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INTERNATIONAL INSTITUTE FOR INFRASTRUCTURAL, HYDRAULIC
AND ENVIRONMENTAL ENGINEERING

PROBABILISTIC DESIGN AND RISK ANALYSIS FOR THE DUONG RIVER DIKE IN THE RED RIVER DELTA IN VIETNAM

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Master of Science Thesis
by
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ABSTRACT

Traditionally, the design of sea and river dikes in Vietnam is primarily based on a water level with a particular frequency of being exceeded. The frequencies of exceedence of design levels and governing discharge are widely regarded as constituting a standard for the safety of the region protected by the dikes and are interpreted in terms of inundation probabilities. However, this is not fully correct since the inundation probability is not only determined by the frequency of exceedence of design levels, but it is also caused by other mechanisms below that design levels.

Normally, the dike crest exceeds the design water level to a certain extent, thus, the probability of overtopping is smaller than the design frequency. However, some parts of the dike may already be critically loaded before the design water level is reached. Water logging may lead to slide planes through the dike or piping may undermine the body of the dike, with sudden failure as a consequence. In short, there are other failure mechanisms that can lead to flooding of the protected area than overtopping. In fact, if all possible causes of dike failure at high water level such as overtopping, piping, macro instability, and micro instability etc. could be listed and the associated probabilities of their occurrence be ascertained, then, in principle, the probability of inundation could be calculated

As a next step, a new approach to safety of the design of dikes, embankments, and other flood defenses was developed. This is the probabilistic design method. The probabilistic approach aims to determine the true probability of flooding of a river system and to judge its acceptability in view of the consequences. In this approach, the stochastic character of the various load and strength parameters is taken into account and the design is based on an analysis of failure probabilities. This new approach is called "risk-based" approach.

During this study, the probabilistic design method is applied for a case study in Vietnam, which is the dike improvement for the right Duong River dike in the Red River Delta in Vietnam. Due to lack of time, only the failure probabilities caused by overflowing, piping, and sliding mechanisms and the failure probability caused by hydraulic structures have been calculated. For each mechanism, uncertainties for various variables are considered by mean of statistical tools. The failure probability of each mechanism was determined based on the defined reliability function and, as a result, the inundation probability of the dike was calculated by combining all the mechanism failure probabilities. By considering the risk and the cost for the improvement, the optimization of the dike design is reached. From the economic point of view, the optimal dike crest elevation is the point corresponding with the minimum total cost. Following this study this leads to a crest level that can withstand a 1/500-year flood approximately.

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1 CHAPTER I: INTRODUCTION

1.1 GENERAL INTRODUCTION

Vietnam is regularly affected by substantial damages due to floods. The reasons of these damages were caused by severe hydraulic loads and rather low safety level of the present dike system. Since 1996, Vietnam was affected by several flood disasters, each disaster responsible for the loss of hundreds of lives and considerable damage to infrastructure, crops, rice paddy, fishing boats and trawlers, houses, schools, hospitals, etc. The total material damage of the flood disasters in these years exceeded US$ 500 million, which damage was accompanied by the loss of almost 1000 lives. The flood disasters occurred in North Vietnam (1996), in South Vietnam (1997) and especially in the Central Vietnam (1999) from Quang Binh to Phu Yen provinces causing the loss of 721 lives and the damage of 4.693.901 million Vietnam dong (Duong flood protection sub-project, 2001). Most floods were initiated by typhoons and occurred in the coastal zone. Heavy monsoon rains also caused failure of river dikes and floods inland. These floods affected seriously the living and working conditions of the people in many regions of Vietnam.

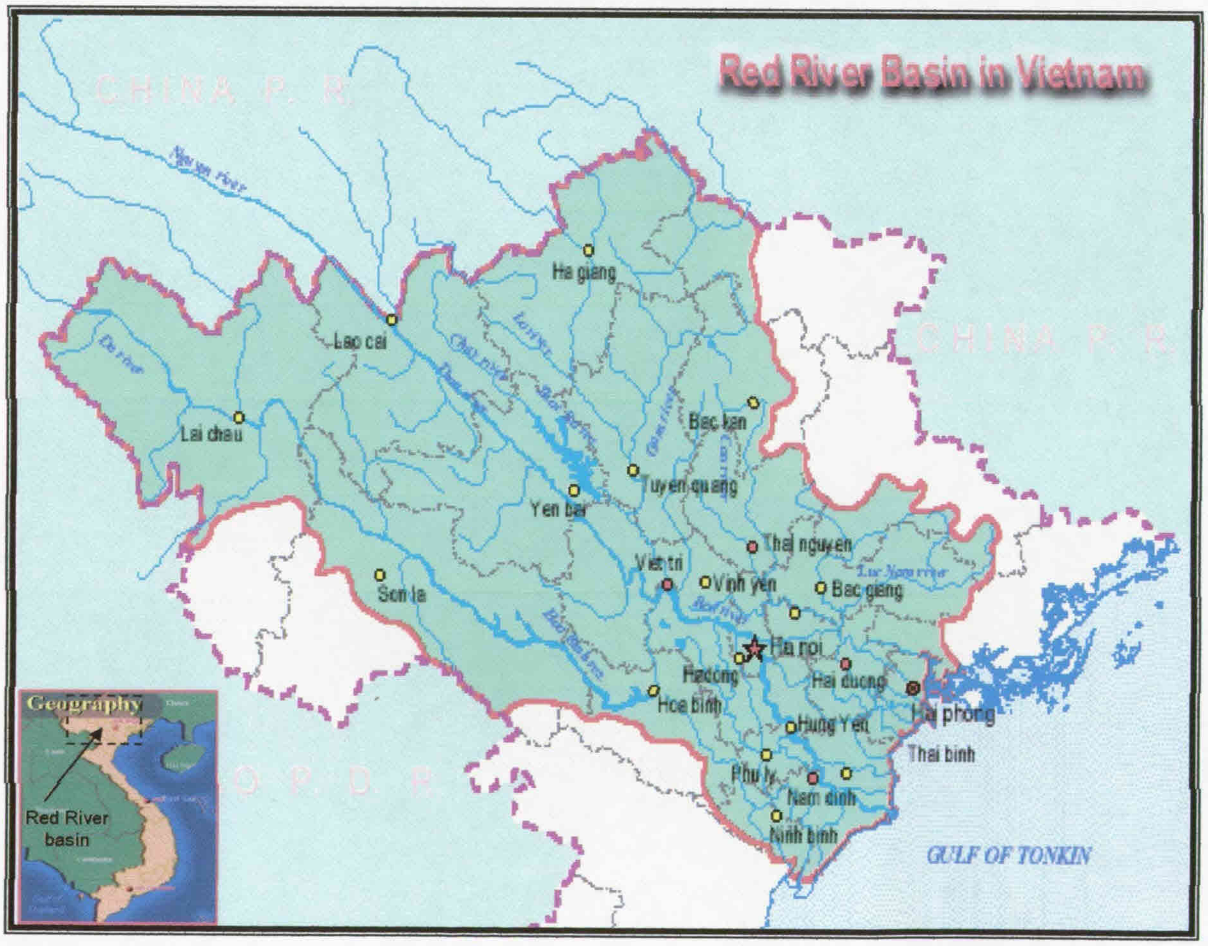

Figure 1.1 Red River Basin in Vietnam

The Duong River basin is located in the Red River Delta and it is a distributary of the Red River. The Duong River naturally diverts about 30 percent of the mainstream Red River flow eastward across the Delta to the Thai Binh River. Flow into the Duong River begins just north of Hanoi. The Duong River is approximately 60 km long, and is almost entirely bounded by dikes along its left and right margins. The channel meanders significantly and the floodplain between the left and right side dikes are 100 m wide at its narrowest point at the Duong Bridge, and more than 4,000 m wide at Song Giang, Giang Son, and Mo Dao. The right river dike system of the Duong River together with the left river dike systems of Red, Luoc and right river dike system of Thai Binh Rivers provide flood protection to a wide region (Figure 1.2), which includes 180.000 hectares of land. Almost this area is rural agricultural land. They also protect 2.5 million people, national highway No 5, the national railway connecting Hanoi city to Hai Phong city, Gia Lam airport, a number of towns such as: Gia Lam, Duc Giang, Sai Dong, and the city of Hai Duong. Like many other parts in the country this region is always threatened by floods. Although this river dike system does not fail frequently, it is still impossible to say that the region is protected safely from flooding based on the observations in the past and the present condition of the dike system. Together with the economic development, the safety insurance for the region from flooding is also very important. The increase of safety level not only protects the region better but also creates a growth of economical activities and therefore the growth of national income. The expected development of higher river discharges requires a pro-active policy, in which the increase of interests and investments to be protected will have to be taken into consideration. Knowledge of water and water defenses is indispensable when considering the desired protection level against flooding.

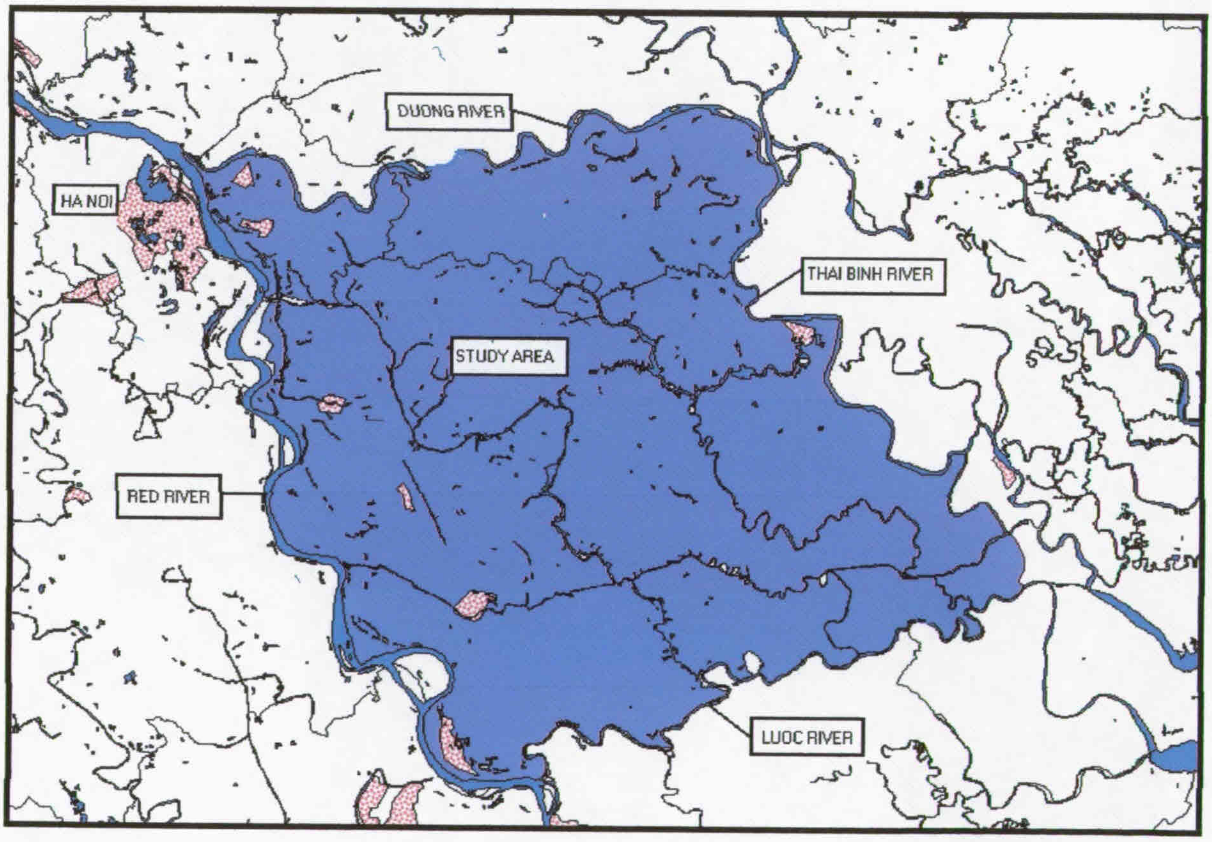

Figure 1.2 The Duong River Flood Protection Study Area

1.2 PROBLEM DESCRIPTION

Most designs of the Vietnamese sea and river dikes in general and the Duong River dike in particular are based on loads with return periods of 30, 40 years or even shorter periods. It should be noted that the adopted return periods are not based on proper risk analysis. The often adopted return period is founded on rather arbitrary, unspoken considerations of the authorities. The theoretical knowledge in the fields of safety approach, risk analysis, policy analysis, and mathematical modelling is not up to date. Failure mechanisms, which differ from overflowing 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 of the river dike system may exceed the design frequency. The present state of this system is insufficient to protect the region against flood disasters.

Besides the above reasons, there are also other imperfections of the present design method, as applied in Vietnam that can be summarized as follows:

- Per section of the dike there is an open question if a balanced design with regard to the various failure mechanisms was reached. It is not known which of the failure mechanisms makes the greatest contribution to the probability of failure of the dike section in question. For a soundly based design it is desirable that these contributions should be inter-adjusted in a well-balanced manner.

- The overall length of the dike is of no influence upon the design per section of dike. But the longer the dike, the higher failure probability is likely to be.

- The magnitude of damage or loss has no influence on the design of the dike.

- Because the actual probability of inundation of the region protected by a dike system is not known, there is no clarity on the background on which the politicians make their decisions on safety standards.

All the above imperfections of the present design method can be overcome by the probabilistic design method.

1.3 OBJECTIVES OF THE STUDY

This study is one of the first attempts towards a new safety philosophy based on risk approach, a calculation method for probabilities of flooding and consequences. It is a continuation of the previous study by Vrolijk (2002). The aim of this study is:

- *To analyze / understand and to apply the probabilistic design method and risk analysis to the improvement of the Duong river dike system in particular, and the Vietnamese water defense system in general*

- *To apply accordingly this knowledge in the Vietnamese situation and to extend it to other locations.*

- *To appraise the present state of the Duong river dike system by probabilistic approach and risk analysis and through that to help Vietnamese authorities get a clearer picture of the Vietnamese water defense system.*

2 CHAPTER II: PROBABILISTIC DESIGN METHOD

2.1 INTRODUCTION

The traditional design is based upon the deterministic approach. In this approach, a limit state condition is chosen with respect to the accepted loading state of the structure. This limit state usually corresponds to a certain strength value or the characteristic strength. An important limitation of the deterministic approach is that once a certain load has been chosen, no account is taken of loading below or exceeding that value, whereas contributions of these loads to the expected damage are neglected. This can be considered as a serious shortcoming when future damage must be estimated and quantified for maintenance assessment.

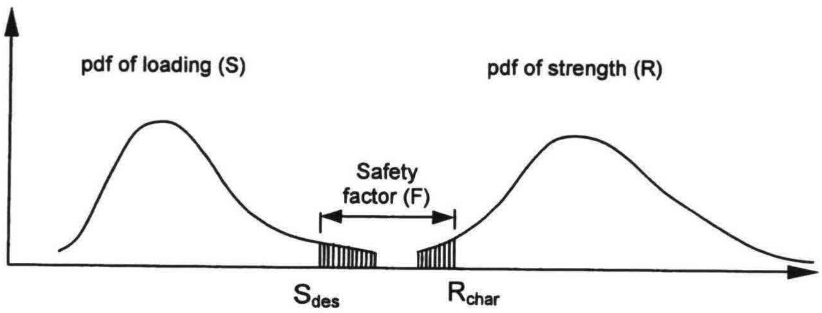

Figure 2.1 Example of a deterministic approach

In fact, the probabilistic design approach is a logical extension of the traditional method. In this approach, the various uncertainties of the loading and strength variables are expressed in terms of probabilities. The probabilistic approach aims to estimate, on the one hand, the failure probability of a flood defense system and on the other hand, the expected damage according to that failure during the lifetime of the system. The design is based on a risk analysis with regard to safety and economy.

2.2 PRINCIPLE OF RISK ANALYSIS

Risk analysis can be applied in order to judge whether a technical system satisfies the requirements that society applies with regard to safety and economy. The term "risk" comprises the probability of an undesirable event (failure, inundation) and the consequence of the occurrence of that event (economic loss, number of death). This may be expressed in general as follows:

Risk = probability x consequence

The aim of risk analysis is to estimate on one hand, the probability of occurrence of an undesirable event and on the other hand, the consequences of occurrence of that event.

The three main elements of the risk analysis are: *hazard – mechanism – consequence*. A risk analysis begins with the preparation of an inventory of the hazard and the mechanism. A mechanism is defined as the manner in which the structure responds to hazards. A combination of hazards and mechanisms leads, with a particular probability, to failure or collapse of the flood defence structure or its component parts. Finally, the consequences of failure or collapse must be considered. In the event of failure of the flood defence structure as a whole, the material damage and non-material loss must be estimated. The probability of failure multiplied by the damage or loss constitutes the risk. For an optimal design it is essential to seek an appraisal in the sense of weighing the risk, on the one hand, against the cost of constructing a flood defence structure, on the other.

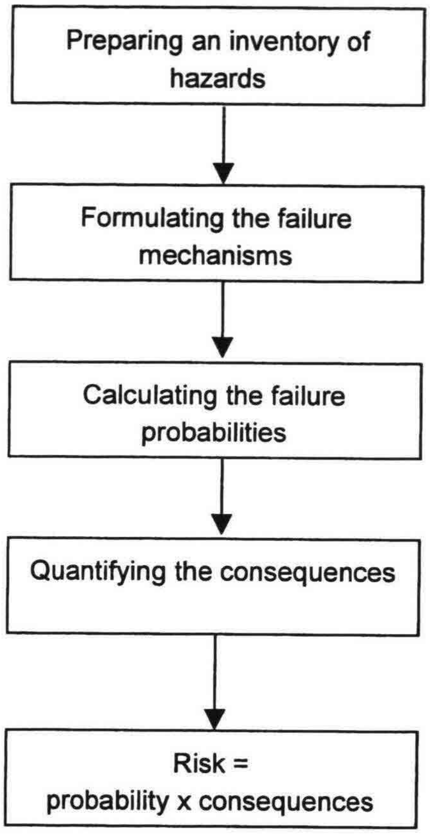

Figure 2.2 Elements of the risk analysis

The advantages of the risk analysis approach is:

- The various uncertainties are rationally incorporated in the assessment of the safety of the system
- It is possible to take into account the cost of improving the system and the damage of loss expectation per protected region. This can lead to greater differentiation of the safety within the country.
- The politicians obtain a clearer conception on the matter on which they have to make pronouncement.
- Better insight into the sensitivity of the failure probability of the system to the various uncertainties is obtained. This enables priorities to be established for further research with a view to improve the description of the system and reducing the margins of uncertainty.
- Better insight into the priority for improving different elements of the system is obtained.

2.3 THEORY OF PROBABILISTIC DESIGN APPROACH

In order to ascertain the probability of failure due to a particular mechanism, a probabilistic calculation should be performed. In many cases the failure of a structure can be reduced to comparing two quantities, the resistance (R) and the load (S). For this purpose, it is necessary to have a computational model of the mechanism. On the bases of that model a so-called reliability function (Z) is established with regard to the limit state considered in such a way that the negative values of Z correspond to failure and positive values to non-failure (Figure 2.3). The reliability function can then be written as:

$$Z = R - S$$

The probability of failure can thus be represented symbolically as P{Z < 0}.The reliability function is a function of a number of variables such as the the water level, the crest level, the angle of internal friction, etc. The variables with a stochastic character are usually called the basic variables.

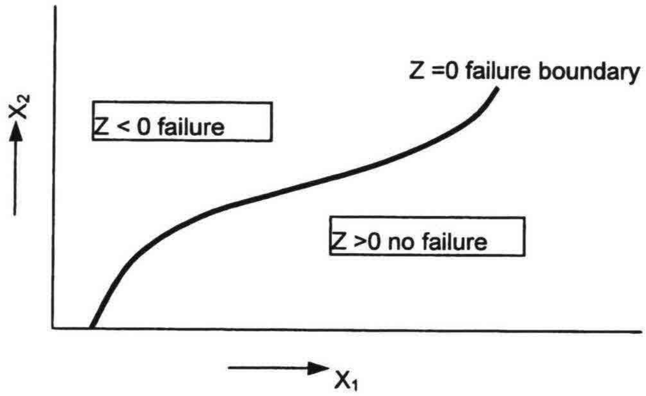

Figure 2.3 Definition of a failure boudary Z = 0

There are various techniques available for determining the probability of failure for a given reliability function. For classifying these techniques, the following levels are to be distinguished

Level III: Comprises calculations in which the complete probability density function of the stochastic variables and the possibly none-linear character of the reliability function are taken into account.

Level II: Comprises a number of approximate methods in which the problem is linearized at the design point. All probability density functions are replaced by probability density functions following the normal distributions.

Level I: Comprises calculations based on characteristic value and partial safety factors or safety margins.

2.4 PROBABILISTIC APPROACH APPLIED IN THIS STUDY

During this study, the probabilistic method with calculations at level II with regard to economic point of view is applied for the design of the improvement of the right Duong River dike. An overview of level II calculations is given below:

For introducing the calculations at level II, the reliability function Z = R – S is considered, whereby it is assumed that R and S have both normal distributions. From the theory of statistics, it is known that Z then also follows a normal distribution. This implies that the mean value μ and the standard deviation σ of Z can be obtained through:

$$\mu(Z) = \mu(R) - \mu(S) \tag{2-1}$$

$$\sigma^2(Z) = \sigma^2(R) + \sigma^2(S) \tag{2-2}$$

The probability of failure of a structure follows from (Figure 2.4):

$$P\{Z < 0\} = \int_{-\infty}^{0} f_Z(Z)dz = \Phi_N(-\beta) \tag{2-3}$$

$$\beta = \frac{\mu(Z)}{\sigma(Z)} \tag{2-4}$$

Where:

$F_Z(Z)$ = probability density function of Z
$\Phi_N(-\beta)$ = distribution function of the standard normal distribution
β = reliability index

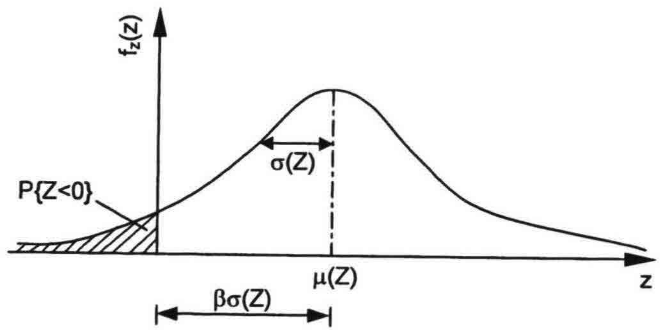

Figure 2.4 Probability density function of Z = R – S, showing the reliability index β

For the general case, Z is an arbitrary function of n stochastic variables $X_1, \ldots, X_n$. Assuming that the variables X_i are mutually independent and that their mean values and standard deviations are known, the basic feature of the level II analysis is that the function Z can be linearized. In case the linearization process is based on expansion through a Taylor series at a point $X_i = X_i^0$, the linearized function becomes:

$$Z = Z^0 + \sum_{i=1}^{n}\left(X_i - X_i^0\right)\left[\frac{\partial Z}{\partial X_i}\right]^0 \tag{2-5}$$

Where:

Z^0 = function value of Z at the point $X_i = X_i^0$.

$\left[\frac{\partial Z}{\partial X_i}\right]^0$ = partial derivative with respect to X_i, evaluated at the point $X_i = X_i^0$.

The mean value and the standard deviation of Z are:

$$\mu(Z) = Z^0 + \sum_{i=1}^{n} \{\mu(X_i) - X_i^0\}\left[\frac{\partial Z}{\partial X_i}\right]^0 \tag{2-6}$$

$$\sigma^2(Z) = \sum_{i=1}^{n} \left\{\sigma(X_i)\left[\frac{\partial Z}{\partial X_i}\right]^0\right\}^2 \tag{2-7}$$

The probability of failure can be again expressed by:

$$P\{Z < 0\} = \int_{-\infty}^{0} f_Z(Z)dz = \Phi_N(-\beta) \tag{2-8}$$

A so-called Mean Value Approximation can be followed through by adopting the mean values of X_i for X_i^0. A more accurate approximation, however, can be obtained by letting X_i^0 coincide with the design point, which is defined as the point on the failure boundary where the probability density attains a maximum. The design point is given by (Figure 2.5):

$$X_i^0 = \mu(X_i) - \alpha_i \beta \sigma(X_i) \tag{2-9}$$

$$\alpha_i = \frac{\sigma(X_i)}{\sigma(Z)} \frac{\partial Z}{\partial X_i} \tag{2-10}$$

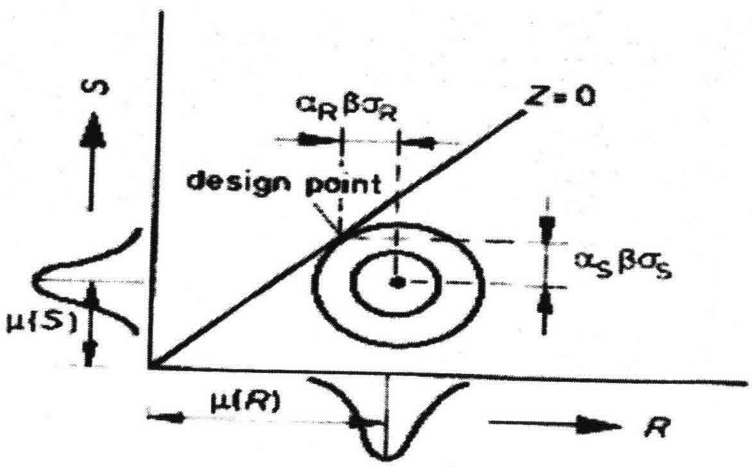

Figure 2.5 Definition of the design point as the point on the failure boundary where the probability density is greatest

The difficulty is that the design point can not be determined directly (unless Z is linear function), so that an iterative procedure must be followed.

In the case the basic variables are non-normally distributed. They can be treated by replacing the non-normal distribution with equivalent normal distribution, for which the values of the density function and distribution function at the point X_i^0 are the same (Figure 2.6)

Summary of the solution to the problem during this study follows a number of steps as below:

- First, determination of the failure probability caused by a number of failure mechanisms such as overflowing, piping, macro-instability and the failure probability caused by hydraulic structures according to the level II approach.

- Second, determination of the overall failure probability of the dike given the component failure probabilities of overflowing, piping, sliding, and hydraulic structures.

- Third, estimation of damage for the study area in case of inundation as a consequence of the failure of the dike.

- Forth, calculation of the total cost for the improvement of the dike, which includes the cost of construction and the cost of damage (risk) in case of inundation of the area.

- Fifth, determination of the optimal dike design. The optimal design return period will be corresponding with the point of the minimum total cost.

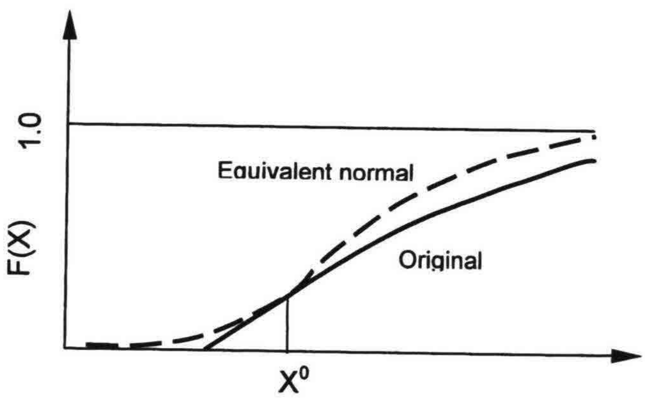

Figure 2.6 Replacing a non-normal distribution by a normal distribution

3 CHAPTER III: AVAILABLE DATA FOR THE DUONG RIVER AND THE STUDY AREA

3.1 HYDRAULIC DATA

3.1.1 Hydrologic overview of the Duong River system

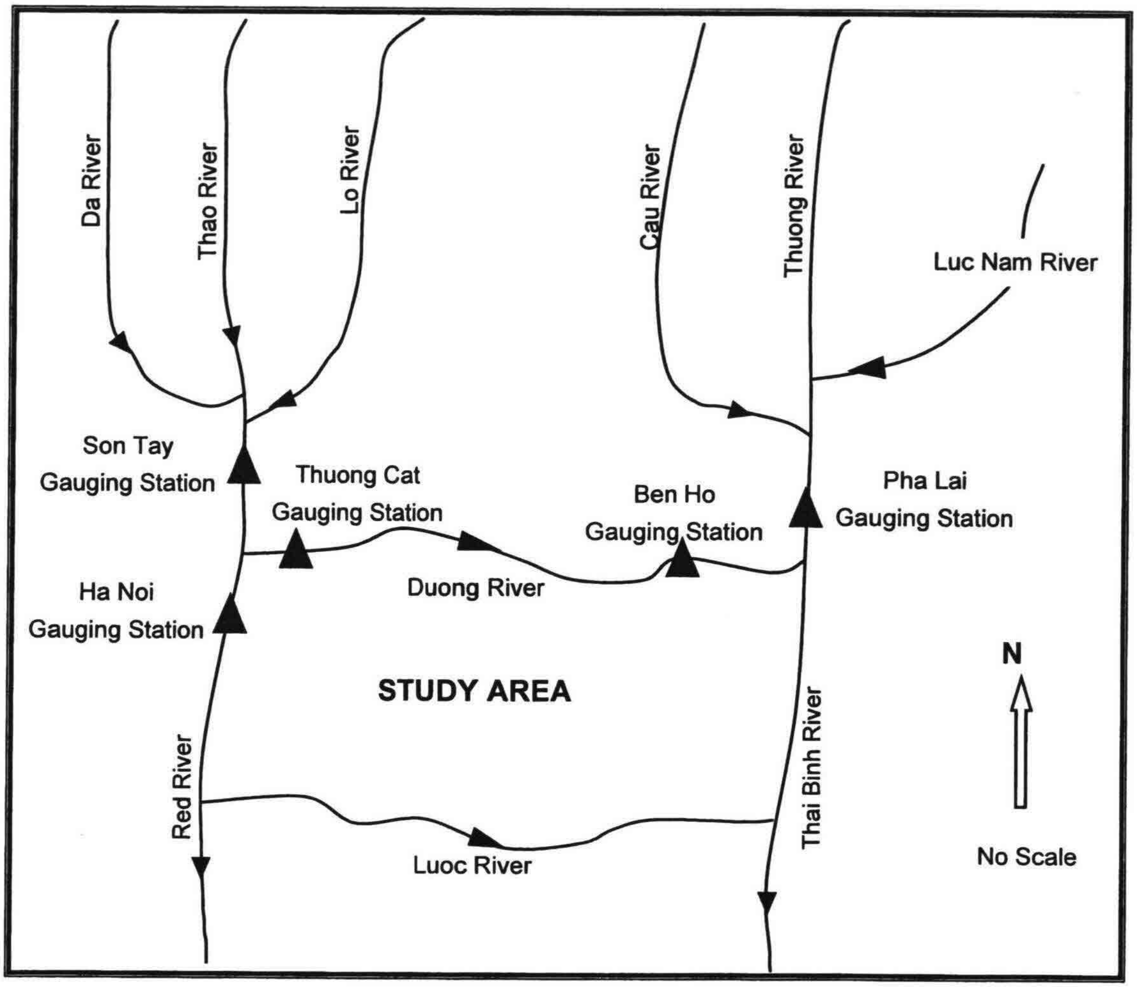

Figure 3.1 Location of gauging stations in the Red River Delta

3.1.2 Flow characteristics of the Duong River catchment

The Duong River itself is a distributary of the Red River. The Duong River naturally diverts about 30 percent of the mainstream Red River flow eastward across the Delta to the Thai Binh River. Flow into the Duong River begins just north of Hanoi capital. The Duong River is approximately 60 km long, and is almost entirely bounded by dikes along its left and right boundaries. The river meanders widely along its course within the dikes, which are generally two to three kilometers apart. But the dikes are up to 5 km apart along a wide bend in the lower reach of the River. Small islands occur within the channel area, particularly in the eastern end of the channel. Lateral inflows enter the Duong River from adjacent agricultural land by pumping and by a network of ditches and gated sluices.

However, the majority of the flow in the Duong River is contributed by the upstream catchments of the Red River. It is estimated that perhaps only about 5 percent of the flow in the rivers on the Delta originates from rainfall occurring on the Delta itself. Major shifts in vegetation cover and the construction of the Hoa Binh Reservoir in the upstream catchments of the Red River have contributed to changes in the flow regime and channel geometry on the lower Red River and on the Duong River.

The entire Duong River catchment is a subsection of the relatively flat, low-lying topography of the Red River Delta. In general, the land surface elevations range from about 5 m to 10 m, with somewhat higher elevation points along the dike and bridges. Significant changes in vegetation and land use have not occurred over the past several decades in the Duong River catchment. Land use in the catchment is dominated by agriculture, with a much smaller surface area utilized for villages and small towns. Cropland is dominated by rice paddies interspersed with smaller fields of maize and other crops. A number of elongated ponds are situated parallel to the Duong River's course on the field side of the dike. In several places, these ponds are 5 km or so in length. Being adjacent to the dike and even lower than the general surrounding land surface, these ponds are a significant feature affecting the seepage and stability characteristics of the Duong River dike. In addition, the flow of the river impinges against the dikes at bends in five or six locations. Revetments have been placed at these locations and are being monitored for river bank and dike bank erosion. No reservoirs or flood detention zones exist along the length of the Duong River

The seasonal occurrence of rainfall in the Duong River catchment is similar to the Red River basin as a whole. A monsoon season extends from May through October, with significantly dryer weather the rest of the year. Typhoons making landfall in northern Vietnam from the Gulf of Bac Bo (Tonkin) affect the Duong River catchment during the monsoon season, primarily in July, August, and September. The mean annual rainfall is approximately 1,600 mm, which is slightly lower than for the interior overall of the Delta. Rainfall reaches a maximum monthly average of 275 to 300 mm in August. The mean annual evaporation is approximately 990 mm, and the average annual relative humidity is approximately 85 percent.

3.1.3 General flow characteristics of the Duong River

Flows in the Duong River are measured primarily at the Thuong Cat gauging station at the western end of the approximately 2 km downstream of the Red River (Figure 3.1). Both river stage and discharge are recorded at Thuong Cat, with river stage recorded at 6-hour time intervals. Additional river stage records are taken at the Ben Ho gauging station approximately two-thirds of the way downstream on the Duong River. River stage and discharge are also measured at the Pha Lai gauging station, on the Thai Binh River approximately 5 km upstream of its confluence with the Duong River. Based on the data from these gauging stations, it is known that tidal effects extend up the Thai Binh River and the Duong River, particularly in low flow months. Similarly, both tides and storm surges from the coast affect flows and water levels in the lower Thai Binh River up to Pha Lai and in the lower Duong River during the rainy season. In spite of these factors and the flat channel slopes in the Delta, flood flows in the Duong River are from the Red River in the west to the Thai Binh River in the east.

The Thai Binh River at the Pha Lai gauging station located above the confluence with the Duong River had a total annual runoff of approximately 10 billion cubic meters for the period 1961 through 1985. Downstream of the confluence, the Thai Binh River received additionally approximately 29 billion cubic meters annually from the Duong River over the same time period. Upstream at the Thuong Cat gauging station, the mean annual Duong River flow from 1961 through 1985 was 876 m^3/s, or approximately 27.6 billion cubic meters per year. Mean annual flow in the Red River at Hanoi over the same time period was approximately 2710 m^3/s.

3.1.4 Hydrologic data analyses

♦ Data Sources and Considerations for Duong River Hydrology

A system of rain gauges and river stage and discharge measurement stations has been in place on the Red River system for many decades (Figure 3.1). Approximately 40 years of record for river stage and discharge are available at the Thuong Cat gauging station on the Duong River. Around 90 years of record for river stage and discharge are available at both the Son Tay and the Hanoi gauging stations on the Red River. Additional river stage data have been recorded at the Ben Ho gauging station on the Duong River, and river stage and discharge have been recorded for approximately 90 years at the Pha Lai gauging station on the Thai Binh River. However, these last three data sets were not available during this study, so they could not be further analyzed and reviewed.

River stage is of particular interest to the Duong River hydrologic analyses, since the potential for overflowing the dikes is a major concern. In addition, water levels on the face of a dike section create some probability of dike failure by mechanisms other than overflowing. Therefore, river stage data are of further interest for the design of remedial improvements for the dikes. Given these considerations and the non-unique relationship between river stage and discharge in the Duong River, it was decided to emphasize the analysis of directly recorded river stage data rather than from discharge data.

♦ Data Analyses for Thuong Cat gauging station on the Duong River

At Thuong Cat gauging station, the data observations for daily water level and river discharge were recorded during the period from 1961 until now, with river stage recorded at 6-hour time intervals. Often maximum stages are observed and recorded for high flow flood events. Discharge measurements involve both velocity and depth measurements, which are integrated together to calculate the flow rate. Data from measurements are processed and screened by qualified and capable Hydro-Meteorological Service (HMS) staff hydrologists. The data set contains 39 continuous years, which includes 14,244 values of observation.

Data analyses for the Duong River on determining stage frequencies at the Thuong Cat gauging station. Since only maximum values of water level are of interest so from the data set, the maximum value of each year was taken out. Finally, a data set of 39 maximum values was selected (Appendix I). In Vietnam, a number of frequency distributions were analyzed specifically using the stage data at the Thuong Cat gauging station. The Pearson III frequency distribution was selected for application by Institute of Water Resources Planning. The results present a reasonable depiction of stage frequencies at the Thuong Cat gauging station for the data used (Marshall, 2002). The stage frequencies at Thuong Cat gauging station developed by Institute of Water Resources Planning are presented in the Table 3.1

Table 3.1 Pearson III River Stage Recurrence Intervals at the Thuong Cat gauging station on the Duong River by Institute of Water Resources Planning*

Cumulative probability	**Probability of being equaled or exceeded in any given year**	**Return period** (years)	**River flood levels by Pearson III distribution** (meter)
0.5	0.5	2	10.8
0.8	0.2	5	11.7
0.9	0.1	10	12.4
0.95	0.05	20	12.93
0.98	0.02	50	13.64
0.99	0.01	100	14.16
0.995	0.005	200	14.66
0.996	0.004	250	14.88
0.998	0.002	500	15.32
0.999	0.001	1,000	15.81

*Source: "Feasibility study for The Duong River flood protection project"

During this study, in order to find out a proper distribution, which can be easily applied for probabilistic calculations but still has enough essential accuracy, the exponential distribution is analyzed using the data at Thuong Cat gauging station. Analyses were done by two methods: the method of linear regression and the method of moments. Detail calculations are given in Appendix I, only the obtained equations of the exponential distribution are presented below:

By the method of linear regression:

$$F(H) = 1 - e^{-\frac{H-9.8}{1.06}}$$

By the method of moments:

$$F(H) = 1 - e^{-\frac{H-9.95}{0.86}}$$

The river flood levels determined by two these distributions are shown in Table 3.2 together with the results obtained by Institute of Water Resources Planning in Vietnam.

Table 3.2 River Stage Recurrence Intervals at the Thuong Cat gauging station on the Duong River using Exponential distribution

Cumulative probability	Prob. of being equaled or exceeded in any given year	Return period (years)	River flood level by Pearson III distribution (meter)	River flood level by Exponential distribution* (meter)	River flood level by Exponentail distribution ** (meter)
0.5	0.5	2	10.80	10.53	10.55
0.8	0.2	5	11.70	11.51	11.33
0.9	0.1	10	12.40	12.24	11.93
0.95	0.05	20	12.93	12.98	12.53
0.98	0.02	50	13.64	13.95	13.31
0.99	0.01	100	14.16	14.68	13.91
0.995	0.005	200	14.66	15.42	14.51
0.996	0.004	250	14.88	15.65	14.70
0.998	0.002	500	15.32	16.39	15.29
0.999	0.001	1,000	15.81	17.12	15.89

Notes: * : by the method of linear regression and **: by the method of moments

From the calculated results it is found that river stages for different recurrence intervals calculated using the Pearson III frequency distribution are slightly different compared to the recurrence intervals calculated using the exponential distribution and the method of moments. In the case of using the method of linear regression the difference is quite big. Because of the simplicity and applicability of the exponential distribution in probabilistic calculation, instead of using Pearson III distribution the exponential distribution is used in this study. The choice of exponential distribution is based on the comparison between the calculated values and the values of the Person III distribution since the application of Pearson III distribution for Thuong Cat gauging station is widely used in Vietnam for its accuracy in the Duong River condition. The exponential distribution calculated using the method of moments gives the values closer to the values of Person III than using the method of linear regression. Therefore, the exponential distribution calculated using the method of moments is used in this study. The distributions for high water levels at Thuong Cat using Exponential distribution and the method of moments are shown in Figures 3.2, 3.3, and 3.4.

The distribution function: $$F(H) = 1 - e^{-\frac{H-9.95}{0.86}} \quad (3\text{-}1)$$

The density function: $$f(H) = \frac{1}{0.86} e^{-\frac{H-9.95}{0.86}} \quad (3\text{-}2)$$

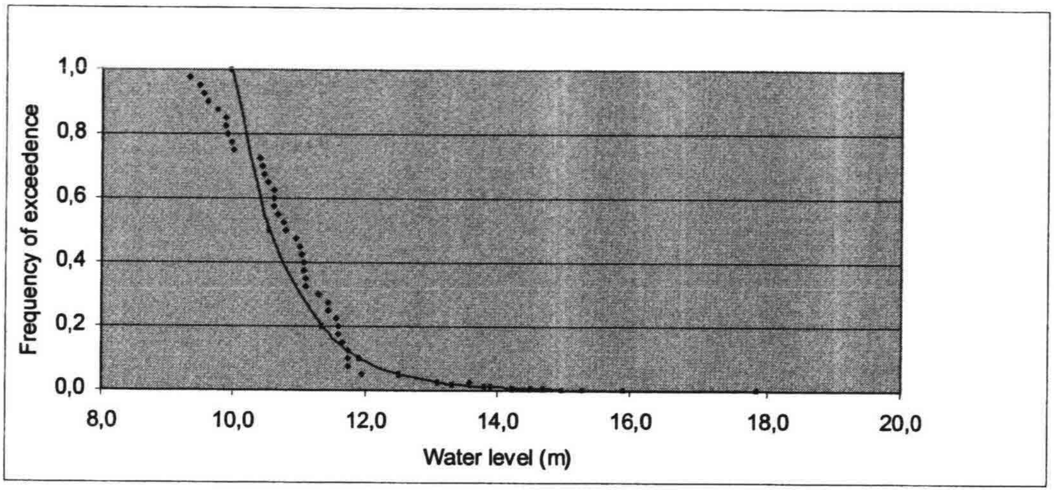

Figure 3.2 High water level frequency curve at Thuong Cat gauging station by using the Exponential distribution (method of moments)

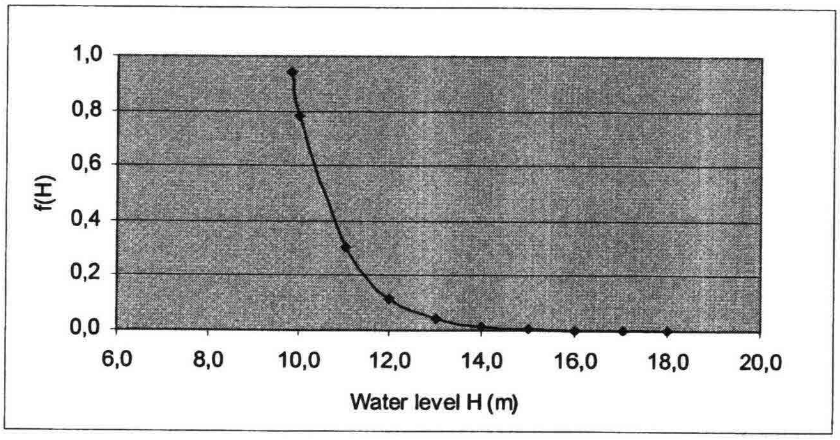

Figure 3.3 Probability density function of the highest water level at Thuong Cat

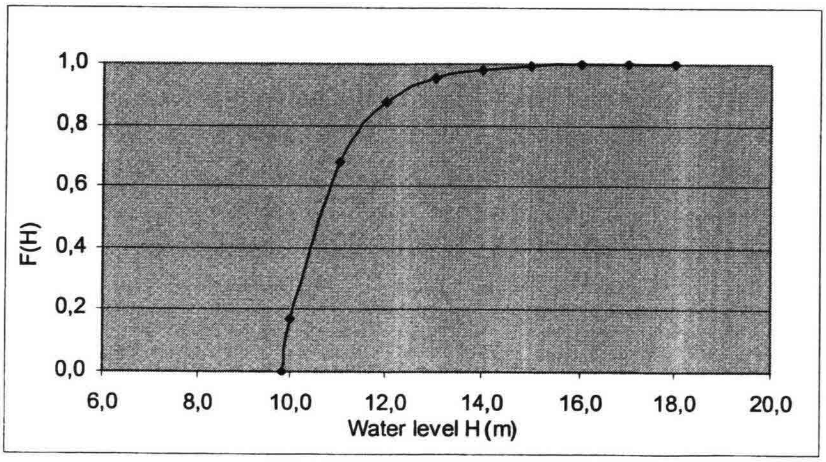

Figure 3.4 Distribution function of the highest water level at Thuong Cat

3.2 DIKE CHARACTERISTICS

Figure 3.5 A dike segment of the right Duong River dike

The right Duong River dike system has a total length of 59.6 km. The Duong River dike system contributes significantly to flood control in the Red River Delta. The right Duong River dikes protect 180,000 hectares of land, which is almost rural agricultural land. The right Duong River dike system also protects 2,5 million people, National Highway No. 5, the National Railway connecting Hanoi City to Hai Phong City, and Gia Lam Airport...Significant investment has been made and great efforts have been expended by the Ministry of Agriculture and Rural Development (MARD) and the People's Committees of Hanoi City and Bac Ninh Province to operate and maintain the Duong River dike system. However, there are still many deficient flood protection issues associated with the Duong River dike system as it currently exists.

3.2.1 Dike geometry

Of the 59.6 Km of dike along the right bank of the Duong River, there are 20.6 Km of dike in which the dike cross section does not meet design standards. Also, there are many Duong River dike sections where the existing dike top or crown width is smaller than the design width of 6.00 m, and where the land side slope is less than the design value of 1: 3 (vertical: horizontal). In Vietnam, the Duong River dike is classified as Grade I. According to Vietnamese norm, if a Grade I river dike is used for transportation, the crest width of the dike must be 6 m. Another reason for choosing this value is that the high water level period during flood seasons is quite long for river dikes in general and for Duong River dike in particular. So the bigger dike body the better dike stability against piping and seepage. The locations of inadequate dike crest width based on a design width of 6 m are summarized in Table 3.3.

3.2.2 Dike Berm

A dike berm is often built for added dike slope stability, and to lengthen the seepage path through and under the dike. For the Duong River dike system, there are many dike sections where no dike berms are constructed.

Table 3.3 Deficient dike crest width along the Duong River dike system

Dike Section Hanoi City	Dike Crest Width (m)	Dike Section Bac Ninh Province	Dike Crest Width (m)
Km 0.000 to Km 2.868	1.20	Km 21.600 to Km 21.727	2.00
Km 2.868 to Km 3.972	1.40	Km 23.121to Km 28.499	5.00
Km 3.972 to Km 4.542	2.00	Km 31.078 to Km 31.621	5.40
Km 4.542 to Km 4.851	2.50	Km 31.621 to Km 38.459	2.00
Km 4.851to Km 6.777	1.50	Km 38.515 to Km 41.109	5.00
Km 6.777 to Km7.789	2.00	Km 41.109 to Km 41.337	4.60
Km 7.789 to Km 8.055	3.50	Km 41.109 to Km 47.798	5.00
Km 8.055 to Km 9.067	2.00	Km 54.287 to Km 54.787	5.00
Km 9.067 to Km 9.440	1.50		
Km 9.440 to Km 10.412	2.00		
Km 10.412 to Km 10.817	3.00		
Km 10.817 to Km 11.429	5.00		
Km 11.429 to Km 13.213	4.50		
Km 13.213 to Km 21.600	5.00		

Table 3.4 Deficient dike berms,right bank of the Duong River

Dike Section	Location	Length of Dike (Km)
Dike Landside Berm		
Km 26.100 to Km 26.700	Bac Ninh	0.600
Km 31.500 to Km 32.400	Bac Ninh	0.900
Km 53.000 to Km 55.060	Bac Ninh	2.060
Km 59.100 to Km 59.600	Bac Ninh	0.500
Dike Riverside Berm		
Km 00.000 to Km 16.500	Hanoi	16.500
Km 21.600 to Km 23.100	Bac Ninh	1.500
Km 27.600 to Km 31.400	Bac Ninh	3.800
Total		**25.860 Km**

3.2.3 Ponds, swamps, and below grade (hollow) agricultural fields

There are many water ponds and swamp areas located very near to the inner toe of dike along the Duong River. These ponds and swamps shorten the seepage path under the dike and can contribute to excessive foundation seepage and the possibility of dike foundation failure from piping. An inventory of ponds and swamps along the Duong River dike system is summarized in Table 3.5.

In landside areas, there are also many below grade elevation agricultural fields called hollow fields. These agricultural areas are generally large and deep due to historical consequences of dike breaks. Below grade agricultural fields are located at Thuong Cat, Thanh An, Vang Loi, Dong Xuyen, and Sen Ho, on the right bank of the Duong River dike.

Table 3.5 Inventory of ponds and swamps, right side of Duong River

Dike Section	Province	Inventory of Ponds and Swamps
Landside Areas		
Km 01.000 to Km 03.750	Hanoi	Ponds
Km 21.600 to Km 24.000	Bac Ninh	Ponds/Swamps
Km 28.000 to Km 29.500	Bac Ninh	Ponds/Swamps
Km 45.000 to Km 47.000	Bac Ninh	Ponds
Riverside Area		
Km 01.000 to Km 03.750	Hanoi	Ponds
Km 21.600 to Km 24.000	Bac Ninh	Ponds/Swamps

3.2.4 Dike Foundation

The foundation conditions under the Duong River dike system are poor to very poor. Topsoil forms a thin impermeable soil layer from 1m to 3m thick. The deeper subsoil layer is a permeable layer or is composed of poor soil including sand or silty sand. Large volumes of seepage water often flow from the river through this layer to landside ponds, swamps, and below grade (hollow) agricultural fields. This results in embankment stability problems caused by piping

3.3 IMPORTANT PROBLEMS OF THE DUONG RIVER DIKE

Based on the inventory of deficient dike geometry and poor foundation soils presented above, the following problems often occur during the flood season along the Duong River dike:

Seepage often occurs through the body of dikes or under the toe of dikes during the high water level period. These seepage problems are often found in locations where the dikes have no berms on the riverside and where there are of ponds, swamps, and below grade hollow agricultural fields located very near to the toe of the dike. Seepage problems also occur where there are voids in the dike body and where the dike embankment is constructed of sandy clay. The location of Duong River dike sections with critical seepage problems is given in Table 3.6.

Table 3.6 Riverside dike sections with serious seepage problems on the right bank of the Duong River

No.	Dike Sections	Province	Length (Km)
1	Km 21.600 to Km 24.000	Bac Ninh	2.400
2	Km 28.500 to Km 29.500	Bac Ninh	1.000
3	Km 30.000 to Km 31.000	Bac Ninh	1.000
4	Km 36.000 to Km 38.000	Bac Ninh	2.000
5	Km 45.000 to Km 49.000	Bac Ninh	4.000
5	Km 53.000 to Km 59.600	Bac Ninh	6.600
	Total		**23.800 Km**

The higher the flood water level increases, the more critical the seepage problem becomes. If high water flood conditions persist over a long period, the landside slope may erode and fail. This condition exists from Km 41.000 to Km 45.000 on the right bank of the Duong River dike. Also, sand boils and piping problems in landside dike sections are often observed at dike sections where there are ponds, swamps, and below grade (hollow) agricultural fields in both landside and riverside areas. The dike sections having critical sand boil and piping problems are given in Table 3.7.

Table 3.7 Landside dike sections having piping problems on the Duong River dike

Left Bank of the Duong River Dike	Right Bank of the Duong River Dike
Km 21.600 to Km 23.000	Km 23.500 to Km 24.500
Km 23.500 to Km 24.000	Km 40.000 to Km 45.000
Km 45.000 to Km 47.000	
Km 57.000 to Km 59.600	

More critically, a very large number of sand boils and piping covering several tens of square meters were found at these dike locations, 30 to 50 m from the toe of the dike. For example, during the flood season in year 2000, a number of seepage problems and sand boil problems were observed along the Duong River dikes as follows:

1. Nine sand boils were observed on 24 July 2000 in Bac Ninh Province. These sand boils were 30 m to 75 m from the toe of the dike.
2. Sand boils were observed on 27 July 2000 at Km 23.290 and Km 23.500. These sand boils and piping were 30 m from the toe of dike.
3. Two sand boils were observed on 27 July 2000 at Km 29.450 and Km 29.455. The sand boils were 35 m from the toe of the dike.
4. Sand boils with average diameters of 10 cm were observed on 08 August 2000 at Km 19.600. The sand boils were 30 m from the toe of the dike.

3.4 GEOTECHNICAL INFORMATION

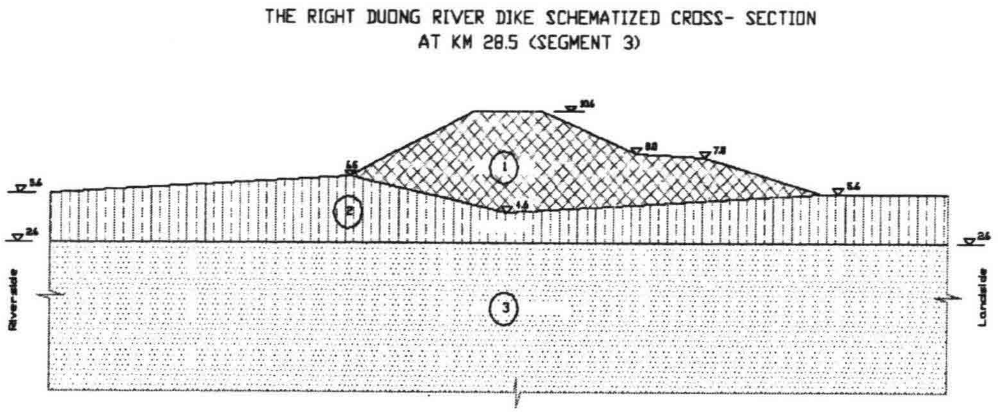

Figure 3.6 Soil investigation for the right Duong River dike

The soil investigation was done along the dike. At most dike cross-sections, from the dike surface until 18m in depth it was found having mainly 3 layers (Appendix III):

- The top layer (layer 1): This is the artificially deposited soil layer lying on the top. The composition of this soil is quite complicated but it consists mainly of clay, sand, and silt. The color of this soil is brown or brownish.
- The second layer (layer 2): Lying just below the top soil layer. This is the sandy clay layer. The composition of this layer is also complicated. The main composition is pink-brown clay. At some places it is interposed with thin layer of grey clay or fine sand. Normally, the thickness of this layer reduces from the riverside until the landside. This factor is not good for the stability of the dike against seepage and piping.
- The third layer (layer 3): Lying under layer 2. At most sections, this layer is sand, only at segment 3 (See appendix III for details)) clay was found at this layer. The sand grain size varies from layer to layer. At some sections it is fine sand but at the others it is coarse sand. The thickness of this layer is undefined.

From the results of the soil investigation, soil parameters for each layer in segment 3 are summarized in Table 3.8. Soil properties for all layers at 6 sections along the dike are given in the Appendix III.

Table 3.8 Average values of soil parameters (segment 3)

Parameters	**Layer 1**	**Layer 2**	**Layer 3**
Water content (%)	28.0	32.0	18.7
Wet density (T/m^3)	1.87	1.83	1.70
Dry density (T/m^3)	1.47	1.34	1.40
Solid density (T/m^3)	2.75	2.72	2.69
Porosity (%)	46.5	46.4	48.50
Degree of saturation (%)	84.3	92.5	100
Liquid limit (W_L)	36.9	39.5	-
Plastic limit (W_P)	23.6	24.0	-
Consistency index (I_C)	13.3	15.5	-
Cohesion (kN/m^2)	21.1	18.0	0
Internal friction angle (degree)	15^0	13^0	26^0
Permeability (m/s)	$6.8x10^{-7}$	$5.5x10^{-7}$	$7.8x10^{-5}$

3.5 HYDRAULIC STRUCTURE CHARACTERISTICS

Figure 3.7 A typical conduit under the right Duong River dike

Conduits exist under the right Duong River dike at Vang, Loi, Phu My, Mon Quang, Ngam Mac, and Tram. All these conduits were built in the end of 1950s and in 1960s. Until now, not all of the conduit are in good condition. At some conduits, cracks were found in the concrete structure. In 1997, the quality assessment of these conduits was done under the request of Ministry of Agriculture and Rural Development but at a simple level. Summarize of the result of the assessment is given in Appendix IV.

3.6 METEOROLOGICAL DATA

3.6.1 Rainfall

The average rainfall for years in the study area

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Rainfall (mm)	22.2	27.1	42.1	105	173.5	255.7	256.2	286.9	247	156.7	72.7	15.8	1661

3.6.2 Wind speed

Wind direction and the maximum observed wind speed in a year in the study area (m/s)

Month	Jan	Feb	Mar	Apr	May	Jun
Value (m/s)	15	15	15	20	30	20
Direction	NE	NE	NNE	W	SW	NNE

Month	Jul	Aug	Sep	Oct	Nov	Dec
Value (m/s)	28	31	28	19	22	18
Direction	NW	NE	ENE	NE	NE	NE

3.7 ECONOMIC DATA

3.7.1 Damage assessment through history flooding

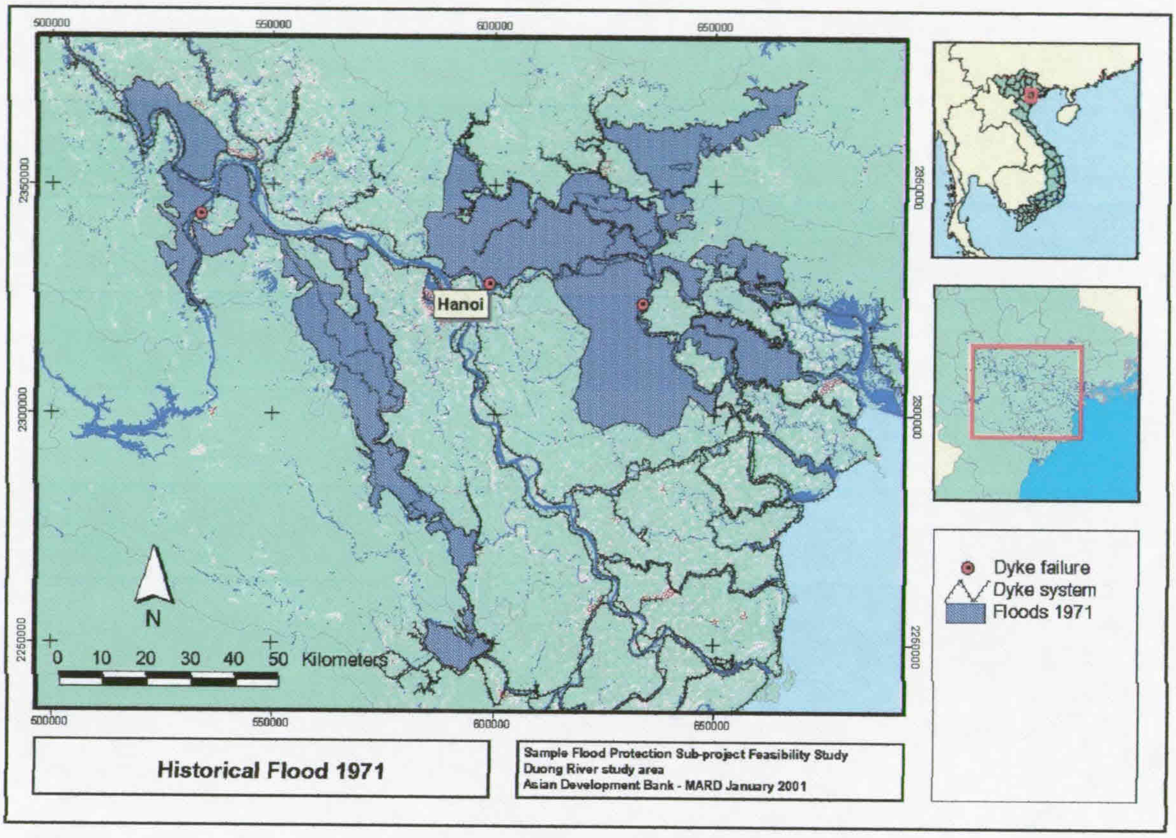

Figure 3.8 Extent of 1971 flooding in the Red River Delta of Vietnam

The study area was affected greatly by the historic flood that occurred in 1971 on the Red River when several dike failures in the study area occurred. The extent of this flood was large compared to the previous ones. It included the study area of this study and also of a number of other regions in different provinces. Estimates of damage caused by this 1971 flood if it were to occur today were calculated for *the area* located on both sides of the Duong River dikes, not for all the flooded regions. Additionally, damage was estimated for a flood covering the entire area (Marshall Silver & Associates, 2002). Damage estimates for a 1-meter deep flood are presented in Table 3.9 and damage estimates for a 1/3-meter deep flood are presented in Table 3.10.

Table 3.9 Damage estimates for the historic flood of 1971 and a flood covering the whole area based on year 2000 prices and a 1-meter depth flood.

Flood losses for 1 meter deep flood	1971	Whole area	1971	Whole area
	(million VND)		(Million US$)	
Crop losses	1,413,000	2,827,000	$99	$198
Livestock losses	363,000	725,000	$25	$51
Housing losses	1,031,000	2,043,000	$72	$143
Commerce losses	12,000	24,000	$1	$2
Highways	38,000	66,000	$3	$5
Major roads	1,000	1,000	$0.07	$0.07
Industrial assets damage	2,707,000	3,090,000	$189	$216
Industrial output losses	1,019,000	1,190,000	$71	$83
Total	6,584,000	9,966,000	$460	$697

Table 3.10 Damage estimates for the historic flood of 1971 and a flood covering the whole area based on year 2000 prices and a 1/3-meter depth flood.

Flood losses for 1/3 meter deep flood	1971	Whole area	1971	Whole area
	(Million VND)		(Million US$)	
Crop losses	707,000	1,413,000	$49	$99
Livestock losses	161,000	322,000	$11	$23
Housing losses	412,000	817,000	$29	$57
Commerce losses	7,000	13,000	$0	$1
Highways	28,000	49,000	$2	$3
Major roads	0	1,000	$0.00	$0.07
Industrial assets damage	1,354,000	1,545,000	$95	$108
Industrial output losses	764,000	892,000	$53	$62
Total	3,433,000	5,053,000	$240	$353

Source: Feasibility study for flood protection projects in the Red River Delta of Vietnam (Marshall, 2002)

The Gross Domestic Product (GDP) for Vietnam was VND 361,500 billion measured in 1998 prices as reported in the *1998 Vietnam Statistical Yearbook*. Flood damage in the area from a repeat of the 1971 flood assuming a 1-meter deep flood would equal 1.8 percent of the total national GDP. The Department of Dike Management and Flood Control reported that flood of 1971 was 3.5 meters deep in some parts of the study area

(Marshall Silver & Associates, 2002). This indicates the serious threat that flooding poses for Vietnam, especially considering that the above estimated damage calculated is only for the Duong River area and does not include the total flooded area of the Red River Delta of the historical flood analyzed.

3.7.2 Construction cost for dike

The unit price for 1 m^3 of embankment of dike is taken at 3.0 US$. This price is based on the estimation by the Department of Dike Management and Flood Control of MARD.

3.7.3 General economic information

The interest rate is taken as 0.04 for the Vietnamese situation
The exchange rate: 1 US$ = 14.500 Vietnamese Dong.

4 CHAPTER IV: FAILURE PROBABILITY CALCULATIONS

4.1 STATEMENT OF THE PROBLEM

The purpose of failure probability calculations is to make possible a prediction of the probability of inundation of the study area protected by the Duong River dike system. In addition, failure probability calculations serve as an aid in determining the risk of inundation of the study area as the risk equals probability multiplied by consequences. Of many mechanisms causing inundation of the area behind the dike, a number will be taken into account, which are believed as the most important failure mechanisms of the Duong River dike, namely:

- Overflowing
- Piping
- Macro-instability

Besides that, failure probabilities caused by *hydraulic structures* are also considered.

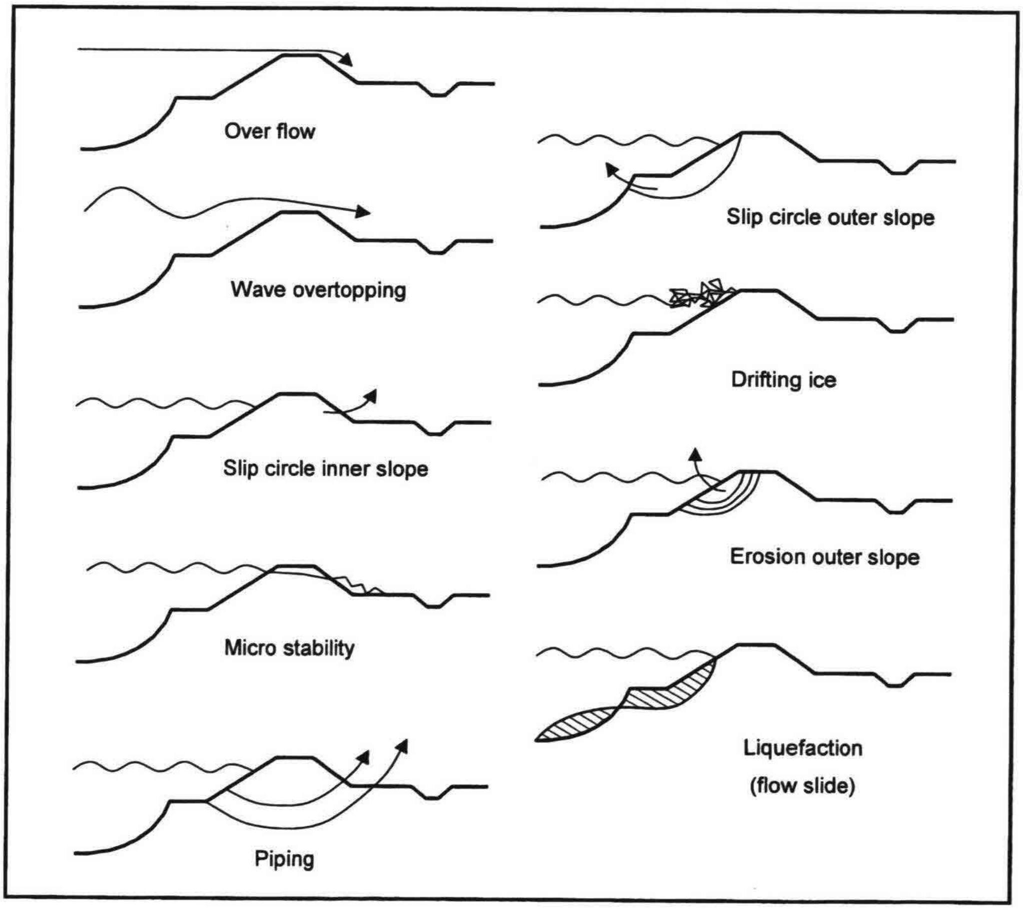

Figure 4.1 Failure mechanisms for dikes

4.2 LENGTH EFFECT

Experience shows that overflowing and wave overtopping of the crest of a dike occurs only over a limited length (though possibly in several places along the crest) and that sliding soil masses are also of limited length. These circumstances lead to the conception of dividing the dike into segments, all of which have a probability of failure depending on the strength parameters (crest level, shearing resistance, etc.) In this approach the dike is conceived as consisting, over its entire length, of a series system of consecutive segments. The system fails if, for at least one segment, the load exceeds the strength. As the dike is longer, the probability of this occurring can be expected to be higher. For the Duong River dike system, in order to take into account these effects, the whole dike, which is 59.6 km in length, is divided in segments as follows:

Table 4.1 Division of the dike into segments

No	Segment	Length (km)	Conduit in the segment	Chosen critical section
1	Km 0 to Km 10	10	Keo Go conduit at Km 5+572	At Thuong Cat
2	Km 10 to Km 20	10	Loi conduit at Km 13+027	At Km 10.5
3	Km 20 to Km 30	10	Phu My conduit at Km 25+520	At Km 28.5
4	Km 30 to Km 40	10	Mon Quang conduit at Km 37+540	At Km 37.0
5	Km 40 to Km 50	10	Tram conduits at Km 44+500	At Km 45.0
6	Km 50 to Km 59.6	9.6	No conduit	At Km 54.5

Since it is impossible to predict the exact location or extent of a dike failure, a critical section of the dike segment was chosen as representative of the whole dike segment for calculations. Critical sections chosen are the ones, which have deficient dike crest elevations, width, piping or seepage problems. Based on the data and information about the present condition of the right Duong River dike, the locations of the critical sections are chosen as follows:

<u>For segment 1:</u>
At Thuong Cat -Km 1.8, this section was chosen for two reasons. First, this section is the location of the Thuong Cat gauging station. Second, this section is located near critical sections where the dike crest and width are deficient.

<u>For segment 2:</u>
At Km 10.5, the crest width of the dike at this location is only 3 meters.

<u>For segment 3:</u>
At Km 28.5, the dike at this location has deficient crest width together with serious seepage problems.

<u>For segment 4:</u>
The critical section was chosen at Km 37 where the dike has problems with deficient crest width and serious seepage.

<u>For segment 5:</u>
At Km 45, the dike at this location has problems with deficient dike crest width, serious seepage, and piping.

<u>For segment 6:</u>
At Km 54.5, the dike at this location has problems with serious seepage and deficient dike crest width.

All these critical dike cross- sections are shown in the appendix III.

The failure probability for each segment comprises of the failure probabilities of mechanisms and of the hydraulic structure in that segment. The name and location of the conduit for each segment, which is taken into consideration, is given in Table 4.1

4.3 CALCULATIONS WITH MECHANISMS

Calculations of failure probabilities by mechanisms will be done for each segment. If the failure probability caused by any mechanism for one segment is known then the failure probability of the segment, which is caused by all mechanisms, can be calculated. Eventually the failure probability of the whole dike, which comprises of all consecutive segments, also can be calculated. Since the calculation process for each segment follows the same principle so the explanation is only given for segment one, of which the critical section is at Thuong Cat gauging station.

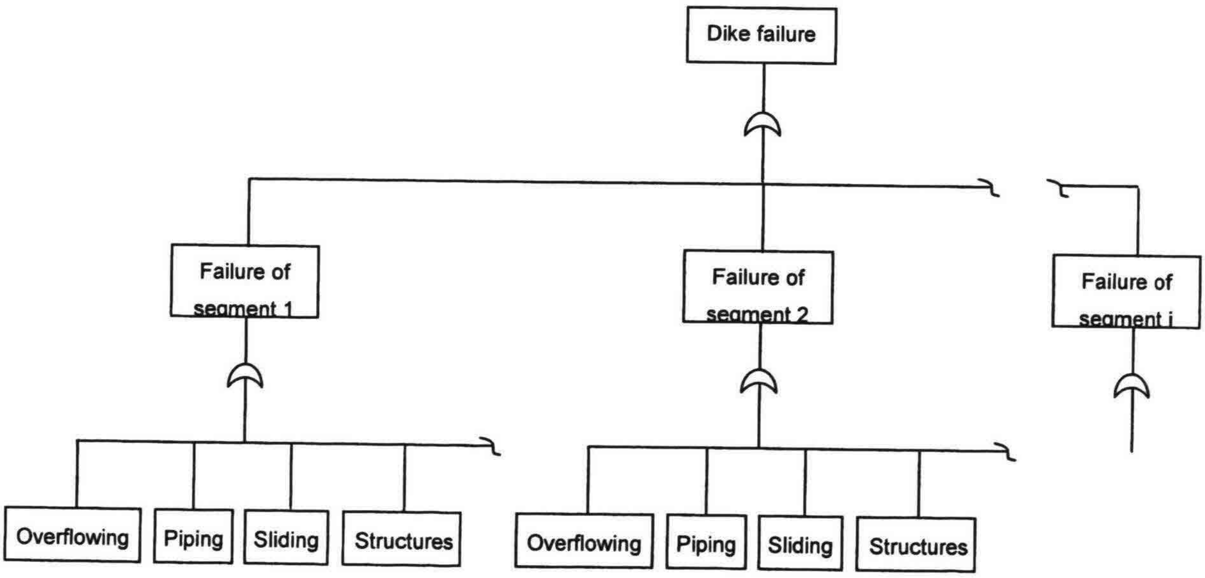

Figure 4.2 Fault tree of the dike

4.3.1 Overflowing mechanism

a. Mechanism of overflowing

If the water level at a dike is higher then the crest level of the dike, ingress of water into the region protected by it will increase and inundation occurs (Figure 4.3).

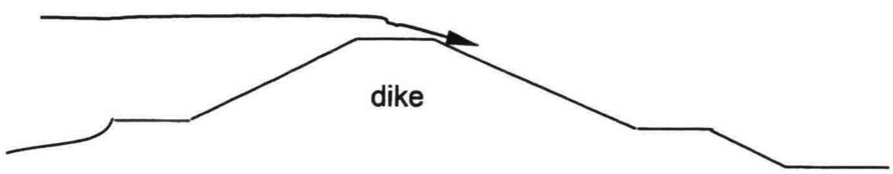

Figure 4.3 Mechanism of overflowing

b. Reliability function for overflowing

With regard to the mechanism "overflowing", the probability of the high water level at the dike exceeding the level of the crest is investigated. Failure can be said to occur if the water level in the river (H) higher than the crest level of the dike. The reliability function then becomes:

$$Z = h_c - H \tag{4-1}$$

Where:

H	=	high water level
h_c	=	crest level of the dike

c. Calculation procedure

- Distributions for water levels at sections (H):

Since the calculation process for all segments is almost the same, explanation is only given for the segment 1. The section chosen for segment 1 during this calculation is located at Thuong Cat gauging station. According to hydrologic data analysis in chapter III, the highest water levels at Thuong Cat gauging station on the Duong River during this study was taken as an exponential distribution:

$$F(H) = 1 - e^{-\frac{H-9.95}{0.86}} \tag{4-2}$$

River flood frequency intervals on the Duong River calculated at the critical section of each segment are shown in Table 4.2. Details of this work are expressed in Appendix I.

Table 4.2 River flood levels calculated for the critical section of each segment using Exponential distribution

Design return period of water level (years)	River flood levels (m)					
	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
2	10.55	9.56	7.51	6.54	5.63	4.54
5	11.33	10.34	8.29	7.32	6.41	5.32
10	11.93	10.94	8.89	7.92	7.01	5.92
20	12.53	11.54	9.49	8.52	7.61	6.52
40	13.12	12.13	10.08	9.11	8.20	7.11
50	13.31	12.32	10.27	9.30	8.39	7.30
100	13.91	12.92	10.87	9.90	8.99	7.90
150	14.26	13.27	11.22	10.25	9.34	8.25
200	14.51	13.52	11.47	10.50	9.59	8.50
250	14.70	13.71	11.66	10.69	9.78	8.69
500	15.29	14.30	12.25	11.28	10.37	9.28
1000	15.89	14.90	12.85	11.88	10.97	9.88

- Determination of dike crest levels

Since calculations for the mechanisms will be done with the dike crest levels, and not with design water levels. The dike crest levels have to be determined for the future conditions when the dike crest is heightened:

Crest level = Design water level + freeboard

If the effect of soil settlement is left out then the freeboard is equal to the run-up and wind set-up. In this case one gets:

Crest level = Design water level + run-up + wind set-up (4-3)

Normally, 2% run-up is applied for the design of a dike. This means that the height of the dike should be such high that less than 2% of the wave run-up tongues should reach the crest of the dike. The background of this idea is that the quantity of water is in that case small enough to guarantee that the overtopped discharge will not cause any damage to the inner slope. The general design formula that can be applied for wave run-up on dikes in this deterministic case can be expressed by (Dikes and Revetments, Pilarczyk, 1998):

$$\frac{R_{2\%}}{H_s} = 1.6\gamma_b\gamma_f\gamma_\beta\zeta_{op} \quad \text{with a maximum of } 3.2\,\gamma_b.\gamma_f.\gamma_\beta \qquad (4\text{-}4)$$

Where:

γ_b = reduction factor due to berm
γ_f = reduction factor due to slope roughness and permeability
γ_β = reduction factor due to oblique wave attack
γ = $\gamma_b \gamma_f \gamma_\beta$ = total reduction factor
H_s = significant wave height (m)
ξ_{op} = breaker parameter for the peak –period and is determined by:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\dfrac{2\Pi H_s}{gT_p^2}}} \qquad (4\text{-}5)$$

In which, α is the angle of the outer slope (degree) and T_p is the peak wave period (second).

The determination of 2% run-up for this deterministic case was done in the Appendix II. The value of 2% run-up according to calculations in Appendix II is:

$$R_{2\%} = 1.62 \text{ m}$$

The relative rise in water level due to wind effect (W_s) can be calculated by the following formula (Revetments, sea- dikes and river levees, Verhagen, IHE lecture notes):

$$W_s = \frac{1}{2}C_w \times \frac{\rho_{air}}{\rho_{water}} \times \frac{U^2 F}{gh}\cos\phi \qquad (4\text{-}6)$$

In which:

C_w = coefficient (from $0.8*10^{-3}$ to $3*10^{-3}$)
ρ_{air} = density of air ($1.25 kg/m^3$)
ρ_{water} = density of water ($1000 kg/m^3$)
U = wind velocity (m/s)
F = fetch length (m)
g = acceleration of gravity (m/s^2)
h = water depth (m)
ϕ = angle between wind direction and axis (degrees)

In current practice, the value of W_s is very small and can be ignored since the wind speed is small and the fetch length is also quite short. At most sections, the river width is only few hundred meters. For example, if u= 15 m/s, F= 3500 m, h= 3.5 m, ϕ= 0, c_W= $2*10^{-3}$ then the value of W_s is equal to 0.028 m. This value compared to the values caused by other effects is small.

The design crest levels that determined by equation (4-3) with corresponding return periods are as in Table 4.3. At the Thuong Cat gauging station, the present dike crest level is 13.8m (Department of Dike Management and Flood Control - Vietnam). If it is considered that this crest level includes the 2% run-up = 1.62m as calculated above, then the design water level for the existing dike is 13.8 – 1.62 = 12.18m. This design water level corresponds to the return period T ≈ 20 years according to flood frequency analysis using an exponential distribution (Table 4.2)

- Distribution of the dike crest level

It is assumed that the dike crest level follows a normal distribution. From experience it is found that if the standard deviation σ is taken as 0.1÷ 0.2 (m) and the mean value is taken as the design level of the dike crest, the normal distribution can reflect the actual condition of the dike crest level. Based on the present condition of the right Duong River dike the standard deviation is taken as σ = 0.15 and the mean values for dike segments at the critical sections were determined from the data of the dike geometry.

Table 4.3 Design crest levels at the critical sections along the right Duong River dike

Design return period of water level (years)	Dike crest levels (m)					
	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
20 (present)	13.80	13.25	10.60	10.10	9.55	8.35
50	14.93	13.94	11.89	10.92	10.01	8.92
100	15.53	14.54	12.49	11.52	10.61	9.52
150	15.88	14.89	12.84	11.87	10.96	9.87
200	16.13	15.14	13.09	12.12	11.21	10.12
250	16.32	15.33	13.28	12.31	11.40	10.31
500	16.91	15.92	13.87	12.90	11.99	10.90
1000	17.51	16.52	14.47	13.50	12.59	11.50

An overview of the problem variables for this mechanism is shown in Table 4.4.

Table 4.4 Overview of the problem variables (overflowing)

Segment	Variables	Description	Type	Unit	μ	σ
1	H	River flood level	E	m	10.81	0.86
	h_c	Dike crest level at present	N	m	13.8	0.20
2	H	River flood level	E	m	9.82	0.86
	h_c	Dike crest level at present	N	m	13.25	0.20
3	H	River flood level	E	m	7.77	0.86
	h_c	Dike crest level at present	N	m	10.60	0.20
4	H	River flood level	E	m	6.80	0.86
	h_c	Dike crest level at present	N	m	10.10	0.20
5	H	River flood level	E	m	5.89	0.86
	h_c	Dike crest level at present	N	m	9.55	0.20
6	H	River flood level	E	m	4.80	0.86
	h_c	Dike crest level at present	N	m	8.35	0.20

Note: E: = exponential distribution, N = normal

The distributions of the stochastic variables in the reliability function are known. It is possible to calculate the failure probabilities. The VAP model, which was developed by Institute of Structural Engineering IBK- ETH Zurich- Switzerland, was used to calculate the failure probabilities caused by overflowing. With the VAP model, a probabilistic analysis of a reliability function G (appendix V), which is the function of a number of stochastic variables, can be carried out at level II. Calculations were done for the present dike crest level and the dike crest levels when the dike is heightened corresponding to the return periods of water levels T = 50, 100, 150, 200, 250, 500, and 1000 years. The results are shown in Table 4.5.

Table 4.5 Failure probabilities caused by overflowing

Design return period of water level (years)	Failure probability					
	Seg. 1	**Seg. 2**	**Seg. 3**	**Seg. 4**	**Seg. 5**	**Seg. 6**
20 (present)	1.15×10^{-2}	6.91×10^{-3}	1.39×10^{-2}	8.04×10^{-3}	5.29×10^{-3}	6.01×10^{-3}
50	3.10×10^{-3}	3.10×10^{-3}	3.10×10^{-3}	3.10×10^{-3}	3.10×10^{-3}	3.10×10^{-3}
100	1.54×10^{-3}	1.54×10^{-3}	1.54×10^{-3}	1.54×10^{-3}	1.54×10^{-3}	1.54×10^{-3}
150	1.03×10^{-3}	1.03×10^{-3}	1.03×10^{-3}	1.03×10^{-3}	1.03×10^{-3}	1.03×10^{-3}
200	7.68×10^{-4}	7.68×10^{-4}	7.68×10^{-4}	7.68×10^{-4}	7.68×10^{-4}	7.68×10^{-4}
250	6.16×10^{-4}	6.16×10^{-4}	6.16×10^{-4}	6.16×10^{-4}	6.16×10^{-4}	6.16×10^{-4}
500	3.10×10^{-4}	3.10×10^{-4}	3.10×10^{-4}	3.10×10^{-4}	3.10×10^{-4}	3.10×10^{-4}
1000	1.54×10^{-4}	1.54×10^{-4}	1.54×10^{-4}	1.54×10^{-4}	1.54×10^{-4}	1.54×10^{-4}

Some comments on the result:

- From the calculated results of failure probabilities caused by overflowing in Table 4.5, it is found that: all the failure probabilities corresponding to the return period of 50 years until 1000 years are equal. This is because of the linear interpolation of water level based on the river slope from the Thuong Cat gauging station. In actual design, the water level at these locations has to be determined by more accurate methods such as an analytical method or by using a flow model, etc.

- The failure probability decreases with increasing dike crest level.

4.3.2 Piping mechanism

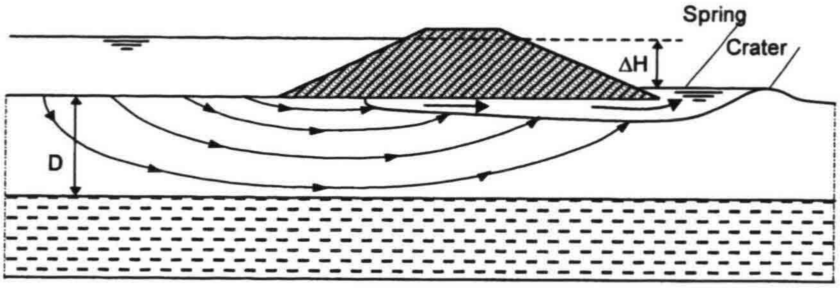

Figure 4.4 Mechanism of piping

a. Mechanism of piping

Piping under dikes occurs as a result of the continuous transport of soil particles by the erosive action of seepage flow. The piping phenomenon is preceded by the formation of boils discharging water in which sand is carried along. Such boils, which manifest themselves especially at periods of high water level retained by the dike, are frequently to be observed along the dike (Figure 4.4).

b. Reliability function for piping

According to Bligh, with regard to piping and underflow associated with rock-fill dams on the basis of a statistical analysis of such structures which had and which had not failed, a minimum necessary seepage path length L_k under the structures was determined (Probabilistic design of flood defences, CUR, 1990):

$$L_K = c^* \Delta H \qquad (4\text{-}7)$$

Where ΔH is the total head loss (overall difference in water level) across the structure and c is a coefficient depending on the soil type.

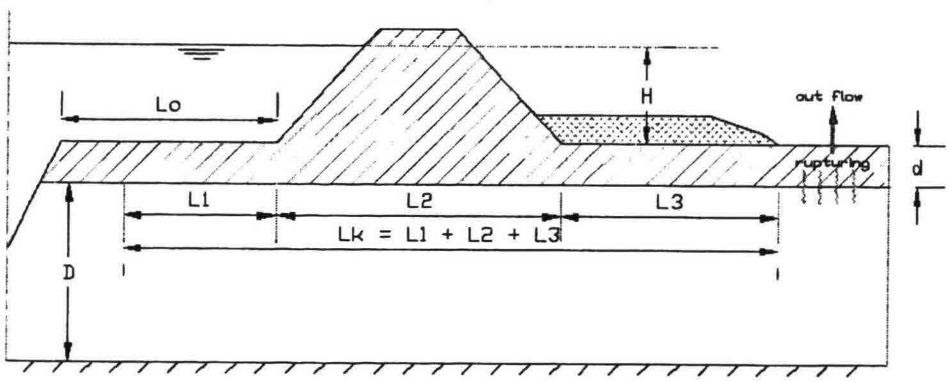

Figure 4.5 Determination of minimum seepage path length

The entry point of the groundwater flow is taken at a distance $L' = \lambda \tanh(L_1/\lambda)$ from the outer toe of the dike. In this formula $\lambda = \sqrt{k.D.c}$ is the dispersion length, while k denotes the permeability of the subsoil, D the thickness of the water-bearing stratum and $c = d/k_V$ the resistance of the top layer (with thickness d and permeability k_V).

For the failure mechanism of piping to occur, two conditions must be satisfied:

- The clay layer under the dike must be ruptured (in case of having a clay layer)
- Continuous transport of sand must take place

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:

$$Z_1 = \rho_c.g.d - \rho_w.g.(H \pm h_b) \qquad (4\text{-}8)$$

Where:

ρ_c = density of the wet clay
g = gravity acceleration = 10 m/s^2
d = the thickness of the clay layer between bottom of the dike and sand layer
h_b = distance from the datum to the bottom of clay layer. Taking + or – sign depends on the datum is higher or lower than the bottom of the clay layer, respectively
H = the levels upstream of the dike follow an exponential distribution as in overflowing mechanism.

After the clay layer has ruptured, a sand–carrying boil may be formed. In order to assess whether it will occur, the Bligh's criterion in formula (4-7) is applied. Piping is assumed to occur if:

$$H > m\left(\frac{L_k}{C_1} + \frac{d}{C_2}\right) \qquad (4\text{-}9)$$

Where:

H = the difference between water levels upstream and downstream of the dike.
L_k = seepage path length = L' + L_2 + B as defined in Figure 4.3
d = the thickness of the clay layer as above
m = a model factor, taking into account the scatter in empirical observations. It is assumed to conform to a normal distribution with μ = 1.67 and the coefficient of variation $V = \sigma/\mu = 0.2$ (Probabilistic design of flood defenses, TAW/CUR, 1990).
C_1, C_2 =constants, depending on the soil type (Table 4.6).

The reliability function for the second condition is therefore:

$$Z_2 = m\left(\frac{L_k}{C_1} + \frac{d}{C_2}\right) - (H - h_b - d) \qquad (4\text{-}10)$$

Piping only occurs when two conditions must be satisfied. The dike fails if:

$$(\mathbf{Z_1 < 0 \text{ and } Z_2 < 0}). \tag{4-11}$$

c. Calculation procedure

According to the equation (4-11) and the theory of statistics, the failure probability by piping equals:

$$\begin{aligned} P\{piping\} &= P\{Z_1 < 0\_and\_Z_2 < 0\} \\ &= P\{Z_1 < 0\} \times P\{Z_2 < 0 \mid Z_1 < 0\} \end{aligned} \tag{4-12}$$

Where: $P\{Z_2 < 0 \mid Z_1 < 0\}$ denotes the probability that $Z_2 < 0$ given $Z_1 < 0$. In practice, the determination of $P\{Z_2 < 0 \mid Z_1 < 0\}$ is difficult because of the complicated behavior of the natural phenomena. Instead of using the formula (4-12) some approximation formulas are used for the determination of $P\{Z_1 < 0\_and\_Z_2 < 0\}$. The best known one is that of Ditlevsen:

$$P\{Z_1 < 0 \text{ and } Z_2 < 0\} \geq \max\{\Phi_N(-\beta_1) \times \Phi_N(-\beta^*_2), \Phi_N(-\beta^*_1) \times \Phi_N(-\beta_2) \tag{4-13}$$

$$P\{Z_1 < 0 \text{ and } Z_2 < 0\} \leq \{\Phi_N(-\beta_1) \times \Phi_N(-\beta^*_2) + \Phi_N(-\beta^*_1) \times \Phi_N(-\beta_2) \tag{4-14}$$

$$\beta_i^* = \frac{\beta_i - \rho\beta_j}{\sqrt{1-\rho^2}} \tag{4-15}$$

$$\rho(Z_1, Z_2) = \sum_{i=1}^{n} \alpha_i^{(1)} \alpha_i^{(2)} \tag{4-16}$$

Where:

β = reliability index
ρ = correlation coefficient
$\Phi_N(-\beta)$ = distribution function of the standard normal distribution
$\alpha_i^{(1)}$ is the α value of Z_1 associated with the variable X_i according to formula (2-10), and similarly for $\alpha_i^{(2)}$.

In order to calculate the failure probability by the equation (4-13) and (4-14), first the failure probabilities by each condition have to be determined.

Table 4.6 Values of C adopted in the methods of Lane and Bligh

Type of soil	C value (Lane)	C value (Bligh)
Very fine sand or silt	8.5	18
Fine sand	7.0	15
Medium grained sand	6.0	15
Coarse sand	5.0	12
Fine gravel	4.0	12
Medium grained gravel	3.5	12
Gravel and sand	3.5	12
Coarse gravel	3.0	12
Boulders and gravel	2.5	12
Boulders, gravel and sand	2.5	4 to 6
Soft clay	3.0	-
Medium firm clay	2.0	-
Hard clay	1.8	-
Very hard clay	1.6	-

At each critical section of a segment, the values of problem variables are different. An overview of the problem variables for this mechanism calculations is shown in Table 4.7. The distributions of high water level for this mechanism are the same with the ones obtained in appendix I.

Table 4.7 Overview of the problem variables for piping mechanism

Segment	Variables	Description	Type	Unit	μ	σ
1	h_b	Distance from the datum to the bottom of the top clay layer	D	m	4	-
	ρ_c	Wet density of the top clay layer	D	Kg/m^3	1800	-
	d	The thickness of the top clay layer	N	m	4.0	0.5
	k_V	The permeability of the top layer	D	m/s	4.5x10^{-7}	-
	C_1	Constant for the top layer	D	-	6	-
	C_2	Constant for the subsoil	D	-	15	-
	k	The permeability of the subsoil	D	m/s	8x10^{-5}	-
	L_K	Seepage path length (present condition)	N	m	48	2
3	h_b	Distance from the datum to the bottom of the top clay layer	D	m	2.6	-
	ρ_c	Wet density of the top clay layer	D	Kg/m^3	1830	-
	d	The thickness of the top clay layer	N	m	3.0	0.5
	k_V	The permeability of the top layer	D	m/s	6.3x10^{-8}	-
	C_1	Constant for the top layer	D	-	6	-
	C_2	Constant for the subsoil	D	-	15	-
	k	The permeability of the subsoil	D	m/s	8x10^{-5}	-

	L_K	Seepage path length (present condition)	N	m	35	2
4	h_b	Distance from the datum to the bottom of the top clay layer	D	m	1.91	-
	ρ_c	Wet density of the top clay layer	D	Kg/m^3	1870	-
	d	The thickness of the top clay layer	N	m	1.2	0.5
	k_V	The permeabilty of the top layer	D	m/s	$6.3x10^{-8}$	-
	C_1	Constant for the top layer	D	-	6	-
	C_2	Constant for the subsoil	D	-	15	-
	k	The permeability of the subsoil	D	m/s	$8.5x10^{-5}$	-
	L_K	Seepage path length (present condition)	N	m	46	2
5	h_b	Distance from the datum to the bottom of the top clay layer	D	m	0.4	-
	ρ_c	Wet density of the top clay layer	D	Kg/m^3	1830	-
	d	The thickness of the top clay layer	N	m	2	0.5
	k_V	The permeability of the top layer	D	m/s	$6.3x10^{-8}$	-
	C_1	Constant for the top layer	D	-	6	-
	C_2	Constant for the subsoil	D	-	12	-
	k	The permeability of the subsoil	D	m/s	$9.5x10^{-5}$	-
	L_K	Seepage path length (present condition)	N	m	45	2
6	h_b	Distance from the datum to the bottom of the top clay layer	D	m	1.29	-
	ρ_c	Wet density of the top clay layer	D	Kg/m^3	1830	-
	d	The thickness of the top clay layer	N	m	3.4	0.5
	k_V	The permeability of the top layer	D	m/s	$6.3x10^{-8}$	-
	C_1	Constant for the top layer	D	-	6	-
	C_2	Constant for the subsoil	D	-	12	-
	k	The permibeality of the subsoil	D	m/s	$9.5x10^{-5}$	-
	L_K	Seepage path length (present condition)	N	m	43	2
Others	ρ_w	Density of water	D	Kg/m^3	1000	-
	g	Acceleration of gravity	D	m/s^2	9.81	-
	D	Thickness of water-bearing stratum	D	m	15	-
	m	Model factor	N	-	1.67	0.334

Notes: D = deterministic, N = normal. Since the thickness of the water-bearing stratum is undefined so in this calculation it is taken as 15 (m).

The occurrence of sand-carrying boil is dependent on the seepage path length. When the dike is heightened, the dike cross-section also is enlarged. Consequently, the seepage path length is longer. The determination of the seepage path length was carried out according to the definition in the Figure 4.1. The results are in Table 4.8. All given values of the seepage path length in Table 4.8 are considered the mean values.

Table 4.8 Determination of the seepage path length

Design return period (years)	Seepage path length (m)					
	Seg. 1	**Seg. 2**	**Seg. 3**	**Seg. 4**	**Seg. 5**	**Seg. 6**
20 (present)	48.0	-	35.0	46.0	45.0	43.0
50	48.0	-	35.7	47.0	46.0	43.6
100	49.0	-	37.0	48.0	47.0	45.0
150	50.0	-	38.0	49.5	48.0	46.0
200	50.0	-	39.0	51.0	49.5	47.0
250	51.0	-	41.0	52.0	51.0	48.0
500	54.0	-	47.0	59.0	57.0	53.0
1000	60.0	-	59.0	72.0	69.0	63.0

The VAP model was used to calculate the failure probabilities for each condition. From the geotechnical condition of the section 2, the subsoil is clay, it is considered that the failure probability caused by piping is very small and can be ignored. Calculations were done for 5 other sections and in two steps:

- First, determining the probabilities for the rupturing condition by applying the formula (4-8)
- Second, determining the probabilities for the occurrence of sand carrying boil by applying the formula (4-10).

The details of these calculations are given in the appendix V. Results are shown in Table 4.9 and 4.10.

Table 4.9 Failure probabilities for the rupturing condition

Design return period (years)	Failure probability					
	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
20 (present)	0.293	0.0	0.313	0.991	0.999	0.369
50	0.293	0.0	0.313	0.991	0.999	0.369
100	0.293	0.0	0.313	0.991	0.999	0.369
150	0.293	0.0	0.313	0.991	0.999	0.369
200	0.293	0.0	0.313	0.991	0.999	0.369
250	0.293	0.0	0.313	0.991	0.999	0.369
500	0.293	0.0	0.313	0.991	0.999	0.369
1000	0.293	0.0	0.313	0.991	0.999	0.369

Table 4.10 Failure probabilities for the sand-carrying boil condition

Design return period (years)	Failure probability					
	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
20 (present)	1.37×10^{-4}	0.0	5.71×10^{-4}	5.47×10^{-4}	1.11×10^{-3}	2.44×10^{-4}
50	1.37×10^{-4}	0.0	5.03×10^{-4}	4.73×10^{-4}	9.54×10^{-4}	2.23×10^{-4}
100	1.20×10^{-4}	0.0	3.98×10^{-4}	4.10×10^{-4}	8.25×10^{-4}	1.81×10^{-4}
150	1.05×10^{-4}	0.0	3.35×10^{-4}	3.33×10^{-4}	7.16×10^{-4}	1.57×10^{-4}
200	1.05×10^{-4}	0.0	2.82×10^{-4}	2.73×10^{-4}	5.83×10^{-4}	1.37×10^{-4}
250	0.93×10^{-4}	0.0	2.03×10^{-4}	2.40×10^{-4}	4.78×10^{-4}	1.19×10^{-4}
500	0.65×10^{-4}	0.0	0.83×10^{-4}	1.07×10^{-4}	2.31×10^{-4}	0.64×10^{-4}
1000	0.35×10^{-4}	0.0	0.20×10^{-4}	0.34×10^{-4}	0.73×10^{-4}	0.23×10^{-4}

From the failure probabilities for the rupturing condition in Table 4.9 it is found that the failure probabilities of segment 4 and 5 are very high, approximately equal to 1. The reason is that the thickness of the top clay layer at these locations is thinner than other locations. Because of very high failure probabilities for the rupturing condition (≈ 1.0), the failure probabilities for the occurrence of the sand-carrying boils at these segments are considered not dependent on the occurrence of the rupturing condition. At these two segments the failure probabilities caused by piping can be determined by the following formula:

$$P\{piping\} = P\{Z_1 < 0\} \times P\{Z_2 < 0\} \tag{4-17}$$

Since $P\{ Z_1 < 0 \} \approx 1.0$ so $P\{ piping \} \approx P\{ Z_1 < 0 \}$ for this case. As for other segments, the failure probabilities have to be determined approximately by using the equation (4-14). Considering section 1 at the present condition (T= 20 years), the stochastic variables of d and H are both present in Z_1 and Z_2, therefore, Z_1 and Z_2 are correlated. From the calculated results for piping by using VAP model (Appendix V), the failure probability of rupturing of top clay layer is 0.293 with Hasofer-Lind reliability index $\beta = 0.545$. The failure probability of sand carrying boil is 1.37×10^{-4} with $\beta = 3.64$. The values of α_i for the water level (H) and the thickness of the top clay layer are (Appendix V):

Function Z_1	**Function Z_2**
$\alpha(H) = 0.711$ $\alpha(d) = -0.703$	$\alpha(H) = 0.631$ $\alpha(d) = -0.146$

By applying the equation (4-16), the correlation coefficient of Z_1 and Z_2 can be determined using the values of α_i as in the table above, the correlation coefficient then becomes: $\rho = 0.551$

By applying the equation (4-15) one gets:

Function Z_1	**Function Z_2**
$\beta_1 = 0.545$ $\beta_1^* = -1.75$	$\beta_2 = 3.64$ $\beta_2^* = 4.0$

And by applying the equation (4-14) one gets:

$$P\{Z_1 < 0 \text{ and } Z_2 < 0\} \leq 1.34 \times 10^{-4} \qquad (*)$$

If Z_1 and Z_2 are considered independent, then the failure probability of $Z_1 < 0$ and $Z_2 < 0$ can be presented by (4-17):

$$P\{Z_1 < 0 \text{ and } Z_2 < 0\} = 0.293 \times 1.37 \times 10^{-4} = 0.4 \times 10^{-4} \qquad (**)$$

Comparing the result between (*) and (**) it is found that in the case of dependency between Z_1 and Z_2 the failure probability is a little bit higher. This result reflects correctly the working condition of a parallel system that correlated elements always give higher failure probabilities than in the case of independent ones.

By applying the same principle for other segments with different return periods, the calculated failure probabilities by piping for all segments are shown in Table 4.11.

Table 4.11 Failure probabilities caused by piping

Design return period (years)	Failure probability					
	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
20 (present)	1.34×10^{-4}	0.0	5.89×10^{-4}	5.47×10^{-4}	1.11×10^{-3}	2.37×10^{-4}
50	1.34×10^{-4}	0.0	5.66×10^{-4}	4.73×10^{-4}	9.54×10^{-4}	2.26×10^{-4}
100	1.25×10^{-4}	0.0	4.18×10^{-4}	4.10×10^{-4}	8.25×10^{-4}	1.86×10^{-4}
150	1.13×10^{-4}	0.0	3.35×10^{-4}	3.33×10^{-4}	7.16×10^{-4}	1.65×10^{-4}
200	1.13×10^{-4}	0.0	2.85×10^{-4}	2.73×10^{-4}	5.83×10^{-4}	1.34×10^{-4}
250	0.93×10^{-4}	0.0	2.07×10^{-4}	2.40×10^{-4}	4.78×10^{-4}	1.24×10^{-4}
500	0.72×10^{-4}	0.0	1.01×10^{-4}	1.07×10^{-4}	2.31×10^{-4}	0.72×10^{-4}
1000	0.36×10^{-4}	0.0	0.22×10^{-4}	0.34×10^{-4}	0.73×10^{-4}	0.31×10^{-4}

4.3.3 Marco-instability mechanism

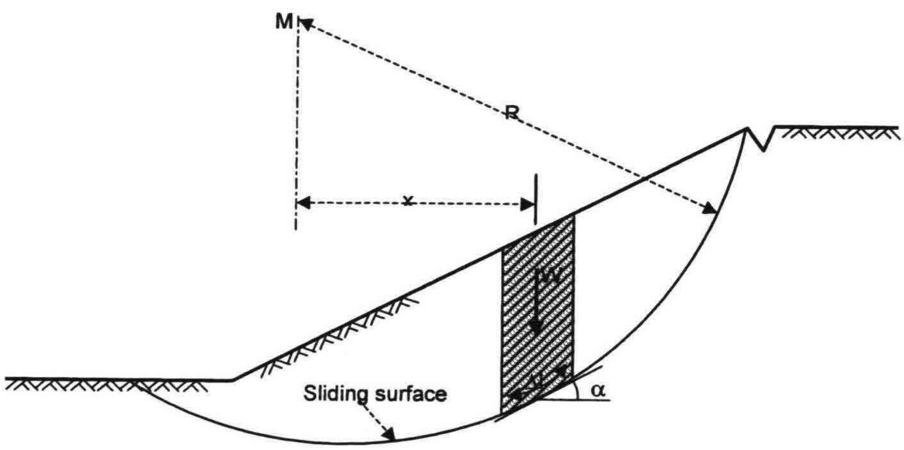

Figure 4.6 Mechanism of macro-instability

a. Mechanism of macro instability

A slope forming the transition between two ground levels is maintained in position by mobilization of the internal shearing resistance of the soil. In the absence of sufficient mobilizable shearing resistance the slope will slide. The term macro-instability is applied to denote that slope failure occurs along a large failure surface (Figure 4.6).

b. Reliability function for macro-instability

In practice, a slope is checked for macro-instability by considering the equilibrium of moments acting upon a mass of soil bounded by the ground levels, the slope and a potential failure surface (failure mode). The soil mass is acted upon by gravity and external forces (striving to induce sliding along the surface through the so-called overturning moment M_o) and, on the other hand, by the shearing forces, which are mobilized along the surface (striving to prevent sliding via the so-called resisting moment). When the resisting moment is equal to the overturning moment, the soil mass is in equilibrium.

The Bishop's method is applied to calculate the stability factor F, which is defined as:

$$F = \frac{M_{resisting}}{M_{overturning}} = \frac{R\sum_{1}^{n}\left\{c'_a b_n + (W_n - u_n b_n)\tan\phi'_a\right\}\times 1/m_\alpha}{\sum_{1}^{n}\left\{RW_n \sin\alpha_n + AW_n(R\cos\alpha_n - \frac{h_n}{2})\right\}} \qquad (4\text{-}18)$$

In which

$$m_\alpha = \left(1 + \frac{\tan\alpha_n \tan\phi'_a}{F}\right)\cos\alpha_n \qquad (4\text{-}19)$$

Where n = number of slices, W = the weight of each slice, b = the width of the slice (m), u = porewater pressure at the slip surface of the slice (kPa), c' = cohesion at the slip surface of the slice (kPa), ϕ_a' = angle of internal friction at the slip surface of the slice, R = radius (m), A = seismic coefficient, α = angle of slip surface of the slice with the horizontal, h_n = the average height of each slice.

To find out the stability factor of the slope, a number of failure circles are taken into account. The failure surface corresponding to the lowest stability factor is called the critical failure circle. The associated stability factor is called the stability factor of the slope (F):

$$F = \min F_i \qquad (i = 1, 2, \ldots n) \qquad (4\text{-}20)$$

The magnitude of the stability factor depends on a large number of variables, including geometry of the cross-sectional profile of the slope and of the soil strata, dead weight of the soil and external loads, shearing strength parameters of the soil and porewater pressure, if any. In principle, these quantities are uncertain variables and therefore the stability factor, too, is an uncertain quantity. In a probabilistic analysis, the variables mentioned can be conceived as stochastic variables. From the probability distributions of these variables, the probability distribution of the stability factor can be derived, and from this in turn can be deduced the probability that the stability factor is less than 1. This will be designated as the probability of instability of the slope.

It is considered that the probability of occurrence of a failure mode is equal to the probability that the associated stability factor is less than 1.0:

$$P\{f_i\} = P\{F_i < 1\} \qquad (i = 1, 2, \ldots n) \qquad (4\text{-}21)$$

Where f_i represents the event "the failure mode associated with circle i occurs". The reliability function for the failure mode can be written as:

$$Z_i = F_i - 1 \qquad (i = 1, 2, \ldots n) \qquad (4\text{-}22)$$

And the corresponding reliability index:

$$\beta_i = \frac{\mu_{Z_i}}{\sigma_{Z_i}} = \frac{\mu_{F_i} - 1}{\sigma_{F_i}} \qquad (i = 1, 2, \ldots n) \qquad (4\text{-}23)$$

Where μ and σ respectively denote the expectation and the standard deviation. With regard to the definition of the stability factor of the slope: the failure surface corresponding to the lowest stability factor (or highest probability of failure) is called the critical failure circle, the associated stability factor is called the stability factor. From that, the probability of instability of the slope in this case can be defined as:

$$P\{f\} = \max P\{F_i < 1\} \qquad (4\text{-}24)$$

c. Calculation procedure

In this section, failure probability calculations for macro-instability were done by MPROSTAB model, which was developed by GeoDelft. With the program MPROSTAB a probabilistic analysis of stability of an earth slope can be carried out. The computational model is based on Bishop's method of slices for equilibrium analysis, random field modeling of spatial variability of soil strength and pore pressure, and first order second moment probabilistic reliability analysis to calculate the probability that the stability factor is less than 1.0.

In the MPROSTAB model, the following assumptions regarding the geometrical soil model and soil properties have been adopted: the geometry of soil layers is considered to be deterministic data, no uncertainty about geometrical data is taken into account. Other data needed in the stability analysis are unit weights of soil, shearing strength parameters and pore pressures. From earlier reported studies on probabilistic stability analysis, it may be concluded that spatial variability of unit weights is of minor importance. Therefore, this parameter is left out of consideration in the stochastic model (User's manual MPROSTAB). The most important variables, which dominate the uncertainty of the stability factor, are the porewater pressures and the shearing strength properties of the soil. In this section, only the following parameters of soil are assumed to be stochastic:

- Angle of internal friction (φ)
- Cohesion (c')
- Porewater pressure

The selected probability distribution for the angle of internal friction and cohesion is of log normal type:

$$F(v)=\int_{0}^{v}\frac{1}{2\pi\chi\delta}e^{-\frac{1}{2}\frac{\ln^2\left(\frac{\chi}{m}\right)}{\delta^2}}d\chi \qquad v\geq 0 \qquad (4\text{-}25)$$

Where m and δ are parameters of the distribution. The mean value and standard deviation of this distribution are:

$$E(v)=me^{\frac{1}{2}\delta^2} \qquad \sigma(v)=E(v)\sqrt{e^{\delta^2}-1} \qquad (4\text{-}26)$$

It is assumed that the pore pressures are normally distributed with the mean value equal to the interpolated piezometric level. The standard deviation will be taken based on the experience and it is equal to 0.25 for all the cases.

Earthquake will be considered as a deterministic variable. Given the Vietnamese situation, light earthquakes, the seismic coefficient is taken to be 0.05.

At each section, the probability of failure of the slope is calculated for the present condition, for the new conditions with the design return period of 50, 100, 150, 200, 250, 500, and 1000 years. With 6 segments, totally there are 48 cases, of which the failure probabilities have to be determined. It can be seen that it is a time-consuming work if all the cases will be analyzed compared to the limited time during this study. Therefore, calculations were carried out for only three sections, which are section 1, 2 and section 3. At these sections, calculations will be carried out for the dike corresponding to the present condition (T= 20 years) for the designed condition (T= 1000 years). It was found that the geo-technical condition for section 1, 3, 4, 5, 6 is more or less similar therefore the failure probability by sliding is expected to be almost equal. At section 2, the soil condition is different from others therefore analysis for this section must be done. The schematized dike cross-sections and general soil information for all sections are given in the appendix III. An overview of problem variables for the calculation with section 3 is shown in Table 4.12.

Table 4.12 Overview of the problem variables of segment 3 (macro- instability)

Layer	Variables	Description	Type	Unit	μ	σ
1	ρ_d	Dry density	D	Kg/m^3	1470	-
	ρ_w	Wet density	D	Kg/m^3	1870	-
	C'	Soil cohesion	LN	KN/m^2	21.1	2.5
	ϕ	Angle of internal friction	LN	degree	15.0	3.0
2	ρ_d	Dry density	D	Kg/m^3	1340	-
	ρ_w	Wet density	D	Kg/m^3	1830	-
	C'	Soil cohesion	LN	KN/m^2	18.0	2.5
	ϕ	Angle of internal friction	LN	degree	13.0	3.0
3	ρ_d	Dry density	D	Kg/m^3	1400	-
	ρ_w	Wet density	D	Kg/m^3	1700	-
	C'	Soil cohesion	LN	KN/m^2	0	-
	ϕ	Angle of internal friction	LN	degree	26.0	3.0
	ρ_w	Density of water	D	Kg/m^3	1000	-
	P	Porewater pressure	N	KN/m^2	-	0.25
	A	Earthquake coefficient	D	-	0.05	-

Calculations were carried out for sections 1, 2, and 3 by MPROSTAB model. The details for these calculations by MPROSTAB are given in the Appendix VI. Only the results are shown in Table 4.13:

Table 4.13 Failure probabilities caused by macro-instability

Design return periods (years)	20	1000
Section 1	7.14×10^{-10}	7.60×10^{-8}
Section 2	6.84×10^{-21}	2.58×10^{-7}
Section 3	5.15×10^{-10}	8.65×10^{-5}

Some comments on these results are:

- The level of the failure probabilities is rather low (<< 8.65×10^{-5} per year). Compared to the level of the failure probabilities of overflowing (> 10^{-4} per year) and piping (around 10^{-4} per year), the failure probabilities of macro-instability can be neglected. This means that the total failure probability of a dike is $(>10^{-4}) + 10^{-4} + (<< 10^{-5}) = (>10^{-4}) + 10^{-4}$.
- The failure probability increases with increasing the crest levels.

4.4 FAILURE PROBABILITIES OF HYDRAULIC STRUCTURES

a. Failure mechanisms for hydraulic structures

As given in chapter 5, the hydraulic structures of the Duong River dike are sluices. These structures were laid in the body of the dike. Together with the dike, during high water level period these hydraulic structures also fulfil a water-retaining function. Safety requirements of these hydraulic structures are also of importance. Failure of these structures will lead to the failure of the dike and eventually, inundation of the region. Normally, assessment for hydraulic structures will be based on the following aspects:

- The probability of run-up and overflowing, or the design height of the hydraulic structure
- The strength and stability of the structure components, stability of soil under and along structures
- The reliability of the closure operation

For the Duong River, all the sluices are for agricultural purposes. The dimension of the sluices is quite small. The height and the width of the rectangular sluices are around 1.0 to 1.5m. The diameter of the circular sluices is also close to those values. All of the sluices were laid down in the body of the dike at the level, which is much lower than the crest level of the dike. Therefore, for the mechanism of run-up and overflowing it is considered that the failure probability belongs to the dike as analyzed in section 4.3.1 above.

All the sluices are equipped with gates. Generally, the closure operation of gates of hydraulic structures might be affected by some factors such as the failure of means of closure, the sudden illness of the operator etc. These unexpected events can lead to the failure of the closure. Considering the existing condition of the sluices it is found that the gate dimension is small. All the gates are in good maintenance since the maintenance work is quite simple. The closure operation of the gates is done manually under the possibility of a group of few people. Therefore, it is assumed that the failure probability caused by the closure operation for all the gates is very small and can be ignored. In addition, there are also other reasons that can lead to the non-closure of the gate. They are human failures. Based on the experience, the failure probability caused by human failures is high but from the existing condition of the conduits of this study there are two possibilities which must be analyzed:

- First, it is assumed that the gate is not close by human failures during high water levels but the conduit and the downstream canal themselves can withstand the high discharge. It means that there is no destruction to them. In this case, the volume of water going through the conduit will cause some damage to the region behind the dike. However, it is known that the dimension of the conduits under the Duong River dike is small the water volume through the conduit is not so big and can not lead to the flooding of the region. Consequences are small and can be neglected.

- Second, the structure and the downstream canal can not withstand for the high water level discharge. The structure components are destroyed and this will lead to the failure of the dike and consequently, inundation of the area. In order to judge this condition whether it happens or not the design data and the present condition information of the conduits must be available. It is recommended for further study.

During the high water level period, the gate of sluices is closed. The sluices are submerged. The strength and the stability of the structure components as well as the stability of the soil surrounding the structures are of major importance during this period. Checking for strength under hydraulic loading takes place is impossible since there are no data available at this moment.

From those reasons, the safety assessment of the structures finally has to deal with the piping problem. Once the sluice is constructed, the top clay layer at the toe of the dike has to remove in order to construct the inlet and outlet canals. This leads to the increase of possibility that piping can occur. In the case the canals are made of concrete of other impermeable material, the piping problem is as not so serious as in the case of permeable material. In the second case, the probability of occurrence of piping under hydraulic structures can be expected to be higher. Given the existing condition of the outer and inner canals of conduits under the Duong River dike. The canal is constructed by rock. Even the top clay layer no longer exists but the seepage path length somewhat increases, therefore, the failure probability is expected not too high at this location. Calculations are done with the assumption that the seepage path length is rather longer than the length of the conduit itself. Since the data for hydraulic structures are not sufficient at every place that needs to determine the failure probability, calculations will only be done for one position, which has most reliable data. The failure probabilities for the hydraulic structures at other positions will be assumed to be the same with the calculated one. The conduit in segment 3 is taken into account as a representative one.

<u>b. Reliability function for piping mechanism under the conduit</u>

Applying the formula (4-10), with the note that the thickness of the top clay layer d is taken as 0, then the reliability function becomes:

$$Z = m\left(\frac{L_k}{C_1}\right) - (H - h_b) \qquad (4\text{-}27)$$

Where:

H = high water level and follows an exponential distribution.
m = model factor and it is normally distributed.
h_b = distance from the datum to the bottom of the hydraulic structure

C_1 = soil factor. At section 3, the subsoil under the conduit is well – graded sand, then $C_1 = 15$ (Table 4.6)
L_k = seepage path length and follows a normal distribution. The length of the conduit is 49.5m (appendix IV). It is assumed that the expected mean value of the seepage path length is 3 times the conduit length, then $\mu \approx 150$m. The standard deviation σ is taken as 5m.

By applying the VAP model to calculate, then the failure probability is:

$$\mathbf{P_f = 3.90 \times 10^{-4}}$$

Some comments on the result:

- The level of the failure probability is quite high. It is less than in the case of overflowing and at the same level with piping mechanism. From the obtained result, it is found that the failure probability caused by hydraulic structures plays an important role to the whole system.

- The calculated value of failure probability above is only for the mechanism of piping under hydraulic structures. Due to lack of information, full calculations with other mechanisms are not done at this moment. In fact, if other failure mechanisms are included the failure probability will be higher. In actual design, the assessment for hydraulic structures must include all the mechanisms and in order to get a better estimate of failure probabilities it is necessary to have enough and reliable data.

4.5 COMBINATION OF FAILURE PROBABILITIES

So far, the failure probabilities of mechanisms such as overflowing, piping, macro-instability, and for hydraulic structures at every segment are known. A dike segment fails if at least the failure of one of those mechanisms occurs. The overall failure probability of each segment in this case can be expressed by the formula of a series system:

$$P\,\{\text{segment fails}\} = P\,\{Z_1 < 0 \text{ or } Z_2 < 0 \text{ or } Z_3 < 0 \text{ or } Z_4 < 0\} \qquad (4\text{-}28)$$

Where:

$Z_1 < 0$ denotes the failure of the segment by overflowing
$Z_2 < 0$ denotes the failure of the segment by piping
$Z_3 < 0$ denotes the failure of the segment by hydraulic structures
$Z_4 < 0$ denotes the failure of the segment by macro-instability

As explained in section 4.3.3 above, the failure probability of macro-instability mechanism is small and can be neglected, then the equation (4-28) becomes:

$$P\,\{\text{segment fails}\} = P\,\{Z_1 < 0 \text{ or } Z_2 < 0 \text{ or } Z_3 < 0\} \qquad (4\text{-}29)$$

In this case, the fundamental lower and upper boundaries are given by:

$$\max\_P\{Z_i < 0\} \le P\{segment\_fails\} \le \sum_{i=1}^{3} P\{Z_i < 0\} \qquad (4\text{-}30)$$

The calculated results for segment 1 are shown in Table 4.14:

Table 4.14 Combination of the failure probabilities for segment 1

Design return period of water level (years)	Component failure probability			Combined Probability	
	Overflow.	Piping	Structure	Lower boundary	Upper boundary
20	1.15×10^{-2}	1.34×10^{-4}	3.90×10^{-4}	1.15×10^{-2}	1.20×10^{-2}
50	3.10×10^{-3}	1.34×10^{-4}	3.90×10^{-4}	3.10×10^{-3}	3.62×10^{-3}
100	1.54×10^{-3}	1.25×10^{-4}	3.90×10^{-4}	1.54×10^{-3}	2.06×10^{-3}
150	1.03×10^{-3}	1.13×10^{-4}	3.90×10^{-4}	1.03×10^{-3}	1.53×10^{-3}
200	7.68×10^{-4}	1.13×10^{-4}	3.90×10^{-4}	7.68×10^{-4}	1.27×10^{-3}
250	6.16×10^{-4}	0.93×10^{-4}	3.90×10^{-4}	6.16×10^{-4}	1.10×10^{-3}
500	3.10×10^{-4}	0.72×10^{-4}	3.90×10^{-4}	3.90×10^{-4}	7.72×10^{-4}
1000	1.54×10^{-4}	0.36×10^{-4}	3.90×10^{-4}	3.90×10^{-4}	5.80×10^{-4}

For other segments, the procedure is the same therefore no explanation is given hereby. Only the results are shown in Table 4.15:

Table 4.15 Combined failure probabilities at segments

Design return period (years)		20	50	100	150	200	250	500	1000
Segment 2	Upper	7.30×10^{-3}	3.49×10^{-3}	1.93×10^{-3}	1.42×10^{-3}	1.16×10^{-3}	1.01×10^{-3}	7.00×10^{-4}	5.44×10^{-4}
	Lower	6.91×10^{-3}	3.10×10^{-3}	1.54×10^{-3}	1.03×10^{-3}	7.68×10^{-4}	6.16×10^{-4}	3.90×10^{-4}	3.90×10^{-4}
Segment 3	Upper	1.49×10^{-2}	4.06×10^{-3}	2.35×10^{-3}	1.76×10^{-3}	1.44×10^{-3}	1.21×10^{-3}	8.01×10^{-4}	5.66×10^{-4}
	Lower	1.39×10^{-2}	3.10×10^{-3}	1.54×10^{-3}	1.03×10^{-3}	7.68×10^{-4}	6.16×10^{-4}	3.90×10^{-4}	3.90×10^{-4}
Segment 4	Upper	8.98×10^{-3}	3.96×10^{-3}	2.34×10^{-3}	1.75×10^{-3}	1.43×10^{-3}	1.25×10^{-3}	8.07×10^{-4}	5.78×10^{-4}
	Lower	8.04×10^{-3}	3.10×10^{-3}	1.54×10^{-3}	1.03×10^{-3}	7.68×10^{-4}	6.16×10^{-4}	3.90×10^{-4}	3.90×10^{-4}
Segment 5	Upper	6.79×10^{-3}	4.44×10^{-3}	2.76×10^{-3}	2.14×10^{-3}	1.74×10^{-3}	1.48×10^{-3}	9.31×10^{-4}	6.17×10^{-4}
	Lower	5.29×10^{-3}	3.10×10^{-3}	1.54×10^{-3}	1.03×10^{-3}	7.68×10^{-4}	6.16×10^{-4}	3.90×10^{-4}	3.90×10^{-4}
Segment 6	Upper	6.64×10^{-3}	3.72×10^{-3}	2.12×10^{-3}	1.59×10^{-3}	1.29×10^{-3}	1.13×10^{-3}	7.72×10^{-4}	5.75×10^{-4}
	Lower	6.01×10^{-3}	3.10×10^{-3}	1.54×10^{-3}	1.03×10^{-3}	7.68×10^{-4}	6.16×10^{-4}	3.90×10^{-4}	3.90×10^{-4}

Notes: Upper = upper boundary and Lower = lower boundary

It is conceived that the dike consists of, over its entire length, a series system of 6 consecutive segments. The overall failure probability of the dike can be determined as follows:

$$P\{\text{dike fails}\} = P\{Z_1 < 0 \text{ or } Z_2 < 0 \text{ or } Z_3 < 0 \text{ or } Z_4 < 0 \text{ or } Z_5 < 0 \text{ or } Z_6 < 0\} \qquad (4\text{-}31)$$

Where $Z_1 < 0, ...and\_Z_6 < 0$ denotes the failure of segment 1 to 6, respectively.

The fundamental upper and lower boundaries are given by:

$$\max\_P\{Z_i < 0\} \leq P\{dike\_fails\} \leq \sum_{i=1}^{6} P\{Z_i < 0\} \quad (4\text{-}32)$$

All calculated overall failure probabilities of the dike in different design return periods according to the equation (4-32) are given in Table 4.16.

Table 4.16 The overall failure probabilities of the dike

Design return period (years)	**20**	**50**	**100**	**150**	**200**	**250**	**500**	**1000**
Overall failure probability (upper boundary)	5.66×10^{-2}	2.33×10^{-2}	1.35×10^{-2}	1.02×10^{-2}	8.33×10^{-3}	7.18×10^{-3}	4.78×10^{-3}	3.46×10^{-3}
Overall failure probability (lower boundary)	5.17×10^{-2}	1.86×10^{-2}	9.24×10^{-3}	6.18×10^{-3}	4.61×10^{-3}	3.70×10^{-3}	2.34×10^{-3}	2.34×10^{-3}

5 CHAPTER V: OPTIMIZATION OF THE DIKE DESIGN

In general, the design of a river dike for the protection of a particular region can be expressed in the following formula:

$$C_{TOT} = C_{CONST} + E(S) \tag{5-1}$$

Where:

C_{CONST} = cost of construction of the dike

$E(S)$ = capitalized loss expectation

The design must be so contrived that the sum of the dike construction cost and the expected loss (due to inundation damage) in the protected region is a minimum.

5.1 ECONOMIC CALCULATIONS

5.1.1 Construction cost for heightening the dike

The cost of construction of the dike is assumed to be dependent only on the volume of the body of the dike. It should be noted that according to the Vietnamese norm for a Grade I dike the minimum dike section for stability is as follows:

- Dike crest width b = 5 to 6 (m). For the Duong River dike b = 6 (m)
- River side slope: $m_{riverside}$ = 2h :1v
- Landside slope: $m_{landside}$ = 3h :1v

It is known from the chapter 3 that the existing right Duong River dike has quite a lot of problems. At many sections of the dike, serious piping and seepage problems often occur during the high water level period. The reason of these problems is the weak foundation of the dike, where the subsoil is sand. In addition, the deficient cross-section of the dike is also the reason causing those problems. In order to avoid those, the dike cross section must be wide enough to have safety against seepage of water through the dike embankment or under the embankment. The cross section of the dike needs to be strengthened to provide dike slope stability under flood conditions, and for rapid drawdown when flood water levels fall. Besides the above reasons, the reason of maintenance condition is also rather important. To fulfill those functions, in the future condition the dike is designed with riverside and landside berms. The width of the berm at both sides is 5m. However, going into detail of technical solutions to the design of a dike is not the purpose of this study. Here, only the overall idea is given to define the geometry of the dike. Based on the above geometry parameters and with the length of 59.6 km, the calculation of the cost for heightening the dike from the present condition to the new conditions corresponding with the return periods T = 50, 100, 150, 200, 250, 500, 1000 years is shown in the table 5.1. The equation for determining the construction cost is:

$$C_{const} = \sum_{i=1}^{6} L_i A_i f_b \tag{5-2}$$

Where:

L_i = the length of the segment i of the dike
Ai = the area of the cross section of segment i
f_b = construction cost per unit volume

Table 5.1 Construction cost for heightening the dike

Design return period (year)	Volume (m^3)	Unit price (US$/$m^3$)	Cost (US$)
20 (present)	0	3.0	0
50	3.242.000	3.0	9.726.600
100	3.746.400	3.0	11.239.200
150	4.250.800	3.0	12.752.400
200	4.755.200	3.0	14.265.600
250	5.259.600	3.0	15.778.800
500	7.781.600	3.0	23.344.800
1000	12.825.200	3.0	38.475.600

5.1.2 Estimation of flood consequences

If inundation occurs, the protected area will be subject to damage and therefore to losses. In carrying out a risk analysis for the design of the dike, it is necessary to know what consequences the failure of the dike will have. Total damage has to be determined. In the current case, only the loss associated with damage affecting property, agriculture and industry is considered. It is assumed that those losses depend only on the inundation depth and on the size and manner of land use in the study area. Generally, the equation for determining the losses is:

$$S = A\sum_{i=1}^{3} \alpha_i S_i c_i \qquad (5\text{-}3)$$

Where:

A = total area of the protected region.
α_i = portion of the area used for the categories: i = 1(residential), i = 2 (agriculture), i = 3 (industry).
S_i = maximum possible loss for category i.
c_i = damage factor as a function of the inundation depth for category i.

The estimate of the expected damage from flooding by the equation (5-3) requires an economic profile of the study area. The economic profile is composed of various sectors such as housing, agriculture, and industry; and represents the property in the protected area that is susceptible to flood damage. Each sector should be compiled at the political boundary level most appropriate for the analysis, for example in Vietnam at the provincial, district, or commune level. Information for the profile should include population, per capita income, asset values, production activities and patterns, employment, and number and size of houses. For each economic activity, forecasts concerning future changes in each sector should be made or obtained. This will aid in characterizing the protected area over the life of the project. Additionally, attention should be given to the ground elevation of the protected area, location of various assets, and other physical conditions. This will allow for more accurate damage estimation.

An assessment of the quality of the data available also needs to be made while constructing the profile. Unavailability of data will place constraints on the calculations made to estimate the flood consequences. If data is limited, assumptions need to be made regarding current and future developments.

The housing profile requires two pieces of data: the number of houses per unit area in the study area and household asset values. Surveys can be conducted to gather information on household asset values. Also, an assessment needs to be made on how susceptible these assets are to flooding. The result will give a listing of the number of houses per hectare for each land use category in a commune, district, or province. Each house will carry with it an average asset value derived from surveys.

The agricultural profile can be determined in the following manner. Since agricultural production varies throughout the year, the profile should incorporate these variations as they occur during the time of flooding, such as crops grown or previous crops in storage. Crop values in Vietnam can be calculated as the agricultural land use area per commune multiplied by the average yield. This will then be multiplied by the average price for each crop. Livestock and poultry are similarly valued according to their market price multiplied by the total number sold per unit of agricultural land area.

The industrial profile can be calculated using official statistics for asset values of state, private, and foreign-owned industries. Production for each type of industry is taken as gross industrial output per unit area. Since floods can temporarily interfere with productive activities, output data may be divided by twelve to derive average monthly production values. This assumes no seasonal fluctuation in production. If more detailed data is available, seasonal production fluctuations can also be addressed.

The determination of damage based on the equation (5-3) with required economic profile of the area as above could not be conducted during this study due to the limitation of time and finance. During this study, the estimation of flood consequences for the study area in case of the failure of the Duong River dike will be based on the estimates, which are given in chapter III. Those estimates were carried out for the economic analysis during the feasibility study of the Duong River Flood protection project - Red River Delta (Marshall, 2002). According to that report, in order to get a good estimate of damage for the area, a questionnaire survey was performed. During interviews, commune officials were asked to make estimates of flood damage to each sector based on their experience. Two floods were considered; one flood at 1/3-meter flood depth and one flood at 1-meter flood depth. Averages for the communes interviewed were calculated and assumed to apply universally to the entire project flood protected area. The damage estimates are given as percentages, except for industrial downtime, which is given in months, and are summarized in Table 5.3.

Table 5.2 Typical average commune profile in the Duong River study area determined from interviews (Marshall, 2002)

Item	Unit	Value
Population	People/ha	11.00
Pop growth rate	%	0.0145
1 story house	unit/ha	2.55
2 story house	unit/ha	0.13
Assets 1 story house	mill VND/unit	13.50
Assets 2 story house	mill VND/unit	25.00
People per house	people/household	5
Agricultural income	mill VND/person/yr	1.65
Assumed income non-farm	% of ag production	0.20
Roads	km/ha	0.01
Road repair cost (paved)	mill VND/km	60.00
Road repair cost (dirt)	mill VND/km	2.00
Average farm size	ha/household	0.27
Spring paddy land	ha/household	0.29
Winter paddy land	ha/household	0.29
Cereal land (corn)	ha/household	0.20
Paddy productivity	t/ha	5.08
Paddy price	mill VND/t	1.70
Corn productivity	t/ha	2.64
Corn price	mill VND/t	2.20
Farm budget (paddy)	mill VND/ha	4.40
Farm budget (corn)	mill VND/ha	1.90
Cow	head/ha	0.66
Cow cull rate	%/year	1.00
Cow price live weight	VND/kg	10000.00
Pig	head/ha	6.75
Pig cull rate	%/year	1.00
Pig price live weight	VND/kg	9000.00
Poultry	head/ha	33.33
Poultry cull rate	%/yr	1.00
Poultry price live weight	VND/kg	13000.00
Industrial production	mill VND/ha/month	0.33
Industrial assets	mill VND/ha	6500.00
Commerce (small shops)	% of households	0.029
Average commerce assets	Mill VND	5.00

Table 5.3 Assumptions of flood damage for flood height of 1/3 m and 1 m

Flood Damage for Different Flood Levels	0.3 m	1 m
Crop loss in percent	50%	100%
Livestock loss in percent	20%	45%
Housing asset loss in percent	10%	25%
Industrial loss in percent	25%	50%
Downtime for industrial production (months)	3	4
Asset loss for shops in percent	15%	28%
Roads destroyed in percent	35%	48%

The obtained data from the feasibility study for flood protection projects in the Red River Delta by Marshall, 2002 give a detail assessment for all economic sectors in the region. Based on those assumptions, an estimate of damage for 1971 flood for the area of 250.000 hectares on both sides of the Duong River, which includes 484 communes, if it were to occur today was done. The calculated results are shown in Table 3.10 and Table 3.11 in chapter III. However, during this study, only the area located on the right side of the Duong River is of interest. The damage for the area is less than the values calculated in Table 3.10 and 3.11 since those values include the damage for the area on both sides of the Duong River. If it is considered that the damage is proportional to the area then the damage for the study area can be determined as follows:

- As for 1971 flood, from Figure 3.8 it is found that the flooded regions on both sides of the Duong River are approximately equal. Therefore, the damage for the study area can be considered to be equal to ½ of the calculated damage in Table 3.10 and 3.11.
- In the case of flooding of the whole study area the damage is estimated by: the area of this study area is 180.000 hectares, and the study area of the Duong River flood protection project is 250.000. So the damage for the study area is equal to the calculated damage times a factor of (180.000 / 250.000). The results are shown in Table 5.4 and 5.5.

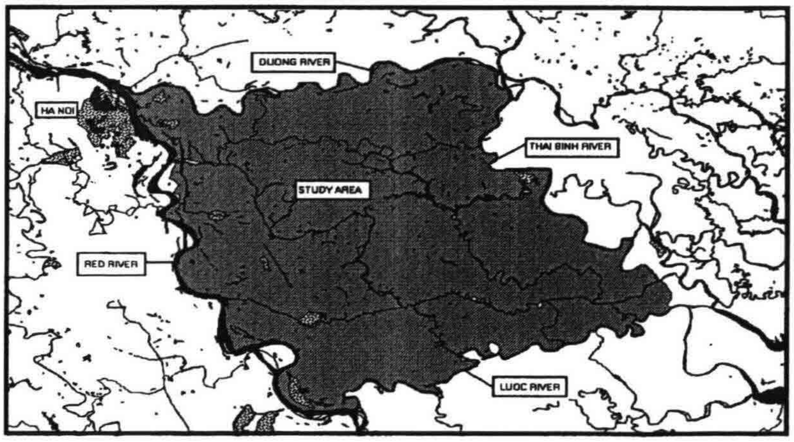

Figure 5.1 Potential inundation area in case of failure of the Duong River dike

Table 5.4 Damage estimates for the historic flood of 1971 and a flood covering the whole area based on year 2000 prices and a 1-meter depth flood.

Flood losses for 1 meter deep flood	1971	Whole area	1971	Whole area
	(million VND)		(Million US$)	
Crop losses	706,500	2,035,000	$49.5	$142.6
Livestock losses	181,500	522,000	$12.5	$36.72
Housing losses	515,500	1,471,000	$36	$103
Commerce losses	6,000	17,000	$0.5	$1.44
Highways	19,000	48,000	$1.5	$3.6
Major roads	500	720	$0.035	$0.05
Industrial assets damage	1,353,500	2,225,000	$94.5	$155.5
Industrial output losses	509,500	857,000	$35.5	$60
Total	3,292,000	7,175,720	$230	$503

Table 5.5 Damage estimates for the historic flood of 1971 and a flood covering the whole area based on year 2000 prices and a 1/3-meter depth flood.

Flood losses for 1/3 meter deep flood	1971	Whole area	1971	Whole area
	(Million VND)		(Million US$)	
Crop losses	353,500	1,017,360	$24.5	$71.28
Livestock losses	80,500	231,840	$5.5	$16.56
Housing losses	206,000	588,240	$14.5	$41.04
Commerce losses	3,500	9,360	$0	$0.72
Highways	14,000	35,280	$1	$2.16
Major roads	0	720	$0.00	$0.05
Industrial assets damage	677,000	1,112,400	$47.5	$77.76
Industrial output losses	382,000	642,240	$26.5	$44.64
Total	1,716,500	3,638,160	$120	$254

In fact, damage assessment for the area is difficult to define. A dike breach at the upper part of the Duong River could result in a larger area of inundation than the one at the lower part. Different floods have different influence to the region as can be seen in Table

5.4 and 5.5 above. However, at this moment, it is assumed that the damage to the region caused by any flood is leading to the same extent of loss. The loss estimate will be based on the obtained values in Table 5.4 and 5.5 with regard to the real flood, which happened in 1971. The assuming floods for the whole area are left out of the consideration. It is known from observations that, during the 1971 flood, the inundation depth at most places in the study area is almost equal to or higher than 1 meter (Marshall, 2002). Therefore, the loss estimate of the 1971 historical flood calculated for the case of 1-meter depth as in Table 5.4 is more practical. Then, the loss is taken as:

S = 230 (million US$)

This value simulates the damage to the study from the extent of 1971 historical flood if it were to occur today, which is the biggest flood observed in the Red River Delta. The extent of this flood is shown in Figure 3.8.

5.1.3 Capitalized loss expectation

The expected loss in year i can be determined by:

$$E(S) = P(f_i) \times S \qquad (5\text{-}4)$$

Where:
$P(f_i)$ = probability of failure in year i.
S = corresponding loss

The present value of the expected loss in year i:

$$E(S) = P(f_i) \times S \times \frac{1}{(1+r)^i} \qquad (5\text{-}5)$$

Considering the intended service life of the dike, the capitalized loss expectation can be presented as:

$$E(S) = \sum_{i=1}^{N} P(f_i) \times S \times \frac{1}{(1+r)^i} \qquad (5\text{-}6)$$

Where:
r = interest rate
N = intended service life of the dike

If it is considered that $P(f_i)$ is constant in time and N is large, then E(S) can alternatively be written as:

$$E(S) = P(f_i) \times S \times \frac{1}{r} \qquad (5\text{-}7)$$

With the overall failure probability of the dike $P(f_i)$ as determined in Table 4.16 , the loss S = 230 million US$, and the interest rate r is taken as 0.04, then the expected loss for each design return period was calculated using the equation (5-7) as in Table 5.6.

Table 5.6 The expected loss for the region

Design return period (years)	20	50	100	150
Failure probability (upper boundary)	$5.66X10^{-2}$	$2.33X10^{-2}$	$1.35X10^{-2}$	$1.02X10^{-2}$
Failure probability (lower boundary)	$5.17x10^{-2}$	$1.86x10^{-2}$	$9.24x10^{-3}$	$6.18x10^{-3}$
Expected loss (upper) (million US$)	325.45	133.98	77.63	58.65
Expected loss (lower) (million US$)	297.28	106.95	53.13	35.54

Design return period (years)	200	250	500	1000
Failure probability (upper boundary)	$8.33X10^{-3}$	$7.18X10^{-3}$	$4.78X10^{-3}$	$3.46X10^{-3}$
Failure probability (lower boundary)	$4.61x10^{-3}$	$3.70x10^{-3}$	$2.34x10^{-3}$	$2.34x10^{-3}$
Expected loss (upper) (million US$)	47.90	41.30	27.50	19.90
Expected loss (lower) (million US$)	26.51	21.28	13.46	13.46

5.2 OPTIMAL DIKE CREST ELEVATION

The optimal dike crest elevation was determined with the use of lower and upper boundaries. The total cost for the design of the dike was calculated by the equation (5-1) and is expressed as a function of the design return periods. The result of calculations is presented in the table below:

Table 5.7 Total cost of the design (use of the upper boundary)

Design Return period (years)	P_f (1/year)	Investment $I(P_f)$ for adjustment of the present dike to the new design of dike crest levels (million US$)	Present value of the expected loss (million US$)	Total cost of the design (million US$)
20 (present)	$5.66x10^{-2}$	0	325.45	325.45
50	$2.33x10^{-2}$	9.73	133.98	143.71
100	$1.35x10^{-2}$	11.24	77.63	88.87
150	$1.02x10^{-2}$	12.75	58.65	71.40
200	$8.33x10^{-3}$	14.27	47.90	62.17
250	$7.18x10^{-3}$	15.78	41.30	57.08
500	$4.78x10^{-3}$	23.34	27.50	50.84
1000	$3.46x10^{-3}$	38.48	19.90	58.38

Table 5.8 Total cost of the design (use of the lower boundary)

Design Return period (years)	P_f (1/year)	Investment $I(P_f)$ for adjustment of the present dike to the new design of dike crest levels (million US$)	Present value of the expected loss (million US$)	Total cost of the design (million US$)
20 (present)	$5.17x10^{-2}$	0	297.28	297.28
50	$1.86x10^{-2}$	9.73	106.95	116.68
100	$9.24x10^{-3}$	11.24	53.13	64.37
150	$6.18x10^{-3}$	12.75	35.54	48.29
200	$4.61x10^{-3}$	14.27	26.51	40.78
250	$3.70x10^{-3}$	15.78	21.28	37.06
500	$2.34x10^{-3}$	23.34	13.46	36.80
1000	$2.34x10^{-3}$	38.48	13.46	51.94

❖ <u>If the upper boundary of the overall failure probability is used, then:</u>

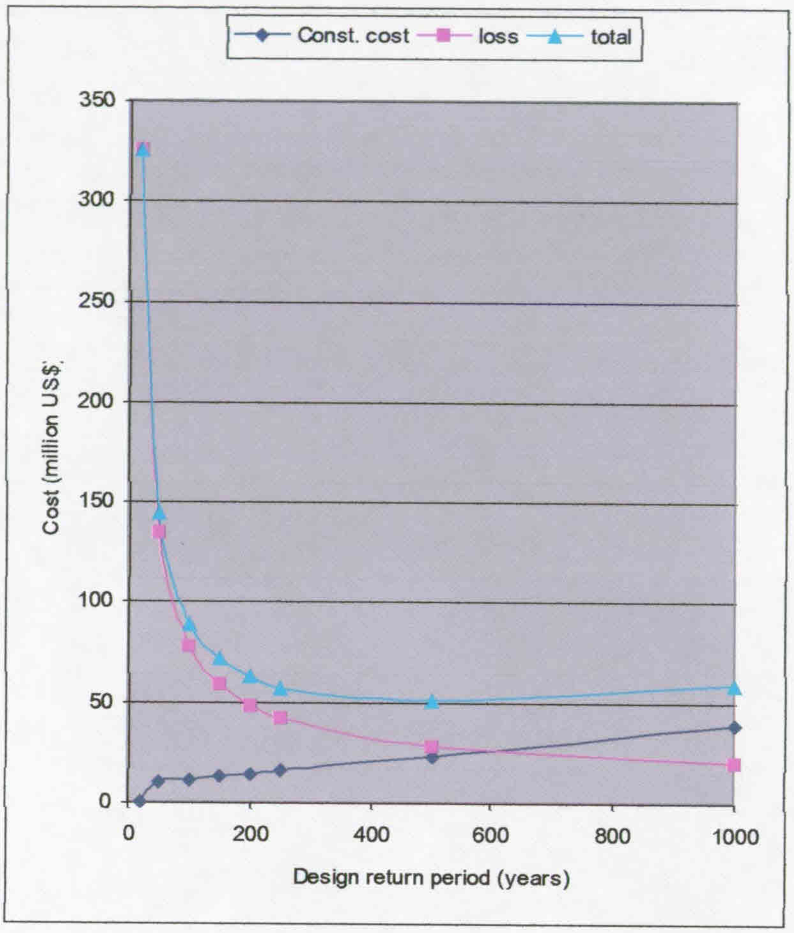

Figure 5.2 Determining the optimal height of the dike (upper boundary)

From the result in figure 5.2 it is found that the point corresponding with the minimum total cost is at T = 500 years. From an economic point of view, if the dike is designed with the return period T = 500 years the cost will be minimum. The corresponding design water level at Thuong Cat gauging station associated with T = 500 years is H = 15.29 m and consequently, the design crest level is h = 16.91m. Compared to the existing condition at this location (the dike crest level = 13.8m), the dike should be increased 16.91 – 13.8 = 3.11 m to meet the new safety level.

❖ In the case of using the lower boundary, one gets:

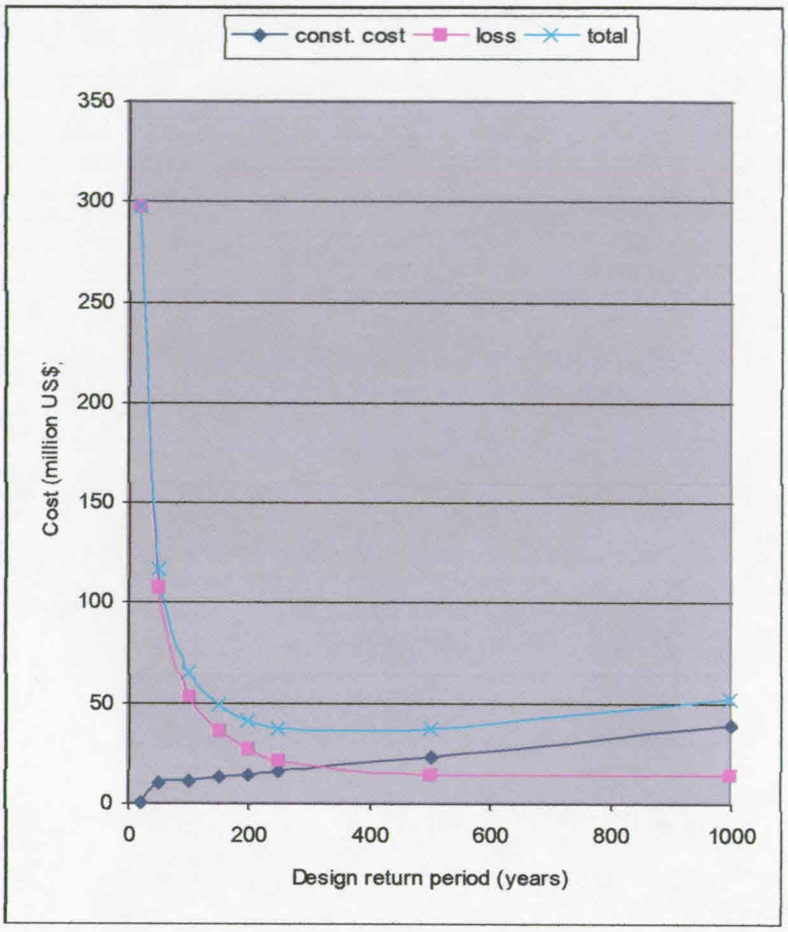

Figure 5.3 Determining the optimal height of the dike (lower boundary)

In this case, the design return period corresponding to the point of the minimum total cost is T= 450 years. The design water level associated with T = 450 years at Thuong Cat station is H = 15.20m and consequently the design crest level is h = 16.82m. Compared to the existing condition at this location (the dike crest level = 13.8m), the dike should be increased 16.82 – 13.8 = 3.02 m to meet the new safety level.

♦ Some comments on the result:

The difference in the design crest levels in two cases is small (0.09m). From engineering point of view it can be said that the use of the upper boundary in this case is applicable. Using the upper boundary in this case always give the design on the safe side but the difference compared to the lower boundary is quite small and can be accepted.

6 CHAPTER VI: CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

Conclusions from this study are:

- The above analyses of safety and the optimization of the design of the Duong River dike show that a significant number of factors can be included in the risk analysis than just the factor "water level"

- From the results of the individual mechanisms it appears that the failure probability associated with the macro-instability mechanism of the inner slope are low in relation with those of overtopping, piping and of hydraulic structures. The failure probability caused by overtopping is dominant but the ones caused by piping and of hydraulic structures also play an important role to the overall failure probability of the dike, especially in the case of long duration of high water.

- From the economic point of view, the chosen design return period is taken as 500 years. With this choice the design is always on the safe side and the difference is small and can be accepted. In order to achieve this safety level, the crest height of the dike should be raised with an average of 3.11m. With this improvement, the failure probability of the Duong River dike will decrease from 5.66×10^{-2} to 4.78×10^{-3} per year.

- The use of FORM method (level II calculation) showed that this is a useful tool to solve the probabilistic design problem. The outcomes of the method give a reasonable estimate of the failure probability. The difference between the upper and lower boundaries is small and can be considered acceptable from the engineering point of view. Using level II calculations enables the designer to investigate the influence of each variable on the probability of failure.

- Based on the obtained result of the failure probability, the first impression of the safety level of the area protected by the Duong River dike can be appraised, through that the safety level for other dike systems also can be estimated.

- The optimization of the dike design through considering construction cost and corresponding loss expectations for the Vietnamese situation is feasible.

- The advantage of the probabilistic design method is that it takes into account the uncertainties of design parameters. The use of the probabilistic approach for the design of flood defense system is very effective in general, especially in developing countries where the accuracy of available data is doubtful.

6.2 RECOMMENDATIONS

- During this study, due to the lack of information quite a lot of assumptions have been made regarding the data, which are necessary for the calculation process. In order to get a better result for the design sufficient and more accurate data such as water levels, soil properties, economic data, etc are recommended.

- The failure probability of overtopping mechanism is sensitive to the overall failure probability of the dike therefore, a good estimate of the failure probability can only obtained as the estimate for water level distributions is accurate. At the moment of this study, water levels along the dike at a number of sections were obtained by the linear interpolation. This is a rather rough estimate. For further study, it is recommended to use a better method such as using a flow model, etc.

- The use of a more accurate formula such as the ones of Ditlevsen or Hohenbichler for the determination of the failure probability is a positive way to reduce the inaccuracy in the result.

- In actual design, besides the above mechanisms, other failure mechanisms such as micro-instability, sliding of the outer slope, etc and other variables affecting the failure probability must be taken into account. Especially for hydraulic structures, more detailed calculations need to be carried out. Additionally, maintenance cost and non-material loss should be included in calculation processes.

- When considering the safety level for the study area, the effect of the Red, Thai Binh, and Luoc River dikes plays an important role, therefore, it should be taken into account, too.

- It is recommended that models for failure mechanisms have to be further developed, taking into account of the length effect and correlation.

- Probabilistic design of water defences should be stimulated by responsible governmental organizations in Vietnam. However, it still requires much effort to achieve this. The proposed step in this direction is proper education in this field at universities and, on short term, through short courses for design offices.

- To make this new design technique visible for future application the proper documentation of flood statistics, state of dikes, hydraulic structures, etc. should be undertaken in early stage.

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APPENDIX I: Determination of the highest water level distributions at sections

1. At the Thuong Cat gauging station on the Duong River

Table 1 Water level and discharge data at Thuong Cat station

Number	Year	Max.water level (m)	Max. discharge (m3/s)
1	1961	10,65	3770
2	1962	9,86	2960
3	1963	9,48	2970
4	1964	11,04	5270
5	1965	9,33	3300
6	1966	11,27	4800
7	1967	10,42	3880
8	1968	11,73	5140
9	1969	11,73	5270
10	1970	11,64	5710
11	1971	13,58	8720
12	1972	9,76	3600
13	1973	10,93	4500
14	1974	9,58	3500
15	1975	9,89	3410
16	1976	10,60	4520
17	1977	10,79	4710
18	1978	11,08	4840
19	1979	11,42	5070
20	1980	11,44	4670
21	1981	10,75	4100
22	1982	10,99	4780
23	1983	11,57	5580
24	1984	10,38	3560
25	1985	11,57	5530
26	1986	11,96	6070
27	1987	10,00	3850
28	1988	9,95	3420
29	1989	9,87	3150
30	1990	11,56	5750
31	1991	11,05	6180
32	1992	11,05	5750
33	1993	9,54	3210
34	1994	10,46	4840
35	1995	11,10	5570
36	1996	11,73	5700
37	1997	10,60	4800
38	1998	10,51	4870
39	1999	10,60	4710

Table 2 Maximum water levels at Thuong Cat gauging station

Number	Year	Max. water level (m)	Rank	m/(n+1)	Ln
1	1961	10,65	13,58	0,025	-3,689
2	1962	9,86	11,96	0,050	-2,996
3	1963	9,48	11,73	0,075	-2,590
4	1964	11,04	11,73	0,100	-2,303
5	1965	9,33	11,73	0,125	-2,079
6	1966	11,27	11,64	0,150	-1,897
7	1967	10,42	11,57	0,175	-1,743
8	1968	11,73	11,57	0,200	-1,609
9	1969	11,73	11,56	0,225	-1,492
10	1970	11,64	11,44	0,250	-1,386
11	1971	13,58	11,42	0,275	-1,291
12	1972	9,76	11,27	0,300	-1,204
13	1973	10,93	11,10	0,325	-1,124
14	1974	9,58	11,08	0,350	-1,050
15	1975	9,89	11,05	0,375	-0,981
16	1976	10,60	11,05	0,400	-0,916
17	1977	10,79	11,04	0,425	-0,856
18	1978	11,08	10,99	0,450	-0,799
19	1979	11,42	10,93	0,475	-0,744
20	1980	11,44	10,79	0,500	-0,693
21	1981	10,75	10,75	0,525	-0,644
22	1982	10,99	10,65	0,550	-0,598
23	1983	11,57	10,60	0,575	-0,553
24	1984	10,38	10,60	0,600	-0,511
25	1985	11,57	10,60	0,625	-0,470
26	1986	11,96	10,51	0,650	-0,431
27	1987	10,00	10,46	0,675	-0,393
28	1988	9,95	10,42	0,700	-0,357
29	1989	9,87	10,38	0,725	-0,322
30	1990	11,56	10,00	0,750	-0,288
31	1991	11,05	9,95	0,775	-0,255
32	1992	11,05	9,89	0,800	-0,223
33	1993	9,54	9,87	0,825	-0,192
34	1994	10,46	9,86	0,850	-0,163
35	1995	11,10	9,76	0,875	-0,134
36	1996	11,73	9,58	0,900	-0,105
37	1997	10,60	9,54	0,925	-0,078
38	1998	10,51	9,48	0,950	-0,051
39	1999	10,60	9,33	0,975	-0,025
	Average	**10,81**			
	Stand.	**0,86**			

1.1 By using the method of linear regression

Assuming that the highest water level follows an exponential distribution, then the equation for cumulative distribution is:

$$F(H) = 1 - e^{-\frac{H-\alpha}{\beta}} \qquad \text{(I-1)}$$

Parameters α and β can be determined from the data set by the following way:

From the data set for maximum values, one gets: $(H_i, \frac{i}{N+1})$

From equation (1):

$$1 - F(H) = e^{-\frac{H-\alpha}{\beta}}$$

$$Ln(1 - F(H)) = -\frac{H-\alpha}{\beta} = -\frac{1}{\beta}H + \frac{\alpha}{\beta}$$

By putting:

$$Y = Ln(1 - F(H)) \text{ with } 1 - F(H) = \frac{i}{N+1}$$

$$a = -\frac{1}{\beta}$$

$$b = \frac{\alpha}{\beta}$$

Equation (1) becomes:

$$Y = a.H + b$$

The relationship between Y and H is linear. By using the least square method the values of a and b are determined from the data set as follows:

$$a = \frac{1}{N}\sum_{i=1}^{N} Y_i - \frac{b}{N}\sum_{i=1}^{N} H_i = -0.945$$

$$b = \frac{\sum_{i=1}^{N} H_i Y_i - \frac{1}{N}\sum_{i=1}^{N} H_i Y_i}{\sum_{i=1}^{N}(H_i)^2 - \frac{1}{N}\left(\sum_{i=1}^{N} H_i\right)^2} = 9.258$$

Therefore the linear equation is:

$$Y = -0.945H + 9.258$$

Consequently, α = 9.8 and β = 1.06. So the exponential distribution for water level is (figure 2):

$$F(H) = 1 - e^{-\frac{H-9.8}{1.06}} \qquad \text{(I-2)}$$

From (I-2), the probability density function is (Figure 3):

$$f(H) = \frac{1}{1.06} e^{-\frac{H-9.8}{1.06}} \qquad \text{(I-3)}$$

1.2 By using the method of moments

From the equation for cumulative distribution function of the exponential distribution:

$$F(H) = 1 - e^{-\frac{H-A}{B}} \qquad \text{(I-4)}$$

Consequently, the probability density function is:

$$f(H) = B^{-1} e^{-\frac{H-A}{B}} \qquad \text{(I-5)}$$

It is known that the following relationship between the first moment, the second moment, and the parameters A, B holds:

The first moment: $\mu_1 = \int_A^\infty x.B^{-1}.e^{-.\frac{x-A}{B}}.dx = A + B = \mu$ (mean value)

The second moment: $\mu_2 = \int_A^\infty x^2.B^{-1}.e^{-.\frac{x-A}{B}}.dx = B = \sigma$ (standard deviation)

From the data set one gets: μ = 10,81 and σ = 0,86. Therefore:

B = σ = 0,86
A = μ - B = 10,81 – 0,86 = 9,95

Parametes A, B are known, the equation for cumulative distribution function is defined:

$$F(H) = 1 - e^{-\frac{H-9.95}{0.86}} \qquad \text{(I-6)}$$

And the probability density function:

$$f(H) = \frac{1}{0.86} e^{-\frac{H-9.95}{0.86}} \qquad \text{(I-7)}$$

2. At other sections along the Duong River dike

Water level measurements at every critical section of all segments along the Duong River are impossible. Only water level measurements were recorded at Thuong Cat gauging station. Therefore, flood frequency calculations at the other sections have to base on the water level data at Thuong Cat gauging station. In general, there are few ways to get the water levels at the critical sections from the water level and discharge data at Thuong Cat on Duong River such as using a flow model or analytical methods. But it is rather complicated and time consuming to use those methods and it is out of the scope of this study. During this study, a simple method is used, which is the method of linear interpolation. The water level at each critical section is interpolated from the water level at Thuong Cat station by the following way:

$$H_{\text{critical section}} = H_{\text{Thuong Cat}} - I \times D \qquad \text{(I-8)}$$

Where:

- I = the river slope
- D = distance from critical sections to the Thuong Cat station

For the Duong River, the slope is $11.4\text{x}10^{-5}$, which was calculated from the data of river cross-section. By substituting this value into equation (I-8) one gets the water levels at every critical section as in Table 3.

By applying the same way as done in the appendix I with the method of moments one gets the distributions for water levels at every section as follows:

For section 2:

$$F(H) = 1 - e^{-\frac{H-8.96}{0.86}} \qquad \text{(I-9)}$$

For section 3:

$$F(H) = 1 - e^{-\frac{H-6.91}{0.86}} \qquad \text{(I-10)}$$

For section 4:

$$F(H) = 1 - e^{-\frac{H-5.94}{0.86}} \qquad \text{(I-11)}$$

For section 5:

$$F(H) = 1 - e^{-\frac{H-5.03}{0.86}} \qquad \text{(I-12)}$$

For section 6:

$$F(H) = 1 - e^{-\frac{H-3.94}{0.86}} \qquad \text{(I-13)}$$

Table 3 Water levels at the critical sections

Number	Year	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
1	1961	10,65	9,66	7,61	6,64	5,73	4,65
2	1962	9,86	8,87	6,82	5,85	4,94	3,86
3	1963	9,48	8,49	6,44	5,47	4,56	3,48
4	1964	11,04	10,05	8,00	7,03	6,12	5,04
5	1965	9,33	8,34	6,29	5,32	4,41	3,33
6	1966	11,27	10,28	8,23	7,26	6,35	5,27
7	1967	10,42	9,43	7,38	6,41	5,50	4,42
8	1968	11,73	10,74	8,69	7,72	6,81	5,73
9	1969	11,73	10,74	8,69	7,72	6,81	5,73
10	1970	11,64	10,65	8,60	7,63	6,72	5,64
11	1971	13,58	12,59	10,54	9,57	8,66	7,58
12	1972	9,76	8,77	6,72	5,75	4,84	3,76
13	1973	10,93	9,94	7,89	6,92	6,01	4,93
14	1974	9,58	8,59	6,54	5,57	4,66	3,58
15	1975	9,89	8,90	6,85	5,88	4,97	3,89
16	1976	10,60	9,61	7,56	6,59	5,68	4,60
17	1977	10,79	9,80	7,75	6,78	5,87	4,79
18	1978	11,08	10,09	8,04	7,07	6,16	5,08
19	1979	11,42	10,43	8,38	7,41	6,50	5,42
20	1980	11,44	10,45	8,40	7,43	6,52	5,44
21	1981	10,75	9,76	7,71	6,74	5,83	4,75
22	1982	10,99	10,00	7,95	6,98	6,07	4,99
23	1983	11,57	10,58	8,53	7,56	6,65	5,57
24	1984	10,38	9,39	7,34	6,37	5,46	4,38
25	1985	11,57	10,58	8,53	7,56	6,65	5,57
26	1986	11,96	10,97	8,92	7,95	7,04	5,96
27	1987	10,00	9,01	6,96	5,99	5,08	4,00
28	1988	9,95	8,96	6,91	5,94	5,03	3,95
29	1989	9,87	8,88	6,83	5,86	4,95	3,87
30	1990	11,56	10,57	8,52	7,55	6,64	5,56
31	1991	11,05	10,06	8,01	7,04	6,13	5,05
32	1992	11,05	10,06	8,01	7,04	6,13	5,05
33	1993	9,54	8,55	6,50	5,53	4,62	3,54
34	1994	10,46	9,47	7,42	6,45	5,54	4,46
35	1995	11,10	10,11	8,06	7,09	6,18	5,10
36	1996	11,73	10,74	8,69	7,72	6,81	5,73
37	1997	10,60	9,61	7,56	6,59	5,68	4,60
38	1998	10,51	9,52	7,47	6,50	5,59	4,51
39	1999	10,60	9,61	7,56	6,59	5,68	4,60
Average		**10,81**	**9,82**	**7,77**	**6,80**	**5,89**	**4,80**
Stdev		**0,86**	**0,86**	**0,86**	**0,86**	**0,86**	**0,86**

APPENDIX II: Wave run-up calculations

The wave period and the significant wave height are determined by the Bretschneider formula (Revetments and Dikes, Verhagen, IHE Lecture notes):

$$\frac{gT}{u} = 2\times\Pi\times1.2\times\tanh\left[0.833\left(\frac{gd}{u^2}\right)^{0.375}\right]\tanh\frac{0.077\left(\frac{gF}{u^2}\right)^{0.25}}{\tanh\left[0.833\left(\frac{gd}{u^2}\right)^{0.375}\right]} \qquad \text{(II-1)}$$

$$\frac{gH_s}{u^2} = 0.283\times\tanh\left[0.530\left(\frac{gd}{u^2}\right)^{0.750}\right]\tanh\frac{0.0125\left(\frac{gF}{u^2}\right)^{0.42}}{\tanh\left[0.530\left(\frac{gd}{u^2}\right)^{0.750}\right]} \qquad \text{(II-2)}$$

Where:

H_s = significant wave height (m)
T = wave period (s)
F = fetch (m)
d = water depth (m)
u = wind velocity (m/s)
g = gravitational acceleration (m/s^2)

For the Duong river dike, based on the data available, if the values of fetch length, water depth, and wind speed are taken respectively as F = 3500 (m), d = 3.5 (m), and u= 15 (m/s), then the values of H_s and T calculated using the formula (II-1) and (II-2) are:

H_s = 0.56 (m) and T = 2.7 (s)

In all design it is common use to apply the 2% run-up. The general design formula that can be applied for wave run-up on dikes is given by (Dikes and Revetments, Pilarczyk, 1998):

$$\frac{R_{2\%}}{H_s} = 1.6\gamma_b\gamma_f\gamma_\beta\zeta_{op} \quad \text{with a maximum of } 3.2\,\gamma$$

Where:

γ_b = reduction factor due to berm
γ_f = reduction factor due to slope roughness and permeability
γ_β = reduction factor due to oblique wave attack
γ = total reduction factor = $\gamma_b\gamma_f\gamma_\beta$
ξ_p = breaker parameter for the peak –period and is determined as follows:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\Pi H_s}{gT_p^2}}}$$

With α = the angle of the outer slope and T_p = peak wave period. Usually the peak period is 1.1 to 1.25 times the mean period (Revetments, sea dikes, and river levees, H.J. Verhagen). Taking T_p= 1.25T_m = 1.25x2.7= 3.4 (s). In addition, according to Vetnamese norm, the Duong River dike is classified as Grade I, therefore the riverside slope is taken as 1:2, so tanα = 0.5. Thus:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\Pi H_s}{gT_p^2}}} = \frac{0.5}{\sqrt{\frac{2\times\Pi\times 0.56}{9.81\times 3.4^2}}} = 2.84$$

Since $\xi_p = 2.84$, the value for relative wave run-up is determined by:

$$\frac{R_{2\%}}{H_s} = 3.2\gamma_b\gamma_f\gamma_\beta$$

In reality, the wave run-up is affected by some factors such as slope roughness, permeability, berm, oblique wave attack, etc. The determination of these factors can be calculated as follows:

- For the Duong River dike, the covering layer is grass, then $\gamma_R = 0.9$
- Reduction factor due to berm can be calculated by Van der Meer formula:

$$\gamma_b = 1 - r_b + 0.5r_b\left(\frac{d_B}{H_s}\right)^2$$

$$r_b = \frac{B/H_s}{2COT\alpha + B/H_s}$$

The purpose of berm design for the Duong River dike is only for maintenance and dike stability so the berm is often designed below the design water level. At this level, the effect of the berm to the wave run-up becomes zero. Thus $\gamma_b = 1$

- Reduction factor due to oblique wave attack with short-crested wave is calculated by:

$$\gamma_\beta = 1 - 0.0022\beta$$

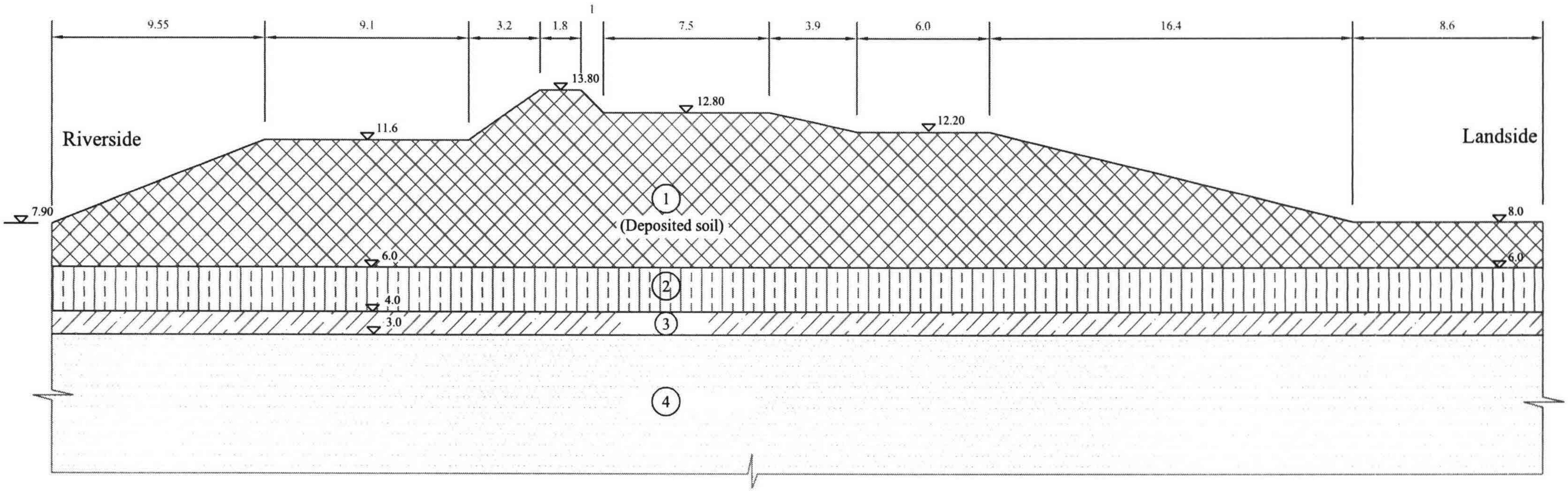
Assuming that $\beta = 0^0$ then $\gamma_\beta = 1$

Therefore,

$$R_{2\%} = 3.2\times H_s\times 0.9\times 1\times 1 = 3.2\times 0.56\times 0.9\times 1\times 1 = 1.62(m)$$

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION
AT KM 1+850 (SEGMENT 1)

SCALE 1/250

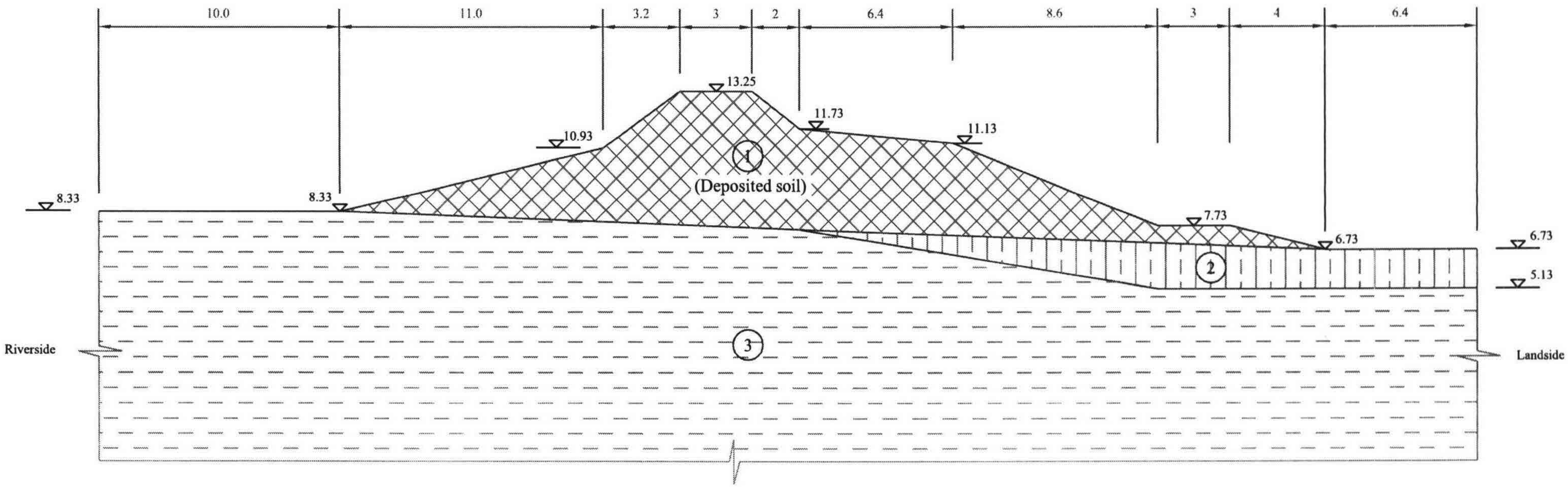


THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1800	1410	47.7	19.0	18.0	6.1×10^{-7}	Sandy CLAY of low plasticity
2	1790	1390	48.3	22.3	12.0	4.5×10^{-7}	Sandy CLAY of intermediate plasticity
3	1760	1360	49.6	14.5	10.0	6.3×10^{-6}	Sandy SILT
4	1700	-	46.2	0	28.0	8.0×10^{-5}	Saturated well - graded SAND

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION AT KM 10+500 (SEGMENT 2)

SCALE 1/250

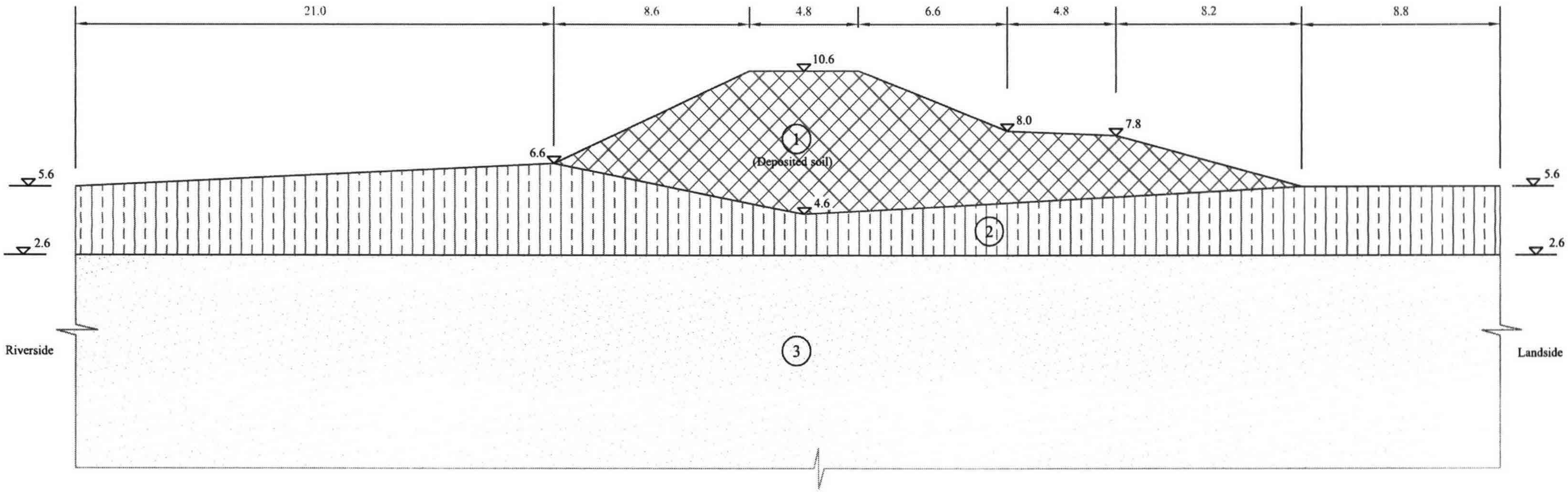


THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1880	1550	43	18.0	17.0	6.5×10^{-7}	Sandy CLAY of low plasticity (brown)
2	1860	1530	43	19.5	10.0	5.6×10^{-7}	Sandy CLAY of low plasticity (red- brown)
3	1820	1370	50	24.0	9.0	1.5×10^{-8}	CLAY of intermediate plasticity

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION AT KM 28.5 (SEGMENT 3)

SCALE 1/250

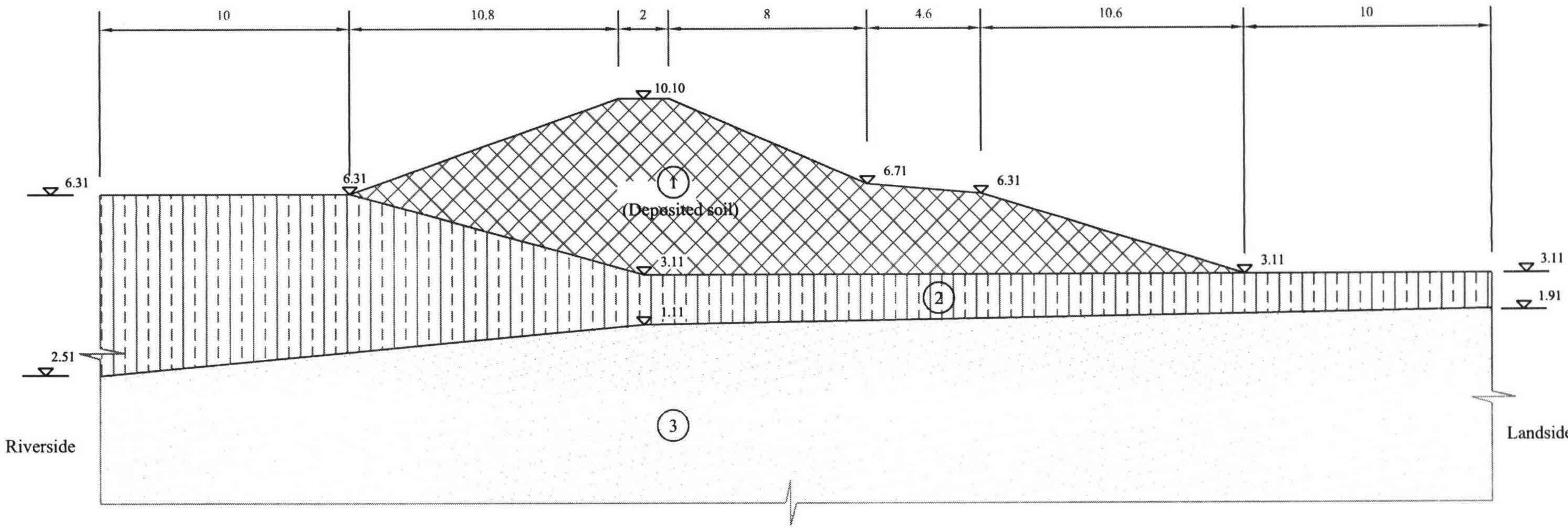


THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1870	1470	46.5	21.1	15	6.8×10^{-7}	Sandy CLAY of intermediate plasticity (brown)
2	1830	1340	46.4	18	13	5.5×10^{-7}	Sandy CLAY of low plasticity (red- brown)
3	1700	1400	48.5	0	26	7.8×10^{-5}	Saturated well - graded SAND

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION AT KM 37+00 (SEGMENT 4)

SCALE 1/250

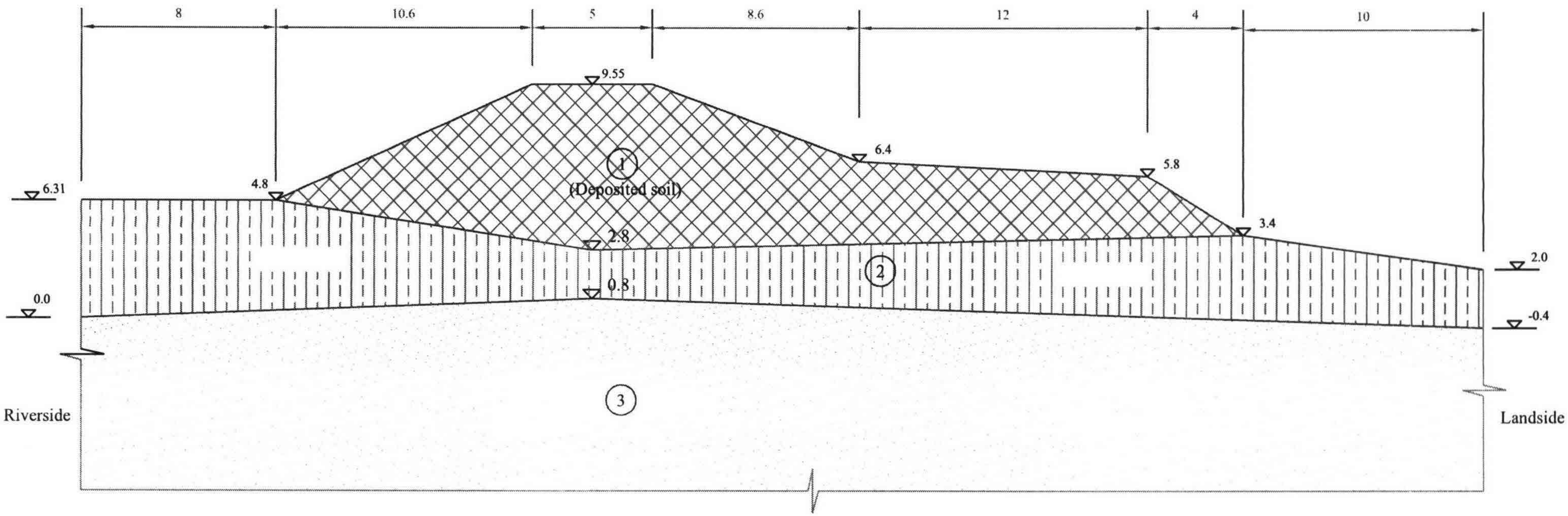


THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1870	1519	44.2	21.0	15.5	$7.1x10^{-7}$	Sandy CLAY of intermediate plasticity (brown)
2	1870	1464	45.8	18.0	12.0	$5.0x10^{-7}$	Sandy CLAY of low plasticity (red- brown)
3	1700	1400	48.5	0	27.0	$7.8x10^{-5}$	Saturated uniform SAND

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION
AT KM 45+00 (SEGMENT 5)

SCALE 1/250

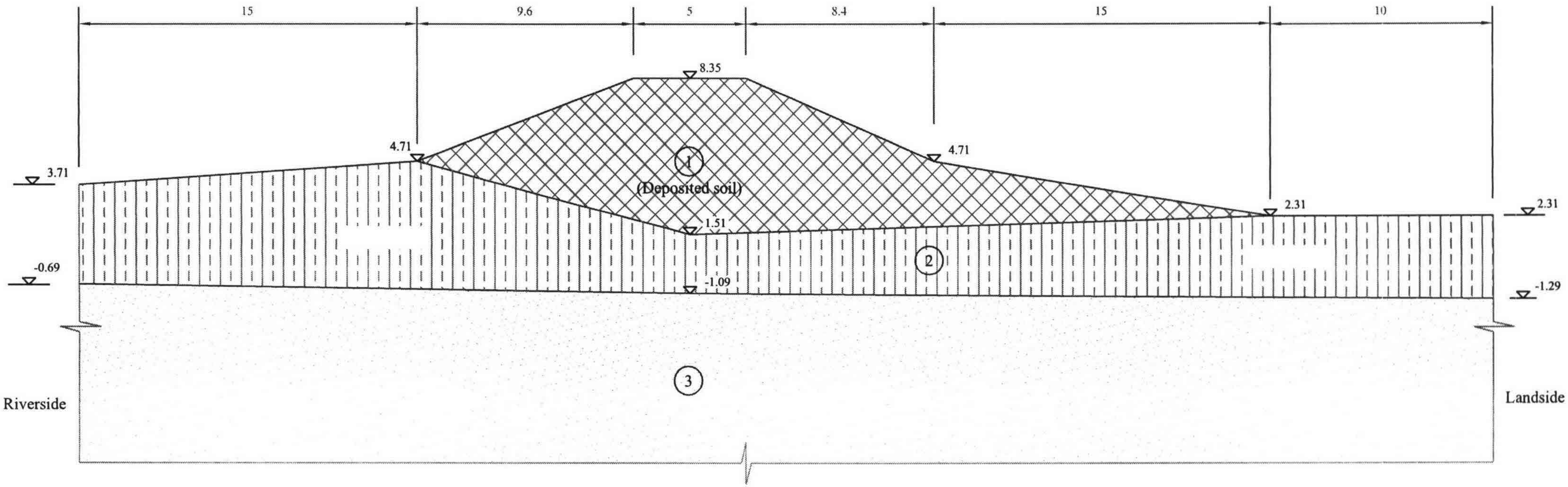


THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1870	1470	46.5	21.1	18.5	6.2×10^{-7}	Sandy CLAY of intermediate plasticity (brown)
2	1830	1340	46.4	18.5	12.0	5.5×10^{-7}	Sandy CLAY of low plasticity (red- brown)
3	1700	1400	48.5	0	25.5	8.2×10^{-5}	Saturated well - graded SAND

THE RIGHT DUONG RIVER DIKE SCHEMATISED CROSS- SECTION AT KM 54+500 (SEGMENT 6)

SCALE 1/250



THE AVERAGE VALUES OF SOIL PROPERTY

Layer	Wet density (kg/m3)	Dry density (kg/m3)	Porosity (%)	Cohesion (kN/m2)	Angle of internal friction (degree)	Permeability (m/s)	Name
1	1870	1470	46.5	20.5	17.5	6.8×10^{-7}	Sandy CLAY of intermediate plasticity (brown)
2	1830	1340	46.4	18.5	10.5	4.4×10^{-7}	Sandy CLAY of low plasticity (red- brown)
3	1700	1400	48.5	0	26	8.0×10^{-5}	Saturated well - graded SAND

Appendix IV: The characteristics of all conduits under the right Duong River dike

Number	Conduit	Location (km)	Type	Length of conduit (m)	Bottom level (m)	Gate dimension (m)	Number of gate	Time of construction	Present condition		Remark
									good	not good	
1	2	3	4	5	6	7	8	9	10	11	12
	In Ha noi										
1	Gia Thuong	K1+936	Circular	32.80	11.87	D = 1,0	1	1959	x		
2	Keo Go	K5+572	Rectangular	28.00	11.50	3.7x1.6	1	1958		x	
3	Giang Bien	K5+768	Circular	24.00	8.80	D = 0,6	1	1964	x		
4	Vang	K9+996	Circular	31.00	7.50	D = 1	1	1958	x		
5	Loi	K13+027	Circular	28.00	5.50	D = 1,2	1		x		
	In Bac Ninh										
6	Phu My	K25.520	Rectangular	49.50	3.00	1.4x1.8	1	1967		x	
7	Mon Quang	K37+540	Rectangular	49.50	2.60	1.4x1.8	1	1968		x	
8	Ngam Mac	K38+383									
9	Tram	K44+500									
	Total	**9**							**4**	**4**	

APPENDIX V: Results of detail calculations for overflowing and piping mechanisms by using VAP model

DETAIL RESULTS OF THE FAILURE PROBABILITY CALCULATIONS FOR OVERFLOWING MECHANISM BY VAP model

Limit state function for all segments:

G = h-H

SEGMENT 1

T = 20 year (present)

Variables of G:

H	sEX	9.950	1.163
h	N	13.800	0.150

FORM Analysis of G:

HL - Index = 2.27   P(G<0) = 0.0115

Name	Alpha	Design Value
H	0.998	13.777
h	-0.067	13.777

T= 50 years

Variables of G:

H	sEX	9.950	1.163
h	N	14.930	0.150

FORM Analysis of G:

HL - Index = 2.74   P(G<0) = 0.0031

Name	Alpha	Design Value
H	0.998	14.906
h	-0.057	14.906

T= 100 years

Variables of G:

H	sEX	9.950	1.163
h	N	15.530	0.150

FORM Analysis of G:

HL - Index = 2.96   P(G<0) = 0.00154

Name	Alpha	Design Value
H	0.999	15.506
h	-0.054	15.506

T= 150 years

Variables of G:

H	sEX	9.950	1.163
h	N	15.880	0.150

FORM Analysis of G:

HL - Index = 3.08   P(G<0) = 0.00103

Name	Alpha	Design Value
H	0.999	15.856
h	-0.052	15.856

T= 200 years

Variables of G:

H	sEX	9.950	1.163
h	N	16.130	0.150

FORM Analysis of G:

HL - Index = 3.17   P(G<0) = 0.000768

Name	Alpha	Design Value
H	0.999	16.106
h	-0.051	16.106

T= 250 years

Variables of G:

H	sEX	9.950	1.163
h	N	16.320	0.150

FORM Analysis of G:

HL - Index = 3.23   P(G<0) = 0.000616

Name	Alpha	Design Value
H	0.999	16.296
h	-0.050	16.296

T= 500 years

Variables of G:

H	sEX	9.950	1.163
h	N	16.910	0.150

FORM Analysis of G:

HL - Index = 3.42   P(G<0) = 0.00031

Name	Alpha	Design Value
H	0.999	16.886
h	-0.047	16.886

T= 1000 years

Variables of G:

H	sEX	9.950	1.163
h	N	17.510	0.150

FORM Analysis of G:

HL - Index = 3.61   P(G<0) = 0.000154

Name	Alpha	Design Value
H	0.999	17.486
h	-0.045	17.486

SEGMENT 2

T = 20 year (present)

Variables of G:

H	sEX	8.960	1.163
h	N	13.250	0.150

FORM Analysis of G:

HL - Index = 2.46   P(G<0) = 0.00691

Name	Alpha	Design Value
H	0.998	13.227
h	-0.063	13.227

SEGMENT 3

.T = 20 year (present)

Variables of G:

H	sEX	6.910	1.163
h	N	10.600	0.150

FORM Analysis of G:

HL - Index = 2.2   P(G<0) = 0.0139

Name	Alpha	Design Value
H	0.998	10.577
h	-0.068	10.577

SEGMENT 4

T = 20 year (present)

Variables of G:

H	sEX	5.940	1.163
h	N	10.100	0.150

FORM Analysis of G:

HL - Index = 2.41   P(G<0) = 0.00804

Name	Alpha	Design Value
H	0.998	10.077
h	-0.064	10.077

SEGMENT 5

T = 20 year (present)

Variables of G:

H	sEX	5.030	1.163
h	N	9.550	0.150

FORM Analysis of G:

HL - Index = 2.56 P(G<0) = 0.00529

Name	Alpha	Design Value
H	0.998	9.527
h	-0.061	9.527

SEGMENT 6

T = 20 year (present)

Variables of G:

H	sEX	3.940	1.163
h	N	8.350	0.150

FORM Analysis of G:

HL - Index = 2.51 P(G<0) = 0.00601

Name	Alpha	Design Value
H	0.998	8.327
h	-0.062	8.327

DETAIL CALCULATIONS FOR THE RUPTURING CONDITION BY VAP Model (Piping mechanism)

SEGMENT 1

Limit state function:

G = 18*d-10*(h-4)

Variables of G:

d	N	4.000	0.500
h	sEX	9.950	1.163

FORM Analysis of G:

HL - Index = 0.545 P(G<0) = 0.293

Name	Alpha	Design Value
d	-0.703	3.808
h	0.711	10.855

SEGMENT 3

Limit state function:

G = 18.3*d-10*(h-2.6)

Variables of G:

d	N	3.000	0.500
h	sEX	6.910	1.163

FORM Analysis of G:

HL - Index = 0.486 P(G<0) = 0.313

Name	Alpha	Design Value
d	-0.720	2.825
h	0.693	7.770

SEGMENT 4

Limit state function:

G = 18.3*d-10*(h-1.91)

Variables of G:

d	N	1.200	0.500
h	sEX	5.940	1.163

FORM Analysis of G:

HL - Index = -2.37 P(G<0) = 0.991

Name	Alpha	Design Value
d	-0.944	2.317
h	0.331	6.150

SEGMENT 5

Limit state function:

G = 18.3*d-10*(h+0.4)

Variables of G:

d	N	1.200	0.500
h	sEX	5.030	1.163

Crude Monte Carlo Analysis of G:

1 run with 100000 samples:
1. m = -40.9996 s = 12.6567 p = 0.99999

SEGMENT 6

Limit state function:

G = 18.3*d-10*(h+1.29)

Variables of G:

d	N	3.400	0.500
h	sEX	3.940	1.163

FORM Analysis of G:

HL - Index = 0.335 P(G<0) = 0.369

Name	Alpha	Design Value
d	-0.748	3.275
h	0.664	4.703

DETAIL CALCULATIONS FOR THE SAND CARRYING BOIL CONDITION BY VAP Model (Piping mechanism)

Limit state function:

G = m*(L1/C1+d/C2)-(H-h-d)

SEGMENT 1

Present condition and T= 50 years
Variables of G:

C1	D	6.000	
C2	D	15.000	
H	sEX	9.950	1.163
L1	N	48.000	2.000
d	N	4.000	0.500
h	D	4.000	
m	N	1.670	0.334

FORM Analysis of G:
HL - Index = 3.64 P(G<0) = 0.000137

Name	Alpha	Design Value
H	0.631	13.839
L1	-0.069	47.496
d	-0.146	3.734
m	-0.759	0.748

L1 = 49 (T= 100 years)
FORM Analysis of G:

HL - Index = 3.67 P(G<0) = 0.00012

Name	Alpha	Design Value
H	0.622	13.814
L1	-0.067	48.508
d	-0.144	3.735
m	-0.767	0.729

L1 = 50 (T= 150 and 200 years)
FORM Analysis of G:

HL - Index = 3.71 P(G<0) = 0.000105

Name	Alpha	Design Value
H	0.613	13.785
L1	-0.065	49.521
d	-0.143	3.735
m	-0.774	0.711

L1 = 51 ((T= 250 years)
FORM Analysis of G:

HL - Index = 3.74 P(G<0) = 9.3e-05

Name	Alpha	Design Value
H	0.604	13.752
L1	-0.062	50.534
d	-0.141	3.736
m	-0.782	0.694

L1 = 54 (T= 500 years)
FORM Analysis of G:

HL - Index = 3.83 P(G<0) = 6.51e-05

Name	Alpha	Design Value
H	0.576	13.635
L1	-0.056	53.570
d	-0.137	3.738
m	-0.804	0.642

L1 = 60 (T= 1000 years)
FORM Analysis of G:

HL - Index = 3.98 P(G<0) = 3.48e-05

Name	Alpha	Design Value
H	0.517	13.320
L1	-0.045	59.640
d	-0.129	3.744
m	-0.845	0.547

SEGMENT 3

Limit state function:
G = m*(L1/C1+d/C2)-(H-h-d)

Present condition
Variables of G:

C1	D	6.000	
C2	D	15.000	
H	sEX	6.910	1.163
L1	N	35.000	2.000
d	N	3.000	0.500
h	D	2.600	
m	N	1.670	0.334

FORM Analysis of G:

HL - Index = 3.25 P(G<0) = 0.000571

Name	Alpha	Design Value
H	0.757	11.191
L1	-0.105	34.320
d	-0.168	2.726
m	-0.622	0.994

L1 = 35.7 (T= 50 years)
FORM Analysis of G:

HL - Index = 3.29 P(G<0) = 0.000503

Name	Alpha	Design Value
H	0.753	11.222
L1	-0.102	35.029
d	-0.167	2.726
m	-0.629	0.979

L1 = 37 (T= 100 years)
FORM Analysis of G:

HL - Index = 3.35 P(G<0) = 0.000398

Name	Alpha	Design Value
H	0.744	11.272
L1	-0.098	36.346
d	-0.163	2.726
m	-0.640	0.953

L1 = 38(T= 150 years)
FORM Analysis of G:

HL - Index = 3.4 P(G<0) = 0.000335

Name	Alpha	Design Value
H	0.738	11.304
L1	-0.094	37.359
d	-0.161	2.726
m	-0.649	0.933

L1 = 39 (T= 200 years)
FORM Analysis of G:

HL - Index = 3.45 P(G<0) = 0.000282

Name	Alpha	Design Value
H	0.731	11.329
L1	-0.091	38.373
d	-0.159	2.726
m	-0.658	0.913

L1 = 41 (T= 250 years)
FORM Analysis of G:

HL - Index = 3.54 P(G<0) = 0.000203

Name	Alpha	Design Value
H	0.717	11.365
L1	-0.085	40.399
d	-0.155	2.727
m	-0.675	0.873

L1 = 47 (T= 500 years)
FORM Analysis of G:

HL - Index = 3.77 P(G<0) = 8.27e-05

Name Alpha Design Value

H	0.669	11.332
L1	-0.069	46.479
d	-0.144	2.729
m	-0.726	0.757

L1 = 59 (T= 1000 years)
FORM Analysis of G:

HL - Index = 4.1   P(G<0) = 2.02e-05

Name	Alpha	Design Value
H	0.557	10.778
L1	-0.045	58.632
d	-0.128	2.738
m	-0.819	0.547

SEGMENT 4

Limit state function:

G = m*(L1/C1+d/C2)-(H-h-d)

Present condition
Variables of G:

C1	D	6.000	
C2	D	15.000	
H	sEX	5.940	1.163
L1	N	46.000	2.000
d	N	1.200	0.500
h	D	1.910	
m	N	1.670	0.334

FORM Analysis of G:

HL - Index = 3.27   P(G<0) = 0.000547

Name	Alpha	Design Value
H	0.616	9.215
L1	-0.083	45.455
d	-0.159	0.941
m	-0.767	0.833

L1 = 47 (T= 50 years)
FORM Analysis of G:

HL - Index = 3.31   P(G<0) = 0.000473

Name	Alpha	Design Value
H	0.608	9.214
L1	-0.081	46.467
d	-0.157	0.941
m	-0.774	0.815

L1 = 48 (T= 100 years)
FORM Analysis of G:

HL - Index = 3.35   P(G<0) = 0.00041

Name	Alpha	Design Value
H	0.600	9.207
L1	-0.078	47.479
d	-0.154	0.942
m	-0.781	0.797

L1 = 49.5 (T= 150 years)
FORM Analysis of G:

HL - Index = 3.4   P(G<0) = 0.000333

Name	Alpha	Design Value
H	0.587	9.189
L1	-0.074	48.997
d	-0.151	0.942
m	-0.792	0.770

L1 = 51 (T= 200 years)
FORM Analysis of G:

HL - Index = 3.46   P(G<0) = 0.000273

Name	Alpha	Design Value
H	0.574	9.163
L1	-0.070	50.515
d	-0.149	0.943
m	-0.802	0.744

L1 = 52 (T= 250 years)
FORM Analysis of G:

HL - Index = 3.49   P(G<0) = 0.00024

Name	Alpha	Design Value
H	0.565	9.141
L1	-0.068	51.527
d	-0.147	0.944
m	-0.809	0.727

L1 = 59 (T= 500 years)
FORM Analysis of G:

HL - Index = 3.7   P(G<0) = 0.000107

Name	Alpha	Design Value
H	0.502	8.910
L1	-0.053	58.606
d	-0.135	0.950
m	-0.853	0.615

L1 = 72 (T= 1000 years)
FORM Analysis of G:

HL - Index = 3.98   P(G<0) = 3.41e-05

Name	Alpha	Design Value
H	0.385	8.326
L1	-0.034	71.726
d	-0.117	0.966
m	-0.915	0.453

SEGMENT 5

Limit state function:

G = m*(L1/C1+d/C2)-(H-h-d)

Present condition
Variables of G:

C1	D	6.000	
C2	D	12.000	
H	sEX	5.030	1.163
L1	N	45.000	2.000
d	N	2.000	0.500
h	D	-0.400	
m	N	1.670	0.334

FORM Analysis of G:

HL - Index = 3.06   P(G<0) = 0.00111

Name	Alpha	Design Value
H	0.588	7.891
L1	-0.090	44.449
d	-0.167	1.744
m	-0.786	0.867

L1 = 46 (T= 50 years)
FORM Analysis of G:

HL - Index = 3.1   P(G<0) = 0.000954

Name	Alpha	Design Value
H	0.581	7.896
L1	-0.087	45.461
d	-0.164	1.745
m	-0.793	0.848

L1 = 47 (T= 100 years)
FORM Analysis of G:

HL - Index = 3.15   P(G<0) = 0.000825

Name	Alpha	Design Value
H	0.573	7.897
L1	-0.084	46.472
d	-0.162	1.745
m	-0.799	0.830

L1 = 48 (T= 150 years)
FORM Analysis of G:

HL - Index = 3.19   P(G<0) = 0.000716

Name	Alpha	Design Value
H	0.565	7.894
L1	-0.081	47.484
d	0.160	1.745
m	-0.805	0.812

L1 = 49.5 (T= 200 years)
FORM Analysis of G:

HL - Index = 3.25   P(G<0) = 0.000583

Name	Alpha	Design Value
H	0.553	7.885
L1	-0.077	4.900e+01
d	-0.156	1.746
m	-0.814	0.787

L1 = 51 (T= 250 years)
FORM Analysis of G:

HL - Index = 3.3   P(G<0) = 0.000478

Name	Alpha	Design Value
H	0.541	7.866
L1	-0.073	50.518
d	-0.153	1.747
m	-0.824	0.761

L1 = 57 (T= 500 years)
FORM Analysis of G:

HL - Index = 3.5   P(G<0) = 0.000231

Name	Alpha	Design Value
H	0.489	7.730
L1	-0.060	56.583
d	-0.142	1.752
m	-0.858	0.666

L1 = 69 (T= 1000 years)
FORM Analysis of G:

HL - Index = 3.8   P(G<0) = 7.33e-05

Name	Alpha	Design Value
H	0.388	7.310
L1	-0.040	68.694
d	-0.123	1.767
m	-0.913	0.513

SEGMENT 6

Limit state function:

G = m*(L1/C1+d/C2)-(H-h-d)

Present condition
Variables of G:

C1	D	6.000	
C2	D	12.000	
H	sEX	3.940	1.163
L1	N	43.000	2.000
d	N	3.400	0.500
h	D	-1.290	
m	N	1.670	0.334

FORM Analysis of G:

HL - Index = 3.49   P(G<0) = 0.000244

Name	Alpha	Design Value
H	0.676	7.974
L1	-0.081	42.432
d	-0.156	3.128
m	-0.715	0.837

L1 = 43.5 (T= 50 years)
FORM Analysis of G:

HL - Index = 3.51   P(G<0) = 0.000223

Name	Alpha	Design Value
H	0.671	7.973
L1	-0.080	43.040
d	-0.155	3.128
m	-0.720	0.825

L1 = 45 (T= 100 years)
FORM Analysis of G:

HL - Index = 3.57   P(G<0) = 0.000181

Name	Alpha	Design Value
H	0.660	7.967
L1	-0.076	44.458
d	-0.152	3.129
m	-0.731	0.799

L1 = 46 (T= 150 years)
FORM Analysis of G:

HL - Index = 3.6   P(G<0) = 0.000157

Name	Alpha	Design Value
H	0.652	7.955
L1	-0.073	45.471
d	-0.150	3.129
m	-0.739	0.780

L1 = 47 (T= 200 years)
FORM Analysis of G:

HL - Index = 3.64   P(G<0) = 0.000137

Name	Alpha	Design Value
H	0.644	7.938
L1	-0.071	46.484
d	-0.149	3.130
m	-0.747	0.761

L1 = 48 (T= 250 years)
FORM Analysis of G:

HL - Index = 3.67   P(G<0) = 0.000119

Name	Alpha	Design Value
H	0.635	7.916
L1	-0.068	47.497
d	-0.147	3.130
m	-0.755	0.743

L1 = 53 (T= 500 years)
FORM Analysis of G:

HL - Index = 3.83   P(G<0) = 6.37e-05

Name	Alpha	Design Value
H	0.590	7.748
L1	-0.057	52.560
d	-0.139	3.134
m	-0.794	0.654

L1 = 63 (T= 1000 years)
FORM Analysis of G:

HL - Index = 4.07   P(G<0) = 2.32e-05

Name	Alpha	Design Value
H	0.491	7.194
L1	-0.040	62.674
d	-0.125	3.145
m	-0.861	0.499

APPENDIX VI: Results of detail calculations for macro-instability by using MPROSTAB model

```
MM    MM  PPPPPP  RRRRR   OOOOOO  SSSSSS TTTTTT AAAAAA BBBBBB
MMM  MMM  PP PP  RR RR  OO OO  SS      TT   AA AA  BB BB
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MM M MM  PP      RR RR   OO OO      SS  TT   AA AA  BB BB
MM    MM  PP      RR RR  OOOOOO  SSSSSS   TT   AA AA  BBBBBB
```

Version : 2.4.0 PROGRAM MPROSTAB
Update : 010206 ==================================
Licence : beta PROBABILISTIC ANALYSIS OF STABILITY
Copy : 1

Date : 2002-03-15
Time : 10:19:02

Name of project : Section 2 - Duong River dike
Present condition

Input file : C:\MPROSTAB\S2\S2-PRE.PSI
Dump file : C:\MPROSTAB\S2\S2-PRE.PSD
Restart file : C:\MPROSTAB\S2\S2-PRE.PSR

Output files :
Mean Value Calculation : C:\MPROSTAB\S2\S2-PRE.POM
Design Point Calculation : C:\MPROSTAB\S2\S2-PRE.POD
A Posteriori Analysis : C:\MPROSTAB\S2\S2-PRE.POA

****

MPROSTAB : Probabilistic analysis of slope stability.

RESTART RUN, restart parameter : 1

Param=1 : Design Point Calculation
Param=2 : A Posteriori Analysis

For a summary of the input data used in the calculations,
see to the clean-start outputfile (ext. *.POM) !

PHASE 1 SLIP CIRCLE NR. 62
==

X-center= 41.14 m. Y-center= 20.00 m. R= 14.15 m.

Bishop factor : 2.358

Hasofer-Lind reliability index : 10.003

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 47
estimated mean value : 68
TAN(phi) : spatial variability : 19
estimated mean value : 26
Correlation cohesion-tan(phi) : -73
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 13
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 7.374E-0024
Including effect of length (P2) : 6.841E-0021
Expected length of instability E(l) : 11 m
Prob. of failure incl. edge effects (P3) : 6.841E-0021

DIRECTIONAL COSINES OF DESIGN POINT.
===

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.493	0.245
2	0.663	0.443
3	0.000	0.000

alfa phreatic line : -0.360

PHASE 1 SLIP CIRCLE NR. 22
==

X-center= 34.29 m. Y-center= 25.00 m. R= 20.00 m.

Bishop factor : 2.173

Hasofer-Lind reliability index : 10.719

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 49
estimated mean value : 72
TAN(phi) : spatial variability : 26
estimated mean value : 38
Correlation cohesion-tan(phi) : -91
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 6
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 4.141E-0027
Including effect of length (P2) : 4.378E-0024
Expected length of instability E(l) : 10 m
Prob. of failure incl. edge effects (P3) : 4.378E-0024

DIRECTIONAL COSINES OF DESIGN POINT.
===

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.338	0.141
2	0.241	0.146
3	0.740	0.581

alfa phreatic line : -0.246

PHASE 1 SLIP CIRCLE NR. 25
==

X-center= 34.29 m. Y-center= 27.50 m. R= 22.50 m.

Bishop factor : 2.215

Hasofer-Lind reliability index : 10.841

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 48
estimated mean value : 72
TAN(phi) : spatial variability : 25
estimated mean value : 37
Correlation cohesion-tan(phi) : -88
Uncertainty exc. porepressure : 0

Uncertainty phreatic line : 6
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.104E-0027
Including effect of length (P2) : 1.172E-0024
Expected length of instability E(l) : 10 m
Prob. of failure incl. edge effects (P3) : 1.172E-0024

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.324	0.108
2	0.245	0.139
3	0.742	0.580

alfa phreatic line : -0.255

End of calculations MPROSTAB
============================

```
MM   MM  PPPPPP  RRRRR  OOOOOO  SSSSSS TTTTTT AAAAAA BBBBBB
MMM  MMM  PP PP  RR RR  OO OO  SS     TT  AA AA BB BB
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MM M MM  PP     RR RR  OO OO    SS  TT   AA AA BB BB
MM    MM  PP     RR RR  OOOOOO  SSSSSS  TT  AA AA BBBBBB
```

Version : 2.4.0 PROGRAM MPROSTAB
Update : 010206 ===================================
Licence : beta PROBABILISTIC ANALYSIS OF STABILITY
Copy : 1

Date : 2002-03-15
Time : 10:09:40

Name of project : Section 3 - Duong River dike
Present condition

Input file : C:\MPROSTAB\S3\S3-PRE.PSI
Dump file : C:\MPROSTAB\S3\S3-PRE.PSD
Restart file : C:\MPROSTAB\S3\S3-PRE.PSR

Output files :
Mean Value Calculation : C:\MPROSTAB\S3\S3-PRE.POM
Design Point Calculation : C:\MPROSTAB\S3\S3-PRE.POD
A Posteriori Analysis : C:\MPROSTAB\S3\S3-PRE.POA

****

MPROSTAB : Probabilistic analysis of slope stability.

RESTART RUN, restart parameter : 1

Param=1 : Design Point Calculation
Param=2 : A Posteriori Analysis

For a summary of the input data used in the calculations,
see to the clean-start outputfile (ext. *.POM) !

PHASE 1 SLIP CIRCLE NR. 37
===

X-center= 35.71 m. Y-center= 27.50 m. R= 27.50 m.

Bishop factor : 1.851

Hasofer-Lind reliability index : 7.093

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 11
estimated mean value : 20
TAN(phi) : spatial variability : 27
estimated mean value : 42
Correlation cohesion-tan(phi) : -21
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 21
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 6.563E-0013
Including effect of length (P2) : 3.425E-0010
Expected length of instability E(l) : 20 m
Prob. of failure incl. edge effects (P3) : 3.425E-0010

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.227	0.087
2	0.383	0.265
3	0.000	0.583

alfa phreatic line : -0.458

PHASE 1 SLIP CIRCLE NR. 34
==

X-center= 35.71 m. Y-center= 23.75 m. R= 23.75 m.

Bishop factor : 1.825

Hasofer-Lind reliability index : 7.035

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 12
estimated mean value : 20
TAN(phi) : spatial variability : 27
estimated mean value : 41
Correlation cohesion-tan(phi) : -21
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 22
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 9.994E-0013
Including effect of length (P2) : 5.145E-0010
Expected length of instability E(l) : 20 m
Prob. of failure incl. edge effects (P3) : 5.145E-0010

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.248	0.102
2	0.368	0.265
3	0.000	0.577

alfa phreatic line : -0.468

PHASE 1 SLIP CIRCLE NR. 19
==

X-center= 32.86 m. Y-center= 23.75 m. R= 23.75 m.

Bishop factor : 1.925

Hasofer-Lind reliability index : 7.327

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 11
estimated mean value : 18
TAN(phi) : spatial variability : 30
estimated mean value : 46
Correlation cohesion-tan(phi) : -20
Uncertainty exc. porepressure : 0

Uncertainty phreatic line : 16
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.181E-0013
Including effect of length (P2) : 6.468E-0011
Expected length of instability E(l) : 19 m
Prob. of failure incl. edge effects (P3) : 6.468E-0011

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.190	0.038
2	0.378	0.282
3	0.000	0.614

alfa phreatic line : -0.400

PHASE 1 SLIP CIRCLE NR. 40
==

X-center= 35.71 m. Y-center= 31.25 m. R= 31.25 m.

Bishop factor : 1.896

Hasofer-Lind reliability index : 7.253

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 11
estimated mean value : 20
TAN(phi) : spatial variability : 28
estimated mean value : 44
Correlation cohesion-tan(phi) : -22
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 19
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 2.034E-0013
Including effect of length (P2) : 1.102E-0010
Expected length of instability E(l) : 19 m
Prob. of failure incl. edge effects (P3) : 1.102E-0010

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.202	0.058
2	0.401	0.283
3	0.000	0.595

alfa phreatic line : -0.432

PHASE 1 SLIP CIRCLE NR. 22
==

X-center= 32.86 m. Y-center= 27.50 m. R= 27.50 m.

Bishop factor : 1.971

NO CONVERGENCY after 30 steps in the iteration-process !

The dumpfile will not be filled (no drawing possible).

Hasofer-Lind reliability index : 7.406

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 10
estimated mean value : 18
TAN(phi) : spatial variability : 29
estimated mean value : 45
Correlation cohesion-tan(phi) : -20
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 16
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 6.505E-0014
Including effect of length (P2) : 3.604E-0011
Expected length of instability E(l) : 18 m
Prob. of failure incl. edge effects (P3) : 3.604E-0011

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.152	0.013
2	0.401	0.274
3	0.000	0.616

alfa phreatic line : -0.399

End of calculations MPROSTAB
============================

```
MM   MM  PPPPPP  RRRRR   OOOOOO  SSSSSS  TTTTTT  AAAAAA  BBBBBB
MMM  MMM  PP  PP  RR  RR  OO  OO  SS      TT   AA  AA  BB  BB
MM MMM MM  PPPPPP  RRRRR   OO  OO  SSSSSS  TT   AAAAAA  BBBBBB
MM  M  MM  PP      RR RR   OO  OO      SS  TT   AA  AA  BB  BB
MM     MM  PP      RR  RR  OOOOOO  SSSSSS  TT   AA  AA  BBBBBB
```

```
Version :    2.4.0          PROGRAM  MPROSTAB
Update  :    010206         ==================================
Licence :    beta           PROBABILISTIC ANALYSIS OF STABILITY
Copy    :    1
```

Date : 2002-03-14
Time : 20:57:25

Name of project : Section 3
Design return period = 1000 years

Input file : S3-PRE.PSI
Dump file : S3-PRE.PSD
Restart file : S3-PRE.PSR

Output files :
Mean Value Calculation : S3-PRE.POM
Design Point Calculation : S3-PRE.POD
A Posteriori Analysis : S3-PRE.POA

****

MPROSTAB : Probabilistic analysis of slope stability.

RESTART RUN, restart parameter : 1

Param=1 : Design Point Calculation
Param=2 : A Posteriori Analysis

For a summary of the input data used in the calculations,
see to the clean-start outputfile (ext. *.POM) !

PHASE 1 SLIP CIRCLE NR. 103
==

X-center= 57.86 m. Y-center= 35.00 m. R= 33.00 m.

Bishop factor : 1.438

Hasofer-Lind reliability index : 5.086

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 27
estimated mean value : 43
TAN(phi) : spatial variability : 34
estimated mean value : 43
Correlation cohesion-tan(phi) : -59
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 13
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.832E-0007
Including effect of length (P2) : 8.647E-0005
Expected length of instability E(l) : 22 m
Prob. of failure incl. edge effects (P3) : 8.647E-0005

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.388	0.271
2	0.524	0.444
3	0.000	0.396

alfa phreatic line : -0.364

PHASE 1 SLIP CIRCLE NR. 88
==

X-center= 55.71 m. Y-center= 35.00 m. R= 33.00 m.

Bishop factor : 1.447

Hasofer-Lind reliability index : 5.155

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 25
estimated mean value : 39
TAN(phi) : spatial variability : 36
estimated mean value : 45
Correlation cohesion-tan(phi) : -57
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 12
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.266E-0007
Including effect of length (P2) : 6.061E-0005
Expected length of instability E(l) : 22 m
Prob. of failure incl. edge effects (P3) : 6.061E-0005

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.374	0.281
2	0.504	0.443
3	0.000	0.421

alfa phreatic line : -0.346

PHASE 1 SLIP CIRCLE NR. 100
==

X-center= 57.86 m. Y-center= 31.25 m. R= 29.25 m.

Bishop factor : 1.449

Hasofer-Lind reliability index : 5.144

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 27
estimated mean value : 42
TAN(phi) : spatial variability : 33
estimated mean value : 42
Correlation cohesion-tan(phi) : -58
Uncertainty exc. porepressure : 0

Uncertainty phreatic line : 13
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.341E-0007
Including effect of length (P2) : 6.371E-0005
Expected length of instability E(l) : 22 m
Prob. of failure incl. edge effects (P3) : 6.371E-0005

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.384	0.247
2	0.526	0.446
3	0.000	0.397

alfa phreatic line : -0.362

PHASE 1 SLIP CIRCLE NR. 118
===

X-center= 60.00 m. Y-center= 35.00 m. R= 33.00 m.

Bishop factor : 1.453

Hasofer-Lind reliability index : 5.195

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 29
estimated mean value : 45
TAN(phi) : spatial variability : 31
estimated mean value : 39
Correlation cohesion-tan(phi) : -59
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 16
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 1.024E-0007
Including effect of length (P2) : 4.888E-0005
Expected length of instability E(l) : 22 m
Prob. of failure incl. edge effects (P3) : 4.888E-0005

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.399	0.252
2	0.541	0.443
3	0.000	0.366

alfa phreatic line : -0.394

PHASE 1 SLIP CIRCLE NR. 85
===

X-center= 55.71 m. Y-center= 31.25 m. R= 29.25 m.

Bishop factor : 1.462

Hasofer-Lind reliability index : 5.294

Distribution of variance of reliability function (%) due to:

COHESION : spatial variability : 25
estimated mean value : 40
TAN(phi) : spatial variability : 36
estimated mean value : 45
Correlation cohesion-tan(phi) : -57
Uncertainty exc. porepressure : 0
Uncertainty phreatic line : 11
Corr. exc.porepr./phreat.line : 0
Model uncertainty factor Q : 0

Probability of a safety factor < Q (P1) : 5.979E-0008
Including effect of length (P2) : 2.934E-0005
Expected length of instability E(l) : 21 m
Prob. of failure incl. edge effects (P3) : 2.934E-0005

DIRECTIONAL COSINES OF DESIGN POINT.
==

materialno.	alfa-cohesion	alfa-tan(phi)
1	0.376	0.266
2	0.506	0.447
3	0.000	0.421

alfa phreatic line : -0.338

End of calculations MPROSTAB
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