Probabilistic Risk Analysis and Safety Evaluation of Dikes

Case study: Structural Safety assessment with Multi-criteria evaluation on Anqing Dikes in China

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Delft - October 2003

Preface

This research is done for the partial fulfilment of requirements for the co-operation project between Directorate-General Rijkswaterstaat (RWS) of the Ministry of Transport, Public Works and Water Management in the Netherlands, represented by Road and Hydraulic Engineering Institute in Delft (DWW), and the Flood and Drought disaster Research Center in Beijing (RCDR) of the Ministry of Water Resources in China.

Within the cooperation between two sides some new technologies can be applied and exchanged. The Dutch's experience has been used for reference in the development of China probabilistic risk analysis system involved the whole report.

According to the proposal by DWW, there is also special interest to learn from Chinese experiences. Since it is a continuation of the previous studies, some conclusions from preceding reports of our research in Chinese also were translated and presented in this report as possible as I can.

Since my colleague MS Wang will concentrate on the economic analysis to assess damage that would result from flood. At that moment it will also be possible to calculate the costs and benefits of some alternative mitigating measures. For this case, some section of this report is empty for her study.

Abstract

What are the new mechanisms of inundation of an embanked area? Flooding may not only occur because of too high water levels, but also because of a collapse of a dike when its revetment is damaged, when a slope slide occurs, or when water seeps through or under a dike and thereby weakening it. When determining the probabilities of flood, all these failure mechanisms have to be taken into consideration. In addition, the flood probability is calculated for an entire dike ring area and not for a single dike section only. In this way, a better insight into the protection level of the area can be obtained, Weak links in the dike ring become apparent and bottle-necks can be ranked from large to small. Uncertainties due to lack of knowledge can also be made explicit.

In this reports, the following contents have been involved:

(1) A typical dike with inclined impermeable facing is selected to investigate its safety against piping. A comparison of the Sellmeijer's method, the Chinese method and the Bligh's method, respectively, shows that the Sellmeijer's requires shorter seepage length and hence, a more economical design for water retaining structures against piping. A sensitivity analysis was performed by using the various geometrical variables and geotechnical parameters to investigate the different model's compatibility and accuracy. These parameters include thickness of clay layer and sand layer, width of foreland and cross dike crest and piping-berm, height of water level, slope ratio, permeability coefficient of sand and clay layer.

(2) A brief introduction on the reliability theory has been given, such as the limit state equations for different failure mechanisms, their solving approach, and fault tree. And then, some soil strength parameters of levee and its foundation are taken as random variable, limit state equations of overtopping, piping and sliding of two typical dikes with inclination watertight facing and the homogenous embankment are formulated, and the influences of various factors such as geotechnical statistic parameters and geometry of dikes on reliability index or structure risk degree for these failure modes, have been investigated systematically.

(3) Some functions and features of two software system on the structural safety assessment used in the Netherlands and China have been discussed.

(4) Description of Anqing dike system has been given. Emphasize on discuss of the geological condition and historical yearly maximum water levels of the pilot has been made. The large-scale reinforcement project on this dike since 1998 has been discussed.

(5) A case study on risk assessment has been carried out for the pilot area. The safety factor, probability of failure, probability of flooding of various failure modes for each individual section and a whole dike section have been given at different water levels. A risk map of structural safety and some multi-evaluation indices along the whole dike has been achieved. The combination probability has been presented for each failure mode and every individual section and a whole section, some valuable remarks and conclusions will be helpful to understand the weakest compartments. The total probability of failure of the dike ring has been obtained, which will be employed in a further calculating of the risk of flooding including the economic loss.

Based on the research, a new approach is under development to implement the methodology in China. However, there are still a number of known limitations for which additional research and development appear warranted.

Contents

Chapter 1 Problem Analysis, Project Objective and Working-method

1.1 Necessity for this study

The objective of this study is to apply reliability analysis on the Anqing' flood defence system in China in order to support an evaluation of the appropriateness of these methods.

Construction of dikes is the most important structural measure of flood protection in China and the Netherlands. China has about 250,000 km of dikes, among which the main dikes have a total length of 65,700km. In the Yangtze River, all classes of dikes together take 30,000 km, of which 3,600 km are main dikes. According to the kind of resisted floodwater, there are two categories of dikes: river levees and sea dikes. Each category can be divided into some grades on the basis of the importance of a dike (for example, the Yangtze River dikes include three grades).

Our increased prosperity means natural threats can cause a great deal of damage. And bearing in mind there is less willingness to accept it, citizens are changing the demands they set on government. People want to live their lives without feeling threatened by the water. But more importantly, the manner in which the land is used means that the consequences would be far greater now than they were a few centuries ago. The rise in population meant that increasing numbers of lower-lying areas were taken into use.

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 its consequences.

The aim of this study is:

(1) To analyze / understand and to apply the probabilistic design method and risk assessment to the improvement of safety evaluation of river dike systems.

(2) To apply this knowledge in the Chinese situation and to extend it to other locations.

(3) To appraise the present state of the Yangtze river dike system by probabilistic approach and risk analysis in order to give technical authorities a clearer picture of the water defense system.

In China, information technology has been intensively used in management of hydraulic engineering, which is aim to improve the decision-making technique on flood defence system and disaster reduction. In order to intensity of flood prevention and rush to deal with an emergency during flood period, it is necessary to present or employ new approach of safety assessment on dike engineering, which will become a useful tool to carry out the safety management and risk assessment for engineers and mangers.

In addition, our government has poured much money into the strengthening of the main dike of Yangtze river basin, to a flood water level with a return period of 1000 year height. What are the new mechanisms of inundation of an embanked area? Flooding may not only occur because of too high water levels, but also because of a collapse of a dike when its revetment is damaged, when a slope slide occurs, or when water seeps through or under a dike and thereby weakening it. The flood probability is calculated for an entire dike ring area and not for a single dike section only. In this way, a better insight into the protected area should be obtained. The question as to whether the dike is safe enough, which standard should be maintained, and whether the current safety approach still meets current standards, will have to be answered at political level.

With this study, some new knowledge and ideals should be functional in answering these questions and to support an anticipating approach to the safety discussion.

1.2 The art-of-review on safety evaluation of flood defences in the netherlands and china

1.2.1 Dutch risk assessment method for flood defences

Before 1953 the risk estimates (safety definitions) were either formulated on the basis of intuition or experience. The 'highest known water level' on site played a crucial role. The flood defence was designed at that level plus a certain margin. If a flood defence proved too low for a new and higher water level, then this automatically became the highest known water level and the dike was raised.

After the 1953 disaster the need for a more unambiguous approach, along the lines of the section above was adopted. The safety requirements in the prevailing guides placed on the flood defences have their origins in the Delta Committee’s body of ideas, collated in its 1960 report. An econometric view was set up for Central Holland. The econometric optimum safety level was fixed at approximately 1/125,000 per year, assuming a complete loss of capital goods. The loss of human life and the breakdown of society was not collating in this view.

Assuming a flood probability of 1/125,000 per year this would mean about the same collapse probability for the flood defence. However, it was impossible to determine the probability of collapse of a dike with any precision due to insufficient numeric insight into the collapse mechanisms. As a result, partly in light of the other uncertainties, another approach was chosen. The requirement was that it must be possible to ‘completely retain’ a water level with an exceedance frequency of 1/10,000 per year (the design level, the Normative High Water, MHW). That was considered to be true when only 2% of the accompanying waves running up the dike were exceeding the crest of the dike.

For Hoek van Holland it was determined that a level of NAP +5m has an exceedance frequency of approximately 1/10,000 per year, the so-called base level of the Delta Committee. For Central Holland the design level (MHW) is the same as the base level. For other locations along the coast the base levels have been determined by assuming conditions with the same exceedance frequency. For the MHW along other parts of the coast an economic reduction was applied however, varying from 0.2 to 0.6m. The accompanying exceedance frequency of the MHW along the coast varies from approximately 1/4000 to approximately 1/1500 per year. The economic reduction was motivated by the fact that the consequences in the case of dike collapse are less serious than in Central Holland.

For the upper rivers area the River Dikes Committee (the Becht Committee) recommended an exceedance frequency of the design discharge (to which the MHW is linked) in 1977 equal to 1/1250 per year. The ‘extenuating circumstances’ in the rivers area in relation to the coast included: fresh versus saltwater and in the long term forecasting river floods versus in the short term forecasting of sea floods. The loss of Nature values also played a role in the choice of this number. The Boertien Committee made a similar choice in 1993, the result of which was the same exceedance frequency of 1/1250 per year. This number was chosen in spite of the fact that a material risk consideration would lead to a lower frequency.

The considerations of the Delta Committee and its elaborations have since formed the core of the safety requirements. The exceedance frequencies of the MHWs form at this time the most tangible expression of the degree of protection against high water, offered to various areas.

The concept 'completely safe' is worked out in the design rules. It should include a margin due to uncertainties in such matters as water levels, wave attack, soil & material characteristics and behaviour of the water retaining structures. The Delta Committee chose the freeboard for this margin, the difference between the crest height of a dike and the design level. This difference is decided in the design by such factors as the wave run-up, any fluctuations in the water level due to wind, estimated settlement of the flood defence and such like. The Delta Committee recommended that the freeboard, even when there is no wave attack to speak of, should be at least a couple of decimetres.

In The Netherlands a similar development with respect to risk-based design and maintenance of flood defence systems has taken place. Until recently the Dutch flood defences were expected to be able to withstand a water level with a certain frequency of exceedance. The accepted frequency of exceedance has been established for different areas. These standards of safety are mainly based on the consideration that everyone has equal rights to the same level of safety.

In the last few years the emphasis in safety approaches with regard to flood defences has shifted from the frequency of exceedance approach to a risk-based approach. To support this risk-based approach, software called PC-Ring has been developed to calculate the probability of inundation of flood defence systems. These calculations are based on certain failure modes. These failure modes take both the strength of the flood defence and the local loading conditions into account. Besides the total probability of inundation the output of PC-Ring consists of the probabilities of failure of the included cross sections and the contributions of the uncertainties of random variables to the total uncertainty of reliability functions representing the failure modes. The knowledge of these probabilities provides the possibility to spot the weak elements in the flood defence system. Knowledge of the mentioned contributions indicates how the system can best be improved.

The exceedance frequency is a standard for a high water level that a dike section should be able to withstand. The flood probability of an area is that the entire area will be flooded due to failure of one or several water defence works of that protected area. The differences between the proposed method to determine probabilities of flooding from the current approach (exceedance) is at three levels:

(1) The analysis of an entire dike ring instead of individual dike sections. In this way, the strength of a dike ring as a whole (consisting of dikes, structures and dunes) is determined.

(2) Taking various types of failure mechanisms of a dike ring mutually into account. This is different from the current approach, in which the type of failure is dominated by the failure modes “overtopping” and “overflow of water”.

(3) Systematically and verifiably discounting all uncertainties in advance when calculating the probabilities of flooding. In the current approach, uncertainties are for the greater part discounted afterwards by including an extra safety margin.

1.2.2 Review of safety management and risk assessment of the Yangze River dike

Chinese dikes were gradually formed in the past. The initial Jingjiang dikes of the Yangtze river date from AD 300, so they have a history of over 1600 years. All these dikes are proof of the struggle of the Chinese people against flood for thousands of years. Because dikes were constructed, damaged, and rebuilt repeatedly, there are some potential dangerous occurred during the flood period.

In China, the safety management is based on the experience and judgment of engineers at some extent, despite the reference to the existing codes and monitoring results and a few years operating status. In 1998, BBC reported that "The decisive battle to beat the flood has begun, and danger may appear at any time, especially around the inland lakes, above the warning level. Some 3,500 local people are now defending the dikes while soldiers and armed police officers are on standby for rescue work". Figure 1.1 gives the picture of local residents have been organized to detect the potential emergency on the dike at higher water level. In Yangtze river basin, during the rainy season most warnings of an imminent failure come from the thousands of dyke inspectors living and working 24 hours per day in inspection shelters on the dykes. They observe weaknesses of the dyke and try to warn as early as possible. The same as for construction techniques holds for maintenance techniques and emergency relief. Age-old methods are used to prevent the dyke of failure. While the techniques are usually successful, the rates of productivity of the repairs are low, despite of the thousands of people working on the dyke.

Figure 1.1 Local residents have been organized to detect the potential emergency on the dike

In the past 100 years, engineers have summarized the experience to find the rational and scientific and effective method for safety design and assessment. The notion of safety factor or safety margin has been updated many times, and it achieves a rather credible grade up to the middle of 20th century. Although safety factor is an empirical coefficient, it generalizes the factors of disadvantage and advantage of effects of structural safety and their combined action. It includes the potential over loading and inhomogeneous of material strength and man-made error occurred during the design or construction or operating. So, this is the reason that safety factor was used to verify and calibrate other design methods.

Since the 1980's, the probabilistic design method based on the stochastic theory has been employed in design code and standard of China. The resistance and loading effect of structure should be taken to be random variables, then the reliability index and failure probability of structure can be calculated according to the probability density distribution curve. But the preconditions of correct and reliable distribution curves should be imposed. In fact, there are many factors of effect on resistance and loading.

In this case, it is a difficult task to transform and extend the new method. It is no doubt that the non-deterministic associated with the deterministic method will be rather scientific methods and means to assess the safety of dike. Thus, the traditional deterministic method and probabilistic method will be involved in this study.

In addition, the development of the software system for safety assessment of dike is helpful to the design, maintenance, safety management of river levee and revetment in China. It is an effective tool to transfer the professional knowledge to engineers. In recent years, some tentative research work aiming at the improvement of the safety management and assessment has been carried out.

In China, the risk analysis of overtopping and slope stability and seepage stability by the reliability method have been studied, but the computation of failure probability taking in account all relational factors are imperfect. The structure risk models of slope stability and seepage stability have been proposed by Wang Zuofu (1998). The correlation of these failure mechanisms and feasibility of generalized analysis are deserved in further study. A practical software system has been developed by using the conventional procedure and probabilistic risk method---SADSS. According to the process for flood prevention, a model and its solution method of structure risk on dike were put forward. Some computing results of whole dike sections can be real-time displayed, and this system has been applied successfully to the modern management on dike of Anqing city.

Based on these results, some design guides and handbooks of this method applying for China can be presented in the near future.

1.3 Traditional method for safety evaluation

The assessment aspects as included in the guide relate to height, stability, means of closure and limit profile. The manager begins with general monitoring on the basis of simple calculation rules and goes into greater detail if such proves necessary. On the basis of monitoring a qualification is given per flood defence element of 'good', 'satisfactory' or 'poor'. A design profile is usually laid down in the data-base corresponding to the assessment level 'good'.

In the classic deterministic approach fixed calculation values are used for the load and the strength, with a safety coefficient in between: Strength = F *Load, with F > 1.

As every parameter shows a specific spread, F does not say anything about the failure probability. A completely different probability can accompany the same 'safety coefficient' so that the load exceeds the strength. The probability that the strength is smaller than the load, in other words $Z<0$, is much smaller in the upper case than in the lower case, while the 'safety coefficient' in relation to the average is the same. This is also true if other calculation values are assumed, with a smaller probability of occurrence than the average value for the definition of the safety coefficient.

In a deterministic design the selection of F is based on experience or intuition. The aim is to prevent failure or collapse, in other words to prevent $Z<0$. An experienced designer will select a larger F as the spread of the parameter increases or the design model is less accurate, but F still says nothing about the failure probability.

1.4 Probabilistic risk analysis

The protection against flooding by flood defences is never absolute. Upper limits of natural phenomena like wind and rain are not known. Instead it must be assumed a certain exceedance-probability of these phenomena. Under extreme conditions flood defences can collapse and the land behind it will flood. Under less extreme conditions the behaviour of the flood defence cannot always be predicted, so there is always a (small) probability of collapse.

In recent years, a number of flooding events occurred in some countries in the last decade has caused an increasing interest in risk-based safety approach of flood defences. Probabilistic risk analysis is the up-to-date research field of safety assessment techniques of dike. It is well-known that the experience of the design and maintenance and assessment of embankment and revetment engineering in the Netherlands is worth to referring.

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.

The main advantages of a risk-based safety approach are: (1) It is based on the concept of risk and therefore considers all the aspects related to failure of a flood defence system: the strength and the loading conditions of the flood defence system as part of the probability of inundation as well as the consequences of inundation in case of failure of the flood defence system. (2) It supports the process of decision-making with respect to maintenance of a flood defence system as a risk-based analysis of flood defence systems points out the system's weakest links and enables the decision-maker to target maintenance activities. (3) In case of large scale flood defence improvements the decision-maker can compare different design options in terms of the actual risk reduction and the costs which are associated with the improvement option.

More and more, the interest in risk-based design and maintenance of flood defences has grown in other countries, for instance USA, Australia, England, Canada.

1.5 Relation between the traditional method and probabilistic method

In fact, the probabilistic design approach is a logical extension of the traditional method. As we known, the deterministic method has been using in the active code of designing of dikes and revetments. Probabilistic calculation techniques are more laborious and complicated than deterministic ones, but correspond better with the aim to produce sophisticated structures and have insight into the actual risks. The 'safety coefficient' used in deterministic practice actually says little about safety: the same value means something completely different depending on the mechanism.

The difference between a deterministic approach and a probabilistic one is good expressed in the determination of the necessary dike height.

The predominantly deterministic determination assumes the normative high water level (MHW) that the dike must be able to retain. This MHW includes a rise in sea level in the plan period (up to now 0.1m for 50 years) and wind effects on the local water level. In addition, the wave run-up is the most important parameter in the determination of the crest height. The wave run-up is calculated on the basis of a certain wind at MHW and the corresponding waves (bearing in mind the geometry of the foreland and the outside slope). This is expressed in the level that is exceeded by 2% of the waves or a certain overtopping discharge. The so-called crest reference height is therefore equal to this level.

Settlement of the soil body over a certain maintenance period is also taken into account to avoid a situation in which the height of the dike is lower than the crest reference level needed. That results in the so-called construction level. The difference between crest reference level and MHW is called the freeboard. At least 0.5m is usually maintained for that, also when the calculated wave run-up is lower. This is in connection with uncertainties in the determination of the MHW's among other things, and to ensure that the crest is easily passable in the case of extremely high water levels.

In a probabilistic approach, as in the inundation probability calculation, the result of the above-mentioned approach can still be used as an initial estimate. The probability that the defence will fail must subsequently be calculated. That may be due to a defence that is too low or too weak. The inundation probability is studied after integration with all failure mechanisms. The whole range of combinations of water levels and waves are included. A lower defence may then be possible than would follow from a deterministic approach, because other dike sections or mechanisms contribute less to the failure probability. The determination of the ultimate dike height is therefore much less direct than in the deterministic method. MHW plus a few decimetres is however retained for the time being for the dike height.

1.6 Structure of this report

In chapter **1** the background leading to the initiative of this project, the definition of the problem and the objective of this study is given. This chapter closes with a description of the safety assessment method in main lines. Chapter **2** provides an overview of the deterministic safety assessment method on dike. Chapter **3** contains the comparison study of the different models on piping between China and the Netherlands. Chapter **4** provides an overview of the reliability methods and risk assessment for flood defence systems, especially for Dutch method. These methods are the tools which are used to perform the probabilistic risk analysis. Chapter **5** gives an outline of safety evaluation technical standard and risk level, and decision-making after a risk analysis. Chapter **6** discussed the function and feature of the practical software system for structural safety assessment. In chapter **7**

the boundaries of the Anqing city flood defence system are defined together with the main components or defence types occurring in the system. In chapter **8** contains a description of the choices which have been made with respect to the data requirements in general and specifically connected to the separate failure modes. In chapter **9** main failure mechanisms and their probability functions and random variables concerned with this case study have been discussed. Chapter **10** the results of the calculations on Anqing river dikes are presented. Finally, in chapter **11**, the conclusions and recommendations are given.
Therefore, the whole report can be subdivided on the basis of the related contents into three parts: Traditional deterministic method (Chapter 2, 3); theory description of probabilistic risk assessment (Chapter 4, 5, 6); case study for Anqing dike ring system (Chapter 7, 8, 9, 10).

PART I Traditional Deterministic Approach

Chapter 2 Failure Mechanisms and Deterministic Safety Assessment Method

[**Abstract**] This chapter provides an overview of the deterministic safety assessment method on dike. Some failure mechanisms and modes of soil structures was described. A flow process of safety evaluation by deterministic method was given.

2.1 Failures modes of flood defences

The manner in which the water retaining capacity is lacking is called a failure mechanism. The first step in the assessment of the safety of a flood defence is the catalogue of all threats and corresponding failure and collapse mechanisms. As soon as a mechanism is known, it is attempted to make a model of it. That model can be of an experimental or mathematical nature. The model must assist the designer or manager in obtaining insight into the conditions in which the defence performs well and those in which it does not. The regularity of the conditions in which performance is unsatisfactory is then estimated. This can then be the basis of a decision on the safety level of the (existing or designed) structure.

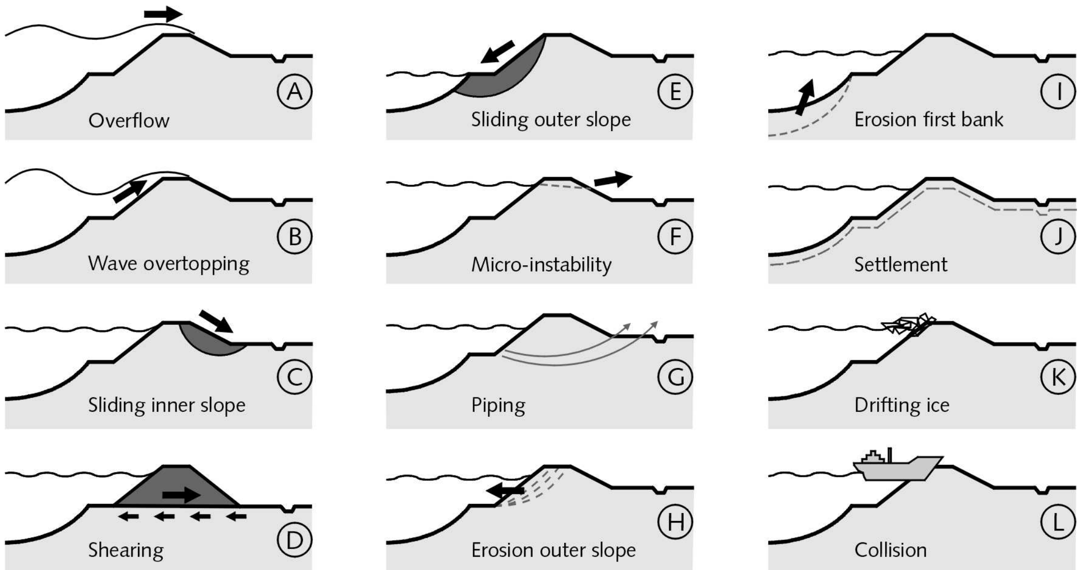


Figure 2.1 Failure mechanisms soil structures

In the assessment of the safety of dikes and dams the following failure mechanisms are important, as illustrated in figure 2.1:

(1) Inundation of the dike ring area through a combination of high water level and wave overtopping without the collapse of the defence structure (A);

(2) Erosion of the inner slope by the force of the flowing water and by a combination of high water level and wave overtopping (B);

(3) Instability (sliding) of the inner slope, due to either infiltration of the overflowing water in a combination of high water level and wave overtopping, or water pressure against the

defence and increased water pressure in the subsoil (C);
(4) Shearing of a soil body, also by water pressure against the defence and increased water pressure in the subsoil (D);
(5) Sliding of the outer slope in the case of a rapid fall in the outside water level after high water (E);
(6) Instability of the inner (or outer) slope by exiting seepage water through the soil body (micro-instability) analogous to failure mechanism C, but at lower water levels (F);
(7) Piping as a consequence of seepage flow through the subsoil so that erosion starts behind the dike and soil is borne along (sand boils) (G);
(8) Erosion of the outer slope or the toe and foreshore by current or wave movement (H, I);
(9) Large-scale distortions of the soil body (I);
(10) Mechanical threats like ice and shipping (K, L).

2.2 Deterministic method of safety assessment

Deterministic means that only a single normative combination of loads is considered, making assumptions about the strength and safety realized via non-explicitly substantiated safety coefficients, but based in part on experience and in part on intuition.
The traditional design or safety assessment 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 2.2 gives an illustration of a deterministic approach.

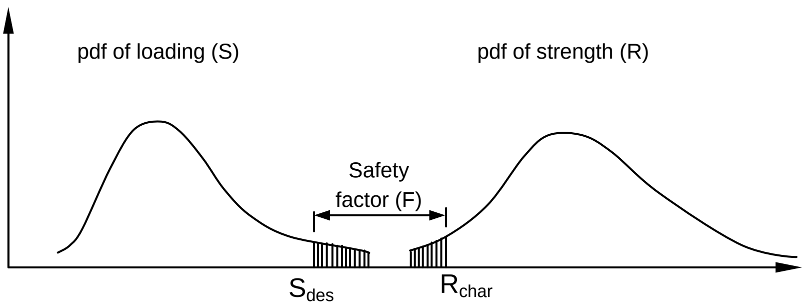


Figure 2.2 Example of a deterministic approach

2.3 Flow process of safety evaluation by deterministic method

Figure 2.3 gives the flow process chart of safety evaluation on dike by deterministic method.

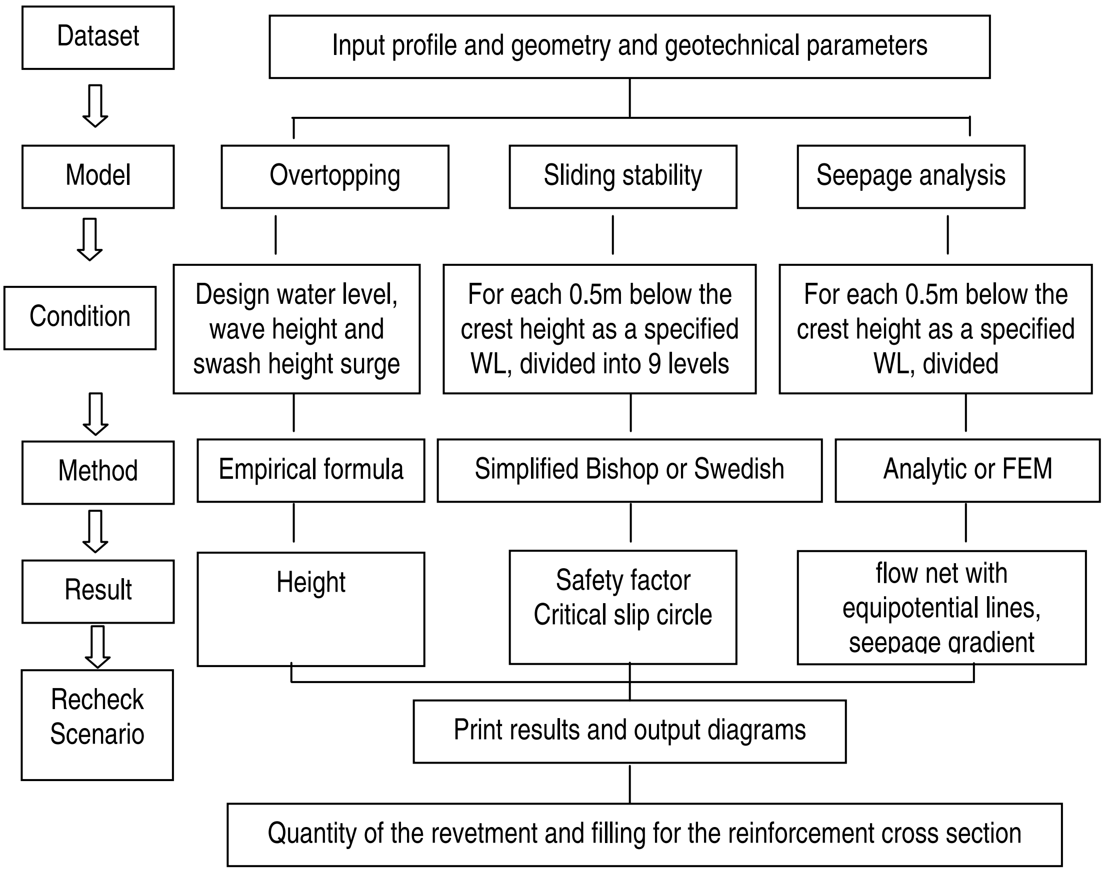


Figure 2.3 Flow process chart of safety evaluation on dike by deterministic method

Chapter 3 Comparison Study of Piping Mechanism on Dikes in China and the Netherlands

[**ABSTRACT**]: A typical dike with inclined impermeable facing is selected to investigate its safety against piping. A sensitivity analysis was performed by using the various geometrical variables and geotechnical parameters to investigate the different model's compatibility and accuracy. These parameters include thickness of clay layer and sand layer, width of foreland and cross dike crest and piping-berm, height of water level, slope ratio, permeability coefficient of sand and clay layer. A comparison of Sellmeijer's method, Chinese method and Bligh's method, respectively, shows that Sellmeijer's method requires shorter seepage length and hence, a more economical design for water retaining structures against piping. The Chinese method also can give a reasonable design within a certain range, and the safety factor by this model is not very sensitive with some variables than the one by other models. An increasing curve, safety factor increase with the permeability coefficient of sand layer, has been gained by Bligh model, which is not very unreasonable.

Seepage failure is a major failure mode of levee in China. According to the statistic, the emergencies caused by seepage failure of river dike are about 70% of the total emergency of the 1998's Flood of Yangtze River basin. Actually, where there is a hydraulic head difference, there is seepage in the dike. With the rise of water elevation during flood period, the phreatic line is formed inside the dike and its position gradually rises. At the same time, the seepage gradient in the dike and subsoil gradually increased. When the actual seepage gradient J is larger than the critical gradient of the subsoil J_{C}, seepage failure will occur. In other words, if the difference between the local water level h and the inside water level h_{b}, reduced with a part of the vertical seepage length L_{v}, exceeds the critical water level h_{p}, the embankment fails as a consequence of piping.

The purpose of the research effort leading to this report was to compare the differences of the piping models and seepage analysis methods between China and the Netherlands by sensitivity analysis of design variables and constants, and the deterministic method being commonly versed by geotechnical engineers was used in this study.

3.1 Determining of variables and constants

An idealized levee cross section with inclined clay layer for example problem was illustrated in Figure 1. It is assumed to be a two-phase fluvial facies structure, with a covering weak permeable clay stratum and an underlying strong permeable sand substratum.

Depending on the computing model used for piping, the data needed about the material composition and the configuration of the soil layer is more or less extensive. The values of some design variables and deterministic parameters are shown in Table 3.1.

Table 3.1 Variables of this study

Variable	Description	Unit	Type	Mean value
h_{bc}	Thickness of clay	m	Design variable	3.5
d_{ks}	Effective thickness of clay	m	Design variable	3.5
L_1	Width of foreland	m	Design variable	0.0
m	Slope ratio		Design variable	2.5
w	Crest width	m	Design variable	5.0
k_c	Permeability coefficients of clay	m/s	Design variable	10^{-8}
k_s	Permeability coefficients of sand	m/s	Design variable	10^{-5}
t_{sb}	Thickness of piping-berm	m	Design variable	0
L_2	Width of piping-berm	m	Design variable	0
h_{bs}	Thickness of sand	m	Design variable	2.5
h_w	Flood water level	m	Design variable	8.3
h_0	Height	m	Design variable	11.0
d_{70}	70-percentile value of the grain distribution	m	Deterministic	0.0003
v	kinematic viscosity of water at 100 Celsius	m^2/s	Deterministic	$1.33*10^{-6}$
γ_{nk}	Wet unit weight of clay	kN/m^3	Deterministic	22.0
γ_w	Unit weight of water	kN/m^3	Deterministic	10.0
γ_{sb}	Bulk gravity of piping-berm	kN/m^3	Deterministic	18.0

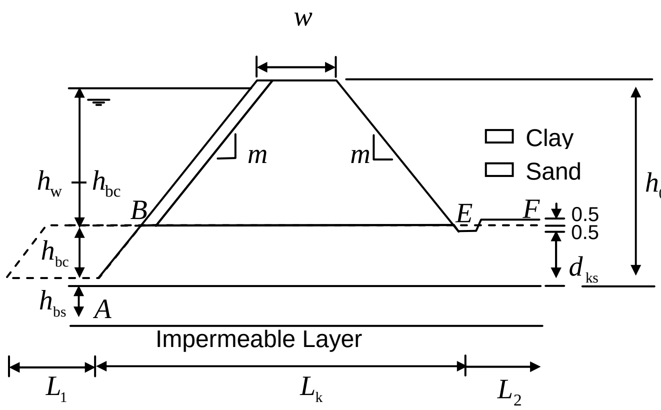


Figure 3.1 Typical dike cross section with inclined facing on two-phase fluvial facies basement

3.2 Modelling of piping mechanism on dike

First uplifting causes openings in the impervious clay layer covering the sand layer. Second, a flow of water through these openings initializes an erosion process. This process can progress from the point of the openings caused by the previous uplifting behind the embankment towards the water outside. The erosion process takes the form of pipes undermining the foundation of the embankment. These pipes can eventually cause failure

(CUR 1991).

By understanding the groundwater flow principles and the mechanism of the seepage failure, it is more important to apply this knowledge to form a design criteria and construction standard in the dike design. In the Netherlands, the criteria are mainly concerned with the relation between hydraulic head applied on the dike and the seepage length of the dike. In China, it is more emphasized on the relation between critical seepage gradient and the impermeability of the soil. Essentially, the principles of the two approaches are same.

Therefore, a comprehensive safety factor used to describable the piping is presented to develop the comparative study. For a hydraulic head over the structure (loading) which is smaller than the critical hydraulic head (strength) a fissure will originate due to erosion which will grow until the fissure length corresponding to this hydraulic head is reached. The inclined facing of above-mentioned typical section is supposed to have good impervious property. Aside from overtopping and sliding, piping (underseepage) is assumed to be the governing mode for the section studied. Under normative high water level, piping will occur at vulnerable area of clay layer because it can be penetrated by artesian head, and then a breach is developed. In this case, the following relation for safety factor can be deduced:

$$F = \Delta H_{\text{strength}} / \Delta H_{\text{loading}} \tag{3-1}$$

In which: $\Delta H_{\text{strength}}$ is critical hydraulic head, [m]; $\Delta H_{\text{loading}}$ is actual loading hydraulic head over the soil structure, [m]; F is safety factor, [-].

Some semi-experimental expressions on the critical head and residual head have been presented by different researchers. Thus, various calculation models or calculation rules on piping have been constructed, here a few typical models will be selected for this case study. The distribution of hydraulic head in sand layer is illustrated in Figure 3.2.

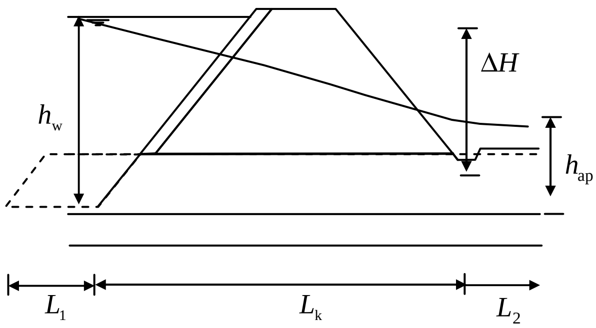


Figure 3.2 Distribution of hydraulic head in sand layer

3.2.1 Chinese model

In the illustrated dike and substrate configurations in which piping may play a role an open connection between the sand layer and the ground surface has been assumed. That connection can originate if the water pressure in the sand layer against the underside of the clay layer corresponding to a sufficiently high outside water level equals the weight of the overlying ground. Due to the vertical strength equilibrium the water pressure can never increase. It is assumed that cracks will form in the covering layer because of this, so that (sand) boils can originate. This phenomenon is called cracking or hydraulic soil failure. According to the Chinese Standard (1998) on dike design, the equation of safety factor F_{CN}

is obtained based on considerations of equilibrium necessary to initiate a sand boil:

$$F_{\mathrm{CN}} = \Delta H_{\mathrm{strength}} / \Delta H_{\mathrm{loading}} = \frac{\gamma_{\mathrm{nk}} d_{\mathrm{ks}} + \gamma_{\mathrm{sb}} t_{\mathrm{sb}}}{\gamma_{\mathrm{w}} h_{\mathrm{ap}}} \tag{3-2}$$

Where, h_{ap} is the residual head of lower bound of the weak permeable stratum which can be substituted by the following expression:

$$h_{\mathrm{ap}} = \frac{h_{\mathrm{w}}}{1 + A * L_{\mathrm{k}} + \tanh A * L_1} e^{-AL_2} \tag{3-3}$$

$$A = \sqrt{\frac{k_{\mathrm{c}}}{k_{\mathrm{s}} h_{\mathrm{bc}} h_{\mathrm{bs}}}} \tag{3-4}$$

Where, tanh() is the hyperbolic tangent function, $\tanh(x) = (e^x - e^{-x})/(e^x + e^{-x})$. A is a coefficient, and relative with the thickness and seepage coefficient of the sand and clay substratum. L_2 is the distance to point E illustrated in Figure 3.1. L_{k} is effective seepage path length, its value can be equal to the expressions: $L_{\mathrm{k}} = 2 * m * h + w + m * h_{\mathrm{bc}}$. According to the Chinese guideline, the safety factor F_{CN} should be larger than 2.0, if not piping will occur. The model are very similar to the calculation model for cracking of the Netherlands (TAW 1999b), especially in theory.

3.2.2 The Netherlands models

The calculation rules of Bligh and Sellmeijer (Sellmeijer 1998) are used in the Netherlands to test piping. In these roles only horizontal seepage is taken into account. In addition to horizontal seepage, the method of Lane takes the vertical seepage water into account. In recent years, when calculating the hydraulic head for river dikes in the west of the Netherlands, it has become common practice to take into account the reduction in loading resulting from resistance in vertical channels from the sand layer to ground level or base of the ditch. In laboratory tests, the potential hydraulic head has been measured for a column of sand that started to fluidize as a result of vertical seepage. Various tests indicate that the average potential hydraulic head over the fluidized column of sand is 0.6 times the height of the column. In a prototype situation in which the eroded sand from the sand layer is transported to ground level via vertical channels, on average, this is the hydraulic head that will occur. For the piping mechanism, the potential hydraulic head between the outer water and the exit point in the sand layer is normative. This hydraulic head is equal to the total hydraulic head over the soil structure, minus the hydraulic gradient over the vertical channel in the covering layer. If this phenomenon is taken into account along with the introduction of a safety factor 2 and the testing rule is then (TAW 1999a):

$$\Delta H_{\mathrm{loading}} = \Delta H - 0.3 d_{\mathrm{ks}} \tag{3-5}$$

$$F = \Delta H_{\mathrm{strength}} / (\Delta H - 0.3 d_{\mathrm{ks}}) \tag{3-6}$$

3.2.2.1 Bligh

Bligh (1912) listed and analyzed a number of small dams that collapsed through piping. According to Bligh, the critical hydraulic head is a function of a material constant C_{creep} (see Table 3.1, it includes all uncertainties and depends on the type of sand. C_{creep} varies from 12 for coarse sand to 18 for the more dangerous fine sands.) and the ‘line of creep’ (also called the continuous line) from entry point to exit point (L).

$$\Delta H_{\text{strength}} = \frac{L}{C_{\text{creep}}} \tag{3-7}$$

Here, the seepage L has been divided into three parts: width of foreland, effective seepage path and width of piping-berm. Where $L = L_1 + L_k + L_2$.

Substituting equation (3-7) into equation (3-6) then the following can be obtained:

$$F_{\text{BL}} = \frac{L}{C_{\text{creep}}\left(\Delta H - 0.3 d_{\text{ks}}\right)} \tag{3-8}$$

The Dutch guidelines prescribe Bligh. Assuming that C_{creep} =15 and $\Delta H_{\text{loading}}$ =5.00m, the required seepage length amounts some 75 meters. As long as a covering layer with sufficient entrance resistance is present in front of the dike, problems are usually limited. Wherever this is not present ($L_1 = 0$ in Figure 3.1) the demand for space behind the dike even doubles the requirements in case of 5$\Delta H_{\text{loading}}$-berms. The awareness that Bligh's rule may be too conservative and social pressure to limit dike dimensions, led to research into the piping phenomenon (Tonneijck 1992).

3.2.2.2 Lane

One or more vertical seepage screens are sometimes used to extend the seepage line for hydraulic structures or soil structures located on a sandy substratum. In structures with seepage screens to prevent under and rear seepage in particular, the normative seepage lines can be very complex. Naturally, seepage water follows the way of least resistance. In a technical report on sand bearing springs (TAW 1999a) an approach is given in which these types of seepage routes are systematically investigated. Lane (Lane 1919) has set down a summary of collapsed structures of which the seepage line consisted in part of vertical sections in the following rule:

$$\Delta H_{\text{strength}} = \frac{\frac{1}{3} L_{\text{h}} + L_{\text{v}}}{C_{\text{w,creep}}} \tag{3-9}$$

In this equation, L_{h} is the total length of the horizontal sections of the seepage line and L_{v} is the total length of the vertical parts of the seepage line. The material constant $C_{\text{w,creep}}$ is based on an upper limit that implies that when Lane's rule is used: $F = 1$. Table 3.2 indicates the values given by Lane for various types of material in the soil layer. The expression of safety factor on piping is deduced as following:

$$F_{\text{LA}} = \frac{\frac{1}{3} L_{\text{h}} + L_{\text{v}}}{C_{\text{w,creep}}\left(\Delta H - 0.3 d_{\text{ks}}\right)} \tag{3-10}$$

3.2.2.3 Sellmeijer

Dutch TAW researcher Sellmeijer (Sellmeijer 1998) developed a calculation model to test for piping. It is valid for an idealized soil composition, or in other words for situations in which the water-bearing sand layer with homogeneous permeability and constant thickness extends far beyond the exit point. In reality, however, soil compositions are frequently encountered, in which the sand is anything but homogeneous. Nevertheless the model can still be used on condition that the product of layer thickness and permeability in the model is so chosen that the seepage current is simulated in the prototype situation.

The above mentioned calculation model was used to perform various numeric calculations, in which the hydraulic head and soil parameters differed. The results from these calculations were then used to calibrate an analytical calculation model. Sellmeijer's calculation model was validated on the basis of a large-scale model test in the Delta channel of the Waterloopkundig Laboratory. On the basis of the model tests in the piping guideline the calculation model of Sellmeijer was validated (Silvis 1991). The nominal values were determined for roll-resistance angle (θ), and slip force (η) these being θ=41 degrees and η=0.25 respectively. There is no simple way to determine these parameters for different cases (expensive laboratory studies). With these parameters, however, the model error in the original model of Sellmeijer can be corrected. If these nominal values are used, the safety factor according to Sellmeijer's original equation gives:

$$F_{\mathrm{SE}} = \frac{\Delta H_{\mathrm{strength}}}{(\Delta H - 0.3 d_{\mathrm{ks}})} = \frac{1}{(\Delta H - 0.3 d_{\mathrm{ks}})} h_{\mathrm{bs}}{}^{\beta} L^{1-\beta} \left(\frac{g d_{70}^{3}}{\nu k_{\mathrm{s}} L} \right)^{\frac{1}{3}} \left(0.25 - 0.037 \ln \left(\frac{g d_{70}^{3}}{\nu k_{\mathrm{s}} L} \right)^{\frac{1}{3}} \right) \tag{3-11}$$

Here:

$$\beta = \frac{0.28}{\left(\frac{h_{\mathrm{bs}}}{L} \right)^{2.8} - 1}$$

In which: g is gravitational acceleration [m/s^2]. Considering the inhomogeneity of the soil parameters, some representative values of parameters in Sellmeijer formula have been given, as listed in Table 3.3.

Table 3.2 Creep factors for the rules of Bligh and Lane

Type of soil	Median Grain diameter	C_{creep} (Bligh)	$C_{\mathrm{w,creep}}$ (Lane)
	µm	(F = 1)	(F = 1)
Extremely fine sand, silt	<105		8,,5
Very fine sand	105-150	18	
Very fine sand (mica)		18	7
Moderately fine sand (quartz)	150-210	15	7
Moderately coarse sand	210-300		6
Very/extremely coarse sand	300-2000	12	5
Fine gravel	2000-5600	9	4
Moderately coarse gravel	5600-16000		3.5
Very coarse gravel	>16 000	4	3

Table 3.3 Representative choice of parameters in Sellmeijer formula

Parameter	Description	Type of representative value	Comments
h_{bc}	Layer thickness covering layer	l.r.v.	V_{c} =0.10
d_{70}	70-percentile sand	l.r.v.	V_{c} =0.25
h_{bs}	Thickness sand layer	h.r.v.	V_{c} =0.10
k_{s}	Permeability of sand	h.r.v.	(see text)
L	Length seepage line	l.r.v.	V_{c} =0.10

Representative or characteristic values:

h.r.v. $\mu(1-\mathrm{t}_{N-10.95}V_{\mathrm{c}})$ high representative value or 95% upper limit);

l.r.v. $\mu(1+\mathrm{t}_{N-10.95}V_{\mathrm{c}})$ low representative value or 95% lower limit);

μ average value from sample, or best estimate;

V_{c} variation coefficient from sample or default variation coefficient; see table;

$\mathrm{t}_{N-10.95}$ student t-factor (if no sample is available: 1.65);

Example of the determination of k if no random sample is available; $k_{\text{calculation}} = k_{\text{average}} * (1+1.65*0.1) = 1.16k_{\text{average}}$.

Indicative values for the permeability of sand containing negligible amounts of silt have been listed in Table 3.4.

Table 3.4 Relation between grain size and permeability of sand (TAW 2001)

Grain size d_{50}	Permeability k_{s}
mm	m/s (*10^{-3})
0.1	0.06
0.15	0.14
0.2	0.24
0.3	0.54
0.4	1.00

3.3 Sensitivity analysis

The sensitivity analysis is conducted for a number of cases, making use of the special fragment types, namely the leak fragments and the settlement fragment and the seepage fragment.

The occurrence of piping under normative conditions is determined by: (1)The length of the seepage flow;(2)The thickness, permeability and erosion sensitivity of the water-bearing layer; (3) The presence and thickness of a weakly permeable top layer; (4) The difference in potential over the length of the seepage flow.

It can be shown from formula above-mentioned that safety factor depends on flood hydraulic head, property and density of fill material of dike body, structure and property of soil layer of dike foundation, width of outer berm, piping-berm behind dike, etc. Furthermore, a sensibility analysis of these parameters is performed as following by using different models. In sensibility analysis only one of the parameters varies, others remain values of Table 3.1.

Before this, because of the explicit expression between the grain size d_{50} and C_{creep} is vacant. Via accurate curve fitting to these experience values an approximate analytical formula is derived.

When the relation between grain size d_{50} and permeability has been given by Table 3.4, we can obtain the following equation by polynomial fitting:

$$k_{\mathrm{s}} = 3.28\mathrm{E}^{-5} - 0.00037d_{50} + 0.0069{d_{50}}^{2} \quad (3\text{-}12)$$

Refer to Figure 3.2A.

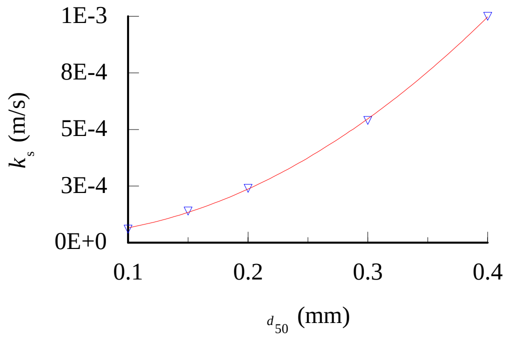
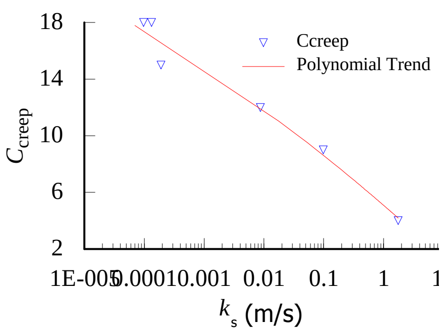


Figure 3.2A Relation of seepage coefficient with grain size d_{50}

Then, a polynomial approximation of C_{creep} with seepage coefficient k_s is represented by the following equation:

$$C_{creep} = 5.09 - 3.65\log k_s - 0.14\log k_s{}^2 \tag{3-13}$$

Refer to Figure 3.2B.



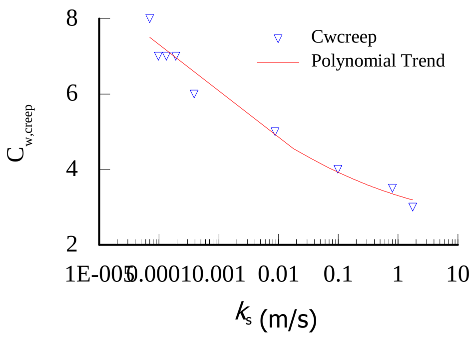


Figure 3.2B Relation of C_{creep} with k_s Figure 3.2C Relation of $C_{w,creep}$ with k_s

The same way, we can obtain the following equation for $C_{w,creep}$ with k_s:

$$C_{w,creep} = 3.3 - 0.49\log k_s + 0.12\log k_s{}^2 \tag{3-14}$$

Thus, the relation between material constant C_{creep} and grain size has been explicitly expressed. It will be convenient to develop the sensitivity analysis of parameters.

It should be also pointed out that when using different empirical calculation models to analyze the same question, only a qualitative indication can be presented, since the common starting point is quite difficult to define.

In addition, estimates of the permeability on the basis of grain distribution analyses can also be gained by the formula (Rooijen 1992).

$$k_s = (c_0 - 1.83 \cdot 10^3 \cdot \ln(U))d_{10}^2 \tag{3-15}$$

In which c_0 is dependent on the packing of the sand: 1) loose packing, c_0 = 1.5 10^4; 2) Natural packing, c_0 =1.2*10^4; 3) Firm packing, c_0 =1.2*10^4. The packing is dependent on porosity and the uniformity coefficient. The uniformity coefficient $U = d_{60} / d_{10}$.

For the calculation of the characteristic value of the permeability, in accordance with formula (3-15), we need (high) characteristic values for d_{10} and for U. In addition, for piping analyses with the rule of Sellmeijer, a low characteristic value for d_{70} must be used. The independent application of the characteristic value procedures on the results of a sieve analysis can lead to a situation in which the calculated (low) characteristic value for d_{70} is lower than the calculated (high) characteristic value for d_{10}. In fact d_{70} and d_{10} are not independent; this should be expressed in the estimation procedures for characteristic value determination. In the report (Calle 1994) it is therefore proposed that the characteristic value for d_{10} be determined using the (low) characteristic value of d_{70} and the (high) characteristic value of U in accordance with the formula:

$$d_{10,kar} = \alpha' \frac{d_{70,kar}}{U_{kar}} \tag{3-16}$$

In which $\alpha' = 0.9$ is a corrective value as U is based on d_{60}.

3.3.1 Thickness of clay layer

The relation between thickness of clay layer and safety factor of seepage stability F is shown in Figure 3.3. It indicates that thickness of clay layer has comparatively influence on seepage stability of dike foundation. In Bligh and Sellmeijer models, F has a similar increasing tendency with the increase of the thickness. The essential reasons results from the expression on these models. In Chinese model, the denominator of equation (3-2) will change very small when the thickness of clay layer increases, so the safety factor almost goes up linearly. Herein, it should be pointed out that the thickness of clay layer will be increased downwards along the vertical direction but the profile of the dike body keeps constant.

3.3.2 Width of foreland

The foreland has a reducing effect on the potential in the sand at the site of a potential crack location. Figure 3.4 shows F increases with the width of foreland, and it appears to be a linear proportion for each model. The reason lies that the increase of width of seepage path in the foundation intensifies the capability of impermeability of soil. It is an effective method to control seepage for the structure to adopt natural covering when there is enough foreland at outer side in the practical engineering. It also can be observed form Figure 3.3 and Figure 3.4 that the computing results are very approach for Bligh and Chinese model.

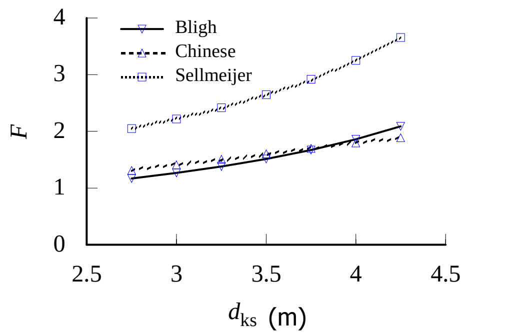


Figure 3.3 Thickness of clay stratum

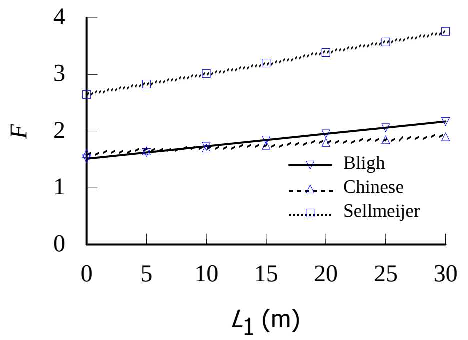


Figure 3.4 Width of foreland

3.3.3 Slope ratio

The differences between the three models are shown in Figure 3.5, safety factor F increases linearly with slope ratio of the dike. The reason is that the inner side and outer side slope ratio are taken as the same value. If the slope ratio is enlarged, the length of seepage path will increase evidently and obtain a lower loading hydraulic head. In addition, the slope coefficient of the relation curve between safety factor and slope ratio will be descended by the following order, Sellmeijer, Bligh, and Chinese model. When the slope ratio is larger than 4, the safety factor by Chinese model and Bligh is more or less smaller.

3.3.4 Width of the dike crest

It is shown in Figure 3.6 that safety factor increases linearly with the width of the dike crest. It is obvious that the increase of slope ratio and width cross dike crest can change the length of seepage path, but it will result in higher construction cost. It is an interesting problem to be further studied to balance the relation of construction costs and the relative benefits with the introduction of conception of economic assessment. But the safety factor obtained by Bligh model and Chinese model is lower than those by Sellmeijer model. It will provide adequate reliability against uncontrolled movement of materials.

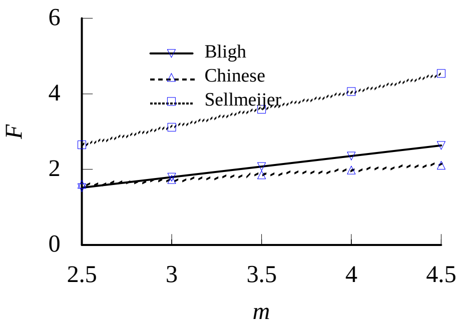


Figure 3.5 Slope ratio

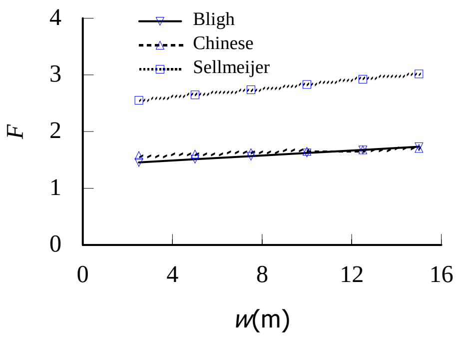


Figure 3.6 Width across the crest

3.3.5 Permeability coefficient of clay layer

The permeability coefficient of soil can reflect mechanical composition, structure, tightness and particle size. The increase of F with the increase of permeability coefficient of clay layer is shown in Figure 3.7 according to Chinese model. If the terms keeping constant are omitted or combined, a simplified expression of safety factor with k_c can be gained, $F_{CN} = C_1 + C_2 * \sqrt{k_c}$. In practical engineering, this conclusion is inexplicable at a certain extent. The safety factors are constants by Bligh and Sellmeijer model because the permeability coefficient of clay is not involved in the expressions.

3.3.6 Permeability coefficient of sand layer

It can be found from Figure 3.8 that F decreases sharply with the increase of permeability coefficient of sand layer according to Chinese model and Sellmeijer model. It is interested that the Bligh model gains an increasing curve, which is a little bit unreasonable. The possible reason is that a bigger seepage coefficient of sand will obtain a lower value of C_{creep}. Besides, the change in seepage coefficient of sand layer will result in final variation of seepage path length, but the expression of Bligh does not take into account. A conclusion can be drawn that embankment filling materials should be chosen carefully and compaction quality should be controlled strictly.

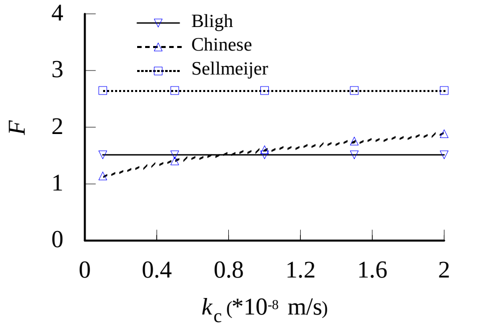


Figure 3.7 Permeability coefficient of clay

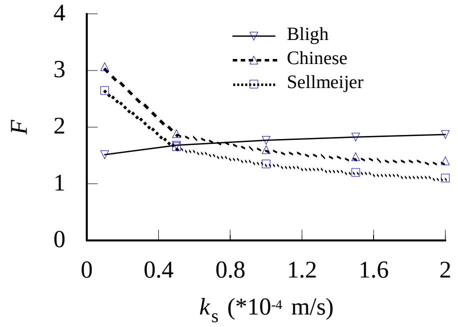


Figure 3.8 Permeability coefficient of sand

3.3.7 Thickness of piping-berm

When there is natural covering at inner side of a levee, the thickness of piping-berm can make the residual hydraulic head at the end of the piping-berm less than the allowable value, otherwise, it will result in a potential seepage failure or uplift. Figure 3.9 shows that F increases with the thickness of piping-berm. Therefore, it is feasible to adopt a solution by using piping-berm in order to decrease width of clay layer of substratum. The safety factor obtained by Bligh model and Chinese model is lower than those by the other, so the piping formula of Bligh and Chinese model is a conservative estimate for the potential failure mode.

3.3.8 Width of piping-berm

The relation of width of piping-berm and F is shown in Figure 3.10. It can be found that F increase with the width of piping-berm. For the Netherlands model, the relation of safety factor with width possesses in a linear proportional manner, with a mathematic function of $F = C_1 \cdot L_2 + C_2$. For Chinese model, the relation is an exponent function, $F_{\text{CN}} = C_1 e^{C_2 L_2}$. In fact, the increase of width of piping-berm prolongs the effective seepage path length of natural covering. The width of piping-berm should be chosen according to geological and topographical conditions and the importance of dike section.

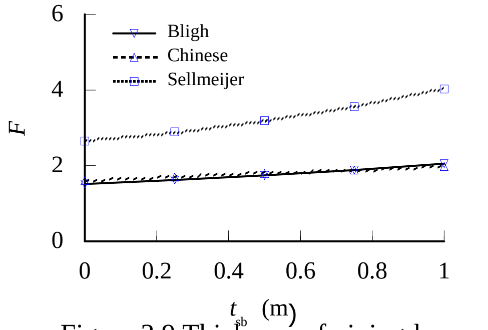


Figure 3.9 Thickness of piping-berm

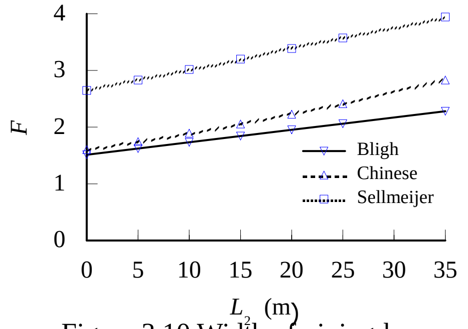


Figure 3.10 Width of piping-berm

3.3.9 Thickness of sand substratum

It can be observed from Figure 3.11 that the thickness of sand substratum has little influence on the safety factor obtained by Bligh model. However the thickness of the acquifer is taken into account for the Sellmeijer's formula, thin layers somehow show extra resistance to piping. The thickness of sand layer is also considered by Chinese model, and it has an evident influence on computing results. It appears that the safety factor is lower when the thickness is larger than 7.5m, thus some modifications should be made in the future. The most important we should point out is the thickness of the sand substratum should be added 20 meters in Sellmeijer's method. Otherwise, safety factor will be too low to meet the demand of the limit value. It seems that a similar decreasing tendency is obtained by Chinese model and Sellmeijer model.

3.3.10 Height of water level

It can be shown from Figure 3.12 that a similar decreasing tendency between safety factor and water level is achieved according to Bligh and Sellmeijer model. The relation curve obtained by Chinese model is smooth.

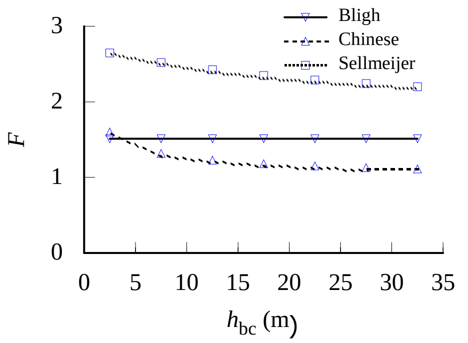


Figure 3.11 Thickness of sand substratum

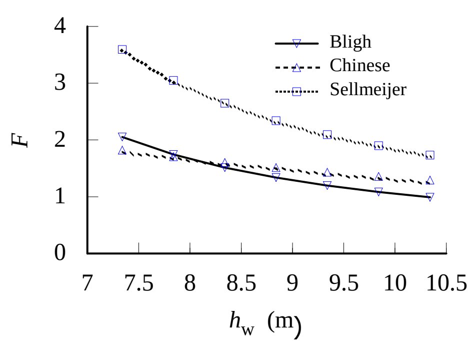


Figure 3.12 Height of water level

Supposed that a certain difference in safety factor gained by different variables is given, the relative influence of these variables on safety factors can be calculated, sensitivity index=($\Delta F/F$)/($\Delta V/V$), V standing for the variables. As illustrated in Figure 3.13, the variables that contribute most to the safety factor are the water level, thickness of clay layer, and the slope ratio. The values are listed in Table 3.5 with different models.

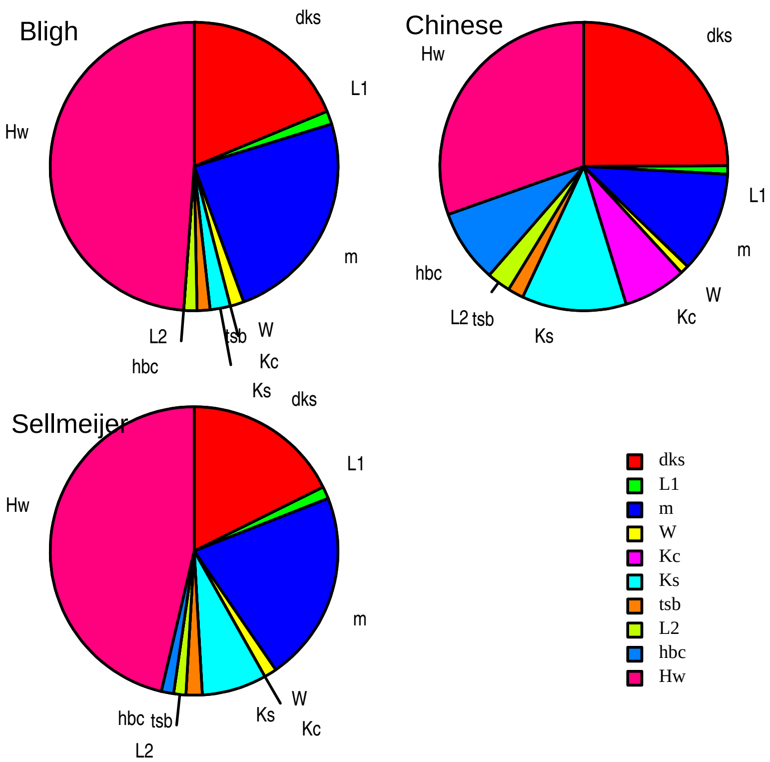


Figure 3.13 Relative influence of the variables on safety factors for different models

Table 3.5 Relative influence of the variables on safety factor with different models

Variables	Bligh	Chinese	Sellmeijer
d_{ks}	0.188	0.249	0.177
L_1	0.015	0.009	0.0136
m	0.242	0.113	0.213
W	0.015	0.01	0.014
k_c	0	0.071	0
k_s	0.023	-0.118	-0.073
t_{sb}	0.0145	0.018	0.018
L_2	0.015	0.027	0.014
h_{bc}	0	-0.082	-0.014
h_w	-0.488	-0.304	-0.463

Based on the above results, we can conclude that seepage stability can be satisfied with the typical values in Table 3.1 for this dike section from this study. The influence of these factors on seepage stability should be considered completely, and reasonable selections of parameters should be made prudently. For example, when the thickness of clay layer is not big enough or the clayey can not be obtained in site, some measures such as widen crest and slope ratio can be taken.

3.4 Conclusions and suggestions

Some of the important points that can be concluded from the investigation into the comparison are:

1) For a long time, the majority of the dikes and dams are designed by applying the rules of Bligh and Lane. These rules contain empirical coefficients which do only partly reflect the relevant soil characteristic they are based on the soil type only. They are therefore less accurate and should be used with great care. By combining the classical theories such as ground water flow, limit equilibrium, hydraulics and particle mechanics, the formulas developed by Sellmeijer's model and Chinese model are specified in terms of soil parameters rather than an empirical soil classification, these models are a better description of the phenomenon at some extent.

2) By comparing the above methods that applied in the case study, it can be observed that: Bligh method can always produce a safe dike section. But it overestimates the width needed. So the related cost for construction and improvement of the dike might be too high. Sellmeijer's method takes into account much more properties of the subsoil and the thickness of the aquifer. In the meantime in various situations these new design rules have been shown to result in more favorable dimensioning of the horizontal seepage length needed, it is shorter piping-berm than Bligh's classical calculation rule. The comparison of the Sellmeijer's method with the empirical methods of Bligh showed that the Sellmeijer's method requires a shorter seepage length and produces a more economical design for water retaining structures.

3) Chinese method also can give a reasonable design within a certain range, and the safety factor by this model is not more sensitive with some variables than the one by other models. In some cases, a few very similar results have been obtained by Chinese model and Bligh model. In technical report (TAW 1999a), there is description of various stages in the creation of piping, i.e., 1) uplift of the covering layer on the inside of the dike; 2) cracking of the covering layer and the creation of boils; 3) erosion of the sand layer; 4) the creation of through pipes; 5) collapse of the flood defence. Chinese model simulates initial stage of the cracking in the covering layer occurred due to uplift, some conservative values can be accepted. It should be kept in mind that if cracking could be prevent from happening, there would no piping. Besides, the increasing relation of safety factor with k_s seems to be unreasonable for Bligh's model.

4) A vertical part in the seepage line at the outflow should be included in the calculation. However this only applies if it concerns a vertical part in the seepage line through a clay layer. Heave occurs if there is a vertical outflow in sand, Sellmeijer's method and Chinese model do not apply in this case.

4) As a result we may state that the risk caused by the piping phenomenon is accessible for calculations. We can further optimize the constructions, minimize the costs a bit further and make faster progress with the construction work because the dikes are smaller.

The following suggestions should be further investigated:

1) In the Netherlands, some probabilistic methods have been integrated far into design part in the seepage line through a clay layer.

2) A sensitivity analysis should be undertaken using collected prototype and laboratory data to investigate the model's compatibility and accuracy on some selected dams or dikes.

PART II Theory Description of Probabilistic Risk Analysis

Chapter 4 Probabilistic Analysis for Water Defence Systems

[**Abstract**] A brief introduction on the reliability theory has been given, such as the limit state equations for different failure mechanisms, their solving approach, and fault tree. Two typical dike sections have been used to an individual case study, and the sensitivity analysis of design variables has been discussed.

4.1 Introduction

Risk is a function of the probabilities of undesired events and their consequences. A part of a risk analysis is the qualitative and quantitative analysis of the undesired events in a system. This part is called a reliability analysis.

In this report, structure risk and probability of flooding and flooding probability have the same meaning, which are equal to the probability of failure multiply the frequency of the specified water level.

4.2 History of probabilistic design of flood defences in the Netherlands

In order to judge whether a technical system (in this case a flood defence system) satisfies the requirements that society applies for with regard to safety and economy, a risk analysis has proven to be very useful. This approach is already used for other technical systems like airports, nuclear power plants and transport of hazardous goods. The term risk in a risk analysis is often considered as the multiplication of the probability of an undesirable event with the consequences of such an event (economic loss, casualties). In formula form: risk = probability x consequence[i].

The first step towards probabilistic design of flood defences in the Netherlands was made by the Delta Commission, which was established in 1953 after the historical flood of the Dutch Delta area. The commission applied statistical techniques to determine the design water levels of various locations.

Based on an econometric optimization the required safety of a flood defence was expressed in terms of a design water level, which had to be withstood. This was first performed for Central Holland. On the one hand the damage of a flooding in Central Holland was estimated and on the other the costs to upgrade the water defence system were calculated. This resulted in an optimum design water level. The probability of exceedance of the extreme water level to be withstood for Central Holland was set at 1/10000 per year. For economically less important regions, the design water level is based on the acceptance of a higher frequency of flooding. Today, the low lying areas of the Netherlands are divided in 53 dyke-ring areas, each with a specified design return period (figure 4.1). From January 1st of 1996 these safety levels have been laid down by a new law, the Flood Defence Act. The lowest design water levels are associated with a frequency of 1/1250 per year for river dykes. The Delta Commission stated however that the frequency of exceedance of the design water level must not be conceived as a frequency of failure, because there are more failure mechanisms that can lead to flooding than overtopping only.

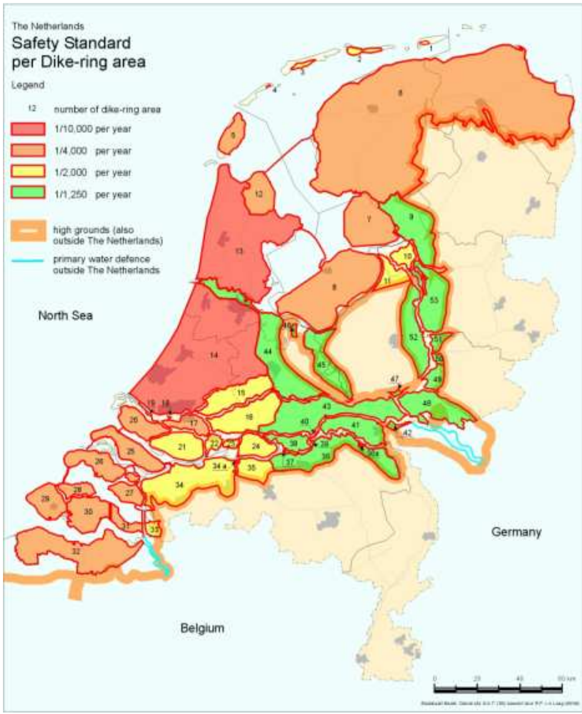


Figure 4.1 Safety standard per dyke-ring area of the Netherlands

In practice the probability of failure is kept sufficiently low by introducing deterministic additional criteria for stability.

Since approximately 1980 studies have been conducted for a complete probabilistic approach to water defences. This new approach puts the event “flooding” in the centre of the analysis. The contribution of all elements of the system and of all failure mechanisms to the probability of flooding is calculated and clearly presented. This method has been tested on four dyke-ring areas in Holland. The results showed that the new approach is a powerful tool to detect weak spots in the water defence system.

4.3 Risk analysis of a flood defence system

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.

The main steps of this approach are as followings:

(1) The first step in the reliability analysis is the definition of the system. This step points out the relevant defence length for the calculation of the probability of inundation and the area which suffers consequences in case the flood defence system fails.

(2) The second step is to define the system's components, or the defence types that occur in the flood defence system. Part of the second step is to determine the different failure modes which can cause failure of the components.

(3) The third step is to select for each failure mode the cross sections in the flood defence system that are regarded as the weakest compartments. These cross sections represent the flood defence system and contribute most to the total system's probability of failure.

(4) After this last step the flood defence system has been translated from reality into a model, the model has been expressed into data and the data can be used to calculate the probability of failure with some programs or softwares. These results point out the weakest links in the system and which random variables contribute most to the variance of the total probability of failure. (add determine consequeses of failure; determine of risk level) relate failure to the consequences of flooding).

(5) Based on the results the evaluation can be made whether reliability methods for flood defence are appropriate to serve as a detailed level methodology in the risk-based assessment of flood defence systems.

The water defence system of an area can consist of different subsystems like dykes, dunes hydraulic structures or higher grounds. Failure of any of these subsystems leads to flooding of the area. Each subsystem can be divided into elements. A dyke for instance can be divided in dyke sections. Failure of any of the sections leads to failure of the subsystem and consequently to flooding of the area. This is shown in Figure 4.2, presented as a fault tree. All possible failure modes for an element can cause failure of the system. The most important failure modes for a dyke section are illustrated in Figure 4.3. The way to present these failure mechanisms in a fault tree is given in Figure 4.4.

The system can be considered as a series system, because any failure mechanism of any element of any subsystem of the flood defence can lead to inundation of the area.

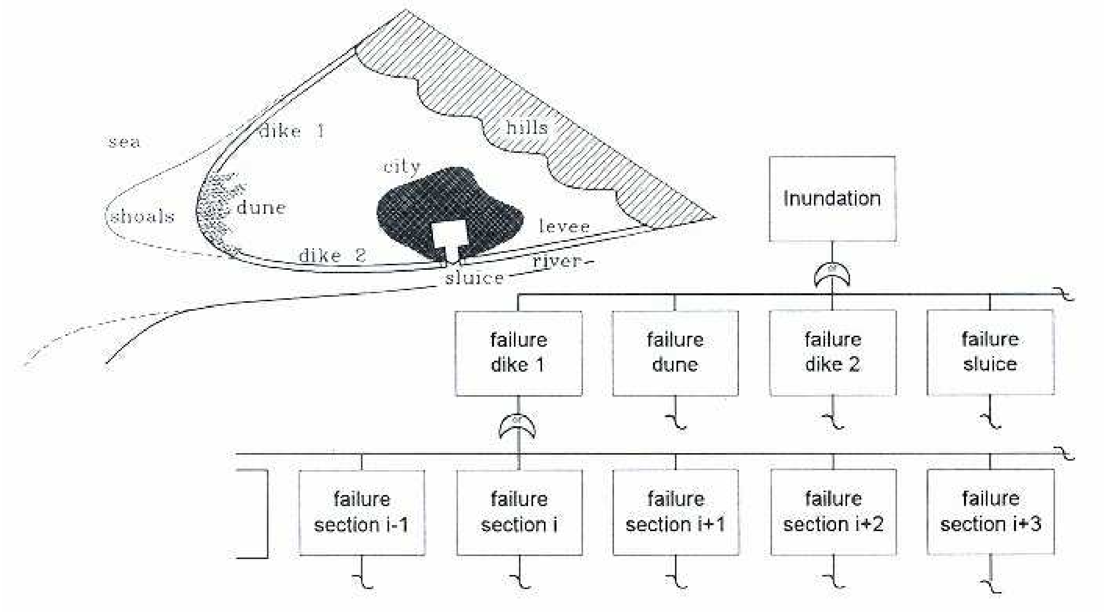


Figure 4.2 Risk analysis of a flood defence system

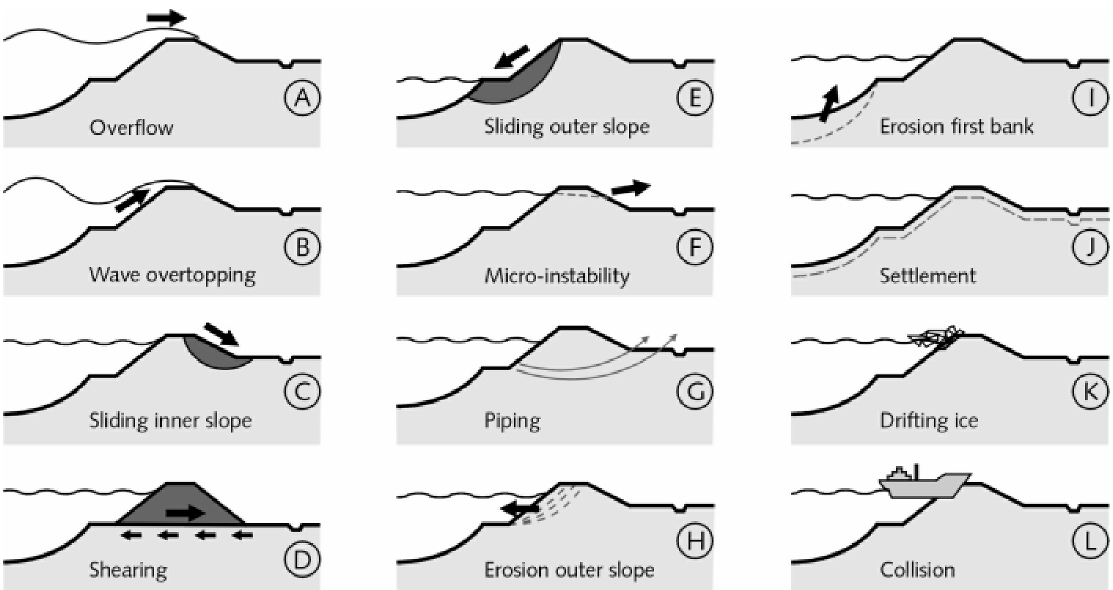


Figure 4.3 Failure mechanisms for a dyke section

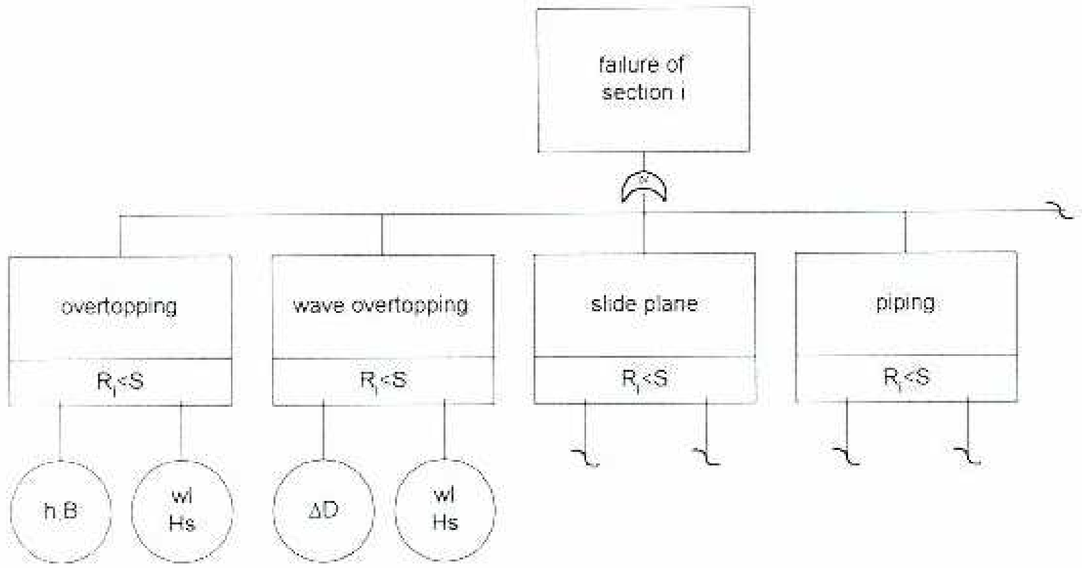


Figure 4.4 Fault tree for a dyke section

4.4 Probabilistic computations of failure mechanisms

4.4.1 Definition of failure probability

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 , as shown in Figure 4.5. The reliability function can then be written as: $Z = R - S$. The probability of failure can thus be represented symbolically as $P\{Z < 0\}$.

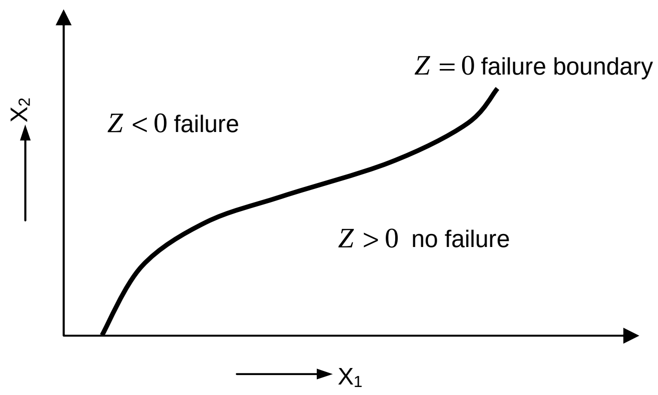


Figure 4.5 Definition of a failure boundary $Z = 0$

When a phenomenon is described by more stochastic variables the probability density function is more dimensional. This can be described in a joint probability density function, which is the multiplication of the probability density functions of the two variables. In case of the strength and load the joint probability function is given by (the load and the strength are considered independent):

$$f_{RS}(r,s) = f_R(r) \cdot f_S(s) \tag{4-1}$$

In the area $Z < 0$ the element will fail. The failure probability can be estimated by summation of the probability density of all the combinations of strength and load in this area.

$$P_\mathrm{f} = \iint_{Z<0} f_{RS}(r,s)\mathrm{d}r\mathrm{d}s \tag{4-2}$$

The general reliability problem is schematized in Figure 4.6.

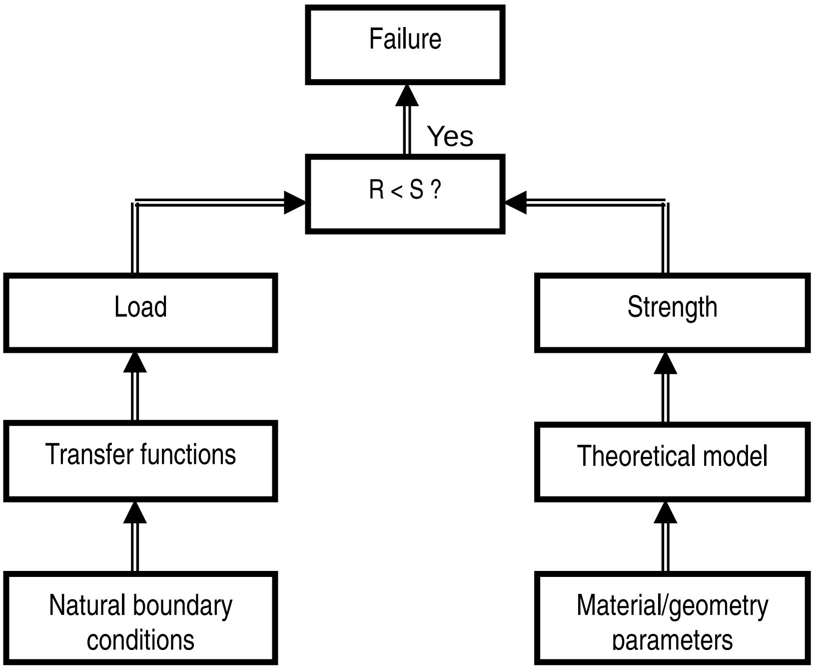


Figure 4.6 General reliability problem

In most cases the reliability function will consist of various variables. For instance the load can contain parameters for the water level and the wave run-up, while the strength contains parameters for the crest height, slope and material, the angle of internal friction, etc. The variables with a stochastic character are usually called the basic variables. Formula (4-2) can now be denoted by:

$$P_\mathrm{f} = \iint_{Z<0} f_{RS}(r,s)\mathrm{d}r\mathrm{d}s = \iint_{Z<0}.....\iint f_{r_1,r_2,..r_n,s_1,s_2..s_n}(r_1, r_2,..r_n, s_1, s_2,..s_n)\mathrm{d}r_1\mathrm{d}r_2...\mathrm{d}r_n\mathrm{d}s_1\mathrm{d}s_2...\mathrm{d}s_n \tag{4-3}$$

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 non-linear character of the reliability function are taken into accounted.

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.

The probability analysis can be implemented using various methods:

- Numeric integration
- Monte Carlo simulation
- FORM (First Order Reliability Method)
- SORM (Second Order Reliability Method)

The first two are level III probabilistic methods, the latter two are level II methods. In this categorization, level I is the design values method, but more on that later.

4.4.2 Level II method

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:

$$\mu(Z) = \mu(R) - \mu(S) \tag{4-4}$$

$$\sigma^2(Z) = \sigma^2(R) - \sigma^2(S) \tag{4-5}$$

The probability of failure of a structure follows from (Figure 4.7):

$$P\{Z < 0\} = \int_{-\infty}^{0} f_Z(Z)\mathrm{dz} = \Phi_N(-\beta) \tag{4-6}$$

$$\beta = \frac{\mu(Z)}{\sigma(Z)} \tag{4-7}$$

Where: $f_Z(Z)$ = probability density function of Z ; $\Phi_N(-\beta)$ = distribution function of the standard normal distribution; β = reliability index.

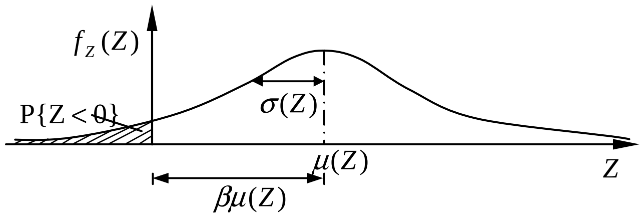


Figure 4.7 Probability density function of $Z = R - S$, showing the reliability index β

For the general case, Z is an arbitrary function of n stochastic variables $X_1, X_2, \cdots, X_n$. 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:

$$Z = Z^0 + \sum_{i=1}^{n} \left(X_i - X_i^0\right) \left[\frac{\partial Z}{\partial X_i}\right]^0 \tag{4-8}$$

Where: Z^0 = function value of Z at the point $\mathrm{X_i = X_i^0}$. $\left[\dfrac{\partial Z}{\partial X_i}\right]^0$ = partial derivative with respect to X_i, evaluated at the point $\mathrm{X_i = X_i^0}$.

The mean value and the standard deviation of Z are:

$$\mu(Z) = Z^0 + \sum_{i=1}^{n} \left\{\mu(X_i) - X_i^0\right\} \left[\frac{\partial Z}{\partial X_i}\right]^0 \tag{4-9}$$

$$\sigma^2(Z) = \sum_{i=1}^{n} \left\{\sigma(X_i)\left[\frac{\partial Z}{\partial X_i}\right]^0\right\}^2 \tag{4-10}$$

The probability of failure can be again expressed by:

$$P\{Z < 0\} = \int_{-\infty}^{0} f_Z(Z)\mathrm{dz} = \Phi_N(-\beta) \tag{4-11}$$

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. As shown in Figure 4.6, the design point is given by:

$$X_i^0 = \mu(X_i) - \alpha_i \beta \sigma(X_i) \tag{4-12}$$

$$\alpha_i = \frac{\sigma(X_i)}{\sigma(Z)} \frac{\partial Z}{\partial X_i} \tag{4-13}$$

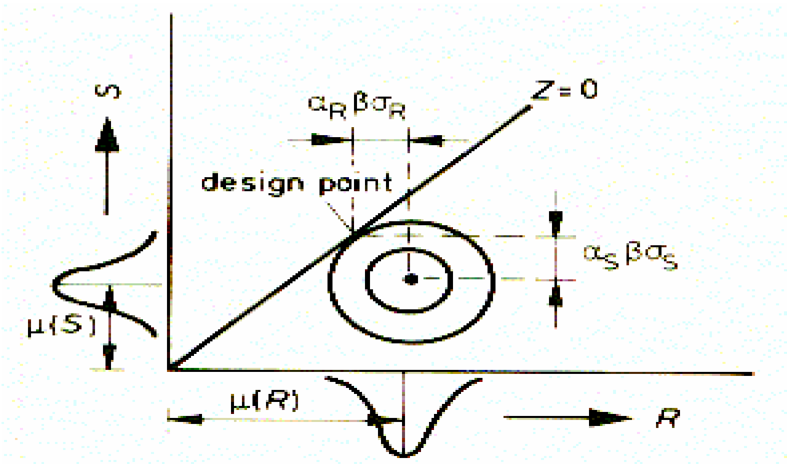


Figure 4.6 Definition of the design point as the point on the failure boundary where the probability density is greatest

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, as shown in Figure 4.8.

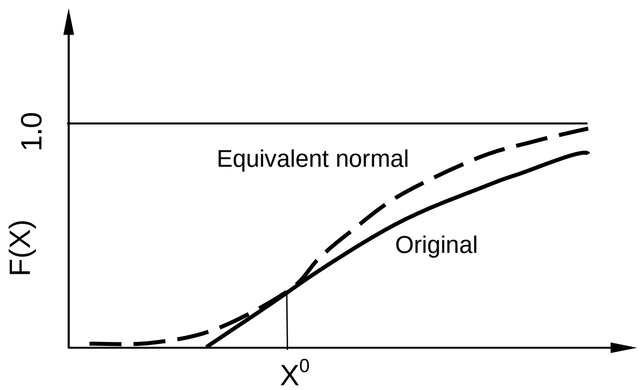


Figure 4.8 Replacing a non-normal distribution by a normal distribution

4.4.3 Monte Carlo method

The Monte Carlo stochastical method based on the central-limit theorem is employed to determine the failure probability for engineering structure. Monte Carlo method calculates the reliability through many random samplings. So it can be applied for reliability analysis of flood defence with non-liner limited state function. The theory of the method described by Huang Kezhong, Mao Shanpei (1987) is as following:

Method of Random Sampling

Firstly, normally distributed random numbers are obtained on interval of (0,1) by mixed congruence method and multiplicative congruential method. The recursion formula of mixed congruence method is:

$$x_i = (\lambda x_{i-1} + C)(\bmod M)$$
$$r_i = x_i / M\,(i = 1,2,\cdots,n) \tag{4-14}$$

Where, λ、x_0、C and M are constants chosen. Formula (4-14) shows the remainder of $\lambda x_{i-1} + C$ divided by M is x_i and random numbers r_i on interval of (0,1) are obtained by x_i divided by M.

The random numbers sequence $\{r_i\}$ is transferred to normally distributed random numbers sequence $\{R_i\}$on interval of (a,b).

$$R_i = a + (b-a)r_i \tag{4-15}$$

By method of inverse function, normally distributed random numbers $\{r_i\}$ are transferred to random numbers in order to meet a certain referred probability distribution. The premise of it is the inverse function of its empirical distribution exists. Otherwise the random variable function method will be adopted.

Suppose X is a continuous random variable whose distribution function is $F_X(x)$ and whose inverse function exists. r is the value of random variable R whose distribution type is uniform distribution function is $F_R(r)$. If the cumulative probability $F_X(x) = r$ is given the following can be drawn:

$$x = F_X^{-1}(r) \tag{4-16}$$

If $\{r_i\}$ has been known sequence of random numbers meet $F_X(x)$ will be obtained:

$$x_i = F_X^{-1}(r_i) \quad (i = 1,2,\cdots,n) \tag{4-17}$$

Basic Process of Monte Carlo Method

(1) Probabilistic distribution models and distribution parameters of the variables related to reliability analysis are determined;

(2) The first random sampling of all variables is done, and the result is used in the reliability function;

(3) Repeat random sampling independently for the total number of simulations n, and then failure probability is estimated.

Results and Accuracy of Monte Carlo Method

In reliability analysis for engineering structures, the limit state function is $Z = g(x_1, x_2, \cdots, x_n)$ and the failure probability is:

$$P_{\mathrm{f}} = P(g(x_1, x_2, \cdots, x_n) \le 0) \tag{4-18}$$

When basic variables are assigned values by random sampling, the result is $g(\cdot) > 0$ or $g(\cdot) \le 0$. So index function can be defined as following:

$$I(g(x_1, x_2, \cdots, x_n)) = \begin{cases} 1, & g(\cdot) \le 0 \\ 0, & g(\cdot) > 0 \end{cases} \tag{4-19}$$

According to Bernoulli's theorem and characteristics of normally distributed random variable the failure probability is:

$$\hat{P}_{\mathrm{f}} = \frac{1}{N}\sum I(g(x_1, x_2, \cdots, x_n) \le 0) = \frac{M}{N} \tag{4-20}$$

Where, M is the total number that $g(\cdot) \le 0$ in the total number N of simulations.
Formula (4-20) is not the only formula to calculate failure probability. The distribution of function is fit according to simulating result and the first moment $\hat{\mu}_x$ and the second moment $\bar{\sigma}_x^2$ are obtained. Therefore, the reliability index β can be obtained as following:

$$\beta = \frac{\bar{\mu}_x}{\bar{\sigma}_x} \quad (4\text{-}21)$$

The error of Monte Carlo method is expressed by $\hat{\mu}_x$. The more discrete the function value Z is, the larger the error is. When simulating number is sufficiently large the standard deviation of estimation values obtained by simulating sample inverses with the square root of simulating number. So the accuracy increases with the increase of simulating number. In general, when simulating number is more than $N \ge 100 / P_\mathrm{f}$ the accuracy may be satisfactory.

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

- First, determination of the failure probability caused by a number of failure mechanisms such as overtopping, piping, macro-instability and the failure probability caused by hydraulic structures according to the Level II approach or the Level III approach.
- Second, determination of the overall failure probability of the dike given the component failure probabilities of overtopping, 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.

4.5 Reliability analysis on two typical dikes or dike sections

As mentioned before, the dike is usually a two-phase fluvial facies structure in Yangtze river basin. Its top layer is relatively impervious soil, with a non-uniform thickness, or a clay layer with sandy lenticle. A quite thick (more than 10 m) sandy layer exists below this layer. The grain size of the sand becomes gradually coarse with increasing depth, and the bottom is sandy gravel. In city areas, the surface consists of a few meters of miscellaneous fill, with a large permeability. Herein, two typical dike sections have been used in this study, which can test the software system to some extent. By comparing the numerical results with the dangerous records or latest geological exploration, the geotechnical parameters and numerical models can be verified. If numerical results and field survey are in qualitative correspondence, the parameters and numerical implementation methods are reliable.
Some soil strength parameters of levee and its foundation are taken as random variable, for the homogenous embankment, the effects of various factors such as geotechnical statistic parameters and geometry of dikes on reliability index or structure risk degree for overtopping and piping and sliding, have been investigated systematically. Then, limit state equations of overtopping, seepage stability and slope stability of the typical dike with inclination watertight facing and on two-phase fluvial facies base are formulated, and the influence of the variables of geometry on various failure modes has been discussed. An overview of some results on the typical dike sections is given below.

4.5.1 Homogenous embankment

Taking a typical homogenous dike as an example, the upstream slope ratio m_1 =3, the downstream slope ratio m_2 =3, the width across the crest w =7m, the height of the dike h_0 =10m, the water level of upstream H_{uw} , the water level of upstream H_{dw} , as shown in Figure 4.9.

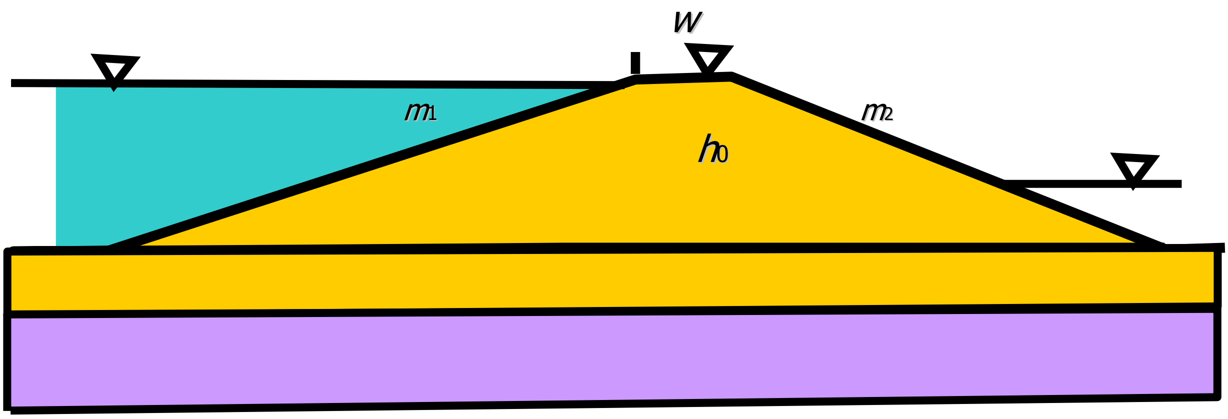


Figure 4.9 Sketch of the cross section of homogenous embankment

In reliability analysis, many variables, such as shear strength parameters, bulk gravity, pore water pressure, critical gradient, water level of upstream and downstream should be taken to be random variables. To simplify, partial geotechnical parameters used in this study as random variables, which are listed in Table 4.1. In each case only one parameter vary, other parameters remain the values as listed.

Table 4.1 Statistic of geotechnical parameters

Stochastic variables	Symbol	Name/unit	Distribution type	Mean value	Standard deviation
x_1	c	Cohensive (kPa)	Normal	12.54	2.8
x_2	ϕ	Inner friction angle(˚)	Normal	21.58	3.5
x_3	γ	Bulk gravity (kN/m³)	Normal	18.84	3.1
x_4	J_c	Critical seepage gradient	Normal	0.25	0.093

In order to investigate the effects of geometry of dikes on the structural risk at a certain water level (0.5 meter below the crest height), different values of slope ratio, width across the crest and height of the dike is performed. The following results are obtained with varying only one of the parameters. It is shown that the instability risk of inside slope is much larger than that of outside slope, only the risk of inner side slope is considered.

(1) Upstream slope ratio

The risk degrees of slope instability and seepage deformation instability of the dike body with different upstream slope ratios are shown in Figure 4.10. It can be seen that the variation of upstream slope ratio has little influence on the stability of downstream slope. The risk of piping obviously decreases with the increasing of upstream slope ratio, because of the increasing of the length of seepage path.

(2) Downstream slope ratio

Figure 4.11 shows the risk degrees of different downstream slope ratios. The risk degrees of slope instability and seepage deformation instability decrease drastically with the increase of downstream slope ratio. It should be noted that more flat slope and platform can improve the capability of the dike body to resist seepage damage in practical engineering, which is rational in theoretical and the costs will increase.

(3) Width cross the crest

The instability risks of flood defences with different width cross the crest are shown in Figure 4.12. The variation of w has little influence on the risk of slope instability, but has obvious influence on the risk of piping.

(4) Height of the dike

Figure 4.13 shows the instability risks with different heights of the dike. The conclusion can be drawn that the risks of slope instability and seepage deformation instability increase with the increasing of height of the dike. There are two reasons for the increasing of risk of slope instability. One is the water level and saturation line increase with the increasing of height of the dike. The other is the increasing of sliding moment is more quickly than the increasing of moment against sliding. The reason for the increasing of risk of seepage deformation instability is the increasing of the height of the seepage exit of downstream slope is larger than the increasing of the water level of upstream, then the seepage gradient increases.

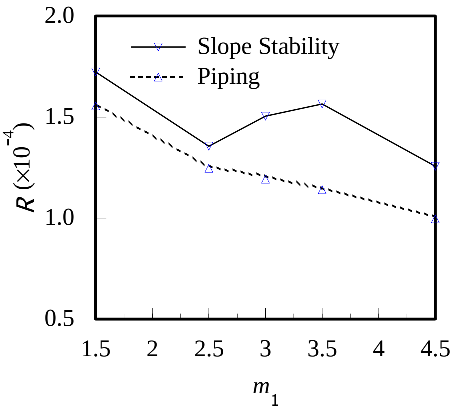


Figure 4.10 Influence of upper slope ratio

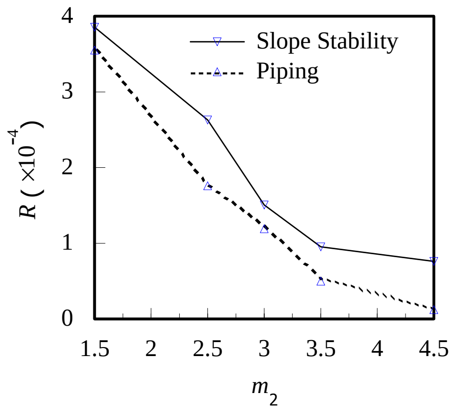


Figure 4.11 Influence of down slope ratio

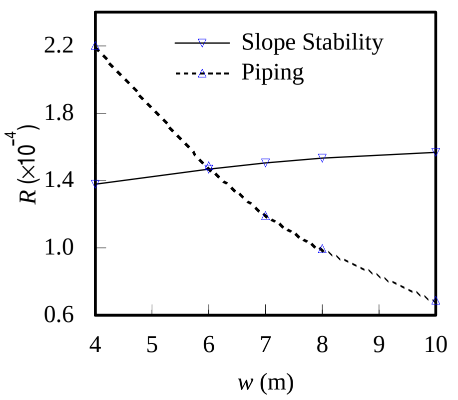


Figure 4.12 Influence of crest width

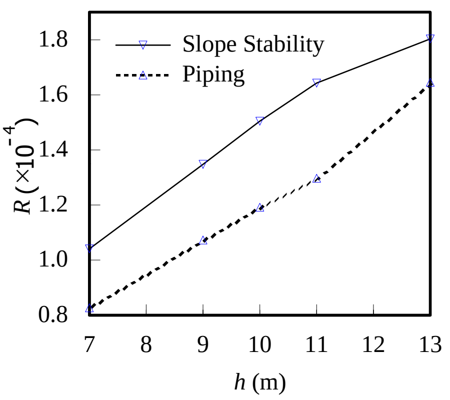


Figure 4.13 Influence of height of dike

The variation of the geometry of dikes has effects on β_s and β_p. The variation of upstream and downstream slope ratio only has effects on the corresponding slope instability risk. Seepage path enlarges with the increasing of slope ratio, and the risk of piping decreases.

Increasing of width cross the crest has great influence on the risk of seepage deformation instability and has little influence on the risk of slope instability.

4.5.2 Inclined-facing embankment

An idealized flood defence with inclination clay layer is analyzed, as shown in Figure 3.1. It is supposed to be a two-phase fluvial facies with its basement constituted of weak pervious clay stratum and strong pervious sand stratum.

The values of some design variables and deterministic parameters and random variables in this analysis are listed in Table 4.2 and Table 4.3. Herein only the random variables characteristic has been given in Table 4.2, other variables can refer to the Table 3.1.

Table 4.2 Random variables of inclined-facing embankment

Variable	Description	Unit	Type	Mean value	Standard deviation
d_{ks}	Effective thickness of clay	m	Normal	3.5	0.7
h_w	Flood water level	m	Exponent	8.34	0.9
k_c	Permeability coefficients of clay	m/s	Normal	$10E^{-8}$	$2*10E^{-8}$
k_s	Permeability coefficients of sand	m/s	Normal	$10E^{-5}$	$0.75*10E^{-5}$

Table 4.3 Variables of overtopping modeling on typical dike

Symbol	Description	Distribution	Mean Value	Deviation of Standard
h_0	height of dike crest	Normal distribution	11.0m	0.051m
h_w	Flood water level	exponential distribution	8.34m	0.9m
h_s	swash height	Normal distribution	0.638	0.44
e	surge height			

(1) Influence of crest height on reliability index of overtopping

Reliability index β_1 by Monte Carlo method corresponding to the mean value of the height of dike μ_{h_0} is obtained, as shown in Figure 4.13. When μ_{h_0} is 11.0m, β_1 is 2.363, and P_f is 0.914%.

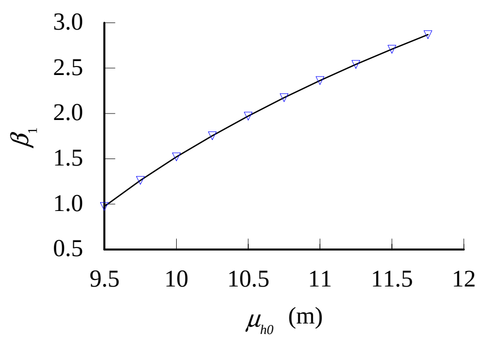


Figure 4.13 Relation of height of dike and reliability index

(2) Influence of geometry parameters on reliability index of piping

(a) Thickness of clay layer

The relation of thickness of clay layer and reliability index of seepage stability β_2 is shown in Figure 4.14. It shows that thickness of clay layer has strong influence on seepage stability of dike foundation.

(b) Width of foreland

Figure 4.15 shows that the increase of β_2 with the width of foreland. The reason is that the increase of width of seepage path of foundation intensifies the capability of impervious of soil. But in this example, β_2 decrease slightly when the width of foreland increases to more than 20.0m. It is an effective method to control seepage for the structure to adopt natural covering layer when there is wide foreland at upstream for practical engineering.

(c) Slope ratio of flood defence

Figure 4.16 shows that β_2 increase with the increase of slope ratio of flood defence.

(d) Width cross dike crest

It is shown in Figure 4.17 that β_2 increase linearly with the width cross dike crest. The increase of slope ratio and width cross dike crest changes the width of seepage path. But it will raise the construction cost. It is an important problem to be deeply studied to balance the relation of construction cost and expected flood damage with the introduction of conception of economic assessment.

(e) Permeability coefficient of clay layer

The permeability coefficient of soil can reflect mechanical composition, structure, tightness and pore size. The decrease of β_2 with the increase of permeability coefficient of clay layer is shown in Figure 4.18. When the permeability coefficient of clay layer is more than 0.5×10^{-8}, β_2 decreases obviously.

(f) Permeability coefficient of sand layer

The decrease of β_2 linearly with the increase of permeability coefficient of sand layer is shown in Figure 4.19. The conclusion can be drawn embankment fill should be chosen carefully and compaction quality should be controlled strictly.

(g) Thickness of piping-berm

Figure 4.20 shows that the reliability index of piping increases with the thickness of piping-berm.

(h) Width of piping-berm

The reliability index of piping increases with the width of piping-berm, as shown in Figure 4.21.

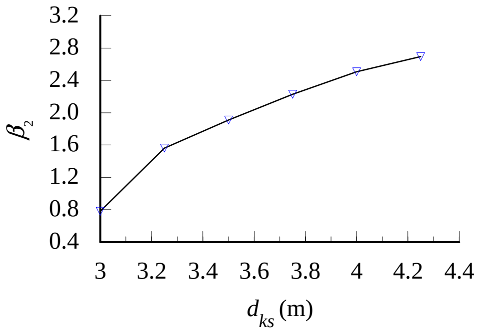


Figure 4.14 Thickness of clay stratum

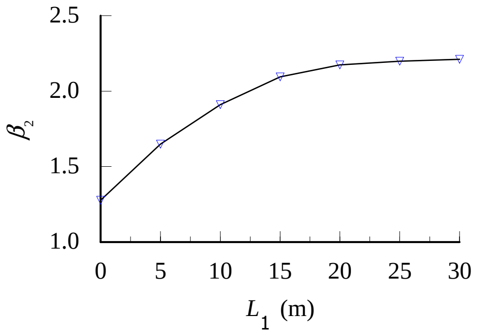


Figure 4.15 Width of foreland

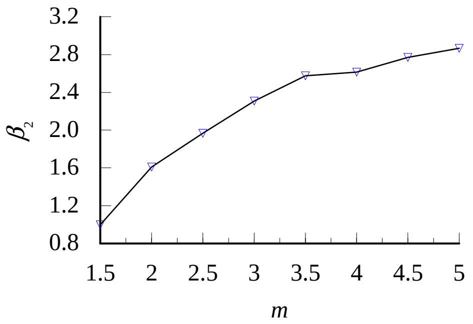


Figure 4.16 Slope ratio

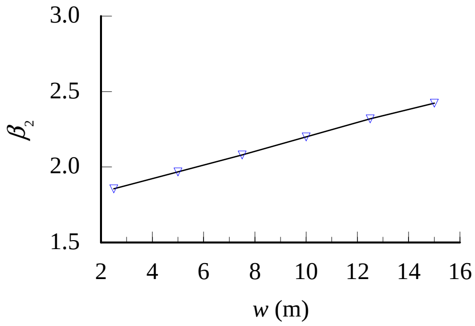


Figure 4.17 Width across the crest

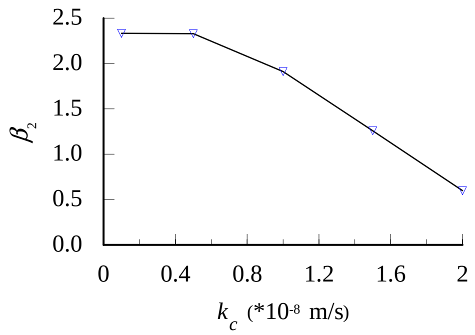


Figure 4.18 Permeability coefficient of clay

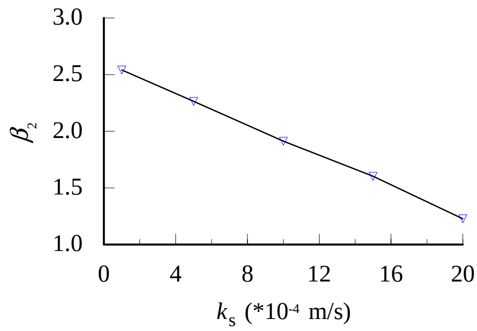


Figure 4.19 Permeability coefficient of sand

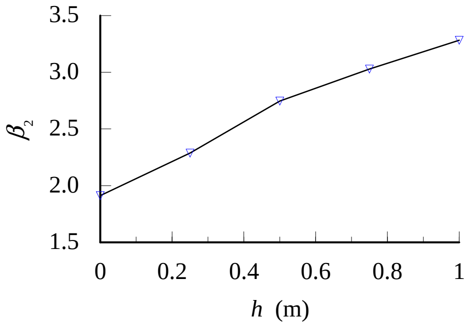


Figure 4.20 Thickness of piping-berm

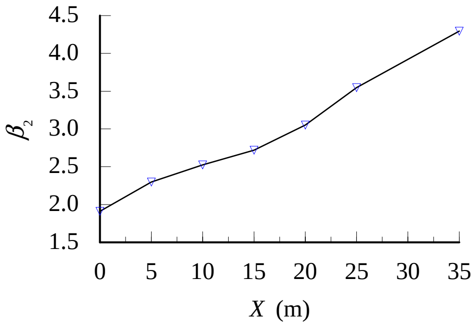


Figure 4.21 Width of piping-berm

(3) Influence of slope ratio on reliability index of sliding

The relationship of slope ratio of dike and reliability index of slope stability is shown in Figure 4.22. It shows that when slope ratio is 2.5, β_3 is 2.756, and the corresponding failure probability is 0.289%.

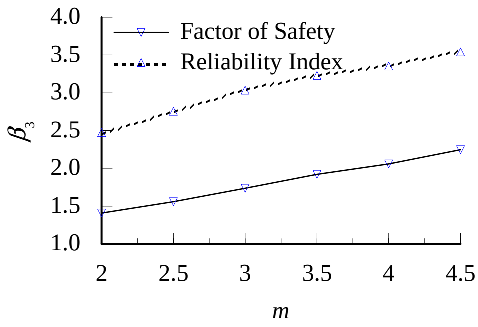


Figure 4.22 Influence of slope ratio on reliability index of sliding

The following conclusions can be obtained: (1) The reliability index of overtopping increases with the height of dike. (2) The reliability index of seepage stability of dike foundation increases with the slope ratio of dike, width cross crest of dike, width of clay layer of dike foundation, width of foreland, width and thickness of gland behind dike. (3) The reliability index of seepage stability of dike foundation decreases with the increase of seepage coefficient of clay layer and sand layer of dike foundation. (4) The reliability index of slope stability increases with the slope ratio of upstream and downstream of dike.

4.6 Breach growth and flood simulation

4.7 Consequence assessment

4.7.1 Economic damage

4.7.2 Other types of damage

Chapter 5 Safety Evaluation and Risk Assessment

[**Abstract**] This chapter will give an outline of technical standard and risk level for safety evaluation corresponding soil structure, and decision-making method after a risk analysis.

5.1 Current Safety requirements

5.2 Risk assessment method

5.3 Choice of the risk level

The consequences of a flood can be far-reaching: loss of human life, goods and means of production and damage to landscape, nature and cultural heritage. The approach is addressed below, along with the choices that have to be made and the developments involved. First the considerations that should play a role in the construction and maintenance of flood defences are indicated. Knowledge is always limited so not all aspects mentioned can be assessed in full.

The starting point for all safety approaches is the risk that accompanies the problem in question. In the case of water defence the risk is the probability of flooding combined with the corresponding consequences. This can be expressed as the probability (so many times a year) multiplied by a certain consequence (a measure of the loss of money and/or human life). The measure for the risk is the average loss of money or human life per year. The definition of a certain accepted risk indicates that the greater the consequences the smaller the probability must be. It is not possible to totally preclude risk, because the probability 0 is impossible, given the lack of an upper limit to natural phenomena. The choice, and so also acceptance of a risk level is accordingly all about pros and cons. In practice emotions also play a role in the ultimate choice.

On the other hand, the improvement of flood defences also demands sometimes great social sacrifices. This is all about the expense for the construction and maintenance of flood defences, and the loss of landscape, nature and cultural heritage that can be the consequence of the construction or improvement of flood defences.

The requirements set for the degree of safety of the areas behind it must therefore be based on a consideration between the social sacrifices and the benefits of flood defences. The risk approach is an aid here, by which both certain occurrences (investments) and uncertain occurrences (probability of dike collapse and the consequences) can be assessed. If the investments and the sacrifices are both expressed in financial terms then an econometric calculation can be made, to determine the optimal safety level. Any loss of human life makes this approach a discussible one to say the least. So in the consideration both objective and subjective elements play a role.

5.3.1 Individual risk

5.3.2 Societal risk

5.3.3 Econometric optimization of the probability of failure

5.4 Decision making after a risk analysis

A risk analysis is aimed at quantifying the above mentioned factors and most important of all using these results in a sensible way for decision making.

5.4.1 Reduction of the probability of failure

5.4.2 Reduction of the consequences

5.4.3 Reduction of the uncertainties

Chapter 6 Software System of Risk Assessment for Water Defences

[**Abstract**] Two software systems for safety evaluation, PC-Ring and SADSS, which have been used in the Netherlands and China, respectively, were described. Main functions and features were discussed.

6.1 PC-Ring

In 1960, the Delta Committee presented the foundations for the current Dutch safety approach against flooding. They noticed that a safety approach should preferably be based on flooding risks. In doing so, probabilities and consequences of flooding would have to be considered together and in coherence. Until 1992 the knowledge necessary to interpret this further was not available.

In recent years an advanced program for reliability analysis of ring dike systems has been under development: PC-RING. It implements reliability analysis of the elements in a ring dike system, considering all principal dike failure modes.

The PC-RING analysis generates an estimate of the probability of system failure and an indication of the contribution that each dike section makes to the probability of failure. Thus weak links in the dike ring can be identified, as well as the mechanism and variables that contributes most to the probability.

The Dutch reliability methods for flood defences mainly consist of two parts. (1) a process that reduces the amount of work related to data gathering by selecting the cross sections that are most representative of the system's probability of failure. (2) software called PC-Ring which calculates the probability of failure of a flood defence system. These selected cross sections are included in the calculations with PC-Ring.

6.1.1 The PC-Ring incorporated failure modes

PC-RING repeatedly calculates the reliability of dikes or of a composite of dike sections for a fixed period. Failure is the result of the occurrence of one or more of the following mechanisms:

–Overflow/overtopping
–Sliding/uplifting
–Heaving/piping
–Erosion of dike revetment
–Dune erosion
–Failure of hydraulic structures

6.1.2 Probability of failure per section and per mechanism

The first step in calculating the probability of failure of a dike ring comprises subdividing the dike ring into several dike sections. A dike section is defined as part of the water defence in which the main characteristics in lateral direction may be seen on average as constants. This refers to characteristics such as slopes, verges, crest height, orientation, but also revetment type, average soil properties, soil layers, etc. It could therefore be necessary to divide a specific dike section for a specific mechanism into two parts, which is not necessary for other mechanisms.

When the dike ring is divided into dike sections, one can calculate the probability of failure of each dike section and each mechanism. In principle, this calculation requires three ingredients: (1) A model as a description of the mechanism; (2) Data for the deterministic variables; (3) Data for the stochastic variables.

In some cases users can choose from several different models. For probabilistic calculations, the mechanism model is usually determined by means of a so-called limit state function, $Z = g(X)$, whereby X is the vector of stochastic variables. Per definition, negative values of Z correspond to “failure ” and positive values of Z, to “non failure ”.

In PC-RING the below mentioned main methods are available to calculate the probability of failure of one limit state function for a single element. (1) FORM (First Order Reliability Method); (2) SORM (Second Order Reliability Method); (3) MC (Crude Monte Carlo);(4) DS (Directional sampling).

A number of combinations of the above mentioned methods: for instance an option that PC-Ring automatically switches to DS if convergence does not occur in a calculation with FORM or SORM. Other options involve the combination of DS and FORM, the first method is then used to find a first estimate of the design point.

The reliability analysis uses a set of around 50 variables that can be regarded as stochastic or deterministic parameters. The user is responsible for the deterministic and the stochastic variables and determines whether a given situation will be calculated for a dike section, a design variant or a potentially temporary situation, etc.

In principle, data should be entered as: (1) Geometric data (slope, orientation, fetch, etc.); (2) Material properties (weight, sliding strength, etc.); (3)Hydraulic load (wind and water level).

For a deterministic variable a single value has to be entered. For a stochastic variable the following numerical values have to be entered: (1) Variation around the average (standard deviation or variation coefficient); (2) Spatial correlation within a dike section and between dike sections; (3)Correlation in time.

The global form of spatial correlation in PC-Ring is:

$$\rho(\Delta x) = \rho x + (1-\rho x)\exp\left\{-\frac{\Delta x^2}{d_x^2}\right\} \tag{6-1}$$

where $\rho(\Delta x)$ =constant correlation, d_x =correlation distance. The following auto-correlation function is used when a 3D spatial deviation has to be taken into account.

$$\rho(r_x, r_y) = \exp(-(r_x / d_x)^2)\left\{(1-\alpha) + \alpha \exp(-(r_z / d_y)^2)\right\} \tag{6-1}$$

where r_x and r_y =horizontal respectively vertical distance between two points, d_x and d_y =horizontal respectively vertical fluctuation scale and α =variance ratio factor.

6.1.3 Probability of failure per dike ring

Subsection 6.1.2 described the first step of a PC-Ring calculation. For example, the calculations are successively performed for the mechanisms of overflow/overtopping, sliding, etc. Per mechanism, a result for each dike section is obtained separately (i.e. reliability index β and a list of α-values indicating the importance of a variable ’s uncertainty).

After calculating all the probabilities per mechanism, the system probability has to be analyzed. Consider a system consisting of n elements. Two elements of the system are picked out and are combined to form one equivalent representative element. In other words two limit state functions are combined to one. The total amount of elements in the system is reduced from n to $n-1$.Repeating this procedure over and over again will eventually reduce the amount of elements in the system to one. In other words, the system is "wrapped up". The procedure to find one equivalent representative Z-function is according to the method of (Hohenbichler & Rackwitz 1983).This procedure calculates $\mathrm{P}(\mathrm{Z}_1 < 0 \text{ AND } \mathrm{Z}_2 < 0)$ taking the mutual correlation into account. If this probability is known, $\mathrm{P}(\mathrm{Z}_1 < 0 \text{ OR } \mathrm{Z}_2 < 0)$ can be determined.

The total procedure is as follows:

(1) Calculation of the probability of failure of one flood defence cross section for one tide, one partial failure mode (for instance failure mode overtopping, partial failure mode saturation),given the wind direction.

(2) Combination of the partial failure modes resulting in the probability of failure of one total failure mode.

(3) Taking the probability of the wind directions into account.

(4) Determining the probability of failure due to one failure mode for the total flood defence stretch for which the under step 1 mentioned flood defence cross section is representative.

(5) Combining the probabilities of failure of all the wind directions.

(6) Determining the probability of failure for the total regarded period.

(7) Combining the probabilities of the different failure modes.

(8) Combining all the flood defence stretches to find a total flood defence system 's probability of failure.

6.2 SADSS

Safety Assessment and Decision Support Software System of Dike (SADSS) is a practical system developed with the ability of the conventional process and probabilistic risk evaluation. This software system is simple to install, uses a mouse and/or keyboard control, and has pull-down menus and interactive dialog boxes, written in visual basic together with dataset platform of SQL Server. It developed by Research Center on Flood and Drought Disaster Reduction of Ministry of Water Resources of China (RCDR).

This system has three functional modules: probabilistic and risk analysis; slope sliding analysis; seepage analysis. Based on the data of typical dike subsection, the real-time computing of the risk degree can be carried out at different flood water heights. Thus the dynamic risk diagram on a whole dike ring system can be submitted. Moreover, the different safety grades can be classified according to specified standards and rules, and the corresponding strengthening measures or forecasting schemes can be obtained.

It provides a user-friendly interface that facilitates easy interaction with operator in data inputting and updating and editing graphics.

This system has three functional modules: probabilistic and risk analysis; slope sliding analysis; seepage analysis, and the major interfaces are shown in Figure 6.1. Main features are as followings:

1) Some mathematical models and solution procedure and specified boundary condition have been modularized and visualized. Which can process a fully automated analysis, and

it almost requires no programming knowledge or skill in this field.
2) For a typical dike subsection, sliding and seepage analysis, probabilistic and risk analysis.
3) For a whole dike section, a graphic displaying based on GIS map enable the user to win a maximum of information from the calculating and to publish the results very easy.
4) Graphics are made on the screen, and may be exported to clipboard or bitmap (*.bmp) file. The graphics can be zoomed and the cursor position corresponding to coordinate value can be shown on the status bar.
5) First, second and third type boundary conditions are available for both flow and transport simulations. A 2D finite element model can simulate flow in both steady and unsteady. Solutions can be displayed as plots of flow vectors and head contours or as a complete flow net with equipotential lines and flow lines.
6) The Swedish and Bishop simplified method determine the stability of circular failure surfaces.
7) The Monte Carlo numerical simulation method has been employed to calculate the probability.
Based on the data of typical cross sections, the risk degree can be real-time calculated at different flood water levels according to the numerical results, such as factor of safety, reliability index and probability of flooding. Thus the dynamic risk diagram of the whole dike can be submitted. Moreover, the different safety grade can be classified according to specified standards and rules, the corresponding strengthening measures or forecasting schemes can be obtained by the existent scenario database.

Figure 6.2 and Figure 6.3 show that the main functional modules and the developing flow process of this system. Figure 6.4 shows that the illustration of main functions based on GIS map.

6.3 Comments

PC-ring presents an approach to select the appropriate cross sections of the flood defence system that contribute most to the total probability of failure. This approach limits the amount of work: instead of modelling the complete flood defence system and gathering data for the total length of flood defence, a limited amount of cross sections is selected.
The computational model for more failure modes should be improved on SADSS.

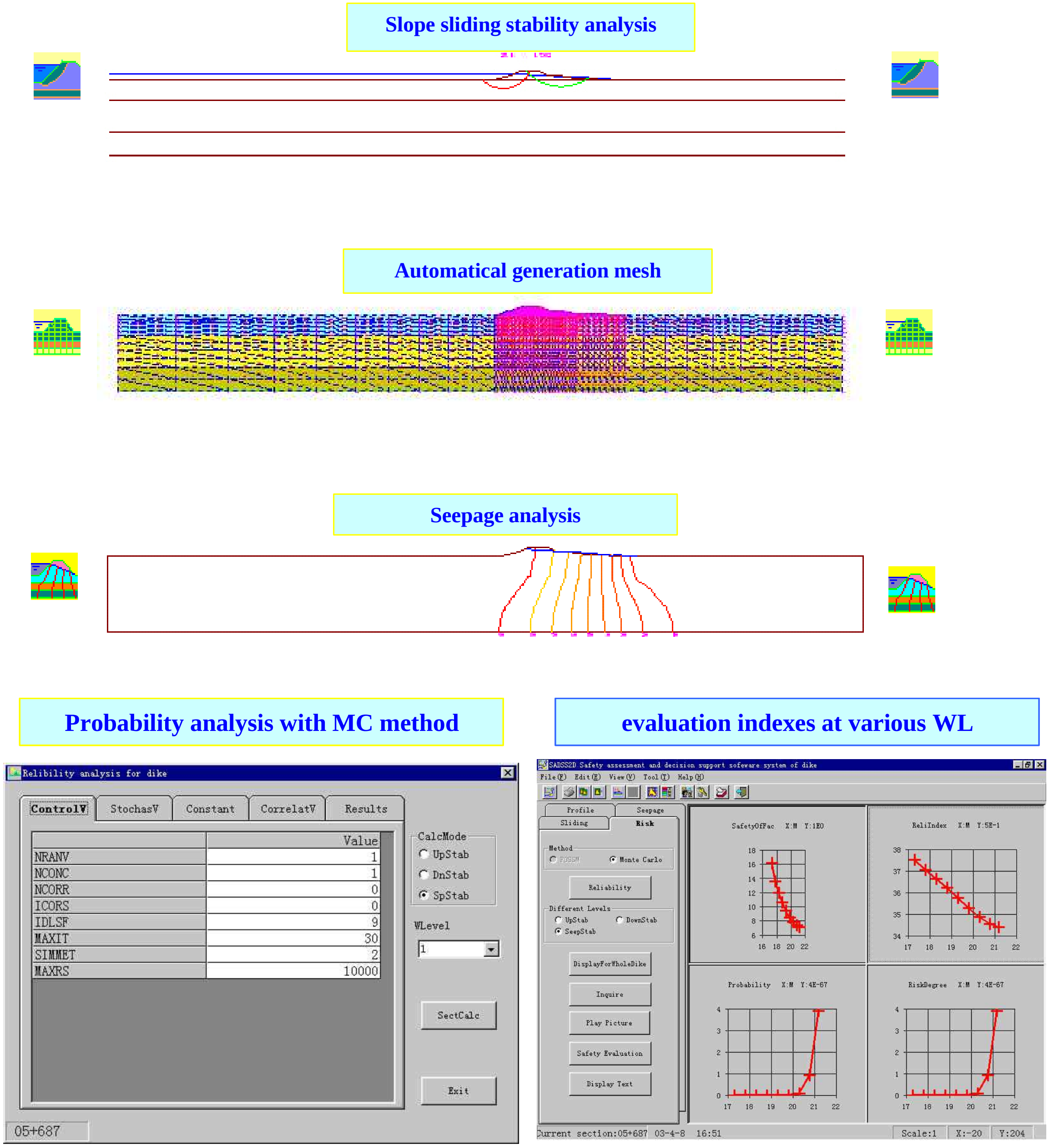


Figure 6.1 Main interfaces of SADSS

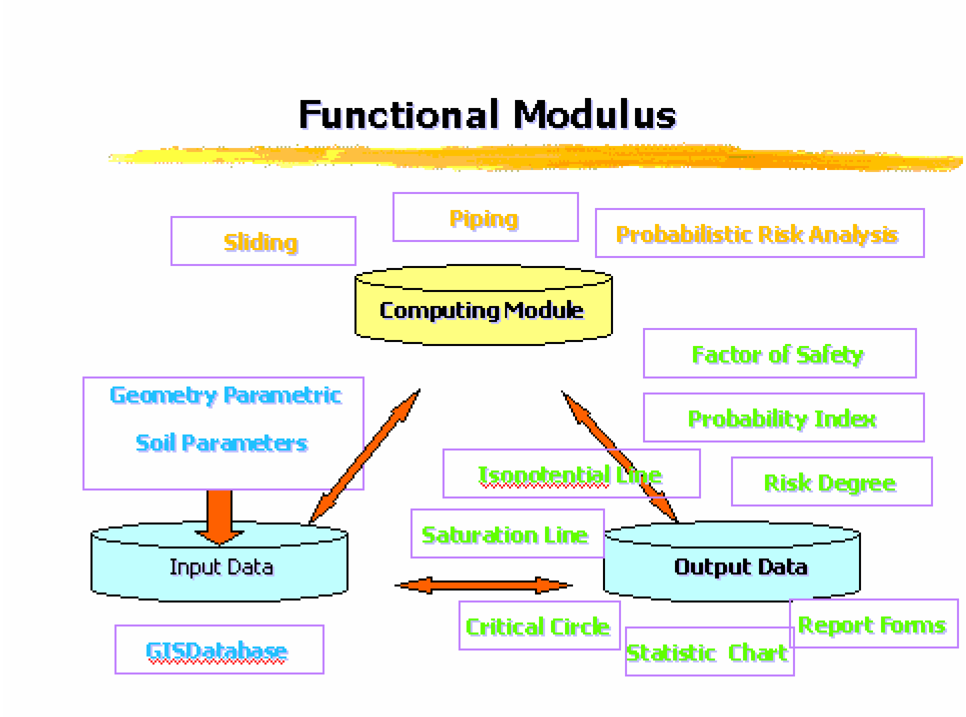


Figure 6.2 Main functional modules of this system

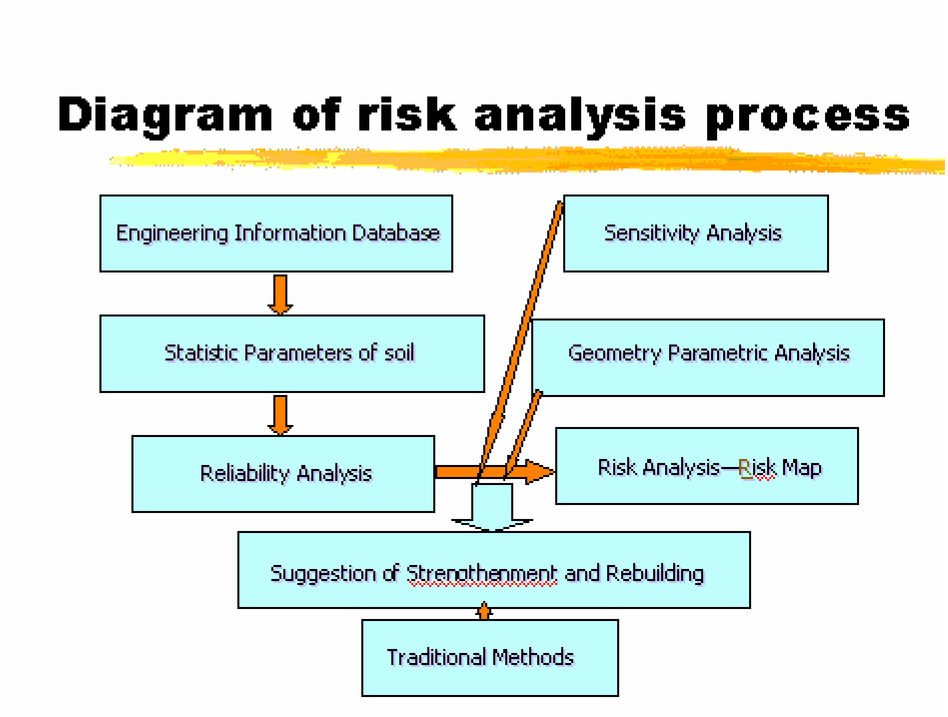


Figure 6.3 Developing flow process of this system

Figure 6.4 Illustration of main function for SADSS

Part III: Case study: Risk assessment for Anqing Dikes

Chapter 7 Description of Anqing Study Area

[**Abstract**] Dike is the oldest and most essential flood protection measures in Yangze basin. The dike construction has been lasted 2000 years, and the relative dike system has formed. Its length is about 36,000 kilometers, with the exception of coastal leeve. The 56 km Anqing section of the Yangtze River is an important part along the 6,300 kilometer Yangtze. The large-scale reinforcing project on this dike also was described in detail.

7.1 The Yangtze River

For many years, flooding is frequent in the Yangtze basin with floods occurring in the middle and lower basins, resulting in huge losses to people's property and lives and serious damage to the local ecology and environment. On the Yangtze, the 1931 event flooded 3.3 million ha and 140,000 people died, and the 1935's flood in the Han and Lishui tributaries led to losses of a similar magnitude. The 1954 flood on the Yangtze, one of the worst in history, caused widespread damage and loss of life that would have been much worse without the new, recently completed dikes.

By contrast, 4,150 people were killed across China during 1998's summer floods which swept over China's 29 provinces, causing 255.1 billion yuan's (US$30.7 billion) worth of direct economic damage, affecting more than 22 million hectares of crops and causing more than 6.8 million houses to collapse. On the middle and lower reaches of the Yangtze's main streams, as well as areas around Dongting and Poyang lakes, more than 2.2 million locals were affected during the flooding which inundated 197,000 hectares of farmland following the crumbling of 1,075 embankments.

Following 1998's devastating flooding, the central government poured 160 billion yuan (US$19.2 billion) into construction of water conservation projects.

For the Yangtze alone, some 29 billion yuan (US$3.4 billion) was earmarked during the 1998-2002 period to consolidate the roughly 3,000-kilometre-long major levees shielding vast areas along the river's middle and lower reaches. "Such massive investment for water conservancy projects was unprecedented in China's history and the world's history as well," Wang (Minster of WRC) said. Wang said confidently that "in the next 50 to 100 years, there will be no need for China to launch another large-scale reinforcement of the levees there." So far, no major flood-related mishaps have occurred along the main levees of the Yangtze, even though the river has been hit by flood crests as reported by the provinces stretching along its middle reaches, the minister said.

7.2 The dyke system of Anqing City

Anqing City is located at the northern bank of Yangtze River downstream and is one of the cities threatened seriously by flooding from Yangtze River main stream, as shown in Figure 7.1. There have been six years in which disastrous floods rushed into the town during the 20th century. Anqing dike and Guangjiwei dike are built from Song and Yuan dynasty and they have been the main flood-prevention projects of Anqing city after hundreds years of

continuous construction and restoration. The two dikes were connected at one's beginning and another's end, starting from Lion Mountain bottom in Anqing suburb and ending at Muqi Mountain bottom in Zongyang county. Combining with Meilin separation levee, they have formed the flood-prevention barrier of Anqing suburb, Zongyang county and Tong.
In the past, Anqing dikes were constructed, damaged and rebuilt repeatedly, so its filling components are very complex. Ditches and ponds inside, outside or near the river bank are widely distributing, ancient watercourses are developed and hidden diseases and disasters. During flood reason, some failure mechanisms such as disperse-soak, piping and collapse frequently occurred.

There are four (main) types of structure for the protection of a dike ring area against high water. These are (see figure 7.2):
- River Dunes;(near Anqing)
- Soil structures (dikes, dams);
- Special water retaining structures (including cofferdam, retaining wall, sheet piling);
- Water retaining hydraulic structures (including locks, cuts, pumping stations).

7.3 Natural conditions

Anqing city is an important flood protection city of China, which has superior natural condition, fertile soil, well-growth economy. The sections of Anqing, Guangjiwei, and Meilinge dike are the principal flood protection works, of which the length is 49.985km. The total area of the protected zone is 233.1 km^2, which lies beneath the flood water elevation, and in which the agricultural acreage is 0.14 million and the population is 0.38 million.

Figure 7.1 The location of the Anqing dikes between Nanjing and Wuhan, northside of the Yangtze dike

7.4 Large-scale Reinforcement on this dike since 1998

7.4.1 Standard

Before 1998, the main dikes along the Yangtze River, could only protect against a flood with a frequency of one in 10 to 20 years. Only 20 percent of the city embankments could protect against flood frequencies of one in 50 to 100 years. There is a great deal of difference between China and foreign countries. For example, the dikes in the basins of the Mississippi River and the Missouri River can control floods with frequencies of one in 50 to 100 years. In 2000, China has stipulated in the "code for the design of levee projects"(GB50286-98) that the first grade dikes should be able to withstand one in 100 years floods, and the second grade one in 50 to 100 years.

According to the plan on Yangtze River basin, the vast flood disaster of 1954 is taken as the defensive target for the river's middle and lower reaches hereinafter the Chenglingji section. Then, the water level of hydrological station in Anqing is 19.34m. The design water level of Meilinge dike section is assumed as the design level of the section linkage section with the Guanjiwei, i.e. 18.30m.

Based on the scenario of urban flood defence in Anqing, the grade of Anqing section (00+000～18+837) is Grade one, and Guangjiwei section (18+837～40+439) is Grade two. The section of Meilinge (M0+000～M6+296) belongs to grade one. The corresponding culvert and sluice are grade one and grade two.

The following aspects have been completed in the reinforcement of Anqing section of the Yangtze River in Anhui province: heighten and widen of dike body, remove or rebuild anti-flood wall, inside and outside berm, grouted cutoff wall, revetment, flood protection highway, fill polder, relief blind ditch, relief well, vertical anti-seepage wall, water system restoration.

7.4.2 Reinforcement of dike body

(1) Widen and heighten

The width of the embankment is 10m for Grade one, 8m for Grade two, and slope ratio is 3. When the net height of dike body is larger than 6m, a platform should be constructed below 3m of the crest at inner slope. The platform or berm located at toe of dike should be determined by calculating results. According to the analysis of wind wave, the elevation of the crest has been defined by the design water level added by 2m freeboard.

(2) Treatment of potential failure

The filling materials of these sections are very complex, composed by clay, silty clay, loamy soil, construction rubbish, and living rubbish. The termite runway distributes along the dike. In this case, some measures, such as grouting or clay anti-seepage wall, have been taken in some sections with bad construction quality of dike body, to improve the density of filling and permeability.

(3) Revetment

(4) Vertical anti-seepage with plastic membrane

Since the existence of penetration sand layer in the top stratum for the section from 11+367 to 15+687, depth of 3-8m, a vertical anti-seepage wall with plastic membrane has been constructed.

(5) Flood prevention roadway

The flood prevention roadway of levee crest have been consolidated, the pavement of placing concrete in site for the urban and upper part of Guangjiwei section, the asphalt concrete pavement used in other sections. The grade is the third.

7.4.3 Reinforcement of dike foundation

Five types of dike foundation can be classified by the geological condition and seepage analysis results on these sections: 1) homogenous clay foundation or upland water retaining; 2) deep silt loam soil, silt clay foundation; 3) top layer is natural filling soil, and then is silt loam soil or sand and gravel foundation; 4) foundation with rather deep covering on the sand layer, top layer included with sandy loamy soil; 5) foundation with thin silt loamy soil on top layer and very deep sand stratum.

For the first one, if there are almost not any potential dangers in some sections, they need not to take some measures. The gravity balance platform should be constructed if deep instability exists in the dike foundation. For the second one, expect for the back-up polder, any other measures need not to be taken. For the third one, it locates the section of urban flood protection, replacement soft soil or grouting or new construction plastic concrete anti-seepage wall. For the forth one, a vertical anti-seepage plastic membrane should be built in the toe of the outer slope. For the fifth one, some measures such as piping berm and relief ditch or relief well can be taken. The detailed treatment is the followings: 1) inner berm with the width of 30m, and outer berm with the width of 50m; 2) piping-berm with the width of 150m; 3) new construction of the relief-ditch; 4) .back-up filling of the invalid relief-well.

7.4.4 Water system restoration

7.4.5 Revetment

7.4.6 Strengthening on the culvert and sluice

7.4.7 Cost - benefit- analysis

The structural measures of reinforcement and the developing of the decision-making system against flooding can provide a safeguard and condition for the social and economic developing. The flooding frequency will be decreased, and the dike can withstand the flood water occurred in 1954. The indirect benefit will be seen by the following aspects: 1) a large amount of casualties and loss on property resulted from the flood disaster can be avoided; 2) The influence of normal order in product and living due to participating the fight against flood can be relieved; 3) The ecological environment of the protected area can be improved; 4) The serious effects of potential epidemic disease and water-quality deterioration resulted by the inundation can be avoided.

The best cost-benefit of the strengthening project will be gained according to the report, mean annual economic benefits is 139 million Yuan. The internal ratio of return of this project is 19.67%, and economic net present value is 548,8 million Yuan. Multiple indexes exceed the state evaluation standard.

Chapter 8 Data Collection

[**ABSTRACT**] According to our ongoing project, some gathering data, especially in geological condition and evaluation, geotechnical problem, geotechnical parameters, the profiles of cross sections, have been description in detail.

8.1 Introduction

Investigating on site visit and gathering data under the guidance and help of the project coordination group. Data and information for the case study have been obtained during the past two years by practicing an ongoing project. Those data came from various organizations related to the case study subject. In particular: Research center of flood and drought disaster reduction of WRC (RCDR); Anqing Dike Department of Anhui province; Committee of Huai River; Committee of Yangtze River.

Gathering information in China is a time consuming and difficult task. In some cases not enough information was available or the involved organizations asked money or were reluctant to give the information. In those cases simplifications and assumptions had to be made.

The foundation data should be included as follows:

1) Topographical maps of urban (scale not less than 1:10000); Topographical maps of suburb (scale not less than 1:10000). Which should not be plotted in earlier stage than the 90's of the 20th century.

2) Thematic maps of hydraulic engineering and their relevant data should be included, such as design parameters of hydrological planning; dike distributions and standards of flood-prevention and elevations; the topographic map along the dike (the scale is 1:2000); the representative section structures of dike; chainage, type, structure characteristic, apertures, bottom elevation of the accessory structure and their relevant photos; flood-prevention, and distribution of stock material for emergency treatment.

3) Information about dike designing and planning, historical evolvement of dike, engineering information of annual maintenance and strengthening dike and appurtenant work (such as, wave and flood protection works, forest against wave, revetment, pavements and drainage facilities).

4) Geotechnical parameters and geological exploration data: nature density and moisture content of soil, specific gravity of soil particle, granulometric composition, liquid limit, plastic limit, organic content, water fused-salt, classification and physical and chemical index; strength parameters such as, cohesive, inner friction angle, compression coefficient, deformation modulus, permeability coefficient. The essential data of appurtenant building, drill data, comprehensive layer situation, soil test results, groundwater, in site testing, profile position and its literal record.

5) Hydrology data of Yangtze watercourses, typical cross section plots of watercourses, the course of water level recorded at control hydrological stations during historical cataclysms.

Historical and existing information of dangerous dike sections, measures of danger eliminating and strengthening (especially after the floods during 1998), prepared schemes for emergency treatment.

6) Information of historical cataclysms (such as in 1954, 1998) should include information of water and rain, situations of flood-prevention, engineering status, dangerous conditions, measures and experiences during flood.
7) Report forms of engineering dangers in flood period, report forms on investigation and statistic of disaster loss.
8) Report forms on construction investment, annual maintenance planning, investment grants, approximate estimate used to annual maintenance and statistical quantity sheet of annual maintained engineering.

8.2 Hydraulic data

The loading side of the reliability function at each location along the flood defence system is determined by the local hydraulic boundary conditions, i.e. the local water levels and the local wave conditions (significant wave height and wave period).
The local maximum water levels at Anqing hydrological station are listed in Table 8.1. For this case study daily records from 1924 to 1998 were available.

Table 8.1 Yearly maximum water level in Anqing hydrological station

Rank	Water level	Year	frequency	Cumulative frequency
1	18.74	1954	1	0.014493
2	18.5	1998	2	0.028986
3	18.05	1999	3	0.043478
4	17.95	1983	4	0.057971
5	17.89	1995	5	0.072464
6	17.55	1996	6	0.086957
7	17.28	1949	7	0.101449
8	17.08	1973	8	0.115942
9	17.07	1931	9	0.130435
10	16.98	1977	10	0.144928
11	16.95	1980	11	0.15942
12	16.88	1969	12	0.173913
13	16.79	1948	13	0.188406
14	16.73	1992	14	0.202899
15	16.65	1962	15	0.217391
16	16.58	1968	16	0.231884
17	16.52	1991	17	0.246377
18	16.43	1988	18	0.26087
19	16.4	1993	19	0.275362
20	16.36	1974	20	0.289855
21	16.34	1937	21	0.304348
22	16.32	1976	22	0.318841
23	16.31	1989	23	0.333333
24	16.19	1955	24	0.347826
25	16.18	1970	25	0.362319
26	16.07	1964	26	0.376812
27	16.07	1990	26	0.376812
28	16.04	1994	28	0.405797
29	16.03	1933	29	0.42029
30	16.02	1982	30	0.434783
31	15.94	1935	31	0.449275
32	15.91	1924	32	0.463768

Rank	Water level	Year	frequency	Cumulative frequency
33	15.91	1975	32	0.463768
34	15.86	1952	34	0.492754
35	15.85	1997	35	0.507246
36	15.61	1926	36	0.521739
37	15.45	1932	37	0.536232
38	15.43	1967	38	0.550725
39	15.4	1987	39	0.565217
40	15.33	1946	40	0.57971
41	15.32	1956	41	0.594203
42	15.18	1984	42	0.608696
43	15.07	1958	43	0.623188
44	14.91	1957	44	0.637681
45	14.9	1927	45	0.652174
46	14.9	1930	45	0.652174
47	14.9	1950	45	0.652174
48	14.87	1959	48	0.695652
49	14.84	1947	49	0.710145
50	14.78	1966	50	0.724638
51	14.71	1979	51	0.73913
52	14.66	1971	52	0.753623
53	14.66	1986	52	0.753623
54	14.57	1981	54	0.782609
55	14.33	1965	55	0.797101
56	14.29	1936	56	0.811594
57	14.2	1929	57	0.826087
58	14.09	1951	58	0.84058
59	13.99	1985	59	0.855072
60	13.91	1953	60	0.869565
61	13.84	1961	61	0.884058
62	13.81	1978	62	0.898551
63	13.72	1934	63	0.913043
64	13.47	1960	64	0.927536
65	13.14	1963	65	0.942029
66	12.71	1925	66	0.956522
67	12.68	1972	67	0.971014
68	12.4	1928	68	0.985507

Figure 8.1 gives the statistic characteristic of yearly maximum water levels by using the Bestfit software. It can be obtained that the water level confirm to normal distribution, with the mean value 15.59 and standard deviation 1.51.

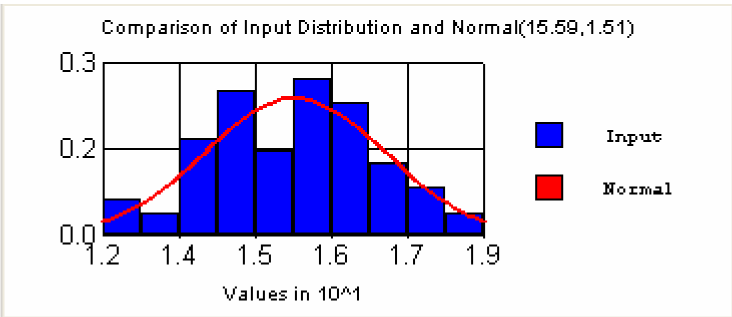


Figure 8.1 Statistic distribution of flood water level according the record at Anqing gauge station

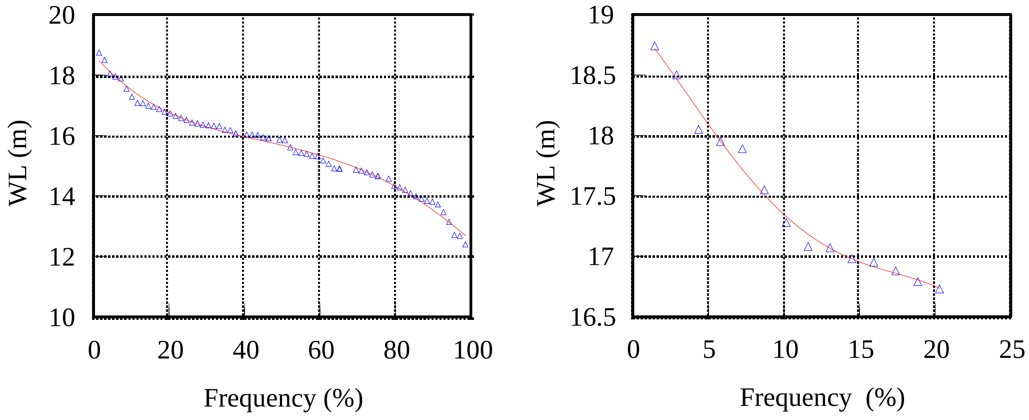


Figure 8.2 Fitting of the frequency curve of flood water levels

Figure 8.2 (a) shows the relation between cumulative frequency and yearly maximum water levels, and then, the fitted polynomial equation to compute curve trends using a least squares method can gain as the following:

$$y = 18.6748 - 14.227x + 28.669x^2 - 28.2219x^3 + 7.63999x^4 \tag{8-1}$$

For the small frequency case, referring to Figure 8.2 (b), another fitted polynomial equation (8-2) employed to fit the curve.

$$y = 18.9491 - 0.146054x - 0.0101275x^2 + 0.00114167 * x^3 - 2.7244E^{-5} * x^4 \tag{8-2}$$

Consequently, occurrence frequency and return period for each water level on Anqing dike, each level from the crest every 0.5m interval, are listed in Table 8.2. The curve is shown in Figure 8.3.

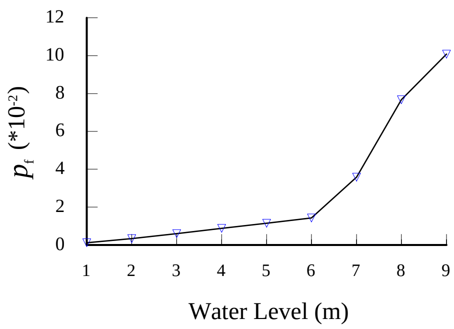


Figure 8.3 Occurrence frequency for each water level in this study

General speaking, the current design approach is the probability of exceedance or return period follows some specified values, such as 5, 10, 20, 50, 100, 200. Therefore, the design water levels corresponding to different design frequencies can be obtained by linear interpolation. In the same way, the design flood water level corresponding to different specified design frequencies are listed in Table 8.3.

Table 8.2 Occurrence frequency for each water level

Rank	WL	Frequency	Return Period (ys)
1	21.19	0.001094	914
2	20.79	0.003281	305
3	20.29	0.006016	166
4	19.79	0.00875	114
5	19.29	0.011485	87
6	18.79	0.014219	70
7	18.29	0.035749	28
8	17.79	0.076726	13
9	17.29	0.100912	10

Table 8.3 Design flood water level corresponding to different design frequencies

Design standard	Return period	5	10	20	50	100	200
	Frequencies	20	10	5	2	1	0.5
Design water level		16.36	16.98	17.89	18.5	18.74	18.89

Table 8.4 listed the basic variables concerned with overtopping for each dike section, such as crest elevation, dike height, design water level, fetch length, average water depth, and wave length.

Table 8.4 Basic variables concerned with overtopping

Section No.	Crest elevation	Inner WL	Dike Height	Design WL	Water height	Fetch length	Average W depth	Wave Length
5687	21.29	14.5	6.69	19.29	4.79	4790	12	8.12407
8687	21.29	14.5	6.69	19.29	4.79	4790	12	9.19916
9387	21.18	14	7.08	19.18	5.18	5180	12	9.76672
11287	21.18	14	7.08	19.18	5.18	5180	12	11.3782
12737	21.08	14.13	6.85	19.08	4.95	4950	12	12.3259
12942	21.08	14.13	6.85	19.08	4.95	4950	12	12.6985
13737	21.07	14.8	6.17	19.07	4.27	4270	12	10.9319
14187	21.06	14.5	6.46	19.06	4.56	4560	12	9.19916
14587	21.06	14.5	6.46	19.06	4.56	4560	12	9.19916
14937	21.04	13.54	7.4	19.04	5.5	5500	14	9.23668
15187	21.04	13.54	7.4	19.04	5.5	5500	15	9.25112
15287	21.03	14	6.93	19.03	5.03	5030	15	9.25112
15787	21.01	12.87	8.04	19.01	6.14	6140	15	9.04702
16337	21.03	14	6.93	19.03	5.03	5030	15	9.25112
16987	20.98	14	6.88	18.98	4.98	4980	15	9.44921
17137	20.98	14	6.88	18.98	4.98	4980	15	9.25112
18187	20.96	14.7	6.16	18.96	4.26	4260	15	9.25112
18647	20.96	14.7	6.16	18.96	4.26	4260	15	9.25112
18837	20.95	14	6.85	18.95	4.95	4950	15	9.64173
18987	20.92	14	6.82	18.92	4.92	4920	15	9.82907

Section No.	Crest elevation	Inner WL	Dike Height	Design WL	Water height	Fetch length	Average W depth	Wave Length
19757	20.92	14	6.82	18.92	4.92	4920	15	10.0116
20287	20.88	14	6.78	18.88	4.88	4880	15	10.1895
20487	20.88	14	6.78	18.88	4.88	4880	15	10.3632
20904	20.88	14	6.78	18.88	4.88	4880	15	10.5328
21087	20.88	14	6.78	18.88	4.88	4880	15	10.0116
21292	20.88	14	6.78	18.88	4.88	4880	15	10.6987
21487	20.85	14	6.75	18.85	4.85	4850	15	10.6987
21682	20.85	14	6.75	18.85	4.85	4850	15	11.0198
22287	20.8	13.8	6.9	18.8	5	5000	15	12.4531
22687	20.77	13.8	6.87	18.77	4.97	4970	15	11.7687
22857	20.77	13.8	6.87	18.77	4.97	4970	15	11.328
23757	20.69	13.8	6.79	18.69	4.89	4890	15	11.0198
23987	20.69	13.8	6.79	18.69	4.89	4890	15	9.25112
24037	20.69	13.9	6.69	18.69	4.79	4790	15	9.25112
24487	20.78	12.9	7.78	18.78	5.88	5880	15	9.25112
24687	20.78	12.9	7.78	18.78	5.88	5880	15	9.44921
29687	20.64	12.9	7.64	18.64	5.74	5740	14	9.23668
30237	20.64	12.9	7.64	18.64	5.74	5740	13	9.41562
30387	20.64	12.7	7.84	18.64	5.94	5940	12	9.58289
31787	20.58	12.7	7.78	18.58	5.88	5880	12	9.76672
33137	20.58	12.7	7.78	18.58	5.88	5880	12	11.5211
34687	20.51	12.7	7.71	18.51	5.81	5810	12	12.9374
36187	20.51	12.7	7.71	18.51	5.81	5810	12	13.5045
37187	20.44	12.7	7.64	18.44	5.74	5740	12	10.9319
38387	20.44	12.7	7.64	18.44	5.74	5740	15	9.25112
40439	20.31	8	12.21	18.31	10.31	10310	15	8.1597
40687	20.31	8	12.21	18.31	10.31	10310	15	6.82659
40709	20.29	9.7	10.49	18.29	8.59	8590	15	6.82659
41644	20.26	9.7	10.49	18.26	8.56	8560	16	6.83152
M0000	19.3	10.9	8.3	18.3	7.4	7400	0	0
M0948	19.3	10	9.2	18.3	8.3	8300	0	0
M1748	19.3	11	8.2	18.3	7.3	7300	0	0
M2448	19.3	10	9.2	18.3	8.3	8300	0	0
M3548	19.3	10.5	8.7	18.3	7.8	7800	0	0
M5248	19.3	10.5	8.7	18.3	7.8	7800	0	0

Note: Wind velocity is 18 m/s^2, gravity is 9.81m/s^2, Kd=0.85, Kv=1.1.

8.3 Geotechnical boundary conditions

8.3.1 Geology Condition

(1) Topography and geomorphy

In these areas, the Yangtze River changes a direction of a stream water to the north, and flow along the Zongyang section then to east direction. A breaching failure has been occurred recorded in historical literatures. These areas are alluvial plain, and there is a surface relief from high ground in the northwest to the lower area in the southeast. Elevations in the area vary from 11m to 17m. Low-relief terrain and hilly ground are located at the suburb of Anqing and Zongyang city. There are Dalong, Muqi, and Baihefeng hill, and the high-point is Dalong which height is 699m.

Lakes, pool or polder, and cannel are distributed in both sides of the major dike. Pogang and Lianhua lake are the larger lake. There are 214 polders distributed within 300m off the toe. There are borrow pit and old channel. Outer side of Meilin section is Chang stream, and inner side is Huashan Lake and Pogang Lake, 29 polders distributed nearby the toe.

(2) Lithologic of the stratum layer

The stratum layer of Anqing and Guangjiwei river levee is Quaternary system alluvial or depositional layer, which is composed by silt clay, heavy clay soil, silt loamy soil, loamy soil, sandy loamy soil, silt fine sand, and sandy gravel, depth of 35.0～50.0m. The elevation of relative impervious cretaceous sandstone plate is -19.4～-39.44m. The dike body of Meilin section is composed by silt clay and slit loamy soil. The dike foundation is composed by silt clay, heavy slit loam, muddy soil, fine sandy lenticle.

(3) Geological structure and earthquake

According to plot plan on earthquake intensity in China, published in 1990, the earthquake intensity of this area is VI.

(4) Hydrological geology

The ground water is storied in the soil layers in the form of pore water. It can be divided into two types: phreatic water and artesian water. pH of the river water is 7.6～7.8, and pH of the ground water is 7.2. They are weak alkaline water but not corrosive to concrete.

(5) Exogenic geological process

Bank collapse is a major exogenic geological process, related with the river-bed variation and erosion or scouring.

8.3.2 Major engineering geology problem

It includes: seepage instability, sliding, bank stabilization, non-uniform settlement.

(1) Seepage instability

Dangerous emergencies have been occurred in many sections of Anqing Guangjiwei and Meilin dike during the flood season and the serve flooding in 1998, as listed in Table 8.5. The main failure modes are leakage, sand boils and piping of dike body and foundation. The one reason is that high water head, uneven rolling, big void ratio of filling. The other reason is that rock character and geological structure of the foundation, breadth of foreland, and maximum depth of scour.

Table 8.5 Some dangerous and measures of Anqing dike in 1998

No	Section Name	No. Section	Modes	Time	Water Level	Description	Measures
1	GuangJiwei	17+100	Piping	Aug. 4	18.46	Piping occurred in three positions of a polder, a distance of 90m off the inner toe of the dike. The area of the polder is about 5 acre.	Conducting seepage and drainage at land side
2	GuangJiwei	27+416	Leakage	Jun. 28	17.43	Sand boiling occurred in No. 2 section of culvert box, especially at the top of the dilatation joint, around 4-5 m3/hour. On July. 4th some clear water drop out at the joint position between the culvert box and abutment wall	Observation
3	GuangJiwei	30+000	Piping	Jul. 28	18.33	Clear water drop out from the drainage ditch corner which located at oil storage facilities of a petrochemical company with 70m distance off the toe, 7cm aperture.	Piping-berm with permeable filter. Technical engineer on duty.
4	GuangJiwei	39+050	Piping	Aug. 3	18.49	Piping occurred in the polder with a distance of 260m off the toe, the diameter is 15cm. The four sand rings formed an entire boiling area. Cracking occurred in the basement of a two-storied house with a distance of 50m off the toe. The maximum width of slit is 1 cm, and the settlement of the house amount to 45cm.	Piping-berm has been constructed by using the sand or gravel. The polder has been filled with sand.

(2) Sliding

The composition of bank slope soil in most of these sections is alluvial-proluvial soil of Quaternary dualistic structure, weak erosion-resisting characteristics. Bank collapse and shoreline fallback can be occurred in part of the sections due to the scour and side erosion. In this case, the slope stability is one of the principal geological problems.

(3) Sliding of deep foundation

A soft muddy clay exists in the foundation of some individual section of Guangjiwei and Meilin, and the sliding of deep foundation is a potential danger.

(4) Non-uniform deformation

The foundation of anti-flood wall is made by loose natural soil, its distribution is non-homogeneous. A non-uniform settlement will be result from the large loading.

The supporting layer of the foundation of some culverts and sluice is lacustrine deposit muddy fine clay or muddy silt loamy soil. Their mechanical strength is lower, big void ratio, high moisture content. In case of the existence of a more or less settlement deformation for these structures, the safety problem will be presented at a certain extent.

8.3.3 Segmentation of engineering geology and evaluation

The segmentation of engineering geology for whole section is based on the geological structure of the ground and the primary geology problem, as shown in Table 8.6. Some different grades can be classified, very good (A), good (B), not good (C), and bad (D), by considering to the breadth of the foreland, polder position, scour pit, and historical dangerous emergencies. In general, the foreland can be divided by the breadth, much wider foreland in 200m, wider foreland between 100m-200m, narrow foreland between 50m-100m, no foreland less than 50m.

Table 8.6 Geological structure type of dike foundation

Foundation		Section No.	Length (m)	Description
Type	Sub type			
homogenous Ⅰ	Ⅰ1	40+439~41+939	1500	Clay layer, depth lager than 20m, sand or sand loamy lenticle soil in some sections
		41+939~42+479	1150	Upland retaining water
		42+579~42+759		
		43+059~43+359		
		43+559~43+689		
		42+479~42+579	600	Clay layer, depth lager than 12m, located on the rock basement
		42+759~43+059		
		43+359~43+559		
		M4+748~M5+048	300	Natural upland retaining water
		M0+000~M4+748	5996	Clay layer, depth lager than 12m, located on the rock basement, sand or sand loamy lenticle soil in some sections
		M5+048~M6+296		
	Ⅰ2	18+300~18+837	537	Clay covering is very thin, less than 2m
Double layer Ⅱ	Ⅱ1	18+837~23+427	4590	Top layer is silt clay, silt sandy loamy soil, depth 5-5m, underneath deep sand layer
	Ⅱ2	0+000~0+960	1785	Top layer is silt clay,depth larger than 15m, and then is permeable fine sand layer, depth 9m, underneath rockstone
		1+277~2+102		
		5+687~11+367	5680	Depth 15m,top layer is silt clay, inclusion of sand loamy soil with lenticle, underneath deep sand layer
		15+987~18+300	2313	Depth of the covering layer 5~10m, silt clay, inclusion of sand loamy soil with lenticle, underneath deep sand layer
		23+427~34+412	13737	Top layer is clay layer, depth 6~16m,and then sand loamy soil, underneath deep sand layer
		37+187~37+687		
		38+187~40+439		
		34+412~37+187	3275	Top layer is clay, depth larger than 15m,underneath deep sand layer
		37+687~38+187		

Multi layer III	III	0+960~1+277	3902	Top layer is natural filling soil, depth 1.5~8.0m, and then silt loamy soil, inclusion of fine sand layer、sand loamy soil, depth 4~10m, then silt clay with gravel, silt loamy soil with gravel, depth 6 ~12m, underneath rockstone
		2+102~3+626		
		3+626~4+726		
		4+726~5+687		
		11+367~15+987	4620	Top layer is fine sand, sand loamy soil, depth 0.5~2.0m,and then silt loamy soil, silt clay, depth 4m, underneath deep sand layer

It can be concluded from above table that most dike foundations consist of sandy soil, or have a "two-phase structure" of clay and sandy soil. According to the related literatures, segmentations of engineering geology on these sections are listed in Table 8.7. Grade A has 1,682km, which accounts for 3.37 percent of the length whole section. Grade B has 29.744km, which accounts for 59.51 percent of the length of whole section. Grade C has 3.941km, which accounts for 7.88 percent of whole section. Grade D has 14.618km, which accounts for 29.24 percent of whole section.

Table 8.7 Evaluation of geology on Anqing/ Guangjiwei/ Meilinge dike section

Type	No. Section	Length	(km)	%	Description of Geological condition of basement	Evaluation of geological condition
A	4+726～5+687	0.961	1.68	3.4	Pleistocene series clay, top layer is raw natural soil with thickness of 4m, next layer is silt clay or silt clay loam, without sand layer	Bedrock or Q_2 clay, high strength, strong anti-scour and anti-seepage capability, no potential dangerous failure in 1998.
	42+18～42+477	0.288			Red sand stone, top layer is eluvial soil or raw natural soil with thickness of less than 1m	
	43+237～43+440	0.203				
	M4+748～M4+978	0.23			Granite, top layer is eluvial soil or raw natural soil with thickness of less than 1m	
B	0+000～0+960	0.96	29.7	59.5	Top layer is silt clay with the thickness of almost 20m, and then fine sand layer with the thickness of 12m, elevation of the impermeable plate-21m	Weak permeable Quaternary Holocene series silty clay, silt clay loam. Sand layer buried in very deep, the elevation of the impermeable plate is high. The breadth of the foreland is quite wide for this dike section. The breadth of the foreland is very small in section 35+750 to 36+990. Due to the scour effect of the water flow, the sliding stability of bank slope sometime is serious. In 1998, major potential dangerous of the dike is sand boils at the flood period, leakage occurred in some sections. And some reinforcement measures has been taken. A sand boil has occurred in 1999 in the Meilinge dike.
	1+277～2+102	0.825			Top layer is silt clay with the thickness of almost 10m, and then sandy loam soil with 15m, and silt layer with lenticular, elevation of the impermeable plate is 4.78～2.40m	
	5+687～11+287	5.60			Top layer is silt clay, silt clay loam of 15m, and then sand and gravel layer, elevation of the impermeable plate is -10～-20m	
	23+387～40+439	17.05			Top layer is silt clay, silt clay loam of 20m, some lenticular muddy clay, and then sandy, sand and gravel layer of 17.40m, elevation of the impermeable plate is -21.60m.	
	40+439～42+189	1.75			Top layer is silt clay, silt clay loam (with muddy clay) of 20m, and then sandy, sand and gravel layer of 10m, elevation of the impermeable plate is -20m.	
	42+477～43+237	0.760			lacustrinesilt clay loam, silt clay of 10m, muddy clay of 13m, without sandy layer	
	43+440～43+689	0.249			10m thick of lacustrine silt clay loam, silt clay, without sandy layer	
	M0+000～M2+548	2.548			Top layer is made up of silt clay loam, silt clay, clay, 21.40～24.10m thick, and then slit fine sand layer with 3.10~5.10m thick	

Type	No. Section	Length (km)		%	Description of Geological condition of basement	Evaluation of geological condition
C	0+960～1+277	0.317	3.94	7.9	Top layer is natural fill soil with the thickness of 5m, and then silt clay loam	Top layer is natural fill soil in 0+750～1+277, 2+102～4+726, porosity is bigger of the soil, high permeable, high permeable sand and gravel layer, inclusion with soft mud clay, be prone to asymmetry settlement. According the historical record of the dangerous is that flood wall was cracked in1991 and clinking in 1983.
	2+102～3+626	1.524			Top layer is natural fill soil with the thickness of 4-6m, and then silt clay loam layer around 0.2~8.20m. Sand and gravel layer with the thickness of about 2.40~18m, elevation of the impermeable plate for red sandstone is 6.68～6.12m,	
	3+626～4+726	1.100			Top layer is 2.4~7.9m in thickness of natural fill soil, and then is andy soil with inclusion of little lenticular silt clay loam, lenticular silt clay loam with gravel, the thickness of 4.0~8.1m, the thickness of sand and gravel layer is 4.2~9.3m, elevation of the impermeable plate is -3.10m,	
	M2+548～M3+548	1.000			Top later is composed by tawny, graysilt clay loam, silt clay, 7.30～19.80m thick, the elevation of the impermeable plate is -15m.	
D	11+287～16+072	4.785	14.6	29.2	8～15m thick of weak permeable layer, with sandy loam soil and silt layer, inclusion of fine sand thin layer with bedding plane, and then the thickness of 50m thick of high permeable sand layer, sand and gravel layer.	The breadth of the foreland is about 60～80m, sliding stability problem is existed. A lot of polder are located in the inner side 100～200m off the dike body. Top layer is thin silt clay, silt clay loam, and the deep slit fine sand, being prone to seepage instability. More than 70 relief well have been installed in this section, but the effect is not very obvious in some sections. During 1998 flood, sand boils had occurred in 5 positions, and sliding had occurred in 3 positions. A weak geological condition is for this kind of dike section, some treatment measures of foundation should be taken.
	16+072～18+837	2.765			10m thick of weak permeable layer, the overlying stratum composed by silt clay loam, silt clay, inclusion with fine sand thin layer, and then the thickness of 50m thick of high permeable sand layer, sand and gravel layer.	
	18+837～23+387	4.550			2～4m thick of weak permeable layer, sand and gravel layer is very deep.	
	M3+548～M4+748	1.200			11.30～18.10m thick of tawny, graysilt clay loam, silt clay, muddy clay, 1.60～4.50m muddy clay thickness.	
	M4+978～M6+296	1.318			Tawny, graysilt clay loam, silt clay, muddy clay, 2.0～8.0m muddy clay thickness.	

8.3.4 Geotechnical parameters

A rather wider scale exploration of dike has been carried out during the strengthening, especially since 1998 flood waters, but these data is usually on the shelf. How to tackle and recycle these valuable resources and to decrease the expenses paid for repeating exploration, which is the key issue for the specialists of geology engineering and geotechnical engineering. The development of technique of data base is the newest means to solve this problem. In this case, the geological exploration likes that a doctor diagnoses a patient. After examining the pathogeny, then suit the remedy to the case. This is so called have a definite object in view. The creation of database tables likes the collecting and sorting prescription. When the similar disease comes up, then you can make up a prescription acoording to codex.

Based on the user requirement analysis, all kinds of data tables will be established after trimming and filtrating and condensation and sorting the collected data. A database server will manage these tables

(1) Dataset of geotechnical parameters

According to the action of engineering design, soil indicator obtained by test should be divided into general characteristic parameter and main parameter. The former, such as volume weight, nature moisture content, specific gravity, granulometric composition, liquid limit, plastic limit, organic content, water fused-salt. It is important to soil classification and naming and physical chemistry clarification. The latter, such as cohesive, inner friction angle, compression coefficient, deformation modulus, permeability coefficient. It is directly useful to define the strength and deformation and stability.

The sorting and statistic of soil parameters in the same region are helpful to evaluate the variability of correlated parameters and distribution types of probability and the sensitivity of the numerical analysis.

(2) Dataset of geological exploration

The following aspects should be comprehensively exposed: 1) soil characteristic of the dike body and basement. 2) topography of the dike within definite region (such as berm and pond) and historical construction. 3) shoreline and river evolution trend and integrity of revetment and shore protection for major dike section. 5) the hidden defects and historical dangers and tackle measures.

Table 8.8 listed the range of geotechnical parameters for different soil types used in this study. It should be pointed that collecting geotechnical data appeared to be the most difficult task. Geotechnical surveys are very expensive and are therefore scarce in many departments. More detailed data should be collected in the near future.

Table 8.8 Geotechnical parameters used in this study

Symbal	Soil type	Moisture content	Dry density	Void ratio	Saturation degree	Plasticity index	Compression coefficient	Cohesive	Inner friction angle	Bearing capacity	Seepage coefficient
		w	ρ_d	e	S_r	I_p	a_{1-2}	c	ϕ	f_k	k
		%	g/cm³		%		MPa⁻¹	kPa	0	kPa	cm/s
rQ	natural fill soil	25.89~31.5	1.42~1.57	0.74~0.89	94~100	15.0~17.0	0.29~0.36	22.4~34.5	12.6~18.8		7.04E-4 ~6.14E-5
	natural fill soil										4.96E-5 ~5.30E-2
rQ	natural fill soil	26.03~31.6	1.43~1.52	0.75~0.88	90~100		0.41~0.64			120~140	5.17E-5 ~3.69E-4
a1Q4	Top sandy loam soil	25~33.1	1.34~1.61	0.92~1.04	87~100		0.15~0.29	1~18.7	30~33.5	140	1×10-3
	Silt clay	29.6~37.6	1.22~1.48	0.86~1.01	97~100	12.4~18.6	0.37~0.45	15.6~28.3	12.5~21.6	130~160	2.59E-6 ~1.96E-7
	Silt clay/ loam soil	32.8~38.6	1.31~1.45	0.89~1.08	97~100	11.4~16.5	0.39~0.55	11.5~15.9	11.5~20.5	150	1.29E-5 ~6.31E-6
	Muddy clay	23.8~77.6	1.32~1.52	0.76~2.21	74~100	8.5~40.5	0.2~3.03	6~30	4.3~18.8	60~80	1.01E-8 ~7.25E-6
	Sandyloam soil	30.4~33.8	1.40~1.43	0.85~0.90	100		0.40~0.81	8~14.5	8.5~16	140	8.45E-5 ~1.84E-4
	Silt fine sand		1.37~1.48	0.79	100					150	6.16E-3 ~2.04E-4
Q1	Clay	17.1~29.6	1.71	0.62			0.16~0.34	32	14.6	260~300	5.85E-6 ~1.37E-8

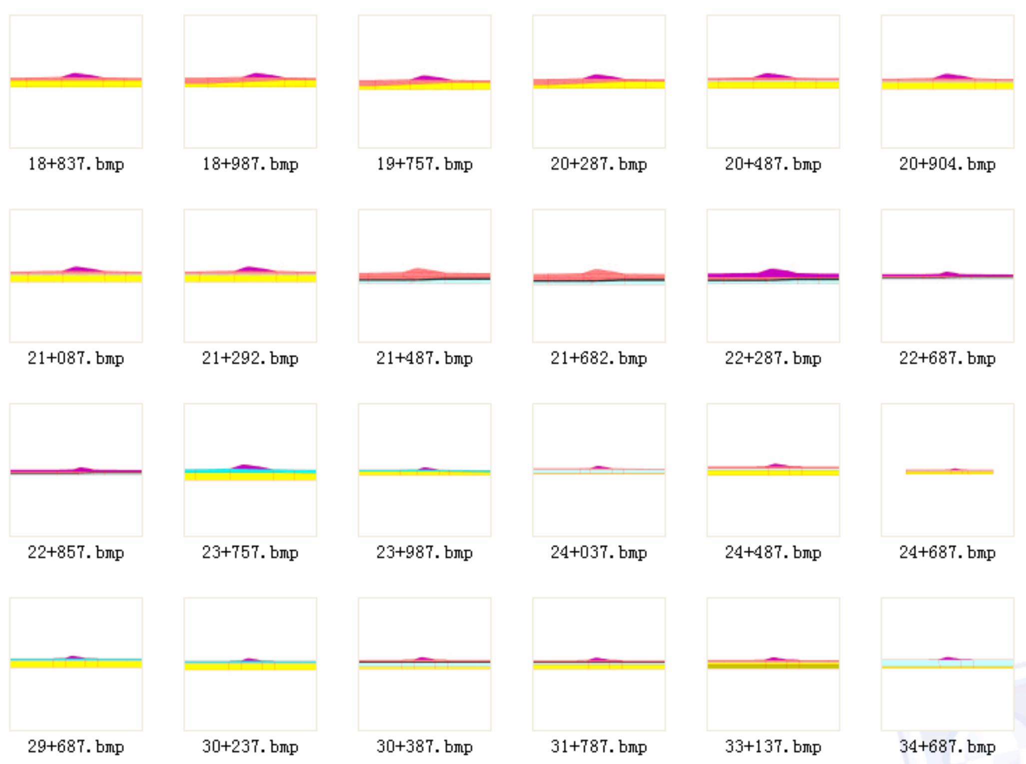
8.5 Types of revetment on the outside slope of the flood defence

Three types of revetment of the outside slope of the flood defences occur in the Anqing flood defences: 1) Placed stone revetment is present at two locations. In both situations the maintenance is in a moderate condition. 2) Grass occurs along large stretches of the flood defences. The overall condition of the grass is good. 3) Rock armour revetment is applied along large stretches of the flood defences. The overall condition of the rock armour is good. The sizes of the stones vary between 0.75 and 1.5 meter.

8.6 Dyke characteristics

Dyke profiles were obtained from some strengthening design reports for these dikes. A numerical representation of the dyke profiles in the form of horizontal and vertical distances has been made from the reports and input to dataset. In total 55 dyke profiles were collected at intervals of about 1 km. Figure 8.4 shows dyke profiles drew by SADSS.


05+687.bmp
08+687.bmp
09+387.bmp
11+287.bmp
12+737.bmp
12+942.bmp
13+737.bmp
14+187.bmp
14+587.bmp
14+937.bmp
15+187.bmp
15+287.bmp
15+787.bmp
16+337.bmp
16+987.bmp
17+137.bmp
18+187.bmp
18+647.bmp
18+837.bmp
18+987.bmp
19+757.bmp
20+287.bmp
20+487.bmp
20+904.bmp


18+837.bmp
18+987.bmp
19+757.bmp
20+287.bmp
20+487.bmp
20+904.bmp
21+087.bmp
21+292.bmp
21+487.bmp
21+682.bmp
22+287.bmp
22+687.bmp
22+857.bmp
23+757.bmp
23+987.bmp
24+037.bmp
24+487.bmp
24+687.bmp
29+687.bmp
30+237.bmp
30+387.bmp
31+787.bmp
33+137.bmp
34+687.bmp

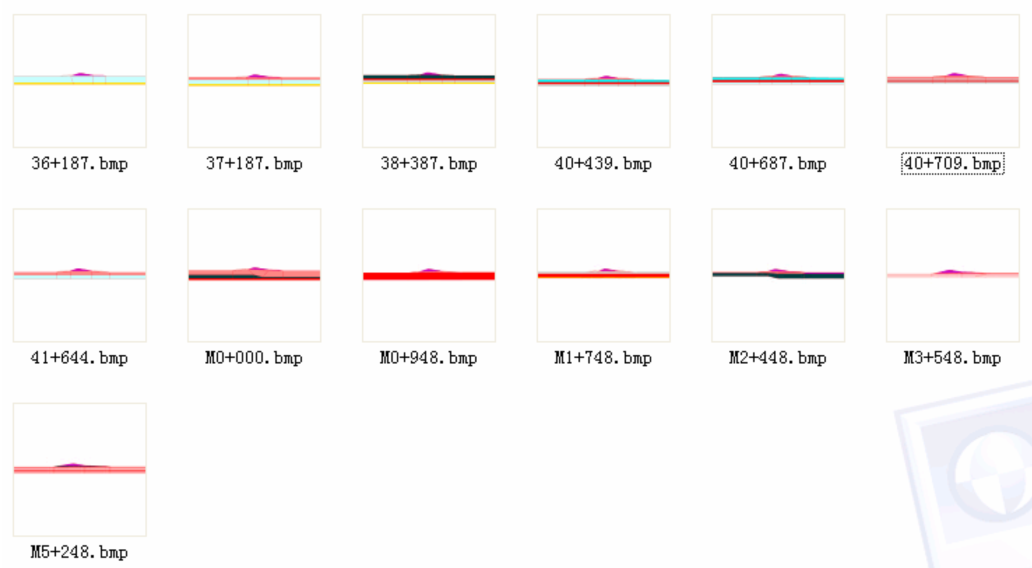


Figure 8.4 Dyke profiles of each cross section of Anqing dike

8.7 Meteorological data

8.8 Economics

Chapter 9 Probabilities of Failure of Anqing Dike

[Abstract]: Based on theory of soil mechanics and reliability theory above-mentioned, some practical limit state functions regarding to the failure mechanisms, such as overtopping and piping and sliding of dikes, have been proposed. Then, these equations are incorporated in the probabilistic analyses of Anqing dike ring. A risk analysis has been carried out for the area. The risk analysis for the dyke sections has taken three failure mechanisms into consideration, namely overtopping, piping and sliding of the inner slope. For the sluice only the failure mechanism ‘not closing due to human failure’ is taken into account.

9.1 Introduction

9.1.1 Anqing flood defence system’s boundaries

The Anqing flood defence system’s boundaries is shown in Figure 9.1. Due to the left side and upper side of this map are highland (highground) or mountains. The WS+18m line represents the high grounds which do not contribute to the system’s probability of flooding.

Under mean high water circumstances the area is at risk. If breach occurs, the water will reach the protected area.

Figure 9.1 Anqing flood defence system’s boundaries

9.1.2 Components of the Anqing' flood defence system

The seven main components, or defence types, of the Anqing' flood defence system are shown in Figure 9.1, and some other structure types that occur in the flood defence system but are not included in this study. Thereby the five types embankments and two sluices have been listed Table 9.1.

In order to provide a first impression of the shape of the Anqing' flood defence system only the homogeneous embankment is discussed, i.e., the five type embankments are assumed to an identical one without additional structures. Another limitation is that there is no information available with regard to where these sluices till now. Other parts of the water defence system are considered infinitely safe compared with these elements.

Table 9.1 Description of the main flood defence components

Component	No. Section		Length (m)	Type of dike	Percent (%)
	Initial position	Terminal			
A	000+000	05+687	5,687	Homogeneous dike of steal-concrete anti-flood wall with foundation treatment	11.26
B	05+687	14+500	8,813	Homogeneous dike with cement soil core	17.45
C	14+500	28+180	13,680	Homogeneous dike with piping berm or relief well	27.08
D	28+180	45+000	16,820	Homogeneous dike	33.31
E	M0+000	M5+000	5,500	Homogeneous dike	10.89
F	05+687			Xin jinjia Sluice	
G	33+201			Po ganghu Sluice	

The component is presented by pictures from a site visit as shown in Figure 9.2. An indication of the dimensions of cross section 05+687 is shown in Figure 9.3 and the coordinates of typical topography point are listed in Table 9.2.

Table 9.2 Topography point coordinate of section 05+687

Layer	Soil Type	1 (X/Y)	2 X/Y	3 X/Y	4 X/Y	5 X/Y	6 X/Y	7 X/Y	8 X/Y	9 X/Y	10 X/Y	11 X/Y	12 X/Y	13 X/Y
1	Natrual filling	0	260.18	271.55	279.05	279.05	289.05	290.55	298.05	304.05	316.50	346.50	350.00	500.00
		285.00	285.50	281.71	279.21	278.71	278.71	279.21	281.71	281.71	284.20	284.80	285.50	285.00
2	Medium sand	0	271.55	298.05	304.05	316.50	500.00							
		285.50	285.50	285.50	285.50	285.50	285.50							
3	Loamy soil	0	260.18	271.55	298.05	304.05	316.50	350.00	500.00					
		299.00	299.00	299.00	299.00	299.00	299.00	299.00	299.00					
4	Fine sand	0	260.18	271.55	298.05	304.05	316.50	350.00	500.00					
		321.00	321.00	321.00	321.00	321.00	321.00	321.00	321.00					

Figure 9.2 Photo of section 05+687

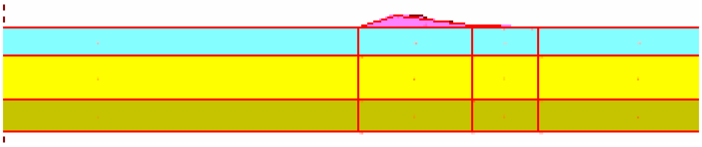


Figure 9.3 Indication of dimensions of cross section of section 05+687

9.1.3 Main failure modes

The deterministic method and reliability analysis methods for flood defences which are discussed in chapter 4 provide the possibility to calculate the probability of failure of the embankment.

In figure 6.2, a number of possible failure modes of an embankment without additional structures are given. However, this study focuses on failure modes that are incorporated in SADSS. In foregoing chapter these failure modes are listed and briefly introduced. These failure modes contribute according to experience significantly to the total system's probability of failure, according to some dangerous records mentioned-above. Besides that, for these failure modes (suitable) models are available to make reliability calculations.

For the sake of completeness the failure modes that are included in SADSS and thus subject to this study are given below: (1) Overtopping; (2) Instability of the inside slope;(3) Instability of the outer slope;(4) Uplifting and consequently piping.

Figure 9.4 shows the sketch of fault tree for the flood defence system.

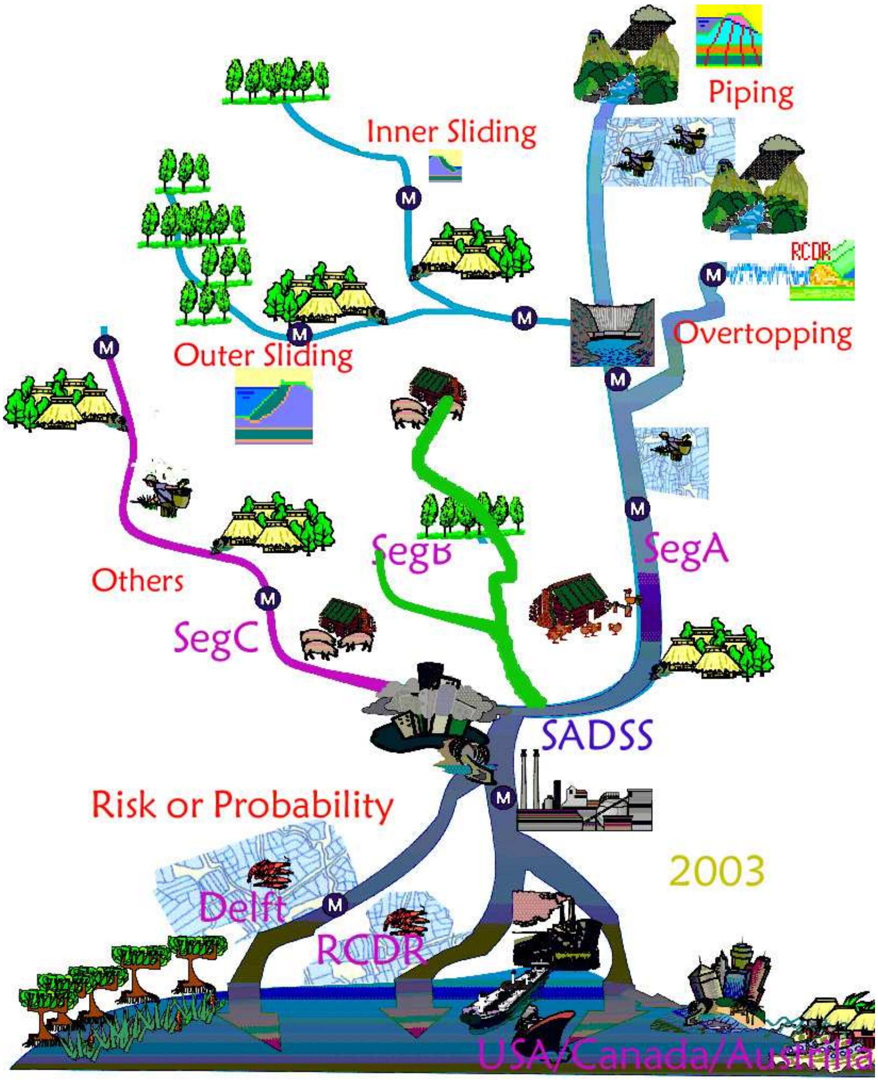


Figure 9.4 Sketch of fault tree of the dike

9.2 Overtopping

9.2.1 Mechanism

The mechanism of overflowing is applicable for dyke sections where wave attack is not important. In the upper river area waves are always small, typically between 0 m and 0.5 m. However, these small waves can still cause wave overtopping, resulting in a higher probability of failure than overflowing. In that case overtopping will always occur sooner than overflowing and therefore overflowing can be left out for the dyke sections where wave attack is important, as shown in Figure 9.5.

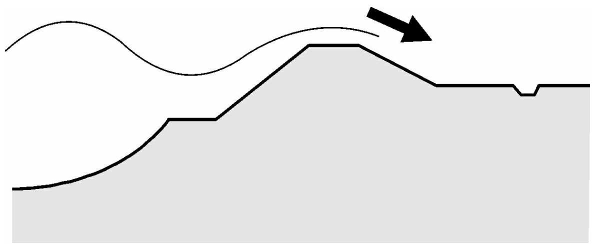


Figure 9.5 Mechanism of overtopping

9.2.2 Limit state function

The water level is the deterministic factor in the determination of the height of crest. The reliability function of overtopping is:

$$z_1 = h_0 - h_{\mathrm{w}} - h_s - e \tag{9}$$

Where, h_s is the swash height; e is the surge height.

There are many factors which have effects on the height of dike crest. The reserved settlement is set in design because of the consideration of construction precision and consolidation settlement after completion. Suppose h_0 is normally distributed, and the probability which the reserved settlement is more than 0.1m is less than 2%. Therefore, according to the expression of normally distribution, the following can be drawn:

$$\frac{h_{\mathrm{d}} - h_0}{\sigma_R} = 1.96 \tag{10}$$

where, h_{d} is the designing height of dike. Then formula (10) can be written as:

$$\sigma_{h_0} = 0.1/1.96 = 0.051 \tag{11}$$

The maximum height of flood level h_{w} is supposed to be exponential distribution with $\mu_{h_{\mathrm{w}}} = 8.34$m and $\sigma_{h_{\mathrm{w}}} = 0.9$ according to general experience. The swash height h_s is supposed to be normal distribution with $\delta_{h_s} = 0.69$.

The average wave height μ_h is recommended by Chinese Standard (1998):

$$\frac{g\mu_h}{V^2} = 0.13\mathrm{th}\left[0.7\left\{\frac{gh_{\mathrm{a}}}{V^2}\right\}^{0.7}\right] \times \mathrm{th}\left\{0.0018\left[\frac{\mathrm{g}F}{V^2}\right]^{0.45}\left[0.13\mathrm{th}\left[0.7\frac{gh_{\mathrm{a}}}{V^2}\right]^{0.7}\right]^{-1}\right\} \tag{12}$$

Where, μ_h is the average wave height, m; V is the calculating wind speed, the value of it is 18m/s; F is the length of wind field, the value of it is 2000m; h_{a} is the average water depth of the water area, the value of it is 10.5m.

Swash height is

$$\mu_{h_s} = \frac{K_\Delta K_V \sqrt{\mu_h \mu_\lambda}}{\sqrt{1+m^2}} \tag{13}$$

Where, $m = 2.5$; $K_\Delta = 0.75$-1.0, 0.85 can be used; $K_V = 1.0$-1.3, 1.1 can be used; μ_λ is the wave length.

$$\mu_\lambda = \frac{g\mu_T{}^2}{2\pi} \operatorname{th} \frac{2\pi h_a}{\mu_\lambda} \tag{14}$$

Where, μ_T is the average wave period, and $\mu_T = 4\sqrt{\mu_h}$.

Surge height is

$$e = \frac{KV^2F}{2gh_a}\cos\beta \approx \frac{KV^2F}{2gh_a} \tag{15}$$

Where, the angle of wind direction β is 0^0 ; K is the combined coefficient of friction resistance, and its value is 3.6×10^{-6} .

The following can be obtained: μ_{h_s} =0.638; $\sigma_{h_s} = \delta_{h_s} \cdot \mu_{h_s} = 0.69 \times 0.638 = 0.4395$.

Certain extra-height should be set in practical engineering according to the grade of dike, when the observation data in hydrograph analysis can not obtained. But it is not considered in this study.

9.2.3 Variables and their distributions

Basic variables concerned with overtopping have been listed in the other chapters. Table 9.3 listed the random variables regarding to the overtopping modeling for the entire dike section. All random variable have been taken as a normal distribution. In this table, the MV denotes mean value, and SD denotes standard deviation.

Table 9.3 Random variables of overtopping modeling

Section	Dike Height		Water Level		Swash height		Surge
No	MV	SD	MV	SD	MV	SD	Height
5687	21.29	0.00239	15.59	1.51	0.48	0.331	0.00743
8687	21.29	0.00239	15.59	1.51	0.544	0.375	0.0099
9387	21.18	0.0024	15.509	1.501	0.577	0.398	0.01139
11287	21.18	0.0024	15.509	1.501	0.673	0.464	0.01634
12737	21.08	0.00241	15.436	1.493	0.729	0.503	0.01981
12942	21.08	0.00241	15.436	1.493	0.751	0.518	0.0213
13737	21.07	0.00242	15.428	1.492	0.646	0.446	0.01486
14187	21.06	0.00242	15.421	1.491	0.544	0.375	0.0099
14587	21.06	0.00242	15.421	1.491	0.544	0.375	0.0099
14937	21.04	0.00242	15.406	1.49	0.546	0.377	0.00849
15187	21.04	0.00242	15.406	1.49	0.547	0.377	0.00792
15287	21.03	0.00242	15.399	1.489	0.547	0.377	0.00792
15787	21.01	0.00242	15.384	1.488	0.535	0.369	0.00753
16337	21.03	0.00242	15.399	1.489	0.547	0.377	0.00792
16987	20.98	0.00243	15.362	1.485	0.558	0.385	0.00832
17137	20.98	0.00243	15.362	1.485	0.547	0.377	0.00792
18187	20.96	0.00243	15.348	1.484	0.547	0.377	0.00792
18647	20.96	0.00243	15.348	1.484	0.547	0.377	0.00792
18837	20.95	0.00243	15.341	1.483	0.57	0.393	0.00871
18987	20.92	0.00243	15.319	1.481	0.581	0.401	0.00911
19757	20.92	0.00243	15.319	1.481	0.592	0.408	0.00951
20287	20.88	0.00244	15.289	1.477	0.602	0.415	0.0099
20487	20.88	0.00244	15.289	1.477	0.613	0.423	0.0103
20904	20.88	0.00244	15.289	1.477	0.623	0.429	0.0107
21087	20.88	0.00244	15.289	1.477	0.592	0.408	0.00951
21292	20.88	0.00244	15.289	1.477	0.632	0.436	0.01109
21487	20.85	0.00244	15.267	1.475	0.632	0.436	0.01109
21682	20.85	0.00244	15.267	1.475	0.651	0.449	0.01188
22287	20.8	0.00245	15.231	1.471	0.736	0.508	0.01585
22687	20.77	0.00245	15.209	1.469	0.696	0.48	0.01387
22857	20.77	0.00245	15.209	1.469	0.67	0.462	0.01268
23757	20.69	0.00246	15.15	1.463	0.651	0.449	0.01188
23987	20.69	0.00246	15.15	1.463	0.547	0.377	0.00792
24037	20.69	0.00246	15.15	1.463	0.547	0.377	0.00792
24487	20.78	0.00245	15.216	1.47	0.547	0.377	0.00792
24687	20.78	0.00245	15.216	1.47	0.558	0.385	0.00832
29687	20.64	0.00247	15.114	1.459	0.546	0.377	0.00849
30237	20.64	0.00247	15.114	1.459	0.557	0.384	0.0096
30387	20.64	0.00247	15.114	1.459	0.566	0.391	0.01089
31787	20.58	0.00247	15.07	1.454	0.577	0.398	0.01139
33137	20.58	0.00247	15.07	1.454	0.681	0.47	0.01684
34687	20.51	0.00248	15.018	1.448	0.765	0.528	0.02229
36187	20.51	0.00248	15.018	1.448	0.798	0.551	0.02477
37187	20.44	0.00249	14.967	1.443	0.646	0.446	0.01486
38387	20.44	0.00249	14.967	1.443	0.547	0.377	0.00792
40439	20.31	0.00251	14.872	1.433	0.482	0.333	0.00594
40687	20.31	0.00251	14.872	1.433	0.403	0.278	0.00396
40709	20.29	0.00251	14.857	1.431	0.403	0.278	0.00396

Section No	Dike Height MV	Dike Height SD	Water Level MV	Water Level SD	Swash height MV	Swash height SD	Surge Height
41644	20.26	0.00251	14.835	1.429	0.404	0.278	0.00371
M0000	19.3	0.00264	14.132	1.432	0.01	0.001	0
M0948	19.3	0.00264	14.132	1.432	0.01	0.001	0
M1748	19.3	0.00264	14.132	1.432	0.01	0.001	0
M2448	19.3	0.00264	14.132	1.432	0.01	0.001	0
M3548	19.3	0.00264	14.132	1.432	0.01	0.001	0
M5248	19.3	0.00264	14.132	1.432	0.01	0.001	0

9.3 Piping

9.3.1 Mechanism

The section has been describable in the forgoing Chapter.

9.3.2 Limit state function

Actually, where there is a water head difference, there is the seepage in the dike. With the rise of water level during flood period, the phreatic line is formed inside the dike and its position gradually rises up. At the same time, the seepage gradient in the dike and subsoil gradually increased. When the actual seepage gradient J is lager than the critical gradient of the subsoil J_c, seepage failure is occurred. In this case, the following equation can be used to evaluation the seepage stability:

$$R_2 = J - J_c \tag{9-4}$$

The actual gradient occurred in foundation and structure can be determined by using FEM calculation or electrical analogy method. For preliminary design or ordinary structures, it can also be determined by empirical method. A free water surface curve can be determined by the intersecting point between the dike body profile and the exit point or water level, the coordinate of intersecting point at different water levels can be gained by contraction or stretching. In the same way, the probability that seepage gradient J exceed critical gradient J_c at a certain water level is calculated by Monte Carlo method.

9.3.3 Variables and their distributions

According to theory and tests of soil mechanics, the critical seepage gradient of the soil depends on many factors, such as grain diameter, size grading, structure, porosity, bulk gravity and quality of construction. For the piping failure, the critical gradient depends greatly on the gradation distribution of the soil grains. For the soil with continuous gradation, the impervious gradient could be calculated by equation (9-5) (Liu, 1993):

$$J_c = 2.2(1-n)^2 (G_s - 1)\frac{d_5}{d_{20}} \tag{9-5}$$

To simplify the analysis, only the variability of the critical seepage gradient of the soil is considered. Normal distribution type of the critical seepage gradient is assumed, and which values have been listed in Table 9.4.

9.4 Sliding

9.4.1 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, as shown in Figure 9.6.

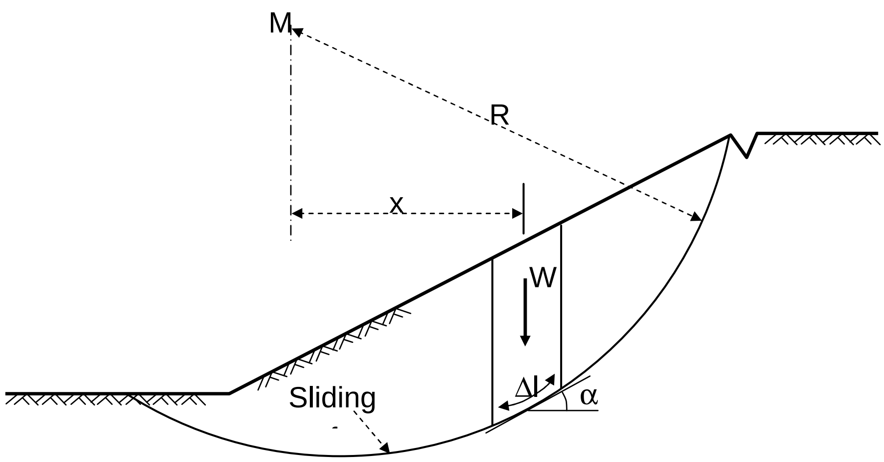


Figure 9.6 Mechanism of macro-instability

9.4.2 Limit state function

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:

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

In which

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

Where n = number of slices, W = the weight of each slice, b = the width of the slice (m), u = pore water pressure at the slip surface of the slice (kPa), c' = cohesion at the slip surface of the slice (kPa), $\phi_a^{'}$ = 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_0 = 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:

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

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 pore water 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:

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

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:

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

And the corresponding reliability index:

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

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:

$$P\{f\} = \max P\{F_i < 1\} \tag{4-24}$$

When we use Monte Carlo method to solved the reliability function, the main calculating procedures are as following:

(1) The minimum safety factor of slope sliding of dikes and the dangerous sliding surface are determined by adopting the simplified Bishop method;

(2) Producing the pseudo random numbers, random sampling for geotechnical parameters c 、 ϕ and γ is performed;

(3) Calculating the sliding moment M_s and moment against sliding M_r for given sliding circle;

(4) Counting the number that $M_s > M_r$ and the number is recorded as m;
(5) Repeating from steps (2) to (4) for n times until convergence has been attained;
(6) According to Bernoulli's theorem and characteristics of normally distributed random variable, $P\{f\} = m/n$ can be obtained, where $P\{f\} = \int_{Mr}^{\infty} f(M_s / H) \mathrm{d}M_s$ is the probability that sliding moment M_s exceed moment against sliding M_r at specified flood water level.

9.4.3 Variables and their distributions

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 computing model, the following assumptions regarding to 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. The most important variables, which dominate the uncertainty of the stability factor, are the pore water 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, Cohesion, Bulk gravity, Pore water pressure. The selected probability distribution for the random variables is of normal type. The input variables for the sliding calculation with section 05+687 and 08+687 are shown in Table 9.4.

Table 9.4 Input variables for the mechanism of sliding

Section	Layer	Name	Cohen	Angle	Gravity	Coeff	CritGrid	CoheSD	FricSD	GravSD	CoeffSD	CritSD
05+687	1	NF	10	16.5	16.22	0.00001	0.257455	6.277	2.093	0.0748	1.16E-05	4.72E-02
05+687	2	LS	10	16.5	16.22	0.00001	0.238955	8.275	4.291	0.0690	6.50E-05	4.38E-02
05+687	3	FS	10	7.5	18.07	0.00001	0.285727	1.665	1.037	0.0713	8.98E-04	5.24E-02
05+687	4	MS	10	7.5	18.07	0.00001	0.285727	2.695	2.985	0.0713	8.98E-04	5.24E-02
08+687	1	NF	10	16.5	16.22	0.00001	0.257455	6.277	2.093	0.0748	1.16E-05	4.72E-02
08+687	2	LS	10	16.5	16.22	0.00001	0.238955	8.275	4.291	0.0690	6.50E-05	4.38E-02
08+687	3	FS	15	7.5	18.07	0.004	0.285727	1.665	1.037	0.0713	8.98E-04	5.24E-02
08+687	4	MS	15	7.5	18.07	0.01	0.285727	2.695	2.985	0.0713	8.98E-04	5.24E-02

9.5 The combined probability of failure

The previous paragraphs showed the calculation methods for the probability of failure per mechanism. Overflowing and overtopping are almost identical failure mechanisms of which the highest probability has to be taken into account. Despite of the small difference the probability of overtopping is almost always higher than the probability of overflowing and thus the latter will be taken into account to calculate the total probability of failure of the dyke section.
All four failure mechanisms, overtopping, piping and sliding (inner or outer), may lead to a breach in the dyke section and consequently inundation.

The lower bound of the equation above can be described as the situation where all four failure mechanisms are completely correlated. The upper bound is the situation where all four failure mechanisms are completely independent. In reality the probability of failure of the dyke section lies somewhere between the lower bound and the upper bound.
Detailed expressions and computed results will be discussed in the next chapter.

9.6 Selection of flooding scenarios and flood simulation

9.7 Consequence assessment

Chapter 10 Risk Assessment and Safety Evaluation of Anqing Dike

[**Abstract**] Based on the method of risk analysis mentioned-above, the safety factor, probability of failure, probability of flooding of various failure modes for each individual section and a whole dike section will be given at different water levels. A risk map of structural safety and some multi-evaluation indices along the whole dike has been achieved. The combination probability also will be presented for each failure mode and every individual section and a whole section, some valuable remarks and conclusions will be helpful to understand the weakest compartments. The total probability of failure of the dike ring can be obtained, which will be employed in a further calculating of the risk of flooding including the economic loss.

10.1 Statistic characteristic of safety indexes at various water levels for the whole dike

Chapter 4 and Chapter 9 explained the methods to calculate probabilities of failure for an individual dyke section. Calculations of failure probabilities by mechanisms will be done for each section. If the failure probability caused by any mechanism for one section is known then the failure probability of the section, which is caused by all mechanisms, can be calculated. Eventually the failure probability of the whole dike, which comprises of all consecutive sections, also can be calculated. The same methods will be used in this chapter to derive probabilities of failure for the 55 identified dyke sections.

The statistic characteristic, including mean value and variance, of multi-criteria of safety assessment, such as safety factor, reliability index, probability of flooding (structural risk degree), can be gained, which is valuable data to evaluate the strength discrepancy of every dike subsection. In addition, the relations of safety indexes with different flood water level are useful to adjust the protected water level.

10.1.1 Overtopping

The mean value of safety indexes for a whole dike at a certain water level can sum up the computed results for individual section. Figure 10.1 shows the reliability index against overtopping at different water levels. It can be seen that the reliability index increases with the water level linearly. The average reliability index of the whole dike is 3.29 at design water level, i.e. the fifth water level. When the water level is almost equal to the crest height, the reliability index is 1.97.

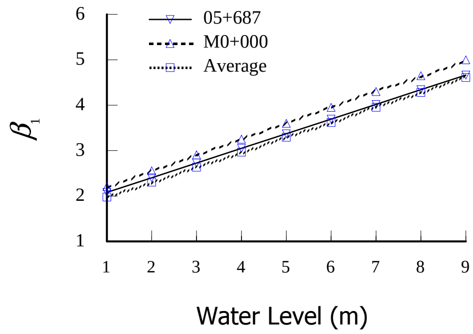


Figure 10.1 Reliability index against overtopping for the whole dike and subsection 05+687 and M0+000

Figure 10.2 shows the relation between the probability of failure of overtopping and the water levels. It can be seen that the probability will be decreased rapidly with the increasing of water level, and the values become very smaller after the fifth water level. The values of the probability at the first and fifth water level are 0.0247 and 0.000546, respectively. In general, heightening the crest is an effective measures in order to reduce the overtopping risk.

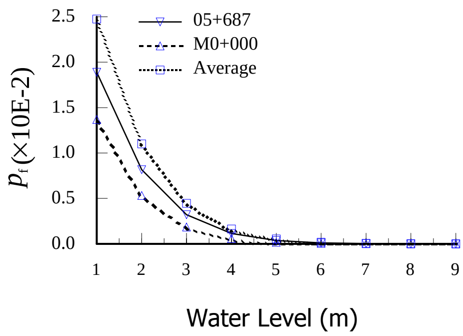


Figure 10.2 Probability of failure of overtopping for the whole dike and dike subsection 05+687 and M0+000

10.1.2 Sliding

The safety factor, reliability index and risk degree of/against sliding in the outer side and inner side for a whole dike section at various water levels are shown in Figure 10.3, Figure 10.4 and Figure 10.5, respectively. It can be seen that the safety factor and reliability index against sliding are appear to increase linearly, and the risk degree of sliding goes down step by step. The reason is that the saturation line will descend when the water elevation becomes lower, therefore the circle sliding surface will change. In addition, the value of evaluation index of outer side is smaller obviously than the one of inner side due to the piping-berm has some good influence against the sliding.

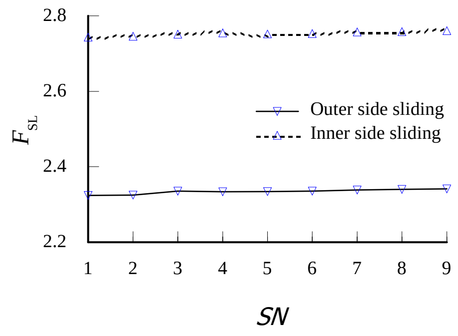


Figure 10.3 Safety factor against sliding at various water levels

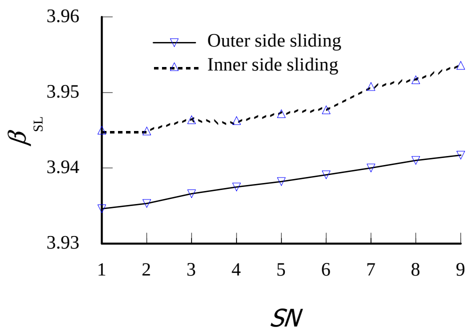


Figure 10.4 Reliability index against sliding at various water levels

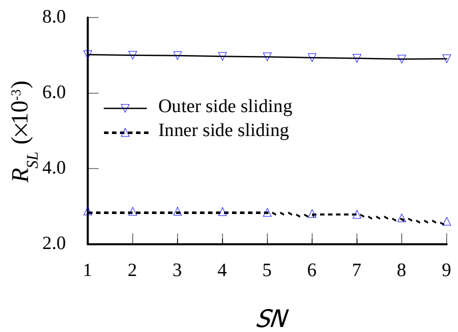


Figure 10.5 Risk degree of sliding at various water level

10.1.3 Piping

The safety factor, reliability index and risk degree of/against piping on the dike body for a whole dike section at various water levels are shown in Figure 10.6, Figure 10.7 and Figure 10.8, respectively. It can be seen that the safety factor and reliability index against piping are appear to increase evidently. The risk degree of piping goes down sharply, and it will be almost zero at higher flood water levels. In this study, the values of evaluation indexes are enough high to meet the demand for safety, perhaps the higher critical hydraulic gradient has been assumed. More detailed data should be gathered and be employed the modeling of piping. Some other elaborate models also should be used to analyze the phenomena of sand boiling.

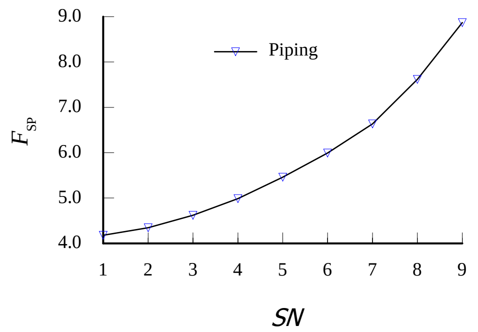


Figure 10.6 Safety factor against piping at various water level

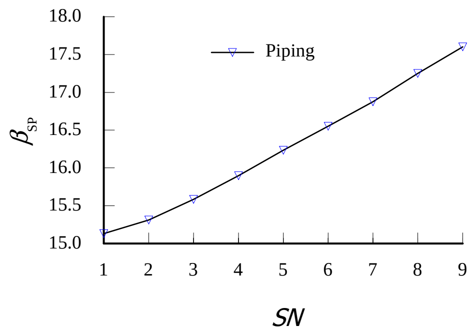


Figure 10.7 Reliability index against piping at various water level

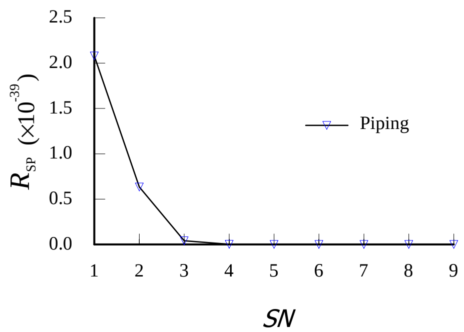


Figure 10.8 Risk degree of piping at various water level

The computing results of the mean value of various safety indexes are also listed in table 10.1.

Table 10.1 Statistic results at different water levels

WL	Outer slope stability			Inner slope stability			Seepage stability		
	Safety factor	Reliability index	Risk degree (*1.0E-2)	Safety factor	Reliability index	Risk degree (*1.0E-2)	Safety factor	Reliability index	Risk degree
1	2.324	3.934	0.702	2.744	3.945	0.288	4.178	15.13	2.08E-39
2	2.325	3.935	0.700	2.746	3.945	0.287	4.343	15.31	6.33E-40
3	2.336	3.936	0.699	2.751	3.946	0.287	4.619	15.583	4.13E-41
4	2.334	3.937	0.697	2.755	3.946	0.286	4.989	15.897	2.97E-42
5	2.335	3.938	0.696	2.752	3.947	0.284	5.458	16.233	2.94E-44
6	2.335	3.939	0.694	2.753	3.948	0.281	5.994	16.552	1.40E-45
7	2.338	3.940	0.692	2.757	3.951	0.279	6.636	16.876	0
8	2.341	3.941	0.690	2.758	3.952	0.270	7.620	17.250	0
9	2.342	3.942	0.691	2.761	3.954	0.261	8.870	17.604	0

10.2 Distribution of evaluation indexes for a whole dike at different water levels

Figure 10.9 shows that the distribution of the design water level of the whole dike section.

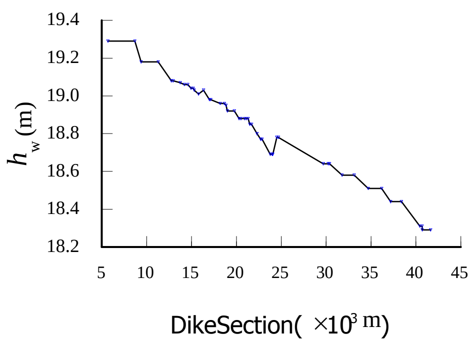


Figure 10.9 Distribution of design water level of a whole dike section

10.2.1 Overtopping

The distribution of reliability index against overtopping and probability of failure of overtopping along the whole dike section at various water levels are shown in Figure 10.10 and Figure 10.11, respectively. The difference of the evaluation index with different subsection at a certain water level is rather obvious, because of the influence of fetch length, wave length, average water depth, and wind direction at the subsection. The probability of failure is bigger than the others for the following sections: No. 34+687, No. 36+187, and No. 22+287.

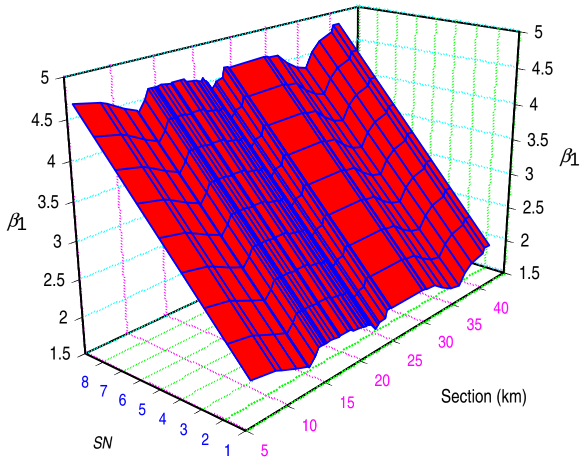


Figure 10.10 Reliability index against overtopping at different water levels

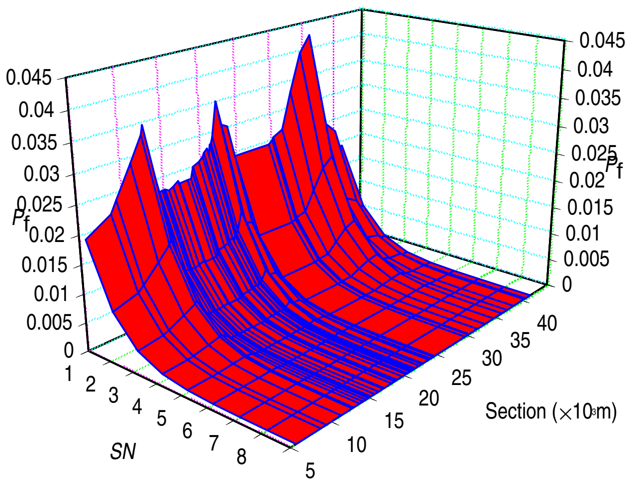


Figure 10.11 Probability of failure of overtopping at different water levels

10.2.2 Sliding

The Distribution of the safety factor of inner and outer slope stability for the whole dike section at design water level is shown in Figure 10.12. The rather difference between different dike subsections can arise, the statistic variance is 0.55. It also can be observed that the value of the outer slope is lower than the one of the inner slope for each individual section, but the distribution tendency of the safety factor is very similar for both sides. During the flood season of the river, some provisions for the upstream side of the dike should be made.

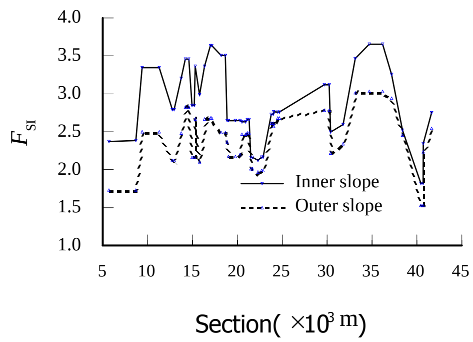


Figure 10.12 Distribution of safety factor of inner and outer slope stability at design WL

The distributions of reliability index and probability of flooding of inner and outer side at design water level are shown in Figure 10.13 and 10.14. It can be concluded that the difference between the value of the inner side and the one of the outer slope is quite small, though the values obtained for different individual section with large variance. The intrinsic reason is that the assumption of the same statistic characteristics of soil strength parameters for the both sides. By comparing the Figure 10.13 with Figure 10.12, the distribution tendency is not identical because of the variance of the soil strength parameters used in the reliability analysis. From these figures, we can obtain the weakest section is No. 05+687 and No. 08+687, and No. 40+439 and No. 40+687.

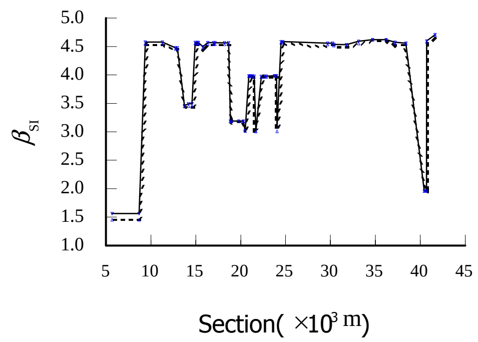


Figure 10.13 Distribution of reliability index of inner and outer slope stability at design WL

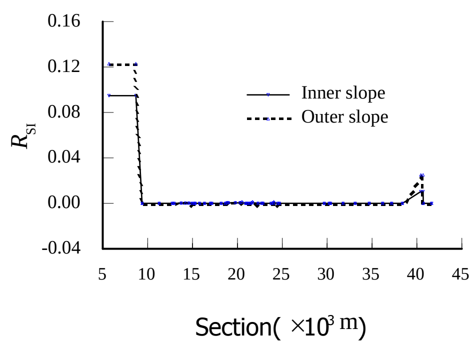


Figure 10.14 Distribution of risk degree of inner and outer slope stability at design WL

10.2.3 Piping

The distributions of safety factor, reliability index and probability of flooding of piping at design water level are shown in Figure 10.15 and 10.16 and 10.17, respectively. It can be seen that the value is approximate to each other for some subsections. By comparing the Figure 10.15 with Figure 10.16, the distribution tendency is nearly identical, and a possible reason is that the coefficient of variation of the soil permeability coefficient has a little influence on the reliability index of piping. The weakest section is No.40+439 and No.40+687, and the reliability index of piping is 3.16.

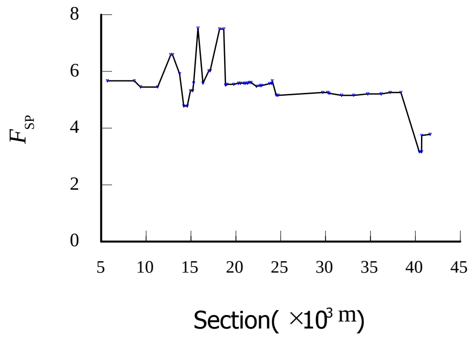


Figure 10.15 Distribution of safety factor against piping at design WL

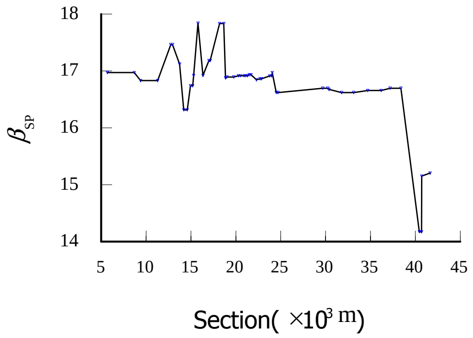


Figure 10.16 Distribution of reliability index against piping for whole dike at design WL

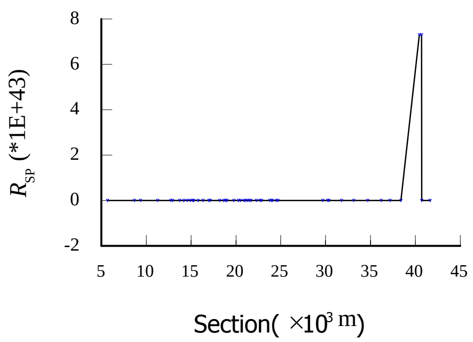


Figure 10.17 Distribution of risk degree of piping for whole dike at design WL

The following table 10.2 listed the safety factor, reliability index, and probability of flooding of various failure modes at design water level of each section.

Table 10.2 The results of whole dike section for various failure modes at design WL

Section	Outer slope stability			Inner slope stability			Piping			Overtopping	
	SF1	RI1	RD1	SF1	RI2	RD2	SF3	RI3	RD3	RI4	RD4
05+687	2.37	1.5615	0.094814	1.730	1.4515	0.12296	5.6675	16.4726	2.85E-61	3.372	3.73E-04
08+687	2.383	1.5615	0.094814	1.730	1.4515	0.12296	5.6675	16.4726	2.85E-61	3.3075	4.71E-04
09+387	3.345	4.5777	9.64E-07	2.497	4.5469	2.6782E-06	5.453	16.3314	2.91E-60	3.273	5.32E-04
11+287	3.345	4.5777	9.64E-07	2.497	4.5469	2.6782E-06	5.453	16.3314	2.91E-60	3.1709	7.60E-04
12+737	2.792	4.4775	1.55E-06	2.120	4.4592	0.000004046	6.5894	16.9634	7.55E-65	3.1072	9.45E-04
12+942	2.792	4.4775	1.55E-06	2.120	4.4592	0.000004046	6.5894	16.9634	7.55E-65	3.0828	1.03E-03
13+737	3.205	3.4603	0.000111	2.482	3.4455	0.00028097	5.917	16.6209	2.42E-62	3.1987	6.90E-04
14+187	3.457	3.4927	9.82E-05	2.827	3.4519	0.00027437	4.7796	15.8158	1.19E-56	3.3075	4.71E-04
14+587	3.457	3.4927	9.82E-05	2.827	3.4519	0.00027437	4.7796	15.8158	1.19E-56	3.3075	4.71E-04
14+937	2.847	4.5724	9.89E-07	2.161	4.5506	2.6314E-06	5.3201	16.2396	1.31E-59	3.3049	4.75E-04
15+187	2.847	4.5724	9.89E-07	2.161	4.5506	2.6314E-06	5.3201	16.2396	1.31E-59	3.3046	4.76E-04
15+287	3.362	4.5707	9.96E-07	2.668	4.5439	2.7172E-06	5.5853	16.4196	6.82E-61	3.3048	4.75E-04
15+787	2.987	4.4979	1.41E-06	2.101	4.4733	0.000003789	7.5263	17.3412	1.13E-67	3.3159	4.57E-04
16+337	3.362	4.5707	9.96E-07	2.668	4.5439	2.7172E-06	5.5853	16.4196	6.82E-61	3.3048	4.75E-04
16+987	3.636	4.5664	1.02E-06	2.681	4.5382	2.7916E-06	6.0247	16.6809	8.89E-63	3.2929	4.96E-04
17+137	3.636	4.5664	1.02E-06	2.681	4.5382	2.7916E-06	6.0247	16.6809	8.89E-63	3.3047	4.76E-04
18+187	3.505	4.5647	1.03E-06	2.487	4.5368	2.8098E-06	7.4975	17.3341	1.28E-67	3.3028	4.79E-04
18+647	3.505	4.5647	1.03E-06	2.487	4.5368	2.8098E-06	7.4975	17.3341	1.28E-67	3.3028	4.79E-04
18+837	2.648	3.189	0.000293	2.365	3.1809	0.0007235	5.5185	16.3773	1.37E-60	3.2788	5.21E-04
18+987	2.646	3.1891	0.000293	2.172	3.1818	0.00072129	5.5455	16.3949	1.02E-60	3.2659	5.46E-04
19+757	2.646	3.1891	0.000293	2.172	3.1818	0.00072129	5.5455	16.3949	1.02E-60	3.2545	5.68E-04
20+287	2.649	3.1893	0.000293	2.195	3.1818	0.00072133	5.5822	16.4161	7.23E-61	3.2454	5.87E-04
20+487	2.630	3.0217	0.000516	2.466	3.017	0.001258	5.5822	16.4161	7.23E-61	3.2334	6.12E-04
20+904	2.631	3.9832	1.40E-05	2.466	3.977	0.000034405	5.5822	16.4161	7.23E-61	3.2231	6.34E-04
21+087	2.660	3.9835	1.39E-05	2.483	3.9774	0.00003434	5.5822	16.4161	7.23E-61	3.2562	5.65E-04
21+292	2.66	3.9835	1.39E-05	2.483	3.9774	0.00003434	5.5822	16.4161	7.23E-61	3.2129	6.57E-04
21+487	2.161	3.9764	1.44E-05	2.009	3.9639	0.000036354	5.6102	16.4338	5.40E-61	3.2117	6.60E-04
21+682	2.161	3.0165	0.000525	2.009	3.007	0.0013001	5.6102	16.4338	5.40E-61	3.1911	7.09E-04
22+287	2.124	3.9752	1.44E-05	1.967	3.9633	0.000036445	5.4744	16.3455	2.31E-60	3.0953	9.83E-04
22+687	2.164	3.976	1.44E-05	1.986	3.9643	0.000036293	5.5007	16.3631	1.73E-60	3.139	8.48E-04
22+857	2.164	3.976	1.44E-05	1.986	3.9643	0.000036293	5.5007	16.3631	1.73E-60	3.1679	7.68E-04
23+757	2.731	3.9867	1.38E-05	2.614	3.9819	0.000033699	5.573	16.4126	7.66E-61	3.1869	7.19E-04
23+987	2.731	3.9867	1.38E-05	2.614	3.9819	0.000033699	5.573	16.4126	7.66E-61	3.2997	4.84E-04
24+037	2.763	3.0779	0.000428	2.572	3.0094	0.0012901	5.6675	16.4726	2.85E-61	3.2997	4.84E-04
24+487	2.759	4.5871	9.21E-07	2.681	4.5653	2.4536E-06	5.1577	16.123	8.66E-59	3.3007	4.82E-04
24+687	2.759	4.5871	9.21E-07	2.681	4.5653	2.4536E-06	5.1577	16.123	8.66E-59	3.2889	5.03E-04
29+687	3.119	4.5579	1.06E-06	2.784	4.5338	2.8499E-06	5.258	16.1972	2.60E-59	3.2991	4.85E-04
30+237	3.119	4.5579	1.06E-06	2.784	4.5338	2.8499E-06	5.258	16.1972	2.60E-59	3.2872	5.06E-04
30+387	2.495	4.5372	1.17E-06	2.217	4.5155	0.000003108	5.228	16.176	3.67E-59	3.2765	5.26E-04
31+787	2.595	4.5323	1.20E-06	2.340	4.5113	3.1699E-06	5.1578	16.123	8.66E-59	3.2648	5.48E-04

Section	Outer slope stability			Inner slope stability			Piping			Overtopping	
	SF1	RI1	RD1	SF1	RI2	RD2	SF3	RI3	RD3	RI4	RD4
33+137	3.464	4.5917	9.01E-07	3.013	4.551	2.6265E-06	5.1578	16.123	8.66E-59	3.1492	8.19E-04
34+687	3.653	4.6191	7.90E-07	3.032	4.6167	0.000001918	5.2071	16.1583	4.89E-59	3.0525	1.14E-03
36+187	3.653	4.6191	7.90E-07	3.032	4.6167	0.000001918	5.2071	16.1583	4.89E-59	3.0138	1.29E-03
37+187	3.255	4.5752	9.75E-07	2.943	4.5565	2.5587E-06	5.258	16.1972	2.60E-59	3.1861	7.21E-04
38+387	2.523	4.5569	1.06E-06	2.454	4.549	2.6512E-06	5.258	16.1972	2.60E-59	3.2976	4.88E-04
40+439	1.818	1.9596	0.010274	1.528	1.9543	0.024963	3.1607	13.6726	7.30E-43	3.3647	3.83E-04
40+687	1.818	1.9596	0.010274	1.528	1.9543	0.024963	3.1607	13.6726	7.30E-43	3.4466	2.84E-04
40+709	2.344	4.5974	8.77E-07	2.235	4.5802	2.2852E-06	3.7397	14.6506	6.58E-49	3.4478	2.83E-04
41+644	2.750	4.7094	5.09E-07	2.544	4.695	1.3119E-06	3.7765	14.7036	3.01E-49	3.4464	2.84E-04
M0+000	1.991	3.5714	7.29E-05	2.09	3.1534	0.00079521	4.1484	15.1803	2.35E-52	3.6019	1.58E-04
M0+948	2.005	2.8493	0.0009	1.238	2.8255	0.0023261	5.9699	16.6491	1.51E-62	3.6019	1.58E-04
M1+748	1.703	1.8028	0.014662	1.476	1.793	0.035961	5.5198	16.3773	1.37E-60	3.6019	1.58E-04
M2+448	1.919	2.9498	0.000653	1.691	2.9334	0.0016521	4.5428	15.5969	3.76E-55	3.6019	1.58E-04
M3+548	3.186	3.1043	0.000392	1.700	3.2084	0.00065771	7.9729	17.493	7.98E-69	3.6019	1.58E-04
M5+248	1.583	4.3461	2.84E-06	1.692	3.8888	0.000049663	5.6926	16.4867	2.25E-61	3.6019	1.58E-04

SF: Safety Factor, RI: Reliability Index; RD: Risk Degree(Probability of flooding).

It can be concluded from this study that various evaluation index for safety assessment on more than fifty dike subsections can meet the design standard according to the current code, and the probability of flooding of potential damage is considerable little, especially in seepage instability. The total risk of sliding in outer side slope for the whole dike section at design water level is 0.696×10^{-2}, yet it is 0.702×10^{-2} at first water level (almost equal to the crest height of dike). The probability of flooding of sliding decreases by around 0.02% with a decreasing of water level for each 0.5m. For seepage instability, the probability of flooding at design water level is 2.94×10^{-44}, and it is 2.08×10^{-39} at first water level. It can be seen that the probability of floodingreduce by 10 to 100 times when the water level is decreased by 0.5m. It is shown that a specified water level for flood protection can be updated appropriately since a rather large scale reinforcement scheme for this dike section has been completed since 1998.

According to the distribution of various evaluation index along the entire dike at design water level, the weakest link can be addressed. For overtopping, the weakest section is: No. 34+687, No. 36+187, and No. 22+287. For piping, The weakest section is No.40+439 and No.40+687, and the reliability index of piping is 3.16. For sliding, the weakest section is No. 05+687 and No. 08+687, and No. 40+439 and No. 40+687. It should be pointed out that the sliding failure mode has a great contribution on the overall probabilities of the dike, which will be discussed in the following section.

10.3 Combination of the failure probabilities

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

$$P\{\text{section fails}\} = P\left\{Z_1 < 0 \text{ or } Z_2 < 0 \text{ or } Z_3 < 0 \text{ or } Z_4 < 0\right\} \quad (10\text{-}1)$$

Where: $Z_1 < 0$ denotes the failure of the section by outer slope stability; $Z_2 < 0$ denotes the failure of the section by inner slope stability; $Z_3 < 0$ denotes the failure of the section by piping; $Z_4 < 0$ denotes the failure of the section by overtopping.

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

$$\max\_P\{Z_i < 0\} \le P\{\text{section\_fails}\} \le \sum_{i=1}^{4} P\{Z_i < 0\} \quad (10\text{-}2)$$

The calculated results for section 05+687 are shown in Table 10.3.

For other sections, the procedure is the same therefore no explanation is given hereby. Only the results are shown in Table 10.4.

Table 10.3 Combination of the failure probabilities for section 05+687

Rank	WL	Return period (years)	Component failure probability				Combined Probability	
			Inner sliding	Outer sliding	Piping	Overtopping	Lower boundary	Upper boundary
1	21.19	914	0.0596	0.0414	2.56E-53	1.89E-02	0.0596	0.1199
2	20.79	305	0.0595	0.0413	3.05E-54	8.16E-03	0.0595	0.1090
3	20.29	166	0.0594	0.0412	5.10E-56	3.21E-03	0.0594	0.1038
4	19.79	114	0.0593	0.0412	2.28E-58	1.15E-03	0.0593	0.1016
5	19.29	87	0.0592	0.1230	2.85E-61	3.73E-04	0.1230	0.1826
6	18.79	70	0.0591	0.1175	4.84E-64	1.10E-04	0.1175	0.1767
7	18.29	28	0.0589	0.0409	1.09E-66	2.92E-05	0.0589	0.0999
8	17.79	13	0.0588	0.0832	3.14E-69	7.04E-06	0.0832	0.1419
9	17.29	10	0.0589	0.0829	2.76E-72	1.54E-06	0.0829	0.1418

Table 10.4 Combined failure probabilities at sections

Sect.	RT	914	305	166	114	87	70	28	13	10
	WL	1	2	3	4	5	6	7	8	9
	Upper	5.96E-02	5.95E-02	5.94E-02	5.93E-02	7.50E-02	1.18E-01	5.89E-02	8.32E-02	5.89E-02
05+687	Lower	1.20E-01	1.09E-01	1.04E-01	1.02E-01	1.35E-01	1.77E-01	9.99E-02	1.42E-01	5.89E-02
	MinSF	1.712	1.714	1.728	1.730	1.730	1.728	1.728	1.732	1.736
	Upper	5.96E-02	5.95E-02	5.94E-02	5.93E-02	7.50E-02	1.18E-01	5.89E-02	8.32E-02	5.89E-02
08+687	Lower	1.23E-01	1.10E-01	1.04E-01	1.02E-01	1.35E-01	1.77E-01	9.73E-02	1.42E-01	5.89E-02
	MinSF	1.712	1.714	1.728	1.730	1.730	1.728	1.728	1.732	1.736
	Upper	2.36E-02	1.05E-02	4.28E-03	1.58E-03	5.32E-04	1.62E-04	4.49E-05	1.12E-05	2.54E-06
09+387	Lower	2.36E-02	1.05E-02	4.28E-03	1.58E-03	5.36E-04	1.66E-04	4.85E-05	1.48E-05	3.48E-06
	MinSF	2.481	2.483	2.510	2.504	2.498	2.496	2.495	2.498	2.500
	Upper	2.89E-02	1.33E-02	5.63E-03	2.17E-03	7.60E-04	2.42E-04	7.03E-05	1.85E-05	4.41E-06
11+287	Lower	2.89E-02	1.33E-02	5.63E-03	2.17E-03	7.64E-04	2.46E-04	7.39E-05	2.21E-05	5.35E-06
	MinSF	2.481	2.483	2.510	2.504	2.498	2.496	2.495	2.498	2.500
	Upper	3.31E-02	1.56E-02	6.71E-03	2.64E-03	9.45E-04	3.08E-04	9.14E-05	2.46E-05	6.02E-06
12+737	Lower	3.31E-02	1.56E-02	6.72E-03	2.65E-03	9.50E-04	3.14E-04	9.70E-05	3.02E-05	7.56E-06
	MinSF	2.115	2.115	2.141	2.126	2.121	2.118	2.115	2.115	2.115
	Upper	3.46E-02	1.64E-02	7.14E-03	2.83E-03	1.03E-03	3.38E-04	1.01E-04	2.76E-05	6.85E-06
12+942	Lower	3.46E-02	1.64E-02	7.15E-03	2.84E-03	1.03E-03	3.44E-04	1.07E-04	3.32E-05	8.39E-06

Sect.	RT	914	305	166	114	87	70	28	13	10
	WL	1	2	3	4	5	6	7	8	9
	MinSF	2.115	2.115	2.141	2.126	2.121	2.118	2.115	2.115	2.115
	Upper	2.78E-02	1.27E-02	5.29E-03	2.00E-03	6.90E-04	2.81E-04	2.80E-04	2.79E-04	1.10E-04
13+737	Lower	2.82E-02	1.31E-02	5.68E-03	2.39E-03	1.08E-03	6.08E-04	4.52E-04	4.05E-04	1.14E-04
	MinSF	2.464	2.466	2.478	2.480	2.482	2.482	2.482	2.485	2.488
	Upper	2.24E-02	9.85E-03	3.94E-03	1.43E-03	4.71E-04	2.74E-04	2.72E-04	2.75E-04	9.71E-05
14+187	Lower	2.28E-02	1.02E-02	4.31E-03	1.80E-03	8.43E-04	5.12E-04	4.08E-04	3.81E-04	9.91E-05
	MinSF	2.806	2.808	2.825	2.825	2.827	2.826	2.824	2.925	2.962
	Upper	2.24E-02	9.85E-03	3.94E-03	1.43E-03	4.71E-04	2.74E-04	2.72E-04	2.75E-04	9.71E-05
14+587	Lower	2.28E-02	1.02E-02	4.31E-03	1.80E-03	8.43E-04	5.12E-04	4.08E-04	3.81E-04	9.91E-05
	MinSF	2.806	2.808	2.825	2.825	2.827	2.826	2.824	2.925	2.962
	Upper	2.26E-02	9.93E-03	3.97E-03	1.44E-03	4.75E-04	1.42E-04	3.82E-05	9.29E-06	2.05E-06
14+937	Lower	2.26E-02	9.93E-03	3.97E-03	1.44E-03	4.79E-04	1.46E-04	4.18E-05	1.29E-05	3.02E-06
	MinSF	2.141	2.149	2.173	2.166	2.161	2.162	2.162	2.164	2.167
	Upper	2.26E-02	9.94E-03	3.98E-03	1.44E-03	4.76E-04	1.42E-04	3.82E-05	9.30E-06	2.05E-06
15+187	Lower	2.26E-02	9.94E-03	3.98E-03	1.44E-03	4.79E-04	1.46E-04	4.18E-05	1.29E-05	3.02E-06
	MinSF	2.141	2.149	2.173	2.166	2.161	2.162	2.162	2.164	2.167
	Upper	2.26E-02	9.95E-03	3.98E-03	1.45E-03	4.75E-04	1.42E-04	3.82E-05	9.27E-06	2.04E-06
15+287	Lower	2.26E-02	9.95E-03	3.98E-03	1.45E-03	4.79E-04	1.46E-04	4.19E-05	1.29E-05	3.02E-06
	MinSF	2.651	2.653	2.686	2.674	2.669	2.667	2.665	2.667	2.530
	Upper	2.22E-02	9.71E-03	3.87E-03	1.40E-03	4.57E-04	1.35E-04	3.62E-05	8.74E-06	1.91E-06
15+787	Lower	2.22E-02	9.72E-03	3.88E-03	1.41E-03	4.62E-04	1.40E-04	4.14E-05	1.39E-05	3.32E-06
	MinSF	2.091	2.091	2.104	2.104	2.101	2.095	2.091	2.091	2.092
	Upper	2.26E-02	9.95E-03	3.98E-03	1.45E-03	4.75E-04	1.42E-04	3.82E-05	9.27E-06	2.04E-06
16+337	Lower	2.26E-02	9.95E-03	3.98E-03	1.45E-03	4.79E-04	1.46E-04	4.19E-05	1.29E-05	3.02E-06
	MinSF	2.651	2.653	2.686	2.674	2.669	2.667	2.665	2.667	2.530
	Upper	2.33E-02	1.03E-02	4.13E-03	1.50E-03	4.96E-04	1.48E-04	4.00E-05	9.72E-06	2.14E-06
16+987	Lower	2.33E-02	1.03E-02	4.13E-03	1.50E-03	5.00E-04	1.52E-04	4.38E-05	1.35E-05	3.14E-06
	MinSF	2.663	2.665	2.686	2.688	2.681	2.679	2.678	2.569	0.754
	Upper	2.28E-02	1.00E-02	4.00E-03	1.45E-03	4.76E-04	1.41E-04	3.79E-05	9.17E-06	2.01E-06
17+137	Lower	2.28E-02	1.00E-02	4.00E-03	1.45E-03	4.79E-04	1.45E-04	4.17E-05	1.29E-05	3.01E-06
	MinSF	2.663	2.665	2.686	2.688	2.681	2.679	2.678	2.569	0.754
	Upper	2.29E-02	1.01E-02	4.03E-03	1.46E-03	4.79E-04	1.42E-04	3.81E-05	9.22E-06	2.02E-06
18+187	Lower	2.29E-02	1.01E-02	4.03E-03	1.46E-03	4.82E-04	1.46E-04	4.19E-05	1.30E-05	3.03E-06
	MinSF	2.470	2.472	2.508	2.493	2.488	2.487	2.485	2.488	2.491
	Upper	2.29E-02	1.01E-02	4.03E-03	1.46E-03	4.79E-04	1.42E-04	3.81E-05	9.22E-06	2.02E-06
18+647	Lower	2.29E-02	1.01E-02	4.03E-03	1.46E-03	4.82E-04	1.46E-04	4.19E-05	1.30E-05	3.03E-06
	MinSF	2.470	2.472	2.508	2.493	2.488	2.487	2.485	2.488	2.491
	Upper	2.41E-02	1.07E-02	4.31E-03	1.57E-03	7.24E-04	7.21E-04	7.19E-04	7.17E-04	2.90E-04
18+837	Lower	2.51E-02	1.17E-02	5.33E-03	2.59E-03	1.54E-03	1.17E-03	1.05E-03	1.02E-03	2.92E-04
	MinSF	2.361	2.363	2.358	2.364	2.365	2.365	2.380	2.384	2.387
	Upper	2.49E-02	1.11E-02	4.47E-03	1.64E-03	7.21E-04	7.19E-04	7.19E-04	7.17E-04	2.90E-04
18+987	Lower	2.59E-02	1.21E-02	5.49E-03	2.66E-03	1.56E-03	1.18E-03	1.05E-03	1.02E-03	2.92E-04
	MinSF	2.162	2.164	2.167	2.170	2.172	2.175	2.177	2.180	2.183
	Upper	2.54E-02	1.14E-02	4.61E-03	1.70E-03	7.21E-04	7.19E-04	7.19E-04	7.17E-04	2.90E-04
19+757	Lower	2.64E-02	1.24E-02	5.63E-03	2.72E-03	1.58E-03	1.18E-03	1.06E-03	1.02E-03	2.93E-04
	MinSF	2.162	2.164	2.167	2.170	2.172	2.175	2.177	2.180	2.183

Sect.	RT	914	305	166	114	87	70	28	13	10
	WL	1	2	3	4	5	6	7	8	9
	Upper	2.61E-02	1.17E-02	4.75E-03	1.75E-03	7.21E-04	7.20E-04	7.18E-04	7.16E-04	2.90E-4
20+287	Lower	2.71E-02	1.27E-02	5.77E-03	2.77E-03	1.60E-03	1.19E-03	1.06E-03	1.02E-03	2.93E-04
	MinSF	2.189	2.192	2.195	2.192	2.195	2.195	2.205	2.208	2.211
	Upper	2.67E-02	1.20E-02	4.90E-03	1.82E-03	1.26E-03	1.26E-03	1.25E-03	1.25E-03	5.12E-04
20+487	Lower	2.85E-02	1.38E-02	6.68E-03	3.60E-03	2.39E-03	1.96E-03	1.82E-03	1.78E-03	5.15E-04
	MinSF	2.471	2.473	2.482	2.463	2.467	2.467	2.487	2.490	2.492
	Upper	2.73E-02	1.23E-02	5.04E-03	1.88E-03	6.34E-04	1.94E-04	5.37E-05	3.41E-05	1.38E-05
20+904	Lower	2.73E-02	1.23E-02	5.09E-03	1.93E-03	6.82E-04	2.42E-04	1.02E-04	6.13E-05	1.68E-05
	MinSF	2.471	2.473	2.482	2.463	2.467	2.467	2.487	2.490	2.492
	Upper	2.55E-02	1.14E-02	4.61E-03	1.70E-03	5.65E-04	1.70E-04	4.64E-05	3.41E-05	1.38E-05
21+087	Lower	2.55E-02	1.14E-02	4.66E-03	1.75E-03	6.13E-04	2.18E-04	9.44E-05	5.93E-05	1.63E-05
	MinSF	2.455	2.457	2.462	2.466	2.483	2.487	2.470	2.473	2.475
	Upper	2.78E-02	1.26E-02	5.18E-03	1.94E-03	6.57E-04	2.02E-04	5.62E-05	3.41E-05	1.38E-05
21+292	Lower	2.78E-02	1.26E-02	5.23E-03	1.99E-03	7.05E-04	2.50E-04	1.04E-04	6.20E-05	1.70E-05
	MinSF	2.455	2.457	2.462	2.466	2.483	2.487	2.470	2.473	2.475
	Upper	2.80E-02	1.27E-02	5.21E-03	1.95E-03	6.60E-04	2.03E-04	5.63E-05	3.61E-05	1.42E-05
21+487	Lower	2.81E-02	1.28E-02	5.26E-03	2.00E-03	7.11E-04	2.54E-04	1.07E-04	6.44E-05	1.74E-05
	MinSF	2.002	2.003	2.006	2.008	2.010	2.011	2.013	2.015	2.016
	Upper	2.91E-02	1.33E-02	5.50E-03	2.07E-03	1.30E-03	1.30E-03	1.30E-03	1.29E-03	5.21E-04
21+682	Lower	3.09E-02	1.51E-02	7.33E-03	3.90E-03	2.53E-03	2.04E-03	1.88E-03	1.83E-03	5.25E-04
	MinSF	2.002	2.003	2.006	2.008	2.010	2.011	2.013	2.015	2.016
	Upper	3.51E-02	1.65E-02	7.09E-03	2.77E-03	9.83E-04	3.17E-04	9.28E-05	3.61E-05	1.43E-05
22+287	Lower	3.52E-02	1.66E-02	7.14E-03	2.82E-03	1.03E-03	3.68E-04	1.43E-04	7.50E-05	2.02E-05
	MinSF	1.960	1.961	1.964	1.966	1.967	1.975	1.971	1.972	1.974
	Upper	3.25E-02	1.51E-02	6.35E-03	2.44E-03	8.48E-04	2.68E-04	7.66E-05	3.60E-05	1.42E-05
22+687	Lower	3.26E-02	1.52E-02	6.40E-03	2.49E-03	8.98E-04	3.18E-04	1.27E-04	7.01E-05	1.89E-05
	MinSF	1.985	1.987	1.989	1.991	1.987	1.980	1.997	1.998	2.000
	Upper	5.96E-02	5.95E-02	5.94E-02	5.93E-02	7.50E-02	1.18E-01	5.89E-02	8.32E-02	5.89E-02
22+857	Lower	3.09E-02	1.42E-02	5.94E-03	2.28E-03	8.19E-04	2.89E-04	1.18E-04	6.75E-05	1.82E-05
	MinSF	1.985	1.987	1.989	1.991	1.987	1.980	1.997	1.998	2.000
	Upper	3.01E-02	1.37E-02	5.65E-03	2.12E-03	7.19E-04	2.21E-04	6.15E-05	3.39E-05	1.36E-05
23+757	Lower	3.01E-02	1.37E-02	5.70E-03	2.17E-03	7.67E-04	2.69E-04	1.09E-04	6.29E-05	1.71E-05
	MinSF	2.609	2.598	2.610	2.615	2.614	2.618	2.625	2.628	2.631
	Upper	2.41E-02	1.05E-02	4.17E-03	1.50E-03	4.84E-04	1.41E-04	3.73E-05	3.39E-05	1.36E-05
23+987	Lower	2.41E-02	1.05E-02	4.22E-03	1.55E-03	5.32E-04	1.89E-04	8.50E-05	5.63E-05	1.55E-05
	MinSF	2.609	2.598	2.610	2.615	2.614	2.618	2.625	2.628	2.631
	Upper	2.41E-02	1.05E-02	4.17E-03	1.50E-03	1.29E-03	1.29E-03	1.23E-03	1.22E-03	4.25E-04
24+037	Lower	2.58E-02	1.22E-02	5.89E-03	3.22E-03	2.20E-03	1.86E-03	1.69E-03	1.66E-03	4.27E-04
	MinSF	2.594	2.581	2.573	2.573	2.572	2.599	2.629	2.631	2.633
	Upper	2.37E-02	1.04E-02	4.12E-03	1.48E-03	4.82E-04	1.42E-04	3.75E-05	8.95E-06	1.93E-06
24+487	Lower	2.37E-02	1.04E-02	4.12E-03	1.48E-03	4.86E-04	1.45E-04	4.09E-05	1.22E-05	2.84E-06
	MinSF	2.666	2.672	2.675	2.679	2.682	2.684	2.687	2.690	2.693
	Upper	2.43E-02	1.07E-02	4.26E-03	1.54E-03	5.03E-04	1.49E-04	3.96E-05	9.50E-06	2.06E-06
24+687	Lower	2.43E-02	1.07E-02	4.26E-03	1.54E-03	5.06E-04	1.52E-04	4.30E-05	1.28E-05	2.97E-06
	MinSF	2.666	2.672	2.675	2.679	2.682	2.684	2.687	2.690	2.693
	Upper	2.43E-02	1.06E-02	4.20E-03	1.50E-03	4.85E-04	1.41E-04	3.71E-05	8.74E-06	1.86E-06

Sect.	RT	914	305	166	114	87	70	28	13	10
	WL	1	2	3	4	5	6	7	8	9
29+687	Lower	2.43E-02	1.06E-02	4.20E-03	1.50E-03	4.89E-04	1.45E-04	4.07E-05	1.24E-05	2.90E-06
	MinSF	2.774	2.778	2.780	2.776	2.784	2.784	2.791	2.794	2.797
	Upper	2.49E-02	1.09E-02	4.34E-03	1.56E-03	5.06E-04	1.48E-04	3.91E-05	9.27E-06	1.98E-06
30+237	Lower	2.49E-02	1.09E-02	4.34E-03	1.56E-03	5.10E-04	1.52E-04	4.27E-05	1.29E-05	3.02E-06
	MinSF	2.774	2.778	2.780	2.776	2.784	2.784	2.791	2.794	2.797
	Upper	2.54E-02	1.12E-02	4.47E-03	1.61E-03	5.26E-04	1.55E-04	4.10E-05	9.77E-06	2.10E-06
30+387	Lower	2.54E-02	1.12E-02	4.47E-03	1.61E-03	5.30E-04	1.59E-04	4.52E-05	1.40E-05	3.24E-06
	MinSF	2.244	2.246	2.248	2.249	2.218	2.265	2.256	2.258	2.260
	Upper	2.63E-02	1.16E-02	4.64E-03	1.68E-03	5.48E-04	1.61E-04	4.28E-05	1.02E-05	2.20E-06
31+787	Lower	2.63E-02	1.16E-02	4.64E-03	1.68E-03	5.52E-04	1.65E-04	4.71E-05	1.45E-05	3.37E-06
	MinSF	2.327	2.320	2.336	2.315	2.340	2.326	2.345	2.348	2.351
	Upper	3.29E-02	1.51E-02	6.30E-03	2.39E-03	8.19E-04	2.54E-04	7.13E-05	1.80E-05	4.13E-06
33+137	Lower	3.29E-02	1.51E-02	6.30E-03	2.39E-03	8.22E-04	2.57E-04	7.48E-05	2.12E-05	5.01E-06
	MinSF	3.002	3.005	3.008	3.011	3.014	3.023	3.020	3.024	3.028
	Upper	3.96E-02	1.88E-02	8.12E-03	3.19E-03	1.14E-03	3.67E-04	1.07E-04	2.83E-05	6.79E-06
34+687	Lower	3.96E-02	1.88E-02	8.12E-03	3.19E-03	1.14E-03	3.70E-04	1.10E-04	3.09E-05	7.56E-06
	MinSF	3.021	3.024	3.027	3.030	3.033	3.036	3.040	3.044	3.049
	Upper	4.25E-02	2.04E-02	8.94E-03	3.56E-03	1.29E-03	4.24E-04	1.27E-04	3.42E-05	8.35E-06
36+187	Lower	4.25E-02	2.04E-02	8.94E-03	3.56E-03	1.29E-03	4.27E-04	1.30E-04	3.68E-05	9.12E-06
	MinSF	3.021	3.024	3.027	3.030	3.033	3.036	3.040	3.044	3.049
	Upper	3.13E-02	1.42E-02	5.80E-03	2.15E-03	7.21E-04	2.18E-04	5.95E-05	1.46E-05	3.23E-06
37+187	Lower	3.13E-02	1.42E-02	5.80E-03	2.15E-03	7.25E-04	2.22E-04	6.30E-05	1.81E-05	4.19E-06
	MinSF	2.931	2.933	2.936	2.940	2.943	2.946	2.948	2.952	2.955
	Upper	2.52E-02	1.10E-02	4.31E-03	1.53E-03	4.88E-04	1.40E-04	3.63E-05	8.40E-06	1.75E-06
38+387	Lower	2.52E-02	1.10E-02	4.31E-03	1.53E-03	4.91E-04	1.44E-04	4.00E-05	1.20E-05	2.80E-06
	MinSF	2.446	2.447	2.450	2.453	2.455	2.457	2.459	2.464	2.463
	Upper	2.51E-02	2.51E-02	2.50E-02	2.50E-02	2.50E-02	2.49E-02	2.49E-02	2.48E-02	1.01E-02
40+439	Lower	5.80E-02	4.50E-02	3.90E-02	3.66E-02	3.56E-02	3.53E-02	3.51E-02	3.50E-02	1.01E-02
	MinSF	1.524	1.525	1.526	1.528	1.529	1.527	1.529	1.531	1.532
	Upper	2.51E-02	2.51E-02	2.50E-02	2.50E-02	2.50E-02	2.49E-02	2.49E-02	2.48E-02	1.01E-02
40+687	Lower	5.44E-02	4.33E-02	3.83E-02	3.63E-02	3.55E-02	3.52E-02	3.51E-02	3.50E-02	1.01E-02
	MinSF	1.524	1.525	1.526	1.528	1.529	1.527	1.529	1.531	1.532
	Upper	1.90E-02	7.79E-03	2.87E-03	9.52E-04	2.83E-04	7.51E-05	1.78E-05	3.78E-06	8.36E-07
40+709	Lower	1.90E-02	7.79E-03	2.87E-03	9.55E-04	2.86E-04	7.82E-05	2.09E-05	6.87E-06	1.55E-06
	MinSF	2.228	2.229	2.231	2.233	2.235	2.237	2.239	2.241	2.243
	Upper	1.91E-02	7.84E-03	2.89E-03	9.58E-04	2.84E-04	7.54E-05	1.78E-05	3.78E-06	7.16E-07
41+644	Lower	1.91E-02	7.84E-03	2.89E-03	9.60E-04	2.86E-04	7.72E-05	1.96E-05	5.55E-06	1.20E-06
	MinSF	2.538	2.539	2.541	2.543	2.544	2.546	2.548	2.550	2.552
	Upper	1.37E-02	5.32E-03	1.84E-03	7.98E-04	7.95E-04	7.91E-04	7.89E-04	7.87E-04	6.92E-05
M0+000	Lower	1.46E-02	6.20E-03	2.71E-03	1.44E-03	1.03E-03	9.03E-04	8.70E-04	8.59E-04	6.95E-05
	MinSF	1.986	1.986	1.988	1.989	1.991	1.993	1.994	1.996	1.998
	Upper	1.37E-02	5.32E-03	2.34E-03	2.33E-03	2.33E-03	2.32E-03	2.31E-03	2.30E-03	8.81E-04
M0+948	Lower	1.70E-02	8.57E-03	5.08E-03	3.80E-03	3.38E-03	3.26E-03	3.21E-03	3.19E-03	8.82E-04
	MinSF	1.236	1.237	1.237	1.238	1.238	1.239	1.239	1.240	1.240
	Upper	3.62E-02	3.61E-02	3.61E-02	3.60E-02	3.60E-02	3.59E-02	3.59E-02	3.58E-02	1.45E-02
M1+748	Lower	6.46E-02	5.62E-02	5.26E-02	5.11E-02	5.08E-02	5.06E-02	5.05E-02	5.04E-02	1.45E-02

Sect.	RT	914	305	166	114	87	70	28	13	10
	WL	1	2	3	4	5	6	7	8	9
	MinSF	1.463	1.464	1.466	1.460	1.477	1.472	1.442	1.442	1.474
	Upper	1.37E-02	5.32E-03	1.84E-03	1.66E-03	1.65E-03	1.65E-03	1.64E-03	1.63E-03	6.35E-04
M2+448	Lower	1.60E-02	7.64E-03	4.16E-03	2.89E-03	2.46E-03	2.33E-03	2.29E-03	2.28E-03	6.35E-04
	MinSF	1.686	1.687	1.688	1.690	1.691	1.693	1.694	1.696	1.697
	Upper	1.37E-02	5.32E-03	1.84E-03	6.59E-04	6.58E-04	6.56E-04	6.54E-04	6.55E-04	3.88E-04
M3+548	Lower	1.48E-02	6.38E-03	2.89E-03	1.62E-03	1.21E-03	1.09E-03	1.05E-03	1.04E-03	3.88E-04
	MinSF	1.697	1.697	1.699	1.700	1.701	1.703	1.704	1.702	1.707
	Upper	1.37E-02	5.32E-03	1.84E-03	5.72E-04	1.58E-04	4.95E-05	6.30E-03	4.88E-05	2.79E-06
M5+248	Lower	1.38E-02	5.37E-03	1.89E-03	6.25E-04	2.11E-04	9.13E-05	6.34E-03	5.33E-05	3.08E-06
	MinSF	1.582	1.582	1.581	1.582	1.583	1.583	3.539	1.590	1.615

It is conceived that the dike consists of, over its entire length, a series system of 55 consecutive sections. The overall failure probability of the dike can be determined as follows:

$$\mathrm{P}\{\text{dike fails}\} = \mathrm{P}\left\{\mathrm{Z}_1 < 0 \text{ or } \mathrm{Z}_2 < 0 \text{ or } \mathrm{Z}_3 < 0 \cdots \text{ or } \mathrm{Z}_{55} < 0\right\} \tag{10-3}$$

Where $Z_1 < 0, ... and\_Z_{55} < 0$ denotes the failure of section 1 to 55, respectively.
The fundamental upper and lower boundaries are given by:

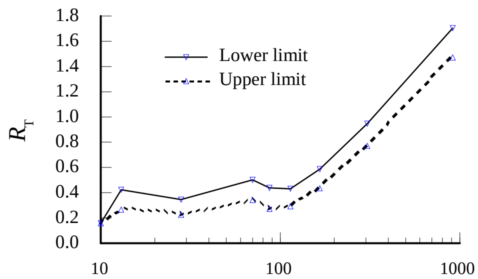
$$\max\_P\{Z_i < 0\} \leq P\{\text{dike\_fails}\} \leq \sum_{i=1}^{55} P\{Z_i < 0\} \tag{10-4}$$

All calculated overall failure probabilities of the dike in different design return periods according to the equation (10-3) are given in Table 10.5 and shown in Figure 10.18.

Table 10.5 Overall failure probabilities of the whole dike

Rank	WL	Return period of WL (years)	Overall failure probability	
			Lower boundary	Upper boundary
1	21.19	914	1.7042	1.4716
2	20.79	305	0.9474	0.7702
3	20.29	166	0.5862	0.4347
4	19.79	114	0.4306	0.2924
5	19.29	87	0.4382	0.2718
6	18.79	70	0.5017	0.3405
7	18.29	28	0.3450	0.2246
8	17.79	13	0.4232	0.2654
9	17.29	10	0.1578	0.1577

It can be seen that the overall failure probability of the whole dike at lower water levels vary within a narrow limits, and then the value will increase rapidly with the increasing of flood water elevation.



Year

Figure 10.18 Relation of upper and lower limit of probability of flooding with return period

According to this idea, the distribution of the maximum and minimum safety factors for various failure modes at design water level along the dike is shown in Figure 10.19, in which the values of overtopping have a great contribution. In this case, the larger safety factors excluding the overtopping are also shown in this figure. It can be seen a larger difference exists in three kinds of limits.

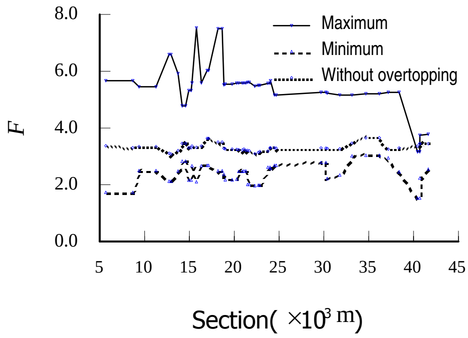


Figure 10.19 Distribution of the upper and lower limit of safety factor along the whole dike

Figure 10.20 shows that the upper and lower limit of probability of flooding along the dike at design water level. The curve of the upper limit and lower limit are very approach at some sections, which means the probability of one failure mode plays a principal role for the overall failure probabilities.

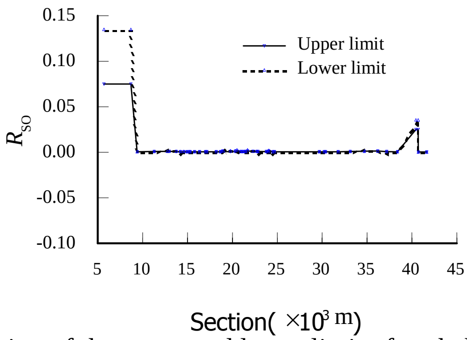


Figure 10.20 Distribution of the upper and lower limit of probability of flooding along the dike

10.4 Classifying of safety grades

Multi-evaluation indexes mentioned-above can be gained for a certain dike subsection at a specified water level, i.e., the different kinds of evaluation indexes, such as safety factor, reliability index, and risk degree, related with the three kinds of failure modes, for instance, sliding stability of outside slope, sliding stability of inner slope, and piping. The different grades can be evaluated according to the specified technical criteria for each failure mode, as listed in Table 10.6. Users also can adjust the range by the local guidelines.

Table 10.6 Technical criteria of safety evaluation

Grade	Definition	Sliding			Piping			Sliding		
		SF	RI	PI	SF	RI	PI	SF	RI	PI
A	Safety	>1.5	>3.8	<1.0E-5	>3.0	>3.8	<1.0E-5	>1.5	>3.8	<1.0E-5
B	Basic Safety	1.3~1.5	3.0~3.8	1.0E-5 ~1.0E-3	2.0~3.0	3.0~3.8	1.0E-5 ~1.0E-3	1.3~1.5	3.0~3.8	1.0E-5 ~1.0E-3
C	Unsafety	<1.3	<3.0	>1.0E-3	<2.0	<3.0	>1.0E-3	<1.3	<3.0	>1.0E-3

Note: SF---safety factor; RI---reliability index; PI---probability of flooding

Referring to the guide of safety assessment for dam (Chinese Standard 2000), the classifying of grade of safety for existing levee follow these rules: when the safety index of all kinds of evaluation are Grade A, then the evaluation Grade is A; when some safety index are Grade B, then the evaluation Grade is B; when more than one safety index is Grade C, then the evaluation Grade is C. If there are one or two index in Grade B and the others in Grade A, it can be assumed as Grade A with better construction quality.

The statistic report can also be printed in tabular form, as shown in Figure 10.21. The statistic pie chart for different modes is shown in Figure 10.22. Figure 10.23 shows the detailed inquiry results of some section with an evaluation index belonging to Grade B.

工程风险评估简明报表

水位级数: 5

上游稳定	安全系数			可靠度指标			风险度		
等级类型	A级	B级	C级	A级	B级	C级	A级	B级	C级
堤段长度 /M	41205	0	0	27665	6970	6570	25137	8026	8042
百分比 /%	100	0	0	67.14	16.92	15.94	61	19.48	19.52

下游稳定	安全系数			可靠度指标			风险度		
等级类型	A级	B级	C级	A级	B级	C级	A级	B级	C级
堤段长度 /M	41205	0	0	27665	6970	6570	25137	8026	8042
百分比 /%	100	0	0	67.14	16.92	15.94	61	19.48	19.52

渗流稳定	安全系数			可靠度指标			风险度		
等级类型	A级	B级	C级	A级	B级	C级	A级	B级	C级
堤段长度 /M	41205	0	0	41205	0	0	41205	0	0
百分比 /%	100	0	0	100	0	0	100	0	0

Figure 10.21 Statistic report forms of the risk evaluation

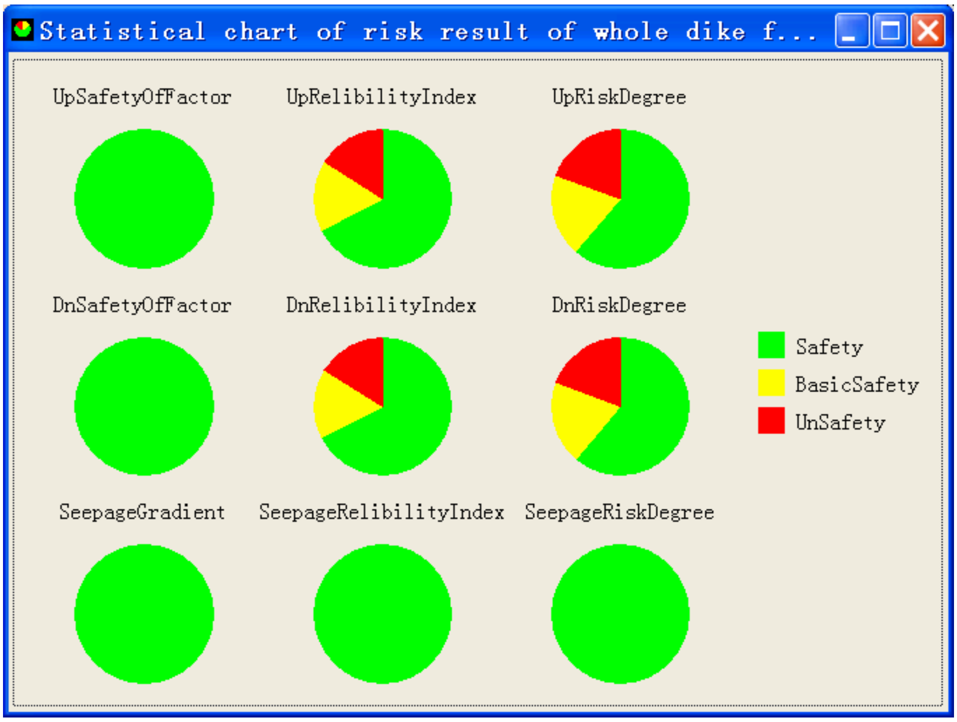
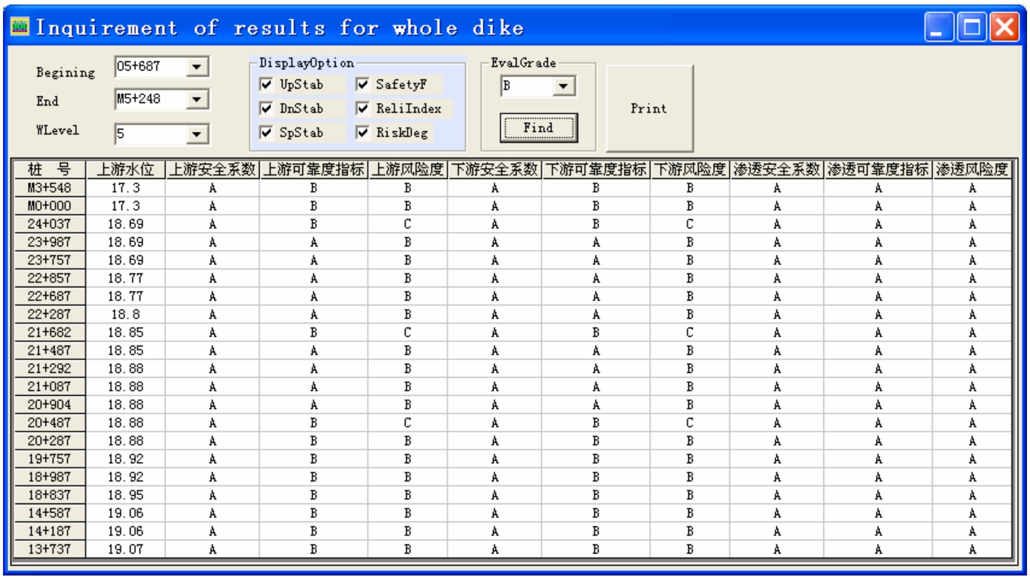


Figure 10.22 Statistic pie chart for different modes



桩　号	上游水位	上游安全系数	上游可靠度指标	上游风险度	下游安全系数	下游可靠度指标	下游风险度	渗透安全系数	渗透可靠度指标	渗透风险度
M3+548	17.3	A	B	B	A	B	B	A	A	A
M0+000	17.3	A	B	B	A	B	B	A	A	A
24+037	18.69	A	B	C	A	B	C	A	A	A
23+987	18.69	A	A	B	A	A	B	A	A	A
23+757	18.69	A	A	B	A	A	B	A	A	A
22+857	18.77	A	A	B	A	A	B	A	A	A
22+687	18.77	A	A	B	A	A	B	A	A	A
22+287	18.8	A	A	B	A	A	B	A	A	A
21+682	18.85	A	B	C	A	B	C	A	A	A
21+487	18.85	A	A	B	A	A	B	A	A	A
21+292	18.88	A	A	B	A	A	B	A	A	A
21+087	18.88	A	A	B	A	A	B	A	A	A
20+904	18.88	A	A	B	A	A	B	A	A	A
20+487	18.88	A	B	C	A	B	C	A	A	A
20+287	18.88	A	B	B	A	B	B	A	A	A
19+757	18.92	A	B	B	A	B	B	A	A	A
18+987	18.92	A	B	B	A	B	B	A	A	A
18+837	18.95	A	B	B	A	B	B	A	A	A
14+587	19.06	A	B	B	A	B	B	A	A	A
14+187	19.06	A	B	B	A	B	B	A	A	A
13+737	19.07	A	B	B	A	B	B	A	A	A

Figure 10.23 Detailed inquiry results of the Grade B

10.5 Scenario of strengthening according to structure risk analysis

The strengthening scheme of construction after the hazard depends on the engineers' subjective decision for long time. Thus, there are three kinds of results after strengthening or rebuilding: one is that the measures is appropriate and the target is accomplished; two is that some excessive measures have been taken, and it results in increasing of cost; three is that insufficient measures have been taken, which can not meet the demand of bearing capacity of the failure structures and then a potential incipient fault of safety still exist. Safety degree or bearing capacity of some structures can be diagnosed by systematical computing and scientific analyzing.

When the safety grade of a specified dike section is Grade B or Grade C, the responding strengthening prediction scheme can be presented by this system. Till now, there are the following schemes (under updating):

Case 1: Some observation of pool level and settlement and river-bed scouring should be enhanced, and upstream revetment should be reinforced.

Case 2: Berm and blanket should be built.

Case 3: Some observation of piping failure and relief-well water level should be enhanced, and seepage control measures should be taken.

10.6 Estimated damage

10.7 Suggestions to reduce the risk according to the structural risk analysis

10.7.1 Reducing the sliding problems

10.7.2 Reduction of the risk by a set of measures

10.8 Conclusions

Some conclusions can be drawn from this case study on the entire dike section:

(1) For statistic characteristic of safety indexes at various water levels for the whole dike, various evaluation index for safety assessment on more than fifty dike subsections can meet the design standard according to the current code, and the probability of flooding of potential damage is considerable little, especially in seepage instability. The safety factor and reliability index of dike stability increase as the decreasing of the water level, but the risk degree of instability decrease. The safety indexes of outer slope stability are lower than the one of inner side, so the monitoring of the revetment or bank collapse in outer side should be enhanced during the flood season. The probability of flooding of sliding decreases by around 0.02% with a decreasing of water level for each 0.5m. For seepage instability, It can be seen that the probability of flooding reduce by 10 to 100 times when the

water level is decreased by 0.5m.

(2) According to the distribution of evaluation indexes for a whole dike at different water levels, the sliding failure mode play a principal role for the overall failure probabilities. The Section No. 05+687~08+687, and No. 40+439~40+687 turn out to be the weakest link regarding to the sliding results. These sections can be selected as the breach spot employed in flood simulation and damage assessment.

(3) At design water level, the overall probability of flooding on Section 05+687 is 0.1826, and the one of the entire section is 0.4382.

Chapter 11 Conclusions and Recommendations

11.1 Conclusions

In order to transform the traditional empirical methods of safety evaluation and management of dike to foreseeable risk management mode, it is important to predict and evaluate the categories of potential dangers, cause of danger occurring and consequences of accidents through more reasonable numerical models and calculation approaches. Probabilistic risk analysis approach is the up-to-date research field of safety assessment techniques of dike. Based on the theory framework of reliability analysis and risk assessment, the interactive software system of dike risk analysis operating on the Windows platform has been developed and used in a pilot area. Some conclusions can be drawn:

(1) A comparison has been given of the piping method. The comparison of the Sellmeijer's method with the empirical methods of Bligh showed that the Sellmeijer's method requires a shorter seepage length and produces a more economical design for water retaining structures. Chinese method also can give a reasonable design within a certain range, and the safety factor by this model is not very sensitive with some variables than the one by other models. An increasing curve, safety factor increase with the permeability coefficient of sand layer, has been gained by Bligh model, which is unreasonable.

(2) Some soil strength parameters are taken as random variable, two typical dikes, an inclination watertight facing dike, and the homogenous embankment have been employed in the studying of individual dike section, and the influences of various factors such as geotechnical statistic parameters and geometry of dikes on reliability index or structure risk degree for overtopping and piping and sliding, have been investigated systematically. Some numerical models and solving approach have been testified.

(3) The case studies are an important part of this research. They are initially oriented to obtaining information to increase insight into the flood risk of the whole dike section. Subsequently, the spatial distribution of multi-indexes, for instance, safety factor, probability of failure and probability of flooding along the entire dike has been obtained at a specified water level. The probability of flooding of potential damage is considerable little, especially in seepage instability. The safety factor and reliability index of dike stability increase as the decreasing of the water level, but the risk degree of instability decrease. The safety indexes of outer slope stability are lower than the one of inner side, so the monitoring of the revetment or bank collapse in outer side should be enhanced during the flood season. According to the distribution of evaluation indexes for a whole dike at different water levels, the sliding failure mode plays a principal role for the overall failure probabilities. The Section No. 05+687~08+687, and No. 40+439~40+687 turn out to be the weakest link regarding to the sliding results. These sections can be selected as the breach spot employed in flood simulation and damage assessment. The overall probability of flooding of the entire section is 0.4382 at design water level. In addition, the relation between the various evaluation indexes and different water levels shows that the flood water level has a great influence on the indexes, but not linear. The adjustment and modification on the flooding-protection water level can be proceeded for some subsections, since some larger safety margin in some sections has been checked.

(5) The new calculation method and software system SADSS were tested for the pilot, and it

shows that the method, works. In brief, risk analysis associated with traditional method used to find weak spots in the dike ring seems to be a very powerful tool. This method also shows the effect of gaps in our knowledge on water defences.

11.2 Recommendations

(1) The combination between more reliability functions should be made in the near future. Since the coefficients of influence in combination with the reliability functions point out what kind of maintenance activities are expected to be most effective, some results of this should be submitted in the further research. The method of combine the different components of a flood defence system should be further study and implement in the practical project.
(2) Some levees may be subjected to significant water heights for many months. When this occurs, the phreatic surface within the levee will rise, increasing pore pressures and increasing the risk of failure due to through-seepage, underseepage and slope stability. This is acknowledged in a rudimentary way by reducing the crest width when the levee is exposed to flood heights for only a limited time. If we can use some time-dependent analyzing model, especially for the seepage simulation, and nonsaturation slope stability, in the real-evaluating of the dike safe, it will be an essential development in this field and deep in diagnose of the disease of the dike during the flood season.
(3) As soil is a continuous medium, the appropriate characterization of uncertainty in a two-dimensional slope or seepage analysis is dependent on the size of the modeled area and free body. Application of spatial correlation theory to soil parameters in SADSS should be taken into account. Similarly, real levees may be many miles in length. Intuitively, a long levee is less reliable than a replicate shorter one.
(4) Computer programs like PC-Ring, SADSS are in single-machine mode, in some extent, there will be in trouble with updating and sharing of data-base and developing of flood emergency action and so on. Therefore, web-based or GIS-based software system would be promising. In addition, these program is “tailor-made”, in which the defence types and failure modes are fixed. Each time when the program is applied to a flood defence system with different defence types, the new failure modes must be imbedded in the program code. “one-size-fits-all” computer program should be set up.
(5) The failure modes are related to structural failure and are limited to the components in this case study: embankments without additional structures, and three failure modes with respect to structures. The failure modes of the wave return wall and relief-wall and anti-seepage wall and sluice should be added to the report.
(6) When the engineers proposes improvement of existing levees (typically raising the height), economic studies are required to assess the benefits and costs. My colleague MS Wang will concentrate on the economic analysis to assess damage that would result form flood according to some computing results mentioned-above. At that moment it will also be possible to calculate the costs and benefits of all alternative mitigating measures.
(7) To support a wide application of risk methodologies in flood defence an overarching risk-based framework is being developed that integrates decisions on different levels (e.g. national, large-scale, strategy, scheme, etc) and across differing functions (local authorities, flood warning, operation and maintenance, etc.). The flood probability is calculated for an entire dike ring area and not for a single dike section only, which can meet the demand in

some extents. In this way, a better insight into the protected area can be obtained, weak links in the dike ring become apparent and bottle-necks can be ranked from large to small. So our government should further carry out and support this kind of research work, especially in research from probability of exceedance to probability of flooding.

(8) If the risk analysis has to be improved, efforts have to be focused on gathering more, and more accurate data as input to the probability calculations. Especially the probability of failure of structural artifacts in the earthen dike, e.g. sluices, was based on possibly inaccurate assumptions. Although usual amount of information is available, lack of information occurs with respect to the following: the original experiment data of the geotechnical parameters, especially in the strength parameters and seepage coefficient. Thus, a number of basic variables with statistical distribution functions and correlations can be gained.

(9) As we known, the safety evaluation and risk analysis can be proceeded after gathering some first-hand materials, such as safety monitoring, Combining the monitoring data with evaluation models is a promising research field.

(10) Finally, as mentioned above, the guideline for dyke safety assessment is a very necessary tool for a dyke owner manager therefore it highly recommended organize a project to build up the legal guideline in China. However, an assess work can be taken a lot of money, thus depend on Chinese's economy condition at present then might be applying for each 5 coming years or 10 coming years for instance. As the Netherlands, they assess the safety of dyke once time in five years. In this way, the specific guides and reports published by the TAW provide the information will be used and recommended.

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Acknowledgements

There is an old saying in China: "Thousand kilometers' walk is started from under your own feet". However, without the constant help and support of so many colleagues and friends, this report would not come true.
In the first place, I would like to thank my supervisor Prof. Pilarczyk, on behalf of the Rijkswaterstaat of the Government of the Netherlands, who has granted the fellowship to me. It is so nice to work under his supervision not only because of his amiable personality but also because of the opening working atmosphere he provides.
I also owe a great deal of gratitude to my promoter Bas Jonkman. His long-lasting patience in helping me correcting and refine this report can be reflected in near every word. I will never forget my nearly-daily bothering to my co-supervisors Dr. Hoffmans in the beginning, his valuable suggestions and comments on comparison of piping mechanism will benefit for my further study life. DWW where I worked for this study is like a family for me. It was this family who made me feels warm and made my stay in Netherlands a pleasant experience. My sincere thanks are given to every member of this family and especially the supporters behind the stage. I am so happy to have these supervisors in one of very important periods of my life, even it is not too long.
I also would like to express my deep and sincere gratitude to Prof. drs. ir. J. K. Vrijling, professor of civil engineering faculty – Technical University of Delft. His valuable advice will be helpful for ever. By the way, his enthusiastic arrangement with Bas fills my empty experience on sailing and booting by fieldtrips. Many thanks to Ir. Ed. Calle, Dr. ir. Sellmeijer at GeoDelft for their advice, guidance and kind support during this study, talks with them still remain fresh in my memory.
Further more, I would like to express my great appreciation to Dr. ir. Van Gelder at TU Delft for his course on the probabilistic method.
Finally, I thank Bart Thonus from TNO for the demonstration of PC-Ring and Peter Blommaart from DWW for the introduction of dike breach in Wilnis with patience and English.
Part of the work described in this report was done in China associated with an ongoing practical project. My thanks are also given to my colleagues, Prof. Cheng Xiao Tao, Prof. Ding Liu Qian, Senior Engineer Sun D. Y., Zhang Jinjie. I would like to express my great appreciation to them for offering this opportunity. I owe a special word of thanks to Senior Engineer Xu Xiao Tong for her great effort to help us collect the data in China.
Thanks to Wang Y. Y., who has given me many interesting discussion, and Ma Jing, my colleagues, and my Chinese friends Zeng Xianghui, Ren Songchang, Teng Bin, Zhao Weimin and Chang Xiao Tao. Thanks must be given to all the people, who have directly or indirectly provided help and support to me during my work here at Delft.
Thank you all,

DWW, Delft, The Netherlands
Oct., 2003

Curriculum Vitae

Xingzheng WU was born in ShanDong province of China on February 29, 1972. In 1990 he finished his high school education at Lianshan high school, Jining, Shandong. In the same year, he entered ShanDong University, Jinan, ShanDong. After four years of study, he got his B. Sc. in 1994. In 1997, he obtained his M.Sc. from the same university with Solid mechanics. The title of his master thesis is “Study of Geosynthetic Reinforced Soil Embankment”. In 2001, he got his Ph.D degree from Dalian University of Technology, Dalian, Liaoning province, China. The title of his doctor thesis is “Constitutive Models of Coarse-grained Soils with Static and Dynamic Loading and Their Applications in High Concrete Faced-slab Rockfill Dam”. In parallel to his study, he worked as a research assistant in the National Science Foundation project of China from 1997 to 2001 and in the 9th Five-Years Plan Project from 1999 to 2000. In these projects, he was active on constitutive model and numerical approach in 3-D finite element analyses.

At the beginning of 2001, he started working towards new research field in the probabilistic design and risk assessment of flood defences at China Institute of Water Resources and Hydropower Research (IWHR), (Research Center of flood and drought disaster reduction of Ministry of Water Resources). And in the subproject of WORLD BANK and Youth Acceleration Fund of National Electric Power Corporation project and some practical engineering projects, he developed the software system of safety evaluation with multi-indexes on flood defences.