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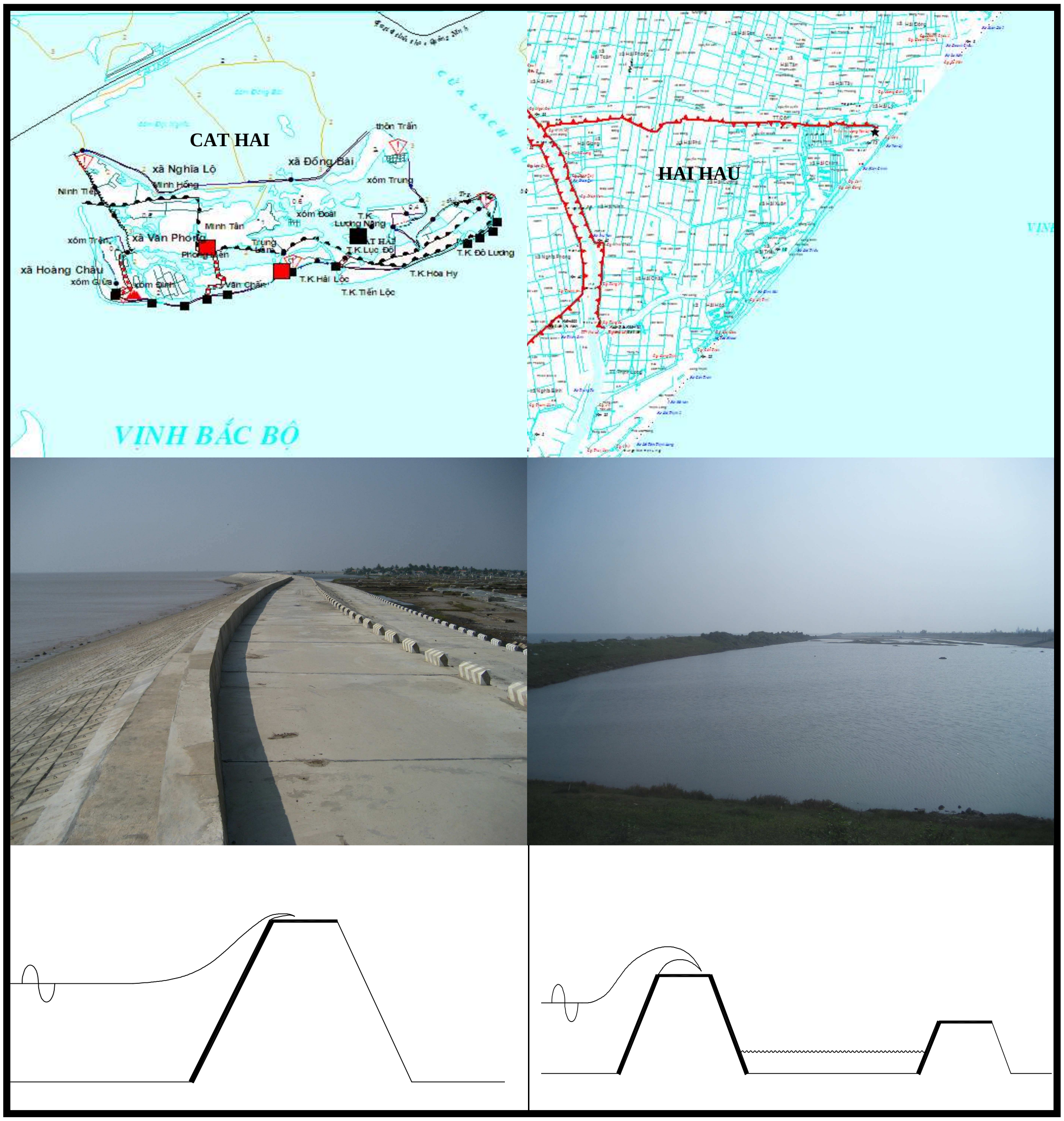

The Application of a Tandem Dike System in Vietnam

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THE APPLICATION OF A TANDEM DIKE SYSTEM IN VIETNAM

Master of Science Thesis

by

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ABSTRACT

In the low-lying coastal regions coastal defence structures are usually designed with a main function to protect the hinterland from highly vulnerable to coastal flooding. Sea dikes are usually the most common and important elements which form the coastal flood defence system. Sea dikes are designed at a pre-defined circumstance and requirement. For instance, dikes are designed where no overtopping; some overtopping; or large overtopping water is allowed. The most interesting issues at the preliminary design stages of a sea dike are its height and layout and associated overtopping discharge criteria. There have been many discussions on whether sea dikes should be designed high enough to not allow any overtopping water or using relatively low but "strong" dikes in order to allow for some to large overtopping water. First option leads to a very high and big dike as, for example, as in most sea dikes in the Netherlands. The second option needs a transitional area to store the overtopping water or to have a way to release and/or collect the overtopped water as well as a proper resistance of upper and inner parts of the dikes to avoid erosion due to overtopped water. Recently, the ComCoast project develops and demonstrates innovative solutions for flood protection in coastal areas. In which different types of dike cross section and layouts of defensive system were proposed. Concepts of defensive zones are presented beside the already existing concept of defensive lines. However, within the ComCoast[1] project, attentions are paid mainly to developments of the innovated concepts rather than focusing on comparison of the proposed system with the conventional sea defences. Thus, there is still lack of guidance and comparative tools to determine the best choice amongst the conventional or innovated options in decision making process. This study focuses on development of the comparative framework and generic guidance to support the decision making process in selection of the best suitable layout option of the sea dike system for a given location. Based on overtopping criteria two situations are considered for the analysis: (i) using one defensive line; (ii) using two defensive lines (defence zone). The multi-criteria analysis, which takes social, economic, environment and technical aspects into account, and the cost-benefit analysis, are used in the selection of the best option. Finally application is made for several case studies of coastal flood defence in Vietnam.

[1] ComCoast - COMbined functions in COASTal defence zones - is a European project which develops and presents innovative solutions for flood protection in coastal areas.

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CHAPTER I INTRODUCTION

1.1 Background information

Sea defense systems are of interests for many nations all over the world, depending on the nature, climate, topography characteristics and development states these systems are at different levels. Sea dike defense systems are built to protect the low-lying coastal regions/countries from sea floods e.g. in Netherlands, Germany, Belgium, Bangladesh, China, Vietnam ... Dikes are also used in the case of land reclamation for urbanization or land expansion. Sometime dikes are needed in the case of fixing shorelines in erosion area, besides/ in combination of other shore protection measures.

Main function of sea dikes is preventing sea water to flood into the polder at a pre-defined circumstance and or criteria. For example, dikes are designed with no overtopped or some overtopped or large overtopped water is allowed. Therefore, at the preliminary design stages of sea dikes, the height of the dikes and its associated overtopping are most important. Experiences in many countries show that, for most of sea dikes, the damage by wave overtopping is a major failure mechanism. The actual situation of sea dikes in Vietnam supports the statement very well (Vinh et al., 1996; Vrijling et al, 2000; Mai et al, 2006). Wave overtopping leads to several consecutive failure mechanisms, i.e. erosion of dike crest, inner slope and dike body; breaking of crown walls; functional failures due to too much overtopped discharges, if duration and intensity of storm are large enough. So, the overtopping mode is the most important aspect which relates to the main function of sea dikes and it safety.

As consequence of climate changes the average sea level is expected to rise. This will increase hydraulic loads on coastal water defences. Until now, traditional dike improvement (step-by-step dike heightening) compensates for increasing hydraulic loads. However, with the ongoing rise in sea level, traditional dike improvement might not be the optimum way to strengthen coastal water defences. From both economic and technical points of view, alternatives are desired.

There have been many discussions on whether the sea dikes should be designed high and strong enough to not allow any water overtopped (it is the case for most of sea defence in The Netherlands) or using relatively low but “strong” dikes in order to allow some to large overtopped water. The first option leads to very high and big dikes, while the second option needs some additional components, in order to ensure the dike function and safety of the dike itself, such as: room to collect the overtopped water or way to release the overtopped water; proper resistance of upper and inner parts of the dikes to avoid erosion due to overtopped water.

As the first attempt, an international research project, named ComCoast, had been appointed within European countries which border the North Sea. Project objectives are to develop and demonstrate innovative solutions for flood protection in coastal areas. A new approach, known as ComCoast approach, has been conceptually proposed. The ComCoast approach is to search for alternative defence systems and new sustainable sea flood management strategies to cope with increasing sea loads. The chosen concept involves the use of a wide coastal area for water defence, which means a gradual transition from sea to land. This coastal transition area is referred to as a coastal defence zone, instead of coastal defence line. The concepts of “Crest Drainage Dike”, “Overtopping resistance dike” and “second dike” and “flood storage area” are mentioned. A number of studies have been done regarding to understanding of social, economic as well as physical insights and safety aspects of the proposed innovated concepts. Case applications have been done for several locations along the North sea [DHV, 2005]. However within the project much attention is put on developments of the innovated concepts rather than focusing on a comparison of the proposed system with the conventional sea defences. Thus, there is still lack of guidance and comparative framework to determine the best choice between conventional or innovated options for decision making.

In order to fulfil the gap, this study focuses on development of the comparative framework and generic guidance to support the decision making process in selection of the best suitable layout option of the defence system given a location. In which two situations will be considered for the analysis: (i) using one defensive line; (ii) using two defensive lines (defence zone). Aspects of social, economic, environmental and technical characters are taken into account to perform analysis criteria.

Finally application is made for the case studies of coastal flood defence in Vietnam.

1.2 Problem definition

For certain low lying coastal regions the main interest is how to protect the regions from sea floods. At present, different solutions may be considered which varied from conventional approach (using one dike line) to integrated coastal zone defence approach (as concepts proposed by ComCoast project). Questions that arise in the decision making process at a certain location are “which solution should be applied” and “why is that so”. To present date there is still lack of tools and knowledge to give a proper answer to the question.

In practice, at different places, the applications of both solutions have been done so far and the defense systems were constructed long time ago. Even though it is not clear why the solution came up. It is exactly the case for Vietnam coastal flood defence systems. For instance, along the coastlines in the North of Vietnam, sea dikes are used as a single defence line for Quang Ninh and Hai Phong province. However, in Nam Dinh and Thai Binh province the dike system consists of two defence lines. The choices of dike configurations as indicated above, were probably rather arbitrary or by local expertise with lots of trials during the time. Thus, there is a need for assessing effectiveness of each existing solution.

1.3 Study objectives

To reconsider/reconstruct dike system in Vietnam, which are often in need for repair, one can choose between a traditional one high dike system or a combination of two dike with land in-between and with varying heights and strength. This study presents a generic approach to the various options in order to help making the right decisions for defence system construction in various cases. Beside this, alternatives for coastal defence systems are an important topic at the moment (see the EU ComCoast project).

Considerations are not only related to technical points of view, but also need to meet the societal needs of the people living in protected area, the economic development and the environmental aspects when the defence solution is applied. This study covers the following aspects:

- Choice of layout of flood defense system, when and where the one defensive line or two defensive lines should be applied;
- Finding the optimal dike height for each layout;

- Cost – Benefit Analysis is taking into account environmental and societal aspect;
- Application of methods to the case study in Vietnam.

The study results aim at providing important basic for establishment of guidelines in sea dikes design in terms of selection of layout (master plan) and dike heights. Specifically based on the analysis results of the case study in Vietnam, guideline for sea flood defences in Vietnam, which take into account country specific aspects, are proposed. Besides that, lesson learnt from existing Vietnam sea flood defences is thought to be useful for other low-lying countries/regions where sea dikes are needed.

1.4 Methodology and study approach

In this study the following steps and methods are adapted:

- Review related studies and literatures;
- Establishment of a generic framework to support decision process in selection of a suitable layout of the sea dike system on the basis of multi-criteria analysis and cost-benefit analysis;
- Risk based approach to establish relation of dike heights and admissible/permissible wave overtopping discharge, given different scenario of safety standards;
- Establishment of sea boundary conditions of the case studies which serve the dike design (wave and water level at the design conditions);
- Demonstration of the approaches by various case studies in Vietnam.

1.5 How to read this report

This report has seven chapters. In which Chapter 1 introduces background information of sea defense system in the world, defines problem of sea defense system, the objectives, the methodology and question is how to approach of this study also explain in Chapter I.

Chapter II introduces background information and development of coastal flood defense system in Vietnam.

Boundary conditions in Northern Sea in Vietnam are introduced and analysed in Chapter III. More detail of analysis is concentrated in Nam Dinh coast and Hai Phong coast.

Chapter IV explains how to choose one defense line or two defense lines based on multi-criteria evaluation. Wave overtopping is considered as the dominant failure mode to determine the height as well as the layout of the sea defence system.

Risk based cost benefit analysis of alternative defense options are introduced and explained in Chapter V. The aim of this chapter is choosing the best solution amongst given alternatives.

Risk based approach for determination of safety standard is introduced in Chapter VI. This Chapter is coming up to answer the question how is safe enough for a current coastal zone. Nam Dinh coastal zone is chosen as an application case of the approach in this chapter.

Finally, conclusions and recommendations of this study are given in Chapter VII.

CHAPTER II VIETNAM COASTAL FLOOD DEFENCES: BACKGROUND INFORMATION AND DEVELOPMENT

2.1 Overview of Vietnam water defences

Vietnam is situated in the tropical monsoon area of the South East Asia and is a typhoon-prone country (Figure 2.1). A large number of populations involved mainly in agricultural and fishery sectors are situated in the low lying river flood plains, deltas and coastal margins. Also, the important ports are located along the coast. On the other side, these areas are the most important potential disaster areas for Vietnam. Typhoons from the South China Sea bring torrential rainfall and high winds to the coast and further inland. On average six typhoons attack the coast annually. The monsoon season coincides with the typhoon season, resulting in heavy damage, loss of life, and destruction of infrastructure facilities and services. One reason that water disasters are so serious is that most of the population lives in areas susceptible to flooding. The main population centres and intensively cultivated lands in the Red River and Mekong Deltas and the narrow connecting coastal strip of the country are particularly vulnerable to flooding from monsoon rains and typhoon storms. Thus flooding is the most important potential disaster facing Vietnam.

In the past, because of lack of funding and lack of access to technology (proper expertise and technical facilities) the methods of dike management and flood control were constrained to simple and cheap techniques such as manual labour, training in dike inspection, procedures for mobilizing the population during emergencies, and strategic positioning of supplies for emergency repairs and for providing relief after a disaster.

The importance of flood mitigation and control has been recognized as fundamental for the continued development and prosperity of Vietnam. In order to minimize the flood damage, the Vietnamese Government is paying great attention to considering both structural (e.g. dike construction) and non-structural flood mitigation (flood plain management and regulations), flood-and disaster-warning systems, emergency preparedness and disaster management.

Actually, Vietnam is seeking to build up the modern system of dike management and flood control based on new design methodologies and technologies, such as using more advanced design and construction methods for flood mitigation (often adopted from abroad), in combination with state-of-the-art warning and forecasting systems to improve the emergency preparedness, and by the construction of storm-proof shelters for disaster management. These objectives are incorporated into the current Vietnam Development Plan, where safety against flooding is recognized as a fundamental requirement for all forms of development

In particular, there is great emphasis placed on strengthening and improvement of the entire system of river and sea dikes to prevent flood damage. Structural measures for flood mitigation consist of the rehabilitation of old dikes or construction of new ones, flood-control structures, and reservoirs. The importance of dike safety has long been recognized as fundamental for the continued development of Vietnam. Vietnam has approximately 5000 km of river dikes, 1000 km estuary dikes, and 2000 km of sea dikes providing protection against flooding. River dikes have been constructed, maintained and enlarged over many centuries using local materials and manual labour. These dikes are essential for the protection of the infrastructure of Vietnam, and rice agriculture would be impossible without the dikes.

In the Red River Delta in the north, people have built 3000km of river dikes and 1500km of sea and estuary dikes to protect them against flooding. Many of these dikes are old and built of poor materials using inadequate manual construction technology. Dike foundation conditions and stability have not always been properly evaluated before construction or improvement. River dikes often suffer damage from under-seepage and piping, slides or local collapse during high flood stages. Further, the construction of dikes has gradually reduced flood plain areas which used to be available for excess flood flows, with the result that river-flood levels have become higher and higher. There are now many areas where the flood waves are as much as five or six metres higher than the land protected by the dikes. However, in general, much more attention has been paid in the past to the river dikes than to the sea dikes and, in the consequence, the condition of the river dikes is usually much better than those of the sea dikes.

Along the coastal plains in northern and central part of Vietnam, the people have built sea dikes to protect themselves against extreme events such as typhoons. These dikes are essential for agriculture in the central provinces. When these dikes are overtopped or breached, the fields are flooded by saltwater and rendered unproductive for years. If this happens frequently, there are not sufficient resources in the community to repair or upgrade the dikes, and the people become impoverished and prone to malnutrition. In some parts, the attacks on coastal lands by the sea have been aggravated by water-resource structures, particularly dams, which have interrupted the natural supply of sediment to the coast. This has sometimes caused the coastline to recede by as much as one kilometre in 20 years. Houses and villages have been destroyed and people have been killed.

Sea-dike systems are still primitive and placed usually on a shallow foreshore and therefore relatively small. After every typhoon landing, a great deal of earth, sand, clay and stone from the sea dikes are washed away. Local people normally use their own resources to repair the broken dikes with the same soil that has been washed away. This process is repeated again and again for each typhoon landing. In areas of structural coastal erosion the shallow foreshore is gradually eroded, the water depth in front of a dike increases allowing development of higher waves, and in the consequence, the dike will break through. Because of the absence of the proper water born techniques allowing the repair of the dike from the seaside, the dike is usually offered to the sea and the new dike is constructed more inland. This retreat strategy is followed already for centuries leading to loss of the valuable coastal areas.

It is clear therefore, that sea dike rehabilitation and construction is a fundamental concern for flood mitigation in Vietnam. Vietnam also recognizes the need for non-structural flood mitigation. Important objectives are land-use management, safety standards, flood-forecasting warning systems, public information and training, hazard mapping, flood proofing and house raising, property acquisition, and education (capacity building). To realize this new approach it is needed to build in future on the new (up-dated) design and construction methods, however, including the own past experience related to the actual Vietnamese situation.

The actual situation in Vietnam is under continuous changes, which should be taken into account when preparing a new strategy/approach. There are in fact a number of new influences:

- Changes resulting of moving from a planned to a market economy.
- The population is increasing rapidly, producing the pressure for more land.

- Many sections of the dikes are reaching bad conditions.
- There has been substantial environmental degradation, such as shore line erosion; destroy of wetland areas due to aquaculture activities…
- The country may already experience the first effects of global warming. According to the IPCC Fourth Assessment Report 2007 due to global warming (0.1 to 0.3^{0}C increase per decade) lead to increase in occurrence of extreme rains causing flash floods in Vietnam. Also increase in hot days and warm nights and decrease cold days and nights.
- The traditional methods of disaster mitigation are less effective than they used to be.
- There are now new techniques available.

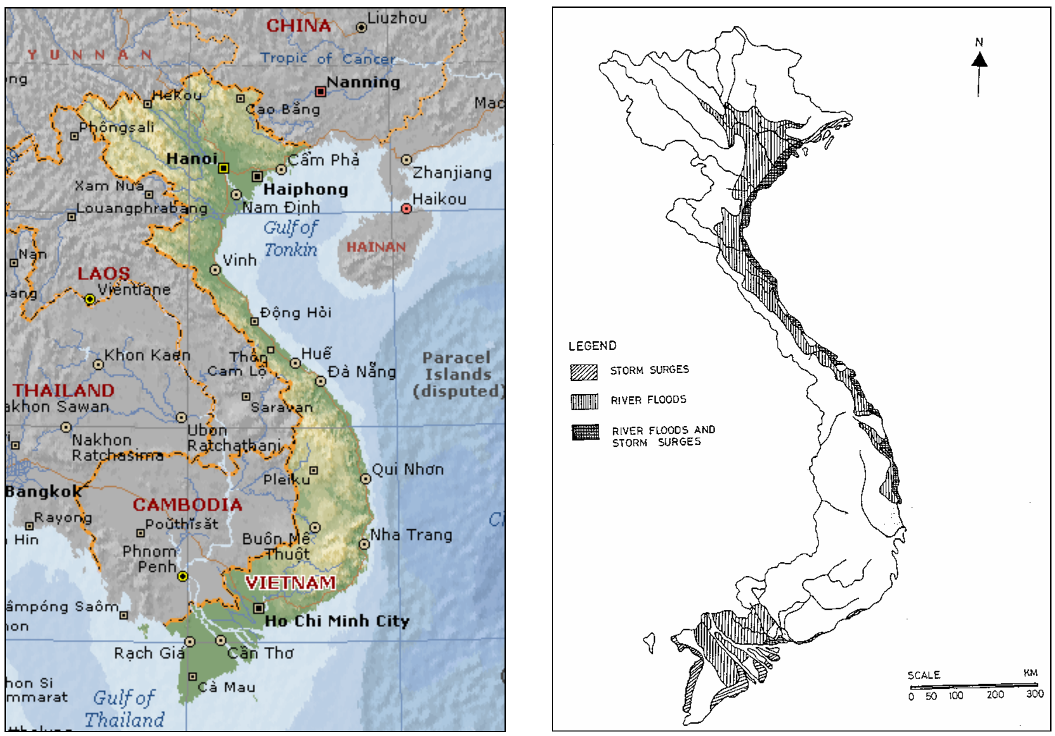


Figure 2.1: Flood situation of Vietnam

According to economic planning, the coastal area is a main location for economy and contributes a big mount of money to national economy and national security. It contributes to the progress to take and develop the main economy areas, industries area, activities for travelling, aquaculture, agriculture, making salt and restore trade village. It means that both the infrastructure of coastal area and sea dike system should have multi-purpose land-use that includes flood control, salt intrusion control to ensure the safety for people and production activities. It also use for transportation which serve for economic development, travelling and international security. Sea dike system is necessary to protect against failure, so it is upgraded and enlarged, that steps support more capacity for flood control which impulse the development of economy and ensure the sustainable developing at coastal areas.

2.2 Attentions to coastal flood defences in Vietnam

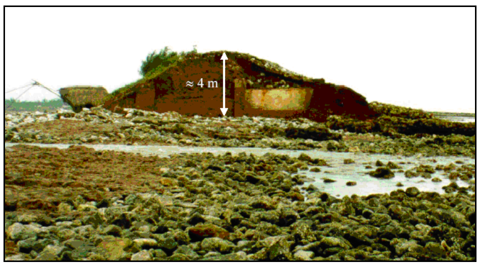
The Vietnamese coasts are frequently hit by typhoons: on average six typhoons attack the coast annually. Much damage is caused by the wind itself and the rainfall that comes afterwards, but at low lying parts of the coastal zone, flooding from the sea is also a serious threat. These areas are protected by dikes of which the quality is largely insufficient to cope with this threat. This becomes evident in frequent emergency repairs, but much worse, with occasional flooding of coastal areas [TLTK].

Vietnam is affected regularly by substantial suffering due to floods. The most severe floods occur during high river discharges and during, and shortly after, typhoons. The typhoons are accompanied by torrential rains causing flash floods which regularly submerge low-lying areas. The deltaic coastal area to a distance of about 20 km behind the sea dikes is threatened in particular because of the combined occurrence of storm surge from the sea and high river discharge at high tides. As a result of these severe loads and the rather low safety level of the present dikes, the water defense system of Vietnam fails regularly. Since 1996, Vietnam was affected by several flood disasters, each of those responsible for the loss of hundreds of lives and considerable damage to infrastructure, crops, rice paddy, fishing boats and trawlers, houses, schools, hospitals, etc... The total material damage of the flood disasters in these years exceeded USD 500 million, which damage was accompanied by the loss of almost over 1000 lives. The flood disasters occurred in North Vietnam (1996 and 2005), in South Vietnam (1997) as well as in the Central provinces (1998, 2005). Most floods were initiated by typhoons and occurred in the coastal zone.

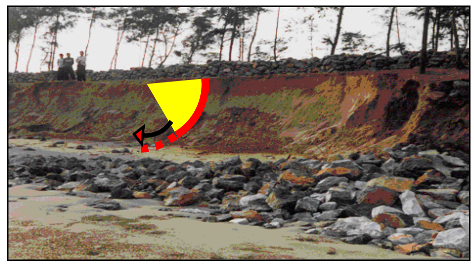
The relative low safety level of the Vietnamese dikes was investigated in 1996 during two visits of Dutch missions to Vietnam. Most designs of the Vietnamese sea and river dikes are based on loads with return period 20 years or even shorter periods. Compared to the Dutch standard (return periods 1000 to 10000 year) these return periods are very small. Besides this fact, the Dutch mission marked most Vietnamese dike designs as poor and disputable (Vrijling & Hauer 2000). Designs are not always based on the right formulae, hydraulic boundary conditions are not always based on proper statistics (for example design water level and wind set up are not always treated as two independent phenomena), failure mechanisms which differ from overtopping are often neglected, no attention is given to length effects, monitoring and timely repair of small damages is often at a poor level, etc. As a result, the true probability of failure (probability that the load is larger than or equaled to the resistance of the flood defence system) of the Vietnamese water defense system may exceed the design frequency. Although designed to fail only once in 20 year of this water defense system might well fail almost every year. The experiences in last 10 years support this assertion.

a. Failure of the toe structure

b. Damage of slope protection



d. Dike breach



c. Slope instability

Figure 2.2: Failures presented at NamDinh sea dikes [Namdinh dikes 1996, Mai et al 2006]

Figure 2.3: Sea dikes failures presented at Hai Phong coast [Hai Phong 2005]

During the year 2005, which showed some severe typhoons, almost 60 kilometers of sea dike were severely damaged or destroyed;

Year 2005 can be treated as a historical year in respect to disastrous impact of typhoons on sea defences in Vietnam. In total 8 typhoons hit the Vietnamese coastal zone in this year, resulting in human casualties and large economic damage. Typhoons No.2 and No.7 (Damrey) were of exceptional strength and belong to the heaviest typhoons in the last 3 decades.

Typhoon No.2, on 31 July, hit mainly the coastal area of Haiphong resulting in a number of kilometers of damaged sea dikes, especially on the island Cat Hai, where 8 km dikes were broken and/or heavily damaged and need total rehabilitation.

Typhoon No.7, on 27 September, hit 3 provinces, Thai Binh, Nam Dinh and Thanh Hoa. The damage was enormous; 25 kilometres of dikes were broken and nearly totally destroyed. In Nam Dinh a stretch of 800m sea dikes was completely washed out. The typhoon affected to about 1 million people, estimated direct loss is about 500 million USD.

Figure 2.4: Consequences of the Damrey typhoon in SEP. 2005. Picture on the left shows a sea dike breach at Hai Hau district while the right one illustrates sea flooding of the protected regions behind the dikes

2.3 Problem induced a wish for new safety policy establishment

Damrey typhoon in 2005 is considered as a turning point for safety policy towards sea defences and flooding prevention of Vietnam, similar as the year 1953 was a turning point for the Netherlands in formulating the new policy and safety standards for protection of the country against flooding. In attempt to rehabilitate the sea dike system in a long run a huge sea dikes program has been established by Ministry of Agricultural and Rural Development (MARD). The sea dike program is implemented for 2005-2015 period and appointed with two important tasks: (i) researches on safety standards, boundary conditions and finding optimal solutions for sea defences along the whole country; (ii) design and construction new dikes, at places where sea dikes has not been existed or were breached, and reinforcement of the existing dikes on the basic of findings in the first task.

At the same time, as an emergency solution to close the breached gaps due to Damrey typhoon in 2005, a new sea dike cross section was designed and approved by MARD in 2005. Coastal zone of Hai Hau district, Nam Dinh province and Cat Hai island, Hai Phong province were selected as a pilot location. Construction works took place in 2005 and had finished in 2007. However design the new dikes is still based on existing safety standards (design frequency of 1/20 year), which is known as out of date. It is necessary to check for consistency of the new constructed dike system at the pilot locations to see if the current rehabilitation works provides enough safety given present situation and if safe is safe enough for current Vietnam development. Findings are important input contributing to the first task of the sea dike program of Vietnam, which aims at providing design guidelines for sea defences.

2.4 Selection and description of the case studies

As mentioned on section 2.3 Hai Hau and Cat Hai coastal regions were selected as pilot locations for emergency response by MARD of Vietnam. These locations are also selected as case studies in this research, because of two reasons:

- Data is made available for these locations by pilot projects and it can be accessed.
- Usefulness for Vietnam since this study is an independence.

Descriptions of the case study areas are in the next sub section.

2.5 Coastal zone of Nam Dinh province and current situation of sea dike system

The coastal zone of Nam Dinh is roughly 80.000 hectares in size and has about 70 kilometres of coastlines. These Nam Dinh coastlines are considered as the most dynamic part of the coastal zone in the Red River delta. This coastline is naturally divided into 3 sections by 4 large estuaries: the Ba Lat (Red River main reach), Ha Lan (So River – has been cut-off some ten years ago), Lach Giang (Ninh Co River) and Day (Day River). From the North to the South these are: Giao Thuy section: from Ba Lat estuary to So estuary belonging to Giao Thuy district, about 27 kilometres; Hai Hau section: from So estuary to Ninh Co estuary, belonging to Hai Hau district, 27 kilometres; Nghia Hung section: from Ninh Co estuary to Day estuary belonging to Nghia Hung district, 16 km long (see Figure 2.5).

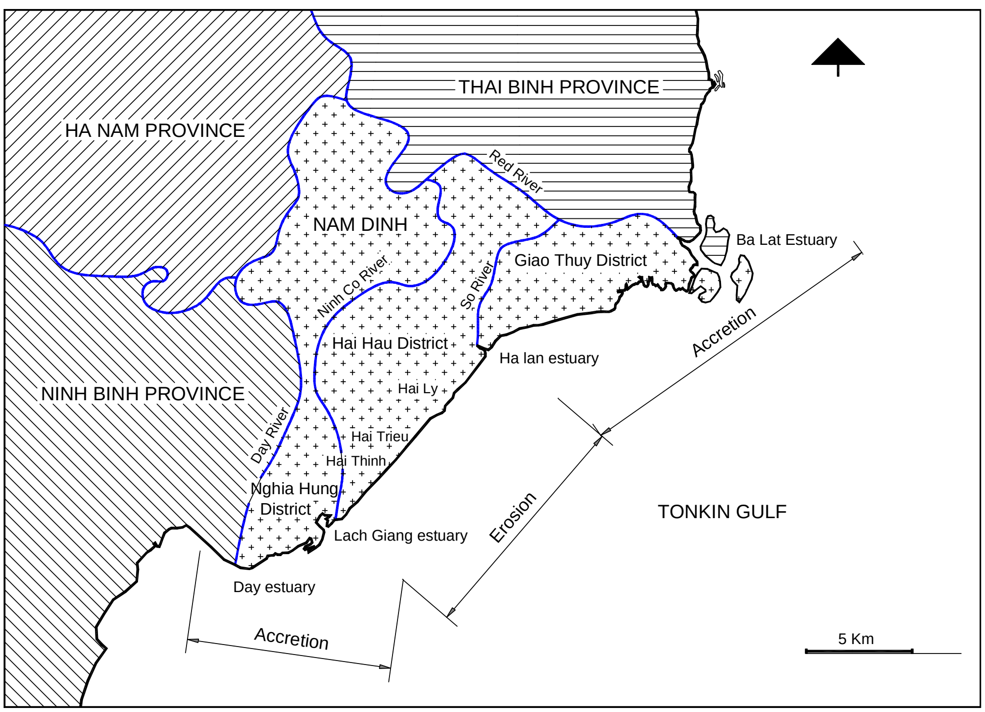


Figure 2.5: Nam Dinh coastlines and its current situation [Source: PDD Nam Dinh, 2003]

In Figure 2.5 we observe two accretion areas, one north (Giao Thuy district) and one south (Nghia Hung district) of the eroded area of Hai Hau district. The main reason of accretion of these areas is due to the rivers provide large volumes of deposited silt every year, which accretes and enlarges the beaches by hundreds of meters. It should be noted that accreted areas usually are stabilized by local people reducing the natural distribution of sediment along the coast. This could have accelerated the erosion rate in the eroded areas of Hau Hau. The section from Hai Ly commune to Hai Trieu commune is at present subject to the most drastic erosion. At that section, the eroding speed is 10 to 20 m per year. In eroded areas the beach is rather narrow with approximately 100 – 200 metres at low tides. According to the records of the Provincial Dike Department (PDD) of Nam Dinh province, the total averaged retreat distance during the last 60 years (from 1952) at the Hai Hau coasts is estimated at 1100 metres. Main reason for that is due to unbalance of long shore sediment transport (Luong Giang Vu, 2003, Mai Van et al. 2006). The coastline erosion results in serious economic consequences such as: loss of cultivated and residential land, adverse impacts on the production establishments, damages and threats to the coastal dykes and embankments and other social consequences to coastal communities, especially the communities of the Hai Hau district, as a result of retreat and resettlement. In order to restrain the possible adverse consequences of the coastline erosion, efforts have been made by both central and local authorities, to concentrate and mobilise financial and human resources for dyke and embankment construction, erosion prevention and protection, and for the resettlement of the people who have to move away. However, such efforts still remain limited to reactive and temporary measures, due to the budget constraints and the lack of a strategic and long-term solution [VNICZM project, 2000-2006]. Figure 2.6 is showing the T-groin system to mitigate the erosion of coast line in Hai Hau coastal zone.

Figure 2.6: T-groin system protects erosion along Hai Hau coast in Nam Dinh

The actual sea flood defence strategy in Nam Dinh province is based on a sea dike system with multiple defensive lines (often two defensive lines). The dike system is typically positioned as shown in Figure 2.7 with two defensive lines and separated sections by sub-crossing dikes. The reason behind using such system is that when a breach takes place at the main dikes (first lines) the sub-crossing and second dikes can limit flooding areas and become a new first line of defence of the system. The design distance between two defensive lines is about 200 metres. The land areas between the dikes are also divided into sections varying between several hundred meters up to 3 km by sub-crossing dikes (Mai et al 2007).

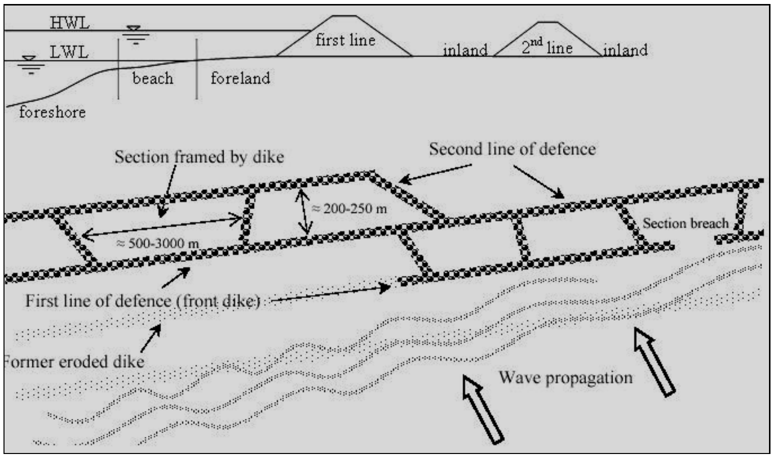


Figure 2.7: Schematized cross section and plan view of the coastal flood defence system in Nam Dinh province

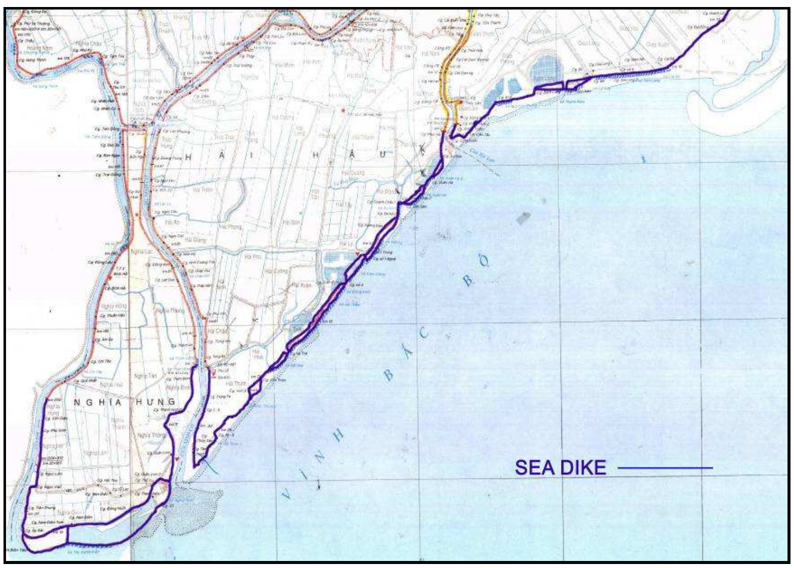


Figure 2.8: Position of existing sea dikes in Nam Dinh

From personal communication with local sea dike managers in Nam Dinh, it is understood that, using two defensive lines system, aims at isolating the sea flood water when a breach takes place at the first dikes. And the second dike is becoming the new first line of defence in case of breach occur. In general, the second dike is mainly made of soil with no slope protected revetments and thus it is weaker than the first dike. However, these dikes must be reinforced when the water reaches them; otherwise they will no longer last.

Configuration of the dike cross section: the front slope of the dikes in NamDinh province is from 1:3 to 1:4 and the crest elevation lies around 5 to 5.5 meters refers to CD (chart datum). The dike body, earth core, consists of material from local sand and clay resources, which strongly affects the durability of the dikes since the fine soil is easily flushed out to sea. On top of outer slope the revetments were constructed of natural stones and/or artificial blocks on a layer of clay. A characteristic dike cross-section is shown in Figure 2.9. In total, dikes protect 95 % of Namdinh coastlines.

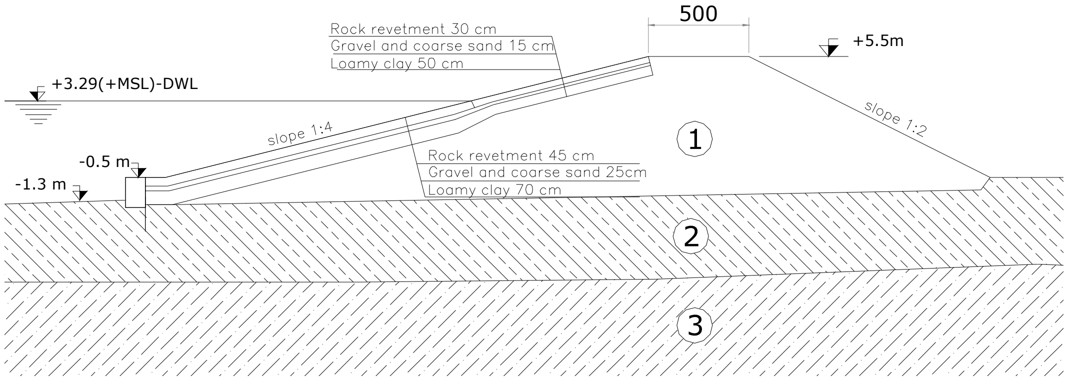


Figure 2.9: Representative cross section of sea dikes in Namdinh

Design of cross section of Nam Dinh sea dikes by applied the current standard 14 TCN 130 – 2002 of Vietnam:

- Design tidal water level MSL + 2.29 m (probability of 5%) storm surge from calculations by formula and observations (+ 1.0 m) design water level MSL + 3.29 m
- Crest freeboard 0.21 m;
- Calculated crest height MSL + 5.50 m (slope 1:4);
- Dike profile: seaside slope 1:4; landside slope 1:2; crest width 4 m;
- The sea slopes are protected by pitched stone revetment:

  Below MSL + 3.5 m the thickness of 45 cm (calculated formula for rock revetment); block dimensions 0.50 x 0.50 x 0.45, the average weight is approximately 250 kg;

  Above MSL + 3.5 m the thickness is of 0.30 m.
- Layer of gravel has the thickness of 25 and 15 cm, and layer of loamy soil is 70 and 50 cm.

Figures 2.10 and 2.11 below are showing the current situation of sea dike defence at Hai Hau coastal area in Nam Dinh province.

Figure 2.10: Current situation of the primary sea dike along Hai Hau coast in Nam Dinh

Figure 2.11: Residents are living behind the sea dike system in Hai Hau coastal zone in Nam Dinh

Economic sectors and developments in the coastal zone of Nam Dinh

Nam Dinh is the mainly agriculture production area of Red River valley. Annual production of agriculture is about 1 million tons that mean of 506kg of food per one person per year. The average development rate of agriculture production in Nam Dinh is reaching of 3.46% in period of 2001 to 2005. The career structure of agriculture has changed in the way of decreasing of farming growth rate (farming growth rate is of 75.12% in 2001 and 69.03% in 2004). On the other hand the growth rates of the cattle-breeding and agricultural services are being developed (e.g. these growth rate in 2001 and 2004 are of 22.7% and 26.8% respectively). In general, the agricultural production in Nam Dinh has developed of stabilizing with rice productivity of 12tons/ha/year and average seedy plants productivity of 106,000 tons/year. However the agricultural development in Nam Dinh has not formed some area specializing in the growing of agricultural crops yet (*Source: Development strategy of period 2001-2010 of Agriculture and Rural Development Department in Nam Dinh province*).

With the length of coastline of 70 km Nam Dinh has rich potentially of aquaculture and fishery. In recent years Nam Dinh has paid much attention on aquaculture production and fishing. Annual aquaculture productively of Nam Dinh is of 19,655 tons. Fishes exploiting productively is reaching to 23,180 tons/year (*Source: Rural Development & Agriculture Department of Nam Dinh province, 2006*).

Salt production is a traditional career in Nam Dinh province. The annual salt production is reached of 89,895 tons with the salt productive area is 886.91 hectares. The strategy development of salt production in future will be reached more than 90,000 tons in total production.

The coastal zone in Nam Dinh is of a rather flat topography with an altitude range from 0 to 6m above sea level. There are two large tidal flats in the Nam Dinh coastal zone. One is situated in Giao Thuy district and has an area of 12,000 hectares. The other is located in Nghia Hung district with the area of 8,000 hectares. These tidal flats present a typical coastal-wetland ecosystem, not only for the Nam Dinh province but also for the whole of Northern Vietnam. In terms of marine economic potentials, according to the assessments of scientists, the Nam Dinh coastal zone has a very rich biodiversity. The total annual sea-product exploitation output of the province ranges from 5,000 to 7,000 tons [VNICZM project, 2000-2006].

Nam Dinh has some long beach as in Thinh Long, Quat Lam and other small beach as Con Mo in Hai Hau district which could be developed in tourism in future. However due to erosion of these beach tourism has difficult in developing, it depend on the natural condition. Ecotourism of province is developing with the Xuan Thuy national park. This national park is formed in 2003 based on the international RAMSAR marsh land of Giao Thuy district which is recognized by UNESCO. Additionally landscape, cultural and historical monuments, and traditional trade villages have potential of tourism which can be developed in Nam Dinh province.

In survey the mainly careers in the coastal zone of Nam Dinh province are agriculture, aquaculture and salt production. Production of these careers makes up 82% of total production of coastal zone in Nam Dinh. Industries in coastal zone are almost not developing. Tourism is developing in step by step but it’s depending on the natural condition (see Figure 2.12).

Figure 2.12: Tourism activities at Hai Hau coastal zone in Nam Dinh

2.6 Coastal zone of Hai Phong province and current situation

The sea territories of Hai Phong are part of north-eastern water area of Gulf of Tonkin. The sea bed compositions and oceanographic characteristics of Hai Phong are closely connected to the common characteristics of Gulf of Tonkin and Eastern Sea.

The water depth of Hai Phong Sea is not large. A depth curve of 2m goes around Do Son Cape and then descends down to 5m far from the coast. At the sea bed where the rivers flow into the sea, the depth is greater due to erosion caused by currents. Further offshore, the level of sea bed descends to the depth of the Gulf of Tonkin of about 30 - 40m. The surface of Hai Phong sea bed is formed by fine components with many deep underwater canals which were used to be river beds and are currently used as daily navigation channels for vessels.

The length of Hai Phong coastlines is 125 km including the length of coast surrounding the offshore islands. The coast has a form of a concave curve as the common sea side of the Gulf of Tonkin. It is low and quite flat with the structure created mainly by muddy sand coming from the 5 major estuaries. At the center of the coast, Do Son cape stretches out into the sea like a peninsula. This cape is also the end of the range of mountains of Devonian sandstone running from the mainland. The highest point is 125m, and it stretched out 5km into the sea in the direction of North West - South East. This advantage of natural structure has given Do Son a status of an important strategic location in the sea and also a famous spot for its beauty. At the foot of the sandstone hills there are beaches making this place a romantic resort and a valuable convalescent area. In the offshore territory of Hai Phong there are many islands which are distributed over the sea with Cat Ba Island as the largest and Bach Long Vi Island as the furthest.

Sea, coast and islands have created special natural landscapes to this coastal city. It is also a special potential advantage of the economy of Hai Phong (see Figure 2.9).

Figure 2.9: Hai Phong province and its coastlines

The length of sea dikes is about 104.26 km, in which sea dike I (17.6 km), sea dike II (10.66 km), sea dike III (21.16 km), Trang Cat (19.72 km), Bach Dang (14.60 km) and Cat Hai (20.52 km).

Descriptions of sea dike cross section design in Hai Phong by applied the current standard 14 TCN 130 – 2002 of Vietnam:

- Design tidal water level MSL +2.29 m (probability of 5%)
- Distinguished alignment: Sea dike I, II and III; Trang Cat; Bach Dang; Cat Hai
- Storm surge calculated by formula: 1.1, 0.9 and 0.8 m
- Design water level: MSL + 3.39, 3.19 and 3.09 m
- Wave run-up: used only one angle for the wind direction to the direction of the dike: H_s = 1.82, 1.57 and 1.17 m

- Crest height: MSL + 5.5, 5.0 and 4.5 m. At the location where mangrove planning is possible: MSL + 4.5 m.
- Crest freeboard: 0.2 to 0.3 m
- Dike profile: seaside slope is of 1:3.5 – 4; land side slope 1:2; crest width 3 – 5 m
- Slope protection: pitched stone and concrete slab calculated by Russian formula and Hudson.
- Thickness of gravel layer is of from 25 cm to 37 cm, and thickness of sand layer is about 5 cm to 10 cm.

Economic sectors and developments in Hai Phong:

Hai Phong is in process of becoming a seaport city, a main gate to the sea, and an industrial, service, tourism and fishery centre in the North with a developed foundation of economy, education and training, technology - environment and infrastructure; a firm security - nation defense and further improving people's living condition.

Continue renovation process, develop the strength of the whole people; rapidly, robustly and stably develop the city, closely related to the economic development of economic focal points of the north; build Hai Phong to become a great city of the Nation.

Invest for rapid and effective development of advantageous and competitive industrial sectors with particular importance to in-depth investment and technology renovation. Focus on developing key sectors for leading products and traditional sectors of high economic value, mainly for export that makes contribution to the economic growth: strive for industrial growth target of 16% to 16.5% per year.

Agricultural production develops toward direction of foodstuff production, growing of plant, livestock of high quality, economic efficiency in connection with safety and sanitation for foodstuff.. Strengthen vital sea dikes; Expedite implementation and largely completion of the concretization program for irrigation channels. Rice productivity is over 11 tones per hectare. The value on each area unit is of 40 million dong/ha (2,000EUR/ha), an average increase of 2.2%/year. Cattle breeding reach 500,000 pigs, 20,000 cows and over 5 million poultry.

Development of fishery in Hai Phong is considered a key economic industry, a breakthrough and an objective to become a Fishery Economic Center of the North. Concentrate on implementation of 5 fishery development programs (catching, raising, processing, protecting the aquatic resources, and logistic services for fishing). Build an intensive aquatic farming area toward industrialized orientation (sugpo prawn, fish breeding in cages at sea...). Develop synchronously fish catching industry in terms of fishing fleets, ports, specialized fish berths, ship-building, and ship repairing, incrementally modernize seafood processing techniques to meet international standards thus increase export turn-over into the USA, EU markets.

Expand and diversify tourism activities, connect tourism with history of culture and tradition and develop oceanic ecological tourism. Invest and build the tourism areas of Do Son, Cat Hai and Cat Ba to become major tourism attractions of the nation.

Economic strategy development in Hai Phong is concentrated to set up a program for development of high quality services such as shipping services, transportation, postal services, banking, auditing, insurance and labors for export... and consider them as an important business that creates revenues of foreign currency. Develop some kinds of services such as trading promotion, legal, advertising, exhibition, restaurant, hospitality, souvenir shops... and strive to increase the proportion of services' value in GDP composition in future.

Cat Hai Island in Hai Phong

Cat Hai is a small coastal island district with its area of about 30 km^2 and population of about 13,000 people. Cat Hai island located at 20^{0}40' latitude and 106^{0}53' longitude. This islands borders the Ha Nam island (Quang Ninh) in the North, Gulf of Tonkin in the South, Lach Tray estuary in the East and Nam Trieu estuary in the West. Cat Hai has rich potentials supporting the development of fishery, forestry, tourism and services.

In period of 1949 – 1965, the segment of shoreline was eroded with 6.4 km long in the southern and rate of 5-8 m/year. In period of 1965 – 1988, it was kept continuously eroding with an extent of 2.9 km long rate of 10 – 25 m/year. From 1988 – 2001, the coast continued to be eroded but the erosion extent and rate were decreased thanks to presence of newly built groins and sea dikes [Pham, 2007]. The shore line erosions of Cat Hai island in period of 1949 to 2001, are illustrated in Figure 2.13 below.

Figure 2.13: Changes of shoreline in Cat Hai island [Pham, 2007]

Figure 2.14 to Figure 2.16 are showing the position and current situation of sea dike system in Cat Hai Island.

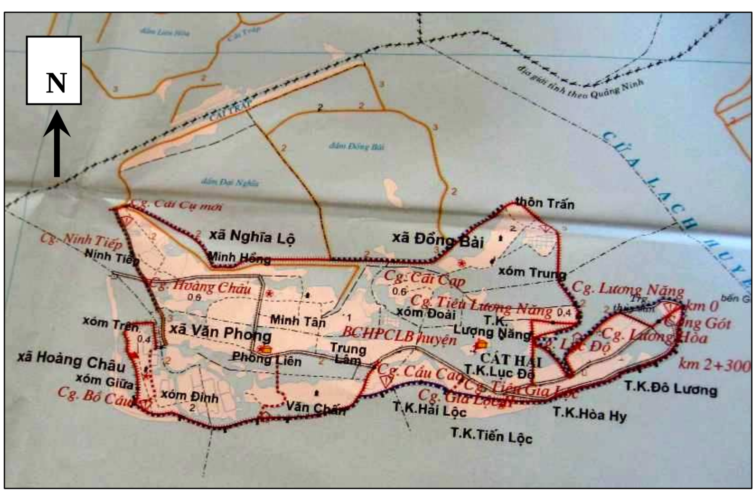


Figure 2.14: Sea dike system of Cat Hai Island, Hai Phong province

Figure 2.15: Sea dike at Northeast side of Cat Hai island

Figure 2.16: Sea dike at Southeast and Southwest (lower right) sides of Cat Hai Island

There is of 20.6 km of total length of sea dike runs around Cat Hai island. In which about 7 km of sea dike which placed from Ben Got to Hoang Chau in the South of island, is direct affected by wave attack from sea, and also affected by the strongest long-shore currents (see Figure 2.16). To avoid long-shore currents, this caused strong coastline erosion in the southern part of islands, a groin system including 14 groins were constructed with the length of each groin is of 150 m and placed perpendicular to the coastline.

Economic sectors and its development in Cat Hai island:

Cat Hai has rich potentials supporting the development of fishery, forestry, tourism and services.

The local fisheries economy has been growing in the traditional way for the past years, which means the intensification of fisheries exploitation, farming and services. The fisheries exploitation achieved 5,730 tones in 2000, but 8,091 tones in 2002. The aquaculture of 608 tones in 2000 was doubled to 1,148 tones in 2002 (*Source: http://www.haiphong.gov.vn)*.

Along with the fisheries development, salt production is also increasingly improved. On the area of 143.7 hectare, in 2002, the production achieved 12,187 tones, increasing by 27.6 percent compared to that in 2001. The total production value of the industry achieved VND 5,480 million in 2002, increasing by 14 percent in comparison to that in 2001. However, since the salt production is facing a lot of difficulties, it is necessary to transform the industry structure. In addition, the increasing salt prices are also helping a lot in the improvement of the farmer's life, ensuring them a better future of the salt production.

Cat Hai also has potentials to develop tourism. For the past few years, Cat Hai has become an attractive place for Vietnamese and international tourists. The number of tourists to the place increase by 25 percent per year, which achieved 205,000 tourists in 2002 particularly (doubled that in 2000). The tourism revenue achieves an annual increase of 19 percent and VND65 billion in 2002 particularly (*Source: http://www.haiphong.gov.vn)*.

It is defined in the development strategy by the district Party committee and people committee that: 'the infrastructure construction should be given with priority thus creating premise for the development of spearhead economic sectors'. Following that guideline, the district has for the past few years intensively implemented the infrastructure construction with high effectiveness.

Towards a better future:

Cat Hai is an island district rich of marine economic potentials, the rear of fisheries in the north coastal area. As a main fishing ground adjoining Long Chau and Bach Long Vi with a large area of ponds, sea and gulf, Cat Hai has a great deal of advantages for the development of fishery.

In the development orientation for future, Cat Hai focuses on the socio-economic development, exploiting to the maximum the potentials of the forest, sea and island ecology thus developing a multi-sector economy. Apart from socio-economic development, Cat Hai will also intensify the national security and defense.

From a remote, poor island district with thatched houses and poor people, Cat Hai has become an attractive centre of tourism and an effective fishing ground. There are more and more multi-storey buildings, the local people's living standard has been much improved, which shows the satisfactory changes marking the increasingly intensive development of the island district with two strong points: tourism and fisheries.

2.7 Actual safety of coastal zones in the study areas

In general in both Nam Dinh and Hai Phong, sea dikes and revetments have been used as the prevalent coastal protection system, protecting coastal areas from erosion, seawater floods and wave attack. This system has been constructed and developed continuously during long time. Existing sea dikes in the region are designed with 20 years return periods of sea loads (1/20 per year design frequency). Dikes are relatively low (1.5 metres of crest free board) with poor protection of dike crest and inner side. Outer slopes are protected by concrete block revetments with 35 cm thickness.

The sea dikes and revetments were designed with two intended functions, (i) sea flood defence; and (ii) prevention of coastline erosion. However, it seems that the system does not fulfil its second function at all, since the dikes itself can only provide flood prevention and the revetments can be used only to protect the dike body/land surface behind from wave and current induced erosion. These structures are not able to avoid erosion of the front beach and foreshore. Moreover, the dike's toe may be threatened by erosion, unless sufficiently deep toe protection is provided.

In addition, due to budget constraints, lack of information on the sea boundary conditions and a proper design methodology as well as good strategic and long-term solutions, the dike system was usually designed and constructed for low design conditions with low quality. As a consequence, the system could fail with high probabilities, even up to once in 10 years as learnt from practical experience. Therefore, the costs of dike maintenance and/or rehabilitation are very expensive. Statistically, costs for maintenance of sea dikes in the areas represent nearly 55 percent of the total budget for coastal flood defence of Vietnam.

As a result of relatively high design frequencies (1/20 per year) the sea dike system fails due to various failure mechanisms. Previous studies of sea dike failures in Vietnam (see, for example, Vinh et al., 1996 and Mai et al., 2007) indicated that the current design frequency resulted in low dikes with a high occurrence frequency of heavy wave overtopping. Wave overtopping induced dike failure is therefore a major problem. Apart from this mechanism, dike collapse due to deepening of foreshores or loss of dike toes are frequent mechanisms. Loss of a dike toe occurs mostly during a storm surge while disappearance of a foreshore is caused by a transport deficit in the long-shore direction under normal sea conditions. A deeper foreshore can accommodate more severe wave conditions and thus more overtopping will occur at the dikes. As a result, in situations where foreshores are retreating, the total failure probability will considerably increase.

An extensive study on probabilistic safety and reliability assessment of the coastal flood defence system is carried out (methods and detail analysis see Mai et al., 2006). Various failure mechanisms, which are possible to occur with the sea dike system in the region, as shown on Figure 15, are considered. The safety assessment is done for both old sea dike system, which was constructed in early 90's, and actual one, which recently constructed. According to the results, the probability that the defence system fails due to above given failure mechanisms is very high. The overall failure probability of the old-existing dikes is at 96% per year (Mai and Pilarczyk, 2005). This implies that the old dike system could fail on average 1/0.96 [$year^{-1}$], more often than once a year. Although the actual dikes were designed for a 20 years return period, the probability of dike failure is much higher, about P_f=0.14 [$year^{-1}$]. This is equivalent to failure occurrence of about once in every 7 years instead of 20 years as expected return period.

In summary, the whole coastal zones of both provinces are not safe according to design condition and existing safety standard. In addition the actual standard safety may not safe enough for the regions.

Table 2.1 next page summaries current situation in both case study of Hai Hau district and Cat Hai island.

Table 2.1: Summary of current situation in Hai Hau district and Cat Hai island

Items	Hai Hau in Nam Dinh	Cat Hai island in Hai Phong
Coastal morphology	Average foreshore slope: 1/175; Erosion rate of 10-20 m/year causes the first dike is damaged	Average foreshore slope: 1/90; Erosion rate: 5-25 m/year (1949-1988); this rate reduced from 1988 to presence.
Hydraulic	Strong waves and long-shore currents	Strong waves and long-shore currents
Socio - economic	Traditional trade villages, agriculture, aquaculture, fishery and salt production, and tourism	Aquaculture, fishery and salt production, and tourism
Ecology	Some mangroves areas in front of the first dike	Some mangroves areas in the northeast and the southwest
Actual safety	The area is flooded and have to evacuate every year	Flooding occurs in severe storms, not every year

CHAPTER III SEA BOUNDARY CONDITIONS

3.1 Data collections

From the pilot projects the following data are collected:

- Water levels at Hon Dau station in Hai Phong from 1961 to 2003.
- Water levels at Phu Le station (1962 – 2001) and Ba Lat station in Nam Dinh.
- Wave heights and wind data at Bach Long Vi Island in Hai Phong from 1976 to 1995.

Based on these collected data, the design sea boundary condition including water levels and the wave heights corresponding to the design frequency can be calculated.

Winds

Since there is no offshore island, and it has relatively flat and low-lying topography, HaiHau is an area exposed directly to the open sea; the area is subject to the winds generated from every direction. In the winter time (from October to March) the dominant wind directions are north, northeast and east. In summer (from May to August) the dominant wind directions are south, southeast and southwest. April and September are considered to be transition times.

Hai Phong is lying in the area which is directly influenced by a lot of storms and tropical low pressures in Vietnam, about 31% in total. Storms and tropical low pressures usually occur in between July to September with probability of 78%. The wind of storms is of level 6 – 7 (Beaufort scale: wind speed is 40 to 62 km/h) and during tropical low pressures, the wind has level of 8 – 12 (Beaufort scale: wind speed is 63 to 120 km/h).

Figure 3.1 illustrated the dominant directions of wind in summer and winter seasons in the Northern of Vietnam.

In this study the observed wind data at Bach Long Vi Island were used (Tonkin Gulf, 20.133^o latitude; 107.72^o longitude). These data are tabled in Table 3.1 and Figure 3.2 below.

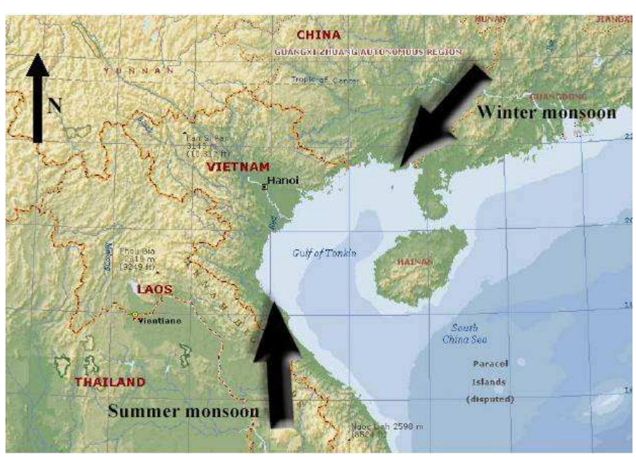


Figure 3.1: Main seasonal wind directions in northern Vietnam

Table 3.1: Wind data at Bach Long Vi Island (observation: 1976 - 1995)

Class (m/s)	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**	**Sum**
1-5	843	3,103	2,843	1,875	1,858	578	277	320	11,697
6-10	505	5,160	1,378	810	3,440	530	77	108	12,008
11-15	156	2,013	73	79	1,043	65	6	9	3,444
16-20	90	863	11	23	77	4	2	19	1,089
21-25	16	27	0	2	5	1	0	5	56
26-30	3	4	0	1	2	3	0	3	16
31-35	3	1	1	0	4	0	0	0	9
36-40	1	0	1	1	0	0	0	1	4
Sum	1,617	11,171	4,307	2,791	6,429	1,181	362	465	28,323

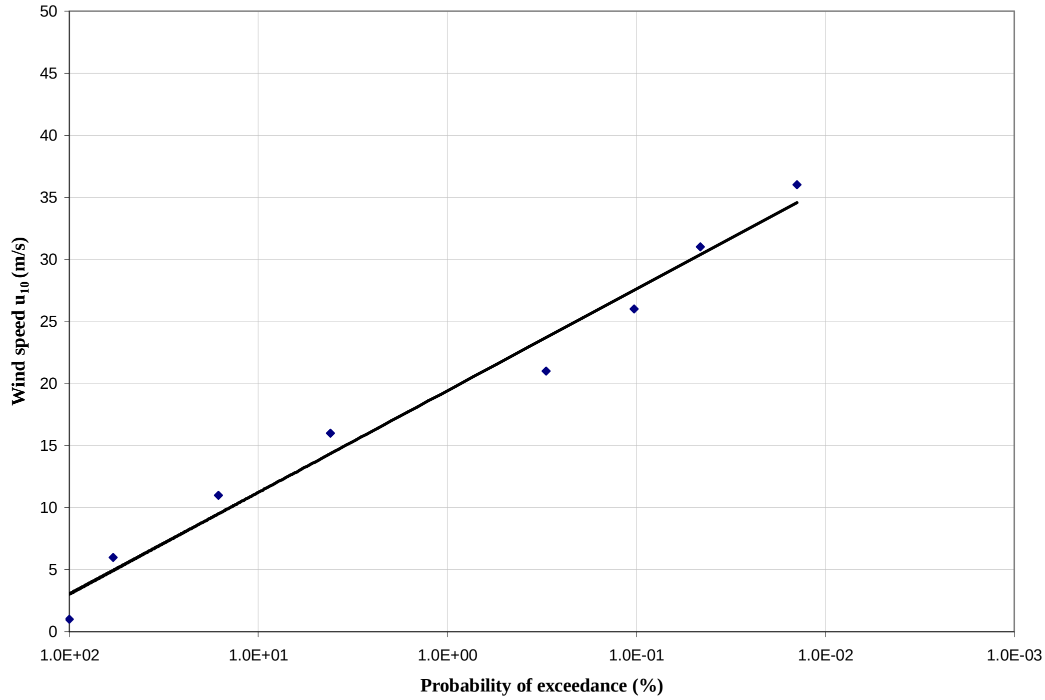


Figure 3.2: Exceedance curve of wind speed at Bach Long Vi Island

Storms/Cyclones

As referring to the topographic map, the beach and the foreshore of the study area has a very gentle slope, which creates a relatively wide zone for wave transformation and energy dissipation. Apparently, only monsoon waves, severe storms or typhoons, with high rainfall, extreme wind speed, high wave and storm surges, cause severe threats to the local natural beach and the existing coastal structure.

In the study area, according to the weather observation record, there were about 4 typhoons occurring in a year on average. August and September are the most critical periods to encounter floods and storms. In August and September, storm winds are generated from NE with velocities of 20 m/s, and in some cases even up to 48 m/s. Typhoons are normally accompanied by storm surges.

Near shore waves

The sea at Nam Dinh is open sea (there is no offshore island) so the wind fetch is long enough for wave growth and approaches the shoreline without any obstacles, which can cause considerable damage to shoreline and sea dikes. According to observation in period from 1975 to 1987 waves at Namdinh had following characteristics:

- In winter (from September to March): In the winter, the sea was much more rough sea than in the summer. Wave height is about 0.8m – 1.0m, with periods varying from 7 to10 seconds. Predominant wave direction was northeast, and makes angles of about 30^{o} to 45^{o} with the shoreline.
- In the summer (from April to August): In the summer there are less rough sea days but strong storms usually happen in this season causing severe damage to the dike system. Average wave height varies from 0.65m to 1.0m with period ranging from 5 to 7 seconds. The prevailing wave direction is south and southeast.

The wave at Hai Phong sea has predominant direction from East, Northeast and South. The wave height is changing in each season and depended on direction and speed of wind:

- In winter period (September to March): The wave in offshore zone has predominant direction of the Northeast (61%) and the East (15%); the wave directions near shore zone are East (34%), Southeast (22%) and Northeast (11%). Average wave height is 1.2 m in offshore and 0.8 m in near shore. The maximum offshore wave height is up to 6 m and 3.5 m in near shore.
- In summer period (April to August): The wave in offshore zone has predominant direction of South, Southeast and East with total probability of 40 - 75%, in which the dominant direction is South (37%); the dominant near shore wave direction is Southeast (24%). Average wave height is 1.2 to 1.4 m in offshore and 1.0 to 1.2 m in near shore. The maximum offshore wave height is up to 7 to 9 m and 4 to 5 m in near shore.

3.2 Design water levels (DWL)

Water level is important because [CIRIA/C683 2007]:

- Most instances of flooding and/or structural damage occur during high water level
- Wave overtopping and wave transmission depend on the Still Water Level (SWL)
- The force on a seawall partially protected from waves by a shallow foreshore depends on SWL
- A structure may be exposed (and possibly vulnerable) to different risks for different water levels, in turn dependent upon SWL
- The wave height may be limited by breaking before arriving at a structure
- Construction and maintenance is generally affected by the overall water level regime.

Various components of water level should be considered. Apart from astronomical tides and very rare seismic (tsunami) effects there are several meteorological components of the water level to be considered, known as residuals. These residuals comprise storm surges, wind setup, wave set-up and seiches.

Some components of water level are partially correlated, meaning that a higher or lower value of one component tends to occur at the same time as a higher or lower value of another component. Correlations often arise between components of meteorological origin, such as storm surge, wind set-up, wave set-up and even seiches. Depending on tidal levels, these components may be affected, notably in shallow areas. For example, surges may propagate differently according to the water depth and current conditions.

Usually the two most important components of the water level at any moment are the astronomical tide and the storm surge. The former is cyclical with a period that depends on the relative significance of astronomic forces at a particular location. The variation with time of the water level due to astronomical tide and storm surge is illustrated in Figure 3.3.

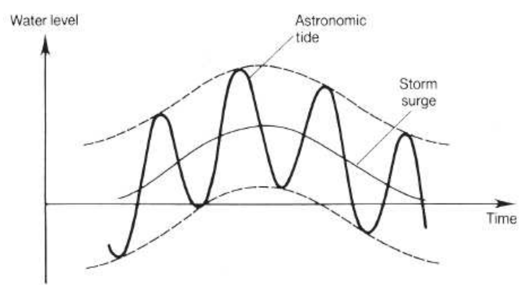


Figure 3.3: Variation in water level due to storm surge and astronomical tide [CIRIA/C683 2007]

In summary, there are two traditional ways to determine the DWL. The first way is based on statistical analysis of observation water level data (see Section 3.2.1). The other way, the DWL is determined by wind speed data which are observed in many years (see Section 3.2.2).

3.2.1 Determination of DWL based on available observed data

For the design of a structure, the definition of design conditions requires knowledge (or estimation of) the distribution of probability of large and extreme events. The purpose of the determination of the long-term climate is to associate a water level (or wave height) to a given return period, and if possible with a confidence level.

Extrapolation of the validity of a distribution beyond the range covered by the measurements should be done with care. However, this is generally the only way of predicting low-frequency (long return period) events. The procedure adopted is to fit to a theoretical extreme-value distribution and then to extrapolate the fitted distribution to extreme values.

There is no theoretical argument in favour of the use of any particular probability density function in all situations.

The following two points are more important than the choice of a particular candidate model to fit available samples of events:

- Is the distribution steep or not? A steep distribution means that very extreme conditions may be much higher than the considered design conditions. This is not the case for flat distributions.
- What is the confidence interval of any estimation? This is particularly sensitive for long return periods, i.e. low occurrence probability.

The recommended procedure for analyzing the water level data comprises the following steps:

1. Select the data for analysis.
2. Drawing empirical exceedance frequency curve.
3. Fit candidate distribution(s) to the data to find the best fit, and estimation of distribution parameters.
4. Compute return values from the fitted distribution(s).
5. Consider confidence interval in the predictions. In this study a confidence level of 5% is considered.

Selection, checking and preparation of data together form probably the most important stage in the analysis procedure. For extraction of water level data, the peaks-over-threshold (POT) method is applied. Following this method, only the water level above some chosen threshold (e.g. water level = 1.5 m) are used in the extremes analysis. It is recommended that the water level threshold is selected to achieve that the average number of selected data values per year (typically 5–10) above the threshold is equal to or less than the average number of storms per year (typically 10–20).

Following the POT method a dataset of events is selected. Based on this data the empirical exceedance frequency per event is built by Equation(3.1) below.

$$1 - Fr(i) = 1 - \frac{i - 0.3}{N + 0.4} \text{ [per event]} \tag{3.1}$$

Where: i is event number and N is total number of collected events.

The POT method is applied to analyses water level data at three observed stations (Hon Dau, Phu Le and Ba Lat). These empirical exceedance frequencies are illustrated in Figure 3.4.

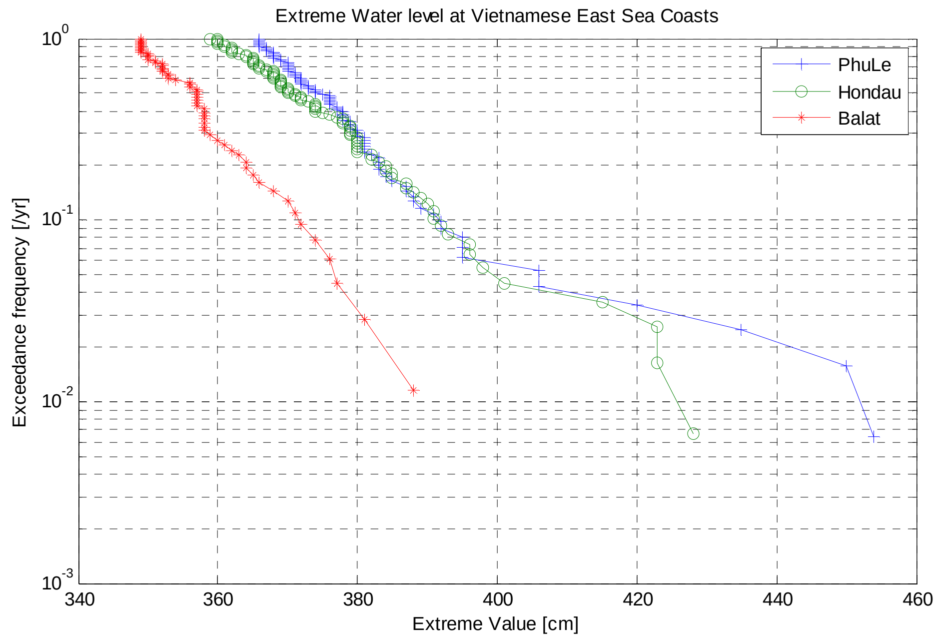


Figure 3.4: Empirical exceedance frequencies per event of water level at Vietnamese East Sea Coasts

These exceedance frequencies per event are also transferred to the curves which show exceedance frequencies per year of water level (Figure 3.5 & 3.6) for each observed station by Equation(3.2).

$$P_i = Coef_{Fr} * \left(1 - \frac{i - 0.3}{N + 0.4}\right) \text{ [per year]} \tag{3.2}$$

In which P_i is empirical probability of exceedance; $Coef_{Fr}$ is coefficient of exceedance frequency which is determined in Equation(3.3):

$$Coef_{Fr} = \frac{n_{observed\_years}}{N} \tag{3.3}$$

Where $n_{observed\_years}$ is the number of observation years; N is the total number of collected events (length of POT data).

Because the record length of water level data at Ba Lat station is less than Phu Le station and in addition the Balat station is inside the Balat Estuaries, therefore the observed water level at Phu Le station is used for the design in Nam Dinh coast. Observed water level at Hon Dau station is used for the design in Hai Phong coast.

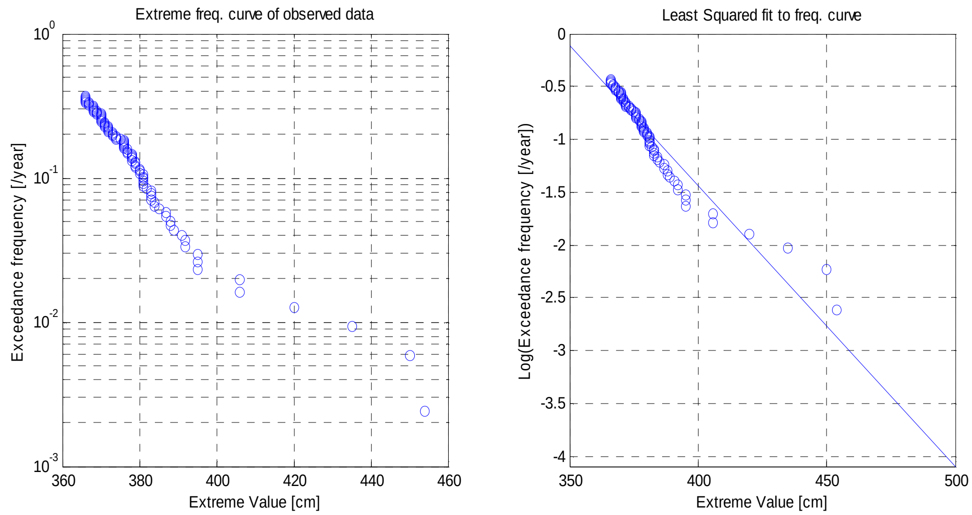


Figure 3.5: Empirical exceedance frequency curve (per year) of water level at Phu Le station, Nam Dinh, along Vietnamese East Sea Coasts and linear regression line (least square)

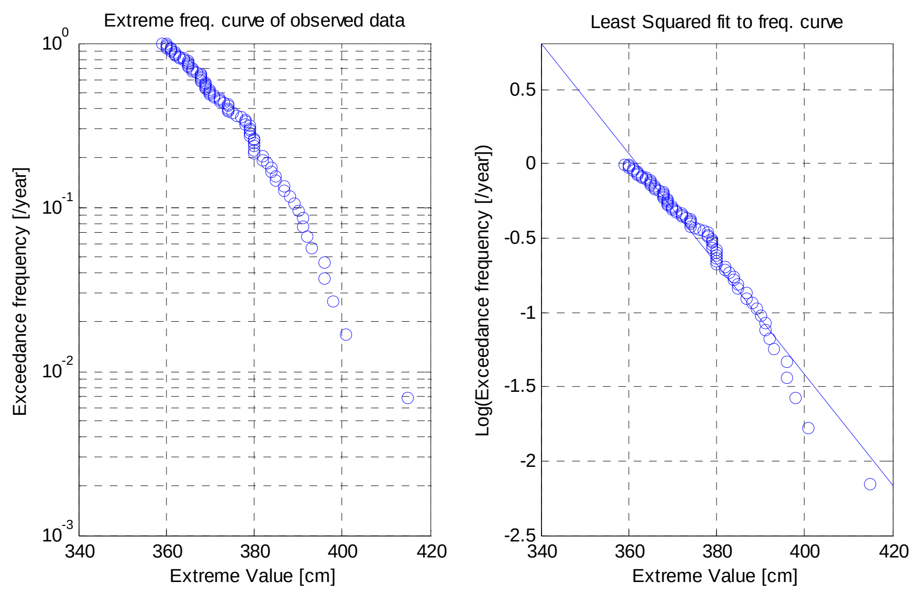


Figure 3.6: Empirical exceedance frequency curve (per year) of water level at Hon Dau station, Hai Phong, along Vietnamese East Sea Coasts and linear regression line (least square)

Table 3.2: Summary of extreme water levels based on empirical frequency curves

Design Frequency	**1/10**	**1/25**	**1/50**	**1/100**	**1/1000**
Log_{10}(Frequency)	**-1**	**-1.4**	**-1.7**	**-2.0**	**-3.0**
Design water level at Nam Dinh coast (m) LS. Line: $Log_{10}(Fr)=-0.0265xWL+9.18$	3.84	4.00	4.11	4.22	4.60
Design water level at Hai Phong coast (m) LS. Line: $Log_{10}(Fr) = -0.0375xWL+13.6$	3.90	4.00	4.08	4.16	4.43

Best fitted distribution and determination of its extreme value.

In previous section observed data is fitted to an empirical exceedance frequency line. In order to reduce uncertainty in estimation of design value this section will try to find the best fit distribution function to model the data. Based on the best fitted distribution the data are re-simulated and the design values are estimated.

Uncertainties in the computed extreme values depend mainly on:

- Inaccuracy or unsuitability among the source data.
- Inherent statistical variability, i.e. sampling variability.
- Uncertainty due to possible incorrect choice of extreme value distribution.
- Uncertainty in the computation of water level due to a record of limited length.

5% of confidence interval formulae take account only of statistical variability, and not of uncertainties in the source data or the choice of fitted distribution. They should therefore be used with some caution, and a confidence estimate based on experience and sensitivity testing may be more reliable.

Various distributions are used to fitting observation data at Phu Le and Hon Dau stations. Based on goodness-of-fit test the best-fitted distribution is found. Best-fit analysis result is presented in Appendix 3. Weibull distribution is the best choice of fitted distribution for both data at Phu Le and Hon Dau (Figure 3.7).

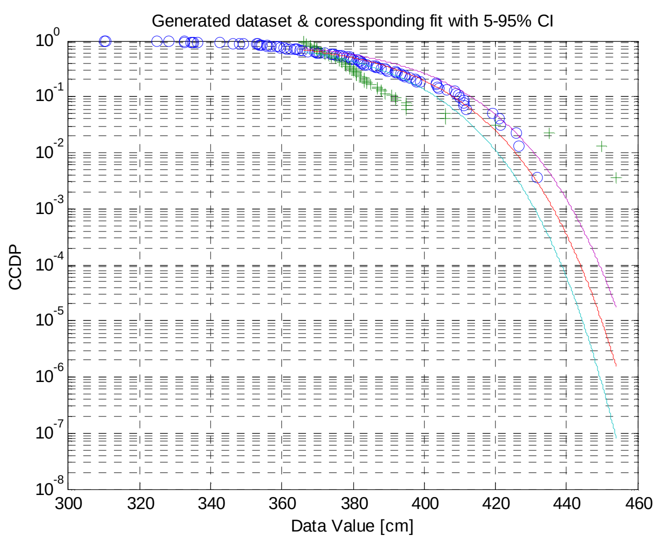


Figure 3.7: Weibull distribution fit to water level data at Nam Dinh coast

Hundred simulations of Weibull distribution of water level in Nam Dinh coast are done and based on these simulations the DWL is determined for variability return period of 10, 25, 50, 100 and 1000 years for design structural procedure of Nam Dinh coast by averaging of these simulations of Weibull distribution. These values are in Table 3.3 and Figure 3.8.

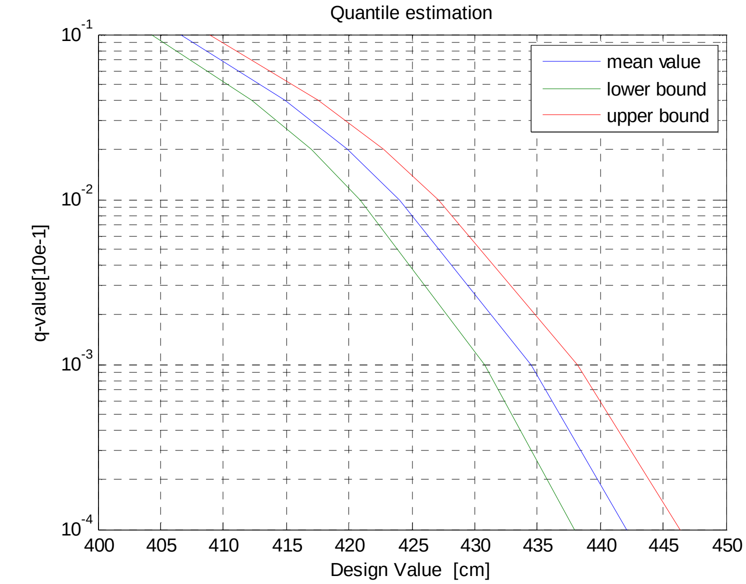


Figure 3.8: Quantile estimation of water level in Nam Dinh coast and with 5% of confidence interval

Similar to water level data analyzing for Nam Dinh coast, water levels in Hai Phong are analyzed and plotted in Figure 3.9 and 3.10. Also design water levels are calculated for various return periods in Table 3.3.

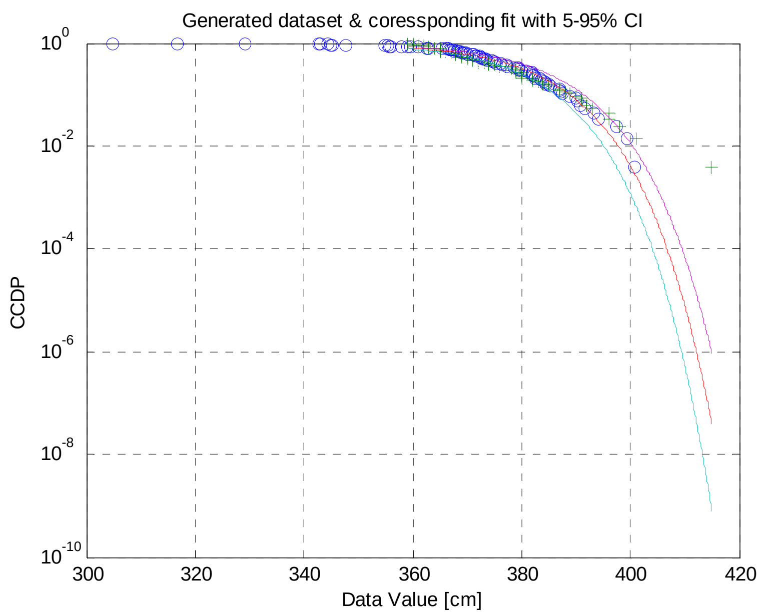


Figure 3.9: Weibull distribution fit to water level data at Hai Phong coast

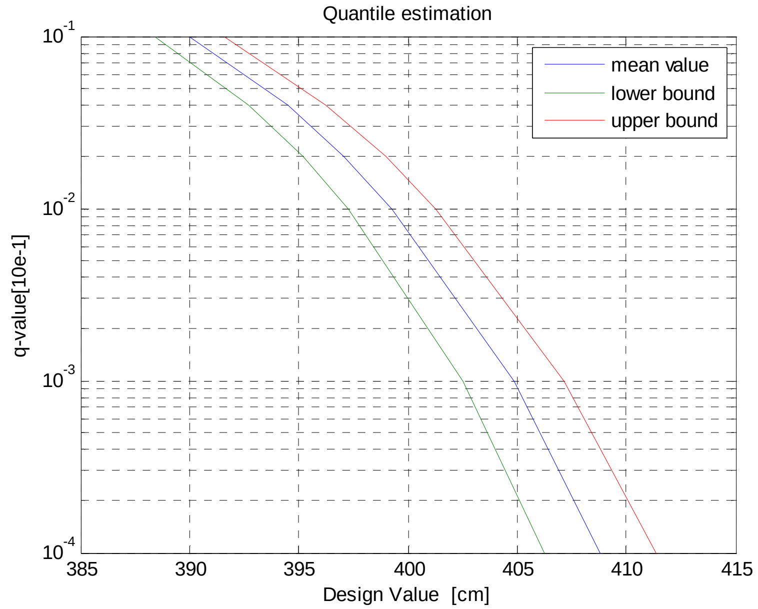


Figure 3.10: Quantile estimation of water level in Hai Phong coast and with 5% of confidence interval

Table 3.3: Design water levels at Nam Dinh and Hai Phong coast based on Weibull fitted distribution

Design Frequency	**1/10**	**1/25**	**1/50**	**1/100**	**1/1000**
DWL at Nam Dinh coast (m)	4.07	4.15	4.20	4.24	4.34.
DWL at Hai Phong coast (m)	3.90	3.94	3.97	3.99	4.06

3.2.2 Determination of DWL based on wind statistic data

Beside the fact that DWL can be estimated statistically by the observed data, another available method is to base on the local weather condition and astronomical tides. The following main components will contribute to design water level:

a. Astronomical tides. Changes in water level are caused by astronomical tides with an additional possible component due to meteorological factors (wind setup and pressure effects). Periodic tidal levels are published annually by the National Hydro-meteorological Centre.

b. Storm surge (wind setup): Storm surge can be estimated by statistical analysis of historical records, or through the use of numerical models. The numerical models are usually justified only for large projects. Some models can be applied to open coast studies, while others can be used for bays and estuaries where the effects of inundation must be considered.

In addition to that for sea dikes design we also take in to account other types of water fluctuations like [Pilarczyk et al 1998]:

- Wind setup;

- Climatologically variations;

- Wind waves;

- Squall oscillations (seiches) and gust bumps.

Thus, DWL can be determined by equation(3.4):

$$DWL = MSL + Z_{tide} + \Delta Z_{wind} + \Delta Z_{gust} + \Delta Z_{rise} \tag{3.4}$$

In which:

MSL - Mean Sea Level: The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.

Z_{tide}: Tidal level which measure at high water spring (refers to MSL).

ΔZ_{wind}: Increase in water level in front of the dikes due to Wind setup (and storm surge). This is a rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress

ΔZ_{gust}: Rise of water level due to gust bump

ΔZ_{rise} : Rise of water level due to Sea level rise.

Determination of the components: The elevations which will be determined are all relative to CD (chart datum- Standard elevation system in Vietnam)

MSL: At the study area, *MSL* is +1.92m relative to Vietnam-HD coordinates (The "Zero" value is at the lowest annual water level of sea surface). It corresponds to +0.0 meter original land coordinates.

Tidal level*:* Z_{tide}, m +MSL, is the averaged highest tidal range for the location according to annual publication of the National Hydro-meteorological Center, Tidal Tables.

Wind setup (Storm surge): ΔZ_{wind} is defined as the difference in still-water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water (Equation 4.12 in CIRIA/C683 2007). The following formula is applied:

$$\Delta Z_{wind} = \eta_w = \frac{1}{2}\frac{\rho_{air}}{\rho_w} C_D \frac{U_{10}^2}{gh} F \tag{3.5}$$

In which: C_w: air/water drag coefficient with typical values of $0.8\text{x}10^{-3}$ to $3\text{x}10^{-3}$ (-)

ρ_{air}: mass density of air (1.25 kg/m^3)

ρ_w: mass density of sea water (1031 kg/m^3)

F: fetch length (m)

h: water depth at calculated position (m)

U_{10}: wind speed at an elevation of 10 m above MSL (m/s) corresponds to design frequency. This is determined from wind statistics data.

Gust bump: this is the increase in water level due to wind surge in normal conditions.

Seiches: this is the increase in water level due to long waves from off-shore**.**

Sea level rise: for many years it has been known that the sea is level rising. In historical time this sea level rise was generally moderate. The sea level rise here is considered as the rise of water level itself plus the changes of the level of the land. The relative sea level rise can determined from long time series of water level observations. In Vietnam there was not any long term record for these observations. However the predictions of sea level rise were made based on the subsidence of the land in Vietnam and the rise in water level of South China Sea. This component will be accounted for when determining the dike crest level as an independent component.

Applying the equation (3.4) and (3.5) with given parameters, DWL and wind setup for different design frequencies are determined such as in Table 3.4 for Nam Dinh coast and Table 3.5 for Hai Phong coast.

Table 3.4: Determination of DWL based on wind statistic data in Nam Dinh coast

Frequency of occurence	1/10	1/25	1/50	1/100	1/1000
Mean Sea Level - MSL, m	0	0	0	0	0
Tidal level - Ztide, m	2.3	2.3	2.3	2.3	2.3
Friction coefficient Cw	8.00E-04	0.0008	0.0008	0.0008	0.0008
Density of air ρ_{air}, kg/m3	1.25	1.25	1.25	1.25	1.25
Density of sea water ρ_w, kg/m3	1031	1031	1031	1031	1031
Fetch length F, km	150	150	150	150	150
Water depth at calculated position h, m (+MSL)	5	5	5	5	5
Angle between wind direction and normal of coastline Φ, deg	30	30	30	30	30
Wind velocity corresponds to design frequency U, m/s	27.6	30.8	33.4	35.9	44.2
Wind setup (Storm surge), ΔZwind (Eq.4.12 in CIRIA/C683), m	1.13	1.41	1.65	1.91	2.90
Gust bump, m	0	0	0	0	0
Seiches, m	0	0	0	0	0
Design Water Level - DWL, m	3.4	3.7	4.0	4.2	5.2

Table 3.5: Determination of DWL based on wind statistic data in Hai Phong coast

Frequency of occurence	1/10	1/25	1/50	1/100	1/1000
Mean Sea Level - MSL, m	0	0	0	0	0
Tidal level - Ztide, m	2.4	2.4	2.4	2.4	2.4
Friction coefficient Cw	8.00E-04	0.0008	0.0008	0.0008	0.0008
Density of air ρ_{air}, kg/m3	1.25	1.25	1.25	1.25	1.25
Density of sea water ρ_w, kg/m3	1031	1031	1031	1031	1031
Fetch length F, km	100	100	100	100	100
Water depth at calculated position h, m (+MSL)	5	5	5	5	5
Angle between wind direction and normal of coastline Φ, deg	30	30	30	30	30
Wind velocity corresponds to design frequency U_{10}, m/s	27.6	30.8	33.4	35.9	44.2
Wind setup (Storm surge) - ΔZwind (Eq.4.12 in CIRIA/C683), m	0.75	0.94	1.10	1.27	1.93
Gust bump, m	0	0	0	0	0
Seiches, m	0	0	0	0	0
Design Water Level - DWL, m	3.2	3.3	3.5	3.7	4.3

3.2.3 Comparing and selecting of DWL

Via three methods, the DWL which are determined based on water level statistic data are larger than that are based on wind statistic data. However, the differences of DWL, which are determined based on available observation water level data, are too small (about 5 cm to 10 cm) in between return period scenarios, these results of determination are not reliability. While the DWL's, which are determined based on wind statistic data, are more reasonable, therefore, these DWL's are used in this study. Table 3.6 summaries the values of DWL for case studies of Nam Dinh and Hai Phong.

Table 3.6: DWL in Nam Dinh and Hai Phong coast

Design Frequency	**1/10**	**1/25**	**1/50**	**1/100**	**1/1000**
Design water level at Nam Dinh coast (m)	3.4	3.7	4.0	4.2	5.2
Design water level at Hai Phong coast (m)	3.2	3.3	3.5	3.7	4.3

3.3 Design wave heights in deep water

3.3.1 Long-term wave statistics – analysis of extreme waves:

In the region of Vietnamese East Sea Coasts there exists only one available wave measurement dataset at Bach Long Vi Island. This dataset was observed in 20 years (1976-1995).

The empirical exceedance curve of extreme wave height at Vietnamese East Sea Coasts is plotted in the Figure 3.11 by applied Equation(3.1).

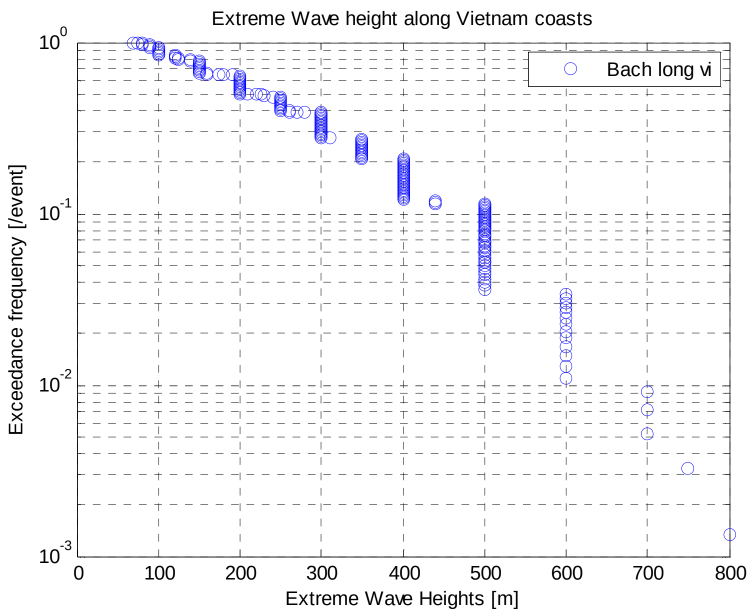


Figure 3.11: Empirical exceedance frequency per event

An empirical exceedance frequency per event will be transferred to an empirical probability of exceedance per year (Figure 3.12) following Equation(3.2).

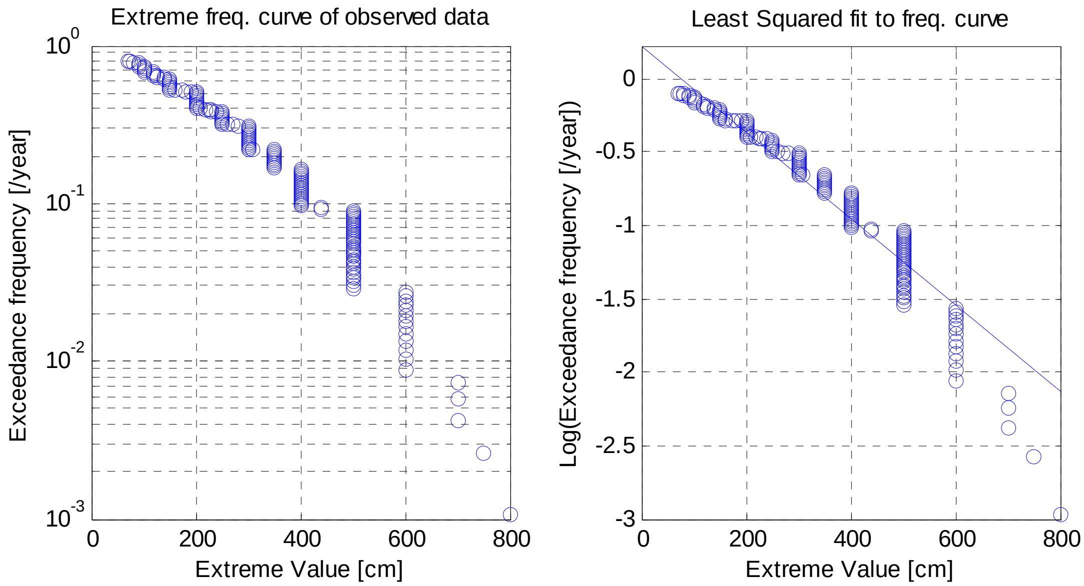


Figure 3.12: Empirical probability of exceedance per year

Table 3.7: Summary of extreme wave height based on empirical frequency curve

Design Frequency	1/10	1/25	1/50	1/100	1/1000
Log$_{10}$(frequency)	-1	-1.4	-1.7	-2	-3
Extreme wave height	4.1	5.4	6.4	7.5	10.9

Based on the dataset of wave height at Bach Long Vi Island, the Exponential distribution is found as the best fitted distribution to the wave observed data. The compliment cumulative distribution (CCDF) with 5% confident interval is plotted in Figure 3.13 with confidence interval of 5%.

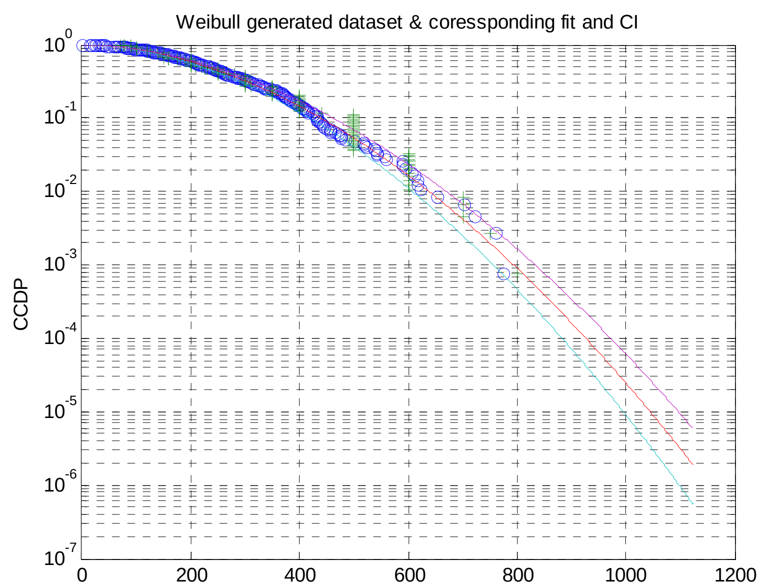


Figure 3.13: Exponential distribution fit to long-term wave height data and simulations of the distribution fit to long-term wave height data (in cm)

Some wave height values are estimated corresponding to design frequencies as in Table 3.8 and Figure 3.14.

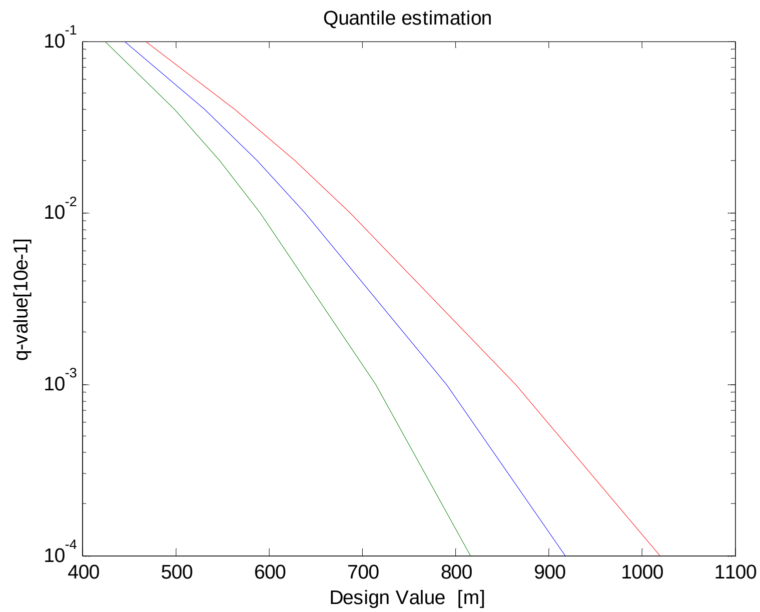


Figure 3.14: Quantile estimation of design wave heights and with 5% of confidence interval

Table 3.8: Long-term design wave heights in deep water (based on best fitted distribution method)

Design Frequency	**1/10**	**1/25**	**1/50**	**1/100**	**1/1000**
Design wave height (cm)	4.5	5.3	5.9	6.4	7.8

The larger wave height values are used to design. Table 3.9 shows the design wave height in deep water which are used to design in this study.

Table 3.9: Significant wave height in deep water

Design Frequency	**1/10**	**1/25**	**1/50**	**1/100**	**1/1000**
Significant wave height in deep water (m)	4.5	5.4	6.4	7.5	10.9

3.3.2 Short-term or daily wave climate:

In order to describe the total hydraulic boundary conditions a distribution of wave conditions have to be defined, both height and period, both in deep and in shallow water.

The climatology of sea-states has to be analysed (based on some representative parameters such as the significant wave height, the mean or peak period, the mean wave direction etc) on the basis of a set of data covering several months to several years (typically one year) and covering a range of storm conditions.

Results of the daily or short-term wave climate are often useful for the design of a structure (which is in fact dictated by the long-term wave climate). Furthermore, they are very important for the definition of operating conditions for the structure, the operability of floating equipment, the knowledge of typical wave conditions during the construction or maintenance phases.

Based on a series of sea-state parameters (given every three or six hours typically), it is possible to build several useful tables and graphs describing the short-term or daily wave climate.

- Histograms of significant wave height, mean (or peak or significant) period, mean wave direction etc. For each parameter a set of classes of values is defined and, from the measured series of sea-states parameters, the number of events per class (and so the empirical probability of occurrence) is estimated from the data. The analysis may be restricted to a particular period of the year (to analyse the seasonal effects), to a range of incoming wave directions (to separate different wave regimes) etc.

- Wave roses, in the same manner as wind roses. This type of representation allows a combined view of the most frequent incoming wave directions and associated wave heights. Again, different wave regimes can thus be separated (e.g. swell and wind-sea conditions). Figure 3.15 plots the wave height rose at Bach Long Vi Island of all year.

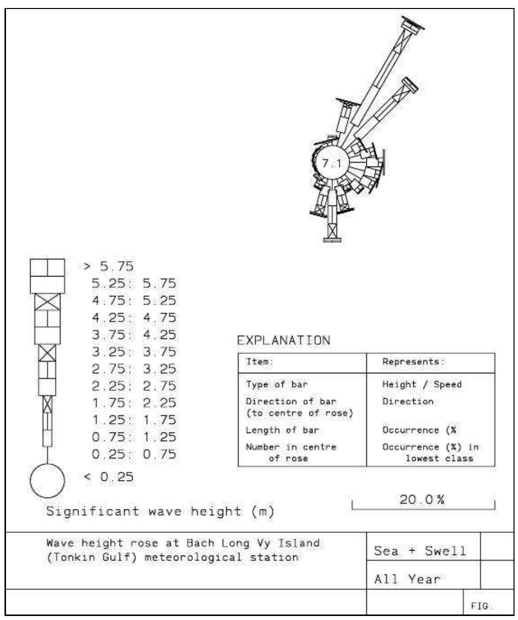


Figure 3.15: The wave height rose at Bach Long Vi Island

- A **scatter diagram** of wave height and period which gives the fraction of waves found within each of a number of predefined classes of H_s and T_m. The scatter diagram is created by counting the total number of individual sea-states falling within classes ΔH_s and ΔT_m. Division by the total number of sea-states gives an estimate of the 2D (H_s , T_m) joint-distribution function.

Table 3.10: Short-term wave climate data at Bach Long Vi

Significant Wave Height (m)		**Spectral Peak Period (s)**							
		< 1	**1-3**	**3-5**	**5-7**	**7-9**	**9-11**	**>11.00**	**Total**
<	0.25	6.44	0.64	.	.	.	.	.	7.07
0.25:	0.75	.	3.71	23.12	.	.	.	.	26.83
0.75:	1.25	.	.	18.92	9.25	.	.	.	28.17
1.25:	1.75	.	.	.	21.55	.	.	.	21.55
1.75:	2.25	.	.	.	11.58	0.39	.	.	11.97
2.25:	2.75	.	.	.	0.21	1.41	.	.	1.62
2.75:	3.25	.	.	.	.	1.79	.	.	1.79
3.25:	3.75	.	.	.	.	0.53	.	.	0.53
3.75:	4.25	.	.	.	.	0.38	.	.	0.38
4.25:	4.75	.	.	.	.	0.08	.	.	0.08
4.75:	5.25	.	.	.	.	0.01	.	.	0.01
5.25:	5.75	.	.	.	.	0.01	.	.	0.01
>	5.75	.	.	.	.	.	.	.	.
Total		6.44	4.34	42.04	42.59	4.6	.	.	100

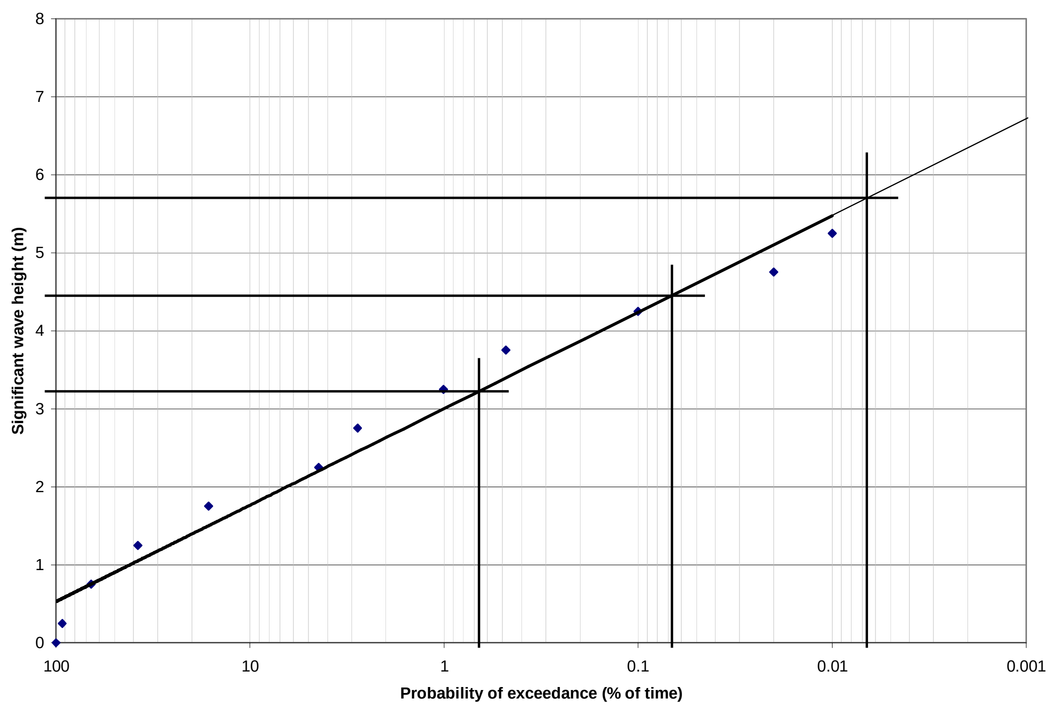


Figure 3.16: Daily wave climate based on data of Table 3.10

The main interest is usually focused on the distribution of wave heights, and in particular on the determination of wave heights that are exceeded 10 per cent, 1 per cent or 0.1 per cent of time on average over one year. This information can be extracted from the empirical distribution of significant wave heights.

Based on distribution of significant wave heights in Figure 3.16 daily wave climate (short-term wave climate) are extracted by considering sea-state duration of 6 hours. The values of short-term wave climate are determined in Table 3.10.

Estimation of wave period:

In the design of sea dike it is usually necessary to know the mean wave period, T_m, as well as the significant wave height, H_s, derived from extremes analysis. Any treatment of wave period is usually based on the assumption that wave height and wave period are strongly correlated, as already analysed for the short-term or daily wave climate, perhaps being related by a constant wave steepness. Any treatment of wave direction is usually based on a conditional analysis where the condition is that the wave records analysed have directions within a particular angular sector. In other words, wave data within different direction sectors are considered as being members of different populations, which can be analysed separately.

A common approach is to look at the joint distribution of (H_s, T_m) among the highest few percent of wave conditions in the source data. The average wave steepness ($2\pi Hs/Tm^2$) for these data can be computed and then applied to the predicted extreme wave heights. Wave steepness in deep water is typically in the range 0.045–0.065. It tends to have a lower value in shallow water, where wave height may have reduced but wave period is little changed from deep water. If there is any doubt about the exact wave steepness to use, note that the use of lower wave steepness (hence higher wave period) tends to be conservative for most design purposes.

There is no observed data available for wave period along Vietnamese coast. Thus in this study the wave period is determined based on empirical formula, which is written in the Chinese code for the South China Sea. The peak wave periods are determined by empirical equation(3.6):

$$T_p = 4.5\sqrt{H_s} \tag{3.6}$$

The ratio between peak period, T_p, and mean period, T_m, varies slightly depending upon the type of weather conditions, and upon the shape and size of the wave generation area, but in the absence of site-specific effects, T_p tends to be about 25 per cent greater than T_m (Equation(3.7)).

$$T_m = \frac{T_p}{1.25} \tag{3.7}$$

Table 3.11: Daily wave climate based on Figure 3.18

Duration of storm (hours)		6
Number of storm per year, N		1460
Probability, P_i (%)	10% / year	0.685
	1% / year	0.0685
	0.1% / year	0.00685
H_s (m)	10% / year	3.21
	1% / year	4.45
	0.1% / year	5.70
T_p (s)	10% / year	8.1
	1% / year	9.5
	0.1% / year	10.7
T_m (s)	10% / year	6.4
	1% / year	7.6
	0.1% / year	8.6

3.4 Correlation analysis

According to CIRIA/C683 2007 joint probability analysis results can be expressed as a range of combinations of wave conditions and water levels, each with the same return period. Each one is expected to be exceeded once, on average, in each return period. In designing or assessing a structure, its resistance to every combination of wave height and water level for the return period being used should be verified. In other words, for each coastal response variable of interest, each combination of extreme water level and wave condition should be tested, to determine the worst case for each response variable.

Alternatively, the results of a joint probability analysis may be presented in the form of a climate scatter diagram with or without extrapolated joint probability density contours. This form of presentation is more appropriate for building up a probability distribution of a coastal response variable, found by integrating the response function over the joint ranges of each of the primary input variables. This might be helpful where damage to a structure builds up over a period of time, as opposed to damage occurring during a single rare event.

If the wave heights and water levels are derived for a location other than the point where they are to be applied, some adjustment of values may be necessary. The most obvious case is the need to modify wave conditions calculated offshore to allow for shallow-water transformations prior to their arrival at coastal defences.

The degree of correlation between large waves and high water levels varies between locations and wave direction sectors, and even between offshore and near shore. It may therefore be inappropriate to assume that the most severe sea-states offshore give rise to conditions as severe as those at the coast. At open coasts, where the largest waves offshore also give rise to the largest waves inshore, the correlation with high water levels is similar for both situations.

However, where waves are strongly depth-limited before arriving at the sea defences, then wave period (and therefore possibly a different type of sea condition) may be much more important nearshore than offshore. In addition, for locations protected by headlands from the largest offshore waves, the nearshore situation may differ from that offshore.

Joint probability analysis is normally based on measurements taken during recent years, assuming that they are representative of longer-term conditions. If the wave conditions or water levels are known to be subject to any long-term variations or if the period of measurement is known to be unrepresentative, then allowance should be made for this.

With regard to Vietnam data availability condition at stations in the interested areas only one variable, either wave height or water level is available. In this case it is difficult to say anything about correlation and /or joint probability distribution.

At Hon Dau station the water level data is available. Wave height can be calculated/ simulated based on numerical model, e.g. sing SWAN model (Nguyen, T.T.H. 2003). By using the computed wave height extracted at the same location of Hon Dau the correlation analysis of wave and water level can be performed.

The correlation of observed water level and calculated wave height at Hon Dau station (Hai Phong) are expressed in Figure 3.17 and Figure 3.18. Upper bound (in Figure 3.17) shows the depth limited wave height line which is physical relevant in this case. All points above the line can be considered as outlier.

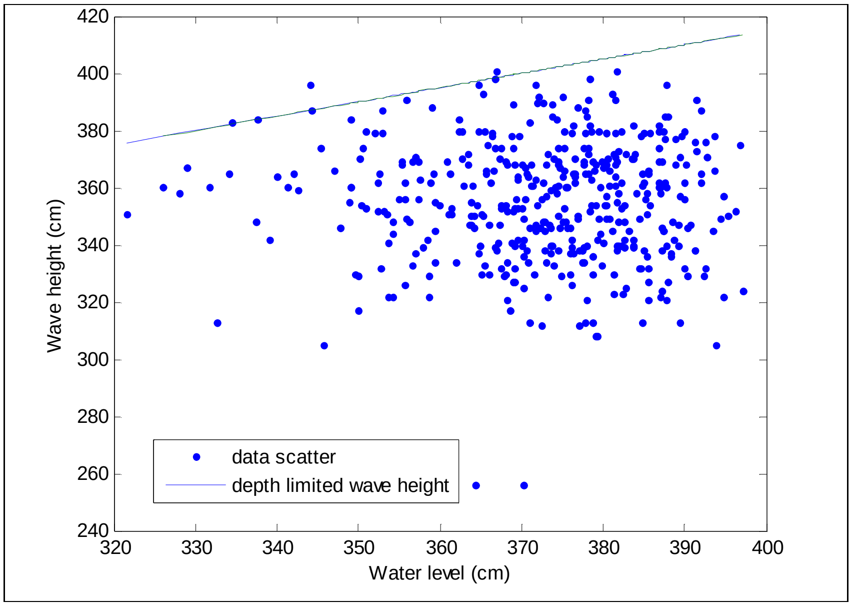


Figure 3.17: Depth limited wave height line at Hon Dau station

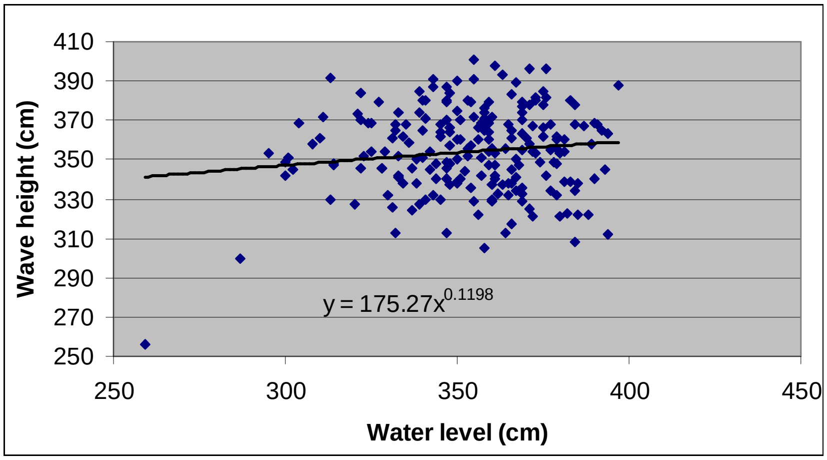


Figure 3.18: Correlation of water level and simulated wave height at Hon Dau, Hai Phong

3.5 Determination of wave height and wave period at the toe of the dike

The hydraulic boundary conditions mentioned here are given at a certain location (Nam Dinh and Hai Phong coasts). Very often this is 50 m – 200 m from the toe of the dike. For calculation of wave run-up or wave overtopping the wave height at the toe of the dike has to be determined. If depths at the given location and the toe of the structure are similar, then the given values can be used. If a sloping foreshore is present it can be required to calculate the wave height at the toe of the dike.

For coastal structures, the effects of water-depth reduction and coastal forms on the incoming waves should be accounted for. These factors transform the incoming waves by refraction, shoaling, diffraction and eventually wave breaking. Wave breaking results in significant dissipation of energy and is often the major factor limiting the design wave height and consequently the loading on the structure. All these phenomena are a function of water depth.

Refraction:

Refraction is the change in the wave propagation velocity, and consequently also in the direction of wave propagation, when waves propagate in varying water depth. In decreasing water depth the direction of wave incidence, β(°), relative to the structure inclines towards the direction normal to the depth contours.

The refraction factor is calculated by Equation(3.8).

$$K_r = \sqrt{\frac{\cos\beta_0}{\cos\beta}} \tag{3.8}$$

In which the local wave direction β at water depth h is found from Equation (3.9) which is applying linear wave theory to a regular wave with wave number k and direction β_0 in deep water.

$$\beta = \arcsin\left(\sin\beta_0 \tanh(kh)\right) \tag{3.9}$$

Shoaling:

Shoaling is a change in wave height when waves propagate in varying water depths. The shoaling effect is normally expressed in terms of the shoaling coefficient, K_s, which is defined as the local wave height H relative to H_o. Using linear wave theory K_s can, for a given wave period T, be written as a function of water depth h. Equation (3.10) is using to determine the shoaling coefficient K_s.

$$K_s = \sqrt{\tanh(kh)\left(1+\frac{2kh}{\sinh(2kh)}\right)} \tag{3.10}$$

Depth-limited significant wave height for constant bottom slopes:

Wave breaking becomes increasingly important in shallow water. The main effect of wave breaking is a lower significant wave height. To determine significant wave height at a certain water depth h we use the graphs of Figure 3.19 which is introduced in CIRIA/C683 2007. Five graphs are given for different wave steepness in deep water, s_{op} = 0.01, 0.02, 0.03, 0.04 and 0.05.

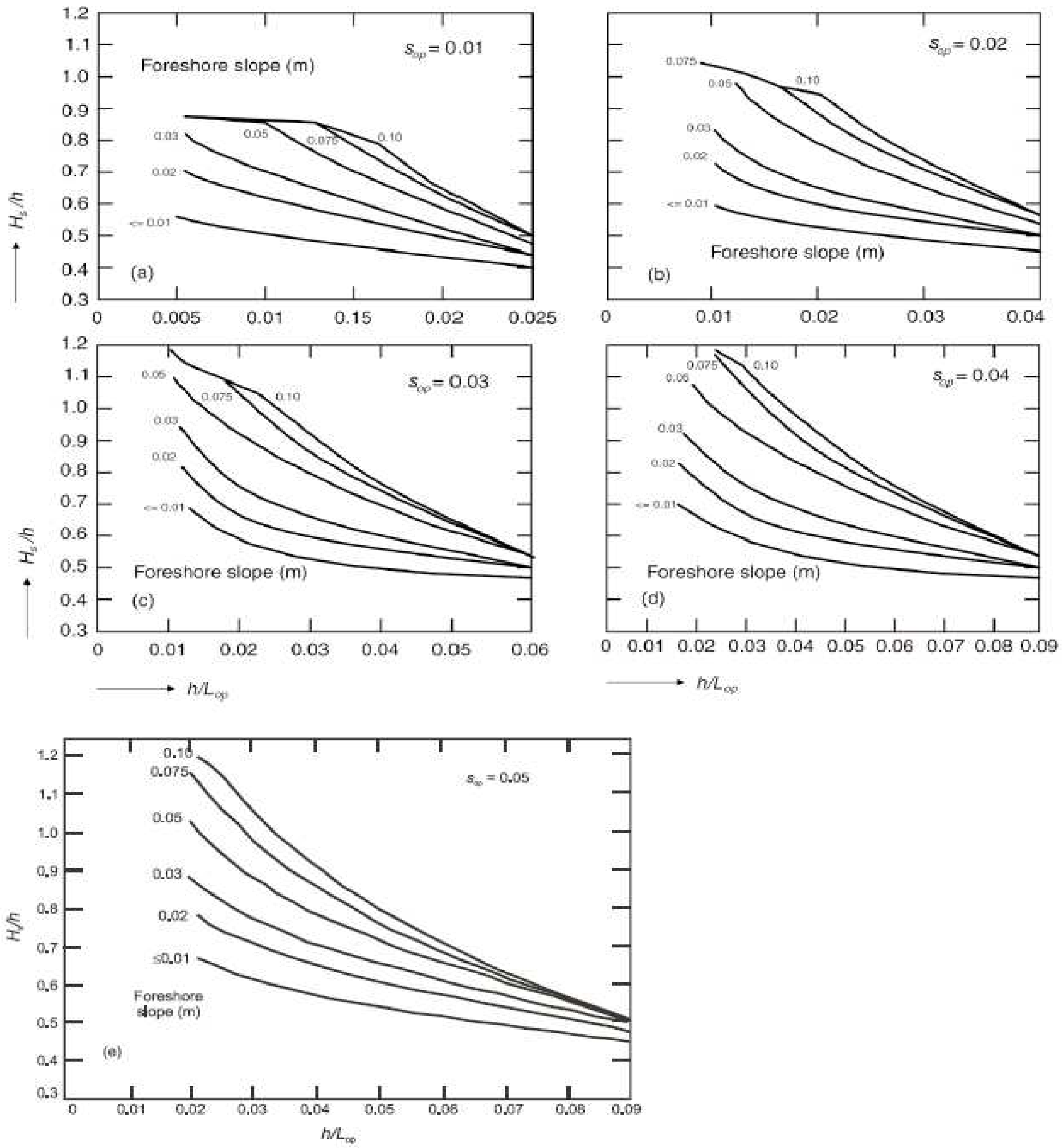


Figure 3.19: Shallow-water significant wave heights on uniform sloping foreshore [CIRIA/C683 2007]

The method for using these graphs of Figure 3.19 is also introduced in CIRIA/C683 2007, as follows:

1. Determine the deep water wave steepness $s_{op} = H_{so}/L_{op}$,
2. Determine the local relative water depth, h/L_{op};
3. Determine the slope of the foreshore, m = tanα. Curves are given for range m = 0.075 to 0.01. For gentler slope the 0.01 slope should be used.
4. Enter the two selected graphs with calculated h/L_{op} and read the breaker index H_s/h from the curve of the calculated foreshore slope.
5. Interpolate linearly between two values of H_s/h to find H_s/h for the correct wave steepness.

The calculation result of significant wave height near the toe of the dike in Nam Dinh coast and Hai Phong coast are introduced in Table 3.12 and Table 3.13, respectively.

Table 3.12: Calculation of significant wave height near the toe of the dike in Nam Dinh coast

Scenario	1/25	1/50	1/100
Design Water Level, m	3.7	4.0	4.2
Sea bed level at 100m far from toe of sea dike, m	-2.0	-2.0	-2.0
Design Water Depth, m	5.7	6.0	6.2
Significant wave height in deep water, m	**5.4**	**6.4**	**7.5**
Peak wave period T_p, s	10.5	11.4	12.3
Mean wave period T_m, s	8.4	9.1	9.8
Wave length in deep water L_{op}, m	171	204	237
Wave steepness s_{op}, (-)	0.032	0.032	0.032
h/L_{op}, (-)	0.033	0.029	0.026
Foreshore slope m, (-)	0.0057	0.0057	0.0057
Hs/h	**0.491**	**0.511**	**0.528**
Incidence wave angle in deep water β_0, deg	30	30	30
Incidence wave angle near shore β, deg	12.8	12.0	11.4
Refraction factor Kr, (-)	0.9423	0.9409	0.9399
Shoaling factor Ks, (-)	1.1027	1.1317	1.1572
Significant wave height in front of structures, m	**2.6**	**2.9**	**3.1**

Table 3.11: Calculation of significant wave height near the toe of the dike in Hai Phong coast

Scenario	1/25	1/50	1/100
Design Water Level, m	3.34	3.50	3.67
Sea bed level at 100m far from toe of sea dike, m	-2.65	-2.65	-2.65
Design Water Depth, m	5.99	6.15	6.32
Significant wave height in deep water, m	**5.4**	**6.4**	**7.5**
Peak wave period T_p, s	10.5	11.4	12.3
Mean wave period T_m, s	8.4	9.1	9.8
Wave length in deep water L_{op}, m	171	204	237
Wave steepness s_{op}, (-)	0.032	0.032	0.032
h/L_{op}, (-)	0.035	0.030	0.027
Foreshore slope m, (-)	0.011	0.011	0.011
Hs/h	**0.492**	**0.515**	**0.526**
Incidence wave angle in deep water β_0, deg	30	30	30
Incidence wave angle near shore β, deg	13.1	12.2	11.5
Refraction factor Kr, (-)	0.9429	0.9412	0.9401
Shoaling factor Ks, (-)	1.3230	1.3230	1.3830
Significant wave height in front of structures, m	**2.8**	**3.0**	**3.1**

3.6 Summary

Table 3.12 is illustrated mainly design parameters in Nam Dinh and Hai Phong coasts. These parameters are used to design the dike for the application coastal areas in Nam Dinh and Hai Phong in next chapters.

Table 3.12: Mainly design parameters in Nam Dinh and Hai Phong coastal areas.

Design variable	**Safety Standard Scenarios**		
	1/25	**1/50**	**1/100**
Significant Wave Height in deep water (m)	5.4	6.4	7.5
Peak Wave Period (s)	10.5	11.4	12.3
Mean Wave Period (s)	8.4	9.1	9.8
Hai Hau coast in Nam Dinh:			
Design Water Level (Phu Le station) (m)	3.71	3.95	4.21
Sea bed level at 100m far from toe of sea dike (m)	-2.0	-2.0	-2.0
Design Water Depth (m)	5.71	5.95	6.21
Significant Wave Height near the toe of the dike (m)	2.6	2.9	3.1
Cat Hai coast in Hai Phong:			
Design Water Level (Hon Dau station) (m):	3.34	3.50	3.67
Sea bed level at 100m far from toe of sea dike (m)	-2.65	-2.65	-2.65
Design Water Depth (m)	5.99	6.15	6.32
Significant Wave Height near the toe of the dike (m)	2.8	3.0	3.1

In Table 3.12 some remarks are realized:

- Design frequency 1/25 wave heights near the toe of the dike are almost equal to design frequency 1/100 wave heights near the toe. The reason is due to wave breaking when propagating to the toe of the dike.
- Wave period is slightly different in different design frequencies (1/25, 1/50 and 1/100 years).

CHAPTER IV ONE VERSUS TWO DEFENSE LINES

4.1 General philosophy

This study uses wave overtopping as the dominant failure mode which plays the most important role in the determination of the height as well as the layout of the sea defence system. Wave overtopping and its permissible overtopped discharges will be used as a central concept in all analyses and comparisons of options that will be considered.

The dike height/ layout of defences are mainly determined by the quantity of wave overtopping.

Dikes can be made resistant to wave overtopping in two ways: making them so high that there is hardly any or no overtopping during storm period or make them lower and strong (especially the crest and the inner slope) so that they can withstand the wave load and allow more overtopping flow.

The first approach (making dikes so high) leads to very high and wide dikes, which also means money and space consuming. By this approach we need only one line of flood defense system.

The second approach (making dikes lower and strong enough) leads to a low and strong first dike line, and possibly a lot of water in the area behind the first dike line. Thus we need another dike line to prevent the water coming further inland. The water due to flooding from sea will be stored in a transition area lying in between these two dike lines.

Therefore, a compromising solution has to be found between the height and the strength of the inner slope of a sea dike. A permissible amount of wave overtopping will determine the dike height. In return, to withstand such an overtopping flow, appropriate reinforcements of the crest and the inner slope should be applied.

Application one of both approaches depends on the lands where you are, the living and the activities in that areas.

Both approaches are analyzed on basis of the current safety standard of Vietnam for sea dike design. In which actual sea dikes in Vietnam are designed for an extreme storm condition with return period of 25 years (design frequency is 1/25). Some concluding remarks will be pointed out by applying different return periods: 50, 100 years.

4.2 Global option 1: One defense line

One defensive line of sea dikes are used in many countries in the world (e.g. The Netherlands, France, Germany, UK...). The dike bodies of this system are very strong and so high that almost no overtopped water is allowed. The outer slope is heavily protected by revetment while it is not a must to protect the inner slope and crest from wave attacks. However it is often the case that dike crest and inner slope are partly protected in combination of other purposes, for instance to avoid soil erosion due to rain water; using crest as traffic roads. Figure 4.1 illustrates a cross-section using one defence line system. The initial construction cost of these dikes is often very high.

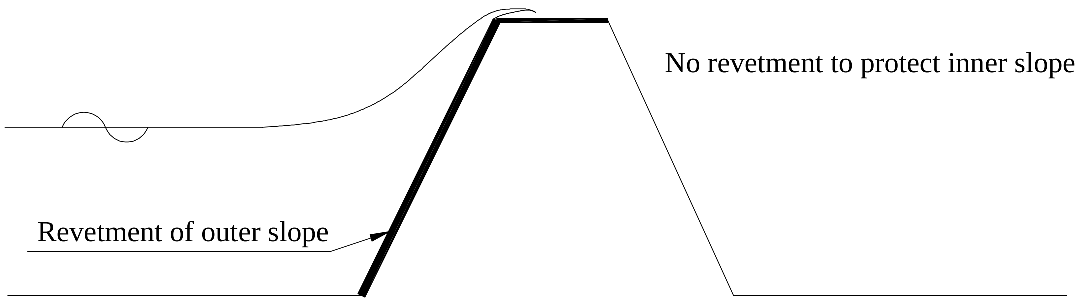


Figure 4.1: Cross-section of one defence line

4.3 Global option 2: Two defense lines

When two defense lines of sea dike are used, it means that some sea water is allowed to overtop the crest of the first dike under some circumstances. Therefore, during storm condition behind the first dike there can be amount of sea water. The first dike not only has revetment to protect outer slope, but also revetment to protect its inner slope, therefore this dike is un-breakable. To prevent the flood water to flow more landward, a second defense line of sea dike is constructed. The area in between two defense lines is considered as space for sea flood water during storm. The questions are:

- Do people allow living in this transitional area?
- If it is possible, how to evacuate when flood occurs?
- What is the height of the second dike?
- How do waves load on the second dike?

Figure 2 illustrates a cross-section of a two defence lines system.

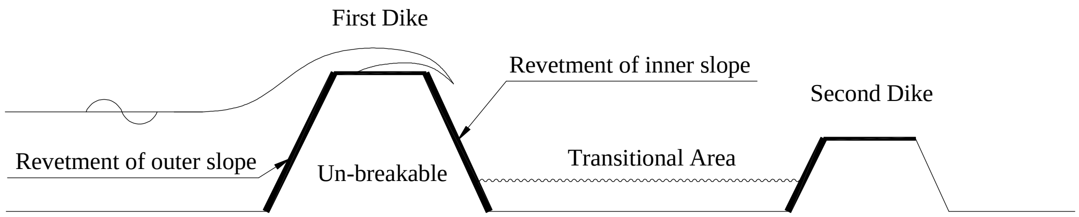


Figure 4.2: Showing of cross-section of using two defence lines

Dimensions of this sea dike system is determined on the basis of cost balancing between initially investment costs of construction, maintenance costs, value of land behind the first and second dikes as well as safety standard of the whole region.

Using one and two defensive lines is analyzed for the applying of the current safety standard of sea dikes design in Vietnam with return period of 25 years for storm occurrence (design frequency is 0.04). After that different return periods will be applied with return period of 50 and 100 years.

4.4 Overtopping criteria for design dike heights

It is obvious that too low dikes lead to flooding by breaching of the dike due to too much wave overtopping. A safe approach is if no significant overtopping is allowed. In general this means that the crest height should not be lower than the 2%-wave run-up level (is often indicated by $R_{u2\%}$) [Pilarzyk, 1998].

Another criterion for dike height design and examination is the admissible wave overtopping rate. This admissible overtopping rate depends on various conditions:

- How passable, practicable or trafficable the dike crest and inner berms must be in view of emergency measures under extreme conditions.
- The admissible total volume of overtopping water with respect to storage or drain off. Storage or drain off problem behind the dike may have an effect on safety of hinterland. If so, overtopping should be limited.
- The resistance against erosion and local sliding of crest and inner slope due to overtopping water. Which criterion applies depend of course also on the design of the dike and the possible presence of buildings.

For examination of the dike height also the governing height has to be defined and determined. The dike has an outer slope, a certain crest width and an inner slope. Heights have to be measured at the outer slope line at least every 20m of a dike section, and the lowest value is then taken as the governing dike height. A dike section is determined by similar characteristics within its length.

If extreme run-up levels exceed the crest level, the structure will be overtopped. This may occur for relatively few waves during the design event, and a low overtopping rate may often be accepted without severe consequences to the structure or the protected area. In the design of hydraulic structures, overtopping is often used to determine the crest level and the cross section geometry by ensuring that the mean specific overtopping discharge, q (m^3/s per meter length of crest), remains below acceptable limits under design conditions. Often the maximum overtopping volume, V_{max} (m^3 per meter length of crest), is also used as a design parameter.

In the design of many hydraulic structures, the crest level is determined by the wave overtopping discharge. Under random waves the overtopping discharge varies greatly from wave to wave. For any specific case usually few data are available to quantify this variation, particularly because many parameters are involved, related to waves, geometry of slope and crest, and wind. Often it is sufficient to use the mean discharge, usually expressed as a specific discharge per meter run along the crest, q (m^3/s per m length or l/s per m length).

Suggested critical values of q for various design situations are internationally proposed in CIRIA/C683 (second edition and as updated version of CUR169) as in Table 4.1. The critical values of q in this table are applied to design structures (verhicles, buildings …) on dike crest and protections of dike (crest, slopes). In this study different options of overtopping criteria are defined on basis of the critical values allowable overtopped discharge given in Table 4.1.

Table 4.1: Critical overtopping discharges and volumes [CIRIA/C683 2007]

	q mean overtopping discharge (m^3/s per m length)	V_{max} peak overtopping volume (m^3/per m length)
Pedestrians		
Unsafe for unaware pedestrians, no clear view of the sea, relatively easily upset or frightened, narrow walkway or proximity to edge	$q > q > 3 \cdot 10^{-5}$	$V_{max} > 2 \cdot 10^{-3} - 5 \cdot 10^{-3}$
Unsafe for aware pedestrians, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	$q > 1 \cdot 10^{-4}$	$V_{max} > 0.02 - 0.05$
Unsafe for trained staff, well shod and protected, expected to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	$q > 1 \cdot 10^{-3} - 0.01$	$V_{max} > 0.5$
Vehicles		
Unsafe for driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	$q > 1.10^{-5} - 5.10^{-5}$	$V_{max} > 5 \cdot 10^{-3}$
Unsafe for driving at low speed, overtopping by pulsating flows at low levels only, no falling jets	$q > 0.01 - 0.05$	$V_{max} > 1 \cdot 10^{-3}$
Marinas		
Sinking of small boats set 5–10 m from wall, damage to larger yachts	$q > 0.01$	$V_{max} > 1 - 10$
Significant damage or sinking of larger yachts	$q > 0.05$	$V_{max} > 5 - 50$
Buildings		
No damage	$q < 1 \cdot 10^{-6}$	
Minor damage to fittings etc	$1 \cdot 10^{-6} < q < 3 \cdot 10^{-5}$	
Structural damage	$q > 3 \cdot 10^{-5}$	
Embankment seawalls		
No damage	$q < 2 \cdot 10^{-3}$	
Damage if crest not protected	$2 \cdot 10^{-3} < q < 0.02$	
Damage if back slope not protected	$0.02 < q < 0.05$	
Damage even if fully protected	$q > 0.05$	
Revetment seawalls		
No damage	$q < 0.05$	
Damage if promenade not paved	$0.05 < q < 0.2$	
Damage even if promenade paved	$q > 0.2$	

Basis approaches for overtopping determination are introduced in Figure 4.3 next page. This figure introduces various methods (Owen 1980, TAW 2002, …) which are applied for various slope protections of dike (smooth slope, rough slope) in many different conditions.

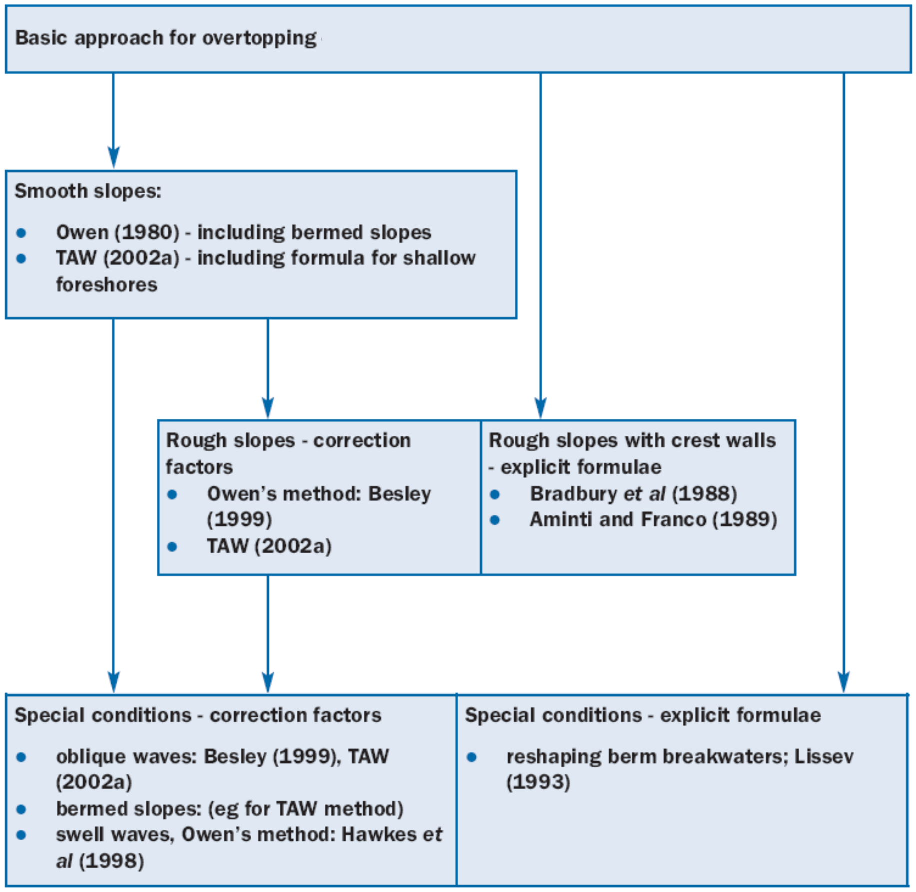


Figure 4.3: Calculation method for wave overtopping [CIRIA/C683 2007]

Definitions of protection options based on overtopping criteria:

In this study four protection options are considered. The coastal flood defence system may design with conditions of (i) Non-overtopping; (ii) Small overtopping; (iii) Medium-large overtopping or (iv) Large overtopping. Each option of overtopping condition will refer to specific design requirement of the dike and its layout (one line or two lines).

- **Non-overtopping criteria** means the dikes is designed high enough to avoid most of the wave overtopping. In this study it is referred to as a limit state of [q] = 0.1 liter/m/s: No damage of buildings; No damage of embankments and sea wall even crest is not protected. No need storage basin for flood water => no second dikes are needed

- **Small overtopping criteria** is based on a limit state of [q] = 1-10 liters/m/s: Damage if crest not protected for Embankment seawalls; No damage for Revetment seawalls. No need storage basin for flood water => no second dikes are needed

- **Medium-large overtopping criteria** is based on a limit state of [q] = 100 liters/m/s. Damage if back slope not protected for embankment seawalls; a basin need to be prepared for storage of overtopped water => Second dikes are needed.

- **Large overtopping criteria** is based on a limit state of [q] = 1000 liters/m/s. Damage if exposed components are not well heavily protected (includes: outer and inner slopes;

crest; outer and inner toes; and transition between components); a basin is certainly needed to be prepared for storage of overtopped water => Second dikes are needed and considered as primary defence line.

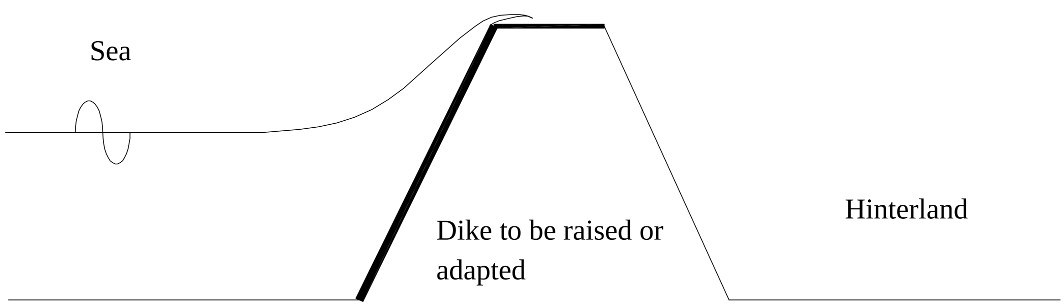
For different options the required crest freeboard of the dike and dike height is calculated on basis of corresponding overtopped discharge. The relation between crest freeboards and wave overtopped discharge is determined by applying method of TAW 2002 in CIRIA C683 – 2007. Detail determination of dike crest freeboard is referred to Chapter 5.

4.5 Defensive alternatives

On basis of overtopping criteria the following defense options are applied:

4.5.1 Non-overtopping dikes: one dike system

This option aims at constructing one primary defence line, which is designed under almost unable-overtop conditions. This one defence line must be able to withstand all loads from the outer water, the water level as well as the waves (see Figure 4.4).



Therefore the crest level is determined by a certain design water level (DWL), which consists of the tidal fluctuation and storm surge with a certain return period and extra increased water level due to local wind induced gustbump, seiches …, and the wave overtopping (or wave run-up) height R_h. The last component is known as the minimum required crest freeboard.

In most of the cases, the total crest freeboard are added with a safety margin due to the expected crest level decrease causes by dike body settlement and the expected sea level rise. For a comparison in this preliminary design stage settlement item is neglected. Thus the crest freeboard in this case is equal to the minimum required crest freeboard determining from overtopping condition: [q] = 0.1 liter/m/s.

Outer slopes are taken at 1:4 as typical existing Vietnamese sea dikes; revetments are applied to protect outer slope from erosion. The inner slope and inner side of crest is not or slightly protected, as overtopping should never occur. This results in high and strong dikes. This type is mostly applied at coastal areas with intensive land use and high land use value such as with urban/ modern industrial areas. One-dike constructions are usually expensive because of their required strength induced large dike body and often accompanied by large amounts of foundation improvement work.

Main features:

The total volume of used materials (core, filter and revetment) is large. High costs of investment but low costs of maintenances compare to the following options.

Range of application:

This option is usually applied for a large coastal low-lying area in which floods may cause catastrophic disaster for the area. This is also applied for important economic and populated areas (a coastal city with many activities along coastlines).

In practice this option has been applied mostly for coastal flood defences in Netherlands and United Kingdom. It is also used at many locations in Vietnam such as Do Son in Hai Phong; Nghia Hung in Nam Dinh; Tien Hai in Thai Binh... where in these areas there is limited space for flooding water.

4.5.2 Small overtopping dikes: one dike system

Philosophy of using this option is quite similar to the first one but its application is found in regions where some overtopped water does not cause significant damages to protect the hinterland. Therefore the dike is designed to accept some overtopping water in order of 1 to 10 liters/m/s. With this little amount of overtopped water there is no need to create a flooding storage basin. However, the inner side and the dike crest can be slightly damaged due to attacks of wave overtopped water; therefore protections of these elements are slightly needed. According to Dutch standards (TAW 2002) the inner slope of the dike is able to withstand this load by providing no extra protection or with grass covers. However experiences in Vietnam show that, although the dikes there are designed with small overtopping (no protection of inner side), damages of dike crest and inner slopes still occurs at many places. Thus, in-time repair/ inspection are required after every storm condition when applying this option.

Dike crest freeboard in this case is smaller than that in the first option. The minimum required crest freeboard determining from overtopping condition: [q] = 1 to 10 liter/m/s.

Main features:

The total volume of used materials (core, filter and revetment) is less than option 1. Lower costs of investment but higher costs of maintenances compare to the first option.

Inspection of minor damages and in-time repairs are required after occurrence of storms.

A small ditch to collect/ return overtopped sea water is recommended at inner side of the dike crest (see Figure 4.5).

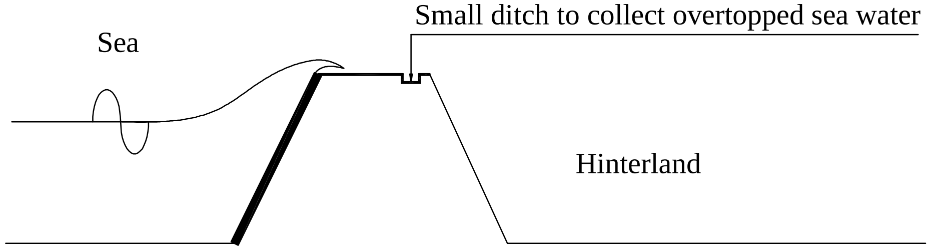


Figure 4.5: Using one dike system with small overtopping

Range of application:

This option is suggested to be used in the case of unavailability of high investment costs while yearly minor maintenance is available with respect to cost and man power. The option is used to protect less important areas as in the previous case, in which due to overtopped water, there is not any significant affect to the daily activities in the regions.

At present this option has been used for most of the coastal flood defences in Nothern Vietnam.

4.5.3 Medium large overtopping: two dikes system

The first approach by using a two dikes system is making a high crest level of the primary dike (first dike line) in combination with a low crested second dike, see Figure 4.6. With this lower first dike some wave overtopping will occur but the overtopping discharge is quite low. The low crested second dike further inland protects the hinterland of inundation by the overtopping water.

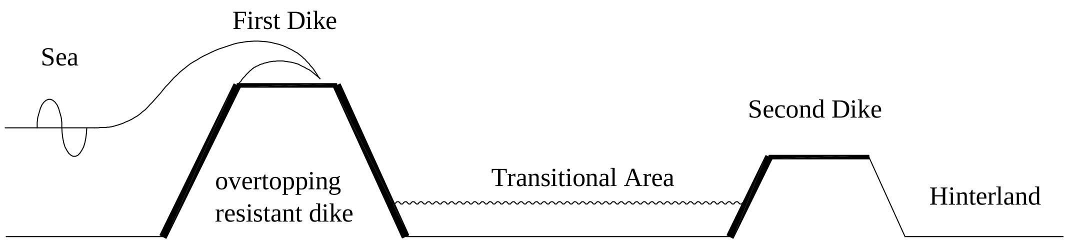


Figure 4.6: Using two dikes system with medium large overtopping

Determination of crest freeboard of the first dike is based on overtopping condition of [q] =100 l/s/m. This means that during storm the basin between two dikes is filled with a unit discharge of 100 l/m/s. With certain storm duration the total volume of sea flooded water can be determined, thus, water level in the basin can be estimated as well. This provide basis for determination of crest level of the second dike, taking into account maximum water level in the basin, increase of the basin water level due to local waves and surge (wave and surge in the basin), that are induced by local storm wind generated waves and surges.

Applying this option, revetments are certainly needed covering whole exposed surface of the first dike (outer slope, inner slope and dike crest) to prevent erosion due to wave overtopping. The second dike endures only local wave attack (wave in the basin) which is relatively small compare to the design waves. Because of this the second dike may not need to protected heavily by revetment, grass cover of the second dike slopes would be sufficient. The area between the dikes must cope with temporary inundation; housing and industrial land uses are not suitable. Salt production and shrimp farms seem an appropriate land use in these areas.

Sub-crossing dikes, with smaller cross section than the second dike, is recommended to split the basin into sub-small basin (ponds). This is useful to narrow the flooded area when it is not necessary to spread over the whole basin.

Main features:

The total volume of earth and materials for dike body is larger than second option and comparable to the first one. Although heavy protection of the first dike is required, the investment costs for this option is lower than that of these two previous options. Permanent land loss (ground used for dikes) is comparable to first option however large area between these two dikes are needed to keep flooded water.

Depending on the land-use between the two dikes, in case housing and industrial sector take place, evacuation of the inter-dike area is needed, but the area is shortly after the storm accessible again. Inspection and regular maintenance of the first dike are necessary to ensure it is “non-breachable” dike.

Range of application:

This option is usually applied for a relatively low land value coastal low-lying region in which floods may not cause serious damages for the regions. In practice this option uses as coastal flood defences of the rural/ aquaculture areas. This approach has been applied and developed in Vietnam during last 20 years at many places in the North. Local people take advantage of the basin between these two dikes to develop aquaculture i.e. shrimp farms.

4.5.4 Large overtopping: Two dike system-first dikes is wave-breaker dike

The dikes in this approach combine a wave breaking line and a water defensive line. A low first dike in combination with a higher second dike is used. The first dike is designed at a reference level with a lower return period than the systems discussed above. The main purpose of the first dike is to absorb wave energy. In principal the first dike is designed as the same as a nearshore breakwater (see Figure 4.7). During storm conditions overtopping and overflow are accepted, resulting in large quantities of salt water stored in the inter-dike area. Because the first dike should be strong enough to resist sea loads (wave and water level) it should be provided as strong amour layer as for breakwater at the same condition. The water level at interdike area can be as high as at sea, therefore no crest free board is required for the first dike. After the storm the flooded area can be drained by gravity during low tides. As even more water will be stored on land, the land use in the area must have a low economical value or cope with the high water levels.

The second dike endures less wave attack because of wave absorption of the first dike. Thus, design waves for the second dikes are chosen as wave condition after the first dike (local wave condition, determined by wave transmission mechanism). Crest freeboard of the second dike is determined based on overtopping criteria of non-overtopping condition, which is similar to the first option, [q]=0.1 l/s/m. However, the crest freeboard in this case is much lower than that of the first option because local wave condition is used.

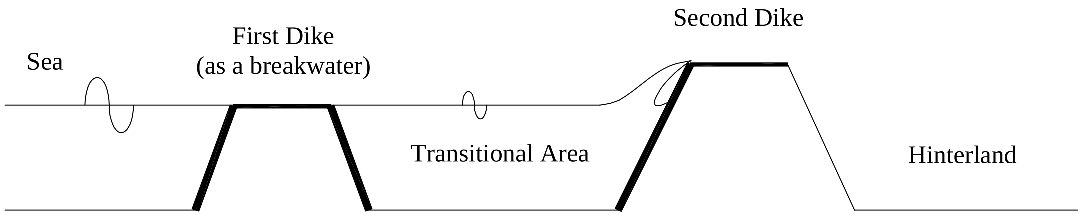


Figure 4.7: Using two dikes system with large overtopping

Main features:

The total volume of earth and materials use for dike body is smaller than in the case of the above options, however, construction of the first dike is very costly.

Land loss is comparable to third option (two dikes with medium overtopping), however, the area between these two dikes is flooded permanently and this may cause some environmental problems.

Range of application:

This option particularly should be used to protect coastal zone in which its shoreline is exposed to large wave attacks. In addition, application of this approach may be suitable for a relatively low land value at a narrow strip along the shore but a high value hinterland. Moreover, use of this approach for flood defences in combination with shore protection in eroded coast would be of help. It could be suitable to protect high productive/value islands by this option.

Of course a wide range of solutions can be interpolated between these types of two-dike systems. Most interesting points in selection of a suitable solution are the amount of overtopping water, groundwork, land value and costs. Extra measures can be taken to guide the overtopping water into storage basins or canals. Whatever solution is chosen, both the first and the second dike will have a crest level which is lower than the first dike in the first two-dike system option. The lower the first dike, the more water needs to be stored and the higher the second dike needs to be to safeguard the hinterland.

4.6 Comparative multi-criteria evaluation

In the previous section alternatives for a coastal defence system have been discussed and presented. Distinction between alternatives is made by overtopping conditions. However, based only on the overtopped discharge criteria is not enough to come up with the proper alternative for selection of a certain sea defence system. Several more criteria are needed for consideration to select the best alternative. In this study, a multi criteria evaluation is used with following criteria taken into account:

4.6.1 Investments

Total investment costs of alternatives are considered as an important aspect. The investment takes into account cost of dike body construction and cost of dike protections. The most expensive solution would be alternative 1 while alternative 3 is the cheapest. Alternatives 2 and 3 are fairly equals. In this section only qualitative volume of materials to construct the dikes is considered.

4.6.2. Maintenance

The potential damages, frequency and intensity of damage repairs of alternatives are considered as a criterion. It is often the case that an alternative with expensive investment results in a cheaper maintenance.

4.6.3 Safety of hinter lands

An assessment of the current safety level of the hinterland and of the safety level of the different alternatives is done. The alternatives are all designed on the same safety level so their value is equal.

In general, the safety of the hinterland is weighted heavily because it is considered very important.

4.6.4 Potential flood risk

Potential risk is defined as:

Risk = Probability of failure x Consequence [CUR/TAW 141, 1990]

In which: Consequence is the loss of assets due to flooding, evacuation cost, etc.

The failure probability is similar for all options and is assumed to be equal to absolute value of design frequency. However, the consequences when a flood occurs are different for the alternatives. Therefore the potential risks are different as well.

This criterion is important for any long term plan of the coastal defence works.

4.6.5 Environmental issue and ecology

Implementation of a system will certainly have its effect on the current ecology and environment. Due to salt water intrusion and higher water levels the ecology can be disturbed and changed severely. Due to construction of a dike natural environment/ wet land area can certainly be damage or destroyed.

4.6.6 Land loss and interferences

When a new system is implemented, extra space has to be created to construct it. Sometimes due to construction and dike rehabilitation local houses, villages have to be demolished or land with a certain use has to be bought from the current owners.

The amount of land with a certain use that has to be sacrificed and possible dispossession procedures are included in this criterion.

The loss of economical value and the amount of damage of the inter-dike area when this area is inundated is an aspect that cannot be neglected.

4.6.7 Re-used materials

At some place the existing dikes are there and can be used as the base for new system. The level of re-use of existing dike bodies and elevations in the new design is important and therefore this is also a criterion used in the analysis.

4.6.8 Technical feasibility

This criterion is about the availability of technology, resources, construction materials and construction equipment in implementation of the system. The available space for construction and conditions such as weather and accessibility of the construction site are also considered.

4.7 Conceptual framework for assessments

A performance matrix will be used in process of assessing eight comparative criteria, which are proposed in previous section, for each option. In this matrix, weight and rank factors are depended on own subjective.

Weighting

Eight comparative criteria are proposed to assess alternative. For every criterion the weight factors are introduced and the value of every weight factor is based on the importance of each criterion for every alternative. The total weight is 100 points and these points are distributed among the eight criteria as an example in Table 4.2.

Ranking

Each option is ranked based on the above eight criteria. Each criterion is ranked for every option from 1 (very bad) to 5 (very good). For example, the construction costs of the option 1 (very high and big dike) are highest so the option 1 is ranked of 2 (bad option based on high construction costs). On the contrary, the option 3 (lower and smaller dikes) has smaller construction costs, and this option is ranked of 5 (good option based on the construction costs).

After that we multiply the weight factor and the ranking point of each criterion together for each option that is called “score” of each criterion. Sum of total scores for each option, comparing them together, the best option has the highest score.

Beside these above four alternatives, when applying this MCE for any case study a “zero option” will be introduced. The zero option demonstrates the present situation of coastal flood defences of the case study area.

Table 4.2: Example of a MCE

No.	Criteria	Weight		Zero Opt	Option 1	Option 2	Option 3	Option 4
1	Investment	15	Weight	15	15	15	15	15
			Rank	5	1	4	3	2
			Score	75	15	60	45	30
2	maintenance	5	Weight	5	5	5	5	5
			Rank	1	5	3	4	2
			Score	5	25	15	20	10
3	Safety of hinter land	20	Weight	20	20	20	20	20
			Rank	1	4	4	4	4
			Score	20	80	80	80	80
4	Potential flood risk	25	Weight	25	25	25	25	25
			Rank	2	2	2	4	4
			Score	50	50	50	100	100
5	Environment Impact	10	Weight	10	10	10	10	10
			Rank	4	2	4	3	1
			Score	40	20	40	30	10
6	Land loss for defence system and its value	10	Weight	10	10	10	10	10
			Rank	5	3	4	2	1
			Score	50	30	40	20	10
7	Re-used material	10	Weight	10	10	10	10	10
			Rank	5	2	2	3	1
			Score	50	20	20	30	10
8	Feasibility	5	Weight	5	5	5	5	5
			Rank	1	3	3	4	1
			Score	5	15	15	20	5
Total Score		**100**		**295**	**255**	**320**	**345**	**255**
Selection: 1 (best) to 5 (worst)				3	4	2	1	4

Zero Option: Do nothing	
Option 1:	Non-overtopping - q = 0.1 l/s/m, one very high dike system
Option 2:	Small overtopping - q = 10 l/s/m, one relatively high dike system
Option 3:	Medium large overtopping - q = 100 l/s/m, two dike system, first dike is higher than second dike
Option 4:	Large overtopping - q = 1000 l/s/m, two dike system (first dike - as breakwater, is lower than second dike)
Ranking factor: 1 (very bad) to 5 (very good)	

Apparently, based on Multi Criteria Evaluation, it can be concluded that applying alternative 3 is the best solution in terms of given criteria.

4.8 Application case of Hai Hau, Nam Dinh

4.8.1 Determination of weight factor for each criterion

Investment cost

The coastal zone of the Nam Dinh province is not an industrial area. The coastal zone mostly aquacultures, salt fields and agricultures, except some small places houses which start to be recreational areas for touristic development. In general, the value of land behind the dikes as well as annual income of local people is not so high. This also means that the available budget is limited for construction of the heavy flood defence system. Due to that fact the investment criterion for coastal area in Nam Dinh is very importance. The weight factor of 25 is chosen in the case of application at Hai Hau coast in Nam Dinh.

Maintenance cost

Weight factor of maintenance cost criterion is chosen as 5, because of the cheaper investment, the more expensive maintenance have to be accepted.

Safety of hinterland & getting less potential risk

Safety of hinter land and getting less potential risk are the most important criteria for most of flood defence system. Therefore the weight factors are chosen of 20.

Environmental impact

When a defence system of a coastal area is implemented the current ecology and environment of the area will be affected. Wet lands are used as ground space for constructing the system, some plants therefore are destroyed. However, along the coastline of Nam Dinh the wetland areas were already destroyed long time ago due to development of agriculture and aquaculture. From all these reason the weight factor of the environment impact is chosen by 10.

Land uses

The quantity of land, which is used for construction of flood defence system, is available at Hai Hau coastal area. There is enough space to construct the defence system in this area. However, local people will not accept easily when their land for life is lost. Therefore, the weight factor is chosen of 10 for this criterion.

Re-use material

There exists already sea dikes system along the coast. New sea defence system or upgrading system will be located at the same location. Uses of existing sea dike’s body and its materials are one of important fact to save money. The value of 10 is chosen for the weight factor of this criterion.

Feasibility

As development of Vietnam in general and Nam Dinh in particular during last, at this moment there is enough available construction technologies and equipments to implement all kind of dike system along the coastlines of Hai Hau. The road system is good enough to mobilize all construction equipments and materials to the coastlines.

This means we do not have to concern too much on this criterion. Its weight factor is chosen as light as 5.

4.8.2 Rank of options for each criterion

Investment cost

Relatively, option 3 of using two dikes, medium-large overtopped water allows (100 l/s/m), good protected of both sides of the first dike appear as the cheapest option. Next to that is option 2 of using one dike. The most expensive option is using two dikes with fisrt line such as wave breaker. The rank for this criterion is presented in table 4.3

Maintenance cost

Ranking of option 1 is the highest, because with small overtopped discharge allow results in the highest and strongest dikes, therefore, less maintenance is expected. While the zero option needs the highest maintenance as from practical experience during the last few years at Nam Dinh sea defiance. Thus its rank is the lowest one. Option 4 needs large maintenance as well.

Safety of hinterland

All options, except doing nothing as the zero option, should have the same rank for safety criterion. This is because for a certain design condition (design return period of loads) all options are required to design and construct in the way that fulfill the design conditions. Therefore, all alternatives have the same safety level for the hinterland and it is theoretically reasonable to consider so in this qualitative analysis. The safety level for each alternative is the design safety level (1/25 in this case) and we have to assume that all design are met this design safety level, which is reflected by using the same design water level and waves. Differences here are overtopped discharges on different "un-breachable" design defence systems.

Getting less potential risk

Although the chance of dike breach for each alternative is the same, however, the risk for the hinterland is not the same because consequences of system failure are not the same for every alternative, since risk is defined as: Risk=(probability of failure)*(consequences). Failure consequences are different within options due to, for instance:

(i) Traditionally people believe more in the higher and bigger one dike system, thus, attention of flood risk of local people is getting less and, consequently, more investments/economic activities are going on;

(ii) Evacuation of the 2 dike system maybe more effective then one dike system when a failure/flood occurs, because with 2 dike system a buffer zone is provided to slow-down flooding process.

Environmental impact

Using of two dike system with one wave breaker line as option 4 seems to be the most serious environmental impact since the breaker line creates a still water area in front of the second dike. Therefore this option has the lowest rank (rank =1). Option 1, with highest dikes, induces large amount of construction works as well as material and its transportation. This disturbs environment considerably (rank=2). Using option 2 and keep doing nothing relatively has the same rank of 4 for this environmental impact criterion. Option 3 has rank of 3, in between these above option.

Land uses

It is clear that doing nothing results in no extra land use, therefore rank of this option is highest at 5. In contrast, using option 3 and 4 need the largest ground space, these give rank of 2. Option 1 need more ground space than option 2 so their rank is of 3 and 4 respectively.

Re-use material

Similarly, doing nothing mean use of existing sea dikes, so 100% of re-use material for this option (rang of 5). Most of the existing dike sections can be used as part of the first dikes in Option 3. So the degree of re-use material is also high, rank of 3. Option 1 and 2 can also be considered as upgrading existing dikes therefore the old dikes can be used as a part of the new dikes, except some weak sections which need to reconstruct completely, so the rank for option 1 and 2 are both equal at 2. The Option 4 can re-use less existing dike body material so its rank is 1.

Feasibility

In term of present construction technology and the way that public reflects its opinion, Option 3 appears as the most feasible (therefore, rank =5). Option zero is hard to be accepted since damage of the system occurs every year and induces serious loss. Amongst other options Option 4 with one wave breaker dam offshore is the most difficult in construction (therefore, rank = 2). Because (i) a shallow foreshore is found along the coastline of Nam Dinh, this make difficult for water born construction equipments to approach the shore lines. (ii) Vietnam has not much construction experience in construction of offshore breakwater.

4.8.3 Summary

Table 4.3 in the next page summarized all weight factors, ranking factors and scores of eight indicated criteria above for the application of Multi Criteria Evaluation in Nam Dinh coastal area.

Table 4.3: The application of MCE in Nam Dinh coastal area

No.		Criteria	Weight	Zero Opt	Option 1	Option 2	Option 3	Option 4
1	Investment	20	Weight	20	20	20	20	20
			Rank	5	2	3	4	1
			Score	60	40	80	60	20
2	maintenance	5	Weight	5	5	5	5	5
			Rank	1	5	3	4	2
			Score	5	25	15	20	10
3	Safety of hinter land	20	Weight	20	20	20	20	20
			Rank	1	5	5	5	5
			Score	20	100	100	100	100
4	Potential flood risk	20	Weight	20	20	20	20	20
			Rank	2	2	2	4	4
			Score	40	40	40	80	80
5	Environment Impact	10	Weight	10	10	10	10	10
			Rank	4	2	4	3	1
			Score	40	20	40	30	10
6	Land loss for defence system and its value	10	Weight	10	10	10	10	10
			Rank	5	3	4	2	2
			Score	50	30	40	20	20
7	Re-used material	10	Weight	10	10	10	10	10
			Rank	5	2	2	4	1
			Score	50	20	20	40	10
8	Feasibility	5	Weight	5	5	5	5	5
			Rank	1	3	3	4	2
			Score	5	15	15	20	5
Total Score		**100**		**310**	**290**	**330**	**390**	**260**
Selection (1=best; 5=worst)				3	3	2	1	5

Zero Option: Do nothing

Option 1: Non-overtopping - q = 0.1 l/s/m, one very high dike system

Option 2: Small overtopping - q = 10 l/s/m, one relatively high dike system

Option 3: Medium large overtopping - q = 100 l/s/m, two dike system, first dike is higher than second dike

Option 4: Large overtopping - q = 1000 l/s/m, two dike system (first dike - as breakwater, is lower than second dike)

Ranking factor: 1 (very bad) to 5 (very good)

As shown in the Table 4.3 total score of the Option 3 – medium large overtopping, which requiring of two dike systems, has the highest score. The second highest in total score is the option 2 – small overtopping with using one dike system. The different of total score of these both options is considerable (60 scores). Therefore using two dike systems allowed medium large overtopping is suggested in Nam Dinh coastal area. It is in good agreement with the actually use of the sea defence system in Nam Dinh (Hai Hau is currently using a two dike system).

4.9 Application case of Cat Hai, Hai Phong

4.9.1 Determination of weight factor for each criterion

Hai Phong is in the process of becoming a civilized and modern seaport city, a main gate to the sea, and an industrial, service, tourism and fishery centre in the North with a developed foundation of economy, education and training, technology - environment and infrastructure; a firm security - nation defense and further improving people's living condition.

Generally the land value in Hai Phong is high, as high as the second most expensive regions in Northern Vietnam, after Hanoi land. Cat Hai Island is selected in this study which is subjected to a fast economic growth region of Hai Phong due to touristic development and fishery sector during last 15 years.

The budget of Hai Phong province is available for constructing a strong flood defence system which causes severe damages for living and other activities production beside coastline if it is weak. Therefore the weight factor of investment cost criterion is chosen by 10. And maintenance has weight of 5 (see Table 4.4).

Protecting of city and industrial areas behind the coastline is a vital importance function in Hai Phong. If these areas are flooded consequence of flooding will be become serious, living and industrial activities will be affected. These cause damage of structures and lose of life. Due to this importance function of flood defence system in Hai Phong, the criteria of hinter lands safety and potential of risk are become of most importance criteria. These comparative criteria are given with weight factor of 25 and 20, respectively, as showing in Table 4.4 below.

Re-use material of existing coastal defence system is not very important in Hai Phong area since local material such as soil and rock is available. Beside, as personal communication with the local authority it is necessary to re-arrange the defence lines in order to fit well with the local development plan. Therefore it is not necessary if the new dike system is on exact place of the existing one. The weight of re-use material criterion in this case is recommended at 5.

Weight factor of others criteria is similar to that of the Nam Dinh case (See table 4.4).

4.9.2 Rank of options amongst each criterion

Since all the options used from previous case of Nam Dinh are introduced for this case, basically, it is reasonable to keep the option ranks as the same as the previous for almost all criteria. Excepting slightly different is found with potential risk criterion. In this case the potential risk of option one is higher than that of option 2. This can be explain that due to local high land value; fast development the consequences of system failure of Option 1 must be higher than Option 2 since local people and investor trust more on Option 1.

4.9.3 Summary

Adapting all weight factors, ranking factors result in scores of eight indicated criteria above for options of coastal flood defence of Cat Hai Island in Hai Phong as shown in Table 4.4 below.

Table 4.4: The application of MCE in Hai Phong coastal area

No.	**Criteria**	**Weight**		**Zero Opt**	**Option 1**	**Option 2**	**Option 3**	**Option 4**
1	Investment	10	Weight	10	10	10	10	10
			Rank	5	2	4	3	1
			Score	30	20	40	30	10
2	maintenance	5	Weight	5	5	5	5	5
			Rank	1	5	3	4	2
			Score	5	25	15	20	10
3	Safety of hinter land	25	Weight	25	25	25	25	25
			Rank	1	5	5	5	5
			Score	25	125	125	125	125
4	Potential flood risk	20	Weight	20	20	20	20	20
			Rank	2	2	3	4	4
			Score	40	40	60	80	80
5	Environment Impact	10	Weight	10	10	10	10	10
			Rank	4	2	4	3	1
			Score	40	20	40	30	10
6	Land loss for defence system and its value	20	Weight	20	20	20	20	20
			Rank	5	3	4	2	1
			Score	100	60	80	40	20
7	Re-used material	5	Weight	5	5	5	5	5
			Rank	5	2	2	3	1
			Score	25	10	10	15	5
8	Feasibility	5	Weight	5	5	5	5	5
			Rank	1	3	3	4	2
			Score	5	15	15	20	10
Total Score		**100**		**290**	**315**	**385**	**360**	**270**
Selection (1=best; 5=worst)				4	3	1	2	5
Zero Option: Do nothing								
Option 1: Non-overtopping - q = 0.1 l/s/m, one very high dike system								
Option 2: Small overtopping - q = 10 l/s/m, one relatively high dike system								
Option 3: Medium large overtopping - q = 100 l/s/m, two dike system, first dike is higher than second dike								
Option 4: Large overtopping - q = 1000 l/s/m, two dike system (first dike - as breakwater, is lower than second dike)								
Ranking factor: 1 (very bad) to 5 (very good)								

As shown in the Table 4.4 total score of the Option 2 – small overtopping, which requiring of one lower dike system, has the highest score. The second highest one is the option 3 – medium large overtopping with using two dike system. Based on analysis result it is recommended to use one dike system with small overtopping allowed for Hai Phong coastal area. It is in good agreement with the actually use of the sea defence system in Cat Hai, Hai Phong (Cat Hai is currently uses one dike system).

For both case studies an interesting result was found between Zero Option and Option 4. Using option 4 is not as good as doing anything using integrated multi-criteria comparison as in this study.

4.10 Discussions

Advantages of MCE method to find out the best solution of sea flood defence system: Based on the general information of socio-economic development, people's living condition, and economic strategy developments and also required safety standard of protected zone etc., the MCE tool could be used to develop the master plan of flood defence system.

Applied MCE tool in process of selecting sea flood defence system for the case study of Cat Hai coastal area in Hai Phong indicate that the using of one defensive line is more suitable than the other.

Also by application of MCE tool, the best solution of sea flood defence system in the case study of Hai Hau in Nam Dinh is found, that it is the flood defence system with two defensive dike lines. This result also indicates that in regions which have space enough to make a large transitional area for storing overtopping water from the sea during storm duration, the two defence lines system should be used.

MCE tool also has disadvantages in process of assessing the performance matrix. It is difficult to distribute weight factor for each criterion and this assessment is depended on own subjective.

Thus, to apply the MCE tool more effectively and close to the actual situation, some activities should be under taken before making the matrix of MCE. They could include:

- Survey public opinion by distribute surveying form to civilians in the region;
- Organize multidisciplinary meetings with all stake holder in the region;
- Final decision on weight factor should be based multidisciplinary expert assessment and practical situations in the area.

Nevertheless, for a preliminary stage, the MCE is a useful tool to orientate well and to indicate briefly which defence option should be considered. In order to come up with more consistent answers, further analysis must be done, based on i.e. cost benefit analysis, partly and/ or fully risk assessment, etc. These issues will be dealt within Chapter 5.

CHAPTER V RISK BASED COST BENEFIT ANALYIS OF ALTERNATIVE DEFENCE OPTIONS

5.1 Methodology and assumption

This Chapter presents the application of risk based cost benefit analysis to find out the best solution amongst given other options. Again three scenarios (with return period are 25 years, 50 years and 100 years) could be considered in the analysis.

According to the method of fundamental economic optimization (see Dantzig, 1956), the total costs in a system (C_{tot}) are determined by the sum of the initial investment cost ($I_{initial}$) for building the dike; the present maintenance cost during the lifetime of the system $C_{maintenance}$; the expected value of land space which is used for building the system (to place the dike system), $C_{land\ use}$; and the present expected value of the economic damage *E(D)* in case of flood occur. The optimal economic solution is the option which has the lowest total cost.

$$C_{tot} = I_{initial} + PV(C_{maintenace}) + C_{landuse} + PV(E(D)) \qquad (5.1)$$

Due to time limited therefore only scenario 1, with return period of 25 years and is actually used in Vietnam, is considered. An analysis of other scenario could be easily done by applying a similar approach. Case studies of both Nam Dinh and Hai Phong coastal flood defences are analysed in Section 5.5 and Section 5.6, respectively.

Chapter 4 already narrowed down better sea defence options, these are: (i) Option 1: non-overtopping dike; (ii) Option 2: Small overtopping dike and (iii) Option 3: Medium-large overtopping first dike and second dike is needed. In this chapter, three better options are considered and compared to find out the best option.

5.2 Establishment of cost function for option of using one dike line

- Initial investment cost function:

$$I_{initial} = f(H_{dike}) = C_1.A_H + C_2.L_{outer} + C_3.L_{inner} + C_4.w \qquad (5.2)$$

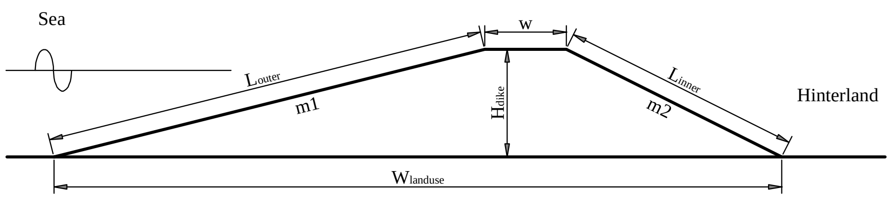


Figure 5.1: Specific parameters of one dike line system

In which: C_1: the rate of constructed dike body, Mil. USD/m^2/km

C_2: the rate of constructed outer slope protection, Mil. USD/m/km

C_3: the rate of constructed inner slope protection, Mil. USD/m/km

C_4: the rate of constructed cost of crest protection, Mil. USD/m/km

And:

Cross section area of the dike:

$$A_H = \left[0.5 * H.(m_1 + m_2) + w\right].H \tag{5.3}$$

Length of outer slope:

$$L_{outer} = H.\sqrt{1+m_1^2} \tag{5.4}$$

Length of inner slope:

$$L_{inner} = H.\sqrt{1+m_2^2} \tag{5.5}$$

- Expected maintenance cost:

$$C_{maintenance} = f(q) \tag{5.6}$$

With q is average overtopping discharge in l/s/m.

The maintenance cost is determined by using inventory data of the actual annual maintenance costs in Vietnam which can be derived from the wave overtopping discharge and total investment cost.

Sea dike at Do Son in Hai Phong province was upgraded to withstand with *[q]=0.1l/s/m*, the maintenance cost was estimated approximately 0.1% of $I_{initial}$ per year. Thus:

$$PV(C_{maintenance}) = 0.1\% * I_{initial} * \sum_{i=0}^{i=T} \frac{1}{\left(1+r'\right)^i} \tag{5.7}$$

Most of other sea dike sections in Vietnam was not designed based on overtopping criteria. However, the actual dikes are corresponded to amount of wave overtopped discharge of about 20 l/s/m. The experienced value of maintenance cost for these dikes was about 0.5 – 1% of the initial investment cost. Therefore the maintenance cost is roughly estimated by:

$$PV(C_{maintenance}) = 1\% * I_{initial} * \sum_{i=0}^{i=T} \frac{1}{\left(1+r'\right)^i} \tag{5.8}$$

In which T is the lifetime of the dikes, in year; r is the economic growth rate.

- Cost of land-use of one dike system is determined by equation (5.9) below:

$$C_{landuse} = C_5.W_{landuse} \tag{5.9}$$

In which: C_5 is the rate of land use, Mil. USD/m/km;

$W_{landuse}$ is the length of land which is used to place dike system, m.

- The expected value of economic damage has to be discounted to present value with the reduced value rate r' = r – g, in which r is the real interest rate and g is the economic growth rate.

The expected value of the economic damage can be calculated from the probability of flooding (P_f), the damage caused by the flood (D), and the reduced value rate (r'), see equation (5.10) below.

$$PV(E(D)) = P_f * D * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{5.10}$$

5.3 Establishment of cost function for option of using two dike lines

- The initial cost of this option is calculated by equation(5.11), (5.12) and (5.13) below:

$$I_{initial} = f_1(H_{dike1}) + f_2(H_{dike2}) \tag{5.11}$$

$$f_1(H_{dike1}) = [C_1.A_{H1} + C_2.L_{outer1} + C_3.L_{inner1} + C_{crest}] \tag{5.12}$$

$$f_2(H_{dike2}) = [C_1.A_{H2} + C'_2.L_{outer2} + C'_3.L_{inner2} + C'_{crest}] \tag{5.13}$$

In which $f_1(H_{dike1})$ and $f_2(H_{dike2})$ are the initial cost function of the first dike and the second dike of defence system, respectively.

C'_1, C'_2 and C'_3 are the unit costs for different components (slopes and crest protection) of the second dike. So it may be smaller than C_1, C_2 and C_3 because it is not necessary to give heavy protection for the second dike.

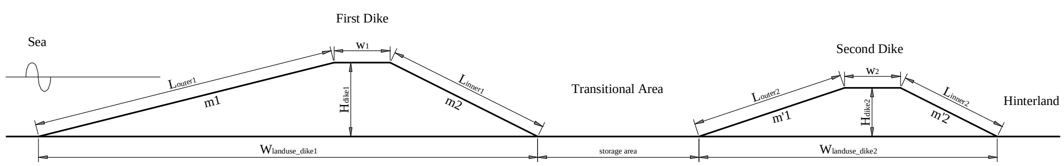


Figure 5.2: Specific parameters of two dike lines system

And:

Cross section area of the first dike:

$$A_{H1} = [0.5 * H_1.(m_1 + m_2) + w_1].H_1 \tag{5.14}$$

Length of outer slope of the first dike:

$$L_{outer1} = H_1.\sqrt{1+m_1^2} \tag{5.15}$$

Length of inner slope of the first dike:

$$L_{inner1} = H_1.\sqrt{1+m_2^2} \tag{5.16}$$

Cross section area of the second dike:

$$A_{H2} = [0.5 * H_2.(m'_1 + m'_2) + w_2].H_2 \tag{5.17}$$

Length of outer slope of the second dike:

$$L_{outer2} = H_2.\sqrt{1+m'^2_1} \tag{5.18}$$

Length of inner slope of the second dike:

$$L_{inner2} = H_2.\sqrt{1+m'^2_2} \tag{5.19}$$

- Expected maintenance cost:

$$C_{maintenance} = C_{maintenance\_dike1} + C_{maintenance\_dike2} \tag{5.20}$$

Corresponding to overtopping discharge *[q]=100 l/s/m* maintenance cost is roughly estimated approximately $C_{maintenance} = 0.7\% * I_{initial}$ per year to maintaining the first dike, and $C_{maintenance} = 0.2\% * I_{initial}$ per year for maintenance of the second dike. Therefore the present maintenance cost of the first and the second dike are determined by equation (5.21) and (5.22), respectively.

$$PV(C_{maintenance}) = 1\% * I_{initial} * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{5.21}$$

$$PV(C_{maintenance}) = 0.1\% * I_{initial} * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{5.22}$$

In which T is the lifetime of the dikes, in year; r' is the reduced value rate.

- The cost of land using of two dikes system is determined in equation (5.23) below:

$$C_{Landuse} = C_5.(W_{landuse\_dike1} + W_{landuse\_storage} + W_{landuse\_dike2}) \tag{5.23}$$

In which:

C_5 is the rate of land use, Mil. USD/m/km;

$W_{landuse\_dike1}$ is the width of land which is used to place the first dike;

$W_{landuse\_storage}$ is the width of land which is used in storage area to place sub-crossing dikes;

$W_{landuse\_dike2}$ is the width of land which is used to place the second dike.

- Expected value of economic damage when flood occurs (during T years of lifetime):

$$PV(E(D)) = P_f * D * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{5.24}$$

In which: P_f is the probability of flooding; D the damage caused by the flood, and r is the reduced value rate.

5.4 Preliminary design of dike cross section

In this section, some main dimensions of the dike are determined. Dike crest level (dike height) is determined by wave run-up and wave overtopping conditions. Amour layers for slope protection are determined by stability condition under wave attack. Slope angles of the dike (outer and inner slope) can be determined by geotechnical stability criteria. Dike crest width depends on requirements of construction, operation and maintenance works, it may also be used as a road for local region. The parameters of slope angles and crest width of the dike are set as same as the actual dimensions of sea dikes in Vietnam:

- Outer slope $m_1 = m'_1 = 4$;
- Inner slope $m_2 = m'_2 = 2$;
- Crest width $w = w_1 = w_2 = 5$ m.

5.4.1 Dike crest level

Dike crest level is determined by:

Dike Crest Level = DWL + R_c + SLR

In which:

DWL: design water level, + MSL m

R_c: crest freeboard, that is the vertical distance between DWL and dike crest, m

SLR: sea level rise due to climate change, m

In this study sea level rise is considered of 0.2 m per century. The lifetime of the dike is considered of 50 years. The sea level rise of 0.1 m therefore is accounted in this study.

Crest freeboard is determined by two conditions (wave run-up and wave overtopping conditions) which are introduced following paragraphs.

❖ **Wave run-up level**

Wave run-up levels are determined by TAW (2002) which presents two equations for determination of wave run-up:

$$\frac{R_{u2\%}}{H_{m0}} = A.\gamma_b.\gamma_f.\gamma_\beta.\xi_{m-1,0} \tag{5.25}$$

With a maximum or upper boundary for larger values of $\xi_{m-1,0}$ of:

$$\frac{R_{u2\%}}{H_{m0}} = \gamma_f.\gamma_\beta.\left(B - \frac{C}{\sqrt{\xi_{m-1,0}}}\right) \tag{5.26}$$

In which:

- Breaker parameter $\xi_{m-1,0}$ is calculated by equation (5.27):

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{s_{m-1,0}}} \tag{5.27}$$

- Wave steepness $s_{m-1,0}$ is determined by equation (5.28).

$$s_{m-1,0} = \frac{2\pi}{g} \cdot \frac{H_{m0}}{T_{m-1,0}^2} \tag{5.28}$$

- The energy wave period $T_{m-1,0}$ can be estimated from peak wave period T_p by equation (5.29) [CIRIA/C683 2007].

$$T_{m-1,0} = \frac{T_p}{1.1} \tag{5.29}$$

- γ_b is the reduction factor of berm. Due to straight slope in this study (no application of berm) therefore γ_b = 1.0;
- γ_f is the reduction factor of roughness slope. This factor is determined based on type of protection. Roughness reduction factors for slope covered with concrete armour unit are of 0.75;
- γ_β is the reduction factor due to oblique wave. This correction factor for oblique short-crested waves is given by equation.

$$\gamma_\beta = 1 - 0.0022|\beta| \qquad \text{for } 0^0 \le |\beta| \le 80^0 \tag{5.30}$$

For angles of approach, $\beta > 80^0$, the result of $\beta = 80^0$ can be applied [CIRA/C683 2007].

Values for coefficients A, B and C in equations (5.25) and (5.26) are given in Table 5.1 in CIRIA/C683 2007 with safety margin – deterministic calculations: A = 1.75; B = 4.3 and C = 1.6.

Wave run-up levels at Hai Hau coast in Nam Dinh and at Cat Hai coast in Hai Phong are presented in Table 5.1 and 5.2 respectively.

Table 5.1: Calculation of wave run-up on the dike slope at Hai Hau coast

Scenario	1/25	1/50	1/100
Significant wave height in front of structures, m	**2.6**	**2.9**	**3.1**
Reduction factor of roughness slope, γ_f, (-)	0.750	0.750	0.750
Reduction factor due to oblique wave, γ_β, (-)	0.972	0.974	0.975
Reduction factor of bermed slope, γ_b, (-)	1.0	1.0	1.0
The fictitious wave steepness (local) $s_{m-1,0}$, (-)	0.019	0.017	0.016
tan(α), (-)	0.25	0.25	0.25
Surf similarity (local) $\xi_{m-1,0}$, (-)	1.83	1.92	1.99
Wave run-up $R_{u2\%}$, m:			
- Eq.5.8 - C683 (Equation (5.25))	6.2	7.0	7.9

Table 5.2: Calculation of wave run-up on the dike slope at Cat Hai coast

Scenario	1/25	1/50	1/100
Significant wave height in front of structures, m	**2.8**	**3.0**	**3.1**
Reduction factor of roughness slope, γ_f, (-)	0.750	0.750	0.750
Reduction factor due to oblique wave, γ_β, (-)	0.971	0.973	0.975
Reduction factor of bermed slope, γ_b, (-)	1.0	1.0	1.0
The fictitious wave steepness (local) $s_{m-1,0}$, (-)	0.020	0.018	0.016
tan(α), (-)	0.25	0.25	0.25
Surf similarity (local) $\xi_{m-1,0}$, (-)	1.78	1.88	1.98
Wave run-up Ru2%, m:			
- Eq.5.8 - C683 (Equation (5.25))	6.3	7.2	7.9

Wave overtopping and crest level

According to wave overtopping criteria the dike crest freeboard (R_c) must be height enough to limit overtopped discharge by waves. In this study Van der Meer formulae in TAW 2002 are used to determine the R_c.

For breaking waves ($\gamma_b.\xi_{m-1,0} < \cong 2$):

$$\frac{q}{\sqrt{g.H_{m0}^3}} = \frac{A}{\sqrt{\tan\alpha}}.\gamma_b.\xi_{m-1,0}.\exp\left(-B.\frac{R_c}{H_{m0}}.\frac{1}{\xi_{m-1,0}.\gamma_b.\gamma_f.\gamma_\beta}\right) \qquad (5.31)$$

With a maximum (for non-breaking waves generally reached when $\gamma_b.\xi_{m-1,0} > \cong 2$):

$$\frac{q}{\sqrt{g.H_{m0}^3}} = C.\exp\left(-D.\frac{R_c}{H_{m0}}.\frac{1}{\gamma_f.\gamma_\beta}\right) \qquad (5.32)$$

Where:

- γ_b = 1.0 is the reduction factor of berm (no berm on the outer slope of the dike)
- γ_f is the reduction factor of roughness slope, for slope covered with concrete armour unit γ_f = 0.75;
- γ_β is the reduction factor due to oblique wave and determined by equation (5.33).

  $$\gamma_\beta = 1 - 0.0033|\beta| \qquad \text{for } 0^0 \le |\beta| \le 80^0 \qquad (5.33)$$

  For angles of approach, $\beta > 80^0$, the result of $\beta = 80^0$ can be applied [CIRA/C683 2007].
- Coefficients A, B, C and D in equations (5.31) and (5.32) are given in Table 5.8 in CIRIA/C683 2007 with safety margin – deterministic calculations: A = 0.067; B = 4.3; C = 0.2 and D = 2.3.

Table 5.3 and 5.4 present freeboards and required crest levels of dikes for each option at Hai Hau (Nam Dinh) and Cat Hai (Hai Phong) respectively.

Table 5.3: Crest level calculation of dike in Hai Hau coast – Nam Dinh

Scenario	1/25	1/50	1/100
Roughness reduction factor γ_f, (-)	0.75	0.75	0.75
Reduction factor due to oblique wave, γ_β, (-)	0.958	0.960	0.962
Reduction factor of bermed slope, γ_b, (-)	1.0	1.0	1.0
Option 1: Non - overtopping	**q =**	**0.1**	**l/s/m**
Freeboard Rc, m	8.4	9.7	11.0
Required crest level, m	**12.2**	**13.8**	**15.4**
Option 2: Small overtopping	**q =**	**10**	**l/s/m**
Freeboard Rc, m	4.7	5.5	6.3
Required crest level, m	**8.5**	**9.5**	**10.6**
Option 3: Medium large overtopping	**q =**	**100**	**l/s/m**
Freeboard Rc, m	2.8	3.4	3.9
Required crest level, m	**6.6**	**7.4**	**8.2**

Table 5.4: Crest level calculation of dike in Cat Hai coast – Hai Phong

Scenario	1/25	1/50	1/100
Roughness reduction factor γ_f, (-)	0.75	0.75	0.75
Reduction factor due to oblique wave, γ_β, (-)	0.957	0.960	0.962
Reduction factor of bermed slope, γ_b, (-)	1.0	1.0	1.0
Option 1: Non - overtopping	**q =**	**0.1**	**l/s/m**
Freeboard Rc, m	8.6	10.0	11.1
Required crest level, m	**12.1**	**13.6**	**14.9**
Option 2: Small overtopping	**q =**	**10**	**l/s/m**
Freeboard Rc, m	4.8	5.6	6.4
Required crest level, m	**8.3**	**9.2**	**10.1**
Option 3: Medium large overtopping	**q =**	**100**	**l/s/m**
Freeboard Rc, m	2.9	3.5	4.0
Required crest level, m	**6.4**	**7.1**	**7.7**

Summary

Selected crest freeboards in this study are based on overtopping conditions, because distinctions are made between three defence options which are proposed in Section 5.1.

The required crest levels of each wave overtopping discharge option are summarized in Table 5.5 at Hai Hau coast in Nam Dinh and Table 5.6 at Cat Hai in Hai Phong coast.

Table 5.5: Summary of crest level calculation of the dike in Hai Hau coast – Nam Dinh

Crest level of each option (m)	**Scenario 1** 1/25	**Scenario 2** 1/50	**Scenario 3** 1/100
Option 1: Non-overtopped dike	12.2	13.8	15.4
Option 2: Small overtopped dike	8.5	9.5	10.6
Option 3: Medium large overtopped dike	6.6	7.4	8.2

Table 5.6: Summary of crest level calculation of the dike in Cat Hai coast – Hai Phong

Crest level of each option (m)	**Scenario 1** 1/25	**Scenario 2** 1/50	**Scenario 3** 1/100
Option 1: Non-overtopped dike	12.1	13.6	14.9
Option 2: Small overtopped dike	8.3	9.2	10.1
Option 3: Medium large overtopped dike	6.4	7.1	7.7

5.4.2 Determination of slope protection

Amour layers to protecting outer slope are determined by Pilarczyk formula (Pilarczyk, 1998). The reasons of using this formula are those (i) it is applicable for various types of revetment; (ii) it is introduced in the sea dike design code of Vietnam (14 TCN-130-2002).

Wave attack on revetments will lead to a complex flow over and through the revetment structure (filter and cover layer). During wave run-up the resulting forces by the waves will be directed opposite to the gravity forces. For semi-permeable cover layers the equilibrium of uplift forces and gravity forces (difined by components of a revetment) lead to the following (approximate) design formula:

$$\frac{H_s}{D\,\Delta} = \frac{\varphi}{\xi_{op}^{\;2/3}} \qquad (5.34)$$

Where:

H_s is (local) significant wave height [m];

ξ_{op} is breaker parameter given by:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\dfrac{H_s}{1.56T_p^2}}} \qquad (5.35)$$

T_p is wave period at the peak of the spectrum [s];

Δ is the relative volumetric mass of cover layer:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \qquad (5.36)$$

With ρ_s is density of the protection material (kg/m^3) and ρ_w is density of water (kg/m^3);

α is slope angle [deg];

D is thickness of a top (cover) layer [m];

φ is total stability factor.

The application of formula (5.34) is only for semi-permeable cover layers (i.e., pitched stone and placed blocks, block mats, concrete- or sand-filled geo-mattresses) which use of the following stability factor φ:

φ = 4 to 4.5 for placed blocks,

φ = 8 for interlocked blocks on properly designed sub-layers and subsoil.

Figure 5.3 shows the given value of the formula (5.34) for slope of 1:4.

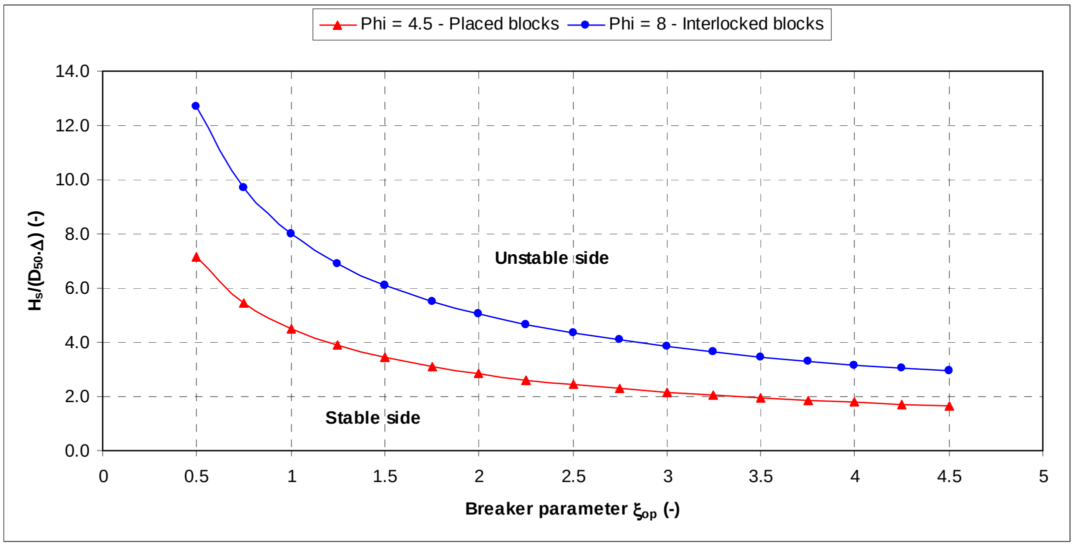


Figure 5.3: Stability of slaps/blocks revetment by applied Pilarczyk's formula.

Applying formula (5.34) the required thickness/size of cover layers for given cases of Namdinh revetments and Hai Phong are in Table 5.7 and Table 5.8, respectively.

Table 5.7: Required thickness of block revetment at Hai Hau coast in Nam Dinh.

P_f	Significant wave height H_s	Peak wave period T_p	Wave length L_{op}	tanα	Breaker parameter x_{op}	D	Placed blocks Δ=1.33, φ=4.5		Interlocked blocks Δ=1.33, φ=8	
							$H_s/(D_{50}.D)$	D	$H_s/(D_{50}.D)$	D
(-)	(m)	(s)	(m)	(-)	(-)	(-)	(-)	(m)	(-)	(m)
1/25	2.6	10.5	171	0.25	2.01	1.33	2.82	0.70	5.02	0.40
1/50	2.9	11.4	204	0.25	2.11	1.33	2.74	0.79	4.86	0.44
1/100	3.1	12.3	237	0.25	2.19	1.33	2.67	0.87	4.74	0.49

Table 5.8: Required thickness of block revetment at Cat Hai coast in Hai Phong.

P_f	Significant wave height H_s	Peak wave period T_p	Wave length L_{op}	tanα	Breaker parameter x_{op}	D	Placed blocks Δ=1.33, φ=4.5		Interlocked blocks Δ=1.33, φ=8	
							$H_s/(D_{50}.D)$	D	$H_s/(D_{50}.D)$	D
(-)	(m)	(s)	(m)	(-)	(-)	(-)	(-)	(m)	(-)	(m)
1/25	2.8	10.5	171	0.25	1.96	1.33	2.87	0.73	5.10	0.41
1/50	3.0	11.4	204	0.25	2.07	1.33	2.77	0.81	4.93	0.45
1/100	3.1	12.3	237	0.25	2.17	1.33	2.68	0.88	4.77	0.49

5.5 Application of risk based cost benefit analysis at Hai Hau coastal area in Nam Dinh

This section is going to determine total cost, C_{tot}, of different sea defense options for Hai Hau coastal zone in Nam Dinh. As mention above three options will be applied to analysis those are:

- Option 1: Non-overtopping (q = 0.1 l/s/m);
- Option 2: Small overtopping (q = 10 l/s/m); and
- Option 3: Medium large overtopping (q = 100 l/s/m).

In Hai Hau the coastal area has a coastline of 25.3 km in length which is directly affected by wave attack from the sea (see Figure 5.4). Therefore this length of coastline will be considered as the length of the sea defense system which has to be designed in each option.

Figure 5.4: Overview of coastal area in Hai Hau district, Nam Dinh province

By applying series equations from (5.1) to (5.24) the total costs of each option are determined for case study of Hai Hau in Nam Dinh.

Based on design documents and expenditure reports of existing Nam Dinh sea dikes given by DDMFC/MARD 2005, taking into account the actual inflammation rates of Vietnam (9 percent in 2007) the unit cost factors C_1, C_2, C_3, C_4 and C_5 are determined as in Table 5.9.

Table 5.9: Unit cost factors for Nam Dinh sea dikes

Para.	Descriptions	Unit	Value
Co	Total Investment costs of existing dike; crest at 5.5m	Mil. $US/km	3.217
C1	Unit cost of constructed dike body	Mil. $US/m^2/km	0.0096
C2	Unit cost of constructed outer slope protection	Mil. $US/m/km	0.0424
C3	Unit cost of constructed inner slope protection	Mil. $US/m/km	0.0024
C4	Unit cost for crest protection	Mil. $US/m/km	0.1204
C5	Unit cost of land use	Mil. $US/m/km	0.0206

Expected economic damage *E(D)* when flooding occurs is determined by equation (5.10), in which D is the damage due to flood. The value of D is estimated based on the damage caused by Damrey Typhoon occurred in September in the Northern Vietnam, which led to a total direct loss of about 120 Million USD in three coastal areas in Nam Dinh (Giao Thuy, Hai Hau and Nghia Hung) [DMWG 2005]. The damage in Hai Hau coastal area is approximately about 40 Million USD.

The actual economic growth rate of 7.5% of Vietnam is taken into account to analysis for the case of Hai Hau (Nam Dinh) coastal flood defense and also for the case of Cat Hai (Hai Phong) coastal flood defense which is analyzed in next section.

Determination of the second dike height of Option 3 (Medium large overtopping – q = 100 l/s/m) is based on overtopped sea water discharge, storm duration, width of storage zone and wind setup in transitional area (storage area). Table 5.10 shows the result of determination of the second dike height for the case study of Hai Hau coastal flood defense.

Table 5.10: Determination of the second dike height for Hai Hau coastal flood defense

Parameters	**Unit**	**Value**
Specific overtopping discharge q	l/s/m	100
Storm duration T_{storm}	hour	6
Length of storage area $L_{storage}$	m	2000
Width of storage area $W_{storage}$	m	1000
Volume of overtopped water from sea $V_{overtop}$	m3	4.32E+06
Storage area $A_{storage}$	m2	2.00E+06
Water depth in storage area $h_{storage}$	m	2.16
Wind velocity corresponds to design frequency U	m/s	30.8
Wind wave in storage area, H_w	m	0.7
Significant wave height in front of the first dike	m	2.6
Transmitted wave height over the first dike, H_t	m	0.3
Incomming wave height at the second dike, $H_{i2} = H_w + H_t$	m	1.0
Design wave height at the second dike, H_{d2}	m	1.0
Overtopped discharge of the second dike, q_2	l/s/m	1.0
Free board of the second dike, $R_{c,2}$	m	1.32
Crest height of the second dike	**m**	**3.5**

The total cost C_{tot} of each option is presented in Table 5.11, 5.12 and 5.13 for sea flood defense in Hai Hau coastal zone (Nam Dinh province).

Table 5.11: Determination of total cost (C_{tot}) of Option 1 (Non-overtopping) for case study of Hai Hau

Parameters	Unit	Value
Probability of flooding P_f	(-)	0.04
Length of dike section	km	25.30
Height of the dike H	m	12.2
Initial investment cost $I_{initial}$	Mil. $US	194.266
Expected maintenance cost $C_{maintenance}$	Mil. $US	2.521
Land use cost $C_{landuse}$	Mil. $US	40.777
Expected value of economic damage E(D)	Mil. $US	20.803
Total costs Ctot	Mil. $US	**258.366**

Table 5.12: Determination of total cost (C_{tot}) of Option 2 (Small overtopping) for case study of Hai Hau

Parameters	Unit	Value
Probability of flooding Pf	(-)	0.04
Length of dike section	km	25.30
Height of the dike H	m	8.5
Initial investment cost $I_{initial}$	Mil. $US	116.645
Expected maintenance cost $C_{maintenance}$	Mil. $US	15.137
Land use cost $C_{landuse}$	Mil. $US	29.134
Expected value of economic damage E(D)	Mil. $US	20.803
Total costs Ctot	Mil. $US	**181.719**

Table 5.13: Determination of total cost (C_{tot}) of Option 3 (Medium large overtopping – two dike lines system) for case study of Hai Hau

First dike:		
Parameters	**Unit**	**Value**
Probability of flooding P_f	(-)	0.04
Length of dike section	km	25.30
Height of the dike H_1	m	6.6
Initial investment cost $I_{initial}$	Mil. $US	85.471
Expected maintenance cost $C_{maintenance}$	Mil. $US	11.091
Land use cost $C_{landuse}$	Mil. $US	23.326
Total costs C_{tot} of first dike	Mil. $US	**119.888**
Transitional area (Storage area):		
Width of landuse to store overtopped water, $W_{landuse\_storage}$	m	1000
Total length of subdivision dike	km	12.65
Height of the subdivision dike, H_{sub_dike}	m	3.5
Initial investment cost $I_{initial}$	Mil. $US	3.1
Expected maintenance cost $C_{maintenance}$	Mil. $US	0.040
Land use cost $C_{landuse}$	Mil. $US	3.2
Construction cost of storage area	Mil. $US	**6.338**
Second dike:		
Length of dike section	km	25.3
Height of the dike H_2	m	3.5
Initial investment cost $I_{initial}$	Mil. $US	23.056
Expected maintenance cost $C_{maintenance}$	Mil. $US	0.299
Land use cost $C_{landuse}$	Mil. $US	13.496
Total costs C_{tot} of second dike	Mil. $US	**47.253**
The total cost of two dike lines system:		
Initial investment cost $I_{initial}$	Mil. $US	108.528
Expected maintenance cost $C_{maintenance}$	Mil. $US	11.391
Land use cost $C_{landuse}$	Mil. $US	43.160
Expected value of economic damage E(D)	Mil. $US	10.401
Total cost of the defense system	Mil. $US	**173.479**

Conclusion: The total cost of Option 3 (with two dike lines system) is the lowest among three considered options. The optimal economic solution for flood defense system in Hai Hau is, therefore, the option of using two dike lines (Option 3). This is similar to the actual use at several places along Hai Hau coastal zone. The application of this two dikes system should be implemented widely for coastal flood defences in Nam Dinh.

5.6 Application of risk based cost benefit analysis at Cat Hai coastal area in Hai Phong

Similar to the case of Nam Dinh, three options will be analyzed for 7 km length of the sea dike which resists directly the wave attack from the sea in southeast and southwest directions (see Figure 5.5). Also the value of damage caused by flood is considered proportionally in the same way as that value in Hai Hau.

Because various material cost rates are fixed to be almost identical for different provinces in Northern Vietnam, thus these unit cost rate factors C_1, C_2, C_3 and C_4 are taken the same as in the case study of Hai Hau coastal zone. However, the rate factor of land use C_5 in Cat Hai island is higher than that in Hai Hau coastal area because land use in Cat Hai island is more expensive due to better local productive economic and touristic activities. In this study the unit cost value of C_5 = 0.0412 Million USD/m/km is applied for the land use cost in Cat Hai island.

Height of the second dike of Option 3 is estimated as in Table 5.14. Due to limited land space in Cat Hai island the Option 3 is applied to analysis with the possible width of transitional zone of about 400 m.

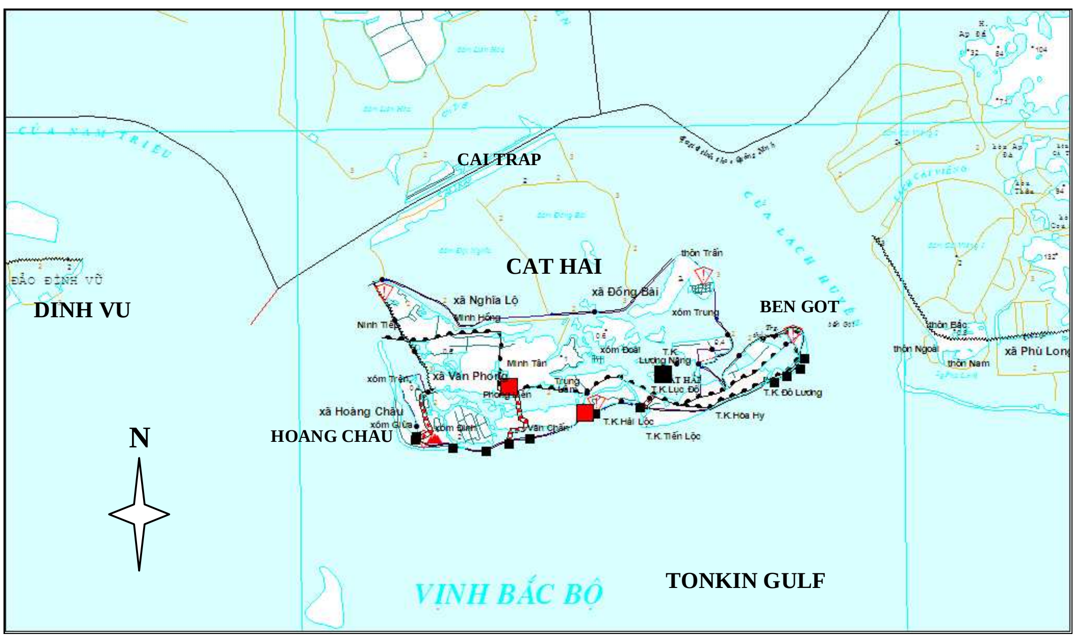


Figure 5.5: Overview of coastal area in Cat Hai island, Hai Phong province

Table 5.14: Determination of the second dike height for Cat Hai coastal flood defense

Parameters	Unit	Value
Specific overtopping discharge q	l/s/m	100
Storm duration T_{storm}	hour	6
Length of storage area $L_{storage}$	m	2000
Width of storage area $W_{storage}$	m	400
Volume of overtopped water from sea $V_{overtop}$	m3	4.32E+06
Storage area $A_{storage}$	m2	8.00E+05
Water depth in storage area $h_{storage}$	m	5.40
Wind velocity corresponds to design frequency U	m/s	30.8
Wind wave in storage area, H_w	m	0.6
Significant wave height in front of the first dike	m	2.8
Transmitted wave height over the first dike, H_t	m	0.3
Incomming wave height at the second dike, $H_{i2} = H_w + H_t$	m	0.9
Design wave height at the second dike, H_{d2}	m	0.9
Overtopped discharge of the second dike, q_2	l/s/m	1.0
Free board of the second dike, $R_{c,2}$	m	1.19
Crest height of the second dike	**m**	**6.6**

The total costs of three options are presented in Table 5.15, 5.16 and 5.17 for the case study of Cat Hai coastal flood defense in Hai Phong province.

Table 5.15: Determination of total cost (C_{tot}) of Option 1 (Non-overtopping) for case study of Cat Hai

Parameters	Unit	Value
Probability of flooding Pf	(-)	0.04
Length of dike section	km	7
Height of the dike H	m	12.1
Initial investment cost $I_{initial}$	Mil. $US	52.907
Expected maintenance cost $C_{maintenance}$	Mil. $US	0.687
Land use cost $C_{landuse}$	Mil. $US	22.340
Expected value of economic damage E(D)	Mil. $US	5.756
Total costs C_{tot}	Mil. $US	**81.689**

Table 5.16: Determination of total cost (C_{tot}) of Option 2 (Small overtopping) for case study of Cat Hai

Parameters	Unit	Value
Probability of flooding Pf	(-)	0.04
Length of dike section	km	7
Height of the dike H	m	8.3
Initial investment cost $I_{initial}$	Mil. $US	31.208
Expected maintenance cost $C_{maintenance}$	Mil. $US	4.050
Land use cost $C_{landuse}$	Mil. $US	15.751
Expected value of economic damage E(D)	Mil. $US	5.756
Total costs C_{tot}	Mil. $US	**56.765**

Table 5.17: Determination of total cost (C_{tot}) of Option 3 (Medium large overtopping – two dike lines system) for Cat Hai coastal flood defense

First dike:		
Parameters	**Unit**	**Value**
Probability of flooding P_f	(-)	0.04
Length of dike section	km	7
Height of the dike H_1	m	6.4
Initial investment cost $I_{initial}$	Mil. $US	22.555
Expected maintenance cost $C_{maintenance}$	Mil. $US	2.927
Land use cost $C_{landuse}$	Mil. $US	12.459
Total costs C_{tot} of first dike	Mil. $US	**37.941**
Transitional area (Storage area):		
Width of landuse to store overtopped water, $W_{landuse\_storage}$	m	400
Total length of subdivision dike	km	1.4
Height of the subdivision dike, H_{sub_dike}	m	6.6
Initial investment cost $I_{initial}$	Mil. $US	1.1
Expected maintenance cost $C_{maintenance}$	Mil. $US	0.014
Land use cost $C_{landuse}$	Mil. $US	1.3
Construction cost of storage area	Mil. $US	**2.324**
Second dike:		
Length of dike section	km	7
Height of the dike H_2	m	6.6
Initial investment cost $I_{initial}$	Mil. $US	14.335
Expected maintenance cost $C_{maintenance}$	Mil. $US	0.186
Land use cost $C_{landuse}$	Mil. $US	12.853
Total costs C_{tot} of second dike	Mil. $US	**33.130**
The total cost of two dike lines system:		
Initial investment cost $I_{initial}$	Mil. $US	36.890
Expected maintenance cost $C_{maintenance}$	Mil. $US	3.113
Land use cost $C_{landuse}$	Mil. $US	27.636
Expected value of economic damage E(D)	Mil. $US	5.756
Total cost of the defense system	Mil. $US	**73.395**

Conclusion: The Option 2 with small overtopping discharge allowed has the lowest total cost of the sea defense system. Therefore, using one dike line defence system is the best choice for Cat Hai island.

5.7 Discussions

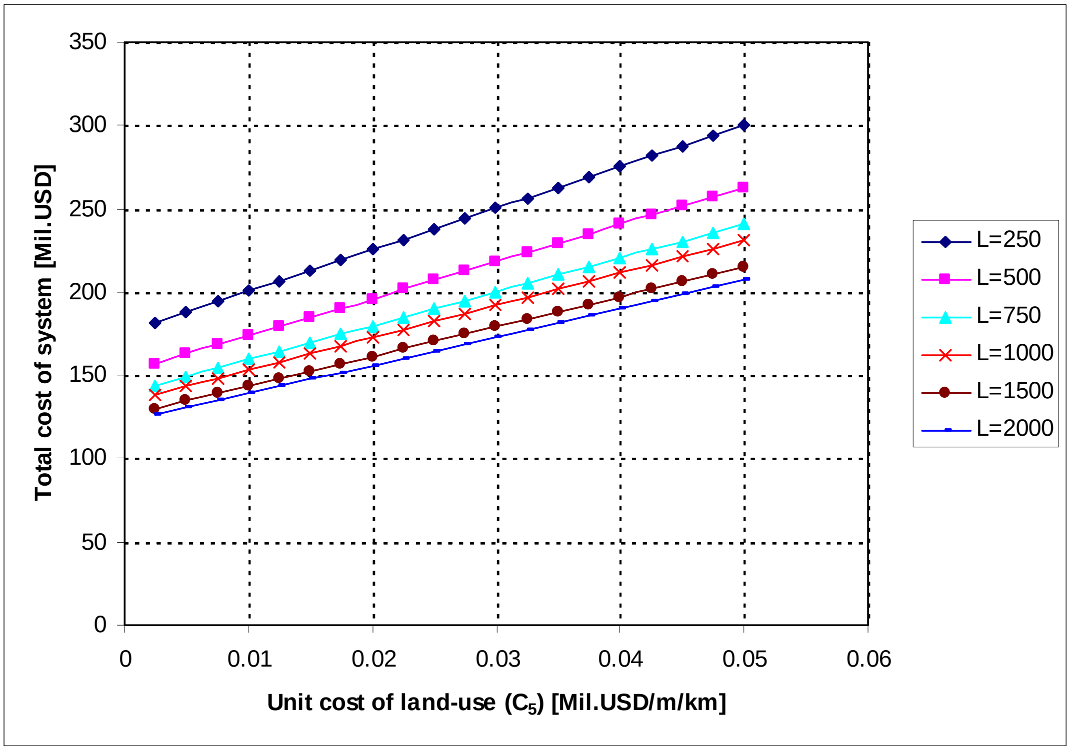
In Chapter 4 five different alternatives of sea defense system are compared by using a multi criteria evaluation (MCE) method, which leads to a preliminary indication for the best solution for coastal defence system. As the analysis shows in Chapter 4, three alternatives, i.e. Option 1, Option 2 and Option 3, are of more interest than the other. Therefore in this chapter these three alternatives are used to find the best sea defence system option in Nam Dinh and Hai Phong, by using a risk based cost benefit analysis.

In the case study of Nam Dinh coast, the analysis shows that Option 3, using two lines defence system, is the best solution. However, for the case of coastal flood defence in Hai Phong, Option 2, using one line defence system and small overtopping discharge allowed, appears to be the best choice.

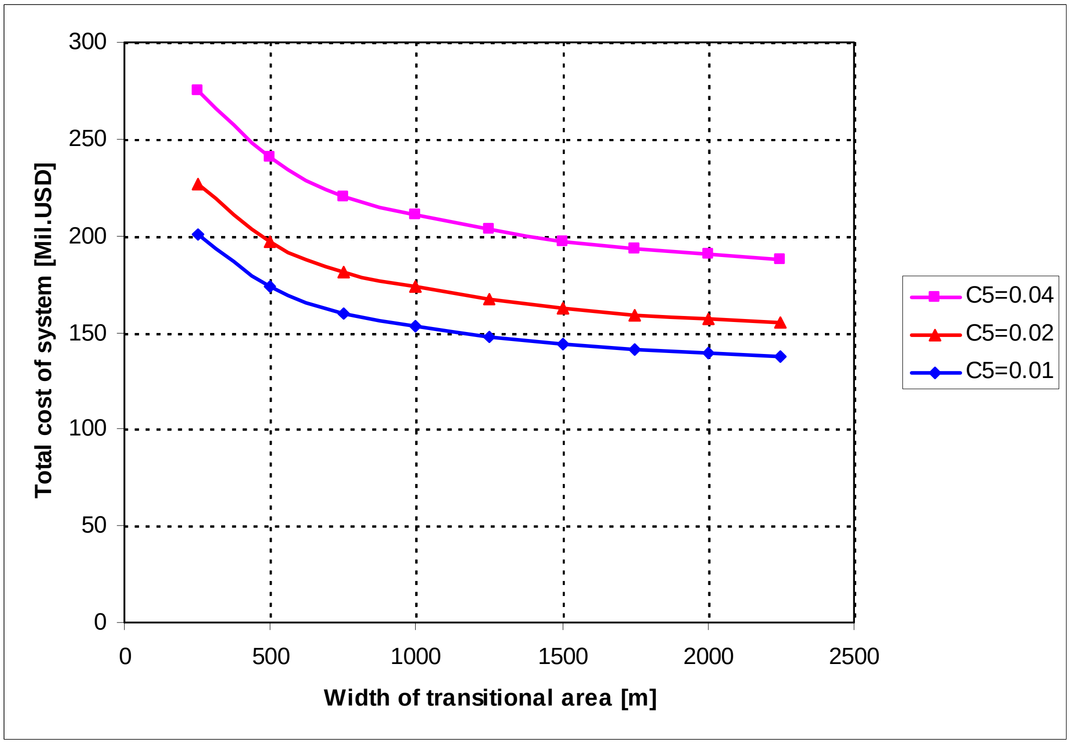
The total cost of Option 3 depends on the width of the transitional area between two dike lines, and the unit cost of land use in the region. Figures 5.6 and 5.7 show the sensitivity of total cost of Option 3 in the case study of Hai Hau and Cat Hai coastal region, respectively. As showing in Figure 5.6 and 5.7 the total cost of defence system is proportional to the unit cost of land use (C_5) and disproportional to the width of transitional area (L). In coastal areas which have widely land space (in the direction of perpendicular to the coastline) and low land values (at present), the two defence lines system could be more applicable, such as in Hai Hau coastal area. On the contrary, such as Cat Hai island which has narrow land space and high land value, it is costly to use a two defence lines system.

In Figure 5.6b – in the case study of Hai Hau coastal area is applied Option 3 (using two defence lines system), shows that if the width of transitional area increase in range of 250 m to 1000 m the total cost of defence system decreases rapidly. And if the width of transitional area increases continuously up to 2500 the cost of defence system decreases slowly, at the end of the curves it look horizontal, that means if the width of transitional area continuity increase the total cost is reduced not considerable.

In the case study of Cat Hai island (see Figure 5.7b) which has different situation (limited land space, high value of land) compare to Hai Hau coastal area, shows that if the width of transitional area increases from 200 m to 400 m (such as the available land space in island to construct defence system), the total cost of defence system decreases much gentler than that in the range of 400 m to 800 m of the transitional area width.

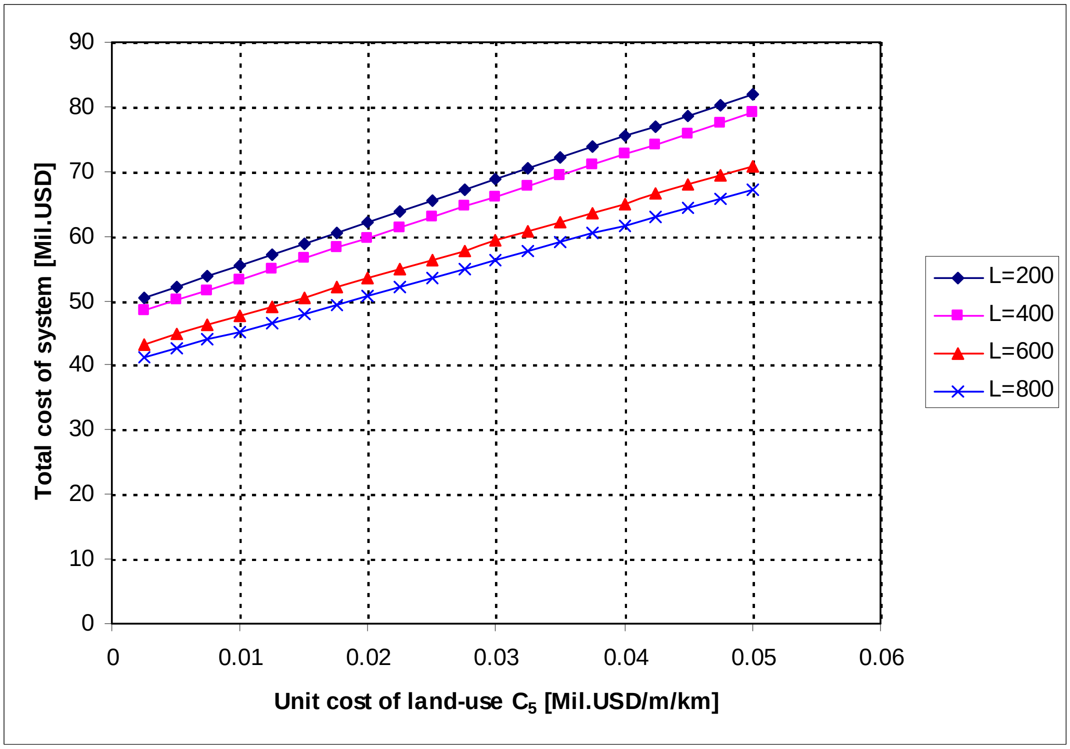


(a)

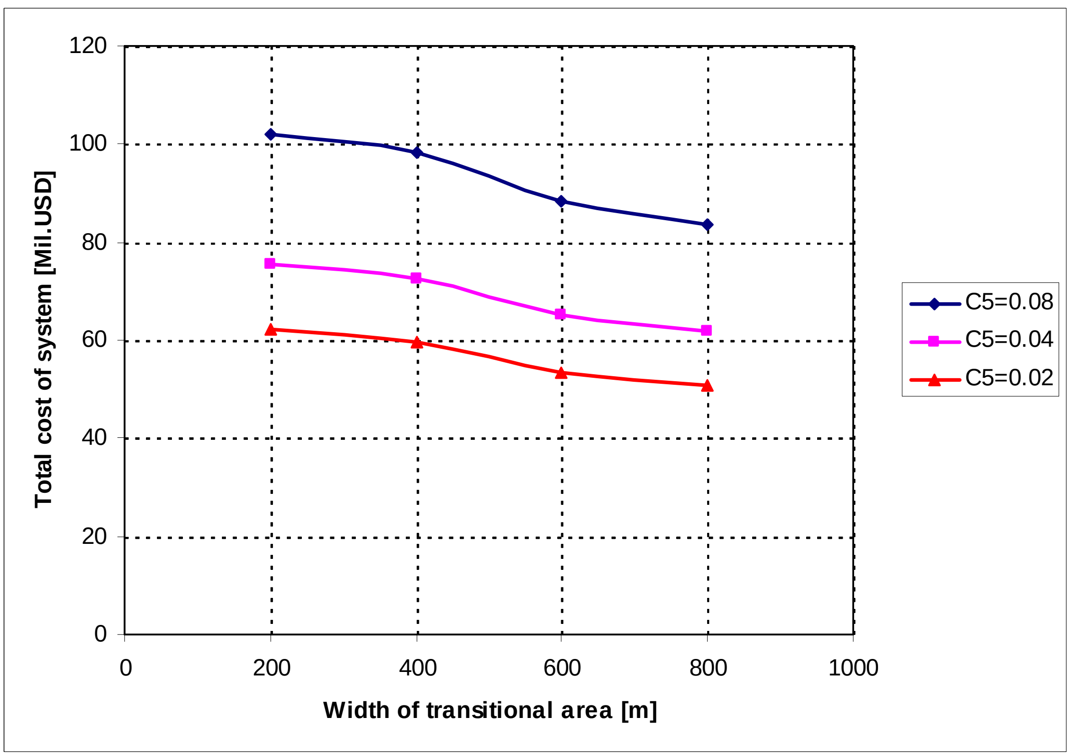


(b)

Figure 5.6: Sensitivity of total cost of Option 3 (using two defence lines) in the case study of Hai Hau, Nam Dinh



(a)



(b)

Figure 5.7: Sensitivity of total cost of Option 3 (using two defence lines) in the case study of Cat Hai, Hai Phong

Because Cat Hai is a small island, the use of two defensive lines is not effective in terms of economic consideration, as the analysis shows. During storms and if a flood would occur, it is hard to evacuate people from the whole island to mainland. By separating the island into various independent small polders people living in the flooded polders can move to other non-flooded areas of the island. For this reason, the best solution for coastal denfences in Cat Hai is using one defensive line, to reduce damage and loss cause by flooding; the island should be separated in many small zones by improving/upgrading its current main traffic road system in the island. This will lead to more effective in evacuation and, thus, to minimize flood consequences when flood would occur.

The result of this study, using a two defence line system, agrees well with the actual defence system at some places in Nam Dinh, which has been tested during more than 10 years. In Vietnam, the discussion on whether one or two defence lines should be used for protecting the coastal zone is on-going. From this study it appears conceptually that selection of defence options is site-specific and depends very much on the local conditions such as economic development plan, cost of land as well as availability of investment. This study could give support in a part of the decision making process.

CHAPTER VI RISK BASED APPROACH FOR DETERMINDATION OF SAFETY STANDARD

6.1 Introduction

6.1.1 General approach

Chapter 4 and 5 already discussed how to select a defence option given a certain safety standard (certain design frequency scenarios). In practice, very often the safety standards need to be assessed to answer question of whether *safe is safe enough* and how we can come up with the optimal safety standards for a certain regions / country regarding to flood defence.

From literatures (e.g. van Dantzig 1956, Vrijling et al. 1980, Voortman 2002, Jonkman 2007 and Mai Van et al. 2006) determination of the safety standards should be based on acceptable risk of the interested regions. Generally there are three points of view on estimation of acceptable risk, with regard to the estimation of the consequences of flooding.

The first point of view is the assessment by the individual. This is translated as the probability of losing one's life due to participating in daily activities (Vrijling *et al.*, 1998). Second point of view concerns the risk assessment by society on a national level related to the number of casualties due to a certain activity. Following the third point of view, acceptable level of risk can also be formulated by economical cost benefit analysis. The total costs in a system are determined by the sum of the expenditure for a safer system and the expected value of the economic damage. The acceptable risk measure can be estimated by comparing the cost of protection to a characteristic value of the consequences of flooding (DMWG, 2005). The optimal level of economically acceptable risk, incorporates with an optimal level of safety, corresponds to the point of minimal total costs.

Since the individuals risk and societal risk is difficult to assess in Vietnam due to lack of input data information and also time limitation of this study, in this chapter, the third point of view is applied. Method of risk based economic optimization is applied and further developed to find the optimal safety level of the coastal flood defences in Vietnam. An acceptable risk, which is determined by other methods, can be found in the above mentioned literatures.

6.1.2 Flood risk and safety standards of flood defence in Vietnam (Mai Van et al. 2008)

Since 1953, Viet Nam was affected by a number of flood disasters, each disaster being responsible for the loss of hundreds of lives and a considerable damage to infrastructure, crops, rice paddy, fishing boats and trawlers, houses, schools, hospitals, etc. The total material damage of the flood disasters over the past 60 years exceeded $US 7.5 billion. Additionally, floods and storms caused the loss of more than 20,000 lives (ADRC, 2006 & DDMFC, 2007). The most severe storm- and flood- induced disasters occurred in North Viet Nam in 1971, 1996 and 2005; in the South in 1997; and in the Central in 1964 and 1998.

Based on data given from (i) the Department of Dike Management and Flood Control (DDMFC, 2007) of Vietnam, yearly economic loss data due to floods and storms, which is collected from 1970 up to 2007, and (ii) online reports of the Asian Disaster Reduction Centre (ADRC, 2006) about the top 25 flood disasters of Vietnam in the 20th century (historical events), Mai van et al. (2008) constructed the exceedance curve of damage (FD-curve). The damage curves are presented in Figure 6.1. A lognormal curve with E(D)= 181.3 and σ=309.5 (*10^6 US$) is found as the best fit to the economic damage dataset.

Based on the FD-curve the total potential damage due to floods could be equal to $E+k*\sigma$=181+3*309= $US 1108*$10^6$ per year for the whole Vietnam. This is comparable with the reported actual flood situation during the last 10 years (total flood damages are estimated as from 1 to 1.5 % of Vietnam GDP during the last 10 years, sources: http://www.vnexpress.net/Vietnam/Xa-hoi/2007/10/3B9FB183/). These values will be used as important input for later calculation in this chapter.

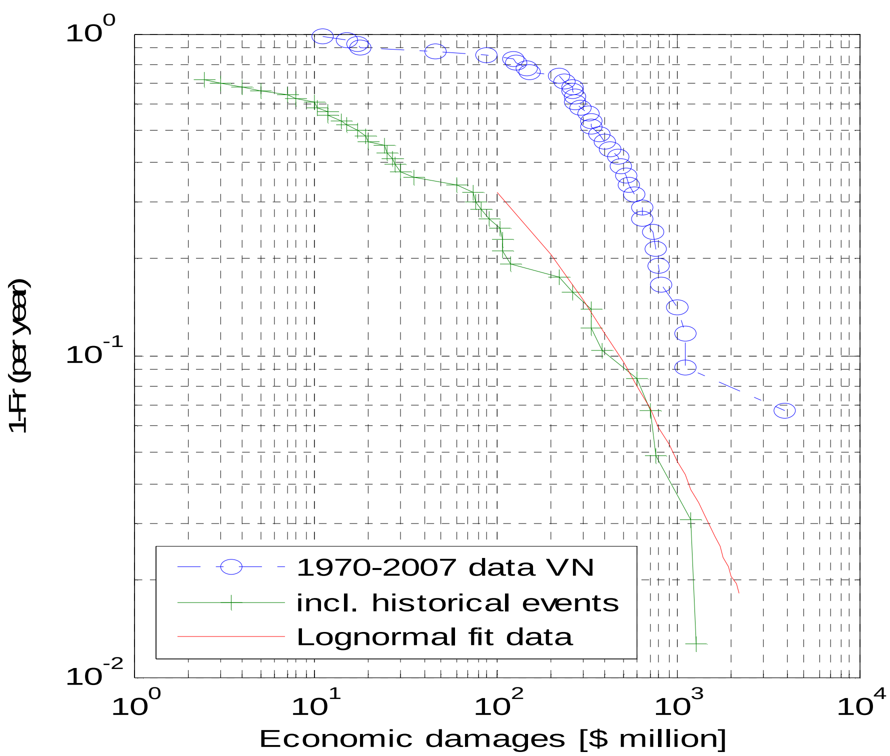


Figure 6.1: Flood FD-curve of Vietnam (Mai van et al. 2008)

The relatively low safety level of the sea dikes in Vietnam was noticed in 1996 during two visits of Dutch expert missions (DWW/RWS, 1996a, b). Most designs of the sea dikes in Vietnam are based on loads with return period 20 to 25 year periods and were disputable. The true probability of failure of the Vietnamese water defense system exceeds by far the design frequency (Mai *et al*., 2006, 2007). Although designed to fail once in 20 to 25 years the sea defense system might fail almost every year. The experiences in the past 20 years support this statement.

The design return periods are not based on proper statistic risk analysis; often the return periods are adopted on a rather arbitrary basis that the safety level of important, valuable areas should be enlarged compared to the safety level of less important areas (Vrijling *et al.*, 2000). This system reflects, however, logical results, which could have been obtained by common risk analysis. Future improvements of flood safety standards might build on the existence of this system by a proper risk analysis.

The improvement of this situation calls for the use of present available knowledge on all levels. The theoretical knowledge in the fields of dike design, reliability and safety approach, risk analysis, policy analysis, statistics in relation to boundary conditions and mathematical modeling is not up to date. Therefore the transfer of this knowledge was strongly recommended (DWW/RWS, 1996b; Vrijling *et al.*, 2000; Mai *et al.*, 2006). An additional important fact is the economic situation of Vietnam, just at the beginning of a developing process, limiting the resources for improvement of the water defence system. On the other hand this situation asks for a more detailed and careful analysis to ensure that the limited resources are used in the optimal way which takes into account the developing characteristics (limited initial investment, fast economic growth, and cheap labor).

As already mentioned in the chapter 1, recently, Damrey Typhoon occurred in September 2005 in Northern Vietnam which led to a total direct loss of over 500 Million USD (DMWG 2005). A huge sea dikes program has been established by the Ministry of Agricultural and Rural Development (MARD) to rehabilitate the sea dike system. This program is implemented for 2005-2015 period and appointed with two important tasks:

(i) Researches on safety standards, boundary conditions and finding optimal solutions for sea defences along the whole country;

(ii) Design and construct new dikes, at places where sea dikes have not existed or were breached, and reinforcement of the existing dikes on the basic of findings in the first task.

Coastlines along Hai Hau district were selected as a pilot location. Construction works took place in 2005 and had finished in 2007. However, design the new dikes is still based on existing safety standards (design frequency of 1/20 year), which is known as out of date. It is necessary to check the safety of the new constructed dike system at the pilot locations to see if the current rehabilitation works provide enough safety given the present situation and if safe is safe enough for current Vietnam development. Findings are important input contributing to the first task of the sea dike program of Vietnam, which aims at providing design guidelines for the sea flood defences. These provide motivation for this chapter in this study.

6.2 Methods of risk based economic optimization of safety level

6.2.1 FD-Curve

The exceedance frequency of damage curve, FD-curve, displays the probability of exceedance as a function of the economic damage. The FD-curve can be constructed by available damage data due to flood. The FD-curve can also be derived mathematically for the probability density function (PDF):

$$1 - F_D(x) = P(D > x) = \int_x^{\infty} f_D(x) \cdot dx \tag{6.1}$$

The main interest in economic optimization of the dike is how to estimate the expected value of economic damage/loss, *E(D)*. This can be determined by the PDF and the FD-curve:

$$E(D) = \int_0^{\infty} x \cdot F_D(x) \cdot dx \tag{6.2}$$

And
$$E(D) = \sum_{i=1}^{N} x_i \cdot p_i(x_i) = \frac{1}{N}\sum_{i=1}^{N} x_i \cdot i \tag{6.3}$$

Where: $F_D(x)$ is the probability distribution function of the economic damage; *E(D):* expected value of the economic damage; x_i (i = 1..N) is vector of random variable, in this case x_i is economic damage per event of flood; $p_i(x_i)$ = i/N; i is the order of x_i and N is the total of events.

6.2.2 Economic optimization of acceptable risk level

In the method of economic optimization the total costs of a system (C_{tot}) are determined by summing up the investments ($I_{\Delta H}$) for a safer system; the expected value of the maintenance cost M and the expected economic damage D (see also van Dantzig, 1956 for a fundamental approach and applications in the Netherlands). The total cost of the system with dike heightening ΔH is:

$$C_{tot}(H_0, \Delta H_{P_f}, P_f) = \left[I_{0,P_{f0}} + I_{\Delta H_{P_f}}(\Delta H_{P_f}) + PV(M) + PV(P_f * D) \right] \tag{6.4}$$

The optimal level of safety indicated by $P_{f\text{-}opt}$ corresponds to the point of minimal cost:

$$\min(C_{tot}) = \min\left[I_{0,P_{f0}} + I_{\Delta H_{P_f}}(\Delta H_{P_f}) + PV(M) + PV(P_f * D) \right] \tag{6.5}$$

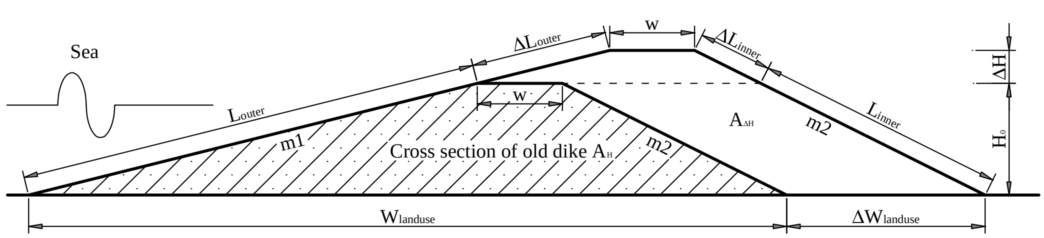


Figure 6.2: Specific parameters of cross section of a heighten dike

Cost of dike heightening in this study is accounted for: Cost of enlarging dike body (heightening the dike crest level which leads to increasing the cross section area ($A_{\Delta H}$); additional cost of outer and inner slope protection due to increase of the protected length of the outer and inner slopes (ΔL_{outer}), (L_{inner} + ΔL_{inner}); additional cost of crest protection; and additional cost of land area use for dikes ($\Delta W_{landuse}$) (see Figure 6.2).

Therefore, the increment cost $I_{\Delta H}$ is determined by:

$$I_{\Delta H} = C_1.A_{\Delta H} + C_2.\Delta L_{outer} + C_3.(L_{inner} + \Delta L_{inner}) + C_4.w + C_5.\Delta W_{landuse} \tag{6.6}$$

Where: C_i, with i=1:5, is the unit cost of different cost components, expressed in $US Million per geometrical unit of dike elements.

Base on the cross section of the dike in Figure 6.2 the increasing of the cross section area, the length of the outer slope, the length of inner slope and the land use consuming are determined.

$$A_{\Delta H} = \left[(H_0 + \frac{1}{2}\Delta H).(m_1 + m_2) + w \right].\Delta H \tag{6.7}$$

$$\Delta L_{outer} = \Delta H.\sqrt{1 + m_1^2} \tag{6.8}$$

$$L_{inner} + \Delta L_{inner} = (H_0 + \Delta H).\sqrt{1 + m_2^2} \tag{6.9}$$

$$W_{landuse} = \Delta H.(m_1 + m_2) \tag{6.10}$$

Substituted the equations (6.7), (6.8), (6.9) and (6.10) to the equation (6.6):

$$I_{\Delta H} = \left\{ C_1.\left[(H_0 + \frac{1}{2}\Delta H).(m_1 + m_2) + w \right] + C_2.\sqrt{1+m_1^2} + C_3.\sqrt{1+m_2^2} + (m_1 + m_2) \right\}.\Delta H + C_3.\sqrt{1+m_2^2}.H_0 \tag{6.11}$$

The present value of the expected maintenance and damage costs are estimated by:

$$PV(M) = E(M) * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{6.12}$$

and

$$PV(P_f * D) = P_f * E(D) * \sum_{i=0}^{i=T} \frac{1}{(1+r')^i} \tag{6.13}$$

Where: P_f is probability of failure per year; *E(M)* is yearly expected maintenance cost; *E(D)* is expected damage in case of flood; *r'* is the reduced value rate, *T* is planning period, in years.

6.2.3 Application to the case of Nam Dinh sea dikes

a- Case application descriptions and limitation of application

In this application the situation of Nam Dinh sea defences is selected. To make things simple, only one defence Option 1 is considered for analysis, as a demonstration of the method.

Nam Dinh coastal zone is protected by 90 km of sea dikes. The dikes system has been constructed based on loads with return period 20 year. However, the true probability of failure of the Nam Dinh defense system is 0.78-0.95 per year (Mai *et al*, 2006, 2007). This exceeds by far the design frequency and reflects that failure of the dike system occurs almost every year.

In response the central and local authorities have undertaken some efforts in order to restrain the possible adverse consequences and as future defensive measures, some sections of new sea dikes have been built. However, such efforts still remain limited to reactive and temporary measures due to budget constrains, lack of information on the sea boundary conditions and suitable design methods as well as strategic and long-term solutions. As a consequence, the system could be destroyed once in every 10 years. Therefore the cost of dike maintenance is finally very expensive. Statistically, for maintenance of Namdinh sea dikes system it is represented nearly 95 percent of the total coastal defence budget of Vietnam (DDMFC, 2006). After Damrey Typhoon 2005, the question of which safety standard is optimal choice for sea flood defences in Nam Dinh is of interests to the Vietnam Government.

b- Determination of Optimal protection levels of Nam Dinh sea dike system

Based on design documents and expense reports of existing Nam Dinh sea dikes given by DDMFC/MARD 2005, taking into account the actual inflation rates of Vietnam (9 percent in 2007) the unit cost factors are determined as in Table 6.1. Costs of dike heightening are presented in Figure 6.3.

Table 6.1 Unit cost factors for Nam Dinh sea dikes.

Para.	Descriptions	Unit	Value
Co	Total Investment costs of existing dike; crest at 5.5m	Mil. $US/km	3.217
C1	Unit cost of constructed dike body	Mil. $US/m^2/km	0.0096
C2	Unit cost of constructed outer slope protection	Mil. $US/m/km	0.0424
C3	Unit cost of constructed inner slope protection	Mil. $US/m/km	0.0024
C4	Unit cost for crest protection	Mil. $US/m/km	0.1204
C5	Unit cost of land use	Mil. $US/m/km	0.0206

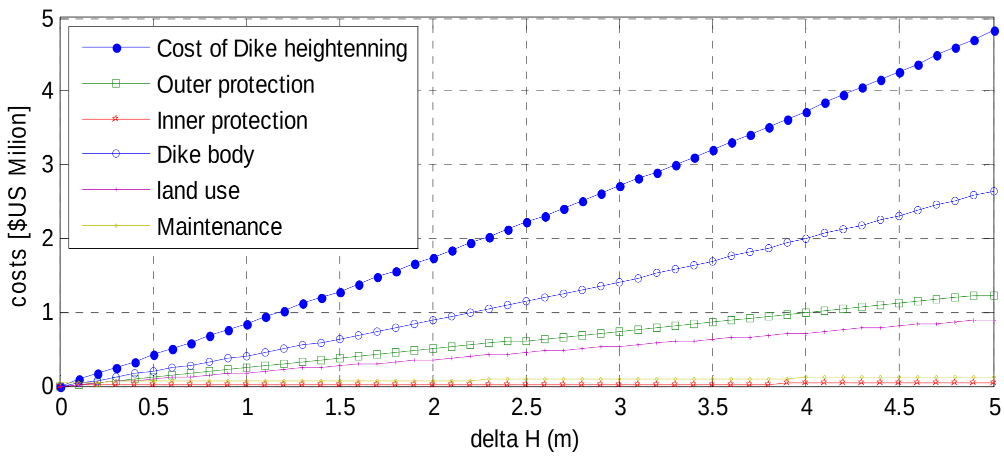


Figure 6.3: Expenditure costs as function of dike heightening

By application of reliability analysis for the case of Nam Dinh coastal flood defences the relationship between dike heights and design frequencies is established by Mai van et al. (2008). That study based on overtopping conditions, required dike heights are calculated with different design conditions which associate with different design frequencies. A linear relation is found between the required dike height and the design frequency in logarithm scale (Figure 6.4). This line can be considered as a limit with safe side (lower left side of the line) and unsafe side (upper right side of the line). Inspection of the actual Nam Dinh sea dikes shows that the existing system is in the unsafe side.

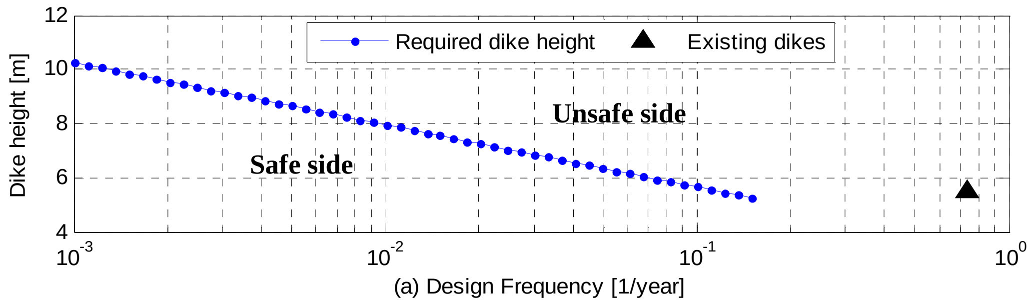


Figure 6.4: Required dike height in relating to design safety level

Economic risk analysis for the case of Nam Dinh coastal flood defences, taking into account the actual economic growth rate (7.5%) of Vietnam and expected damage from the FD-curve (Figure 6-1), $E(D)=\mu+\boldsymbol{k}*\sigma$, gives results as in Figure 6.5.

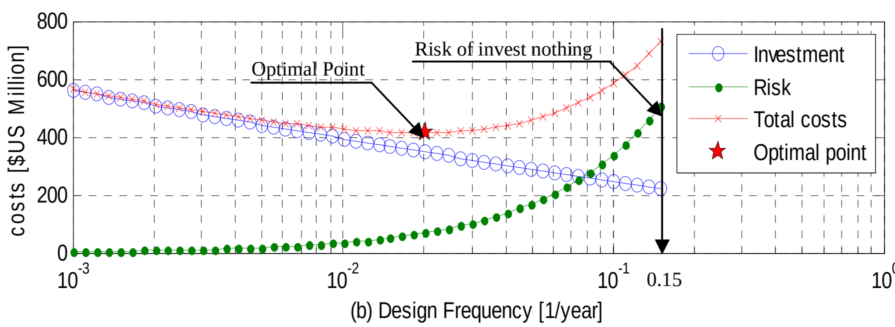


Figure 6.5: Economic risk based optimal safety levels

It is clear that the optimal level of safety is around 1/50 years. The design of the sea dike system should be based on a return period of 50 years or more. A supplementary of design for a return period of 100 years might turn out to be an even better choice since double safety level is archived with relatively small increment of investment cost. Selection of the design return periods of less than 30 years leads to very high risk as well as high expenses for maintenance and repair and is therefore a bad choice in this situation. Selection of 100 years return period is recommended for the future planning of coastal protection in Nam Dinh. Obviously, invest nothing leads to the fact that dike height remains at 5.50 m with annual failure probability of 0.15 for the dike system in Hai Hau (Mai van et al. 2006). This may lead to an economic risk of over $US million 500, which is in the same order of magnitude as the total direct loss of the Damrey 2005 (DMWG, 2005).

6.3 Simplify approach and practical tools for engineers and supporting tools for decision makers

From the previous section we found that the investment cost as a function of safety level [P_f], as I =f(-ln(P_f)). This is, in general, non-linear function and for practical reason this is rather complicated to determine. On the same basic of the method of economic optimization, in order to become familiar with the risk based economic approach to the design of flood defences, a common engineer may prefer to start with a simplified approach.

For these above more practical application and in order to form engineering tools to support a decision making processes, the following simplification is adopted:

- The total investments in dike heightening (I_{tot}) can be simply determined by a linear function of the initial costs (I_0) and the variable costs (I'). The dike is heightened X meters (or transformed to –ln(Pf)) , the difference between the new dike height (h) and the current dike height (h_0).
- All failure mechanisms other than overtopping will be neglected. In most cases overtopping is the major cause of failure of dikes. Therefore dike height is an important parameter to this extent, which is related to design condition of waves and water levels. Other dimensions of the dike are required to be designed in the way that fulfils the given design condition.

Therefore:

$$I_{tot} = I_0 + I' \cdot X \qquad AND \qquad X = h - h_0 \tag{6.14}$$

The expected value of the economic damage can be calculated from the probability of flooding (P_b), the damage caused by the flood (D), and the discount rate (r'). The flood level h is modeled as exponentially distributed with parameters A and B.

$$P_f = e^{-\frac{h-A}{B}} \quad and \quad E(D) = \frac{P_f \cdot D}{r'} \tag{6.15}$$

Now the total costs are formulated as the sum of investments and the expected value of the economic damage. The economic optimum is found by minimizing the total costs. By taking the derivative of the total costs and the dike height, the optimal flooding probability ($P_{b,opt}$) and the optimal dike height (h_{opt}) are found.

$$P_{f,opt} = \frac{I' \cdot B \cdot r'}{D} \qquad and \qquad h_{opt} = A - B \cdot \ln(P_{f,opt}) \tag{6.16}$$

In the equation (6.16) the flooding level (h) and parameters A and B are determined in this application for dike heightening:

h = H (the new dike height)

A = H_0 = 5.5 (the current dike height)

B = 1, scale parameter of the standard exponential distribution (NIST/SEMATECH, 2003), in meter.

As from the damage curve (see FD curve as in Figure 6.1) which is constructed based on available data in Vietnam, the average annual expected damage caused by typhoon is D = 181.3 Mil. USD. The subsequent cost of dike heightening is I' = 50.7 Mil. USD per one meter high. Applying equation 6.15 the optimal probability of flooding therefore is $P_{f,opt}$ = 0.021 per year(corresponding to return period of 47.6 years). This agrees very well with the optimal safety level which is determined from previous section (1/50 year). The corresponding optimal dike height can be found, based on the relation of dike height and design frequency (Figure 6.2), at h_{opt} = 9.4 m.

Sensitivity analysis of optimal flooding probability due to inflation rate:

The inflation rate has affected the reduced interest rate r' which is determined by the real interest rate (r), the economic growth rate (g) and the inflation rate (i).

In a planning period T_p of a sea defence system investment, assume a constant failure probability $P_{f,0}$.

Potential economic damage will increase over time due to economic growth (g) and inflation rate (i). Economic damage at time t, D(t) can now be written as a function of the economic damage at time t = 0, D_0.

$$D(t) = D_0.(1+g+i)^t \approx D_0.e^{(g+i).t} \tag{6.17}$$

The expected economic value can be found over planning time T_p, by discounting to the present value with the real interest rate (r) [Jonkman et al, 2005] as equation (6.18) below.

$$E(D) = P_{f,0}.\int_0^{T_p} D(t).\frac{1}{(1+r)^t}.dt = P_{f,0}.D_0\int_0^{T_p} e^{(g+i).t}.e^{-r.t}.dt = \frac{P_{f,0}.D_0}{\delta}.(1-e^{-\delta.T_p}) \tag{6.18}$$

In which: δ = r – g – i

The total costs can be formulated in equation (6.19) when a linear relation is assumed between investments and the (negative logarithm of the) initial flooding probability. In equation (6.19), expected economic damage is a time dependent variable.

$$C_{tot} = I_0 + I'.(-\ln(P_{f,0})) + \frac{P_{f,0}.D_0}{\delta}(1-e^{-\delta.T_p}) \tag{6.19}$$

By a similar derivation as given in equation (6.16) the level of protection that should e chosen at t = 0 can be derived as a function of planning period T_p (in years, see equation below).

$$P_{f,opt,0}(T_p) = \frac{I'}{D_0}.\frac{\delta}{1-e^{-\delta.T_p}} \tag{6.20}$$

$$h_{opt,0}(T_p) = A - B.\ln(P_{f,opt,0}(T_p)) \tag{6.21}$$

In the situation of long period ($T_p = \infty$) and no inflation (i = 0), the optimal $P_{f,opt,0}$ in equation (6.20) is equal to the optimal derived of equation (6.16), $P_{f,opt}$ = 0.021. Equation (6.20) shows that a lower optimal flooding probability that means of a higher protection level will be found when inflation rate is taken in to account. Figure 6.6

illustrates the optimal flooding probability as a function of planning period with different inflation rate.

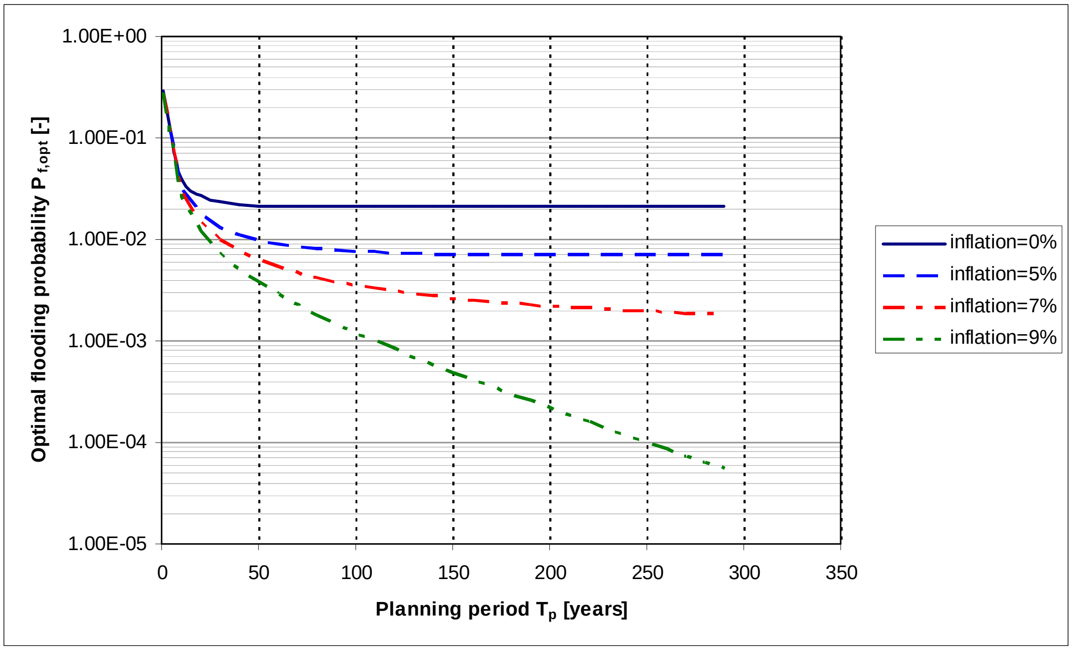


Figure 6.6: The influence of inflation rate to optimal flooding probability in planning period

As showing in Figure 6.6, in the situation no inflation rate, i = 0, the optimal flooding probability will be constant after 50 years of planning period. But in the case of the inflation rate i = 9%, the optimal flooding probability is not constant in long period that means the optimal flooding probability will be found in difference values every year, this optimal value will be constant in long period (more than 350 years).

6.4 Discussions

By using the method of risk based economic optimization, the optimal probability of flooding in Nam Dinh coast is carried out with a value of 0.021 corresponding to a return period of 50 years. Obviously the actual safety standard of coastal flood defence in Vietnam is set at P_f = 0.04 to 0.05 (return period of 25 to 20 years) is not safe enough in terms of economic risk and in the view of the current Vietnamese development with fast economic growth. The analysis results thus in an increase of the safety level of coastal flood protection in Nam Dinh is necessary.

The situation of the Nam Dinh sea defences is typical and representative for sea defences in Vietnam. Therefore, an updating of safety standards of coastal flood defences for the whole country is highly needed. The presented risk based models are thought to be powerful tools to support the decision process to set (or re-set) the safety levels of protection in relation to investments and acceptable consequences for various scales of protection in Vietnam.

Refer to the equation (6.16) the sensitivity of every parameter to the outcome is mathematically similar. However, very often the variable r, the economic growth rate, is fluctuated in developing countries. As consequence the damage value D may change as well (the faster economic growth, the higher economic loss due to flood may be expected). In Vietnam the rate r is currently at 7.5 % per year and be quite stable during last few years. But if we compare to 15 years ago or more, the rate was very much different. Therefore, updating safety standard during a certain period of time is also necessary to ensure safety for the protected region in the view of economic risk approach.

CHAPTER VII CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions:

Main objective of this study is to provide generic guidelines for the selection of coastal flood defense systems with different overtopping criteria, ranging from non-overtopping to large overtopping allowable discharges:

- Option 1: Non-overtopping - q = 0.1 l/s/m;
- Option 2: Small overtopping - q = 10 l/s/m;
- Option 3: Medium large overtopping - q = 100 l/s/m.

These criteria are associated with different defence options of a sea dike system i.e. non-overtopped high dikes to a low crested two dikes system. Subsequently, for a certain given protected region with a specific choice of the defence option the optimal safety level/ optimal safety standard to protect the region is of interest. With efforts in the framework of the M.Sc study program at UNESCO-IHE, this study fulfils its objectives and allows to draw the following concluding remarks.

Multi Criteria Evaluation (MCE), which is developed and applied in this study, is a useful tool for a preliminary decision making stage when comparing different coastal flood defence options. The MCE tool takes into account various criteria in the comparison process such as cost of investment and maintenance, safety of hinter lands, potential risk of protected region, environmental issue, land use problem as well as technical feasibility and re-using the material from old defence systems. These criteria reflect well considerations of both technical and social economic aspects regarding to the local condition of a certain protected region. By using the MCE tool, not only better defence options are detected but also the best defence option can be found in view of social, economic and technical considerations.

As soon as some better defense options are narrowed down by application of the MCE tool, a so-called “Risk Based Cost Benefit Analysis” approach (RBCBA) is used to validate and confirm in the comparative process to find out a more consistent answer. In this study three better options among five indicated options are selected by applying the MCE tool. The best option is defined consistently based on the risk based cost benefit analysis of given three options.

Applications of the MCE framework and risk based cost benefit analysis to the Vietnamese case studies, give us interesting results. For the case of coastal flood defence in Hai Hau, Nam Dinh, the Option 3 – medium large overtopping which requires a two dikes system has the highest score in MCE and lowest total cost in RBCBA. The second highest in total score is the Option 2 – small overtopping with using one dike system. The different of total score of these both options is considerable (20 scores). Therefore the use of a two dikes system, allowing medium large overtopping, is suggested in Nam Dinh coastal area.

Regarding to the case of coastal flood defence in Cat Hai island, Hai Phong, it appears that the Option 2 – small overtopping, which requires one dike system lower than

Option 1, has the highest score in MCE and lowest cost in RBCBA. Based on the results it is recommended to use a one dike system with small overtopping to protect the island.

The main factor that leads to difference in the choice of coastal flood defence option in these two different study areas was mainly due to differences in land use value and space available for the construction of the dike system. In Nam Dinh case, the coastal zone is mainly used for agriculture and salt production which makes the land value lower. Cat Hai has very limited land space and it is an interesting touristic location as well as good place for fishery sector, which induces high land values.

The results of this study agree well with the actual defence systems in Cat Hai island because one defence line actually has been used. On the other hand, the use of two defence lines is increasing in Hai Hau, Nam Dinh during the last 10 years. In Vietnam the discussion on whether to use one or two defence lines for protection of the coastal zone, is still going on. From this study, it appears conceptually that selection of defence options is site-specific and depends very much on the local conditions such as economic development plan, cost of land as well as availability of investment. This study could support a part of decision making process very well.

Since the defence option is already selected, the interest now is which protection level should we apply and which safety level is safe enough. The optimal safety level associated with its optimal dike height of a flood denfence system could be found by applying Risk Based Economic Optimization method, as developed and applied in Chapter 6, which has been applied widely in the Netherlands. Applying this to the case study of Nam Dinh coastal flood defence system, the optimal safety standard should be re-set at $P_{f,\, opt} = 0.021$ (return period of approximately 50 years) and the corresponding optimal dike height is about $H_{opt} = 9.4$ m. Obviously the actual safety standard of coastal flood defence in Vietnam, which is set at $P_f = 0.04$ to 0.05 (with the return period of 25 to 20 years) is not safe enough in terms of economic risk and in the view of the current Vietnamese development with fast economic growth. Therefore, increase of the safety level of coastal flood protection in Nam Dinh is necessary.

This study could provide important basic elements for establishment of guidelines in sea dike design in terms of the selection of layout (master plan) and dike heights. The methods and approaches used in this study have shown to work out well in the case of coastal flood defences in Vietnam. This could be useful for the actual Sea dike research program of Vietnam [Proposal of Subproject No.4, 2007] and for many other cases of coastal flood defences.

7.2 Recommendations

1. Recommendation on the study approach

❖ MCE tool

In process of using the MCE tool, weighting and ranking for each criterion and option are difficult to assess. These assessments depend on qualitative and subjective estimates. Therefore, to apply the MCE tool in more effective and consistence way and in order to gain an outcome which is close to the actual situation, some additional work should be done before using the matrix of MCE. For instance:

- Survey of the public opinion by distributing questionnaires to civilians in the region;
- Organize multidisciplinary meetings with all stake holders in the region;
- Final decision on weight factor should be based on a multidisciplinary expert assessment and practical situations in the area.

❖ Risk based cost benefit analysis

- In this study some simplifications have been made such as taking only average damage D based on statistical data; taking only recent economic growth rate (which normally fluctuated in different years), only overtopping failure mode is used in the analysis. Therefore, a full risk based approach which takes into account the uncertainties of these parameters and considers various other failure modes should also be done.

2. Recommendation of the selection of allowable overtop-discharge, [q]

In this study different flood defence options are defined based on an important parameter, the allowable overtop-discharge [q]. However, selections of these allowable overtop-discharges [q] are just based on existing figures, which were based on world-wide experiences and empirical approach. More physical insight into the determination of these [q] values are, therefore, advised to test for different type of dikes, different type of dike crest and dike slope protections, in order to come up with a more consistent answer. Actually, the EroGRASS[2] project is a good example.

3. Recommendation on other failure mechanisms

Other failure mechanisms should be taken into account in the process of comparison and selection of defence system. These failure mechanisms are erosion/toe scour outer

[2] The EroGRASS project is a project of European countries. The main objective of this project is to perform large scale model tests (scale 1:1) to investigate in detail the failure of grass cover layers due to (i) wave impact, (ii) wave run-up and wave run-down flow and (iii) wave overtopping.

slope, clip circle inner slope, slip circle outer slope, micro instability, piping, etc. The probability of various failure mechanisms should be balanced to avoid dangerous weak spot of dike defence system by making the fault tree of system.

4. Recommendation on boundary conditions of two case studies

❖ <u>Water level</u>

- Design water levels at Nam Dinh coast are 3.7 m, 4.0 m and 4.2 m corresponding to return period of 25, 50 and 100 years, respectively.
- In Hai Phong coast, DWLs are slightly lower than the water levels in Nam Dinh coast. Correspond to return period of 25, 50 and 100 years, DWLs at Hai Phong coast are determined in values of 3.3 m, 3.5 m and 3.7 m respectively
- These values of water level in Nam Dinh and Hai Phong coast are more different to the design water level of 3.29 m which is applied now in Vietnam (according to standard 14 TCN-130-2002 of Vietnam)
- In the determination of DWL, two following ways should be applied:

(i) DWL = MSL + Astronomical tide level + Storm surge + Gust bump + Seiches + Sea level rise

(ii) Based on statistical analysis of observed water level => extrapolated

Then compare both ways together, the larger values are used to design and in the analysis.

❖ <u>Waves</u>

- Significant wave heights in deep water are much different (about 1 m) between different scenarios (1/25, 1/50 and 1/100).
- Significant wave heights near the toe of the dikes are slightly different, of 10 cm to 20 cm, between scenarios because waves are breaking when propagating to the toe of the dikes.

5. Recommendation on the application of the analysis results/ study approach in case studies

Because Cat Hai is a small island, the use of two defensive lines is not effective because of economic considerations, as the analysis shows in this study. When severe storm floods occur, it is hard to evacuate people from the whole island to the mainland. It is suggested to divide the island into several polders. Thus, by separating the island into various independent small polders, people living in the flooded polders can move to other non-flooded areas of the island. For this reason, the best solution for coastal denfence in Cat Hai is using one defensive line, to reduce damage and loss caused by flooding; the island should be divided in many small sub-zones by improving/upgrading its current main traffic road system. This will lead to more effective evacuation and, thus, minimize the flood consequences when a flood occurs.

The Nam Dinh sea defence is typical and representative for sea defences in Vietnam. Therefore, it is necessary to update the safety standards of coastal flood defences for the whole country. The present risk based models are thought to be powerful tools to support the decision process to set (or re-set) the safety levels of protection in relation to investments and acceptable consequences for various scales of protection in Vietnam.

6. Recommendation on wetland conservation/ restoration

The wetlands in between two dike lines of the using two dike system alternative (transitional zone) have many values. It should be cared by development and conservation or restoration if it is degraded and destroyed, to get its values.

The values of the wetlands can be realized in different ways that humans interact and benefit from them. The most important values are *use values* that are realized through human interaction: (i) direct values which are the most tangible and relate to the products and benefits that can be derived from use of the wetlands (fish, food, reeds or tourism); (ii) indirect use values can also be through of as services, and arise from benefits provided to existing activities or resources through their occurrence. An example would be the protection of existing property by regulation of flooding; (iii) potential future use values which arise when there is uncertainty over the future demand for a product or service and/ or its availability in the wetland in the future. It reflects a need to estimate the benefit of conserving them for this purpose. An example would be the protection of future property by regulation of flooding.

The other values of the wetlands are *non-use values* which can be realize in: (i) biodiversity: many species of animals and plants and their habitats depend on the wetlands for their continued existence. Some species live permanently in the wetlands and others depend on them for key aspects of their life circles. Many rare or threatened species depend on the wetlands and people value their continued presence in their own right and not as a source of food or other direct use values; (ii) cultural and heritage values: people who do not directly utilize the wetlands can also place a value on it because of its essential character or meaning to them. For instance they may wish to see it preserved, for its cultural and heritage values.

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APPENDIX

<u>A-1:</u> Best fit distribution analysis for observed water level data at Phu Le station, Nam Dinh coast. Data is picked up by POT method with threshold level of 175 cm.

Weibull is the best fitted distribution (top rank of the second column in Figure A.1)

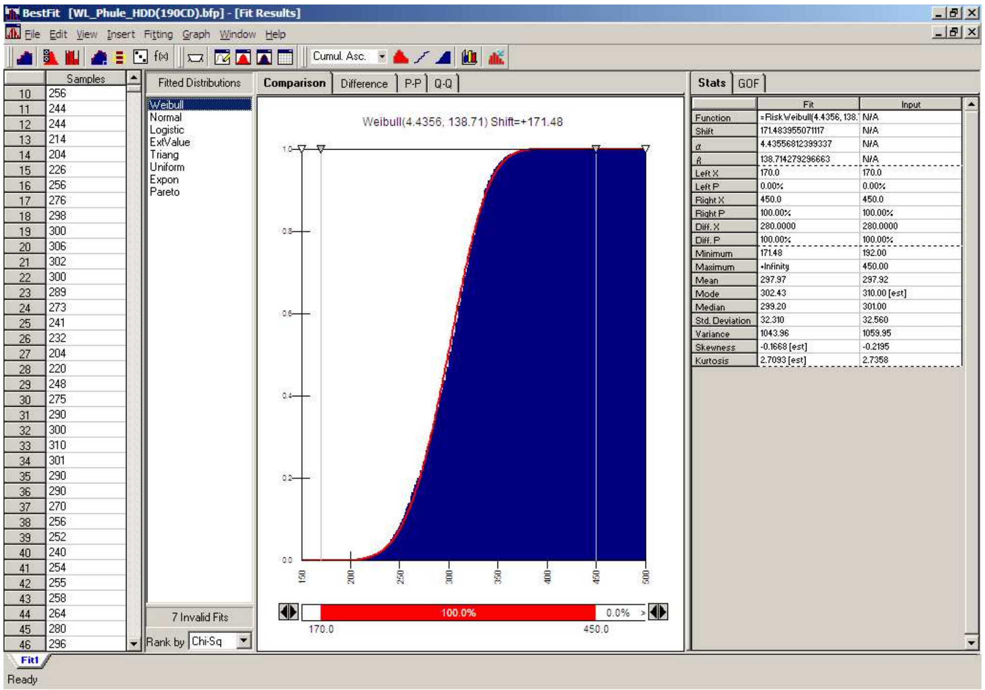


Figure A.1: Weibull distribution fit to observed water level data at Phu Le station, Nam Dinh

Table A.1: Summary statistic parameters of water level at Phu Le station, Nam Dinh

Weibull	Fit	Input
Shift	171.5	N/A
α	4.4	N/A
β	138.7	N/A
N/A	N/A	N/A
N/A	N/A	N/A
N/A	N/A	N/A
Left X	170	170
Left P	0.00%	0.00%
Right X	450	450
Right P	100.00%	100.00%
Diff. X	280	280
Diff. P	100.00%	100.00%
Minimum	171.48	192
Maximum	infinity	450
Mean	297.97	297.92
Mode	302.43	310.00 [est]
Median	299.2	301
Std. Deviation	32.31	32.56
Variance	1043.96	1059.95
Skewness	-0.1668 [est]	-0.2195
Kurtosis	2.7093 [est]	2.7358

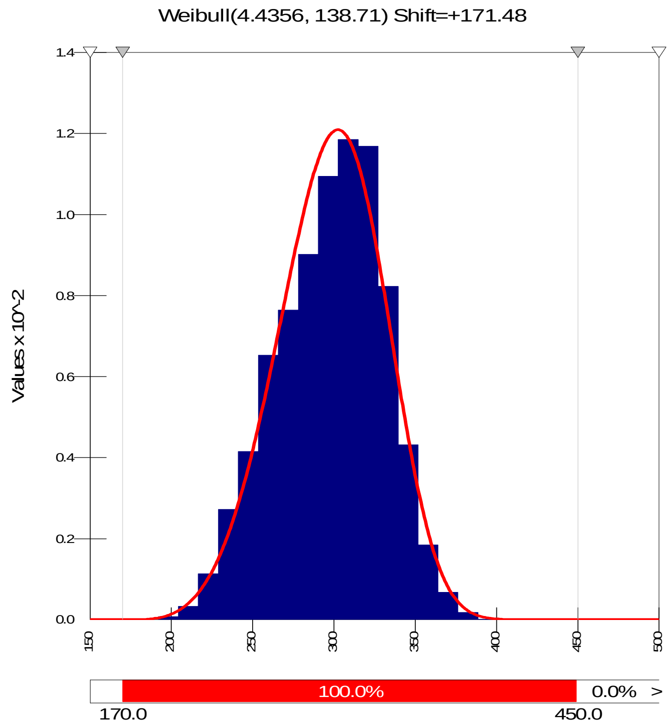


Figure A.2: Comparison of the fitted distribution and the observed water level data at Phu Le station by probability density function (PDF)

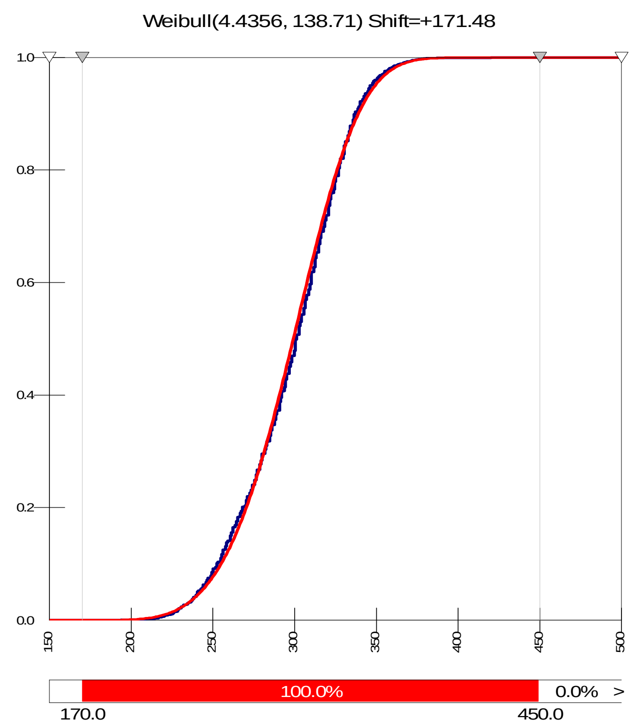


Figure A.3: Comparison of the fitted distribution and the observed water level data at Phu Le station by cumulative distribution function (CDF)

<u>A-2:</u> Best fit distribution analysis for observed water level data at Hon Dau station, Hai Phong coast. Data is picked up by POT method with threshold level of 175 cm.

Weibull is the best fitted distribution (top rank of the second column in Figure A.4)

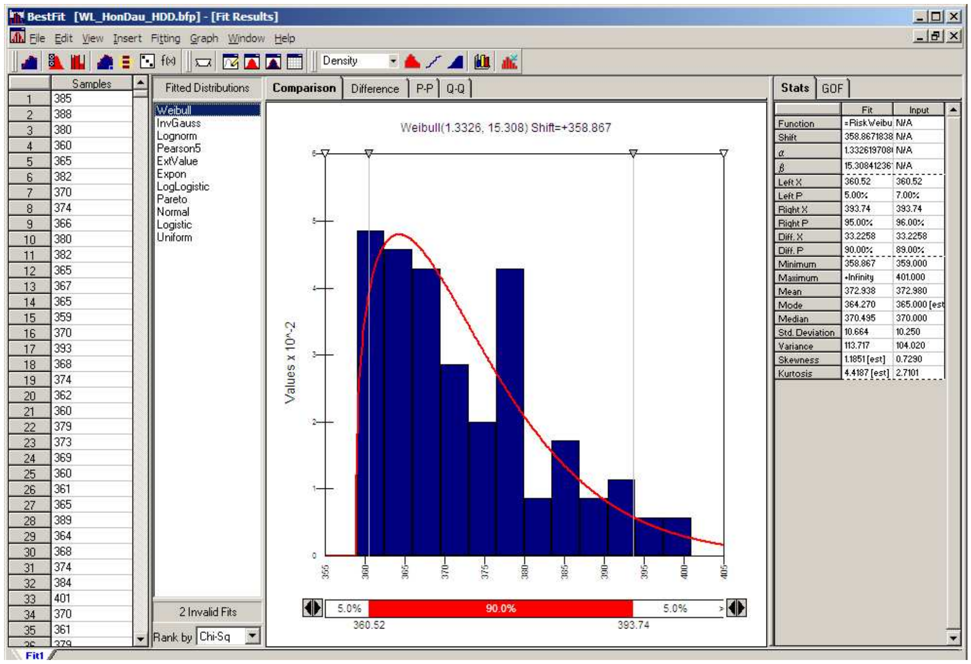


Figure A.4: Weibull distribution fit to observed water level data at Hon Dau station, Hai Phong

Table A.2: Summary statistic parameters of water level at Hon Dau station, Hai Phong

Weibull	Fit	Input
Shift	358.9	N/A
α	1.3	N/A
β	15.3	N/A
N/A	N/A	N/A
N/A	N/A	N/A
N/A	N/A	N/A
Left X	360.52	360.52
Left P	5.00%	7.00%
Right X	393.74	393.74
Right P	95.00%	96.00%
Diff. X	33.23	33.23
Diff. P	90.00%	89.00%
Minimum	358.87	359.00
Maximum	Infinity	401
Mean	372.94	372.98
Mode	364.27	365.00 [est]
Median	370.5	370
Std. Deviation	10.66	10.25
Variance	113.72	104.02
Skewness	1.1851 [est]	0.729
Kurtosis	4.4187 [est]	2.710

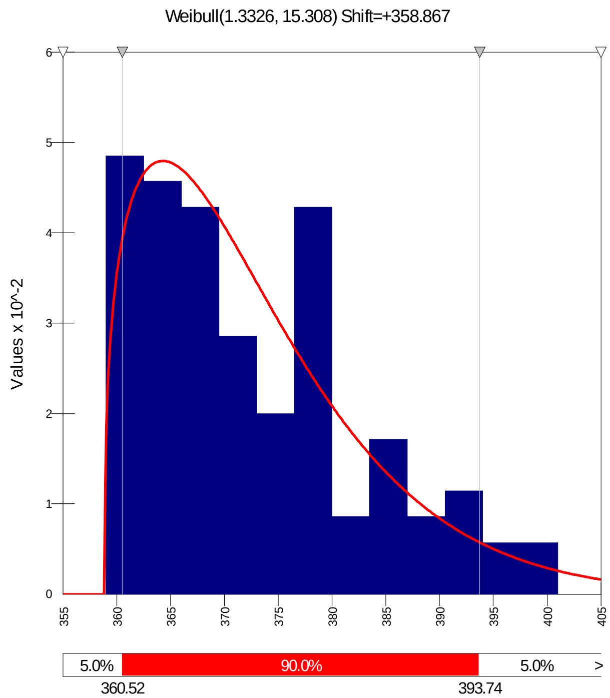


Figure A.5: Comparison of the fitted distribution and the observed water level data at Hon Dau station by probability density function (PDF)

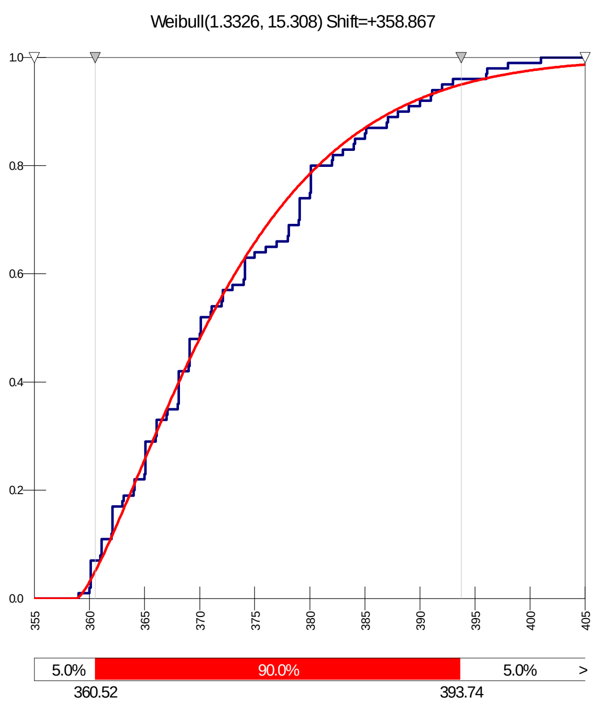


Figure A.6: Comparison of the fitted distribution and the observed water level data at Hon Dau station by cumulative distribution function (CDF)

A-3: Best fit distribution analysis for observed wave height data at Hon Dau station, Hai Phong coast. Data is picked up by POT method with threshold level of 250 cm.

Weibull is the best fitted distribution (top rank of the second column in Figure A.7).

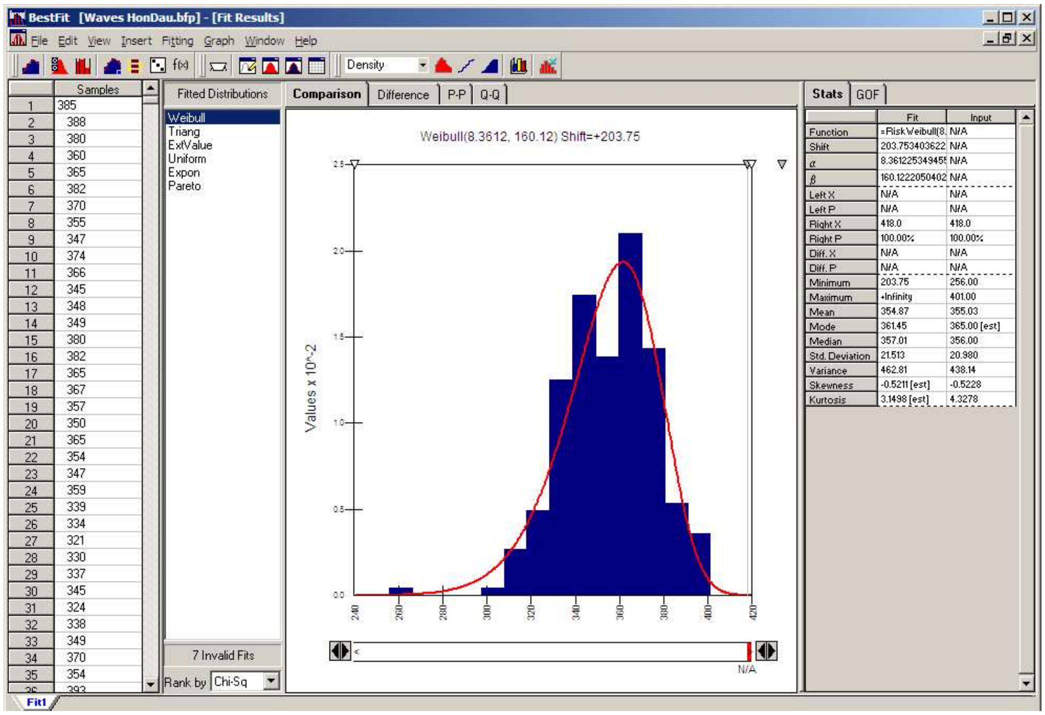


Figure A.7: Weibull distribution fit to observed wave height data at Hon Dau station, Hai Phong

Table A.3: Summary statistic parameters of wave height at Hon Dau station, Hai Phong

Weibull	Fit	Input
Shift	203.75	N/A
α	8.36	N/A
β	160.12	N/A
N/A	N/A	N/A
N/A	N/A	N/A
N/A	N/A	N/A
Left X	N/A	N/A
Left P	N/A	N/A
Right X	418	418
Right P	100.00%	100.00%
Diff. X	N/A	N/A
Diff. P	N/A	N/A
Minimum	203.75	256
Maximum	infinity	401
Mean	354.87	355.03
Mode	361.45	365.00 [est]
Median	357.01	356
Std. Deviation	21.513	20.98
Variance	462.81	438.14
Skewness	-0.5211 [est]	-0.5228
Kurtosis	3.1498 [est]	4.3278

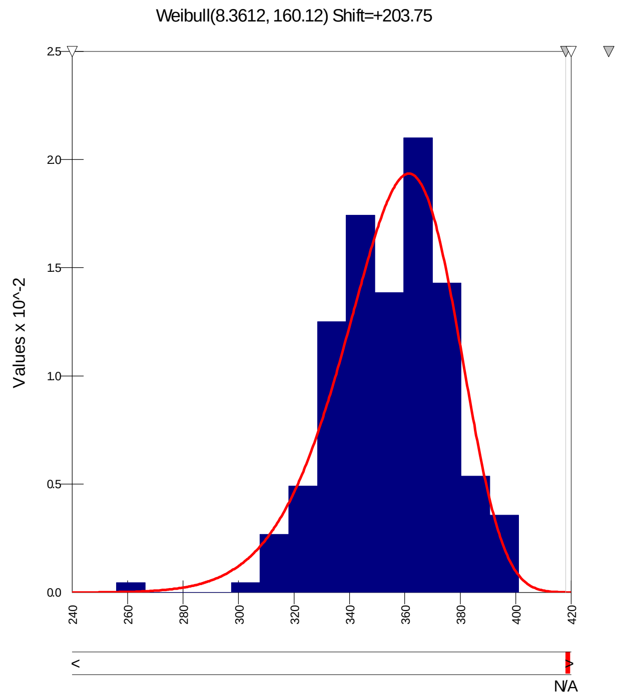


Figure A.8: Comparison of the fitted distribution and the observed wave height data at Hon Dau station by probability density function (PDF)

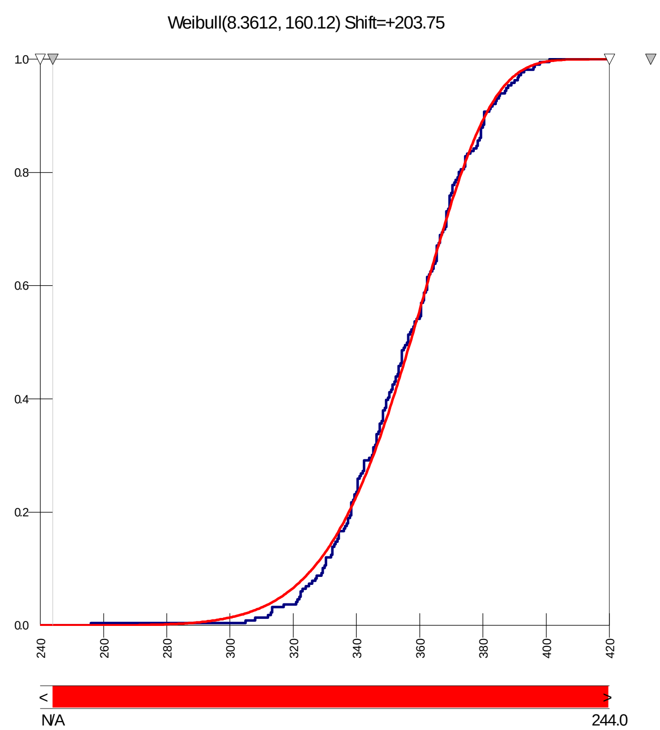


Figure A.9: Comparison of the fitted distribution and the observed wave height data at Hon Dau station by cumulative distribution function (CDF)

<u>A-4:</u> Best fit distribution analysis for observed Wave height data at Bach Long Vi station, offshore of South China sea coast.

Exponential is the best fitted distribution (top rank of the second column in Figure A.10)

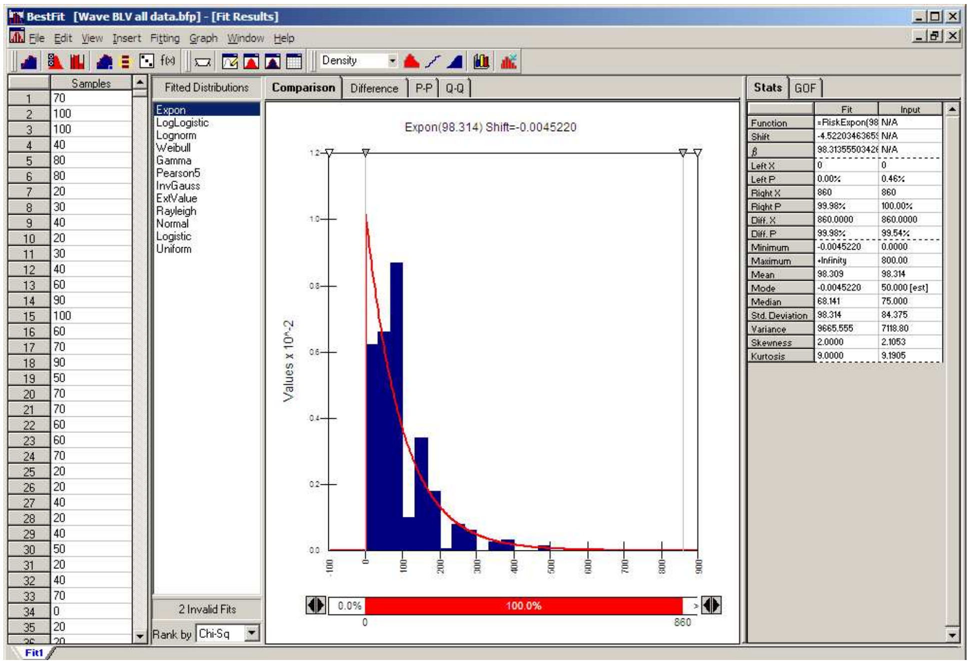


Figure A.10: Weibull distribution fit to observed wave height data at Bach Long Vi station

Table A.4: Summary statistic parameters of wave height at Bach Long Vi station, Hai Phong

Exponential	Fit	Input
Shift	-4.50E-03	N/A
β	98.31	N/A
N/A	N/A	N/A
N/A	N/A	N/A
N/A	N/A	N/A
N/A	N/A	N/A
Left X	0	0
Left P	0.00%	0.46%
Right X	860	860
Right P	99.98%	100.00%
Diff. X	860	860
Diff. P	99.98%	99.54%
Minimum	-0.004522	0
Maximum	infinity	800
Mean	98.309	98.314
Mode	-0.0045	50.000 [est]
Median	68.141	75
Std. Deviation	98.314	84.375
Variance	9665.555	7118.8
Skewness	2	2.1053
Kurtosis	9	9.1905

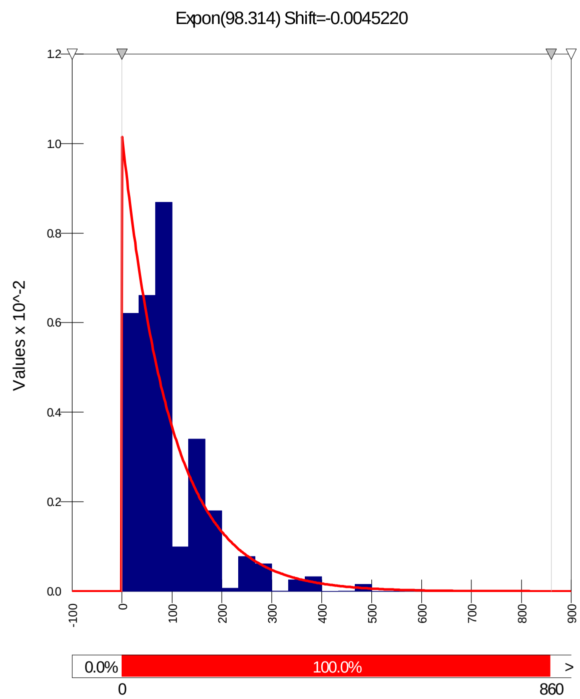


Figure A.11: Comparison of the fitted distribution and the observed wave height data at Bach Long Vi station by probability density function (PDF)

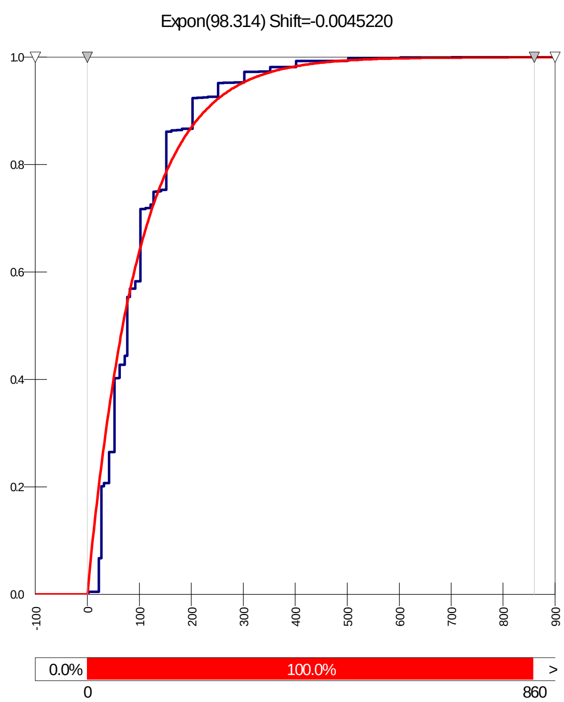
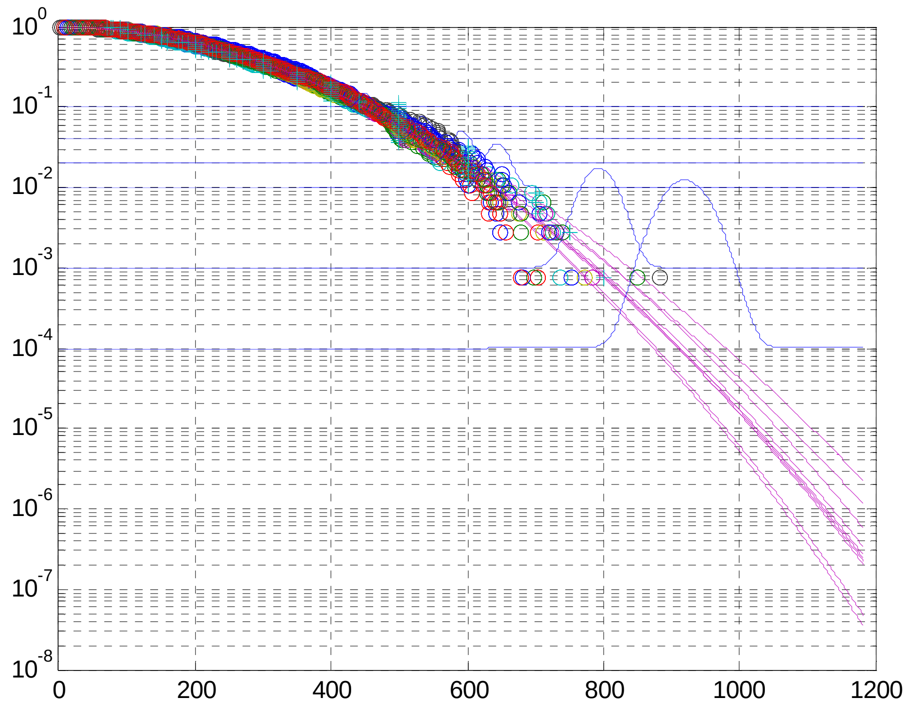
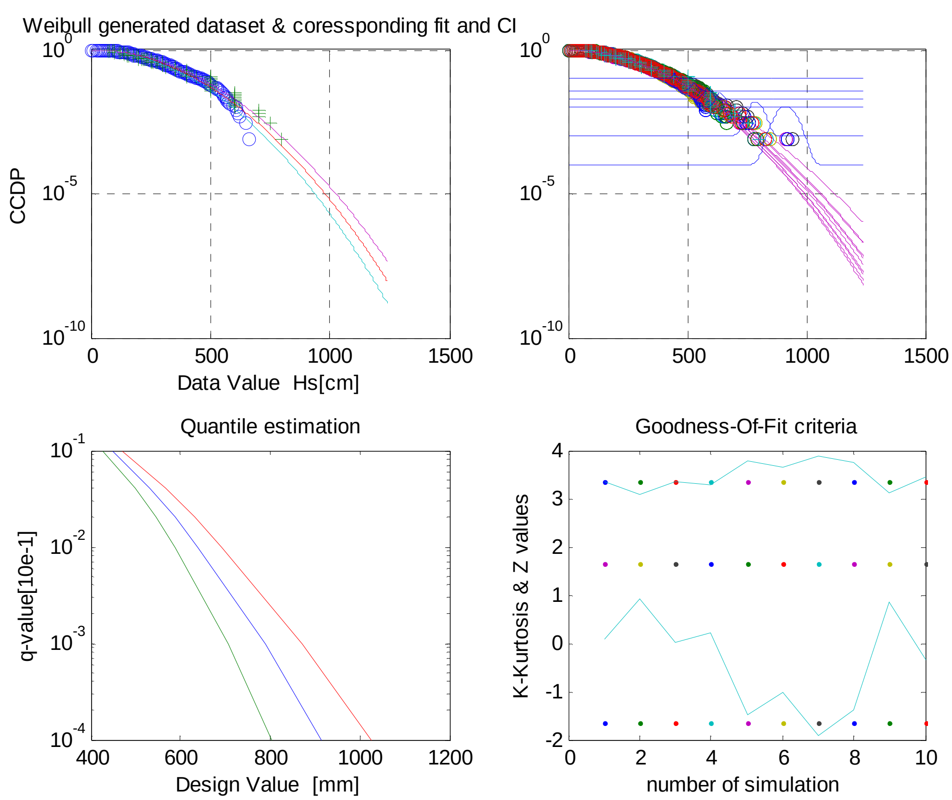


Figure A.12: Comparison of the fitted distribution and the observed wave height data at Bach Long Vi station by cumulative distribution function (CDF)




Weibull generated dataset & coressponding fit and CI
CCDP
Data Value Hs[cm]
Quantile estimation
q-value[10e-1]
Design Value [mm]
Goodness-Of-Fit criteria
K-Kurtosis & Z values
number of simulation