APPLICATION OF RELIABILITY ANALYSIS IN DESIGN OF A RIVER DIKE

P. Q. TU [1,2], T. M. THU [1], K. VRIJLING [2], P. H. A. J. M. VAN GELDER [2],
[1] Faculty of Civil Engineering, Water Resources University of Vietnam, Hanoi, Vietnam
[2] Faculty of Civil Engineering and Geosciences, Delft University of Technology, The Netherlands

ABSTRACT: The application of reliability analysis in civil engineering is still not very widely applied in geotechnical engineering in general and in application to river dikes, in particular. This paper focuses on the general framework of reliability analysis in geotechnical engineering design and an example for river dike reliability analysis is illustrated. Reliability analysis is related to some other terminology such as: risk assessment, probabilistic design etc. but they all deal with analyzing the relevant uncertain parameters. A river dike ring was examined as a series system with many dike sections which have similar characters (geometry, geotechnical condition, protection area and so on). Considering each dike reach, there are several failure mechanisms such as instability, under seepage, erosion, thus all of them contribute to the dike breach probability with different levels. Practically, an example for a Red river dike section analysis is shown with some reliability methods and the joint probability function for dike segments was formulated.

INTRODUCTION

Reliability methods in civil engineering were developed by mathematicians and structural engineering researchers in 1950s and 1960s, but in the early 1970s reliability began to spill over into geotechnical engineering research, see Baecher (2003). However, with the demanding economic development, the more uncertainties we deal with, for instances in the blooming of infrastructures construction, energy or the environmental problems, the more significant influence of reliability analysis in civil engineering as well as in geotechnical engineering we have.

Nowadays, the application of reliability analysis in civil engineering is still an emerging technology, more so in geotechnical engineering and even more so in application to flood defenses such as river dike, sea dike…etc which also relate to a global issue, the increasing of sea water level. Much experience remains to be gained, recently, especially in European countries (the Netherlands, German, United Kingdom, Norway…) as well as in the United States of America. This paper focuses, only, on the general framework of reliability analysis in geotechnical engineering design and an example for river dike reliability analysis is illustrated.

THEORY OF RELIABILITY ANALYSIS

Fundamentals of reliability analysis

In civil engineering, generally speaking, we approach with deterministic methods which calculate the structures, materials or loads… with certain values following current codes and standards. In this approach, loads and strengths, mostly, are assumed to be determined and the structures is safe when the margin between the design value of the load and the characteristic value of the strength is large enough for all limit states of all elements. Therefore, the safety level of a structured system is not explicitly known, Vrijling (1998),

Probabilistic design approach with reliability-and risk-based analysis concepts deal with uncertainties in loads and strengths input, however, all the failure mechanism is described and the possible failure of each elements and whole systems are taken in to account as well. Therefore, in some large projects or complicated structures, probabilistic methods has many advantages, but because of its complicated in calculation, until now, it does not bloom rapidly as expected, except in developed countries.

Limit state function

Theoretically, the limit state function is defined as:

$$Z = R - S \tag{1}$$

where: Z is the limit state function; R: strength; S: load. Both R and S combine many uncertainties, for instance, the inherent and epistemic uncertainty, so that probability of failure ($P_f\{Z<0\}$) is defined as the probabilistic failure if $Z \leq 0$ and $Z = 0$ is the boundary between safe and unsafe area, see figure 1.

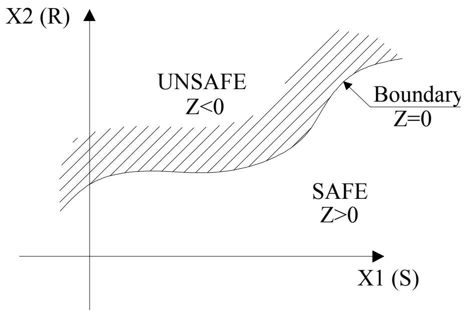

Figure 1 Limit state function in R-S plane

The probability of failure is calculated with the following integral, see TAW 190, (1996):

$$P_f = P\{Z < 0\} = \int_{-\infty}^{+\infty} F_R(x) f_s(x) dx \tag{2}$$

in which $F_R(x)$ is the cumulative distribute function of the strength R; $f_S(x)$ is the probability density function of the load S, x is the random variable.

Mathematically, in R-S space, joint probability density function of R and S is $f_{RS}(R,S)$, therefore, formula (2) can be written by:

$$P_f = \iint_{Z<0} f_{RS}(R < S).dR.dS \tag{3}$$

Limit state function is formulated for each element of structures and analyzing system with number of elements will be considered, in each case, stochastic variables are considered.

Levels calculation

The following four levels of approach were distinguished in determination of the safety of a structure (TAW, 1996):

- Level 0: Deterministic approach, the design is based on average situations and an appropriated safety factor is introduced for obtaining a safe structure;

- Level I: Semi-probabilistic approach, a characteristic value is used in the design, like the load which is not exceeded in 95% of the cases, or the strength which is available for 95% of the construction material;

- Level II: Probabilistic approach with statistical distributions of all variables are taken into account. Level II comprises a number of approximate methods in which the distribution functions are transformed into standard normal or standard Gaussian distributions. In order to approximate the probability of failure, mathematical formulation of the problem has to be linearized.

- Level III: a highest level probabilistic approach and the probability distribution functions of the stochastic variables are fully taken into account. In this calculation level, the problem is solved for both linear and nonlinear functions, for the dependent or independent variables…etc.

Hereafter, we will discuss more detail about each calculation levels in reliability of a system with multi-elements.

Elements reliability analysis

In the real life, the load and strength (in the limit state function) are always function of multiple variables, therefore, formula (3) can be express as following:

$$P_f = \iint_{Z<0} .. \iint f_{R_1,R_2...R_n,S_1,S_2...S_n}(R_1,R_2...R_n,S_1,S_2...S_n) dR_1 dR_2 ... dR_n dS_1 dS_2 ... dS_n \tag{4}$$

where R_i, S_i: are stochastic variables of strength R and load S.

Probabilistic calculation at level III, normally, the Monte Carlo simulation method is applied, in which, a large number of variables is generated and with the using of indicate function, $I(g(x)$, the number of failures is counted as following:

$$m_f = \sum_{j=1}^{m} I(g(x_j)) \tag{5}$$

in which, m_f is the number of failures; m is the total number of simulations; $g(x)$ is the limit state function; $I(g(x))$ is the indicate function, with the value is taken as following rules:

$I(g(x))=1$ if $g(x) \leq 0$
$I(g(x))=0$ if $g(x) > 0$

From number of failures, m_f, the probability of failure can be estimated by:

$$P_f = \frac{m_f}{m} \tag{6}$$

The requirement for the number of simulations to reach the demanding accuracy of this method is given by:

$$m > 400\left(\frac{1}{P_f} - 1\right) \tag{7}$$

This is a very strict requirement, for the total number of simulations, of the Monte Carlo simulation method. Some recent researches, nowadays, are trying to modify this method to reduce the steps of calculation and the numbers of simulation as well as given more accurate calculation results.

In summary, using Monte Carlo simulation methods, the uncertainties of both load and strength can be modeled whether it is dependent or independent, correlated or not. By this approach, fully probabilistic methods will be applied, however, a large number of simulations and complicated steps calculation are some limitation of this methods. Nevertheless, Monte Carlo methods still is a powerful calculation technics for reliability analysis, especially, with the blooming of computer applications.

At the level II calculation, an approximate method is applied by idealized and linearize solutions for Z function. From the limit state function, a transformation will be used to generated limit state function into standard normal distribution, from variables x_1, $x_2 \ldots x_n$ to u_1, $u_2 \ldots u_n$. Generally, Z function is assumed as a non linear, so that it will be estimated with a Taylor series:

$$Z = g_u(u_1, u_2 \ldots u_n) = g_u(u^*) + \sum_{i=1}^{n} \frac{\partial Z}{\partial u_i}(u_i - u_i^*) \quad (8)$$

in which, $Z=g_u(u_1,u_2 \ldots u_n)$ is the limit state function in u-space; u_i^* is the design point. Note that, equation (8) is estimated with the first degree in Taylor series, that why, we have many extensive methods such as : First Order Reliability Methods (FORM); First Order Second-moment Reliability Methods (FOSM)…etc.

In this case, Z function is linearized, and the design point is defined as a point with minimum distance to failure boundary, see figure 2.

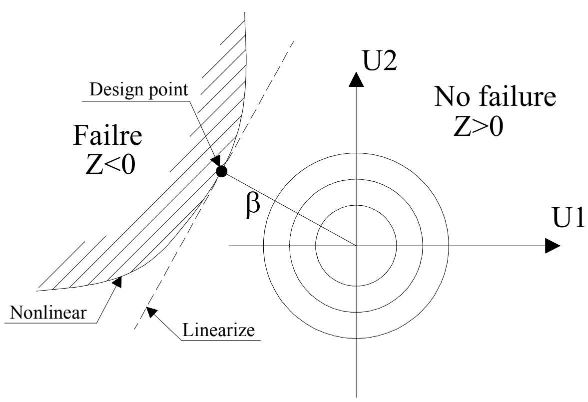

Figure 2 Reliability index and design point

Some importance probabilistic characteristics of this calculation methods are:

Mean value:

$$\mu(Z^{lin}) = Z(u_1^*, u_2^* \ldots u_n^*) + \sum_{i=1}^{n} (\mu_{u_i} - u_j^*)(\frac{\partial Z}{\partial u}) \quad (9)$$

Standard deviation:

$$\sigma^2_{Z^{lin}} = \sum_{j=1}^{n} \sigma^2_{u_i} \left(\frac{\partial Z}{\partial u}\right)^2 \quad (10)$$

Influence coefficient:

$$\alpha_j = \frac{\sigma(u_j)}{\sigma(Z^{lin})} * \frac{\partial Z}{\partial u} \quad (11)$$

Reliability index:

$$\beta = \frac{\mu(Z_{lin})}{\sigma(Z_{lin})} \quad (12)$$

In this case, probability of failure is defined as:

$$P_f = 1 - R = 1 - \Phi(\beta) = \Phi(-\beta) \quad (13)$$

where, *R* is reliability function

Lastly, probabilistic calculation with level II method is quite popular in reliability analysis which can be applied for most cases of civil engineering concepts with appreciated error. Therefore, it is used popularly in many developed countries.

Level I calculation, in every day design, is generated in many codes and standards, Eurocode for example. Basically, this method is related with level II calculation by adding the partial safety factor in calculation.

System reliability analysis

In civil engineering, mostly, we deal with the failure probability of a system with many components, so that the failure mode of all elements are considered by combination depending their connection: series, parallel or combined both of them (see figure 3)

Incase of series system, the failure probability of system is defined if one of elements failure

$$P_f^{series} = P(Z_1 < 0 or Z_2 < 0 or \ldots . Z_n < 0) \quad (14)$$

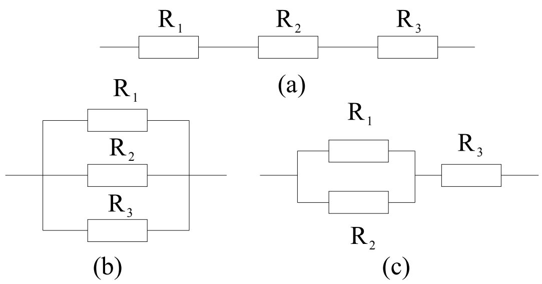

Figure 3 System description
(a) Series system; (b) Parallel system; (c) Combined system

in which $Z_i<0$ denotes at least one of n failure mechanisms occurs, the system will collapse. Boundary of probability of failure can be estimated by:

$$\max(P_f(i)) \le P_f^{series} \le \sum_{i=1}^{n} P_f(i) \tag{15}$$

or narrower by Ditlevsen (1979) as following:

$$\begin{aligned} & P_f(1) + \sum_{i=2}^{n} \max\left(P_f(i) - \sum_{j=1}^{i-1} P_f(i \cap j) \right) \le P_f \\ & P_f \le \sum_{i=1}^{n} P_f(i) - \sum_{i=2}^{n} \max_{i<j}(P_f(i \cap j)) \end{aligned} \tag{16}$$

In term of parallel system, if all elements are not functioned, the whole system will be failed, so probability of system, in this case, can be formulated as following:

$$P_f^{parallel} = P(Z_1 < 0 and Z_2 < 0 and ....and Z_n < 0) \tag{17}$$

and the failure boundary can be calculated by:

$$\prod_{i=1}^{n} P_f(i) \le P_f^{parallel} \le \min(P_f(i)) \tag{18}$$

UNCERTAINTIES IN GEOTECHNICAL DESIGN AND RELIABILITY ANALYSIS

Basically, uncertainties in the real life reflect incompletely knowledge about phenomena, mechanism, structures…etc. Primarily, it can be divided into two categories: inherent and epistemic uncertainty. The former is related to the variability in known (or observable) populations and therefore represents randomness in samples and the latter comes from basic lack of knowledge of fundamental phenomena. Both inherent uncertainty and

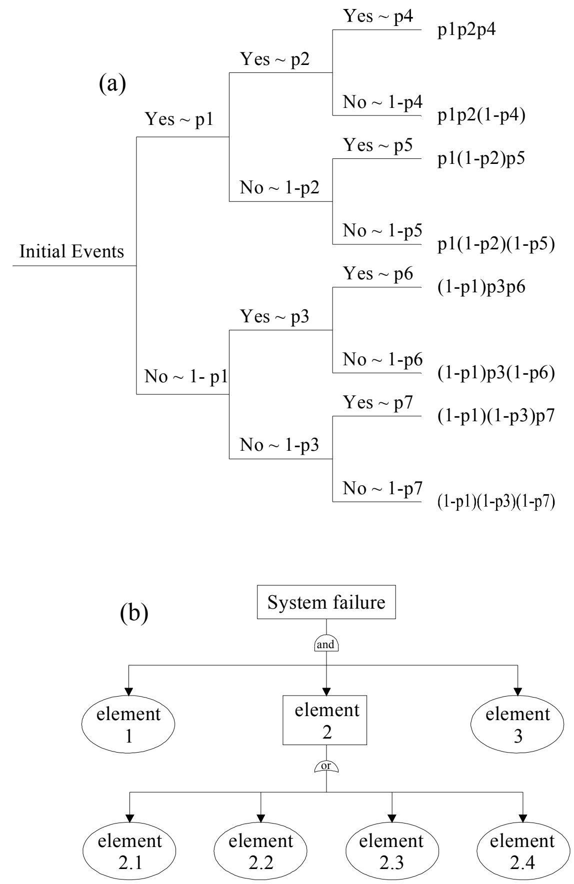

Figure 4 System analysis, (a) Event tree; (b) Fault tree
Note: p_i is probability of failure of element i

epistemic uncertainty can be subdivided in the following five types of uncertainty: inherent uncertainty in time and in space, parameter uncertainty and distribution type uncertainty (together also known as statistical uncertainty) and finally model uncertainty, see Peté-Cornell (1996).

In geotechnical field, we can faced with many uncertainties in the spatial variability of soil and rock (three dimensions), change of its properties in space and time… etc, therefore, it is widely supposed these categories of uncertainties as following, see Baecher (2003):

Natural uncertainty is associated with the “inherent” of nature phenomena and processes, with the variability over the space and time. Sometime, it takes place at difference places in a short period of time, or at a single place during a long period, or combined both of them. In the world of perfect information, soil properties, for instance, at every location in the field are known, there is no need to discus about this issues any more. In fact, data are limited as well as the analytical capacity, so that we have to model the variation of soil properties as random processes although itself does not.

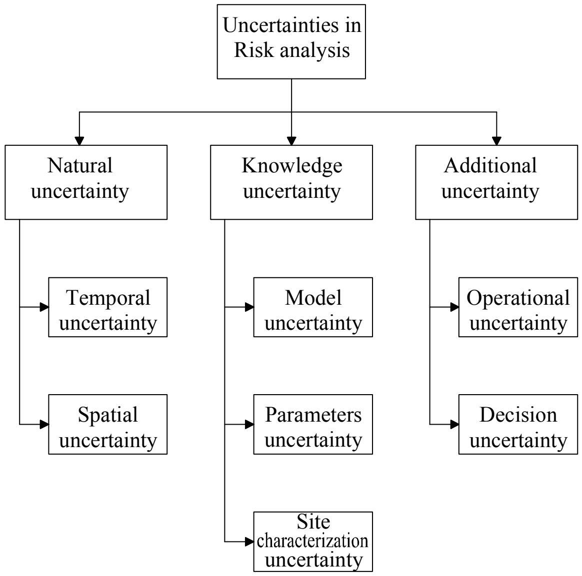

Figure 5 Uncertainties in geotechnical engineering

Knowledge uncertainty is concerned to lack of information, understanding as well as statistical data about the natural phenomena or processes. It is common known as epistemic or internal uncertainty. When enough research could and would be done, epistemic uncertainties may change as knowledge increases, Vrijling at el (2005). Model uncertainty reflects some inaccurate ways to fit the natural phenomena due to its real physical mechanism, for instance, how the failure surface is modeled in the lands slide analysis. Parameters uncertainty result in our poorly accession to the data collected from laboratory or field test or other observations. Site characterization uncertainty related to the data investigated, measurement errors, representativeness… etc, any inadequate things from mentioned lists can cause epistemic uncertainties.

Additional uncertainty is attributed with our inability to know about construction, manufacture, deterioration, maintenance and social objectives...etc. This type of uncertainty, normally, is disregarded in some concepts.

RELIABILITY ANALYSIS FOR A DIKE SYSTEM

General

The reliability analysis has been developed for flood defense since 1990s in United State and European countries (such as the Netherlands, Germany…etc). By evaluating the reliability of existing levees system of Mississippi river, Wolff (1994) created a general framework of risk-based analysis for U.S government. In the Netherlands, Delta commission was found in 1953, after a flooding disaster, to prevent similar situation happened again. Group 10 "Probabilistic method" of the Technical Advisory Committee on Waters Defenses (TAW) has been assigned the task of making the results of this development applicable to flood defense structures, CUR 141 (1990). Recently, many researchers also developed more detail the application of reliability analysis not only in coastal flood defense but also through river dike.

In the light of reliability analysis, dike ring is considered as series system with number of dike reaches, which can be evaluate as a mono-system with many failure mechanism, see figure 6.

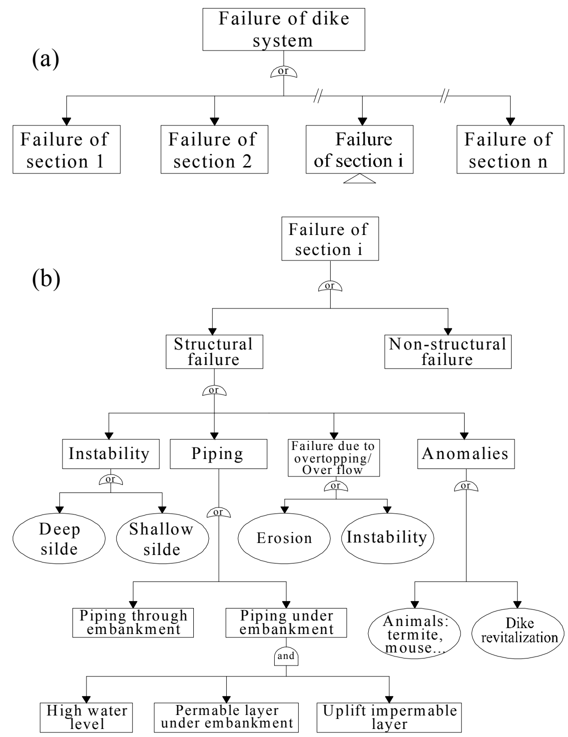

Figure 6 Fault tree analyses
(a) Dike system; (b) Dike section

Failure mechanism

As mentioned above, the dike system will be failed due to one of the dike reach collapse. Basically, for each dike section, failure mechanism can be named as: slope instability, piping, over topping/over flow or other anomalies, following analysis for each failure mechanism base on some research results from Floodsite (2007).

In terms of instability, soft-soil under embankment, high pore pressure, uplift pressure… can cause dike breach, so that it is necessary to evaluate the contribution of these factors to this failure mechanism deeply. Soft-soil often lead to deep slide of the dike embankment with typical circle failure surface, however, uplift pressure also participate in instability of inner slope. Pore pressure, in addition, play an importance role for instability of both embankment (shallow) and deep foundation. The limit state function, basically, can be expressed as following:

$$Z_{stab} = Fs - 1 \tag{19}$$
$$where:$$

$$Fs = \frac{\sum R_M}{\sum S_M} = \frac{\sum_{i=1}^{n}(G_i - u_i b_i)tg\varphi_i + c_i b_i}{\sum_{i=1}^{n}(\cos\alpha_i + \frac{tg\alpha_i.\sin\alpha_i}{Fs}).G_i\sin\alpha_i}$$

in which, *Fs* is factor of safety; R_M, S_M: are resistance and driving moment of each slide respectively; G_i: is the weight of slide; u_i: is the pore water pressure; b_i: is the width of the slide; c_i: is the cohesion of soil along failure surface; α_i: is the angle of slide compare to vertical direction.

For under seepage analysis point of view, piping under dike embankment is considered as a dominant failure mechanism, following formula is assumed:

$$Z_{pip} = i_{cri} - i_o \tag{20}$$

in which, Z_{pip} is the limit state function of piping analysis, i_{cri} , i_o are critical gradient and actual gradient of under seepage respectively.

Critical gradient depends on soil properties, crack developed in soils and the anomaly as well. The actual gradient can be evaluated by the equation-based that developed by Corps of Engineers (USACE, 2000):

$$i_o = \frac{H}{z} = \frac{Hx_3}{(x_1 + x_2 + x_3)z} \tag{21}$$
$$x_3 = \sqrt{\frac{K_{per}}{K_{cov}}.z.d}$$

in which, H is residual head at the dike toe; z is the thickness of cover layer; K_{per}, K_{cov} are permeability of cover and permeable layer respectively; x_1, x_2,x_3 are explained in the figure 7.

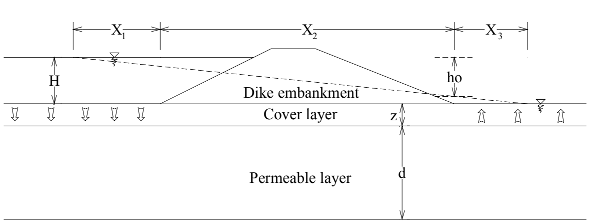

Figure 7 Typical model for under seepage analysis (USACE, 2000)

For overflowing assessment, failures of the dike embankment have often been caused because of overflow. If the flow velocities are high, grass cover may be damaged then eroded, leading to direct erosion of embankment materials. This mechanism may dominate where the flood water level exceeds the embankment crest level and waves are small. Damage is assumed to occur when the overflow discharge (or velocity) exceeds a limit given for the type and condition of grass cover on the crest and/or inner slope. reliability equation can be expressed by:

$$Z_{over} = m_{qc}.q_c - m_{q0}.q_0 \tag{22}$$
$$with: q_0 = c_D.c_v.h_{over}\sqrt{2.g.h_{over}}; q_c = \frac{v^3}{tg\alpha_i.C^2}$$
$$C = 25(\frac{q_c}{k}v_c)^{1/6}; v_c = 3.8\frac{f_g}{1+0.8^{10}\lg t_e}$$

in which: q_0, q_c are actual and critical overflowing discharge respectively; m_{q0}, m_{qc} are model factors of actual and critical overflowing discharge respectively; c_D is coefficient for weir shape, crest width; c_v is dissipation coefficient; h_{over} is depth of flow over (local) crest; v_c is critical flow velocity; α_I is angle of the inner slope; C is roughness factor according to De Chézy; f_g is condition quality of grass, varying between: f_g = 0.7 for bad turf; and f_g = 1.4 for good turf; t_e is overflow duration.

For judgmental evaluation of other modes, there are some more failure mechanisms not already treated by analytical model such as animal burrows, cracks, roots, poor maintenance…etc and it is also difficult to take them in to account. So that, during the field inspection, expert opinion could be used to consider these effects qualitatively.

Failure probability of a dike section

Generally, these mentioned above failure mechanisms lead to dike breach, so they could be judged as a series system and the failure probability is evaluated by:

$$P_f^{\sec} = P_f(Z_{sta} < 0 or Z_{pip} < 0 or Z_{over} < 0 or ...) \tag{23}$$

with Z_{sta}, Z_{pip}, Z_{over} are reliability function of stability, piping and overflow analysis respectively.

In other way, upper boundary failure probability can be calculated as following:

$$P_f^{\sec} = 1 - R^{\sec} = 1-(1-P_f^{sta})(1-P_f^{pip})(1-P_f^{over})(1-P_f^{judg}) \tag{24}$$

with P_f^{sta}, P_f^{pip}, P_f^{over}, P_f^{judg} are failure probability of stability, piping, overflow and judgment failure mechanisms respectively.

Failure probability of a dike system

Theoretically, a dike system composes a lot of elements such as dike sections, hydraulic structures… etc, and the protected areas will be inundated if one of these components fails. Generally speaking, a series system will be used to model the dike ring, so that the upper boundary of failure probability can be express as follows:

$$P_f^{sys} = P_f(Z_{\sec 1} < 0 or Z_{\sec 2} < 0 ... or Z_{\sec i} < 0 ... or Z_{\sec n} < 0) \tag{25}$$

with Z_{sec1}, Z_{sec2}, Z_{seci}, Z_{secn} are reliability functions of dike sections 1, 2, i, n respectively

Similarly, P_f^{sys} can be calculated in case of independent sections by:

$$P_f^{sys} = 1 - R^{sys} = 1 - (1 - P_f^{\sec 1})(1 - P_f^{\sec 2})...(1 - P_f^{\sec i})...(1 - P_f^{\sec n}) \tag{26}$$

with P_f^{sec1}, P_f^{sec2}, P_f^{seci} *and* P_f^{secn} are failure probabilities of dike sections 1, 2, i and n respectively.

CASE STUDY FOR A RED-RIVER DIKE SECTION IN HANOI AREA

Introduction

Red river (known as Song Hong) flows from mountain areas of southern China to the Gulf of Tonkin with the total length over 1150km (the length in Vietnam's area is about 510km). At the border of Vietnam and China, Red river enters Laocai province (northern Vietnam). After that, it runs through mountain areas to Viettri where Red river is contributed by two other tributaries named as Da river and Lo river respectively. In Hanoi area, Red river separates in to two major branches such as: Duong river, Luoc river which make the dike system in this region become more complicated. There are about 250km dike length grade 1 or higher in total 470km dike in Hanoi area.

According to the historical statistics, there was lack of data about the failure of dike in study area from 12th century, the assumed reasons may be because of poor archive works, wars …etc. Generally, many dike failures with damages of properties and lost lives are recorded, total each impacted value comes up thousands victims and billions US dollars every year. However, with the developed of economic system as well as growth of population, the impactions will run up so far.

Basically, there are 4 typical cross dike sections in this area (Man, 1999) with different geotechnical conditions. For actual location and typical section, some related failure mechanism will be considered. In this paper, we only discuss about a typical section in Sen Chieu village, Phuc Tho district, Ha Noi (formerly Ha Tay province). At this location, the dike crest level is around 18m with the width of 6m. Ground level changes from 8.5 to 10m for the landside and from 9 to 10.5m for the riverside.

For geotechnical condition, simply, there are three main soil layers such as: dike embankment – layer 1(natural condition) or layer 2 (saturated condition): grayish brown, yellow, stiff to very stiff, sandy clay; cover layer – layer 3: reddish yellow, yellowish brown, firm to stiff, clay and permeable layer – layer 4: brownish yellow, grayish brown, medium sand or coarse sandy gravel, typical properties of these layers are given in table 1.

Table 1 Geotechnical properties of soils

Layer		Parameters			
		Unit weight	Cohesion	Internal friction	Permeability
		γ_w, (kN/m^3)	C, (kN/m^2)	φ, (Degree)	K, (m/s)
1	μ	18.55	24.20	13.70	$2\ 10^{-5}$
	σ	0.58	2.72	2.40	$0.17\ 10^{-5}$
	V	0.33	7.40	5.75	12.10
2	μ	19.40	14.40	11.40	$2\ 10^{-5}$
	σ	0.61	1.62	1.98	$0.17\ 10^{-5}$
	V	0.37	2.62	3.90	12.10
3	μ	18.89	20.20	13.10	$2\ 10^{-6}$
	σ	0.73	2.67	1.66	
	V	0.53	7.12	2.75	
4	μ	18.51		28.63	$1\ 10^{-3}$
	σ	0.53		2.17	
	V	0.28		4.72	

Following current maintenance guide, the design water level is 13.4m in Hanoi (at Longbien hydrology station) similar to 16.5m at study location. If the flood water head is above that level, some emerging solutions will be applied such as: opening the Day dam or diverging the discharge to some certain areas in upper regions…etc. So, in this paper, the failure analysis is focused on some geotechnical failure mechanisms only, for instance instability, piping, uplift…, other hydraulic failure mechanisms (overflowing, overtopping, erosion…) will be disregarded.

Failure mechanisms

Instability analysis

With the variability of the soil properties, for each water level, factors of safety (FS) are calculated following Morgenstern – Price method combined with Monte - Carlo simulation. Lognormal distribution is assumed for factor of safety (Wolff, 1994). The probability of failure is calculated following formula (19) and the results are shown in table 2.

Piping analysis

Under seepage analysis will concern to piping and uplifting which could lead to failure of river dike. For piping calculation, properties of soil are used in table 3.

In lower areas of Red river dike, some researchers carried out both field test and experiment in laboratory which shows the critical gradient of the cover layers are around

0.76 – 1.98, (Truong, 2009). So that, the critical gradient leading to failure at dike toe at the study location will be assumed at $i_{cri}=0.9$

Table 2 Stability analysis results

Reliability characteristic	H(m)					
	17.5	16.5	15.5	14.5	13.5	12.5
μ_{FS}	1.14	1.22	1.30	1.41	1.43	1.46
σ_{FS}	0.106	0.105	0.104	0.100	0.110	0.109
V_{FS}	0.0927	0.0864	0.0802	0.0707	0.0870	0.0954
$\sigma_{\ln FS}$	0.0925	0.0863	0.0801	0.0706	0.0915	0.0920
$\mu_{\ln FS}$	0.1294	0.1911	0.2565	0.3446	0.4521	0.4630
β	1.40	2.22	3.20	4.88	4.94	5.03
P_f	$8.1\ 10^{-2}$	$1.3\ 10^{-2}$	$6.8\ 10^{-4}$	$5.3\ 10^{-7}$	$3.9\ 10^{-7}$	$2.4\ 10^{-7}$

The failure plane is illustrated in figure 8.

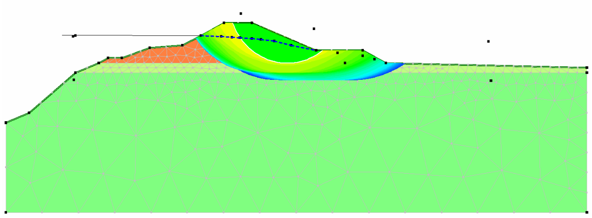

Figure 8 Typical failure model for stability analysis

Table 3 Typical soil properties using in under seepage analysis

Layer	Statistical characteristic	Parameters	
		Thickness (m)	Permeability ratio $r=K_4/K_3$
1		8.50	
3	μ(z)	3.00	500.00
	σ	0.87	0.95
	V	15.70	20.00
4	d	20.00	

in which: d, z: are the thickness of sand and cover clay layer respectively; r is ratio of permeability of sand (K_4) and cover clay (K_3) layer.

By using level 2 calculation, Fist Order Reliability Method (FORM), the piping analysis concerning to uncertainties of soil properties and thickness of cover clay layer, calculation results are shown in table 4.

Table 4 Under seepage analysis results

Characteristic	H(m)					
	17.5	16.5	15.5	14.5	13.5	12.5
β	-1.55	-1.36	-0.97	0.50	1.45	5.62
P_f	0.94	0.91	0.83	0.31	$7.35\ 10^{-2}$	$9.55\ 10^{-9}$

Judgment other failure modes

Besides all mentioned failure mechanisms, there are numbers of uncertainties that could result in the failure of river dike, for instance: animal burry, anomaly in dike embankment, poor maintenance, human error… etc. Unfortunately, these influences can not be accounted qualitatively, so it should be judged by expert opinions combining with field inspection. It is assumed the reliability index and probability of failure for all components are given in table 5.

Table 5 Judgment analysis results

Characters	H(m)					
	17.5	16.5	15.5	14.5	13.5	12.5
β	-0.13	0.25	0.67	1.28	1.64	5.00
P_f	0.55	0.40	0.25	0.10	0.05	0.00

Failure probability of dike section

Deterministically, dike analysis is often carried out by separate calculations, such as stability, seepage…etc. In reliability analysis for a dike section as well as a dike system, series system is used to figure out the different failure probabilities of each component. Following formula (24), in this case we have a failure probability of a dike section as:

$$P_f^{\sec} = 1 - R^{\sec} = 1 - (1 - P_f^{sta})(1 - P_f^{pip})(1 - P_f^{judg}) \quad (27)$$

so that, the results of the combined failure probability can be seen in table 6.

Discussion

It is clear from figure 9 that the failure probability of Sen Chieu dike section is dominated by piping phenomena. At 12.5m of water level (alarm level 1), probability of failure by piping is very small, about $1.2\text{x}10^{-6}$, but it increases dramatically till 84% at alarm level 3 (14.5m). This calculation result is accurate with observation data in this dike section in the past, see DDMFC (2009). At higher water level, failure due to under seepage can occur, so the solution to reduce the actual gradient at the dike toe should be done. However, the combined failure probability is very high (38% at alarm level 3), in terms of piping mechanism, there should be a solution to deal with over acceptable risk of this dike section.

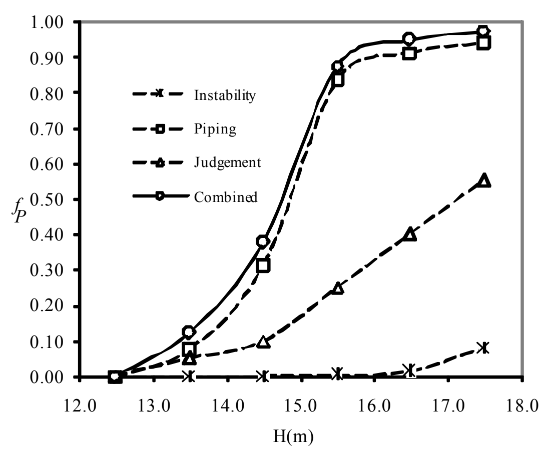

Figure 9 Failure probability of a dike section

Table 6 Summary of analysis results

H (m)	Stability			Piping		Judgement		Combined
	Fs	β	P_f	β	P_f	β	P_f	P_f^{sec}
12.5	1.46	5.03	2.45E-07	5.62	9.55E-09	5.00	0.00	0.000
13.5	1.43	4.94	3.91E-07	1.45	7.35E-02	1.64	0.05	0.120
14.5	1.41	4.88	5.30E-07	0.50	3.09E-01	1.28	0.10	0.378
15.5	1.30	3.20	6.87E-04	-0.97	8.34E-01	0.67	0.25	0.876
16.5	1.22	2.22	1.32E-02	-1.36	9.13E-01	0.25	0.40	0.949
17.5	1.14	1.40	8.08E-02	-1.55	9.39E-01	-0.13	0.55	0.975

On the other hand, further research should be carried out, such as sensitivity analysis, spatial variation of soil properties, time dependency in each failure mode and hydraulic boundary conditions as well.

CONCLUSIONS

Generally, reliability analysis is a powerful tool not only in civil engineering but also in geotechnical engineering. By taking into account uncertainties of both load and strength, engineers could figure out accurate solutions for design problems. However, in the geotechnical field, the variation of soil and rock properties as well as their spatial distribution are faced with our daily life, so the tool should be developed futher.

In the case study, combined failure probability is demonstrated with a very high probability of failure of Sen Chieu dike section. Further study should be done for this location and the whole Red-river dike system as well.

REFERENCES

14 TCN 84-91, (1991). Design standards and guidelines for shore protections and river flood defences. (in Vietnamese)- 14 TCN 84-91. *Sectoral design standard*. Ministry of Water resources and irrigations, Hanoi, Vietnam 1991.

Baecher,G.B., Christian, J.T (2005). Reliability and statistics in geotechnical engineering, John Wiley & Sons

Binh, H.T. (2004), Study the failure mechanism of under seepage and propose design solution at dike section in Sen Chieu – Phuc Tho – Ha Tay. MSc thesis, Hanoi university of Mining and Geology CUR -141,(1990). Probabilistic design of flood defences, Gouda, the Netherlands.

CUR – 190, (1997), Chances in Civil Engineering, Part 1: Probabilistic design in theory, Gouda, the Netherlands.

Department of Dikes Management and Flood Control of Hanoi.(2009). Report on current condition of river dike systems in Hanoi area before flood season, 2009.

Floodsite, (2007), Failure mechanisms for flood defence structures, Floodsite Task 4, edited by: HR Wallingford, Wallingford, UK.

Ditlevsen, O., 1979. Narrow reliability bounds for structural systems. *Journal of Structural Mechanics*, pp. 453-472.

Man, N.C., (1999) Some geotechnical engineering aspects of dike in Red river delta, calculation – revitalization, *Proc. of geology engineering and geo-environment in Vietnam.*

TAW, (1997), Leidraad waterkerende kunstwerken en bijzondere constructies (*Guideline water retaining structures and special stuctures, in Dutch*). Rijkswaterstaat, DWW, division public works.

Truong, B.X., (2009). Study on the failure of under seepage of lower Red river dike in Thai Binh province and field experiment-based analysis the design solution. *PhD thesis*, Hanoi university of Mining and Geology.

U.S Army Corps of Engineer (USACE), (2000). Design and construction of levees. *Engineering manual* 1110-2-1913, April 2000.

Van Gelder, P., Buijs, F.A., ter Horst, W., Kanning, W., Mai Van, C., Boer, E., Gupta, S., Shams, G., & van Erp, H.R.N., 2009. Reliability analysis of flood defence structures and systems in Europe. In P. Samuels, S. Huntington, W. Allsop & J. Harrop (Eds.), Flood risk management. *Research and practice*. Pp. 109.

Vrijling, J.K., Van Gelder, P.H.A.J.M., (2005), Implications of uncertainties on flood defence policy, *Proc.Int. Conf. ISSH, Nijmegen, the Netherlands*, 23-24, May, 2005.

Vrijling, J.K., Van Gelder, P.H.A.J.M., (2000), Probabilistic design, lecture notes CT4310, TU Delft

Vrijling, J.K., Hengel, W.van & Houben, R.J. (1998). Acceptable risk as a basic for design. *Journal of Reliability Engineering and System Safety*, pp. 141-150. Elsevier 1998.

Vrijling, J.K., Van Gelder, P.H.A.J.M., (1998), The effect of inherent uncertainty in time and space on the reliability of flood protection, Safety and Reliability

(Lydersen Hansen & Sanotorv (eds)), pp.451-456, Balkema, Rotterdam.

Wolff, T.F. (1994). Evaluating the reliability of exiting levees. ETL 1110-2-556, Risk-based analysis in geo-technical engineering for support of planning studies, 28 May, 1999.