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Probabilistic investigation of failure mechanisms of coastal flood defence structures

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ABSTRACT: This paper aims at a probabilistic investigation of various possible failure mechanisms which are often experienced by coastal flood defence structures under the impact of sea loads. The factors that affect structure performance are usually diverse. In order to undertake an effective assessment of the overall reliability of the coastal flood defence it is essential to have a thorough knowledge and understanding of all possible failure modes and their contribution to the total failure probability of the whole system. All failure mechanism of these structures often relate with various stochastic variables. Sensitivity analysis is therefore necessary to perform in order to see the importance of these variables for each certain failure mode. In this paper, with the use of probabilistic design approaches at level III and level II in combination with fault tree analysis by Monte Carlo simulation, efforts will be made to provide more insight into these issues. Application of these methods is implemented with a case study of coastal dike rings in Vietnam.

KEYWORDS: Coastal defences, sea dikes, risk analysis, reliability assessment, failure modes, failure mechanism of coastal structures, sensitivity analysis of stochastic variable, fault trees.

1 Introduction

Various coastal flood defence structures react differently under the impact of sea loads. The factors that affect structure performance are usually varied. In order to undertake an effective assessment of the overall reliability of a coastal flood defence it is essential to have a thorough knowledge and understanding of all possible failure modes. Over recent years considerable effort has been devoted to improving our knowledge of how defences fail. However, gaps in knowledge still remain. The appropriate characterisation of failure mechanisms of coastal flood defences is a key component in effective reliability analysis and flood risk assessment. The quantification of these failure mechanisms is facilitated by a number of methods ranging from indicative equations to more physical process-based models. Currently, there is an increasing interest to quantify the reliability of coastal flood defences using probabilistic approaches, i.e. reliability assessment with the outcome is

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in terms of the failure probability of a structural component and of the whole defence system. A central concept in probabilistic reliability-based assessment of coastal flood defences is a limit state equation. This equation links the performance target to the processes that lead to failure to fulfil that target. Starting with a proper definition of the coastal flood defence functions and its functional requirements, the structures is considered to be failed since its performance does not meet its functional requirements. The structure system can fail in different ways, which are normally referred to various failure modes. The process of which the structure is failed by a certain failure mode is called failure mechanism. The failure mechanism can be described by a reliability function, which is based on the physical process of the failure and represented by a combination between characteristic strength and the load of the structure in the form of limit state equation: $Z= R–S$, in which both R and S comprise a number of uncertainty/stochastic variables. At a system level, for instance a dike ring, the system can be discomposed by its system components. Each of those components can be failed in various failure modes. The overall outcome of system reliability analysis is considered as the combination of failure probability of all failure modes and of all system components. The question of which failure modes are the most dominant to the overall failure of the system is very important. Besides that finding the weakest component of the system is also necessary for design, maintenance and management purposes. Influences of load and strength variables to the failure are important to know which parameters contribute the most to the occurrence of failures. In this paper, efforts will be made to give appropriated answers for these previous issues with an application of coastal dikes in Vietnam.

2 Brief overview of probabilistic approach in coastal flood defences

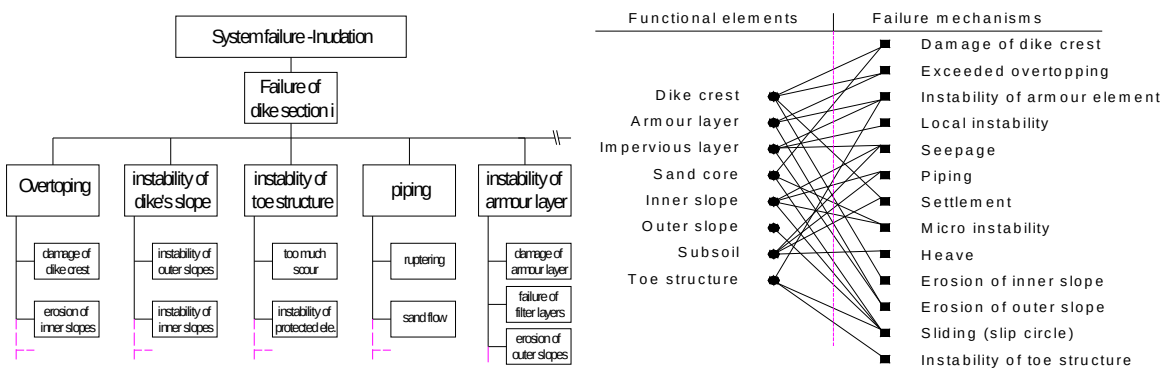
In the low-lying coastal regions the coastal flood defence system is important in protecting hinter lands from flooding. Typically this system contains sea dikes, dunes, estuarine levees, sluices, pumping stations, etc. Sea dikes are usually the most common and important components of the system. In general, design of these coastal structures is still based on a deterministic approach which often not accounts for uncertainties of sea boundary and the resistance. The design hydraulic boundary condition of such structures is normally based on the averaged water depth front with a certain design waves under a design extreme condition of the actual cross shore profile. However, coastal dikes are usually under impacts of many coastal processes and natural phenomena which happens randomly both in time, space, their intensity and amplitude, thus the boundary situation can change subsequently compare to the design situation. Normally the dike crest exceeds the design water level by some measure, thus the probability of overtopping is smaller than the design frequency. However, as learn from practices, 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 dike crossing structures (e.g. sluices, gates, ship logs...) are not functioned well in time before the high water moment (i.e see Katrina case in US by W. Kanning et al 2007; Damrey case in Vietnam by C. Mai Van et al 2006a, ...). Furthermore the length of a dike ring has a considerable influence e.g. with a multiple section dike system of more than 90km long, the probability of dike failure may increase by factor 3 to 10 (see C.Mai Van et al

2006b). A chain is as strong as the weakest link. So a single weak spot determines the actual safety of an entire dike ring (see Vrijling, J.K.,2001).

Since the last few decades probabilistic design concepts have been increasingly proposed and applied to the field of water defences (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 to take into account these uncertainties of the input parameters and to treat them as the random variables. Moreover 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. 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 by various failure modes.

Due to the action of sea boundaries and their uncertainties the failure of sea dikes may presented in various mechanisms. Some of the most possible failure mechanisms are: overtopping, instability of slope protected element, sliding of outer and/or inner slope, piping, erosion of outer and/or inner slopes, dike's toe instability, etc.. The relation between the failure mechanisms in a dike section and the unwanted consequence flooding can be schematised with a fault-tree (see Figure 1.a).

On the other hand these failures of sea dikes can be presented in relation to their functional elements (see Figure 1.b). The failure is considered to occur if the functional component is not fulfil its pre-defined functional requirements.



a) Consecutive failure mechanisms b) Functional element in relation to failure mechanism

Figure 1 Example of failure mechanisms at the sea dikes

Failure mechanisms can also be described by a “failure matrix” which shows clearly relations between types of structures, functional components and types of driven loads leading to failures. The “failure matrix”, was first proposed in FLOODSite’s approach, has been developed as a means of referencing information on each failure mechanism of flood defences. Simplify version can be derived for coastal flood defence structures and is presented in Figure 2.

Coastal Flood Defence\ Failure Mechanisms		1 Water level difference across structure	1.1 Erosion of surface by overflow	1.2 slip / slide	1.3 Piping and/or internal erosion	1.4 Erosion of landward or downstream slope	1.5 Erosion at transition between structures	1.6 Crest level too low - overflow	2 Wave loading	2.1 Erosion of seaward face / slope	2.2 Bulk displacement (slip / overturning) of defence / seaward element	2.3 Local surface failure or element displacement	2.4 Erosion of landward face / slope	2.5 Crest level too low - overtopping	3 Lateral Flow Velocity	3.1 Erosion (scour) of bed or bank
A	Foreshores, dunes and banks															
Aa	Sand Dunes		x							x			x			
Ab	Gravel/ rock beach									x						
B	Sea dikes and revetments															
Ba	Sea dikes (sand/clay)		x	x	x	x	x	x		x		x	x	x		x
Bc	Revetment (rock/block)		x		x					x		x	x			x
C	Walls															
Ca	Mass concrete vertical									x	x					
Cb	Crown or parapet wall\										x					
D	Point Structures															
Da	Barriers			x												
Db	Sluices, gates			x												

Notation: x means failure

Figure 2 Simplified failure matrixes developed for coastal flood defence structures

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 i^{th} of the system can be simply written by Bakker & Vrijling (1980):

$$Z_i=R_i-S_i \tag{1}$$

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 $f_{Zi}(X)$.Then the probability of failure of the element i^{th} which has the limit state function Z_i is:

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

In reality Z_i may be a function of number of stochastic variables $X_1, X_2,...,X_n$, as both the "load", S_i, and the "strength", R_i, and may depend on more than one variable. The failure probabilities of all failure mechanisms can be determined using the methods of the modern reliability theory by different approach levels i.e. level III with numerical integration and Monte Carlo simulation, level II with first order second moment approximation (FORM). The later is more popular since it requires not much calculation time and relatively simpler. In the level II with FORM analysis, the failure surface $Z(X_j)$ =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:

$$Z_{lin} = Z(X_1^*, X_2^*, ...X_n^*) + \sum_{j=1}^{n}(X_j - X_j^*) * \left(\frac{\partial Z}{\partial X}\right)_{X_j = X_j^*} = 0 \tag{3}$$

Z_i^{Lin}: Linearized reliability function of Z_i in $\{X_j^*\}$;

$\left(\frac{\partial Z_i}{\partial X}\right)_{Xj=Xj*}$ is 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_i^{Lin} are:

$$\mu(Z_i^{Lin}) = Z_i(X_1^*, X_2^*, ...X_n^*) + \sum_{j=1}^{n}(\mu_{X_j} - X_j^*) * \left(\frac{\partial Z}{\partial X}\right)_{X_j = X_j^*} \tag{4}$$

$$\sigma^2_{(Z_i^{Lin})} = \sum_{j=1}^{n} \sigma^2_{X_j} * \left(\frac{\partial Z_i}{\partial X}\right)^2_{X_j = X_j^*} ... \tag{5}$$

If mean values $X_1^* = \mu_{(Xj)}, \ldots, X_n^* = \mu_{(Xn)}$ 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:

$$X_j^* = \mu_{X_j} - \alpha_j . \beta . \sigma_{X_j} \tag{6}$$

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

$$\beta = \frac{\mu(Z_{Lin})}{\sigma(Z_{Lin})}; \alpha_j = \frac{\sigma(Xj)}{\sigma(Z^{Lin}{}_i)} * \frac{\partial Z_i}{\partial X_j} \tag{7}$$

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

$$g(z,X) = R(z,X) - S(z,X) \tag{8}$$

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 $f_{\bar{x}}(X)$ denotes the joint probability distribution of random input boundaries. The probability of occurrence of every failure mode is given by:

$$P_f(z, X) = P(g(z, X) < 0) = \int_{g(z,X),0} f_{\bar{x}}(X) dX \tag{9}$$

The overall failure probability of a dike section number i^{th} is given by:

$$P_{failure}^{sec.i} = P\ (Z_1<0;\ Z_2<0;\ ...\ Z_i<0;...\ Z_m<0) \tag{10}$$

where (Z_1<0; Z_2<0; ... Z_i<0;... Z_m<0) denotes at least one of *m* failure mechanisms occurs;
All calculated probabilities of failure can be presented in a form of Table 1.

Table 1. Example table of overall probability of flooding.

section	overtopping	piping	etc.	total
dike 1.1	$p_{1.1}$ (overtopp.)	$p_{1.1}$ (piping)	$p_{1.1}$(etc.)	$p_{1.1}$(all)
dike 1.2	$p_{1.2}$ (overtopp.)	$p_{1.2}$(piping)	$p_{1.2}$(etc.)	$p_{1.2}$(all)
etc.	...	...	...	...
dune	p_{dune}(overtop.)	p_{dune}(piping)	p_{dune}(etc.)	p_{dune}(all)
sluice	p_{sluice}(overtop.)	p_{sluice}(piping)	p_{sluice}(etc.)	p_{sluice}(all)
total	p_{all}(overtop.)	p_{all}(piping)	p_{all}(etc.)	p_{all}(all)

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.

3 Physical process-based probabilistic descriptions of failure mechanisms

Description of failure mechanisms is an important component in the design and risk analysis of coastal flood defence system. In order to determine overall safety in probabilistic design and reliability analysis of coastal structures one of key components is the related failure mechanisms and their reliability equations (as we call in this paper from now on as “limit state equation”). In this section number of popular failure modes for coastal flood defence structures will be analysed and described. The probabilistic description of failure mechanisms in the form of limit state function Z=R-S will be presented based on the physical process-based limit state equation, which often used as limit state equation in conventional design approach. The failure mechanisms will be discussed including: Overflowing, excessive wave overtopping, instability of amour protected elements, macro instability of dike slopes, instability of toe structures and piping.

(1) Overflowing of a dike: This failure mechanism occurs when the actual water level of the sea exceeds the crest level of the dikes. The dominant load which drives this failure mode are sea maximum possible high sea water levels which often are a combination of high astronomical tides, storm surge level, gust pump, increased water level due to long waves such as seiches, and wave run-up due to short waves acting on slope of the dikes.

Limit state equation:

$$Z=Z_{dc}-Z_{wl} \tag{11}$$

where Z_{dc} = actual dike crest level [m]; and Z_{wl}= actual high sea water level [m]

Loading equation:

$$Z_{wl}=MHWL+Z_{Surge}+Z_{Seiches}+Z_{gust}+Z_{run\text{-}up2\%} \tag{12}$$

Resistance (strength) equation: Actual dike crest level Z_{dc} can be obtaining from monitoring measurement data/ design dike crest level.

(2) Excessive wave overtopping at a dike: Sea dikes is considered functional failure when total wave overtopped discharge exceeds the design value in which by excessive wave overtopping, either by reference to the receiving area or by virtue of failing to deliver adequate resistance to hazards.

Reliability equation: Wave overtopping rate exceeds admissible rate of water behind the sea defence. The "load" is the actual wave overtopping rate; the "strength" is a critical rate which is higher than the limit for inundation. The reliability function is expressed by:

$$Z = q_{adm} - q \tag{13}$$

where: q_{adm} = admissible wave overtopping rate [l/(s·m)]; q = actual wave overtopping rate [l/(s·m)]

*Loading equations:*Overtopping discharge (J. Van der Meer et al, 1991):

$$\frac{q}{\sqrt{2gH_s^3}} = Q_0 \cdot \exp\left(-5{,}5\frac{R_c}{z_{98}}\right) \text{ where: } Q_0 = \begin{cases} 0{,}038\,\xi_d & \text{for } \xi_d < 2{,}0 \\ \left(0{,}096 - \dfrac{0{,}160}{\xi_d^3}\right) & \text{for } \xi_d \geq 2{,}0 \end{cases} \tag{14}$$

$$\xi_d = \frac{\tan\alpha}{\sqrt{H_s / L_{0m}}} \text{ and } L_{0m} = \frac{gT_m^2}{2\pi} \tag{15}$$

*Resistance (strength) equations:*Admissible wave overtopping rate e.g. q_{adm}= 10 l/s/m for well protected inner side or 1 l/s/m for partly-protected inner side. More detail information can be found in CUR169.

Parameter definitions:z_{98}: wave run-up height at slope (2% probability of exceedence) [m]; H_s: significant wave height at toe of dike [m]; T_m: mean wave period of incident waves [s]; α: outer slope [°]; L0m: wave length at toe of dike relating to T_m [m]; R_c : crest freeboard [m].

(3) Instability of amour unit: Under the attack of waves and current the amour layer can be failure due to instability of protected elements. For coastal structures, e.g. revetment to protect outer parts of sea dikes, this failure mechanism often occur under storm condition which mainly induced by action of waves.

Reliability function: $Z= (H_s/\Delta D)_S-(H_s/\Delta D)_R$ (16)

Loading equation: $(H_s/\Delta D)_S$ is the required stability number which depends upon the boundary condition and the applied formula. For sea dike revetment, this can be determined based on Van der Meer's and/or Pilarczyk's formulae.

$$\text{Van der Meer's: } \frac{H_s}{\Delta D_{n50}} = A * P^B \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \text{; Pilarczyk's: } \frac{H_s}{D_{50}\Delta} = \psi_u \frac{\varphi}{\xi_{op}^{\ b}} \tag{17}$$

Resistance equation: $(H_s/\Delta D)_R$, is design stability number corresponds to the actual/design size of protected elements (e.g. characteristic diameter of protected element or thickness of amour layer).

Parameter definition.

H_s is significant wave height at local condition

φ stability factor; ψ_u system parameter; P: Notional permeability factor, for Namdinh revetment, the armour layer lies on granular filter layer, underneath is clay layer, so P should take value of 0.1; N: number of waves; S: damage level; A, B, b are model factors.

(4) Toe foot instability due to local scour and beach erosion: Erosion of toe of sea dike protection leading to toe failure which may be due to a) localized scour along the toe of the revetment, b) general erosion of the bed or, c) long-term degradation of the bed.

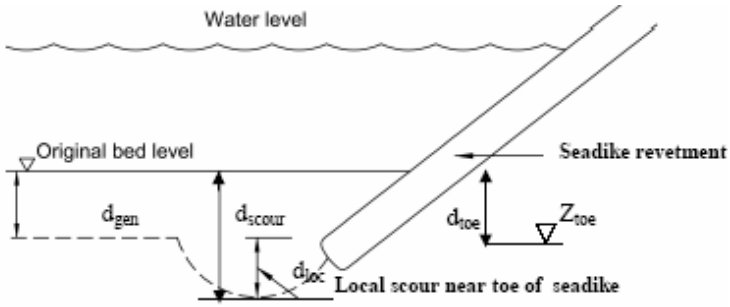


Figure 3 Scour and erosion induced dike's toe failures

*Limit state equation:*The reliability function is expressed in terms of the depth of erosion of the bed at the toe of the dikes. This may be due to localised scour along the toe of the revetment, general erosion of the sea bed and long-term erosion of the beach:

$$Z=d_{toe}-d_{scour} \tag{18}$$

$d_{toe::}$ Protected depth of toe, determine from original bed level to the lowest point of the toe structure.
$d_{scour:}$ total scour depth in front of the toe structure, it includes general erosion, long term degradation of the bed and localised scour along the toe structure.
Loading equations:

$$d_{scour}=d_{gen}+d_{loc} \tag{19}$$

Resistance (strength) equations:

$$d_{toe}=Z_o-Z_{toe} \tag{20}$$

Parameter definitions: d_{gen}: can be determined by using numerical models; d_{loc}: localised scour along the toe structure can be determined by Sumer & Fredsoe, 2001; Z_o is determined from design profile/ actual measurement; Z_{toe} is the possible lowest point of the toe structure; Z_{toe}, level of toe of protection, taking into account the deployment of any falling or launching apron (the lowest point of the toe structure).

(5) Piping: Piping occurs under the dike due to the erosive action of seepage flow which causes the continuous transport of soil particles. The physical based failure mechanism of piping was described in Figure 4, (see also CUR/TAW 1990).

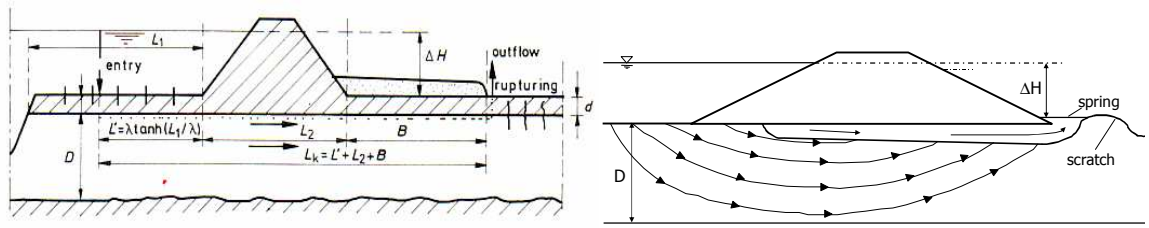


Figure 4 Piping at a dike (CUR 141, 1990).

*Limit state function:*The failure mechanism of piping occurs when two conditions must be satisfied:

- The clay layer under the dike must be ruptured (1)
- Continuous transport of sand must take place (2)

Reliability function 1: The rupture of clay layer occurs when the water pressure caused by high water level is higher than the wet density of the clay layer. So the reliability function that follows from the first condition is:

$$Z_1 = \rho_c * g * d - \rho_w * g * \Delta H \tag{21}$$

Reliability function 2: Based on the Bligh's criterion in the reliability function of piping is:

$$Z_2 = m * L_t / c - \Delta H \tag{22}$$

Parameter definitions: ρ_c is density of the wet clay;ρ_w is density of water; g is gravity acceleration; d is the thickness of the clay layer between bottom of the dike and sand layer; ΔH is the difference in water levels between sea side and inland; $L_t = L' + L_2 + B + d$; $c = c_B$ (constant depending on soil type, according to Bligh); ΔH is the difference in water levels between sea side and inland; L'; L_2; B defined as shown in Figure 4; m is a model factor, taking into account the scatter in empirical observations.

(6) Macro instability of dike's slopes: Macro instability in outer face of dike may be initiated by (rapid) draw-down of water level on outer face of the dike, when material properties may altered in time / space. While macro instability in inner face may be driven by high water level and infiltration over a long period, or by water penetration into tension cracks at crest or into surface of inner face.

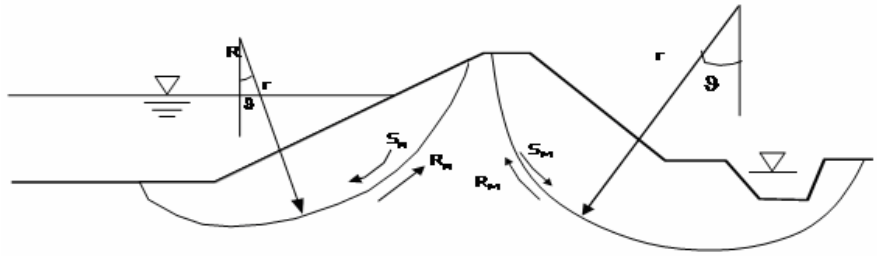


Figure 5 Geotechnical instability of dike slopes (slip circle approach)

*Reliability equation:*The reliability function is expressed by:

$$Z = \sum R_M - \sum S_M \tag{23}$$

where: ΣR_M = sum of resisting moments of single segments [kNm]; ΣS_M = sum of driving moments of single segments [kNm].
Loading equations:

$$\sum S_M = r \sum G_i \cdot \sin \vartheta_i \tag{24}$$

with weight of single segment:

$$G_i = \gamma_i \cdot A \tag{25}$$

Resistance (strength) equations:

$$\sum R_M = r \sum T_i = r \sum \frac{(G_i - u_i b_i) \cdot \tan \varphi_i + c_i \cdot b_i}{\cos \vartheta_i + \frac{1}{\eta} \cdot \tan \varphi_i \cdot \sin \vartheta_i} \tag{26}$$

Parameter definitions: G: mass force of segment [kN/m^2]; T: shear resistance in cracking gap [kN/m^2]; H: deterministic safety coefficient [-]; b: width of segment [m]; h: height of segment [m]; A: area of segment [m^2]; u: pore water pressure at segment [kN/m^2]; c: cohesion at segment [kN/m^2]; r: radius of slip circle [m]; γ_i: volume weight of single soil segment [kN/m^3]; φ: internal friction angle [°]; θ: direction angle of segments [°];

By applying similar way, other limit state function can be build up for other failure mechanism. More possible failure mechanisms of coastal flood defence structures and their limit state functions are summarized in table.

Table 3 Sumary of LME of indicated failure mechanisms

Failure mechanism	Probabilistic description- Limit state equation
Overflowing	Zc- (MHWL+Surge+S.L.Rise+$Z_{2\%)}$
Excessive wave overtopping	Z=[q]-0.06*g_b*irri*exp(-4.7*R_c/H_s/irri/g_b/g_f/g_a/g_v)*(g*(0.9*Hs)^3)$^{0.5}$
Instability of armour unit	Z={8.7*$P^{0.18}$*(S/N0.5)0.2*(tanα/SQRT(S0)) -0.5 }-{Hs/Δ/D} or Z={ϕ*Δ*D}-Hs*(tanα/SQRT(S_0))b/cosα
Geo instability of outer slope	$Z = \sum R_M - \sum S_M$
Geo instability of inner slope	
Instability of toe protected element	Z =A*(2+6.2*(h_t/h)$^{2.7}$)-Hs/Δ/D
Excessive toe erosion	Z=$h_{protected} - h_{scour\ holes}$
Overall instability of toe structure	$Z = \sum R_M - \sum S_M$
Piping condition 1	Z_1= ρc*g*d-ρw*g*ΔH
Piping condition 2	Z_2= m*Lt/c - ΔH

4 Reliability based sensitivity analysis

4.1 Introduction of study case and input data.

A case study selected is HaiHau sea dikes belong to Namdinh coastal defence system in Namdinh province, Vietnam. Total length of the Haihau sea defence system includes around 32 kilometres of sea dikes and 14 dike crossing structures i.e. sluices and pumping stations. Due to Damrey Typhoon 2005 several dikes section of Namdinh were breached, including some Haihau seadike sections (C.Mai Van et al 2006). In attempt of rehabilitation of the sea dike system, a new design cross section was introduced by MARD, Haihau is selected as one of pilot locations. Composition and dimension of the design cross section is shown on Figure 6. This study will focus on analysis of possible failure mechanisms of the sea dike section with application of the new cross section only and point structures are therefore excluded.

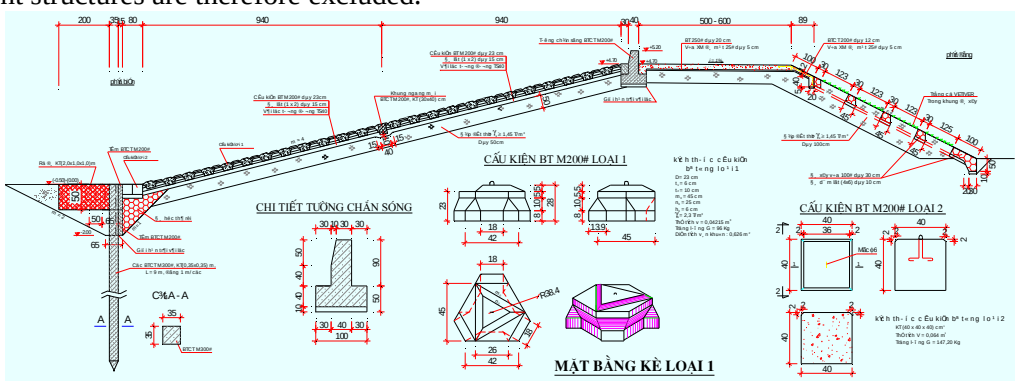


Figure 6 New design cross section of Hauhau seadikes

4.2 Failure modes and failure probability

Follows the method given above, the reliability of Namdinh sea dike system is conducted from reliability analysis. All possible failure mechanisms in previous section will be analysed for Hai Hau case based on given above limit state functions. Using level III method Monte Carlo simulation the failure probability of possible failure modes are tabulated in Table 4.These results will be used as input of fault-tree analysis in the next section.

Table 4 Failure probability vs. failure mode

F.M	Description	Failure probability	Absolute contribution	important
a	Overflowing	3.81E-03	3.81E-03	12.80%
b	Excessive wave overtopping	1.30E-02	1.30E-02	43.69%
c	Instability of armour unit	6.25E-03	6.25E-03	21.00%
d	Geo instability of outer slope	3.10E-05	3.10E-05	0.10%
e	Geo instability of inner slope	5.70E-03	5.70E-03	19.15%
f	Instability of toe elements	1.10E-03	1.10E-03	3.70%

g	Excessive toe erosion	1.89E-04	1.89E-04	0.64%
h	Overall instab. of toe structure	1.00E-02	1.89E-04	0.64%
i	Piping condition 1	6.57E-04	5.91E-15	0.00%
j	Piping condition 2	9.00E-12	5.91E-15	0.00%

4.3 Fault trees analysis

4.3.1 Basic concept of fault trees analysis

A fault tree is formed of events often described by binary (Boolean) variables (the event occurs or not) and related by logical functions, popularly OR and AND. Graphically, these logical functions are represented by Boolean "gates". The output of a gate (event represented immediately above the gate) is exactly equal to the Boolean function of the inputs of the gate (events represented immediately below). Each input event can be the result of a logical function of a set of events down to the point where all inputs are primary events that cannot be practically analysed any further.

Fault trees are constructed by using inductive logic by identifying a top event - failure of all or part of the system - and sequentially identifying unions or intersections of events that entirely describe each successive binary variable, see Paté-Cornell (1984).

Finally, what the fault tree allows one to obtain is a logical identity between the "top event" and a set of primary events. Then, on the basis of that identity, one can compute the probability of the top event as a function of the probabilities of the primary events. Phenomena that are not necessarily part of a failure scenario, but increase the probability of failure of the system, can be introduced at that stage. The probability of the top event can be computed conditional on the occurrence of these events at different "levels" if it is appropriate.

Actually two methods often use in fault trees analysis are numerical probability calculations and Monte Carlo Simulations. The earlier uses a method of direct computation from the probabilities of the primary events. The method depends upon the logically reduced tree, but the precision of the result does not. This relies on the number of terms calculated in the expression for the top level probability. The complexity of this calculation arises because the same primary event may occur in several places in a fault tree or, in other words, the component probabilities in the tree are not independent.

The principle behind the Monte Carlo methodology is to simulate occurrences of the primary events (component failures), using a random number generator. For each trial, each primary event is simulated by generating a (pseudo-)random real number in the range 0 to 1 inclusive. If this number is less than or equal to the probability of the primary event, the event is deemed to have occurred and its value is set to TRUE. Otherwise it is deemed not to have occurred and its value is set to FALSE. The fault tree is then evaluated with these values for the primary events to see if the top event occurs (system failure). The number of top event occurrences is stored, together with the corresponding failure mode (the list of primary events which occurred to cause the top event). The data is then used to obtain both the top-level probability and the probabilities of individual cut sets.

Further details on principal knowledge on fault trees can be found in Modarres (1993), Schneider (1994), Stewart & Melchers (1997), Barlow (1998). In this study method of Monte Carlo simulation, which is deployed in OpenFTA[4], is used for analysis.

4.3.2 Application of fault-trees analysis

Given failure probability of each failure mechanism, the following questions may arise: what would be the overall failure probability? and which failure mode contributes the most to the overall failure probability? In this section these two questions would be answered based on fault-tree analysis with Monte Carlo simulation.
Given information of failure modes and it mechanisms the fault trees can be constructed for Hai Hau sea dikes is in Figure 7. The primary event is drawn by a circle associated with its failure probability. Top event and intermediate top event are described by rectangular shape. Gate "AND" and "OR" is used to link between related events.

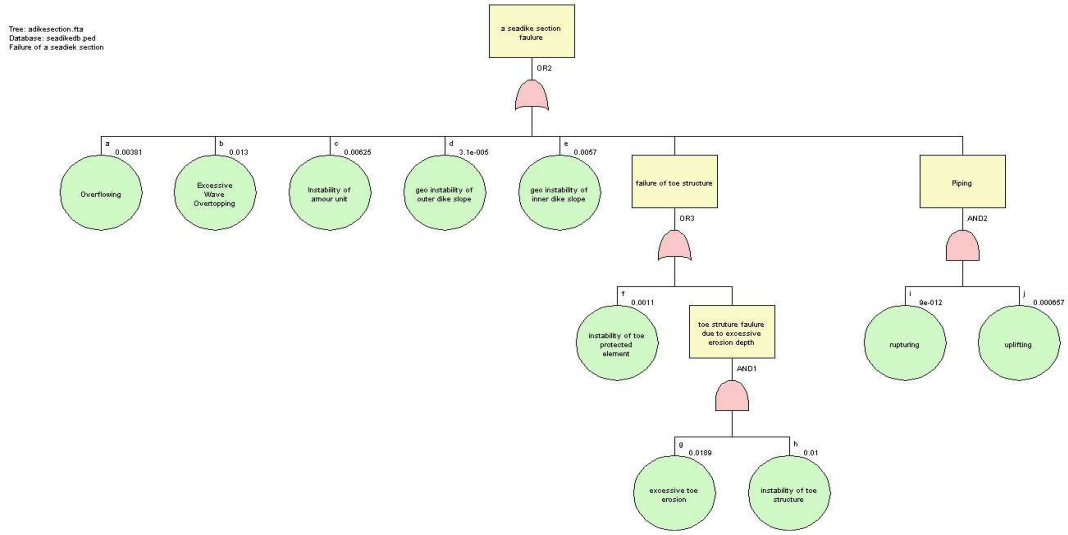


Figure 7 Construction of fault trees and assigning failure probabilities of basic events

Using Monte Carlo method with 10^7 simulations provided results as summary below.
As can be seen from the results: Number of failure of each primary event and its corresponding contribution to the total failure probability was well indicated. Within 10^7 simulations the primary event label a, overtopping mode, occurs 224483 times and this mode contribute the most (43.75%) to failure of the top event. Instability of amour layer (event label c) also happens relatively high which has 21% of the influences. Overflowing and geotechnical instability of inner slope contribute less but considerable amount. Piping, geotechnical instability of outer slope and toe foot instability modes

[4] OpenFTA free and open source software for fault trees drawing and analysis provided by Formal Software Construction (FSC) Ltd, Cardiff, Wales, United Kingdom

are likely not occurred. The absolute failure probability of piping and toe instability (intermediate top events) are 5.91×10^{-15} and 1.9×10^{-4} respectively. The overall failure probability of top event, dike section failure, is 2.98×10^{-2} with lower and upper bound of $-/+4.1 \times 10^{-5}$.

```
Monte Carlo Simulation
======================
Tree   : adikesection.fta
Time   : Wed Mar 07 17:59:47 2007
Note: Only runs with at least one component failure are simulated
Number of primary events  = 10
Number of tests           = 1000000
Unit Time span used       = 1.000000
Number of system failures = 513081
Probability of at least   = 5.805508E-002  ( exact )
one component failure
Probability of top event  = 2.978696E-002  ( +/- 4.158467E-005 )
Compressed result:
Rank   Failure mode         Failures  Estimated Probability    Importance
  1    d                    564       3.274307E-005 ( +/- 1.378732E-006 )   0.11%
  2    f                    18972     1.101421E-003 ( +/- 7.996442E-006 )   3.70%
  3    a                    65690     3.813638E-003 ( +/- 1.487955E-005 )  12.80%
  4    e                    97855     5.680980E-003 ( +/- 1.816067E-005 )  19.07%
  5    c                    107759    6.255958E-003 ( +/- 1.905755E-005 )  21.00%
  6    b                    224483    1.303238E-002 ( +/- 2.750629E-005 )  43.75%
  7    g h                  3345      1.941943E-004 ( +/- 3.357672E-006 )   0.65%
```

4.4 Sensitivity analysis of stochastic variables

The failure probability of the whole system can be reduced by reducing the failure probability of every individual failure mode. In order to archive this increasing in the strength parameters or reducing the loads of considered failure modes is appropriated. For this purpose the sensitivity analysis of related variable is necessary to see which variables contributing/ take the most influent to the failure probability of each individual failure mode.

With use of FORM analysis, values of influenced factor, α_i, of related stochastic variable X_i of failure mode which has LSF $Z_i=R_i-S_i$ will be given by equation (7). The absolute contribution of stochastic variable X_i to Z_i will be α_i^2 and within a single failure mode the total absolute contribution of all related stochastic variables is equal to one. Influences of each stochastic variable to the total variance of reliability function of given failure modes are presented graphically from Figure 7 a-h.

With overflowing and overtopping, it seems that wave height is the most dominant load parameter leading to the failure. Design water level is another important load variable. It is physically and practically relevant because under storm condition waves often in combination with high storm surge and sometime high tides acting on the coastal dikes resulting on large wave run-up height and much wave overtopped water.

Two different stability criteria of amour layer, Van der Meer's and Pilarczyk's, were applied however similar contributions of related variables to the failure mode are found. The influences of model factors/ parameters are almost by 50% to the failure mode probability (variable **model** in Van de Meer's, **b** and **Phi** in Pilarczyk's criteria). Wave height contributes 41.72% to total failure according to Pilarczyk criteria and 18% by Van der Meer formula.

Analysis of other failure modes gives the result also shown in Figure 7.

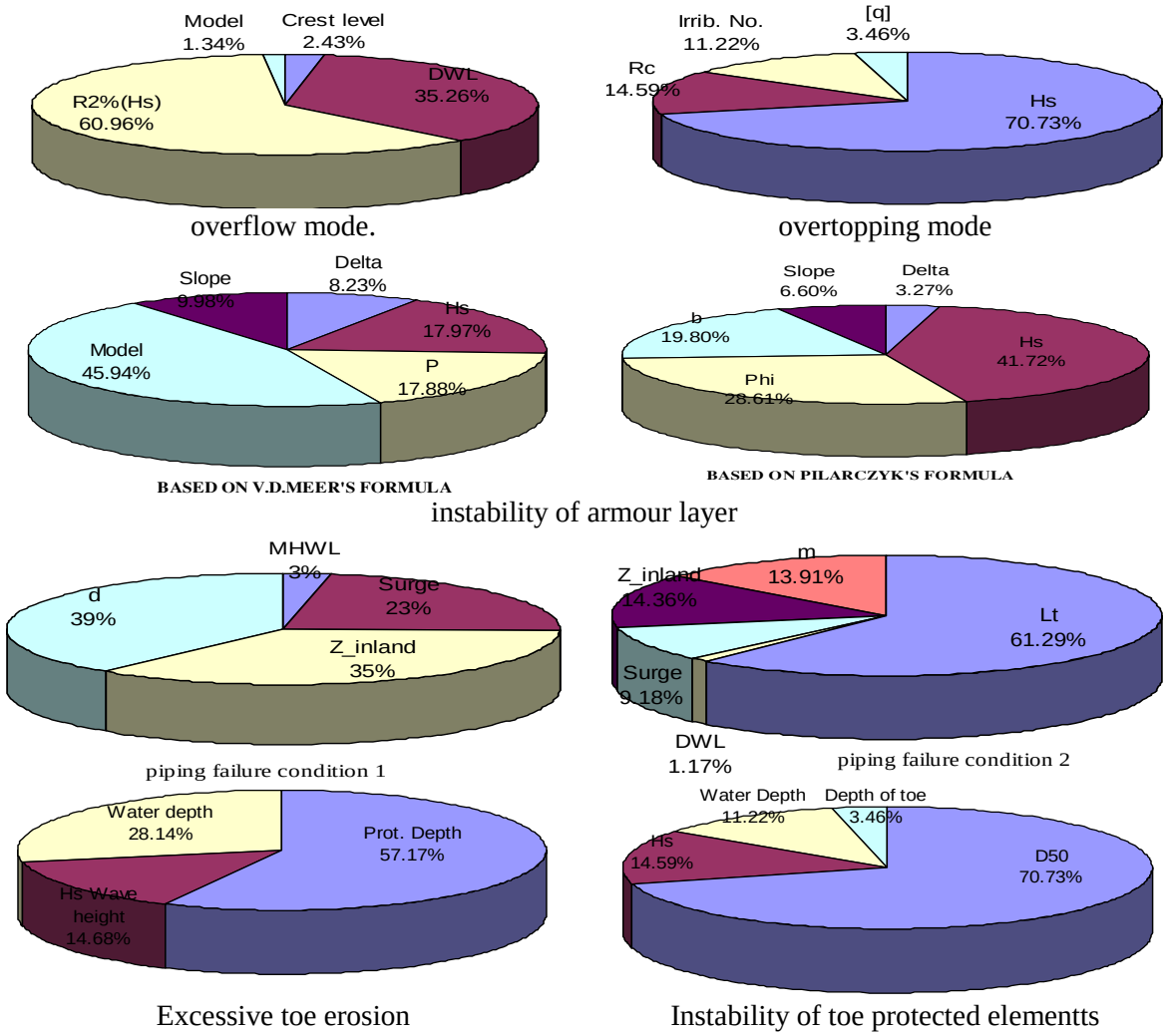


Figure 7 Influences of related stochastic variable to the failure mechanisms of sea dikes

In combination with result from fault trees analysis, it is possible to determine the contribution of main loads to the overall probability of dike section failure given an assumption that the occurrence of failure modes is statistically independent. Findings are presented on Figure 8. Wave height seems to be the most dominant loads (41 % of contribution relatively) which cause sea dike failure.

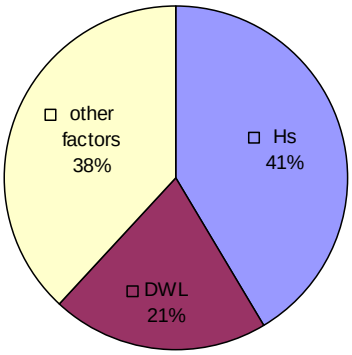


Figure 8 The importance of hydraulic loads

5 Discussions and recommendations

Probabilistic design approach is a powerful tool in reliability analysis of coastal flood defences. This allows us to determine the true probability of the component failures and of the whole system. By introducing simple failure probability table (Table 3) 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 α from FORM analysis.

This study is just focussed on sea dikes and revetment with one section. Further detail analyses should be performed for a whole sea defence system which often contains more components other than just dikes and revetments. Application of methods can also be done for other types of coastal structure system by similar approach. Nevertheless, application of the methods for the case of Hai Hau sea dikes in Namdinh province, Vietnam gave us interesting results and allows the following remarks:

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%. Overflowing and geotechnical instability of inner slope contribute less but considerable amount (12 and 19 %, respectively). Piping, geotechnical instability of outer slope and toe foot instability modes are likely not to occur (5.91×10^{-15} and 1.9×10^{-4} respectively). The overall failure probability of top event, dike section failure, is 2.98×10^{-2} with lower and upper bound of $-/+4.1 \times 10^{-5}$.

Because of high occurrences of wave overtopping, well protection of upper part of outer slope, dike crest, and inner dike slopes must be given. Transition between these parts should also be well treated. Design wave height and design water level are the most important loading parameters in design of sea dikes which provide 41% and 38 % of influences, respectively, to the total failure probability. Subsequently the design water level contributes of influence. Therefore attention should be paid carefully in determination of these parameters in design of the sea dikes.

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