Analysis of failure situations for sea dike in Northern Vietnam and point out some suitable solutions

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Subject: Individual study
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CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

Vietnam consists of about 3,260 km of coastline stretching from latitude 8° North to 22° and from longitude 105° East to 108° East as shown in Figure 1.1. Along the coastline, there concentrate areas of densely populated and industrial areas. Also along the coast, there are many lowland areas needing to be protected by sea dikes.

The most remarkable hydrometeorological feature of Vietnam is the presence of typhoons with very high frequency of occurrence. Every year, in average 8 to 10 typhoons strike the coast of Vietnam, mainly concentrating on the central and north part of the country. Strong winds, large waves and high waters usually combined with intense rainfalls cause a lot of damages to coastal structures and losses to coastal economy.

Figure 1.1 General location of Vietnam
The sea dike system in Vietnam has been set up for a very long time. Most of them are mainly made of soil, sand constructed by hand based on local experiences and directly affected by actions of waves, tide and currents.

With understanding the important of sea dikes system, the sea dikes have been maintained every years, the outer slope has been covered by stone and concrete block, the crest and inner slope has been covered by grass. Recently, a new standard, 14 TCN 130-2002 has been published as a guideline for design sea dikes in Vietnam.

1.2 CERTAIN SITUATION OF SEA DIKES IN NORTHERN VIETNAM

The study area in this report is the North of Vietnam that constitutes part of dynamic coastline within the Red River Delta, which very often due to erosion and accretion processes. The North’s coastline with 720 km of sea dikes from MongCai - QuangNinh to Hauloc - ThanhHoa, which 454 km is directly affected by the sea. The sea dikes in the North area are mainly used to protect agricultural, aquaculture and salt production activities from salt intrusion and sea waves. In this case, seawater is not allowed to overtop the crest of these dikes; therefore, crown walls are usually seen at the top of the dikes. Most of sea dikes have been built with the core is sand with cover layer is clay thickness 50-80cm and the armour layer is revetment.

Figure 1.2 The study area
However, revetments are applied to only 219 km of dikes. The typical cross profile of the
dikes in the North is illustrated in Figure 1.3. The crest level arranges from (+4.50) to
(+5.50). The outer slope is m = 3 to 4 and covered by interlocking concrete block from the
toe to (+3.50). From (+3.50) to crest is covered by pave stone with diameter D = 30cm.
Thickness of revetment is δ = 20 to 50cm. Under armour layer is filter layer by gravel 0.1-
0.15m thick and geotextile. The inner slope is m = 2.5 to 3 and covered by grass. Crest
width about 4.0m to 6.0m. (14TCN 130-2002 - MARD, 2002)

![Figure 1.3 The sea dikes in Northern area](image1)

The toe is important part for stability of sea dike in general and revetment particular protect
structure from scour hole. Normally in Vietnam, three kinds of toe protection has been used
are trapezium of riprap, concrete cylinders or combine of them. The cylinders have
diameters of 1.0m to 1.2m and length 1.5m to 2.0m.

![Figure 1.4 The sea revetments in CoVay - NamDinh](image2)
In some areas (HaiPhong and ThaiBinh) one can see in front of the dike is a sedimentation area with some mangroves (figure 1.5). Mangroves play significant roles in environment as well as the protection for the coast and structures against erosion, damage. In Haiphong, mangrove planting has been tried at several places but was not successful (probably because of the high sand content of the soil). In some places were sedimentation was abundant and also some mangroves were growing, new shrimp ponds were created outside the dike areas.

![Figure 1.5 The mangroves forest and the shrimp ponds along the coast of DoSon - HaiPhong](image)

**1.3 PROBLEM IDENTIFICATION**

According to the annual report of provinces in the North of Vietnam, most of failure occurs in dikes and revetments due to the strong impacts of wind waves in storm combine with high sea water level. Instance, during the period from 1976 to 1995 about 934,000 m$^3$ of stone were taken away from the sea dikes of NamDinh. The damages of dikes and revetment as result of structure collapse as show in Figure 1.6. Beside it, the less in data and design method, poor in construction also effect to quality and dimensions of dikes and revetments.

![Figure 1.6 The damage of sea dyke in HaiHau – NamDinh](image)
Some causes of failures in dikes and revetment in North of Vietnam are listed below:

* **Boundary condition, design method:**
  + Less of data, measurement instruments in waves data, geotechnical
  + Design method is not unification
  + Less maintenance
  + Quality of constructions

* **Physical condition:**
  + Failures in outer slope of revetment cause of instability of armour material under actions of waves and currents
  + Sliding in outer slope because of internal friction in the body of the dike decreases, that mean the friction along the slip circle decreases and the core is no longer stable
  + Damage in toe and bottom protection occurs when the waves reflection and concentrated currents remove the stones of toe
  + Settlement occurs because of the deadweight of the dikes body.

**1.4 OBJECTIVES OF THE STUDY**

This study mainly focuses on the analysis of failure situation for sea dikes represent in HaiPhong, ThaiBinh, NamDinh and ThanhHoa to answer the questions:

1. Analysis the existing situations of dikes and revetments at the North of Vietnam
2. What are the causes of failures on sea dikes and revetments?
3. What will be the solutions for new design of sea dikes?

The solutions must be combining of technical and economic demands for correspond to Vietnam’s conditions.

**1.5 APPROACH OF THE STUDY**

The first step of the study is collect data from present conditions of sea dikes system in the North of Vietnam. Analysis the regular failures in parts of dikes based on available data. Using geotechnical data to calculate the slope stability factor of dikes and revetments by GEO-SLOPE program (Canada) with Bishop method. Comparing the results of apply Dutch formula and Vietnamese formula in design dikes and revetments. From that, come up with conclusions on the cause of damages of dikes and revetments. Base on these conclusions, the suitable solutions for new design criteria will be suggested.
CHAPTER 2. NATURAL CONDITIONS AT THE NORTH OF VIETNAM

2.1 COAST LINE

The North of Vietnam is constitutes part of dynamic coastline within the Red River Delta. The orientation of coast is approximately 40° to the North direction. The main types of coastline are low coastline types and low beach types with the normal beach slope ranging from 1:250 to 1:800. The grain size is $d_{50} = 0.1$mm to 0.15mm; $d_{90} = 0.2$mm (NamDinh)

Most of sea dikes have been built in soft soil foundation composed of sand and clay. Outside the sea dike, the coastline has been shaving due to action of waves and tide current, the small grains soil being taking away causing of coarsening of grain size on the beach so this condition is mainly effects to the cross-shore profile of the beach.

![Figure 2.1 The beach in DiemDien - ThaiBinh](image)

2.2 CLIMATES AND METEOROLOGY

The climate in the North of Vietnam is tropical with a pronounced maritime influence. The summer is warm and humid and the winter is cool and dry. The rainy season starting from April to October but the heaviest rainfall is in August and September. There are about six or seven strong typhoons occur frequent between July and October, every year. Typhoons storms usually come from the West pacific, through the Philippines or Eastern Sea and shoot into the coastal areas of South China and Vietnam.

Go with the typhoons is heavy rains. Base on annual raining measure data, the raining quantity in storm at the North of Vietnam is about 100mm/day, possible 300-400mm/day causes several flooding. When such storms break over the main land, a huge amount of water is released, damaging the sea dikes and flood in coastal areas.
2.3 OCEANOGRAPHY

2.3.1 Tide

According to tidal map of Vietnam, Field observations have showed that the astronomical tides are full diurnal type in Doson - Haiphong (Root, 1997) and regular diurnal type in NamDinh coast. In Table 2.1 introduces some mean values of tidal amplitude at observation stations in the North of Vietnam. In general, the tidal range from 1.84 to 2.19m.

Table 2.1 Observation mean water level at stations datum (m)

<table>
<thead>
<tr>
<th>Stations</th>
<th>HonDau</th>
<th>HonGai</th>
<th>CuaOng</th>
<th>CoTo</th>
<th>BaLat</th>
<th>LachTruong</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20°40',106°49'</td>
<td>21°57',107°04'</td>
<td>21°02',107°22'</td>
<td>20°58',107°46'</td>
<td>20°21',106°38'</td>
<td>19°53',105°56'</td>
</tr>
<tr>
<td>Mean water level (cm)</td>
<td>191</td>
<td>206</td>
<td>219</td>
<td>208</td>
<td>192</td>
<td>184</td>
</tr>
</tbody>
</table>

Note: the data is referred to mean low water level (chat datum)

2.3.2 Wind

For the study area, there is no offshore island, so that the North of Vietnam is subject to the wind generated from every direction. The wind climate can be divided into a summer (May to August) and winter monsoon period (October to March). During the summer monsoon period moderate wind have the average force 3Bft from Southwest direction prevail. In winter the predominant wind direction is Northeast. The average wind force during the winter monsoon period is about 4Bft.

In this study the observed wind data at Bach Long Vy island was used (Tonkin Gulf, 20.133° latitude; 107.72° longitude).

![Location Bach Long Vy Island](image)
Individual Study - Analysis of failure situation for sea dike in Northern Vietnam

Figure 2.3 Main wind directions in Northern Vietnam

Table 2.2 The wind data at Bach Long Vy island (observation 1975 – 1995)

<table>
<thead>
<tr>
<th>Class (m/s)</th>
<th>Calm</th>
<th>N</th>
<th>NE</th>
<th>E</th>
<th>SE</th>
<th>S</th>
<th>SW</th>
<th>W</th>
<th>NW</th>
<th>Sum</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-5</td>
<td>843</td>
<td>3103</td>
<td>2843</td>
<td>1875</td>
<td>1858</td>
<td>578</td>
<td>277</td>
<td>320</td>
<td></td>
<td>11697</td>
</tr>
<tr>
<td>6-10</td>
<td>505</td>
<td>5160</td>
<td>1378</td>
<td>810</td>
<td>3440</td>
<td>530</td>
<td>77</td>
<td>108</td>
<td></td>
<td>12008</td>
</tr>
<tr>
<td>11-15</td>
<td>156</td>
<td>2013</td>
<td>73</td>
<td>79</td>
<td>1043</td>
<td>65</td>
<td>6</td>
<td>9</td>
<td></td>
<td>3444</td>
</tr>
<tr>
<td>16-20</td>
<td>90</td>
<td>863</td>
<td>11</td>
<td>23</td>
<td>77</td>
<td>4</td>
<td>2</td>
<td>19</td>
<td></td>
<td>1089</td>
</tr>
<tr>
<td>21-25</td>
<td>16</td>
<td>27</td>
<td>0</td>
<td>2</td>
<td>5</td>
<td>1</td>
<td>0</td>
<td>5</td>
<td></td>
<td>56</td>
</tr>
<tr>
<td>26-30</td>
<td>3</td>
<td>4</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>0</td>
<td>3</td>
<td></td>
<td>16</td>
</tr>
<tr>
<td>31-35</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td>36-40</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td><strong>536</strong></td>
<td><strong>1617</strong></td>
<td><strong>11171</strong></td>
<td><strong>4307</strong></td>
<td><strong>2791</strong></td>
<td><strong>6429</strong></td>
<td><strong>1181</strong></td>
<td><strong>362</strong></td>
<td><strong>465</strong></td>
<td><strong>28859</strong></td>
</tr>
</tbody>
</table>

2.3.3 Storm surge

According to Tran (Tran, 2001; HWRU, 2000) said, “Typhoons are normally accompanied by storm surges. During past 30 years, 50% of storms has caused storm surge of over 1m, 30% typhoons over 1.5m and a few typhoons were coupled with a surge exceeding 2.5m. The high water levels destroyed sea dikes, and initiated flooding of lowland coastal areas through overtopping and breaching of dykes”

Table 2.3 The maximum value of storm surge

<table>
<thead>
<tr>
<th>Area</th>
<th>Position</th>
<th>Maximum storm surge</th>
<th>Storm surge can occur (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>North of Vietnam</td>
<td>22°N - 21°N</td>
<td>2.2</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>21°N - 20°N</td>
<td>2.2</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>20°N - 19°N</td>
<td>3.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>
2.3.4 Sea level rise
Sea level rise is the result of climatologic changes in a long term. In design the crest height, sea level rise has been taken into account. Normally, in the Northern Vietnam the sea level rise take approximately 20cm.

2.3.5 Currents
According to field observations done by Hung et al. (2001), the wave-induced long-shore currents have average value of 0.2 to 0.4 m/s and maximum of 0.7 to 1.0 m/s at depth of 2.5m. The long-shore wave driven currents are southwestward in winter and northeastward in summer.

2.3.6 Waves
The sea at the North of Vietnam is open sea so the wind fetch is long enough for wave growth and approaches the shoreline without any obstacles, which can cause of damage to shoreline and sea dikes system. According to the observation in period from 1975 to 1987 waves at NamDinh coast, the wave height about 0.65 to 1.0 m in summer (from April to August) with period ranging from 5 to 7 seconds and 0.8 to 1.5m in winter (from September to March) with period 7 to 10 seconds.

In this case, the wave data selected from www.hydrobase.net website (9/2003), which taken from 1960 to 1997 with 32452 number of observation for all year season. The wave scenario is calculated from the wave data as follows:

* Calculating wave period.

1. Calculate the average value of significant wave height with the wave data of the area.
2. Multiply number of occurrence with corresponding wave period in one line (for one value of averaged wave height)
3. Make summation in one line, then divide by total number of occurrence, the results is the value of mean period corresponding to averaged wave height in that line.
4. Peak period is equal to 1.2 times mean period.

Based on all the steps above to wave height seeks. The value of averaged significant wave height, the period each direction are then calculated.

* Calculating transformation from deep water to the shallow water depth.
Next step is to calculate significant wave height, wave angle and duration of each wave condition at the seaward boundary of the cross-shore profile. In this case, calculate the wave height at water depth $h = 3$m (at location of dykes).
The applied formulas are as follows:

\[ H_i = H_0 \times K_{sh} \times K_r \]

Where:

- \( H_0 \): Wave height at deep water
- \( H_i \): Wave height at shallow water.

and

\[ K_r = \frac{\sqrt{\frac{C_{g_0}}{C_g}}}{\sqrt{2\pi \tanh(kh)}} \]
\[ K_r = \frac{\cos \alpha_0}{\cos \alpha} \]
\[ \sin \alpha = \sin \alpha_0 \tanh(kh) \]

In this calculation, the data of wave, which have direction from the range of 225° to 345° and -15° to -45° does not take into account since it is assumed that the waves come from landward. As can be see on the map, the depth contours in the North of Vietnam are rather parallel. Hence assume that refraction factor is constant. The final result is show in appendix 1.

![Figure 2.4 Orientation coastline and wave direction in Northern Vietnam](image)

**Table 2.4** The relationship between wave height and % exceedance.

<table>
<thead>
<tr>
<th>Significant wave height (m)</th>
<th>60</th>
<th>90</th>
<th>120</th>
<th>150</th>
<th>180</th>
<th>210</th>
<th>Total</th>
<th>Percent occurrence</th>
<th>Percent exceedance</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.25</td>
<td>1.68</td>
<td>1.47</td>
<td>1.43</td>
<td>1.42</td>
<td>0.85</td>
<td>9.85</td>
<td>16.70</td>
<td>7.2</td>
<td>100</td>
</tr>
<tr>
<td>0.25-0.5</td>
<td>17.86</td>
<td>11.89</td>
<td>18.59</td>
<td>26.56</td>
<td>15.77</td>
<td>7.87</td>
<td>98.54</td>
<td>42.6</td>
<td>92.8</td>
</tr>
<tr>
<td>0.5-0.75</td>
<td>18.11</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.36</td>
<td>19.47</td>
<td>8.4</td>
<td>50.1</td>
</tr>
<tr>
<td>0.75-1.0</td>
<td>5.51</td>
<td>7.19</td>
<td>15.59</td>
<td>28.96</td>
<td>12.51</td>
<td>0.46</td>
<td>70.22</td>
<td>30.4</td>
<td>41.7</td>
</tr>
<tr>
<td>1.0-1.5</td>
<td>8.90</td>
<td>3.07</td>
<td>3.89</td>
<td>7.43</td>
<td>2.77</td>
<td>0.12</td>
<td>26.18</td>
<td>11.3</td>
<td>11.3</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>52.06</strong></td>
<td><strong>23.62</strong></td>
<td><strong>39.50</strong></td>
<td><strong>64.36</strong></td>
<td><strong>31.90</strong></td>
<td><strong>19.66</strong></td>
<td><strong>231.10</strong></td>
<td><strong>100.0</strong></td>
<td></td>
</tr>
</tbody>
</table>
Since the bigger part of the coastline at the North of Vietnam has a shallow foreshore in most cases the design wave heights for the sea defences will be depth limited. Based on the present bathymetry the design wave height for the sea defences design can be calculated.

The wave transmits from deepwater to shallow water (at dikes location) when it reach to wave height equal to the half of water depth, it will be break.

According to McCowan, 1984, Munk, 1949 the breaker index can be calculate by the formula:

\[ \gamma_b = \frac{H_b}{d_b} = 0.5 \]

So, at the water depth \( h = 3 \text{m} \Rightarrow H_s = 1.50(\text{m}) \). The value significant wave height approximately 1.50 has been taken into account in dikes and revetments design at most Northern provinces.

Because of lack data in profile cross-section to analysis changing of profile at front of the sea dike. In this case, assuming that the profile with slope 1:80, value of water depth \( h=3.0\text{m} \) has taken into account the wave height at shallow water (no scour hole include).
CHAPTER 3. ANALYSIS OF FAILURE SITUATIONS OF THE SEA DIKES

3.1 THE INACCURACIES EFFECT OF BOUNDARY CONDITIONS

The design of sea dikes is determined by the local conditions like tides, waves, currents, bottom profiles, morphological processes and the characteristics of sediments and soils. These conditions can be obtained by measurements and analyses are not available data. However, data always are not available or not sufficient due to the insufficient of funds to collect or measure on one hand and a loosely cooperation between the different parties like Department of Dyke Management and Flood control; Vietnam Institute for Water Research; Hanoi Water Resources University; Hydro-meteorological Service of the marine Hydro-meteorological Center and Vietnam Science Institute on the other hand.

Some hydraulic boundary conditions that are usually insufficient are follows.

3.1.1 Waves
+ As wave climate, lack of data for a specific area to know the deep-water wave climate as wave height, wave period, frequently distribution.
+ No data on wave period and wave height during typhoons

3.1.2 Water levels
+ The water level is considered as one of important parameter to determine the crest level of the sea dikes. The water level at the dikes is a combination of several effects. The most important are astronomical tide, wind set up (storm surge) and sea level rise. However, sea level rise has not been taken in to account as it is a long-term effect.
+ No data of storm surge for local area.

3.1.3 Bottom topography
The bottom topography is important since it determines how waves and currents as on the sea dikes. A shallow foreshore can be very advantageous with respect to the maximum wave forces acting on the dikes. In case of an eroding coast the water depth in front of the dikes will increase in time, which will also result in increasing wave attack. Therefore, it is important to know the bottom topography in front of the dikes. However, so far bathymetry is not significantly considered except eroding area such as HaiHau district and GiaoThuy district - Namdinh province.

3.1.4 Currents
Currents are the result of a superposition of a number of difference driving forces. It is of significant important with respect to the development of the coastline and the stability of sea structures. However, designers have to use data to distinguished wave driven current from another currents like wind driven current or tide driven currents etc. Especially typhoons will cause significant impacts on the current and sea dikes but it can not be measured due to without instruments.
3.2 DESIGN METHOD

In general, there are two types of dike in the Red River Delta are sea dikes and estuary dikes. Dikes and revetments are both types of shoreline protection. The most important aspects in a dike design are crest level and stability under design conditions to prevent flooding and salt intrusion of the area behind the dike. Where revetment is to prevent of erosion of the dike-front due to wave action by the both slope protection and toe protection.

3.2.1 Design of crest height

According to PAM project, for calculation of the crest height of the dikes in the 6 Northern provinces of Vietnam from QuangNinh to ThanhHoa, the following formula is used (14TCN 130-2002-MARD, 2000).

\[ H_d = H_t + H_{nd} + H_{sl} + a \]

Where:
- \( H_d \): Design crest height (m)
- \( H_t \): Design tidal level corresponding to probability of \( P = 5\% \) (m)
- \( H_{nd} \): The height of storm surge
- \( H_{sl} \): The height of wave run up
- \( A \): Free board

The results of the calculations, which were made, are summarized in table 3.1

+ Water level

When design the crest height of a dyke, the most important parameter is water level (exceedance probability) that is combination of astronomical tide (5% exceedance probability) and wind set-up. Normally, the recording in over 30 years has been applied so that water level will statistically be exceeded once in every 10 years. The result of this combine with sea level rise is using to calculate water level for sea dikes.

Table 3.1 The water level data in some Stations along the coast of Northern Vietnam

<table>
<thead>
<tr>
<th>Station</th>
<th>Hon Ngu</th>
<th>Hon Dau</th>
<th>Hon Gai</th>
<th>Cua Ong</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max water level recorded (30 years)</td>
<td>1.98</td>
<td>2.35</td>
<td>2.67</td>
<td>2.55</td>
</tr>
<tr>
<td>Mean sea level</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Min water level recorded (30 years)</td>
<td>-1.76</td>
<td>-1.95</td>
<td>-2.32</td>
<td>-2.10</td>
</tr>
</tbody>
</table>

Note: The data relative to MSL (m)

Wind set-up is the water surface rising which result of wind causes a friction force on the water surface, resulting in a stress on the surface. Parameter effect to wind set up is fetch length, water depth and the wind velocity and geometry of basin.
In reality, most of dikes system in Vietnam has been design with the critical to stand in storm level 9 (20% frequency occurs - TCN14) that mean the most dangerous in case the wave direction perpendicular with the dike, the wind velocity is 25m/s and wind fetch approximately 100km.

According to Vietnam Institute for Water Research, the storm surge level measured from 1960 to 1990 at the North -21°N is:

- \( H_{nd} = 1.5 - 2.0m \) is 6 - 8% (frequency occurs of storm surge)
- \( H_{nd} = 1.0 - 1.5m \) is 8 - 38%
- \( H_{nd} = 0.5 - 1.0m \) is 38 - 50%

That mean the storm surge with frequency 20% along the coastline Northern Vietnam is approximately 1.10m.

Checking with Netherlands formula to calculate storm surge value (storm level 9):

\[
\Delta S = \frac{1}{2} C_W \frac{\rho_{air}}{\rho_{water}} \frac{U^2 F}{gh} \cos \phi
\]

Where:
- \( C_W \): Coefficient = 0.8*10^{-3}
- \( \rho_{air} \): Density of air = 1.25kg/m^3
- \( \rho_{water} \): Density of water = 1030kg/m^3
- \( U \): Wind velocity = 25(m/s)
- \( F \): Fetch length = 100km
- \( \phi \): Angle between wind direction and axis (degrees)

Thus:

\[
\Delta S = \frac{1}{2} \times 0.8 \times 10^{-3} \times \frac{1.25}{1030} \times \frac{25^2 \times 100000}{10 \times 2.29} \times 1 = 1.18 \text{ (m)}
\]

Base on the astronomical tide and the wind set up data above, there might be significant differences in design water levels between provinces from QuangNinh to ThanhHoa cause of the difference in tidal amplitude and storm surge level (see table 3.2).
Table 3.2 Design water level along the coastline Northern Vietnam.

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
<th>Thai Binh</th>
<th>Nam Ha</th>
<th>Ninh Binh</th>
<th>Thanh Hoa</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_t ) (m)</td>
<td>2.56</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
</tr>
<tr>
<td>( H_{rel} ) (m)</td>
<td>0.30</td>
<td>1.1</td>
<td>0.9</td>
<td>0.8</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>Design water level</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(m + MSL)</td>
<td>2.86</td>
<td>3.39</td>
<td>3.19</td>
<td>3.09</td>
<td>3.39</td>
<td>3.29</td>
</tr>
</tbody>
</table>

+ Wave run-up: Wave run-up is most important parameter used in calculating the dyke crest. In the present design, wave run-up 2% has been applied. There is no real physical justification of the number 2 but all of design criteria are based upon Ru2%. Normally, to predict wave run up in the design of sea dikes following two Russian formulas were mostly used:

\[
H_{sl} = (3.8 \times H_s) / m \times \cos \beta
\]

Where

- \( H_s \): local wave height.
- \( m \): Slope angle = \( \cot \alpha \)
- \( \beta \): Angle of wave front approaching the coast

In the formula above we see that no effect of the slope roughness is included in the formula. It is similar to the Dutch formula \( H_{sl} = 8 \times H_s \times \tan \alpha \) which is valid for \( \cot \alpha \geq 3 \) and relative smooth revetments. If compare two these formula, the numerical coefficient in Russian formula (=3.8) equal to a half of the coefficient in Dutch formula (=8), which has been proved to work satisfactorily for smooth slopes and wave steepness of about 4% (or 3% to 5%). That means the Russian formula should only be used for riprap slopes where run-up is reduced by factor 0.5. In case using this formula for revetments or pitched stone and 2% wave run-up the numerical coefficient of about 7.5 should be used (instead of 3.8).

The second Russian formula is

\[
H_{sl} = 2 \times K_n \times (h_s / m) \times (\lambda s / h_s)^{1/3}
\]

Where

- \( K_n \): Reduction coefficient due to roughness of a slope
- \( \lambda s \): Local wave length

This formula is including effect of slope roughness, the reduction coefficient and wave steepness \((\lambda s / h_s)\).

It can be rewritten to:

\[
H_{sl} = 2 \times K_n \times h_s \times \xi \times (h_s / \lambda_s)^{1/6} \quad \text{where} \quad \xi = \tan \alpha / (h_s / \lambda_s)^{1/2}
\]

The component \((h_s / \lambda_s)^{1/6}\) in the range of storm waves with \( 2 \leq h_s / \lambda_s \leq 5 \) which provides the numerical values equal to 0.52 to 0.60. Thus, \( H_{sl} = (1 \div 1.2) \times K_n \times h_s \times \xi \).
In the period 1990 to 1993 in the Netherlands, one formula for the calculation wave run up has been applied. 
\[ R = R_{2\%} \cdot \gamma_r \cdot \gamma_B \cdot \gamma_\beta \cdot \gamma_h \]
Where:
- \( R_{2\%} \): Run-up on smooth plane slopes, defines as the vertical height above still water level, which is exceeded by 2% of the waves in a wave field.
- \( \gamma_r \): Reduction factor due to slope roughness and permeability = \( K_n \)
- \( \gamma_B \): Reduction factor due to berm
- \( \gamma_\beta \): Reduction factor due to oblique wave attack
- \( \gamma_h \): Reduction factor due to shallow water.

For random waves \( R_{2\%} \) can be expressed by (Van Der Meer, 1993)
\[ \frac{R_{2\%}}{H_s} = 1.6 \cdot \xi_p \quad \text{for} \quad \xi_p \leq 2 \]
and \[ \frac{R_{2\%}}{H_s} = 3.2 \quad \text{for} \quad 2 < \xi_p < 4 \]
Where:
- \( H_s \): Significant wave height
- \( \xi_p \): Breaker parameter for the peak-period (research has indicated that run-up can better be described using the peak-period instead of using the mean period. Usually the peak period is 1.1 to 1.25 times the mean period and also the peak period is nearly equal to \( T_{1/3} \).

Thus \[ \frac{R_{2\%}}{H_s} = 1.6 \cdot \xi_p \cdot \gamma_r \cdot \gamma_B \cdot \gamma_\beta \]
Because of \[ R_{2\%} = 1.4 \cdot R_{13\%} \]
\( R_{13\%} \): Significant wave run-up = 13% exceedance.
\[ \xi = \frac{\tan \alpha}{\sqrt{H_s / L_s}} \]
So: \[ \frac{R_{2\%}}{H_s} = 1.6 \cdot \frac{\tan \alpha}{\sqrt{H_s / L_s}} \cdot \gamma_r \cdot \gamma_\beta \]
Take value: In storm the wave steepness is \( H_s / L_s = 0.03 - 0.05 \)
The revetment slope covered by riprap have: \( \gamma_r = 0.70 \)
So: \[ \frac{R_{2\%}}{H_s} = 1.6 \cdot \frac{\tan \alpha}{\sqrt{0.04}} \cdot 0.7 \cdot \gamma_\beta = 5.6 \cdot \tan \alpha \cdot \gamma_\beta \]
\[ R_{2\%} = 5.6 \cdot \frac{H_s}{\cot g \alpha} \cdot \gamma_\beta \]
\[ R_{13\%} = 4.0 \cdot \frac{H_s}{\cot g \alpha} \cdot \gamma_\beta \quad \text{compare with Russian formula:} \quad H_{sl} = 3.8 \cdot \frac{H_s}{\cot g \alpha} \cdot \cos \beta \]
We see that in the present design Russian formulas as well as Netherlands formula are used. It point out that the run-up on dike will be strongly influenced by the slope angle (\(\cot \alpha\)) and the roughness and permeability of slope. (reduction factor \(\gamma_r\)). Instance incase the slope covering by grass (3cm) and rough/permeable pitched stones have the reduction factor 0.90 to 1.00 but incase slope covering by 2 layers of rock (\(H_s/D = 1.5\) to 6), the reduction factor only 0.50 to 0.55.

In one hand, the wave run-up reduction factor will be increase if the slope of revetment more height roughness and more permeability. Also, if the slope milder (\(\cot \alpha\) increase) the wave run-up will be decrease. In another hand, when the slope milder that mean the slope of revetment more stability so material amour layer can be reduce in the size and thickness. In the guidelines 14 TCN, 2002 suggest that in the upper part of the slope, at location DWL+1/2\(H_s\) the pitched rock can be applied. That why in most of revetments at the Northern Vietnam pitched rock has been applied at elevation MSL+3.50.

**Table 3.3 Summarized results of wave run-up calculations (Russian formula No1).**

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
<th>Thai Binh</th>
<th>Nam Ha</th>
<th>Ninh Binh</th>
<th>Thanh Hoa</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_{sl}) (m)</td>
<td>1.77</td>
<td>1.82</td>
<td>1.57</td>
<td>3.00</td>
<td>1.5</td>
<td>2.00</td>
</tr>
<tr>
<td>Outer slope</td>
<td>1:4</td>
<td>1:3 (\div) 1:4</td>
<td>1:4</td>
<td>1:4</td>
<td>1:4</td>
<td>1:4</td>
</tr>
<tr>
<td>(H_{s}) (m)</td>
<td>2.18</td>
<td>1.82</td>
<td>1.57</td>
<td>1.17</td>
<td>1.56</td>
<td>2.14</td>
</tr>
</tbody>
</table>

Apply CRESS module 241 – *Calculation of wave run-up along a sloping structure* to check the result above. Choose the case study input with \(H_s = 1.50\)m, \(T_p = 8\) seconds, revetment slope \(\cot \alpha = 4\). Water depth at front \(h = 3\)m. (in case rip-rap slope)
We see that the results from CRESS program with wave run-up $R_{2\%} = 1.60\text{m}$ and Russian formula (table 3.3) results approximately the same. That mean applied the Netherlands formula as well as Russian formula.

In practical, the wave run-up at revetment has to calculate with pitched rocks that mean in case the smooth slope revetment.

Calculate with pitched rocks. Roughness coefficient $f = 0.9$ to $1.0$

+ **Free board:** In practical design, the settlement of the upper layer and dikes foundation is caused by compression of sub-layers due to the initial weight of the dike. The settlement normally extend in a long time, it may vary considerably over the route of the dike.

In Vietnam, the free board value 0.2m to 0.3m has been applied. This value is based on local experience because lack of geotechnical data.

Beside it, the value for the settlement of subsoil was not improved in design dikes so it is not clear whether the foundation of the dike sections was stable enough to assure the stability of the dike.
Individual Study - Analysis of failure situation for sea dike in Northern Vietnam

**Results:** Based on the calculation water level, wave run-up and free board above, the results of calculations the crest of dikes are summarized follows:

*Table 3.4 Summarized results of crest height calculations (Vietnam code).*

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
<th>Thai Binh</th>
<th>Nam Ha</th>
<th>Ninh Binh</th>
<th>Thanh Hoa</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_t$ (m)</td>
<td>2.56</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
<td>2.29</td>
</tr>
<tr>
<td>$H_{rd}$ (m)</td>
<td>0.30</td>
<td>1.1</td>
<td>0.9</td>
<td>0.8</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>DWL (m + MSL)</td>
<td>2.86</td>
<td>3.39</td>
<td>3.19</td>
<td>3.09</td>
<td>3.39</td>
<td>3.29</td>
</tr>
<tr>
<td>$H_{sl}$ (m)</td>
<td>1.77</td>
<td>1.82</td>
<td>1.57</td>
<td>3.00</td>
<td>1.50</td>
<td>2.00</td>
</tr>
<tr>
<td>$H_{sl}$ (m + MSL)</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.20</td>
<td>0.30</td>
<td>0.21</td>
</tr>
<tr>
<td>$H_{zd}$ (m + MSL)</td>
<td>4.94</td>
<td>5.51</td>
<td>5.06</td>
<td>4.46</td>
<td>5.20</td>
<td>5.50</td>
</tr>
<tr>
<td><strong>Round</strong></td>
<td><strong>5.00</strong></td>
<td><strong>5.50</strong></td>
<td><strong>5.00</strong></td>
<td><strong>4.50</strong></td>
<td><strong>5.20</strong></td>
<td><strong>5.50</strong></td>
</tr>
<tr>
<td>Outer slope</td>
<td>1:4</td>
<td>1:3 1:4</td>
<td>1:4</td>
<td>1:4</td>
<td>1:4</td>
<td>1:4</td>
</tr>
<tr>
<td>$H_s$ (m)</td>
<td>2.18</td>
<td>1.82</td>
<td>1.57</td>
<td>1.17</td>
<td>1.56</td>
<td>2.14</td>
</tr>
</tbody>
</table>

Result of crest height calculations by Dutch code:

Crest height = Design water level + Wave run-up + Free board

+ In case riprap slope revetment.
  
  \[ H_{\text{crest}} = 3.39 + 1.60 + 0.30 = 5.29 \text{m} \approx 5.30 \text{m} \]

+ In case pitched slope or interlocking blocks revetment.
  
  \[ H_{\text{crest}} = 3.39 + 2.30 + 0.30 = 5.99 \text{m} \approx 6.00 \text{m} \]

**Conclusions:** Compare the results of crest height calculations by Vietnam code and Dutch code; it is clearly that the average difference of dike’s crest level is 0.3m to 1.0m. It is obviously that by lower design crest level, the dikes could be more damages due to overtopping during extreme conditions. The design guideline 14TCN-2002 suggests choose the crest height for dikes at the North of Vietnam is MSL+ 5.50.

**Overtopping:** In dikes system at the North of Vietnam, because of purpose of dikes is protect for the flood area in Red River Delta so that in design criteria the wave overtopping is not larger than $Q = 1 \text{l/s}$.

Apply *CRESS module 242 – Calculation of overtopping over a dike* with the same case study above to check overtopping discharge.

Choose the case study input with:

+ $H_s = 1.50 \text{m}$,
+ $T_p = 8 \text{ seconds}$,
+ Revetment slope $\cot \alpha = 4$.
+ Water depth at front $h = 3 \text{m}$.
+ Crest free board = $5.5 - 3 = 2.5 \text{m}$.
We see that with the value of overflow \( Q = 0.31 \) liters/s, the dike's crest height \((\text{MSL} + 5.50)\) can be accepted in design to protect the low land behind.

### 3.2.2 Design of revetment

The revetment is to prevent erosion of dike front due to currents, waves action or both.

In general, the revetment has been designed to include 3 layers: Armour layer; gravel layer and geotextile.

+ The most important safety criteria for the revetment is the stability of the armour layer. However, most of the design for that were still applied by the old method of the year 60. The comparison of previous design and Dutch design will be studied in section 4.2 on Chapter 4.

+ Armour layer is interlocking concrete blocks or rocks. However, in Vietnam it is very difficult to find the big rocks, which have the diameter larger than 30cm cause of the rocks has been exploited by explode. Thus, in Vietnam the suitable rocks diameter choose for design revetments very limited.

+ Next to the armour layer is gravel layer diameter 1cm \(\leq 2\) cm, some case 2cm \(\leq 4\) cm base on local gravel resources, thickness 10cm to 15cm with purposed to easy when place the blocks on revetments slope and also work as a filter layer.

+ In the past, the filter is granular filter applied but that kind of filter include initial weight of filter material like gravel, coarse sand .etc so it make more weight place on revetment slope that mean the stability of slope will be reduce. Recently year, using geotextile has been popular in design filter layer in Vietnam as TS40; TS70.etc but using geotextile filter required the accuracy core diameter which less data in Vietnam so most of case the designer choose geotextile type by experience and apply resources.
3.2.3 Comments

In general, the design of dikes and revetments are mostly based on experience instead of the generally accepted and validate calculation methods. The actual level of knowledge on design of dikes and revetments is rather poor. In addition to this, the “in put” data for the calculation is an insufficiently. Therefore, the results in some cases are inconvincible.

For example:

+ A free board in the design of 20 to 30cm is used. But it is not clear why these values are applied.

+ Base on the design guidelines, the calculation of crest height is considered of the design tidal water level ($H_t$) corresponding to probability of $p = 5\%$, the height of storm surge $H_{ns}$ of $p = 20\%$.

+ In cases that the tidal water level and storm surge exceeding the design probability overtopping of the dikes will take place. No measure for protecting crest and inner slope is considered in the designs.

+ The dikes have a various exposition in relation to the dominant wind direction. But only one direction is considered for each dike line

+ In the past, the calculation of wave run-up and the dimensions of protection layer base on Russian formulas as well as the Hudson formula were used. The values of wave run-up are less than several percentages in comparison with the Netherlands formula. Where these formulas are based on the weight of elements and originally developed for riprap and rubble structures. New knowledge shows that the design of block revetments should be based on the thickness of a block and not on the weight of a block design transitions from the slope protection into the toe protection is not a proper lead to subsoil from slope through to the toe of structures.
+ The wave run-up on the dikes will be strongly influenced by the slope angle and the roughness and permeability of a revetment. In the design there is no distinction for the various types of revetment.

+ In design revetment, the geotextile is commonly used. For wave height higher than 1m the deformation of the slope should may be occurred due to local liquefaction or the migration of the subsoil along the slope.

3.3 MAINTENANCE

Maintenance works for the dikes and revetments need to carry out periodically after construction to ensure the structure can perform its design functions at an acceptable level during the service life. The maintenance program, in principle, composes of two main elements/steps are:

+ Inspection and monitoring of structural state and changes in hydraulic conditions
+ Repair or replacement of less acceptable structural elements.

The sea dikes in the Northern part of Vietnam are degraded due to a poor maintenance. After each storm season, the dikes are repaired with a large number of government funds instead of small damage, which must be directly repaired by the local authorities and people. The unprepared locations can lead to the serious damages during the next storm in the same season even wave climate, which can be very costly to repair. Among the season, the annual budget is always limited for such operations as well as a poor management base.

The relevant data of sea dikes and the boundary conditions can change due to geotechnical aspects and the morphological change has not yet been reported periodically in a database.
3.4 CONSTRUCTIONS

The sea dikes in the North of Vietnam commonly degraded due to construction aspects which can be explained simply as follows:

+ Specifications for dikes and revetment construction should not be applied or prepared.
+ Almost cases, the execution is done by unskilled and inexperienced voluntary labors.
+ Supervision during the construction is lack, in addition, a poor quality control system lead to insufficient aspects as:
  - Dimensions of the dike (height and width of crest, slope)
  - Compaction of the earth fill
  - Dimensions of the revetment (top layer, filter layers, stone or concrete units. etc)
  - Quality and quantity of materials (rock, concrete, geotextiles)
  - The toe and transition construction.

Most of dikes and revetments are mainly constructed by hand based on local experiences and directly affected by actions of waves, tide and currents.

![Figure 3.4 Construction revetments in Haihau - Namdinh](image)

3.5 DESIGN CODE

A new design guideline 14TCN-130-2002, for coastal protection in Vietnam is just finished. This is including the selection of design formulas for various revetment types. However, it may not be updated according with the actual international knowledge in this field. This code is not including sufficient aspects on construction and maintenance. It also seems to be difficult for the application in design.
CHAPTER 4. ANALYSIS OF TYPICAL FAILURES

4.1 DIKES IN FRONT OF ERODED COASTLINE

Dikes and revetments prevent further erosion of the coastline behind them but do not stop the physical processes, which cause the erosion. Therefore, it does not stop erosion of the inshore zone and even probably increases erosion in front of a dike can lead to the undermining of the dike. Thus, toe protection is needed to prevent this.

On the other hand, to minimize effect of scouring due to the reflection of waves, the slope of the sea dike is usually choose as \( m = 4 \) or even milder.

Practice in Namdinh province show that the beach in front of Covay and Haitrieu dike lines is scoured 0.2m to 0.3m after 4 years upgrading of dikes and revetments. Therefore, the need for proper studies before the decision to construct a new system of groins or upgrading of to protection is taken.

4.2 STABILITY OF STRUCTURES

4.2.1 Sub-soil settlement consideration

The upper soil layers are settled down due to the weight of dike structure. This story is concerned with geotechnical aspects. To predict accurately soil settlement is not of difficulty with the help of computational models since sufficient geotechnical data is provided.

However, due to the lack of information as mentioned in previous. One can only roughly estimate something, which are based on certain limited conditions.

The sandy subsoil in general has a very quick creep and the final settlement is considerably small with respect to another types of subsoil. For a long lasted sandy foundation, the settlement is negligible.

In the locations where considerable settlements have been found in short duration of time, special attention needs to pay to geotechnical survey. The consideration of settlement in this case should be done by specialists with computational models.

Apart from that, some characteristics of clayey or mixed clayey subsoil need to be considered. The creep process of clay subsoil happens slowly, it takes years or even decades to reach final settlement. For the design lifetime of the particularly considered structures (10 - 20 years) and for good quality of clay subsoil, a settlement of 0.20 to 0.30 m is expected.
4.2.2 Stability of rocks and interlocking concrete blocks

In Vietnam, using rocks and concrete blocks in construct sea dikes and revetment become very popular. Rocks has been use in riprap toe and also paved in run-up zone of outer slope. In waves action zone, the interlocking concrete blocks has been applied to protect waves and current actions. Thus, the required size and weight of the armour material is most important input needed to calculate when design a revetment.

From the general point of view the resistance of the protection is deal with friction, cohesion, weight of the units, friction between the units, interlocking and mechanical strength. As a result of the difference of strength properties, critical loading conditions are also difference.

In the Red River Delta, the revetment slope is divided into 3 categories:

+ Natural material (grass)
+ Protected by loose units (riprap)
+ Protected by interlocking units (concrete blocks, pitched stones)

There are some formulas has been applied to calculates stability, weight and thickness of amour material in revetments is:

+ Hudson in the Shore Protection Manual [CERC, 1984].
+ Van Der Meer formula
+ Pilarczyk formula.

For the grass dikes and riprap, maximum velocities will cause displacement of the material. Critical loading conditions vary both with respect to the position along the slope and the time during the passage of a wave. Instability for grass and riprap will occurs around the water level, when in the area just below the still water level.

Instability paved revetment without to much interlocking occurs near the point of maximum downrush, where uplift forces are higher, just before the arrival of the next wave front. Instability will occurs due to the combined effect of uplift and impact forces just after wave breaking. That mean in case the forces lift larger then the initial weight of concrete block, the interlocking will be fail, the soil below amour layer will be deformation and mitigation. Result is the revetment will be fail. Show in figure 4.1 and figure 4.2; 4.3.

![Figure 4.1 Deformation of interlocking blocks if the lift force >> block gravity](image-url)
Figure 4.2 Forces on top-layer during a wave cycle

Where:

- **a**: Forces due to back-wash
- **b**: Quasi-stationary pressures due to wave set-up
- **c**: Pressures due to wave front
- **d**: Velocity-field in wave
- **e**: Wave shock (impact)
- **f**: Pressures due to wave breaking
- **g**: Low pressure due to air in water
- **h**: Forces due to up-rush
- **i**: Gradient perpendicular to the slope
- **j**: Gradient parallel to the slope

Figure 4.3 Critical situations for blocks during wave cycle
4.2.2.1. Applied Methods.

* Hudson formula:
The original Hudson formula is written as:
\[
M_{50} = \frac{\rho_s H_s^3}{K_D \Delta^3 \cot g \alpha}
\]
Where:
- \(H_s\): Design wave height (m)
- \(\rho_s\): Density of rocks (2650 kg/m\(^3\))
- \(\Delta\): Relative mass density (\(\Delta = \rho_r/\rho_w - 1\))
- \(\alpha\): Angle of revetment slope.

\(K_D\) is a stability coefficient taking into account all other variables. \(K_D\) values suggested for design correspond to a “no damage” condition where up to 5% of the armour units may be displaced.

- \(K_D = 3.5\) for breaking waves
- \(K_D = 4.5\) for non-breaking waves

In case the interlocking concrete blocks, \(K_D\) coefficient takes value 5.8 (based on the shape, roughness, and the kind of interlocking concrete blocks)

* Van Der Meer formula:
Based on earlier work of Thompson and Shuttler (1975) an extensive series of model tests was conducted at Delft Hydraulics (Van Der Meer). These include structures with a wide range of core/under layer permeabilities and a wider range of wave conditions. Two formulas were derived for plunging and surging waves, respectively, which are now know as the Van Der Meer formulas:

For plunging waves: \(\xi \leq 2\) to 3
\[
\frac{H_s}{\Delta D_{\xi=50}} = 6.2 \times P^{0.18} \times \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5}
\]

For surging waves: \(\xi \geq 3\)
\[
\frac{H_s}{\Delta D_{\xi=50}} = 1.0 \times P^{-0.13} \times \left( \frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot g \alpha} \times \xi_m^P
\]

The value of factor \(P\) should be lie between 0.1 and 0.6

- \(P = 0.1\) to 0.2 if revetment (riprap on filter layer and impermeable subsoil, sand, clay)
- \(P = 0.4\) if rubble structure with fine coarser core
- \(P = 0.6\) if rubble structure with homogeneous stone.

The value \(S\) takes:
- \(S = 2\) to 2.5 if \(\cot g \alpha \leq 2\)
- \(S = 3\) if \(\cot g \alpha \geq 3\)
*Pilarczyk formula:*

Pilarczyk formula has been applied to calculate the size of rock and blocks revetments.

\[
\frac{H_s}{\Delta D_{50}} = \phi \frac{\cos \alpha}{\xi^b}
\]

- if \( \xi < 3 \)
- if \( \xi > 3 \) take value as calculate for \( \xi = 3 \)
- if \( \cotg \alpha \geq 3 \) so \( \cos \alpha = 1 \)

Where:

- \( H_s \): Design wave height (m)
- \( \rho_S \): Density of rocks (2650 kg/m\(^3\))
- \( \Delta \): Relative mass density (\( \Delta = \rho_r/\rho_w - 1 \))
- \( \alpha \): Angle of revetment slope.
- \( \phi \): Stability factor

Take value:

- \( b = 0.5 \) for riprap
- \( b = 2/3 \) for block revetments
- \( \phi = 3 \) for pitched nature stone
- \( \phi = 4 \) to \( 4.5 \) for placed block
- \( \phi = 5 \) for placed block on geotextile and good smooth surface clay
- \( \phi = 6 \) for pitched Basalton washed in by coarse material of interlock blocks
- \( \phi = 8 \) for interlock blocks on properly designed sub-layers and subsoil

4.2.2.2. Calculation.

In case calculate the rock and blocks size with Hudson formula and Vander Meer formula, the CRESS program routine 511 and 512 has been used.

Case study at \( H_s = 1.50 \text{m}, T_p = 8 \text{ seconds}, \) revetment slope \( \cotg \alpha = 4 \). Water depth at front \( h = 3 \text{m} \).

a) Hudson formula.

+ For rocks:
+ For concrete blocks:

For concrete blocks:

b) Van der Meer formula.

+ For rocks:

For Concrete blocks:
c) Pilarczyk formula.
+ Calculate with pitched stone. \( \varphi = 3 \);
\[
\frac{H_s}{\Delta D_{n50}} = \varphi \cdot \frac{\cos \alpha}{\xi^2}
\]
and.
\[
\xi = \tan \alpha = \frac{1/4}{\frac{H_s}{L_s} \sqrt{\frac{1.5}{99.84}}} = 2.04 \quad \Rightarrow \quad \xi^{2/3} = 1.61
\]
\[
D_{n50} = \frac{H_s}{\varphi \Delta} \xi^{2/3} = \frac{1.5}{3 \cdot 1.59} \cdot 1.61 = 0.51 \text{ (m)}
\]
\[
W_{\text{rock}} = 0.51 \cdot 3 \cdot 2650 = 350 \text{ (kg)}
\]
+ Calculate with stone. \( \varphi = 2.25 \)
\[
D_{n50} = \frac{H_s}{\varphi \Delta} \xi^{2/3} = \frac{1.5}{2.25 \cdot 1.59} \cdot 1.61 = 0.68 \text{ (m)}
\]
\[
W_{\text{rock}} = 0.68 \cdot 3 \cdot 2650 = 833 \text{ (kg)}
\]
+ Calculate with concrete interlock block. \( \varphi = 6 \)
\[
D_{n50} = \frac{H_s}{\varphi \Delta} \xi^{2/3} = \frac{1.5}{6 \cdot 1.34} \cdot 1.61 = 0.30 \text{ (m)}
\]
\[
W_{\text{block}} (0.4*0.4*0.30) \text{ cm} = 0.4*0.4*0.30*2400 = 115 \text{ (kg)}
\]
+ Calculate with concrete block column shape (no interlocking). \( \varphi = 4 \) to 4.5
\[
D_{n50} = \frac{H_s}{\varphi \Delta} \xi^{2/3} = \frac{1.5}{4.0 \cdot 1.34} \cdot 1.61 = 0.45 \text{ (m)}
\]
\[
W_{\text{block}} (0.25*0.25*0.45) \text{ cm} = 0.25*0.25*0.45*2400 = 67.5 \text{ (kg)} \approx 68 \text{kg}
\]

4.2.2.3. Results.

<table>
<thead>
<tr>
<th>Method</th>
<th>Hudson</th>
<th>Van Der Meer</th>
<th>Pilarczyk</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dimensions</strong></td>
<td>( D_{n50} ) (m)</td>
<td>( W_{50} ) (kg)</td>
<td>( D_{n50} ) (m)</td>
</tr>
<tr>
<td><strong>Rocks</strong></td>
<td>0.45</td>
<td>147</td>
<td>0.40</td>
</tr>
<tr>
<td><strong>Interlock blocks</strong></td>
<td>0.49</td>
<td>170</td>
<td>0.48</td>
</tr>
</tbody>
</table>

4.2.2.4. Calculate the filter layer.

In the past, the granual filter has been applied in most of old revetment at the Northern Vietnam. Normally, granual filter is layers gravel, coarse sand, fines sand. One hand, it work as a filter to prevent the washing away of the material to be protected. That means that the underlying grains (base layer) should not pass the pores of the upper layer (filter layer) and the other hand is set up a flat slope for place the rocks or concrete blocks more easy. Thus,
the size and thickness of filter layer is very important parameter to take into account stability of dikes.

According to lecture note in technical aspect – De Groot, there are some criteria to choose the filter layer:

- Stability condition: \( D_{15f}/D_{85b} < 5 \)
- Permeability condition: \( D_{15f}/D_{15b} > 1 \)
- Internal stability condition: \( C_u = D_{60}/D_{10} \leq 10 \)

Where:

- \( D_f \): Grain size of the filter layer
- \( D_b \): Grain size of the basic layer

Recently, the geotextile has been used to reduce the thickness of filter layers, useful and also more economical in large project.

Because of under the under layer, the geotextile has been placed so that the filter layer now work as main functions to prevent damaging the geotextile by place directly individual large stones and to prevent flapping of the cloth, possibly causing looses of the base material. Thus, the under layer between armor layer and geotextile can be reducing the size and thickness.

In case the armor layer is interlocking blocks, the maximum gap between each blocks have to smaller than 1cm. That mean the dimension of filter layer has to \( \geq 1 \text{cm} \) to prevent the washing away of under layer thought the pore of armour layer under action of waves and current. In Vietnam, one layer of gravel \( D_{n50} = 1.5 \text{ to } 2.0 \text{cm}, \text{ thickness } T_u = 10 \text{ to } 15 \text{cm} \) has applied in to design.

**4.2.2.4. Conclusions and recommendations**

Based on calculation dimensions of rocks and concrete blocks above. We see that in Hudson formula and Van Der Meer formula the rocks and blocks have been calculated in case it is locate individual. In reality, at the revetment, the rocks are pitched together. Thus, the diameter and the weight of rocks can be reduced.

*Figure 4.4 The paved rocks revetments in Haihau - Namdinh*
In other side, the popular rocks in Vietnam have been using to construct revetment have $D_{50} = 30\text{cm}$, the weight of rocks $W_R$ approximately 72kg (Design guideline - 14TCN-130-2002) cause of the rocks has been exploited by explode.

Compare the dimensions of available rocks with the results in table 4.1 we see that the $D_{50}$ of rocks smaller the $D_{50}$ required 0.1m to 0.2m. So that the rocks at armour layer of revetment could be more damages due to the wave attack during extreme conditions.

Apply CRESS programs routine 513 - Calculation of revetments under wave attack to check.

![513 Calculation of revetments under wave attack](image)

We see that the rocks $D_{50} = 30\text{cm}$, $W_R = 72\text{kg}$ can be applied in design revetments.

The weight of concrete blocks has been choosing correspond to construction conditions which mainly by hand. (See figure 3.4). The Pilarczyk’s formula to calculate the dimension of concrete block has been applied widely in design interlocking revetments. In this formula the factor $\varphi$ has taken in to account as the interlocking coefficient of blocks.

In “Sea dikes Northern part of Vietnam - 1996”, Mr. Pilarczyk suggest to use the concrete blocks with the column shape. In this case, the height of concrete blocks has to calculate to safety with the uplift forces but the surface area of blocks can be reduced. That mean the weight of individual block can be reduce to suitable for the construction conditions in Vietnam. Thus, to design armour revetments material, the designer of has to consider in calculation data, constructions conditions and funds. Detail see figure below.
Individual Study - Analysis of failure situation for sea dike in Northern Vietnam

**Table 4.2** Some dimensions of amour layers has been use popular in Northern Vietnam.

<table>
<thead>
<tr>
<th>Material</th>
<th>Dimensions (m)</th>
<th>Weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rocks</td>
<td>$D_{50} = 0.3$ m</td>
<td>72 kg</td>
</tr>
<tr>
<td>Interlocking concrete blocks</td>
<td>$(0.4<em>0.4</em>0.28)$ m</td>
<td>108 kg</td>
</tr>
<tr>
<td>TSC 178 blocks</td>
<td>$(0.25<em>0.25</em>0.25)$ m</td>
<td>38 kg</td>
</tr>
</tbody>
</table>

**Table 4.3** Summarized results of calculations protection layers.

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
<th>Thai Binh</th>
<th>Nam Ha</th>
<th>Ninh Binh</th>
<th>Thanh Hoa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pitched stone ($d_{50}$ in m)</td>
<td>0.45</td>
<td>0.35</td>
<td>0.40</td>
<td>0.50</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Gravel ($d_{50}$ in m)</td>
<td>0.2</td>
<td>0.25</td>
<td>0.37</td>
<td>0.15</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Round</td>
<td>0.15</td>
<td>0.05</td>
<td>0.12</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Coarse sand ($d_{50}$ in m)</td>
<td>0.15</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Concrete plate ($d_{50}$ in m)</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Using Pilarczyk formula to check the stability in of concrete blocks in table 4.2 with the wave parameters: $H_s = 1.50$ m, $T_p = 8$ seconds, $h = 3$ m.

Calculate with interlocking concrete block. $\varphi = 6$

+ Check with interlocking concrete blocks $(0.4*0.4*0.28)$ cm

$$D_{n50} = \frac{H_s}{\varphi * \Delta} \Rightarrow H_s = \frac{D_{n50} * \varphi * \Delta}{\varepsilon^{2/3}} (m)$$

$$H_s = \frac{D_{n50} * \varphi * \Delta}{\varepsilon^{2/3}} = \frac{0.28 * 6 * 1.34}{1.61} = 1.40 (m)$$

+ Check with TSC178 concrete blocks $(0.25*0.25*0.25)$ cm

$$H_s = \frac{D_{n50} * \varphi * \Delta}{\varepsilon^{2/3}} = \frac{0.25 * 6 * 1.34}{1.61} = 1.25 (m)$$
We see that in both of interlocking concrete blocks, which is applied in Vietnam only suitable with the significant wave height at 1.25m and 1.40m. That mean, in case the design wave height at 1.50m, it will be instability.

4.2.3 Stability of slope
From the geotechnical point of view the failure mechanisms can in general be related to geotechnical limit state conditions. Base on the geotechnical data, the stability of slope has been calculated by Bishop method.

For the sea dikes, the mechanisms are:
+ Informal erosion: The finer soil particles of a layer are washed out through the pores of the coarser grains of the same layer
+ Slope instability: It happens if the actual shear stress along a potential failure plane exceeds the shearing resistance along that plane
+ Local instability in the soil below the revetment due to wave forces. This may take place suddenly as a result of an exceptional heavy wave attack or more gradually due to cyclic loading of many less rate waves
+ A large soil mass deforms and displaces as a whole in vertical direction due to large horizontal deformation in the lower very soft layer
+ Sand slide due to liquefaction may take place in loose sand

* Apply Geo-slope Program to calculate the stability of dikes slope.
The SLOPE/W program (Canada) calculate stability factor by Bishop method, the water pore pressure has been take from SEEP/W program.
Choose case study:
+ Inner slope: The water level at sea side is normal, the phreatic line stable
+ Outer slope: Water level at sea side reduces fast in short time.

1. Method. In SLOPE/W program, the critical equilibrium condition of forces and moments has take into account stability factor of slope.
* Steps:
  + Choose one slip center and radius slip surface
  + Divide the cylinder slip to many slides; calculate the acting forces and moments in each slide.
  + Take value of soil at upper part of phreatic line is nature density $\gamma_n$
  + Take value of soil at lower part of phreatic line is saturated density $\gamma_{bh}$
  + Calculate with many slip center and radius slip surface
  + Compare the results to find out the $K_{min}$ is stability factor.
Where:

- **W:** Gravity of soil slide which have the width \( b \) and height \( h \)
- **N:** Normal force at slide bottom
- **S:** Friction force at slide bottom
- **E:** Horizontal action force between slides
- **X:** Vertical action force between slides
- **L; R:** Predict the left side and right side of slide
- **D:** External force
- **KW:** Horizontal force at center of slide
- **R:** Slide slip radius
- **x:** Horizontal distance from slip center to slide center
- **e:** Vertical distance from slip center to slide center
- **d:** Distance from slip center to external force
- **u:** Pore water pressure

+ Value of force \( S_m \) that satisfies critical equilibrium conditions is:

\[
S_m = \frac{S \cdot \beta}{F} = \beta \left[ c' + (\sigma_n - u) \tan \varphi \right]
\]

\[
\sigma_n = \frac{N}{\beta} \quad \text{is average value of normal force at slide surface.}
\]

\[
F: \quad \text{Stability factor.}
\]

+ Value of normal force \( N \):

\[
N = \frac{W + (X_R - X_L) \cdot c' \beta \sin \alpha + u \beta \sin \alpha \tan \varphi \cdot (D \sin \omega)}{\cos \alpha + \sin \alpha \tan \varphi} + \frac{(D \sin \omega)}{F}
\]
Total moments of whole slide to slip center.
\[ \sum W_X - \sum S_m R - \sum kW_e \pm (Dd) \pm Aa = 0 \]

\[ F_m = \frac{\sum (c' \beta R + (N - u\beta)R \tan \phi')}{\sum W_X - \sum Nf + \sum kW_e \pm (Dd) \pm Aa} \]

Total of whole horizontal forces.
\[ \sum (E_L - E_R) - \sum (N \sin \alpha) + \sum (S_m \cos \alpha) - \sum (kW) \]
\[ \sum (E_L - E_R) = 0 \quad \text{total action forces at cylinder slip has to annul} \]

Thus: \[ F_f = \frac{\sum [c' \cos \alpha + (N - u\beta) \tan \phi' \cos \alpha]}{\sum N \sin \alpha + \sum kW - (D \cos \omega) \pm A} \]

2. Calculation and results.

+ Input data:

Table 4.4 Input data to calculations stability factor by Bishop method.

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma )</td>
<td>( \gamma )</td>
</tr>
<tr>
<td></td>
<td>(T/m(^3))</td>
<td>(T/m(^3))</td>
</tr>
<tr>
<td>Layer 1</td>
<td>1.85</td>
<td>1.40</td>
</tr>
<tr>
<td>Layer 2</td>
<td>1.82</td>
<td>1.42</td>
</tr>
<tr>
<td>Layer 3</td>
<td>1.71</td>
<td>1.20</td>
</tr>
<tr>
<td>Layer 4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Province</th>
<th>Thai Binh</th>
<th>Nam Dinh</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma )</td>
<td>( \gamma )</td>
</tr>
<tr>
<td></td>
<td>(T/m(^3))</td>
<td>(T/m(^3))</td>
</tr>
<tr>
<td>Layer 1</td>
<td>1.94</td>
<td>1.56</td>
</tr>
<tr>
<td>Layer 2</td>
<td>1.83</td>
<td>1.29</td>
</tr>
<tr>
<td>Layer 3</td>
<td>1.85</td>
<td>1.43</td>
</tr>
<tr>
<td>Layer 4</td>
<td>1.76</td>
<td>1.25</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Province</th>
<th>Thanh Hoa</th>
<th>Earthfill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma )</td>
<td>( \gamma )</td>
</tr>
<tr>
<td></td>
<td>(T/m(^3))</td>
<td>(T/m(^3))</td>
</tr>
<tr>
<td>Layer 1</td>
<td>1.71</td>
<td>1.36</td>
</tr>
<tr>
<td>Layer 2</td>
<td>1.77</td>
<td>1.35</td>
</tr>
<tr>
<td>Layer 3</td>
<td>1.88</td>
<td>1.52</td>
</tr>
</tbody>
</table>
Output data:

Table 4.5 Summarized results of calculations stability factor by Bishop method.

<table>
<thead>
<tr>
<th>Province</th>
<th>Quang Ninh</th>
<th>Hai Phong</th>
<th>Thai Binh</th>
<th>Nam Dinh</th>
<th>Thanh Hoa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case</td>
<td>1 2</td>
<td>1 2</td>
<td>1 2</td>
<td>1 2</td>
<td>1 2</td>
</tr>
<tr>
<td>Stability factor</td>
<td>1.30 1.21</td>
<td>1.25 1.20</td>
<td>1.32 1.24</td>
<td>1.28 1.22</td>
<td>1.23 1.20</td>
</tr>
</tbody>
</table>

3. Conclusions.
Base on the geotechnical data, which collected from 6 provinces along the North coastal of Vietnam, the program SLOPE/W has been applied to calculate the stability factor. In the table results, most of the stability factor of slope have value $\geq K_{critical} = 1.20$ (14TCN,130-2002-MARD) so that the slope of dikes stable in both case when the water at sea side reduce fast in a short time and in case normal conditions. (see Appendix 2)

4.2.4 Stability of toe
The toe is supports the slope of revetment and prevents waves from scouring and keep stability of initial revetment weight. Thus, stability is essential because failure of the toe will lead to failure though out the entire dikes and revetments. The maximum scour occurs where wave down rush on the structure face extends to the toe or the wave is breaking near the toe. These conditions may take place when the water depth at the toe is less than twice the height of the maximum expected unbroken wave.

It is assumption that the maximum depth of a scour trough due to wave action below the natural bed is about equal to the maximum expected wave at the toe. A trench must be excavated with a depth of the expected scour. An apron must be placed on the existing bottom with its width should not be less than twice the local wave height. The minimum thickness of cover-layer over the toe apron should be two quarry stones. Geotextile is also used as a secondary layer to prevent undermining.

* Statistic data on toe of sea dikes in the North of Vietnam

<table>
<thead>
<tr>
<th>Description</th>
<th>Sketch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name of revetment: Debe 1 and 2</td>
<td></td>
</tr>
<tr>
<td>Location: Doson - Haiphong ThaiThuy - ThaiBinh</td>
<td>Concrete block (0.2x0.4x0.8)m</td>
</tr>
<tr>
<td>Form of structure: Concrete block</td>
<td>3.2m</td>
</tr>
<tr>
<td>Stone</td>
<td></td>
</tr>
<tr>
<td>Situation: Normal</td>
<td>2m</td>
</tr>
<tr>
<td>Volume ratio: 7.8%</td>
<td></td>
</tr>
<tr>
<td>(Compare to total of revetment)</td>
<td></td>
</tr>
</tbody>
</table>
The required size of rock used for the toe, in conservative way is equivalent to the rock size of the revetment. However, it is much more acceptable if one reduces this size accordingly to each distinct situation of hydraulic impacts. It was concluded that apart from the wave impact the local water depth above the toe is an important parameter in determination of the toe stability. In case the toe close to the bottom, the required stone size might be as twice as smaller than that toe in case situated about half distance between the still water level and the bottom.

Apply CRESS program routine 531-Calculation of riprap toe protection to check the rocks diameters at toe of the revetments.

According to Ha (MSC.Thesis H.E.134, 2003), using physical model test (MatLap) in regular wave flume of the Laboratory of Fluid Mechanics of Delft University of Technology to determine the cause of damages at toe of sea dikes and Revetments have some result:

Waves will be partly reflected when obstructed by a slope. The reflected wave, which is manifested by reflection coefficient, depends on angle of the slope, water depth and incoming wave. The combination of incoming and reflected wave will depend on the slope angle as well as the water depth. In shallow water condition the wave-induced kinetic energy is very high and distributed over the water depth. More over, there are horizontal and vertical net flows in the area in front of the slope. In general, the horizontal net flows do
not only have seaward direction, but also landward direction. The vertical net flows were found to have upward direction, however sometimes downward direction. Finally, near the toe, the relative turbulent energy intensity is rather high.

The wave-induced kinetic energy and the net flows will affect the bottom in front of sea dikes. Due to the fact that the grain size of the bottom (sand seabed) is too small compared to the wave-induced kinetic energy positions, the sand will be stirred up and then moved by the net flows. Moreover, the water level changes during a storm causing the positions change. As a result, the bottom will be totally affected and lowered. Although the bottom is lowered, the wave-induced kinetic energy is still rather high and the erosion still continues. This causes the concrete cylinders of the toe used as toe protection in the sea dikes and revetment in Vietnam to lose stability and move. Afterward, the slope (revetment) loses the support (toe construction) and slides.
CHAPTER 5. SOME SUITABLE SOLUTIONS FOR THE SEA DIKES IN VIETNAM

5.1 BOUNDARY CONDITIONS ASPECTS
The most importance aspects with respect to boundary conditions for sea dike design are:

+ Setting up a long-term measurements of wave, current, bathymetry along the entire coast. Analysis all the relevant available data in order to define the boundary conditions.

+ Prepare boundary condition guideline based on the analysis of measurements. This will be useful to calculate certain design parameters.

5.2 DESIGN CALCULATION ASPECTS
The following recommendations with respect to design calculation aspects are:

+ Freeboard values can be estimated on the base of the quality of the subsoil. The settlement of the subsoil must be considered. Freeboard may be considered in the combination of subsoil subsidence and sea level rise.

+ Design of crest height must be estimated for each homogenous section of the dike. In case that the run-up is calculated for an exceedance of more than 2%, the stability of crest and inner slope can be considered.

+ The choice of the formula for calculation of thickness of pitched stone and concrete blocks revetment should be carefully to find out a suitable. When using concrete interlocking blocks special attention should be paid to a stable design of revetment foundation (sub-layer and sub-soil)

+ For the wave higher than 1m, there will be necessary to place an additional granular layer instead of geotextile with aim to reduce the hydraulic gradients and to prevent the migration of the sub-soil under the revetments.

+ When designing slope protection it should be taken in to consideration the future increasing of safety requirements. For their purpose the dimensions of revetments should be calculated also for water levels, and resulting waves, with frequency of exceedance of 2% and 1%.

+ In the design the attention should be paid to the toe protection and the transition between slope and toe, revetment and crest.
5.3 DESIGN CODES

The code 14 TCN 130, 2002 needs to be updated according with the actual international knowledge. The new code must be consists the design, construction and maintenance aspects.

5.4 CONSTRUCTION

In construction, the attention should be paid to:
+ Supervision during construction and a proper quality control system are essential for increasing the stability of the dikes and revetments. In order to archive a proper construction special attention should be paid to the execution of the revetments (type and dimensions of armour layer, filter layers and quality of materials) and the transitions (revetment with crest and revetment with toe) and the compaction of the earth fill (if possible).

+ Specification on construction should be included in the new code.

5.5 MAINTENANCE

The solutions for the maintenance aspects are:
+ Establish a proper maintenance system which clear definitions of responsibilities of various manage levels is needed. It is essential benefit in respect to decision and necessary actions concerning the repair of damages just after a storm and even with small damage in the normal conditions.

+ A management scheme should be established for collecting all relevant data of the structures, boundary conditions which change during the lifetime of the structures.
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

For analysis the failures of the sea dikes at the Northern of Vietnam, the boundary conditions, the design method and practical construction condition must be consider. The following conclusions and recommendations can be given:

* **Natural boundary conditions**
  + To analysis the present condition at the study area, the data must be collected in topography, morphology and also climate conditions to predict the main wave direction and calculate the wave transition from deep water to shallow water condition in front of the dike and wave periods. The wave length also must be considered.

  + In case of lacking data of extreme wave conditions, the depth limited wave height can be used to design, that mean when the wave transition to shallow water it will be break by location water depth. For a certain water depth, the maximum wave height can be occurred as depth limited one.

* **Design method**

  * **Design criteria and formula**
    Most of the design criteria and formula, which were used for existing design are need to update. By doing so, the new design method will be more realiable.

  * **Crest height**
    + The crest height at the Northern Vietnam must be consider to prevent the flood due to the combination of the storm and height tidal level. Normally, the crest height equal to sum of design water level, wave run-up and settlement. The settlement of subsoil, which occurs due to the load of earthfill.

    + The design water level should be determining base on the measurement and calculation of astronomic tidal level, wind set-up and sea level rise.

    + To calculate the wave run-up, the present design Russian formula as well as Netherlands formula has been used. The run-up on dike strongly influenced by the mild of slope and the roughness and also permeability of slope. In case the wave run-up calculated larger than 2% exceedance, the wave overtopping must be taken into account in calculation stability of dikes.

  * **Revetments**
    + The stability of revetments is based on the stability of rocks or concrete block of the armour layer, stability of toe to prevent scour hole occurs and also the stability of dike slope.
+ In present design for sea dikes, the stability of rock and concrete blocks has been calculated by Hudson formula, Van Der Meer formula and Pilarczyk formula. The thickness and weight of rock is important factor to calculate stability of whole revetment. Rocks or blocks not only heavy enough to resist the uplift force cause of wave actions but also include pores between blocks to easy drain water inside internal dikes body. So the blocks with column shape has been suggest.

+ When using the interlocking blocks, the stability may increase 1.5 to 2 times in comparison with loose blocks.

+ In Vietnam condition, the rock size and also concrete blocks is very limited cause of the exploitation and less in construction equipments.

+ The geotechnical aspect is influence directly to the stability of dikes and revetment, such as foundation of sub-soils and compaction of earthfill.

+ The toe of structure should be strong enough to supports the slope of revetment and prevents wave from scouring and keep stability of revetment weight. In some case, the riprap trapezoid has been applied for increasing stability of toe.

+ The filter layer and geotextile has to be considering in design revetment. The geotextile is working as a filter to prevent the core layer particle washing out but the water can easy go through it so the pore water pressure in internal dike body will be decrease.

+ The attention should be paid to re-use of materials in general and the transition between revetment and crest.

* **Constructions**
+ The constructions work must be control by local authorities in quantity and quality of material and construction process based on design documents.
+ Using more suitable equipment to improve the working conditions.

* **Maintenance**
+ The maintenance works need to carry out periodically after construction to ensure the structure can perform its design functions at an acceptable level during the service life.
+ All failure at dikes and revetments must be repaired before next storm season starts, small damages can be repair directly by the local authorities and people.

+ In life time of structure, the boundary conditions and strength of structure can be change so that the DDMFC should be control and order the sufficient data base from local authorities report to update.
APPENDIX 1

RESULTS OF WAVE CALCULATION
APPENDIX 2

CROSS SECTIONS AND GEOTECHNICAL DATA

OF DIKES

AT 6 PROVINCES ALONG THE NORTHERN COAST
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