \right] \]  \hspace{1cm} (17)

where \( Q \) refers to the total cost, \( PV \) is the present value operator and \( S \) refers to the total damage in case of failure.

Cost-benefit analysis in principle is limited to quantifiable aspects of the decision with all consequences of flooding are measured in monetary terms. In reality, there are several dimensions of flooding risk that can in principle be quantified, but not necessarily in monetary terms. An important aspect that falls in this group is the risk of loss of life due to flooding. Furthermore, it is observed that disastrous events become less acceptable to the general public if the magnitude of the consequences is larger. This behaviour is referred to as risk aversion.

One way to deal with monetary and non-monetary aspects of flood protection is to define constraints on the solution space, which limits the range of acceptable flooding probabilities to values that are deemed acceptable in the light of the non-monetary consequences of flooding. Observed risk aversion may be included in the definition of the constraint (Vrijling et al., 1995, 1998).

Another way to include monetary and non-monetary aspects of flood protection is by application of the Life Quality Method, proposed by Nathwani
et al., 1997. In this method, decisions concerning risk are judged using a social indicator called the Life Quality Index (LQI), which combines the effects of the decision on the life expectancy at birth and the gross domestic product per capita. From a practical point of view, this is a convenient method since the properties of the life quality model may be established from observed values of the properties of society. However, the validity of this approach model is very hard to verify.

3.2 Discussions

Reliability-based design depends on the availability of a pre-defined failure probability requirement. In a situation where acceptable safety levels are defined in regulation (e.g. pre-defined design frequency), the methods outlined in the previous section are sufficient to obtain a cost-effective design that fulfills the requirements. In some cases, the required failure probability is not yet defined. For instance the case when a design is made for a situation where no regulation is available or when an analysis is performed in a process of defining/redefining required safety levels.

From the review of risk analysis in previous section, the acceptable probability of failure can be defined by comparing the cost of protection to a characteristic value of the consequences of flooding. In a purely economic sense this leads to risk-based cost-benefit analysis. Applications in coastal engineering in the Western countries have been published by Bakker & Vrijling (1980), Burcharth et al., (1995), Voortman et al., (1998, 1999, 2001 and 2002), Vrijling et al., (1998). In risk-based design, a model is established that provides a measure of the effectiveness of the protection system as a function of its failure probability. The two main components of such a model are the cost of protection and a characteristic value of the consequences of flooding, both as a function of the probability of failure.

Actually, for Vietnamese situation of flood defences and coastal protection, the safety levels were set by design frequencies which varied by locations and importance of the elements. It is known as a fixed values and indicated in National Design Standards (e.g. for provincial sea dike is 1/20 years, river dike 1/25 years). These safety levels were selected approximately 30 years ago. There is no clear explanation of the case when a design is made for a situation where no regulation is available or when an analysis is performed in a process of defining/redefining required safety levels.

The present situation of flood defences in Vietnam is well indicated in Section 1.2. Apart from that all design are made in two dimensions without paying attention to the third one: the system length.

This section focuses on importance of the consequences of the two dimensional dike design approach. A simple example shows the influence of system length effects at Haiphong sea dikes.

The Haiphong dike ring consists of 47 km. This dike ring may be subdivided in 47 independent sections with identical length. Each section is constructed according to a design which is based on loads with return period 20 years. Suppose the probability of failure of a section truly reflects this return period: probability of failure section = $P_{f, sect} = 1/20 = 0.05$ year$^{-1}$. This probability is the minimum probability of failure of the entire dike ring, which refers to the case of full correlation of all sections. In this case all sections should fail simultaneously. However practical experience shows that all sections of a dike ring never fail simultaneously: failure occurs to the weakest section(s) only. Therefore failure of the dike ring depends on the behaviour of the weakest section. The dike ring should be conceived as a series system of $n = 47$ independent elements. The maximum probability of failure of such a series system may be calculated conform:

$$P_{f,ring} \leq 1 - (1 - P_{f, sect})^n = 1 - (1 - 0.05)^{47} = 0.9102 \quad (18)$$

This maximum probability $P_{f,ring} = 0.9102$, which is approximately equal to a return period of 1 year, applies to the hypothetical situation of perfect independence of all elements. In most cases this situation differs from reality too. Although strength parameters as crest height and geotechnical properties of a section might well be uncorrelated in the case of the chosen length scale 1000 meters, the hydraulic load will probably be more or less the same for adjacent sections. Therefore the probabilities of failure of adjacent sections are often still somehow correlated.

Taking into account the correlation due to hydraulic loads a more proper upper bound for $P_{f,ring}$ was estimated at (Vrijling & Hauer 2000):

$$P_{f,ring} \leq \frac{P_{f, sect}}{1.1} \times (1.036 + 0.064 \times n)$$  \quad (19)
where $n$ refers to the number of sections of the dike ring under consideration. It should be noted that this result belongs to a specific case, the formula above should not be conceived as a theoretical result for all kinds of situations. However the starting-points of this example are rather common and therefore the formula might well apply to a Vietnamese situation too. In that case the upper bound of the probability of failure of the dike ring in the Vietnamese example might be $P_f \leq (0.05/1.1)^*(1.036 + 0.064*47) = 0.1850$, which probability corresponds with a return period $1/0.0.185 \approx 5.5$ year. This return period might well be the true return period of failure of the Haiphong dike ring.

As far the example only concerned the possibility of overtopping of sections. Some sections might have failed earlier as a result of other failure mechanisms too. Therefore the return period of failure might be even smaller than 5.5 year. In case of a dike ring consisting of both sea dikes and river dikes and independent load characteristics of the rivers and the sea the value $P_f \approx 0.185$ might be an optimistic value. Taking it all in all the true frequency of failure of the dike ring as a whole might well exceed the design frequency of a two dimensional section by a factor 10. For the Vietnamese situation a factor of at least 5 (preferably 10) should be adopted for the choice of the design frequency.

Besides of the length effect, the true frequency of flooding depends on the total number of dike rings in Viet Nam too. The probability of flooding in a country will increase according as the number of dike rings increases. This effect may be considered in the same way as the length effect. The more dike rings in a country, the more severe the requirements should be to the safety level of each dike ring on its own.

4.2 Example of Namdinh sea defence system

4.2.1 Present situation and protection strategy

The NamDinh Province constitutes part of the Red River Delta in Northern part of Vietnam. The total length of Namdinh coastline is about 90 km suffering from severe erosion and serious damages of coastal defences. This can be considered as the representative for coastal problems in the region. The coastline is mainly protected by dikes and revetments.

The failure of the sea dikes and revetments occurs frequently due to actions of severe typhoons, strong waves in combination of high tides while their design parameters were not sufficient. Moreover due to the action of waves and currents the foreshore erosion has occurred seriously which also leads to the collapse of dikes and the revetments.

In response the central and local authorities have undertaken some efforts in order to restrain the possible adverse consequences and as future defensive measures, some sections of new sea dikes had been built. However, such efforts still remain limited to reactive and temporary measures.

The dike system is characteristically positioned as shown in Figure 6 with two defensive lines and separated section by section with sub-crossing dikes. When a breach takes place at the main dikes, the sub-crossing dikes can limit flooding areas and the second dikes will be a new first line of the system. The distance between two defensive lines is about 200 meters.

Due to budget constrains, lack of information on the sea boundary conditions and suitable design methodology as well as strategic and long-term solutions, the dike system was usually designed and constructed in poor conditions. As the consequence, the system could be destroyed once in every 10 years. Therefore the cost of dike maintenance is finally very extensive. Statistically, for maintenance of Namdinh sea dikes system it is represented nearly 95 percent of the total coastal defence budget of Vietnam.

4.2.2 Possible failure mechanisms

Namdinh sea dikes are selected in this study as the most typical sea defence system in Northern Vietnam. Possible failure mechanisms of Namdinh sea dikes are various. For more detail on failure description, see also in Mai et al., 2004. Due to lack of information and time limited, in this paper, the following dominant failure modes are discussed:

1 – overtopping; 2 – instability of slope protection; 3 – geotechnical instability of inner and outer slope; 4 – scour induced dike failure and; 5 – piping.

Hypothesis: The failure of the dike system will not occur if all dike sections, which are consecutive and forming the dike system, are stable. For a dike section the failure may occur since there will be presented one of any failure mechanisms. The fault tree of the dikes, in this case, is shown similarly on Figure 3.

4.2.3 Reliability analysis

Follows the method given above, the reliability of Namdinh sea dike system is conducted from reliability analysis of all dike sections. For each of those the consideration is taken of five given possible failure mechanisms. The reliability functions of these failure mechanisms are given in Table 2. Statistic description of related variables are listed in appendix table. Using

![Figure 6. Schematization of Namdinh sea dike system.](image-url)
Vietnam: p

Total number of sections is 30, with each section length

Total length of Namdinh sea dikes is 90 kilometers.

Table 3. Extensive description of the failure mech-

anism contributions to the total failure probability of

Table 3. Failure probability of Namdinh sea dikes.

<table>
<thead>
<tr>
<th>Failure mode\case</th>
<th>Old dikes</th>
<th>Actual dikes</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Dikes with slope protection by rock revetment:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overtop overflowing</td>
<td>0.4740</td>
<td>0.0474</td>
</tr>
<tr>
<td>Armour failure</td>
<td>0.4730</td>
<td>0.0157</td>
</tr>
<tr>
<td>Sliding Outer SI</td>
<td>3.1E-5</td>
<td>3.1E-5</td>
</tr>
<tr>
<td>Sliding inner SI</td>
<td>0.005709</td>
<td>0.005709</td>
</tr>
<tr>
<td>Piping</td>
<td>3.0E-12</td>
<td>3.0E-12</td>
</tr>
<tr>
<td>Scour depth exceed</td>
<td>0.068</td>
<td>0.068</td>
</tr>
<tr>
<td>Upper bound Pf1</td>
<td>0.96</td>
<td>0.14</td>
</tr>
<tr>
<td>b. Dikes with slope protection by block revetment:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overtop overflowing</td>
<td>0.6320</td>
<td>0.0464</td>
</tr>
<tr>
<td>Armour failure</td>
<td>0.1320</td>
<td>0.0123</td>
</tr>
<tr>
<td>Sliding Outer SI</td>
<td>3.1E-5</td>
<td>3.1E-5</td>
</tr>
<tr>
<td>Sliding inner SI</td>
<td>0.005709</td>
<td>0.005709</td>
</tr>
<tr>
<td>Piping</td>
<td>3.0E-12</td>
<td>3.0E-12</td>
</tr>
<tr>
<td>Scour depth exceed</td>
<td>0.068</td>
<td>0.068</td>
</tr>
<tr>
<td>Upper bound Pf1</td>
<td>0.78</td>
<td>0.13</td>
</tr>
</tbody>
</table>

FORM these reliability functions given the results in

Table 3. Extensive description of the failure mech-

anism, statistical load and strength boundaries and

formulation of reliability functions can be found in

May 2004.

4.2.4 Reliability-based design values

Total length of Namdinh sea dikes is 90 kilometers.

Total number of sections is 30, with each section length

of 3000 meters. Design frequency of whole system can

be obtained from the actual sea dike design standard of

Vietnam: p = 1/20 = 0.05. Applying equation (18) the

failure probability of a single section is determined:

Pfsec = 0.0186 <= 1/55.

Assuming that proportion of every failure mech-

anisms contributes to the total failure probability of

a section is similar to the previous case. The failure

probability of each failure mechanism can be derived

from Pfsec. Since acceptable probability of the fail-

ure is known, applying reliability-based approach, the

design parameter can be determined.

In this case, Pfsec = 0.0186, the design values of

interested parameters are obtained in Table 4.

5 DISCUSSIONS

Most of the existing design of sea dikes and revetments

in Vietnam were based on the old design methods

(standards), which no longer be used nowadays, and

were constructed under poor conditions of execution,

quality control, investment and management. The future

effect of boundary condition was normally not

taken into account (e.g. increase in water depth due to

foreshore erosion and morphological changes, occurrence

of scour holes, settlement of dike body and sea level

rise, etc…). As the result the design load boundary was

underestimated. The strength of the dikes is insuffi-

cient to the actual condition. Consequently, the failures

of sea dikes often occur even with lower design storms.

The failure probability of the old dikes in Nam-

dinh, which were applied old design codes, is very high

at 96%. This means that the old dike could be fault

1/0.96 ≥ 1 time per year. This is unacceptable safety

level by any safety regulations. Obviously, the dikes

were constructed from 2-dimensional design with low

design frequency (1/20 years) of which the length

effect were not taken into account. If this would be

account for, the design frequency of each dike sec-

tion should be decrease to Pfsec = 0.0186 according

to (18). It corresponds to return period of about 55

years (instead of 20 years) for Namdinh case. There-

fore the risk of the old system is increased by factor 3.

The failure probability of inundation due to dike failure is

given much higher, about p = 0.14 (see Table 3). This corresponds
This study reviews partially existing methods of reliability based approach with its specific fault tree, the case of Namdinh coastal protection, followed the example cases given interesting results. Regarding to scale of protection system, investment levels and acceptable consequences for any (or re-set) the safety levels of protection in relation to being a powerful tool supporting decision process to set function of the probability of failure. This model could be applied and need more further studies.

6 CONCLUSIONS AND RECOMMENDATIONS

This study reviews partially existing methods of reliability and risk based approaches which applied in the field of flood defences and coastal protection. The reliability analysis starts with a fault tree at the lowest level. Overall failure probability of the system is calculated at the top level of the tree. The reliability based design is an essential design tool for flood defence and coastal protection when having a pre-defined failure probability and the geometry of the structure needs to be found in order to fulfill the design requirement in combination with minimisation of the construction cost of the system. On the other hand, if the geometry of every component of the flood defence system is known and the joint probability distribution of load and strength variables is quantified, the probability of failure of the system can be found. This can be applied for technical management purpose to determination of safety levels of any existing system and to find out the weaker/weakest points of the system.

Additionally in risk-based design the effectiveness of the protection system as a function of its failure probability can be modeled. The two main components of the model are the cost of protection and a characteristic value of the consequences of flooding, both as a function of the probability of failure. This model could be a powerful tool supporting decision process to set (or re-set) the safety levels of protection in relation to investment levels and acceptable consequences for any scale of protection system.

Application of reliability and risk analysis in these example cases given interesting results. Regarding to the case of Namdinh coastal protection, followed the reliability based approach with its specific fault tree, the actual relative low safety level of the whole system were figured out. Analysis shows that the failure may occur once every year at the existing old-design dike sections and about 7 years with the actual dikes. This safety levels are unacceptable with Vietnamese actual design codes (once in every 20 years). The effect of system length to probability of failure was clearly indicated by the case of Hai Phong dike ring. From that one can realize that with 2-dimensional design (no consideration of system length) the actual safety level of whole system may reduce by factor 10 or more!

Reliability and risk analysis in the field of flood defence and coastal protection has been developed/developing effectively in some developed countries. Applying this modern approach in the same field in developing countries, therefore, should be implemented. Especially for the country where the safety levels of the actual flood protection are relatively low and safety regulations are not usually clear defined. A general framework of implementation of state of the art is thus important. Specific reliability and risk based models should be developed in those countries which should account for some developing characteristics (e.g. limited initial investment, cheaper man power, fast economic growing) and some limitations of data availability, lack of modern construction equipments, poor professional knowledge etc. This may become an interesting topic for the future researches. Beside, extensive studies on determination of sea boundaries, interaction between them and the coastal defensive system as well as descriptions of their failure mechanisms in relations to their physical processes should also be carried out.

REFERENCES


Appendix: Statistical description of related random variables.

<table>
<thead>
<tr>
<th>Not.</th>
<th>Description</th>
<th>Unit</th>
<th>Dist.</th>
<th>Mean value</th>
<th>Std. dev.</th>
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<tr>
<td>φ</td>
<td>Empirical factor -Pilarczyk</td>
<td>–</td>
<td>Norm</td>
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<td>0.5</td>
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<tr>
<td>ϕ</td>
<td>Friction angle</td>
<td>°</td>
<td>Nor nom</td>
<td>2°</td>
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<td>ρc</td>
<td>Saturated density of subsoil</td>
<td>kG/m$^3$</td>
<td>Deter</td>
<td>1800</td>
<td></td>
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<tr>
<td>γsat</td>
<td>Soil unit weight (saturated)</td>
<td>kN/m$^3$</td>
<td>Nor nom</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td>γunsat</td>
<td>Soil unit weight (unsaturated)</td>
<td>kN/m$^3$</td>
<td>Nor nom</td>
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<tr>
<td>ρw</td>
<td>Water density</td>
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<td>8.7</td>
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<td>–</td>
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<td>a</td>
<td>Safety free board</td>
<td>–</td>
<td>Norm</td>
<td>0.5</td>
<td>0.05</td>
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<tr>
<td>A</td>
<td>Wave loads on outer slope</td>
<td>kN</td>
<td>Nor nom</td>
<td>50</td>
<td></td>
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<tr>
<td>b</td>
<td>Power factor</td>
<td>–</td>
<td>Norm</td>
<td>0.65</td>
<td>0.15</td>
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<td>B</td>
<td>Transportation load on dike’s crest</td>
<td>kN</td>
<td>Nor nom</td>
<td>100</td>
<td>10</td>
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<tr>
<td>C</td>
<td>Cohesion</td>
<td>kN/m$^2$</td>
<td>Nor nom</td>
<td>5%</td>
<td></td>
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<tr>
<td>cB</td>
<td>Bligh constant</td>
<td>–</td>
<td>Deter</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>cosα</td>
<td>Cosine of slope angle</td>
<td>–</td>
<td>Norm</td>
<td>0.97</td>
<td>5%</td>
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<tr>
<td>D</td>
<td>Required block thickness/rock size</td>
<td>m</td>
<td>Deter</td>
<td>nom</td>
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<tr>
<td>d</td>
<td>Thickness of the top layer</td>
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<td>Norm</td>
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<td>Hs</td>
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<td>Lognor</td>
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<td>k</td>
<td>Permeability</td>
<td>m/s</td>
<td>Deter. nom</td>
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<td>KΔ</td>
<td>Reduction factor for slope roughness</td>
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<td>Nom</td>
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<td>Kp</td>
<td>Transformed coefficient % of wave exceedance</td>
<td>m</td>
<td>deter</td>
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<td>Lk</td>
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<td>High tidal level (+MSL)</td>
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<td>Deter</td>
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<td>Permeability factor</td>
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<td>S</td>
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<td>S0</td>
<td>Wave steepness</td>
<td>–</td>
<td>Deter</td>
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<td>tanα</td>
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<td>Tm</td>
<td>Wave mean period</td>
<td>s</td>
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</table>

**nom**: normative value; **Norm**: Normal distributed; **Weil**: Weibull distributed; **Deter**: Deterministics; **Lognor**: lognormal distributed.