MINISTRY OF AGRICULTURE AND RURAL DEVELOPMENT

2nd DRAFT

(This version is used for discussion with Dutch experts)

TECHNICAL GUIDELINES

ON

SEA DIKE DESIGN

HANOI - 2011

PREFACE

The ***Technical Guidelines and Standards in Sea Dike Design*** has been issued in the framework of the Program "*Strengthening, upgrading and rehabilitating Sea Dikes from Quang Ninh to Quang Nam (phase 1) and from Quang Ngai to Kien Giang (phase 2)"*. The compilation of this guidelines is based on specific standards 14TCN 130-2002 "*Sea dike Design Guidelines*", *"Technical Standards in Sea Dike Design applied in the framework of Program of strengthening, upgrading and rehabilitating Sea Dikes"* accompanying Decision No. 57/QD-BNN-KHCN dated 08/01/2010. This version has been amended, updated and supplemented with some research findings from the Subject in the second phase of Sea Dike Research Program, namely "*Scientific and Technological Program for the Construction of Sea Dikes and Hydraulic Works in Coastal and Estuarine Areas"*.

Editorial Organization

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LIST OF SYMBOLS

Ký hiệu	**Thông số, đại lượng**	**Đơn vị**
1	**Đất, đá và vật liệu xây dựng**	
α	Góc nghiêng giữa mái đê và đường nằm ngang	Độ (^{0})
γ, γ_B	Trọng lượng riêng của nước, của vật liệu	(t/m^3)
γ	Dung trọng khô của đất	(t/m^3)
ρ, ρ_B	Khối lượng riêng của nước, của vật liệu	(t/m^3)
δ_d	Chiều dày lớp gia cố bằng đá hộc	(m)
δ_B	Chiều dày lớp gia cố bằng tấm bê tông	(m)
δ_f	Chiều dày lớp bảo vệ mái bằng khối phủ	(m)
P	Áp lực	
e	Hệ số rỗng của đất	
n	Độ rỗng	[-]
S	Độ lún	(m)
R_s	Độ nén chặt thiết kế của đất có tính dính	
R_{ds}	Độ nén chặt tương đối của đất rời	
G	Trọng lượng của cấu kiện khối phủ	(t)
A	Khối lượng bê tông	(t)
M	Khối lượng của viên đá	(kg)
W	Trọng lượng của viên đá	(kN)

Ký hiệu	Thông số, đại lượng	Đơn vị
C	Lực dính (của vật liệu dính kết)	KG/cm^2
2	**Thuỷ lực**	
β	Góc giữa đường bờ và hướng sóng tới	Độ (^{0})
υ	Hệ số nhớt động học	
g	Gia tốc trọng trường	(m/s^2)
n	- Hệ số nhám - Số lần	
m	- Hệ số mái dốc, m = cotgα - Các loại số mũ	
x	Khoảng cách theo chiều dòng chảy	(m)
t	Thời gian	(s)
p	Tần suất	(%)
B	Chiều rộng lòng sông	(m)
h	Chiều sâu nước	(m)
i	Độ dốc đáy	
Q	Lưu lượng dòng chảy	(m^3/s)
q	Lưu lượng tràn đơn vị	(m^3/s)
V	Vận tốc dòng chảy	(m/s)
H_{sl}	Chiều cao sóng leo	(m)

Ký hiệu	Thông số, đại lượng	Đơn vị
H_{nd}	Chiều cao nước dâng do bão	(m)
H_s	Chiều cao sóng	(m)
$\overline{H}_S$	Chiều cao sóng trung bình	(m)
$H_{s1/3}$	Chiều cao trung bình của 1/3 số con sóng lớn nhất trong liệt số thống kê về chiều cao sóng	(m)
$H_{s1\%}$	Chiều cao sóng có tần suất tích lũy là 1%	(m)
L_s	Chiều dài sóng	(m)
T_s	Chu kỳ sóng	(s)
Z_{tk}	Mực nước thiết kế	(m)
C	- Vận tốc truyền sóng - Hệ số Chezy	(m/s) ($m^{1/2}$/s)
W	Vận tốc gió	(m/s)
D	Đà gió	(m)
Z_t	Cao trình mực nước triều	(m)
Δ	Chênh lệch mực nước triều	(m)
$Z_đ$	Cao trình đỉnh đê	(m)
a	Trị số gia tăng độ cao an toàn	(m)
B_d	Chiều rộng đỉnh đê	(m)
b_d	Chiều rộng cơ đê	(m)

Ký hiệu	Thông số, đại lượng	Đơn vị
b_f	Chiều rộng thềm giảm sóng trên mái ngoài của đê	(m)
n_k	Số lượng cấu kiện bê tông khối phủ	
k	Các loại hệ số an toàn (trong tính toán ổn định công trình)	
ξ	Chỉ số sóng vỡ hoặc max (chiều cao sóng/độ sâu nước) tại vị trí tính toán	[-]
γ	Hệ số chiết giảm khi tính sóng leo	[-]
γ_b	Hệ số chiết giảm khi bố trí cơ đê, đập	[-]
γ_f	Hệ số chiết giảm độ nhám	[-]
γ_h	Hệ số chiết giảm khi sóng đi vào nước nông	[-]
γ_β	Hệ số chiết giảm khi hướng sóng không vuông góc	[-]
Δ	Hệ số mật độ tương đối của vật chất ; với đất, đá, $\Delta = (\rho_s-\rho_w)/\rho_w$; với $\rho_{s,}$ ρ_w là mật độ đất đá và của nước tương ứng	[-]
η	Mực nước tức thời khi sóng dâng nước	(m)
η_{max}	Mực nước lớn nhất tại bờ biển do sóng làm dâng nước	(m)
η_{min}	Mực nước tại điểm sóng vỡ do sóng làm rút nước	(m)
v	Hệ số nhớt động học	(m^2/s)
ξ	Tham số tương tự sóng vỡ hay gọi là số Iribarren	
$\mu(x)$	Giá trị trung bình của chuỗi số x_i	
ρ_w	Dung trọng của nước biển	(kg/m^3)

Ký hiệu	Thông số, đại lượng	Đơn vị
ρ_a	Dung trọng của không khí	(kg/m^3)
ρ_r	Dung trọng của vật chất (đất, đá)	(kg/m^3)
ρ_s	Dung trọng khô của vật chất	(kg/m^3)
σ	Ứng suất	(N/m^2)
σ_x	Khoảng lệch quân phương của x	[..]
σ^2_x	Bình phương khoảng lệch quân phương	[..]
τ	Ứng suất cắt của đất đá	(N/m^2)
τ_c	Ứng suất cắt đáy ứng với trạng thái chảy ổn định	(N/m^2)
τ_w	Ứng suất cắt đáy do chuyển động quĩ đạo của sóng	(N/m^2)
τ_{cw}	Ứng suất cắt đáy do cả sóng và dòng chảy	(N/m^2)
φ	Hướng sóng, góc giữa hướng sóng tới và đường vuông góc với bờ	Độ $(^0)$
φ_b	Góc giữa hướng sóng tới và đường vuông góc với bờ khi sóng vỡ	Độ $(^0)$
φ_0	Hướng sóng ngoài nước sâu	Độ $(^0)$
ψ	Góc pha của sóng	(rad)
ω	Tần số góc của sóng	(s^{-1})
∞	Vô hạn	
a	Gia tốc	(m/s^2)
a_o	Biên độ của sóng chuyển động trên mặt phẳng ngang gần đáy	(m/s)

Ký hiệu	Thông số, đại lượng	Đơn vị
A	Diện tích	(m^2)
A_c	Chiều cao gia cường đỉnh công trình so với mực nước tĩnh	(m)
A_e	Diện tích xói trên mặt cắt ngang của đê phá sóng ứng với mực nước tĩnh	(m)
B	Chiều rộng công trình	(m)
c	Tốc độ sóng	(m/s)
c_g	Tốc độ nhóm sóng	(m/s)
c_w	Hệ số ma sát giữa không khí và nước	[-]
C	Hệ số Chezy	$(m^{1/2}/s)$
C_D	Hệ số kéo	[-]
C_r	Hệ số phản xạ	[-]
C_t	Hệ số truyền	[-]
D_{50}	Đường kính sàng, đường kính của các viên đá, hạt cát có kích thước vượt quá 50% đường cong tiêu chuẩn	(m)
D_n	Đường kính tương đương (qui đổi cho các vật không là khối cầu)	(m)
D_{n50}	Đường kính tương đương ứng với 50%	(m)
D_z	Đường kính sàng, đường kính viên đá vượt 50% đường cong của s	(m)
e	Cơ số mũ tự nhiên	

Ký hiệu	Thông số, đại lượng	Đơn vị
$E_{\eta\eta}(f)$	Phổ tần số sóng	(J)
$E_{\eta\eta}(f,\theta)$	Phổ hướng sóng	(J)
E_d	Phổ năng lượng hấp thụ hoặc tiêu tán	(J)
E_i	Năng lượng sóng đến	(J)
E_r	Năng lượng sóng phản xạ	(J)
E_t	Năng lượng sóng truyền qua	(J)
f	Tần số sóng	(s^{-1})
f_p	Tần số đỉnh phổ sóng	(s^{-1})
f_w	Hệ số ma sát sóng	[-]
F	Lực	(N)
F_c	Chênh lệch giữa đỉnh tường và đỉnh đập đá đổ	(m)
Fr	Số Froude	[-]
F_s	Hệ số hình dạng	[-]
h	Độ sâu nước	(m)
h_t	Độ sâu của chân công trình dưới mực nước tĩnh	(m)
H_b	Chiều cao sóng tại đường sóng vỡ	(m)
H_i	Chiều cao sóng tại chân công trình hay đường mép nước	(m)
H_{max}	Chiều cao sóng lớn nhất trong tài liệu	(m)
H_{rms}	Chiều cao sóng bình phương trung bình	(m)

Ký hiệu	Thông số, đại lượng	Đơn vị
H_r	Chiều cao sóng phản xạ chân công trình và bờ biển	(m)
H_s	Chiều cao sóng hiệu quả, trung bình của 1/3 số sóng lớn nhất	(m)
H_{sb}	Chiều cao sóng hiệu quả khi vỡ	(m)
H_t	Chiều cao sóng truyền qua công trình	(m)
H_0	Chiều cao sóng ngoài nước sâu	(m)
H_{0s}	Chiều cao sóng hiệu quả nước sâu	(m)
$H_{2\%}$	Chiều cao sóng vượt quá 2%	(m)
$H_{1/10}$	Chiều cao sóng trung bình với 10% các con sóng lớn nhất	(m)
i	Chỉ số dưới các kí hiệu	
i_w	Độ dốc mặt nước do gió gây ra	(m^{-1})
k	Số sóng $k = 2\pi/L$	(m^{-1})
k_s	Chiều dài độ nhám đáy	(m)
K_d	Hệ số nhiễu xạ	[-]
K_D	Hệ số ổn định trong công thức Hudson	[-]
K_r	Hệ số khúc xạ	[-]
K_s	Hệ số nước nông	[-]
L	Bước sóng	(m)
L_m	Bước sóng ứng với chu kỳ trung bình	(m)
L_0	Chiều dài sóng nước sâu hay ngoài khơi xa $L_0 = gT^2/2\pi$	(m)

Ký hiệu	Thông số, đại lượng	Đơn vị
m	Độ dốc bãi biển	[-]
m_2	Moment phổ bậc 2	
m_0	Moment phổ bậc 0	
N	Số sóng trong một trận bão theo tài liệu quan trắc hoặc thí nghiệm	[-]
N_s	Chỉ số ổn định $N_s = H_g/\Delta D_{n50}$	
O_i	Độ mở trong vải địa kỹ thuật	(m)
p_a	Áp suất không khí	(mbar)
p(x)	Xác suất của biến cố x	
P_b	Hệ số thoát nước của đập phá sóng	[-]
P	Thông lượng	(J/s)
P(X<x)	Xác suất một biến cố nhỏ hơn 1 giá trị xác định	[-]
q	Lưu lượng đơn vị	(m^3/s/m)
Q	Lưu lượng nước	(m^3/s)
Q^*	Lưu lượng tràn không thứ nguyên $Q^* = T_m/gH_s$	[-]
r	Bán kính	(m)
R	Ký hiệu cường độ trong tính toán tần suất	
R_c	Độ cao gia cường tính từ mực nước tĩnh thiết kế đến đỉnh công trình	(m)

Ký hiệu	Thông số, đại lượng	Đơn vị
R_d	Mực nước rút tính từ mực nước tĩnh	(m)
R_v	Mực nước sóng leo tính từ mực nước tĩnh	(m)
Re	Số Reynolds	[-]
s	Độ dốc sóng	[-]
S	Hệ số hư hỏng	[-]
S_p	Ký hiệu tải trọng trong thiết kế xác suất	[-]
t	Thời gian gió	(s)
T	Chu kỳ sóng	(s)
T_o	Nhiệt độ	(^{0}C)
T_m	Chu kỳ sóng trung bình	(s)
T_p	Chu kỳ đỉnh phổ, giá trị ngược của tần số	(s)
T_r	Chu kỳ lặp lại của 1 biến cố	(s)
T_s	Chu kỳ sóng hiệu quả	(s)
T_z	Chu kỳ sóng trung bình tìm từ phân tích thống kê	(s)
u_0	Lưu tốc sóng lớn nhất tại đáy biển	(m/s)
U_w	Tốc độ gió	(m/s)
U_{10}	Tốc độ gió tại độ cao 10 m trên mặt biển	(m/s)
v	Lưu tốc	(m/s)
v_c	Lưu tốc tới hạn	(m/s)

Ký hiệu	Thông số, đại lượng	Đơn vị
v_*	Lưu tốc cắt	(m/s)
w	Tốc độ chìm lắng của bùn cát trong nước tĩnh	(m/s)
z	Chiều cao trên mực chuẩn	(m)
z_a	Tăng của mực nước do áp suất khí quyển	(m)
Z	Hàm tin cậy trong thiết kế xác suất	

ABBREVIATIONS

No.	Abbreviation	Full text
1	BT	Concrete
2	MNTK	Design Water Level
3	NBNT	Beach Nourishment
4	SPT	Standard Penetration Test
5	TCAT	Safety Standard

1. GENERAL STIPULATIONS

1.1. This guideline is applied in designing new sea dikes, rehabilitating and upgrading various types of sea dikes and other related structures, such as:

- Dikes protecting populated areas, coastal economic areas (aquaculture, salt fields, tourism etc.)

- Dikes encroaching into the sea for the purpose of land reclamation;

- Retaining dikes for island protection;

- Estuarine dikes under impacts of tides and waves from the sea;

- Combining dikes for multiple purposes

1.2. General bases and principles of sea dike design

Conforming with the current regulations of Construction Investment Project management and provisions:

- Law on Dykes and dyke-maintenance

- Law on Basic Construction

- Master plans for socio-economic development and natural disaster prevention and response in the area;, plans for coastal transportation and other related plans;

- Applying other concerned Codes and Technical standards;

- Applying new achievements of science and technology which are suitable for the dike conditions in Vietnam;

- Active loads are calculated as per current stipulations in Hydraulic works design;

- The elevation system and coordinate system used in sea dike design is the National Elevation System;

- Technical solutions: Combination of structural and non-structural solutions; appropriate to the scenarios of climate change impacts must be applied.

2. DATA REQUIRED IN SEA DIKE DESIGN

2.1 Topographic data

- Components and amount of topographic survey for sea dike design are stipulated in Technical Code 14 TCN 165-2006[1] ;

- In addition, the following requirements should also be satisfied:

+ Upon collecting existing data, the measuring time must be less than 5 years in case of stable foreshores, and less than 1 year for the foreshores being accreted or eroded;

+ Topographic surveys must be extended at least 100m from the dike toe to both sides of the design dike route, and up to 200m in case of variable topography;

+ For attending to calculate the wave propagation from depth water to dike toe, for each dike route direction, a representative dike cross section must be surveyed from waterfront to 10m depth offshore.

+ For frequently eroded coastlines, the historical data on the coastline evolution for at least 20 years up to the timing of project setup need to be collected;

+ In case of largely-distributed soft soil areas (swamp, for example), the aerial measurement method can be applied in order to obtain the topographic and geomorphic data.

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[1] Applying National Technical Code when 14TCN Code is converted correspondingly

2.2 Geological data

- Components and amount of geological surveys for sea dike design are stipulated in Technical Code 14 TCN 195-2006[1] for the project setup stage and hydraulic works design.

- During the preparation of bidding documents and performing the consulting activities, the determination of detailed geological survey components and amount must be based on the actual conditions and the content of above-mentioned Technical Code, and they must be submitted to the competent authorities for approval.

2.3 Meteorological, hydrological and oceanographical data

- Collecting data on the impacts of typhoons and natural disasters in the territorial waters within the project area;

- Forecast of natural disasters;

- Data on tides, nearshore currents, sediment transport, wave setup, waves, flood current (including collected data and newly-measured data);

2.4 Resident livelihood, economic and environmental data

- Collecting data on existing population and development trend, current economic condition and development orientation, environmental condition and evaluation of impact level in the future.

- Requirements and urgency of the structure construction.

3. SAFETY STANDARD DETERMINATION AND SEA DIKE GRADE CLASSIFICATION

3.1 Determination of safety standards

In this section, the designer will be instructed to select safety standard through the features of protected area such as population, economy, the disadvantages of natural conditions, flooding level.

Safety standard is determined on the basis of risk based optimal results taking into account economic risk, potential loss of life in the protected area and the investment capability into consideration. The safety standard is represented by the exceedance frequency of occurrence (1/ years) or return period (in years).

Safety standard is determined on the minimum acceptable value of flooding occurrence probability of protected area. It isn't homogeneous with structure incident probability and elements. Structure safety have to observe by followed safety coefficients of Codes and Law in basic construction.

Based on the features of protected area, safety standard (SS) is determined by following special characteristic protected region:

3.1.1 Characteristic protected Region of Type 1: is coastal regions which have no potential inundated due to local rain or due to upstream rain water coming down. With this type of region the construction of sea/ estuarine dikes will not cause extra inundation for the protected area because dikes will not prevent discharging local rain water to the sea/ estuaries. SS for the Region of Type 1 is defined as per ***Table 3.1***.

Table 3.1. *Safety standards for characteristic region type 1*

Characteristics of protected area	Safety standards (SS) (return period: years)
Developed industrial urban area: - Protected area > 100.000 ha - Population > 200.000 people	150
Rural areas having developed industry and agriculture: - Protected area : 50.000 ÷ 100.000 ha - Population: 100.000 ÷ 200.000 people	100
Developed rural and agricultural area - Protected area :10.000 -50.000 ha - Population: 50.000 – 100.000 people	50
Medium-developed rural and agricultural area - Protected area : 5.000 – 10.000 ha - Population: 10.000 – 50.000 people	30
Under-developed rural and agricultural area - Protected area : < 5.000 ha - Population : < 10.000 people	10 < SS < 30

Most of coastal regions in the North to Northern Central (from Quang Ninh to Ha Tinh) and Southern Central and South (from Vung Tau to Kien Giang) is considered to be characterized as region type 1.

3.1.2 Characteristic protected Region of Type 2: is coastal region which has been potentially inundated due to local rain and/or due to upstream rain water coming down through river system/overland. With this type of region the construction of sea/ estuarine dikes will cause extra inundation for the protected area because dikes will prevent the discharge of local rain water to the sea/ estuaries. SS for the Region of Type 2 is defined as per ***Table 3.2***.

Table 3.2 *Safety standards for characteristic region type 2*

Characteristics of protected area	**Safety standards (return period: years)**
Developed industrial urban area: - Protected area > 100.000 ha - Population > 200.000 people	50
Rural areas having developed industry and agriculture: - Protected area : 50.000 ÷ 100.000 ha - Population: 100.000 ÷ 200.000 people	20
Developed rural and agricultural area - Protected area :10.000 -50.000 ha - Population: 50.000 – 100.000 people	10
Medium-developed rural and agricultural area - Protected area : 5.000 – 10.000 ha - Population: 10.000 – 50.000 people	5
Under-developed rural and agricultural area - Protected area : < 5.000 ha	1 ≤ TCAT < 5

- Population : < 10.000 people	

Most of coastal regions in the central part(from Quang Binh to Binh Thuan) is considered to be characterized region type 2.

***** *Notes:*

- Developed industrial and agricultural areas are determined on the basis of the percentage of economic structure in the protected area. If the industrial rate is greater, then it is a developed industrial area and vice versa.

- Upon using Table 3.1, firstly the protected areas must be classified using the given criteria. Then the two criteria are considered in order to determine the safety standard. In case the protected area meet only one criterion, the level is lowered by one. The spatial planning must take the planning for socio-economic development up to 2020 and vision for 2050 into consideration.

3.2 Determination of sea dike grade

- Sea dikes are classified into 5 grades: grade I, grade II, grade III, grade IV and grade V;

- The dike grade depends on the range of safety standard of the protected area, given in Table 3.3.

In addition, dike grade can also classified by potential inundation depth of protected area (see ***Table 3.4***).

Table 3.3 *Criteria for dike classification*

Dike grade	I	II	III	IV	V
SS (return period: year)	150	100	50	30	10 < SS < 30

Table 3.4. *Dike classification by potential inundation depth of protected area*

Potential inundation depth compare to design water level (m)	Dike grade
Greater than 3	I – II
2-3	II – III
1-2	III – IV
< 1	V

Chosen dike grade is smaller one from these above two criteria and is in the range of grade I to V.

** Note: In case that the dike system combines with other functions from flood defences, e.g. plays an important role concerning security, national defence or is located in an area with specific social characteristics, there must be specific regulation for chosing dike grade in that case.*

4. DESIGN OF DIKE ROUTE

4.1 General requirements

Sea dike route is selected on the basis of economic-technical comparison of different schemes after considering the following aspects:

- Conforming to the master plan for development of the entire area;
- Topographic and geologic conditions;
- Evolution of the coastlines, beaches and estuaries;
- Location of the existing structures and the structures built as per planning;

- Safety and advantages in sea dike construction, management and operation, and the facilitation of preserving and planting mangrove forests in front of the dike;

- Protection of the cultural, historical remains and administrative land boundaries;

- Conforming to maritime navigation development strategy;

- Maximized combination of coastal roads.

- Conforming to adaptable solutions to the impacts of climate change;

4.2 Requirements of sea dike route location

- Running through the areas of high topography and passable geological conditions of the foundation.

- Smooth and stable connection with existing structures.

- Running through the areas which are favourable for the arrangement of dike-crossing and auxiliary structures;

- Less impacts on the flood discharging and river training structures (for estuarine dikes);

- Minimizing impacts on prevention of discharging local rain induced inundated water to the sea and estuaries.

- Meeting the requirements of sea ports and lands behind the dike, beaches, tourism areas, historical remains and beauty spots;

- In case the dike route is combined with the transportation system, national security and defence, it must also conform to the other corresponding regulations;

- Making the maximum use of the natural sand dunes, hills, existing structures in order to close the dike route with stable connection;

- Dike runging through eroded coasts/ unstatble estuaries under impact of complex systems to protect the important cities, a study on the trend of shoreline changes must be done to set a suitable dike route.

- Making a comparison of the economic-technical efficiencies among 2-3 locations of dike route in order to select the one which shows the best total efficiency;

4.3 Requirement of sea dike route shape

- Dike routes should be designed as straight lines or smooth curves without many zigzags which can cause local concentration of wave energy. The construction volume and the capital cost should be compared in order to select the most appropriate dike route;

- In case the dike route must be in concave shape, appropriate solutions to wave attenuation or dike resistance strengthening need to be adopted;

- No weak chain links created at the connection with other nearby structures and no impacts on relevant areas;

- In case of rehabilitated and upgraded sea dikes, the aforementioned requirements must be considered in order to adjust locally necessary sections.

4.4 Design of route for each type of sea dike

4.4.1 Design of border dike route for new land reclamation

4.4.1.1. General requirements

- Border dike route must be determined on the basis of the study on trend of accretion and other influence factors, such as: hydrodynamic conditions at the connection zone, waves, nearshore sediment flow, imbalance of sand transport in nearby areas, forecast of development trend of the foreshore in the future;

- Border dike route must be convenient for construction, especially in case of dike closure, drainage, new land reclamation, soil improvement (leaching and desalinization), plant structure, operation procedure and environment conservation.

4.4.1.2. Foreshore elevation for land reclamation

Alternatives for the elevation of dike constrcution for land reclamation need to be compared and selected on the economical-technical basis in the following cases:

- The design dike is constructed on the foreshore located above mean sea level. In Northern delta, the foreshore level is from +0,5m to +1,0m with reference to mainland coordinate system.

- The design dike can also be built on lower-elevation foreshores, adopting appropriate technical approaches in order to speed up the accretion process for the coastal areas behind the dike after the land reclamation has satisfied the operation requirements.

4.4.1.3. Secondary dike routes for zone division

It is necessary to build the secondary dike routes behind the main one, which separates the entire area behind into different zones and sections depending on natural conditions and service requirements. This will help to narrow down the damages in case of failure of the main dike.

4.4.2 Dike route at the eroded coasts (ingression)

4.4.2.1 General requirements

- At the eroded coastal areas, the dike route is usually damaged due to the direct impacts of waves on the dike body, failure of outer slope and dike toe. In this case, the evolution of the coastline, mechanism and causes of the coastal erosion and other influence factors need to be studied thoroughly in order to decide the appropriate alternative;

- Consideration of dike route must be related to the solutions for erosion restraining, accretion facilitating and foreshore stabilizing;

When there are no eroding mitigrating solutions, - dike route position must comply with set back line of the region on basis of expected life time of the dike system. Apart from the main dike, space must be reserved for dike set back. The secondary dike route can be built in combination with non-structural approaches in order to minimize the damage in case the main dike route has been destroyed.

4.4.2.2 Main dike route

As per Article 4.1 and 4.2, the following criteria must be taken into consideration in order to locate the route of main dike at the eroded areas:

- The dike route is located behind the first breaker line at least to a distance of one design wave length);

- Parallel to the waterline at low tide;

4.4.2.3 Secondary dike route

- Secondary dike is usually located behind the main dike. The distance between them is at least 2 times of the design wave length.

- Enclosure dikes should be arranged between the main dike and secondary dike, with a distance of 3-4 times of the distance between the two dikes.

- Upon designing the enclosure dikes, traffic and rescue requirements must be taken into consideration.

4.4.3. Estuarine dike routes

- Estuarine dike is the connection between river dike and sea dike, under the overall impact of river and marine factors;

- Estuarine dike route must ensure the flood discharging and safety under the impacts of the river and sea;

- For deltaic estuaries with many branches, analysis of the evolution of each branch must be performed in order to obtain the most favourable dike route planning for flood discharging;

- For funnel-shaped estuaries, the curve shape of dike route need to be restrained (by means of calculation or empirical data) in order to control the increase of wave height due to the amplification factor without endangering the river banks.

5. DESIGN OF SEA DIKE CROSS-SECTION

5.1. Requirements of sea dike cross-section design

5.1.1. General requirements

Appropriate design cross-sections of sea dikes and other related structures on each section of sea dikes must be selected on the basis of geological conditions of foundation, embankment materials, active external loads, construction plan and service requirements.

In case the existing sea dike system is upgraded and rehabilitated, the cross-sections of current and supplementary dike routes must be appropriated to the natural conditions.

5.1.2. Technical requirements

The most important requirements of sea dikes and revetments is the reliability in withstanding storms and floods, also coping with the problem of sea level rise as a result of global climate change. In addition, sea dikes and revetments must be appropriate to local natural conditions in each area.

Technical Standards of Sea dike design is established on the basis of the factors such as natural, hydrodynamic and geological conditions, as well as construction technology, operation and maintenance of sea dikes and revetments.

5.1.3. Requirements of national security and defence

Apart from the requirements of protecting inhabitants, infrastructure and the coastline, the design route of sea dikes must also function as defensive routes, vital transportation routes in keeping clear connection between mainland and the islands, and military posts for guarding and patrolling units.

5.1.4. Requirements of extensive and multipurpose exploitation

Sea dikes and revetments must fulfil the requirements of extensive and multipurpose exploitation, serving the needs of nearshore navigation, petroleum and mineral industry, tourism and aquaculture; drainage of inland water and downstream flood water; prevention of salt intrusion; retention of fresh water for agricultural production; prevention of coastal and estuarine erosion, enlarging the foreshore for the purpose of marine economic development and prevention of natural disasters; adapting to the problem of sea level rise as a result of global climate change.

5.1.5. Economic requirements

Sea dikes and revetments must fulfil the economic requirements such as: lowest construction cost; optimization of multipurpose effect; lowest costs of dike management, exploitation and maintenance. Attention should be focused to the selection of optimal cross-section of sea dikes and revetments in order to satisfy all of the above-mentioned requirements.

5.1.6. Environmentally-friendly requirements

The design cross-section of sea dikes and revetments must be environmentally-friendly with appropriate structural solutions without disrupting the nearshore marine ecology as well as the local landscape, especially in case of the coastal tourism and densely populated areas.

5.1.7. Requirements of the adaptation to climate change – sea level rise

The crest elevation and other geometric dimensions of sea dikes and revetments must be designed on the basis of actual situation and data on the rates of sea level rise.

5.1.8 Preventive requirements of common failure mechanisms

The requirements of sea dikes and revetments design is that structural solutions for dike elements must be durable and stable against design external loads, without failure and damage. In case of sea dikes and revetments in Vietnam, the main failure mechanisms include:

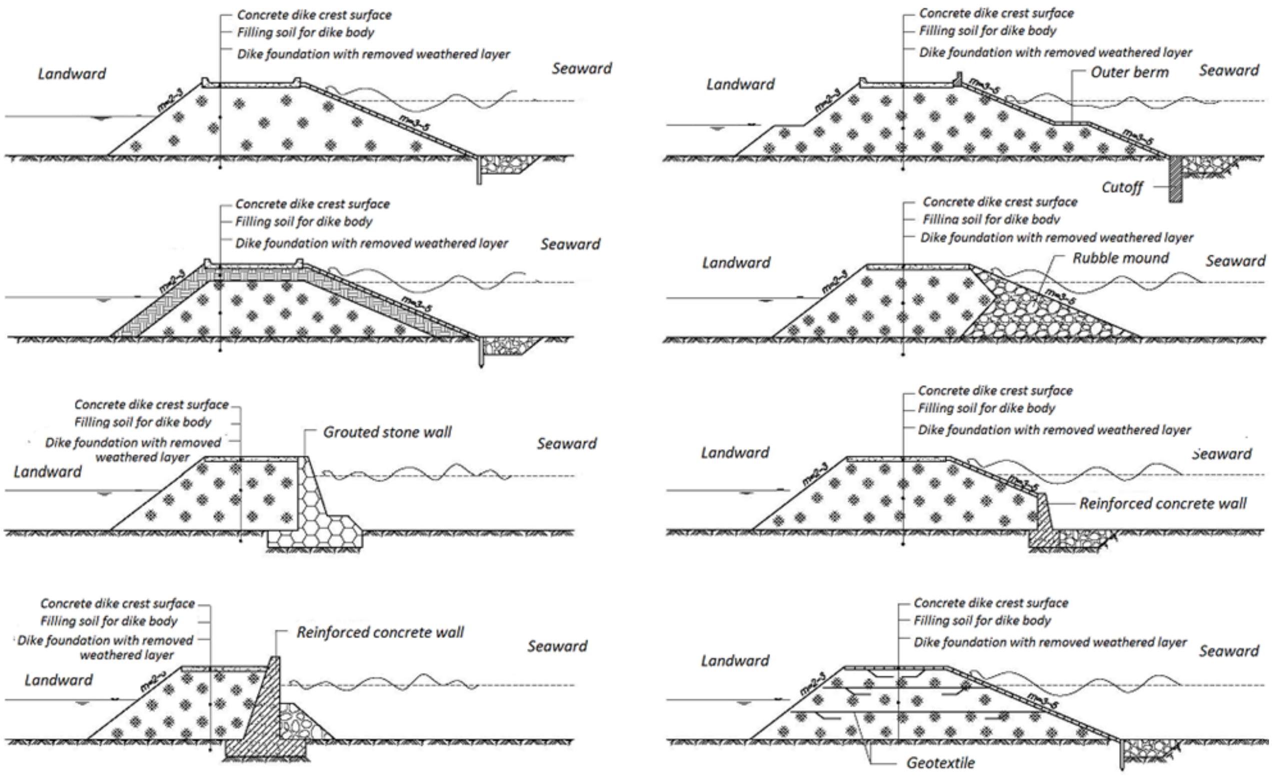
- Wave overtopping discharge exceeding the allowable limit;
- Dike slope sliding (both outer and inner slope);
- Toe erosion;
- Failure of slope protection structures, of dike crest, and erosion of dike body
- Dike settlement;
- Failure of structures on dikes;
- Failure at the transitions;
- Erosion of coastal natural dikes/ sand dunes;

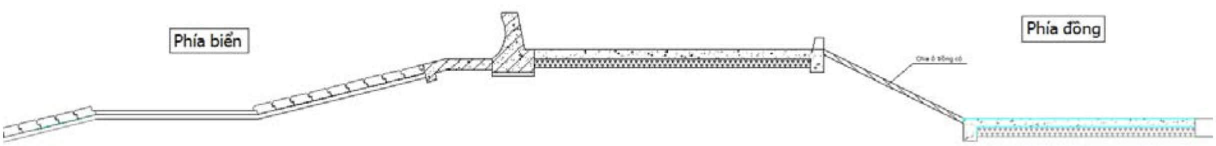
The details of main failure mechanisms of sea dikes and revetment in Vietnam are given in *Section N.1* of ***Appendix N***.

5.2 Types of sea dike cross-section

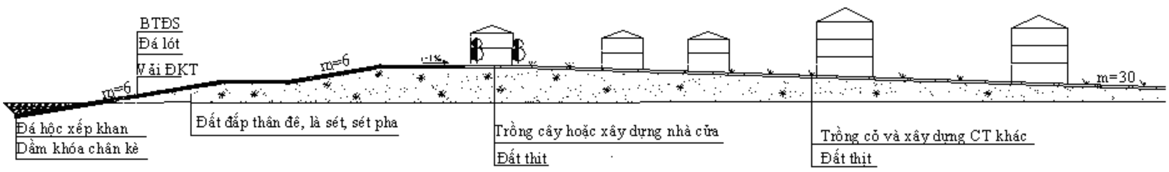
Based on the geometrical characteristics of outer dike slope, the dike cross sections are classified into 3 types: sloping dikes, vertical wall-typed dikes and composite dikes (upper slope and lower vertical wall or upper vertical wall and lower slope). Selection of a cross section must depend on the topographical, geologic, hydrological and oceanographic conditions, as well as construction material, construction conditions and service requirements in order to analyse and decide.

Some types of sea dike cross sections which can be selected are shown in ***Fig. 5.1*** below.





i. Dikes in combination with transportation routes



k. Environmentally-friendly dikes (superdikes)

Figure 5.1 *Types of sea dike cross-sections and corresponding arrangement of materials*

5.3 Content of sea dike cross-section design

Items in the design of sea dike cross section include: crest level, dimensions of cross-section, crest structures, dike body and dike toe, which fulfil technical and economic requirements.

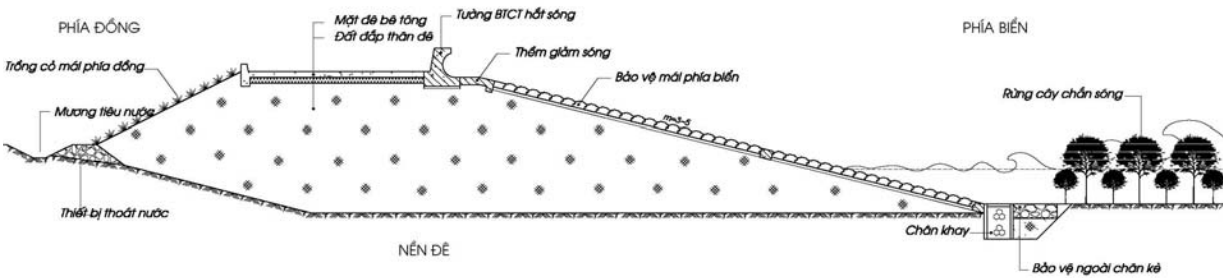


Figure 5.2 *Diagram of sea dike cross-section*

General cross-section of a dike consists of: (1) Outer embankment footing protection, (2) Embankment footing, (3) Lower outer slope, (4) Outer dike berm, (5) Upper outer slope, (6) Dike crest, (7) Inner slope, (8) Landward drainage facilities, (9) Inner drainage channel, (10) Dike body, (11) Dike foundation and (12) Transitions between the dike elements.

Contents of sea dike design are as follows:

1) Design of crest level;
2) Design of dike body;
3) Design of filter layers;
4) Design of slope protection layers;
5) Design of toe protection;
6) Design of dike crest structures;
7) Design of crown wall (if necessary);
8) Design of transition structures;
9) Stability calculation.

5.4. Determination of dike crest level

Crest level of sea dike is defined as the elevation of the dike crest after the settlement has become stable. In case of a crown wall placed on the crest, the elevation of its top is considered the crest level of sea dike.

5.4.1. General formula

$$Z_{đp} = Z_{tk,p} + H_{lk} + a \qquad (5.1)$$

where,

$Z_{đp}$ - Design crest level (m);

$Z_{tk,p}$ - Design water level (MNTK);

H_{lk} - Cress freeboard above design water level;

a - Safety height increment of dike, determined as per ***Table 5.1***;

Table 5.1 *Safety height increment of sea dike*

Dike grade	I	II	III	IV	V
a (m)	0,6	0,5	0,4	0,3	0,2

The above-mentioned safety height increment excludes the provision height for settlement, thus this parameter must be taken into consideration in the design of dike crest.

The following notice must be taken when defining the dike crest level:

- In the same dike route with different dike crest level at different segments, the highest level must be chosen as design level for the entire route;

- In case the strong and stable crown wall is placed on seaward side, the dike crest level is that of the crown wall.

Methods for the determination of each term in the formula for dike crest level design in each specific case will be given in the following sections.

5.4.2 Determination of design water level

Design water level is a combination of astronomic tidal level and the components of surging height generated by different factors corresponding to the design frequency such as: storm surge, wind setup, depression and the setup of flood water from the river.

5.4.2.1. Seaward frontal dikes

For the sake of simplicity in application, the Design Water Level of seaward frontal dikes mostly consists of 2 main components: maximum astronomic tidal water level and storm surge height corresponding to the design frequency. In case the impacts of sea level rise due to global climate change are taken into consideration, the Design Water Level can be determined as follows:

$$Z_{tk,p} = (Z_{TB}+\Delta Z_{NBD}) + A_{tr,\,max} + H_{nd,p} \qquad (5\text{-}2)$$

where,

Z_{TB} - Mean sea level in the study area, with reference to national elevation system;

ΔZ_{NBD} – Average increment of sea level considering impacts of sea level rise, determined as follows:

+ Without impacts of sea level rise, $\Delta Z_{NBD}=0$.

+ With impacts of sea level rise: $\Delta Z_{NBD} = T_{ct} \times R_{NBD}$

where,

T_{CT} - Proposed service life of sea dike (in years);

T_{CT} is determined on the basis of dike grade, as per Construction Standard of Vietnam (TCXDVN 285-2002: *Hydraulic Structures – Principal Guidelines in Design*", summarized in the following table:

Dike grade	***Life time (years)***
I-II	100
III-IV	50
V	30

R_{NBD} - Average rate of sea level rise (in *m/year*) in the scenario stipulated by Ministry of Natural Resources & Environment.

Example: *The average rate of sea level rise along North Coastal areas of Vietnam is 0,006 m/year. The sea dike system is designed to protect urban areas with grade III and service life of 50 years.*

The increment of sea level due to the impacts of sea level rise in this case is:

$$\Delta Z_{NBD} = 50 \text{ (year) x } 0{,}006 \text{ (m/year)} = 0.3 \text{ m.}$$

$A_{tr,\, max}$ - Maximum astronomic tidal amplitude, with reference to mean sea level (m);

H_{nd} - Storm surge height corresponding to design frequency P%. The frequency curve of storm surge heights is established on the basis of observation data in a sufficiently long duration, at least 40 years.

(a) Determination of Design Water Level in case of sufficient measurement data

In case of sufficient measurement data on tidal water levels as well as storm surge height, the Design Water Level can be determined as per Formula (5-2);

(b) Determination of Design Water Level in case of no measurement data available

In case of insufficient measurement data, the Design Water Level is determined by means of frequency curves of combined water level at each specific locations along the coastline of Vietnam (see Appendix A). In order to incorporate the impacts of sea level rise in this case, the Design Water Level determined from the combined water level frequency curve should be supplemented by the average increment of sea level (ΔZ_{NBD}), using the following formula:

$$Z_{tk,p} = MNTK_p^{look\text{-}up} + \Delta Z_{NBD}$$

in which $MNTK_p^{look\text{-}up}$ is the sea level corresponding to frequency P(%) determined from the frequency curve of combined water level in Appendix A.

5.4.2.2. Dikes surrounding estuaries and lagoons

In the design of dikes surrounding estuaries and lagoons, the Design Water Level must include the setup due to combined impacts of the flood water from the rivers and from the sea. In this case, 1-D hydraulic model must be employed in order to determine the combined water levels of riverine and coastal factors. Boundary conditions of seaward water level are determined according to Section 5.4.2.1. Riverward boundary condition is the water level and flood discharge in the river, in which the flood frequency corresponds to the design frequency.

5.4.3 Determination of required freeboard (H_{lk}):

5.4.3.1 Seaward frontal dikes with no overtopping

Seaward frontal dikes withstand direct impacts of waves on the outer slope, thus the required crest freeboard (H_{lk}) is determined on the basis of wave run-up height.

In case no overtopping allowed, H_{lk} is defined as the height of design wave run-up. This can be considerd a specific case, in which the allowable overtopping discharge is very small, inconsiderable and, or non-overtopping waves. In this case, the inner slope and crest of sea dike can be protected only by normal grass if no more specific requirement is considered.

The required freeboard is defined as:

$$\mathrm{H_{lk}} = \mathrm{R_{sl,p}}$$

where,

$\mathrm{H_{lk}}$ - Freeboard of dike crest with reference to Design Water Level;

$\mathrm{R_{sl,p}}$ – Wave run-up height (with a frequency of 2%) generated by design wave height (see ***Appendix D***);

5.4.3.2. Seaward frontal dikes, with allowable unit overtopping discharge [q] (l/s/m)

In this case, the active factors is similar to those explained in Section 5.4.2.1. However, the required crest freeboard $\mathrm{H_{lk}}$ must be sufficient so that the mean overtopping discharges do not exceed the design allowable values *[q]* (l/s/m).

The required crest freeboard is determined as follows:

$$H_{lk} = R_{c,q}$$

where, H_{lk} - Crest freeboard of sea dike with reference to Design Water Level;

$R_{c,q}$ – Required crest freeboard of sea dike with reference to Design Water Level, at which the overtopping discharges do not exceed the allowable values *[q]* (l/s/m) in the design conditions. Required crest freeboard using overtopping criteria is determined with the design frequency P(%) and explained in ***Appendix D***.

[q] - Design allowable overtopping discharge (l/s/m).

The allowable overtopping discharge [q] is selected on the basis of the durability against erosion of different protective solutions for dike crest and inner slope. In addition, the selection of design overtopping discharge must take damage extent and impacts on the landward areas in case of design overtopping discharge.

Allowable overtopping discharges are given in Table 5.2, which are now commonly used all over the world. Based on this, the alternatives to protect the inner slope of sea dike, as well as the collection, storage and drainage of overtopping water can be proposed.

Table 5.2. *Relation between allowable overtopping discharges and protective solutions for inner dike slope*

Quality/ Protection extent of inner slope	**Allowable mean overtopping discharge [q] (*l/s/m*)**
Undefined quality, non-protected	0,1 (equivalent to run-up condition)
Protected by well-grown grass on compacted clay base layer	10
Well protected/strengthened on compacted clay layer; in design of dike toe protection; in design of overtopping water drainage; in design of overtopping water storage and water drainage after storms.	> 10, depends on specific condition of inner protection

More details of the relation between allowable overtopping discharge and protective solutions for inner dike slope, as well as the damage extent are given in ***Table 5.3*** below.

5.4.3.3 Dikes surrounding estuaries and lagoons

Depending on specific conditions at construction sites, direct loads acting on outer slope of sea dikes must be considered.

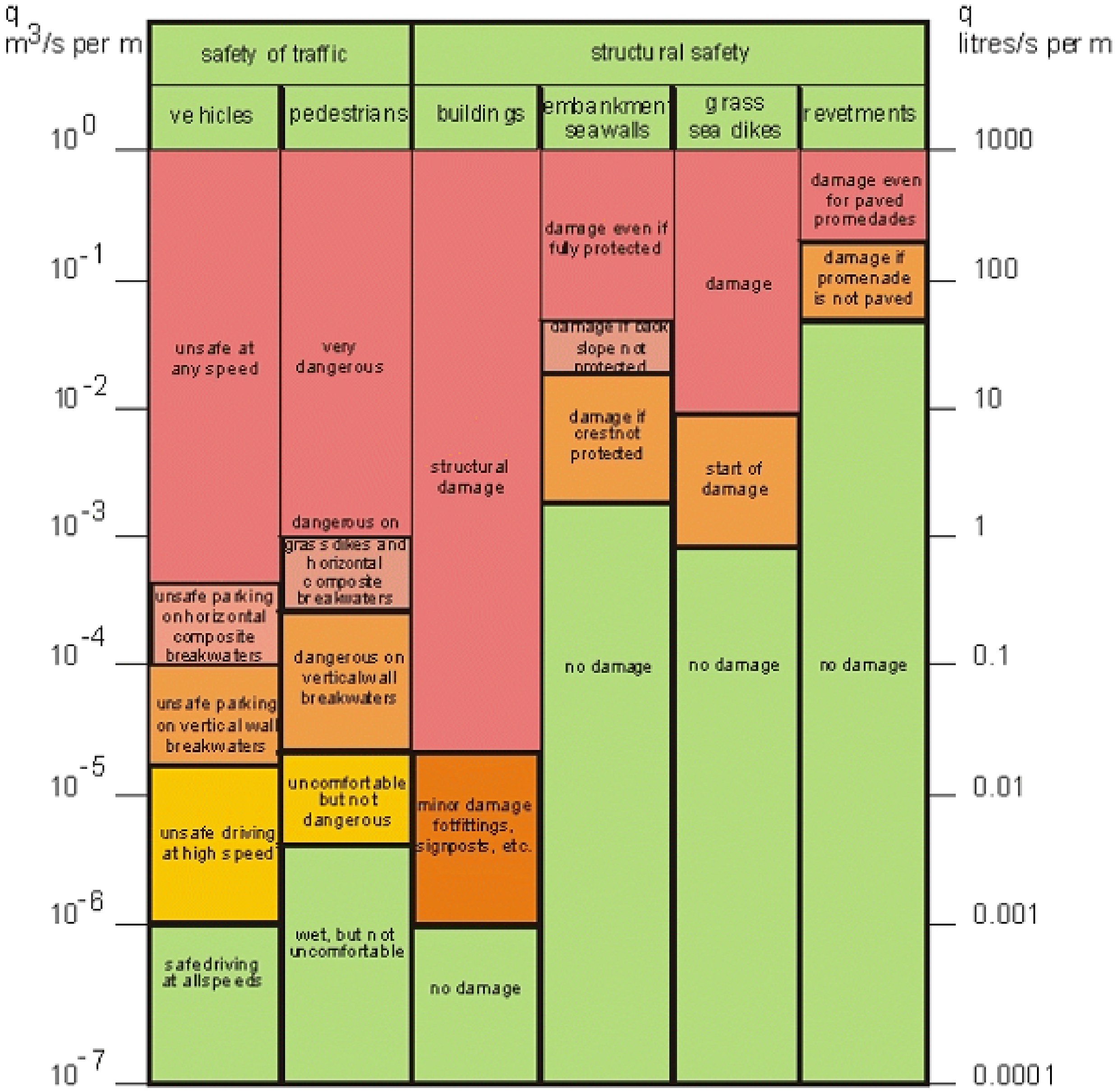
In case of openly-enlarged estuaries under impacts of waves, the determination of crest freeboard with reference to Design Water Level is performed in a similar way as given in Section 5.4.3.1 or 5.4.3.2. However, the wave parameters (wave height and wave period) used in design is the results of computing wave propagation from deepwater boundary to the construction sites in the estuarine areas.

In the design of dikes surrounding large estuaries or in combination with frequent navigation, the impacts of locally generated waves at the estuarine areas must be taken into consideration, such as: locally wind-generated waves (see details in ***Appendix C***) or ship-induced waves etc. When the local wave height in these areas (due to above-mentioned reasons) is greater than or equal to 0.5 m, the required crest freeboard H_{lk} due to the impacts of local waves must be considered in the design of dike crest level.

In other cases when the impacts of the waves from the sea on the construction locations in estuarine areas are inconsiderale (less than 0.5 m), locally generated waves are low, the only main active factor that should be considered in the design of dike crest level is the Design Water Level. The required crest freeboard H_{lk} can be neglected in this case.

Table 5.3 *Allowable mean overtopping discharge*

(source: PIANC - WG 28)



- *Note*:

In case of dikes with the requirements of discharging inland flood seawards, two-way overtopping (landwards and seawards) and appropriate solutions for dike strengthening (crest, slope, body, toe etc.) must be ensured (see ***Fig. 5.4***).

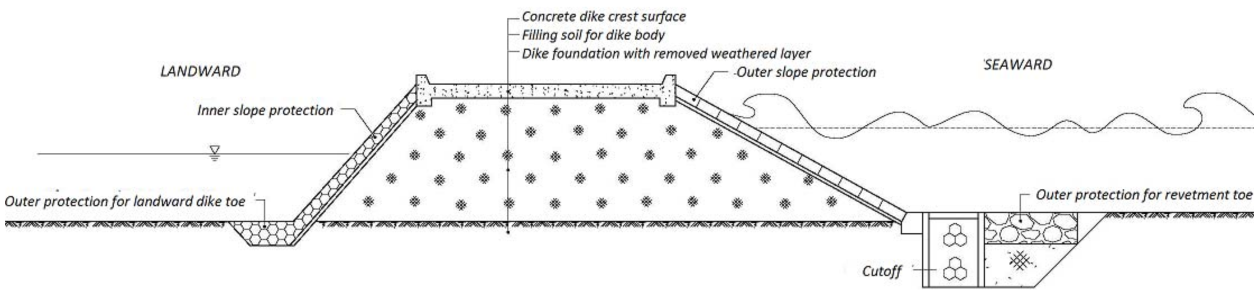


Figure 5.4 *Sea dike profile with three-faced protection*

5.4.4. Determination of design wave parameters

The wave height used in the formula for wave run-up and wave overtopping calculations is the significant incident wave height H_s determined at the dike toe, corresponding to the design frequency (design wave height in short).

The wave parameters at the dike toe is determined by means of the propagation of design deepwater waves to the study location.

- *Location of dike toe:* The dike toe in this guideline is defined as the location at a distance of $L/2 \div L_0/4$ from the water edge in the seaward direction perpendicular to the coastline, in which L is the local wave length and L_0 is the deepwater wave length.

- *Wave-propagating computation*: There are many models can be employed in order to predict the decay of wave energy due to breaking, and from that the corresponding wave height at the toe can be determined. This Guideline recommends the one-dimensional Wave-propagating Model SWAN-1D developed by Delft Technology University, the Netherlands in the determination of design wave parameters at the design dike toe.

In case the design dike route is shielded by the mangrove forests, wave-attenuating effect of mangrove forests must be taken into consideration in the computation of wave propagation as explained in *Section 8.1.* In this case, the design wave parameters at the dike toe include the impacts of mangrove forests.

When using one-dimensional model in wave propagation, the representative topological cross-section must be employed for analysis. Due to difficulty and high costs in topographic survey and measurement, the representative cross-section and the available bathymetry can be combined in order to interpolate the depth contour (or depth points) down to 20 m deep so that the correct input wave parameters are in deepwater area.

Wave-propagating Model SWAN - 1D and User's Manual can be downloaded for free on the website of Dike Management and Flood Control Department (*http://www.ccfsc.gov.vn*). A summary of this manual is given in ***Appendix M.***

In addition, other wave-propagating models are also recommended for the purpose of calibrating and comparing the results. For example, the graphic methods proposed by GODA (1980) and OWEN (1980) applicable to the foreshore slope in the range of 1:10 to 1:100 (for gentler foreshore, the results achieved in case of foreshore slope of 1:100 can be used).

Design deepwater wave parameters at different locations along the coastline of Vietnam can be determined (as a reference) in ***Appendix B (Section B-1)***.

Furthermore, for the purpose of comparison, the design wave parameters at the dike toe in each location in different cross-sections along the coastline of Vietnam can be directly determined according to ***Appendix B (Section B-2)***.

The results of design wave height at the dike toe must be verified and compared with the empirical formula for depth-limited wave height:

$$H_s = a \times h$$

where,

a – Empirical coefficient, depending on the coastal areas, the value a = 0,55 can be taken along the coastline of Vietnam;

h - Water depth corresponding to Design Water Level at the study location (in front of the dike toe).

5.5 Dike crest width

- Based on the dike grade, the dike crest width can be determined as per ***Table 5.4***.

Table 5.4 *Dike crest width based on dike grade*

Dike grade	I	II	III	IV	V
Crest width $B_đ$ (m)	6 ÷ 8	6	5	4	3

In case the dike route in the project area is combined with transportation routes, the crest width and other requirements of the base must be determine as per the standards of main road design, more specifically is TCVN 4054-2005 - *Requirements of Roadway Design*.

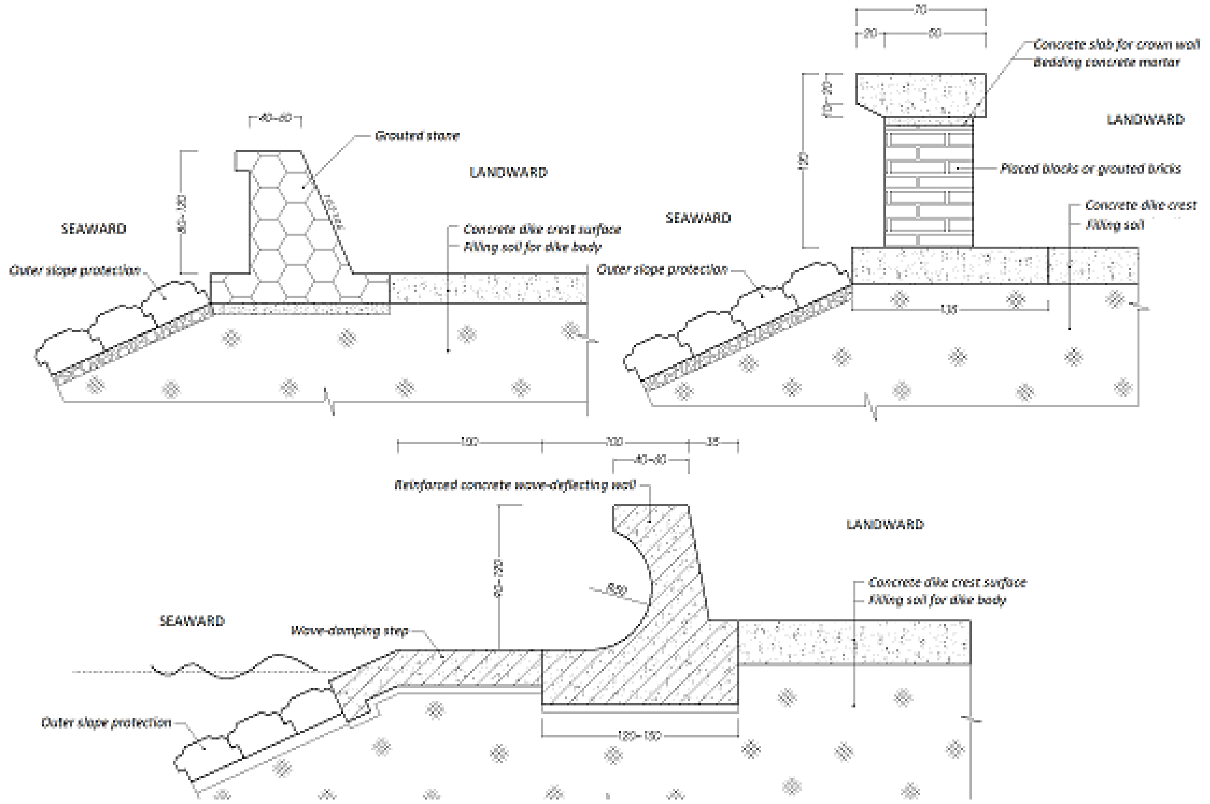


Figure 5.5 *Types of crown wall*

5.6 Dike crest structure

The structures of dike crest must fulfil the technical, economic and service requirements. Dike crest surface should slope either to one side or both sides (at a gradient of 2% - 3%), and drainage system should be arranged on the inner slopes and on dike berms so that rain water and splashed wave foam can be gathered to surface drainage trenches.

In case the dike crest also functions as transportation route, it must be designed as per the technical standards of road ways (see TCVN 4054-2005). If not, protective solutions against erosion due to rain water and overtopping water must also be adopted.

5.7 Design of transition structures

Transition structures are placed between the dike elements such as: dike body and the toe; dike foundation and dike body; dike body and the outermost revetment, etc. (see ***Fig. 5.6***). Transitional parts must meet the technical and aesthetic requirements.

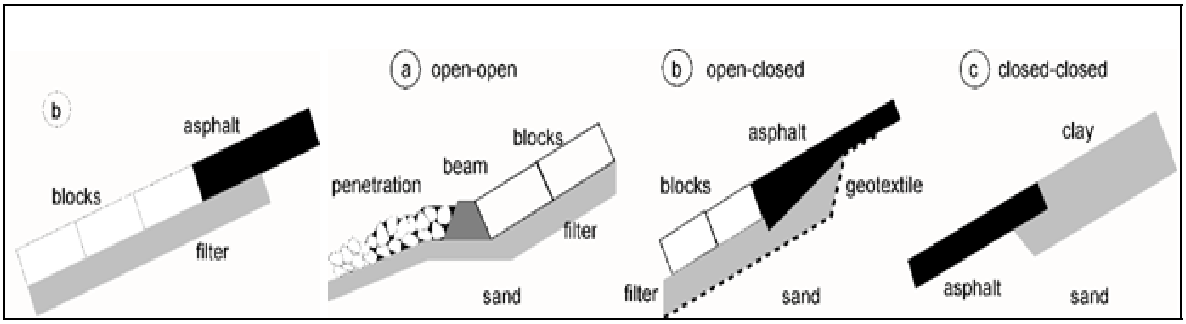


Figure 5.6 *Some types of transition structures*

5.8 Dike slope

5.8.1. Dike slope angle

Dike slope angles should be determined through the stability requirements, taking the cross-section shape, types and material of slope protection, embankment materials, wave energy, geological conditions, construction conditions, service requirements etc. into consideration.

In case of earthen dikes, the preliminary slope coefficients can be determined as follows:

+ $m = 2,0 \div 3,0$ for inner slopes;

+ $m = 3,0 \div 5,0$ for outer slopes.

The selection of preliminary slope coefficients of sea dikess must be examined by means of stability calculation and wave run-up height, from that appropriate values can be determined.

In case of steep outer slopes (for example $m = 1.5 \div 2.0$), the wave run-up height is greater and therefore the crest level increases. However, if the outer slope is too gentle, the construction volume will be enormous. Therefore, in case of large sea dikes, the selection of appropriate slope coefficients is usually performed by means of technical and economic analyses.

If the dikes are embanked on soft soil foundation, berms can be placed on both slopes for high dike body and for the purpose of stability enhancement. These berms can fulfil the requirements of transportation, maintenance and flood control.

5.8.2 Outer berm

Seaward dike berms or wave-attenuating berms are applied in the areas with severe conditions of waves and wind in order to reduce the wave run-up height and to enhance the stability of dike body.

Outer dike berms are usually introduced at the Design Water Level. The width must be greater than 1.5 times of the incident wave height and should not be less than 3 m, and should not exceed 0,25 time of deepwater wave length ($B_{cơ} \leq 0,25L_0$).

Berms can be designed with a slope of $1/15 \div 1/20$ and in combination with the drainage system. On outer dike berm, wave-attenuating blocks can be placed in order to attenuate wave run-up, to dissipate wave energy in front of dike crest, to enhance the stability and safety of the design dike route.

In case of important dike system, the crest level and wave-attenuating berm dimensions must be determined by means of experiments using physical models.

5.8.3 Inner berm

Sea dikes are normall under frequent impacts of waves, tides and storm surges from the sea. Furthermore, the difference in water levels on seaward and landward sides is insignificant, so does the dike height; thus dike berms are prioritized on outer slopes for the sake of their effect in wave attenuation

Only in special cases and for specific purposes, the inner berms are considered. For example, When the dike height is greater than 6 m, the inner slope coefficient is less than 3 ($m < 3$) with traffic demand, the inner berms can be introduced at the level which is $2 \div 3$ m from the dike crest. The width depends on the traffic requirements but should not be less than 5 m. The lower slope is usually gentler than the upper one.

5.9. Dike body and foundation

5.9.1. Embankment materials

The routes of sea dike and revetment go through different regions with variable geological conditions, and require enormous volume of materials. The usage of local and in-situ materials has an economic significance. Maximum use of the embankment soil from the nearby areas should be made. For the homogeneous earthen dikes, the clayey soil with clayey content of 15% ÷ 30%, the plasticity index of 10 ÷ 20% without dirt should be used. The allowable difference between the water content of embankment soil and the optimum water content should not exceed ±3%.

Alluvial silty soil, clay with high natural water content and excessive clay particles, swelling soil and the dissolved soil should not be used for the embankment. In case these types of soil must be used, it is necessary to adopt appropriate technical solutions.

If only loose sand with the fine grain content of 25% is available, cover layers are required (heavy soil layer with a minimum thickness of 0.5 m can be used).

5.9.2. Dike body compaction criteria

For cohesive soil: $K_c = \dfrac{\gamma'_{ds}}{\gamma'_{d\max}}$

where,

K_c - Design compaction degree;

γ'_{ds} – Design dry density of the dike body soil;

γ'_{max} – Maximum dry density, determined in the laboratory;

For non-cohesive soil: $K_{ds} = \dfrac{e_{\max} - e_{ds}}{e_{\max} - e_{\min}}$

where,

K_{ds} – Design relative compaction degree;

e_{ds} – Design compact void ratio;

$e_{max.}$ e_{min} – Maximum and minimum void ratios determined by the standard test.

Required compaction degree of embankment soil is given in ***Table 5.5***.

Table 5.5. *Required compaction degrees of dike body*

Sea dike grade	I	II & III	IV & V
K_c	≥0.94	≥0.92	≥0.90
K_{ds}	≥0.65	≥0.62	≥0.60

In case sea dikes and revetments also function as traffic routes, the compaction of dike body must conform to the design guidelines of road way (TCVN 4054-2005: *Requirements of road way design and 22 TCN 333-05*)

5.9.3. Dike foundation and technical solutions

Dike foundation must ensure the stability (in terms of stresses and deformation, seepage, etc.) under the impacts of active loads. In case the natural dike foundation does not meet the design requirements and standards, additional appropriate solutions for treatment must be applied, such as counter-pressure prism, replacement of soft soil layers, geotextiles or other measures (see ***Appendix F***).

5.10. Calculation of sea dike stability

5.10.1. Introduction

The calculation of sea dike stability must be performed as per current standards and codes of earthen structures and hydraulic works. The following contents must be taken into special consideration:

- Stability of dike slopes against sliding (seaward and landward) ;
- General stability of dike body and foundation;
- Settlement of dike body and foundation;
- Stability against seepage

In addition, designers should also pay attention to other specific contents,

5.10.2. Calculation of stability against slope sliding

- *Section of cross-section:* the selected section must be representative based on the dike functionality, dike grade, topographical conditions, geological conditions, dike structure, dike height, embankment material, etc.

- *Specific cases:*

+ Under normal (working) condition: The inner slope at high tide; the outer slope during rapid falling tide and other base load combinations depending on the detailed conditions;

+ Abnormal (inspecting) conditions: The inner and outer slopes during construction stage; the inner and outer slopes bear the loads at the design water level and other special load combinations depending on the detailed conditions.

Note: In case of dikes built at the areas with heavy rainfall, the stability against sliding of the dike slope during rainy periods needs to be inspected thoroughly.

- *Calculation method:* The calculation is performed as per Technical Codes for rolled earth-filled dam design (14TCN157-2005[1]) and other related Technical codes and Standards. GEO-SLOPE/W software package can also be used in the calculation.

- *Stability safety factor against sliding (K):* Required stability safety factor against sliding is shown in ***Table 5.6***.

Table 5.6: *Required safety factor against sliding for dike slope*

Dike grade / Load combination	I	II	III	IV	V
Base	1,30	1,25	1,20	1,15	1,10
Special	1,20	1,15	1,10	1,05	1,05

- *Stability safety factor against planar sliding (K):*

Table 5.7. *Required safety factor against sliding on non-lava foundation*

Dike grade / Load combination	I	II	III	IV	V
Long-term	1,35	1,30	1,25	1,20	1,15
extreme	1,20	1,15	1,10	1,05	1,05

+ For concrete or grouted stone structures, the safety factor against planar sliding on the interface with non-lava foundation is described in ***Table 5.7***.

+ For concrete or grouting stone structures, the safety factor against planar sliding on the interface with the lava foundation is described in ***Table 5.8***.

Table 5.8. *Required safety factor safety factor against sliding on lava foundation*

Dike grade / Load combination	I	II	III	IV	V
Base	1,15	1,10	1,10	1,05	1,05
Special	1,10	1,05	1,05	1,00	1,00

- *Stability safety factors against overturning (K)*: Safety factors against overturning are given in ***Table 5.9***:

Table 5.9. *Stability safety factor against overturning*

Dike Grade	I	II	III	IV	V

Load combination					
Base	1,6	1,5	1,5	1,3	1,3
Special	1,4	1,3	1,3	1,2	1,2

**** Notes:***

- *Basic loading is the combination of loads under normal working conditions of the structure;*
- *Special loading is the combination of loads during the construction stages or seismic activities;*
- *The calculated actual safety factors should not exceed 20% for basic loading condition and 10% for the special loading condition.*

5.10.3. Calculation of vertical-typed sea dikes

For sloping or vertical-typed sea dikes, the stability is based on gravity. The stability calculation must be performed according to the following 5 items:

+ Stability against overturning;

+ Stability against general sliding;

+ Stability against planar sliding;

+ Stability of subsoil (in terms of stresses, deformation, seepage, etc.).

a) Stability against planar sliding

a.1) In case of no cohesion on sliding plane

The stability factor against sliding along the base of buffer layer are determined by the following formula:

K_{at} = **Error!**

where,

G - Vertical resultant force exerted on the sliding surface (KN);

P - Horizontal resultant force exerted on the sliding surface (KN)

g - Weight of the material of buffer layer and counter-pressure blocks in the range CD÷EE' (kN);

P_E - Passive soil pressure on surface EE' (kN).

F - Friction coefficient;

a.2) In case of cohesion on sliding plane

The stability factor against sliding along the base of buffer layer are determined by the following formula:

K_s = **Error!**

where,

φ_o - Internal angle of friction between the wall bottom and the foundation base;

C_0 - cohesive force on the sliding surface;

A - Area of wall bottom (m^2);

b) Stability against overturning

Examination should be performed in order to check if the structure is unstable due to overturning around the heel seawards in case of small pressures exerted by seaward water and waves, high groundwater level on rear side.

Furthermore, the stability against overturning landwards should aslo be examined in case of construction duration with high sea level, low embankment on the rear side.

Stability factor against overturning can be determined as follows:

$$K_{at} = \frac{M_C}{M_G}$$

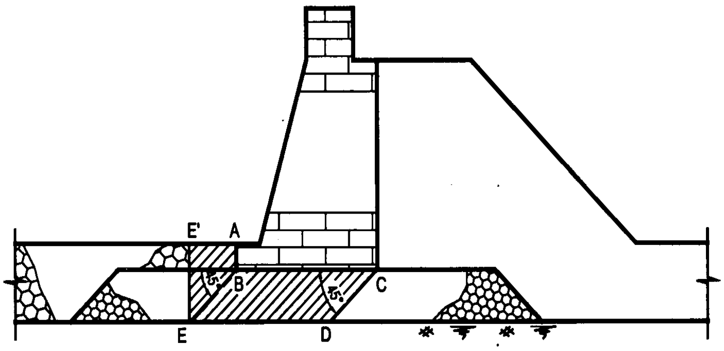


Figure 5.7 *Diagram for the calculation of vertical-typed sea dikes*

where, M_C - Stability moment against overturning;

M_G - Overturning moment around the front edge.

5.10.4. Calculation of crown wall stability

In case of sloping sea dikes, when the base of crown wall is located in the range of wave-runup, it will bear the impacts of waves. Therefore the stability of crown wall must be examined in this case. The horizontal and vertical components of wave pressure are the principal loads exerted on crown wall. In addition, the impacts of wind (pushing and sucking loads) and passive earth forces (if any) should also be taken into consideration.

Guidelines on wave pressure exerting on crown wall of sloping dikes are given in the Appendix. For steep or vertical-typed sea dikes, based on the water depth in front of dike toe and the water depth at the dike toe protection structures, types of wave and appropriate formulae can be selected. The wave pressure diagram exerting on crown wall is determined as per the general diagram on the entire front of sea dike and is used in order to calculate the wave pressure.

The calculation of crown wall stability includes the calculation of stability against sliding and overturning with the load combination corresponding to Design Water Level and design wave height.

6. DESIGN OF SLOPE REVETMENT AND DIKE TOE

6.1 Sea dike slope protection

The outer (seaward) slope of sea dike is under direct impacts of waves, wind and currents. There are two main types of slope reinforcement: grass and revetment.

The revetment includes: crest, outer protection layer and dike toe. Other supplementary components include: buffer layers or filter layers, wave-attenuating berms, wave-dissipating slabs, wave-deflecting crown wall and deformation control joints.

6.1.1 Selection of protection structures

6.1.1. Outer (seaward) slope

The selection of slope protection types depends mainly on many factors such as topography, geology, hydrodynamics etc. in deciding the material and structure of revetment body, toe and crest, which is appropriate to hydrodynamic conditions as well as the economic and aesthetic requirements. This must be made by means of analysis and calculation.

Based on the economic and technical conditions, types of evetment can be selected, as shown in ***Table 6-1***.

Common types of revetment are:

- Paved quarry rocks;

- Precast concrete blocks;

- Combination of precast concreted slabs (on the lower slope) and paved quarry rock (on the upper slope)

Table 6.1. *Types of revetment and applicable conditions*

No.	Slope protection structures	Applicable conditions
1	Grass	- Wave height $H_s \leq 0,5$m, current v < 1m/s or the mangroves are planted on the foreshore; - The dike slope is favourable for grass to grow;
2	Turbulent dumped quarry rock	- Abundant rock supply; - Gentle dike slope, low aesthetic requirements.
3	Paved quarry rock	- Abundant rock supply, available type of rock meeting the requirements; - Dike foundation with good drainage.
4	Jointed quarry rock	- Rather good dike slope; - High wave Hs > 0,5m, strong current V > 1 m/s, the loose type of rock does not meet the requirements
5	Gabion layers	- The rock supply is limited; - High wave, strong current; - Salinity-proof steel gabions.
6	Precast and loose-joined concrete elements	- High wave, strong current; - Aesthetic requirements;
7	Precast, array-joined concrete slabs	- High wave, strong current; - Aesthetic requirements. - Dike slope with little subsidence and drainge; - Available conditions for construction and

8	Combination of many structures	array manufacturing. - Large variance of water level, long revetment slope; - Different service requirements

- *Some applicable slope protective structures*

Some applicable types of dike slope protective structures include (see ***Table 6-2***):

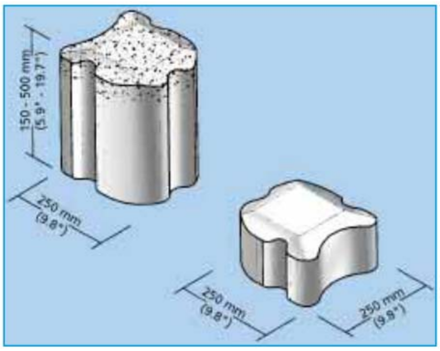
- Independently-placed elements;

- Array-joined elements;

Table 6.2 *Types of slope protection elements made of precast concrete*

Type of structural elements	Shape	Structure of surface exposed to waves	Linking mode
Independently-placed elements	- Column-typed - Hexagonal - T-shaped - Rectangular	- Smooth ; - Concave opening; - Convex abutment; - Drainage openings ; - Smooth, with drainage openings;	Close joining
Array-joined elements	- Rectangular - Hexagonal	- Smooth; - Convex; - Drainage openings;	- Cable-threaded - Jointing slit; - Positive-negative patterns

Recent applied types of independently-paved precast concrete elements show their effectiveness in the world, especially in the Netherlands. For example Basalton® và Hydroblock® elements (see ***Fig. 6.2***). Details of these elements are given in ***Appendix H***.



(a) Basalton®elements *(b) Hydroblock® elements*

Figure 6.2 *Some recent types precast concrete slabs paved independently*

Precast concrete elements of these types in reality are shown ***Appendix H.***

Elements of outer dike slope protection will be designed in terms of thickness using Pilarczyk's formula. In the calculation of protective layer thickness and extent, effect of wave-attenuating structures in front of sea dikes and geometric structure of dike slopes must be taken into consideration.

6.1.1.2. Inner slope protection

Based on rainfall intensity, requirements of overtopping, dike height, soil properties, service requirements etc., types of revetments can be selected. Grass is a common solution in protecting inner slopea; forming beautiful landscape and being environmentally-friendly.

6.1.2. Protection layer thickness

Current design policy is that the protective layers for outer dike slope is designed in terms of layer thickness instead of the weight of protective elements.

For sea dike design, the thickness of slope protection layers can be determined by Pilarczyk's formula (*Pilarczyk, 1990*)

$$\frac{H_s}{\Delta_m.D} \leq \Psi_u.\Phi.\frac{\cos\alpha}{\xi_p^b} \qquad \textit{(for } cotg\alpha > 1{,}5\textit{)}$$

or $$D \geq \frac{H_s}{\Psi_u.\Phi.\Delta_m.\cos\alpha}.\xi_p^b$$

In terms of strength – load: $$\Delta_m.D = \frac{H_s.\xi_p^b}{\psi_u.\phi.\cos\alpha}$$

where, Ψ_u- System-determined (empirical) stability upgrading factor (see ***Table 6.3***);

+ Ψ_u = 1.0 for riprap as a reference;

+ Ψ_u > 1 for other revetment system;

Table 6.3 *System-determined (empirical) stability upgrading factor (Ψ_u)*

Types of revetment	System-determined (empirical) stability upgrading factor (Ψ_u)
2-layer riprap (standard value)	1,0
Natural rock revetment with low quality (non-uniform dimensions);	1,0
Natural rock revetment with good quality (uniform dimensions);	1,5
Loosely-place elements;	1,5
Basalton, Hydroblock;	2,0
Grouted elements (30%);	1,5
Bituminous rock	2,0
Gabions;	2,5
Armorflex	2,5

Φ - Stability factor or stability function for incipient of motion;

For loose aggregates, Φ can be more generally defined using the Van der Meer's formula (1984) as follows:

$$\Phi = 6.2P_b^{0,18}.\left(\frac{S_b^2}{N}\right)^{0,1} \quad (\xi < 3) \quad \textit{(breaking waves)}$$

With: P_b - Notional permability factor of core materials;

+ P_b = 0,5 for breakwaters

+ $P_b = 0,1$ for revetments;

S_b - Initial damage level;

+ $S_b = 1 \div 2$ for independently-placed precast concrete elements;

+ $S_b = 3$ for paved and dumped rocks;

N - Number of incident waves during a storm;

ξ_p - Breaker similarity index on a slope (or Iribarren number) corresponding to peak wave period (T_p);

$$\xi_p = \frac{\tan\alpha}{\sqrt{s_0}} = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_0}}}$$

This formula is applicable for $\xi_p \le 3$ (breaking waves). In case $\xi_p > 3$ the dimensions for $\xi_p = 3$ can be used;

With α - Slope angle (degree);

H_s - Design (significant) wave height (m);

D - Specific size or thickness of protection unit (m);

Δ_m - Relative density of a system-unit (-);

D and Δ_m are defined for specific system as follows:

- Rock: $D = D_n = \sqrt[3]{\frac{M_{50}}{\rho_b}}$

$$\Delta_m = \Delta = \frac{\gamma_b}{\gamma_n} - 1$$

- Blocks: D = block thickness and $\Delta_m = \Delta$;
- Mattresses: D = d = average thickness of mattress and

$$\Delta_m = (1-n).\Delta = (1-n).(\frac{\gamma_b}{\gamma_n} - 1)$$

With: n – Bulk-porosity of fill material;

Δ – Relative density of fill material.

For common quarry rocks: $\Delta_m = (1-n).\Delta = 1$

b - Exponent related to the interaction process between waves and revetment type (roughness, porosity/permeability etc.) (*0,5 ≤ b ≤ 1,0*);

+ b = 0,5: for rough and permeable revetments as riprap;
+ b ≈ 1: for smooth and less permeable revetments;
+ b ≈ 2/3: Common representative value for other systems (i.e more open blocks and block-mats, mattresses of special design etc.)

6.2.3. Drainage openings and deformation joints

6.2.3.1. Drainage openings

The function of drainage openings is to reduce the uplift pressure on the slope protection elements. Drainage openings are arranged in alternate patterns with the opening diameter of 5 ÷ 10 cm. The distance between the openings of 2 ÷ 3 m.

6.2.3.2. Deformaton joints

Based on the stability calculation, appropriate distance between the deformation joints can be determined, normally the required distance is 5 ÷ 15 m.

6.2. Dike crest

6.2.1. Dike crest without crown wall

In case the dike crest is designed without crown wall, it is necessary to introduce the trafic barriers on both edges of dike crest in order to ensure the traffic safety, with a height of 0.2 ÷ 0.3m; the barriers can be placed discontinuously with segment length of 0,5 ÷ 1,0 m.

6.2.2. Dike crest with crown wall

In the areas with high waves, concrete, precast concrete, reinforced concrete or jointed-stone crown walls are usually introduced on the seaward edge of dike crest (see ***Fig. 6.4***).

In some cases, the crown walls are placed on the inner edge of dike crest for the purpose of using dike crest as a berm in order to reduce the impacts of high waves, and thus considerably lower the crest level. In this case, the traffic route is placed on the inner slope.

The distance between 2 settlement joints of crown wall is $15 \div 20$ m for (reinforcced) concrete walls and $10 \div 15$ m for other materials.

The height of crown wall should not exceed 1 m and the foundation should be independent of the revetment. Crown walls should be built only after the dike has become stable, ensuring the requirements of load bearing, stability against overturning and sliding, permeability and foundation bearing capacity as per current stipulations.

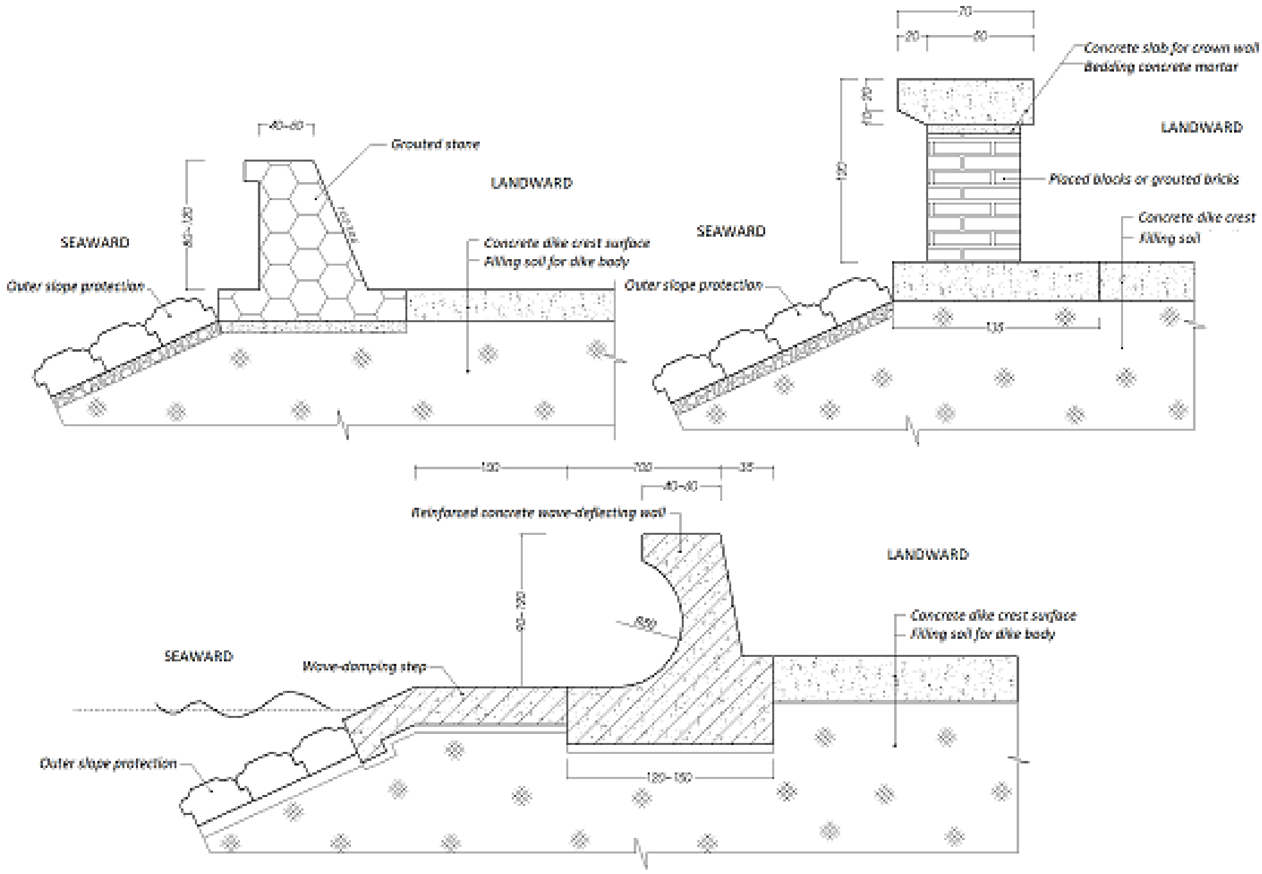


Figure 6.4 *Types of crown wall structures*

In reality, in order to enhance the wave-attenuating efficiency, wave-deflecting nose must be introduced on crown wall, also the frontal wave-attenuating platform with wave-attenuating blocks (see ***Fig. 6.5***).

Figure 6.5 *Crown wall with wave-deflecting nose and wave-attenuating blocks on the frontal platform*

6.3. Buffer layers and filter layers

In order to prevent the loss of embankment soil from the dike body under the impacts of waves and seepage flows, and to function as a buffer layer for the revetment, a buffer layer or filter layer should be introduced between the dike body and the revetment.

Filter layers have many functions: preventing the surface erosion of foundation soil, or preventing the generating of uplift pressures in the foundation soil layer (water drainage), or a combination of the two above-mentioned functions.

Buffer layer for the revetment can combine the functions of a filter layer (inverted filter layer), using traditional materials (loose materials) or geotextiles.

Inverted filter layers normally consist of 2 or 3 layers of loose materials, with increasing diamters in the direction of seepage flow from the dike body outwards.

6.3.1. Granular filter layer

Design of granular filter layer should be taken in sequence and the technical requirements of similar regulations for dam design (Earth Dam design standard).

- The design of filter layers must satisfy the following conditions:

+ Stability: $\frac{d_{15F}}{d_{15B}} < 5$

+ Internal stability: $\frac{d_{60}}{d_{10}} < 10$

+ Permeability: $\frac{d_{15F}}{d_{15B}} > 5$.

where,

d_F – diameter of soil particle in filter layer;

d_B – diameter of soil particle in base layer.

- Thickness of each filter layer d_0 is determined by the following formula:

$$d_0 = 50.d_{15}$$

or empirically:

+ Inner layer: d_{02}= (10÷15) cm;

+ Outer layer: d_{01}= (15÷ 20) cm;

6.3.2. Filter layer formed by geotextile

Geotextiles are placed closely to the embankment soil. There are a gravel buffer layer between the geotextiles and the revetment in order to prevent the rip or tight filling of geotextiles caused by outer protection elements.

Main functions of geotextile are: separating, filtering, strengthening, water guiding and drainage. Arrangement of filter layer structure with geotextile is shown in ***Fig. 6.8***.

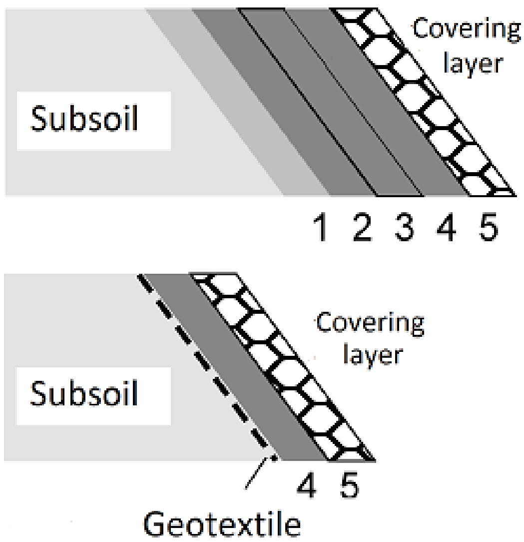


Figure 6.8 *Arrangement of geotextiles in filter layer*

- *Note:* Geotextile can change its shape due to the Ultraviolet (UV),and it can be eroded and pierced by chemical, biological or mechanical processes.

In case the embankment soil consists of dust or clay with a content greater than 50%, the fine grains can go through the opening of geotextiles or fill the openings and increase the hydrostatic pressure in the dike body under the impacts of currents. It is then necessary to place a coarse sand layer between the dike body and geotextile layers with a thickness of 15÷20 cm. The advantages of using sand buffer layer and geotexiles as common components in the filter layer is the lower costs and simpler control of construction quality.

In using geotextiles, it is necessary to select the appropriate types with required technical standards based on the grading of embankment soil in order to fulfil the requirements of water drainage and soil retention; and also to examine the stability against sliding along the surface of filter layers.

The design of filter layer using geotextile is performed as per "*Guidelines for design and usage of geotextile for filtering in hydraulic works*" 14TCN 110-1996[1].

6.4. Dike toe

6.4.1 Definition

Revetment toe (or cutoff) is the transitional structures between the dike slope and the foreshore or the inner base of sea dike, with the following main functions: supporting and enhancing the stability of the revetment against sliding along dike slope; strengthening the foreshore in order to rerduce the impacts of currents, waves; and preventing the erosion of dike toe erosion.

6.4.2 Types of dike toe

6.1.2.1 Seaward dike toe structure

The selection as well as the calculation of dike toe dimensions must be performed on the basis of principal factors governing the rate of dike toe erosion, such as: wave breaking (near the dike toe), wave run-up and run-down on dike slopes, wave reflection, physico-mechanical properties of foreshore materials, especially the grain diameters. Therefore, it is necessary to analyse the actual situation of foreshore erosion, wave parameters and dimensions of slope protection layers.

** Note*:

- Footing level of outer dike toe must ensure the stability of the dike cross-section (including the dike toe), i.e. sufficient depth in order to prevent the impacts of scour on the dikes;

- Depending on the properties of dike foundation and the dike structure, appropriate solution for the treatment of dike foundation can be employed. Pile foundation can be applied in order to improve the bearing capacity of the dike base and to enhance the stability of dike toe in case of mud, silt, sand or clay sand. Based on the physico-mechanical properties of dike foundation and construction conditions, appropriate types of pile can be selected, for example bamboo piles, steel piles, concrete piles etc.

In case the enlargement of horizontal protection extent of dike is possible and the protective depth of dike toe is not too great, no piling is needed for the foundation improvement.

There are two common types of dike toes, including:

- Shallow dike toe: applied to protection type that covers the width of predicted scour; usually applied with normal sloping beach area;
- Deep dike toe: applied to protection type that covers the predicted scour depth; recommend to apply to steep sloping beach area under large waves, in which the shallow toe is not possible to applied

a. Shallow dike toe

Shallow revetment toes are applicable at the areas where the foreshore erosion rate is low; the revetment toes only withstand the impacts of waves and wave-induced currents at the dike toe.

Principal types of shallow dike toe protection are as follows:

- Covering platform (footing) or supporting abutment;
- Buried footing (buried trenches);
- Quarry mattress or placed gabions;

Footing-typed revetment toes are made of quarry rocks, dumped in covering platform at the dike toe forming submerged or emerged footings as a connection to the upper revetment. According to Shore Protection Manual (1984), the footing thickness is equal to $1 \div 2$ times of the revetment thickness for emerged footing, and $1 \div 2$ times of the revetment thickness for submerged footing.

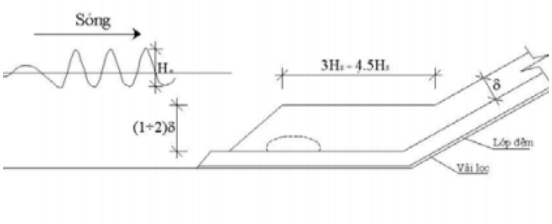
Suppporting abutment is a combination of emerged footing (with large dimensions) and submerged footing (small dimensions), which is applicable in the areas with low foreshore (see ***Fig. 6.9***).

Burried trenches is usually applied in the areas with high foreshore (in case of low foreshore quarry prisms can be used). The footing depth is determined on the basis of topographic conditions of foreshore, wave height, currents etc.

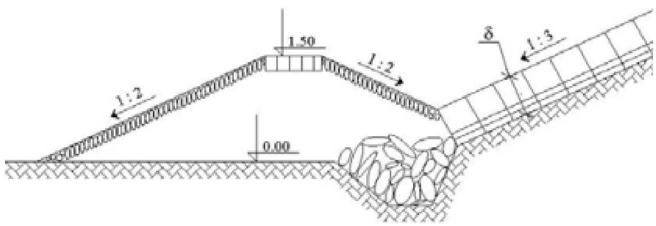
In low foreshores, it is difficult to place the footing down to the required depth. This problem can be solved by supplementing with the prisms against erosion close to the footing.

Mattresses are usually made of quarry rocks or placed gabions with efficiency in moving the scour far away from the dike toe, reducing the threat of dike toe or outer slope erosion. According to Shore Protection Manual (1984), the length of quarry mattress must be $3 \div 4$ times of significant wave height in front of dike toe (for emerged mattress), and $2 \div 3$ times of significant wave height in front of dike toe (for submerged mattress).

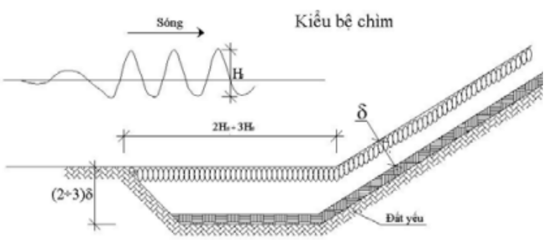
Some common types of shallow dike toe protection are shown in ***Fig*. *6.9***.



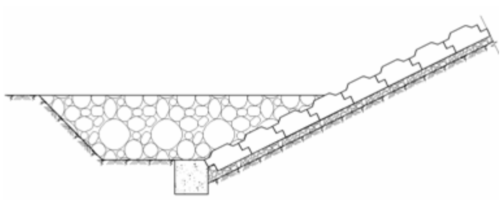
Emerged footing



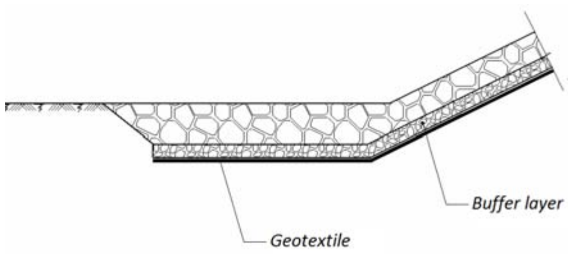
Supporting abutment (mixed type)



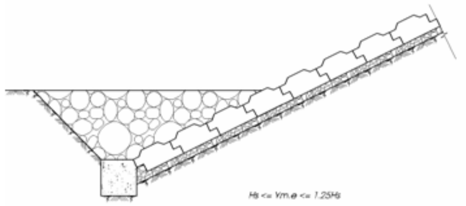
Submerged footing



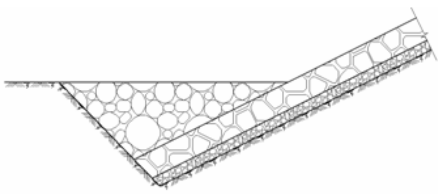
Submerged footing with locking beam



Submerged footing without locking beam



Buried footing with locking beam



Buried footing without locking beam

Figure 6.9 *Types of shallow dike toe protection*

In general, applied protective materials of shallow revetment toes are: turbulent dumped rocks, concrete element or loose-grained materials etc. Quarry rocks are highly appropriate materials in dike toe protection due to their flexibility. In case geotextiles are placed below the footing, the extent of geotextile must not exceed the footing width in order to maintain a flexible distance (at least 1 m) for the prevention of undermining, or can be rolled in the opposite direction (landward) and then covered with rocks and sand.

b. Deep dike toe

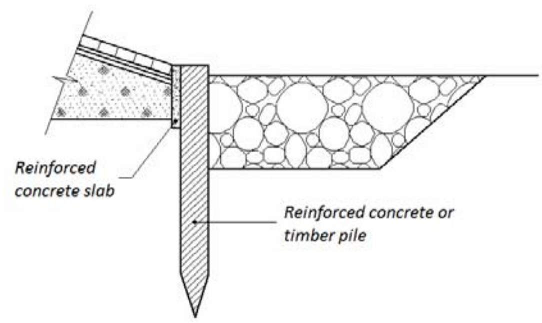
Deep revetment toes are applicable at the areas where the foreshore erosion is severe, ensuring the stability of dike toe in case the foreshore is eroded seriously. Common types of deep revetment toe protection are as follows:

- Timber, concrete or steel sheet piles, in combination with quarry prisms;
- (Reinforced) concrete cylinders (one or more layers) in combination with quarry rocks and quarry prism;
- Mixed types;

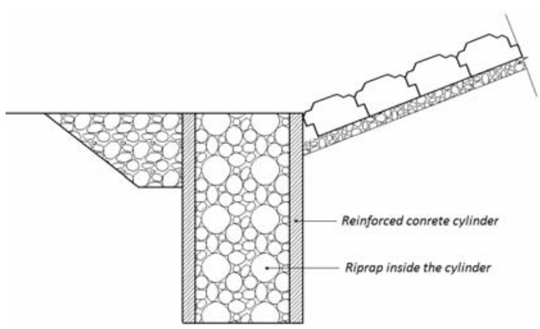
Cylinders have been used in Vietnam since 1992 (Ha, 2003) in order to enhance the stability of seaward dike toe. Current cylinders have length of $(1 \div 2)$ m, with round or hexagonal cross-sections with inner diameter of $(0{,}8 \div 1{,}2)$ m and quarry core. Cylinders are fabricated in situ and then placed vertically down to dike toe using manual labour or machinery.

In the design of deep revetment toe, the limit of water depth in front of the dike toe and the stability of revetment must be determined. If the foreshore is likely to be severely eroded resulting in the water depth in front of the dike toe exceeding the limited depth, appropriate solutions must be adopted in order to reduce the water depth in front of the toe, such as groynes or beach nourishment. Revetment toe must penetrate into natural subsoil to a minimum depth of 1.0 m.

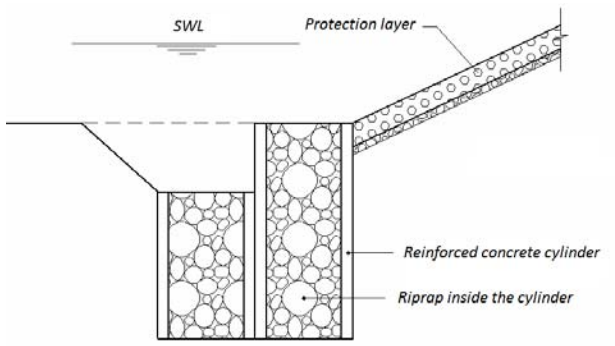
Some common types of deep dike toe protection are shown in ***Fig. 6.10***.



Sheet piles + quarry prism (footing)



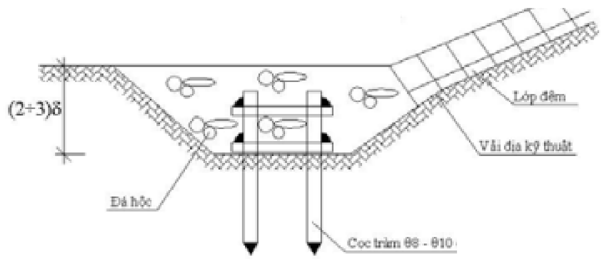
Single cylinder + quarry prism (footing)



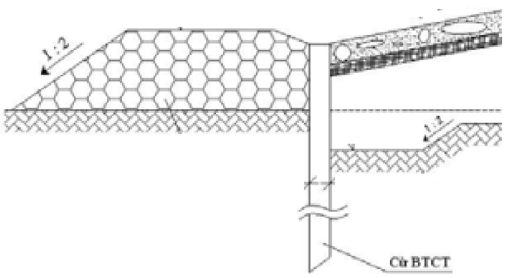
Double cylinders + quarry prism (footing)

Figure 6.10 *Types of common deep dike toe protection*

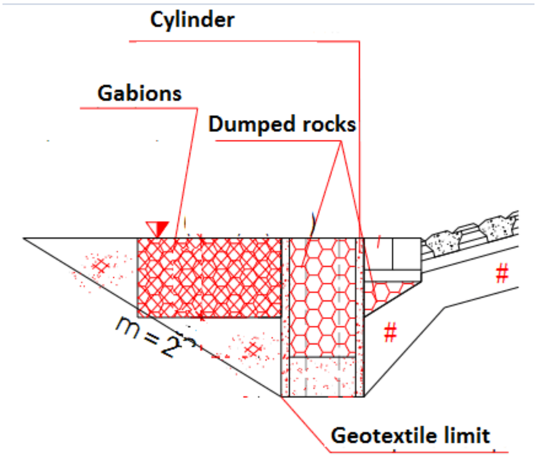
Mixed types of deep dike toe protection is a combination of sheet piles and emerged/submerged footings, or sheet piles and gabions & quarry prisms (see ***Fig. 6.11***).



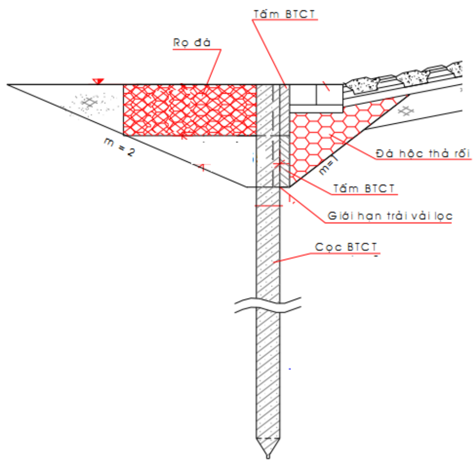
Combination of submerged footing and timber piles



Combination of emerged footing and (reinforced) concrete piles



Combination of cylinders and gabions



Combination of sheetpiles and gabions

Figure 6.11 *Mixed types of deep dike toe*

Table 6.4 is a summary of protection types for dike toe with applicable conditions, advantages and disadvantages for each types.

Table 6.4 *Summary of protection types for dike toe*

	Shallow dike toe	*Deep dike toe*
1. Applicable conditions	- Stable foreshore with accretion potential; - Eroded foreshore from low to medium rate; - Insignificant nearshore currents; - Foreshore made of sand.	- Eroded foreshore from medium to high rate, large scour depth; - Strong nearshore currents; - Foreshore made of cohesive or clay sand
2. Structural types	- Covering platform (footing) or supporting abutment; - Buried footing; - Rock mattress or placed gabions.	- Sheet piles made of timber, concrete or steel, in combination with prisms made of quarry rocks - (Reinforced) concrete cylinders (one or more layers) in combination with quarry core and quarry prism. - Mixed types (combination of submerged/emerged footings + sheet piles)
3. Materials	- Quarry rocks; - Concrete and reinforced concrete; - Geotextile.	- Quarry rocks; - Timber; - Concrete and reinforced concrete;
4. Advantages	- Flexibility of quarry	- Efficiency of sheet piles in

	rocks; - Making use of local materials; - Simple structures and construction; lower costs; - Enhancement of filtering capacity in case of seepage flow from dike body, especially when geotextiles are incorporated.	improving the bearing capacity of soft soil foundation and in enhancing the stability of dike toe against horizontal and vertical sliding due to their deep penetration into the foundation. - Prevention of seepage flow from the sea into dike foundation.
5. Disadvantages	- Low efficiency in prevention of displacement or deformation of revetment; less stable. - No efficiency in prevention of seepage flow from the sea into dike foundation.	- Difficulty in construction; requiring more complex machinery and equipments; higher costs. - Cylinders are vulnerable to sliding or overturning after some time when there is no seaward rock mattress or gabions

Details of protection types for sea dike toes as well as the illustration of common types in Vietnam are given in ***Table H-3***, ***H-4*** and ***H-5*** in ***Appendix H***.

6.4.2.2 Structural solutions for inner dike toe

Inner dike toes are mainly under impacts of overtopping flow from dike crest. Therefore, the selection and calculation of appropriate structural solutions for inner dike toes must be based on overtopping volume and corresponding types of slope protection, as well as the drainage capacity on landward side.

There are now poor variety of inner dike toe protection. The common types include:

- Dike toe without protection;
- Jointed rock footing;
- Sheet piles (in case of ponds, lakes or canals which are located very close to dike toe).

These common types of inner dike toe protection are illustrated in ***Fig. H-15*** and ***H-16*** in ***Appendix H***.

In case of sea dikes allowing overtopping, the drainage system must be designed in an appropriate way, for example the drainage system along inner dike toe, counter-pressure and drainage prisms at the dike toe etc.

6.4.3. Critical scour depth at dike toe

Critical scour depth of the revetment toe depends on the wave energy and geological conditions of foreshore foundation at the dike toe. The following empirical formula can be used:

$$S_{max} = 1{,}5.H_s$$

where

S_{max} - Equilibrium scour depth (m);

H_s - Design wave height (m);

L_s - Design wave length (m);

Based on the scour depth, the depth of the revetment toe and the extent of protective mattress can be determined. The required protective depth for dike toe is at least 1.25 times of scour depth. The width of seaward protective mattress is at least $3 \div 4$ times of design wave height from the dike toe. In case of combined protective solutions, the depth and width of protective structures for dike toe must be determined on the basis of protection types and in specific cases

6.1.4 Determining the dimensions of dike toe protective materials

In reality, quarry rocks are commonly used in the counter-pressure prisms or protective mattress for dike toe protection. The main parameter having impacts on these rocks is the bed current velocity generated by the wave run-down. In low water, these currents cause severe erosion at the dike toe, thus the protective rocks must have sufficiently stable weights. These weights depend mainly on wave parameters and water depth at the design location.

The stable weigth of quarry rocks for dike toe protection ($G_đ$) can be determined as per ***Table 6.5***.

Table 6.5 *Maximum current velocities (V_{max}) and corresponding stable weights of quarry rocks for dike toe protection*

V_{max} (m/s)	2,0	3,0	4,0	5,0
$G_đ$ (kG)	40	80	140	200

in which V_{max} is the maximum wave-induced current velocity at the dike toe, determined by the following formula:

$$V_{max} = \frac{\pi.H_s}{\sqrt{\frac{\pi.L_s}{g}.\sinh\frac{4.\pi.h}{L_s}}}$$

V_{max} - Maximum current velocity (m/s);

L_{sp}, H_{sp} - Design wave length and wave height (m);

h - Water depth in front of the dike (m);

g - Gravity acceleration (m/s^2);

Design wave length (L_s) can be determined using the look-up table of wave parameters presented by Wiegel (1964);

* *Note*:

- In shallow water areas (with the raito h/L < 0,05), the design wave length can be determined as: $L = T\sqrt{gh}$;

- When using ***Table 6.5***, other values of rock weights can be interpolated or extrapolated from corresponding values of maximum current velocities at the dike toe.

7. DIKE-CROSSING AND AUXILIARY STRUCTURES

7.1. Dike-crossing structures

Dike-crossing structures are specifically designed as per related specialized standards and codes for each types (sluice, bridge, spillway, navigation lock etc. However, the construction project setup must conform to the planning approved by competent authorities, especially in case of dike-crossing sluices. In addition, the transition between the dike body and dike-crossing structures must be treated smoothly, ensuring the stability, safety of these structures.

7.2. Dike pathways in combination with dike management and guarding routes

Dike pathways in combination with dike management and guarding routes are usually combined with seaward and landward dike berms. They must be wide enough to arrange machinery and equipments, as well as materials if necessary, with the minimum width is $2 \div 3$ m. In case of transportation routes, these must ensure the dike safety and conform to the planning, standards and codes in transportation.

7.3. Surface water drainage system

Drainage gutters are usually introduced on dike crest, dike slope and dike toe, also at the interface between dike slope and earthen bank or other structures. Drainage gutters along the dike are placed at the inner edge of dike berm or dike toe, connected by cross drainage gutters which are located at a distance of 50 – 100 m from each other. Applicable materials for drainage gutters are concrete, grouted bricks or rocks etc. Their dimensions and bed slopes are determined by calculation or empirically.

7.4. Managerial buildings

Buildings for sea dike management is designed specifically as per the standards and codes in construction. These buildings are normally located close to sea dikes in order to facilitate the management and rescue activities, Therefore, they withstand frequent direct impacts from the sea, especially the impacts of wind and erosion caused by the coastal climate. The selection of materials, structures and directions etc. must ensure the longterm stability and safety in stormy conditions. Apart from managerial buildings, stockpiles of materials to be used for rescue and frequent maintenance should be introduced.

7.5. Observation posts for sea level

The location of these posts must ensure the frequent obvervation of sea level, long-term stability and facilitation during the management and monitoring. Therefore, they should be attached to dike-crossing structures, such as bridges, sluices (if any), estuarine areas.

In addition, the designer must consider the following items depending on specific requirements:

7.6. Boat/ship passing way over the dikes;

7.7 Steps on dike slopes;

7.7. Dike open section gate;

7.9. reserved materials areas

When hard structures are constructed on dikes, special attention should be paid to transitions between the embankments and the structures;

8. FORESHORE PROTECTION STRUCTURES

Foreshore protection is very important to the safety of sea dikes, especially in case of eroded foreshore. The foreshore can be protected by different solutions depending on the causes of erosion, location of structures and specific characteristics of the study area. Common solutions are:

- Planting mangrove forests for the purposes of wave attenuation and foreshore retaining;
- Building groynes, breakwaters etc.
- Artificial beach nourishment.

The detailed design of the above-mentioned solutions must conform to the specific guidelines and technical standards related to each type of solution. This section only give general guidelines and principles. Note that in the stage of setting up construction investment project, the proposed solutions must satisfy the following criteria: effective erosion alleviation, no disadvantageous impacts on nearby areas, limitation to environmental vulnerability, taking advantage of the availability of local materials, and appropriateness to general plan approved by authorized agencies.

8.1 Mangrove forests

8.1.1. Functions and requirements

Mangrove forests have effect on wave attenuation, prevention of coastal and estuarine erosion, improvement of sediment deposit, marine environmental and ecological protection.

In case wave attenuating effect of mangrove forests is taken into consideration in the design of sea dike cross section, the maintenance is compulsory to maintain the design status of mangrove forests throughout the service life.

8.1.2. Applicable conditions for mangrove forests

8.1.2.1. Favourable areas

Mangroves grow steadily on mud flats, coastal and estuarine areas, alluvial deposit with much organic silt. Appropriate types of mangrove can be selected and planted depending on the conditions of climate, rainfall, soil and water salinity, topography and geology.

8.1.2.2. Hardly favourable areas

Mangroves are difficult to grow in deepwater areas, with high waves, frequent foreshore erosion; in areas with nutrient-poor bed soil, in which sand accounts for more than 90%. Therefore, the following solutions should be adopted prior to mangrove planting in these areas:

- Using seedlings planted in soil pots, with sufficient height and holding poles, in combination with the structures having functions as wave shields, foreshore formation and stabilization in the areas with high waves (see Appendix G).

- Local improvement of planting holes in the areas with innutritious bed soil.

8.1.2.3. Unfavourable areas

Mangroves cannot be planted and grow in coastal areas with the water depth greater than 3 m; no exposing time for the foreshore or very little (< 4 hours/day); duration of tidal submergence is less than 10 days or greater than 20 days/month; within the period of severe erosion; unstable bed, highly dilute mud; areas with annual salinity is greater than 30‰ and the areas polluted by oil spill, domestic and industrial refuse.

8.1.3. Design of mangrove planting

8.1.3.1. Selection of mangrove types

Each type of mangrove adapts to specific conditions of topography, geology and salinity, and can be selected depending on the conditions in each area (see ***Table 8-1***).

Table 8-1. *Conditions for foreshore and appropriate types of mangrove*

No.	Foreshore conditions	Type of mangrove
1	- Unstable alluvial deposit; - Continuous tidal submergence during the month; - Harsh natural conditions, high waves, strong wind, frequently high salinity (≥ 30‰)	*Avicennia marina* *Sonneratia alba*
2	- Newly-formed alluvial deposit, high salinity; - Submerged mud flats during tides, with frequent impacts of wind and waves	*Avicennia alba*
3	- Medium submerged foreshore, stable bed, duration of tidal submergence is 24 - 26 days per month	*Rhizophora apiculata* or *Avicennia officinalis*
4	- Estuarine brackish water areas, frequently low salinity (≤ 15‰)	*Sonneratia caseolaris* *Nypa fruticans* *Acanthus ilicifolius*
5	- High submerged foreshore, duration of tidal submergence is 15-22 days per month	*Excoecaria agallocha* *Lummitzera racemosa* *Acanthus ilicifolius*
6	- Seldom submerged banks of lagoons and ponds, duration of tidal submergence is 5-7 days per month. Các bờ đầm	*Thespesia populnea*

Appropriate types of mangrove for different coastal regions (based on natural conditions) are as follows:

North East Region: From Ngoc Cape to Do Son Cape:

- Sub-region 1: From Mong Cai to Ong Estuary:

Avicennia marina, *Aegiceras corniculatum*, *Rhizophora stylosa*, *Kandelia candel*, *Bruguiera gymnorrhiza*.

- Sub-region 2: From Ong Estuary to Luc Estuary:

Avicennia marina, *Aegiceras corniculatum*, *Rhizophora stylosa*, *Kandelia candel*, *Bruguiera gymnorrhiza*, *Lummitzera racemosa*).

- Sub-region 3: From Luc Estuary to Do Son Cape:

Avicennia marina, *Aegiceras corniculatum*, *Rhizophora stylosa*, *Kandelia candel*, *Bruguiera gymnorrhiza*), *Sonneratia caseolaris*, *Acanthus ilicifolius*, *Hibiscus tiliaceus*, *Derris trifoliata*.

North Delta Region: From Do Son Cape to Lach Truong Estuary:

- Sub-region 1: From Don Son Cape to Van Uc Estuary: *Sonneratia caseolaris*, *A. corniculatum*, *Acanthus ilicifolius*.

- Sub-region 2: From Van Uc Estuary ot Lach Truong Estuary, within the accretion areas of Red Rivers system:

A. corniculatum), *Acanthus ilicifolius*, *Sonneratia caseolaris*, *Kandelia candel*, *A. marina*.

North Central Coast Region: From Lach Truong Estuary to Cape of Hai Van Pass:

- Sub-region 1: From Lach Truong Estuary to Ron Cape:

Sonneratia caseolaris), *Acanthus ilicifolius*, *Derris trifoliata*, *A. marina*, *Rhizophora stylosa*, *Kandelia candel*, *A. corniculatum*, *Bruguiera gymnorrhiza*.

- Sub-region 2: From Ron Cape to Cape of Hai Van Pass:

A. marina, *Rhizophora stylosa*, *Kandelia candel*, *Bruguiera gymnorrhiza*, *A. corniculatum*, *Sonneratia caseolaris*, *Acanthus ilicifolius*, *R. mucronata*, *Lummitzera racemosa*, *Derris trifoliata*, *Rhizophora stylosa*.

South Central Coast Region: From Cape of Hai Van Pass to Vung Tau Cape:

R. mucronata, *R. apiculata*, *Bruguiera gymnorrhiza*, *Bruguiera paviflora, Sonneratia caseolaris*, *Acanthus ilicifolius*, *A. alba*, *A. offcinalis*, *Hibiscus tiliaceus*, *Excoecaria agalocha*, *Phoenix paludosa*, *Derris trifoliata*.

South East Region: From Vung Tau Cape to Soai Rap Estuary:

Sonneratia alba, *R. mucronata*, *R. apiculata*, *Kandelia candel*, *Derris trifoliata*, *A. corniculatum*, *A. alba*, *A. officinalis*, *Ceriops tagal*, *C. decandra*, *Excoecaria agalocha*, *Phoenix paludosa*, *Sonneratia caseolaris*, *Acanthus ebracteatus*, *Nypa fruticans*, *Hibiscus tiliaceus*, *Thespesia populnea*.

Cuu Long River Delta Region: From Soai Rap Estuary to Ha Tien:

- Sub-region 1: From Soai Rap Estuary to My Thanh Estuary (coastal area of Cuu Long River Delta)

A. alba, *Sonneratia alba*, *R. mucronata*, *R. apiculata*, *Bruguiera paviflora, Ceriops tagal*, *A. marina*, *A. lantana*, *A. officinalis*, *C. decandra*, *Excoecaria agalocha*, *Phoenix paludosa*, *Sonneratia caseolaris*, *Acanthus ilicifolius*, *Nypa fruticans*, *Hibiscus tiliaceus*, *Thespesia populnea*, *Lumnitzera racemosa*, *Derris trifoliata*.

- Sub-region 2: From My Thanh Estuary to Bay Hap Estuary (South West of Ca Mau peninsula):

A. alb, *A. marina*, *A. officinalis*, *R. mucronata*, *R. apiculata*, *Ceriops tagal*, *C. decandra*, *Excoecaria agalocha*, *Acanthus ilicifolius*, *Nypa fruticans*, *Lumnitzera racemosa*, *Bruguiera sexangula*.

- Sub-region 3: From Bay Hap Estuary (Ba Quan Cape) to Nai Cape – Ha Tien (West Coast of Ca Mau peninsula):

A. alba, *A. marina*, *A. officinalis*, *Sonneratia alba*, *R. apiculata*, *Ceriops tagal*, *Bruguiera gymnorrhiza*, *Excoecaria agalocha*, *Sonneratia caseolaris*, *Nypa fruticans*, *Thespesia populnea*, *Lumnitzera racemosa*, *Derris trifoliata*.

8.1.3.2. Determination of wave-attenuating coefficient

Basis for the calculation of mangrove forests protecting sea dike is their wave-attenuating effect. Mangrove forests with their root and foliage system absorb a part of wave energy. The energy dissipating process is due to the barrier created by mangroves, in which the foliage exerts friction and inertia on the moving water body. The energy dissipation has two mechanisms: wave motion interacting with mangrove forests and bed friction. Thus, after the wave propagation through the mangrove forests, wave height and wave velocities will be attenuated.

Each type of mangrove forest has specific density and foliage coverage (depending on the plant height, plant diameter, number of branches/plants etc.), resulting in different capacities in wave attenuation. Based on those criteria, mangrove forests can be divided into 3 states: thick, medium and sparse (see ***Table 8-2***).

Table 8-2. *Mangrove forest states corresponding to different densities and foliage coverage*

Density	**Mangrove foliage coverage (%)**					
	100	95	90	85	80	75
20.000	thick	thick				
16.000	thick	thick	thick			
12.000	thick	thick	medium	medium		
8.000	medium	medium	medium	medium	medium	
4.000	medium	medium	medium	medium	medium	sparse
3.000	medium	medium	medium	medium	sparse	sparse
2.000	medium	medium	medium	medium	sparse	sparse
1.500	medium	medium	medium	sparse	sparse	sparse
1.000	medium	medium	medium	sparse	sparse	sparse

* *Notes*: - Density (N): number of plants per hectare.

- Foliage coverage (TC): percentage of total foliage projection area on horizontal surface and the ground area.

Above-mentioned classification of mangrove forest status is the basis for the calculation, design of mangrove forests in each specific area and the requirements of wave attenuation of sea dike.

**** Determination of wave-attenuating coefficient and width of mangrove forest belt***

- The reduction of wave height is normally interpreted by the value of K_t (wave-attentuating coefficient):

$$K_t = \frac{H_d}{H_o}$$

where, H_d - Wave height at the dike toe;

H_0 - Wave height in front of mangrove forest belt;

- According to Quartel (Quartel et.al, 2007), wave-attenuating coefficient (R) can be calculated as follows:

$$R = \frac{H_0 - H_d}{H_0} = 1 - K_t \quad (1)$$

Both K_t and R depend on the width of mangrove forest belt (x) and status of mangrove forests. On the other hand, each status of mangrove forest is characterized by a value of wave-attenuating parameter (r).

The correlation between the coefficient K_t and r is shown by the following formula:

$$K_t(x) = e^{-rx} \quad (2)$$

- Wave-attenuating coefficient (K_t) can also be determined by the following formula devised by Vuong Van Quynh et al. (2010) as follows:

$$K_t = e^{-bx} \quad (3)$$

Equation (3) is identified with equation (2), meaning that value of b in the formula of Vuong Van Quynh et al. (2010) can be identified with the value of r in the formula of Quartel (2007).

- From actual surveys and analyses, wave-attenuating parameter (r) for different status of mangrove forest can be determined as per ***Table 8-3***.

Table 8-3. *Wave-attenuating parameter (r) corresponding to different status of mangrove forests*

Status of mangrove forest	Wave-attenuating parameter *r*
Thick	0.010
Medium	0.007
Sparse	0.004

Figure 8-1 illustrate the wave-attenuating coefficient (K_t) corresponding to different width of mangrove forest belt in different actual status.

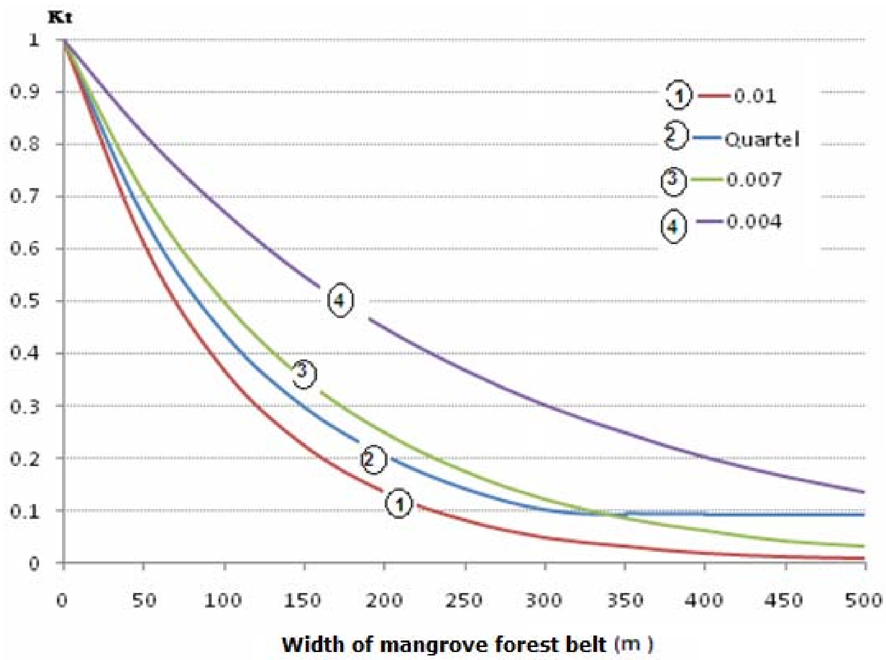


Figure 8-1. *Relation between the mangrove forest belt and wave-attenuating coefficient*

It can be seen that the curves illustrating correlation between wave-attenuating coefficient and width of mangrove forest belt are equivalent to the calculation results made by Quartel (2007).

Curve (1) is used for thick mangrove forests; curve (3) for medium mangrove forests; curve (4) for sparse mangrove forests. Curve (2) is the calculation results made by Quartel.

Thus with available status of mangrove forests (thick, medium or sparse), the wave-attenuating coefficients (K_t) corresponding to certain width of mangrove forest belt can be determined, and can be used in the design of sea dikes in those areas.

In case of newly-planted mangrove forests or under certain specific conditions, guidelines for sparse mangrove forest or the calculation results made by Vuong Van Quynh et al. can be applied (see the attached appendix).

8.1.4 Planning mangrove forest belts in natural ecological succession

The formation and growth of mangrove forest are always accompanied by the formation and evolution of deposit areas. Therefore, appropriate types of mangrove must be selected for each area based on natural conditions.

In case mangrove forests are planted in landward direction, different types of plant are selected and arrange based on the natural ecological succession of mangrove with 3 main belts, in the following orders:

- First (primary) belt, including original plants which are appropriate in the following conditions:

+ *Avicennia alba* Bl.: appropriate to dilute mud flat, and yearly salinity ≥15‰;

+ *Sonneratia caseolaris*.: appropriate to mud and sandy flat, salinity from 5 to 15‰ in rainy season and close to estuaries;

+ *Avicennia marina*: appropriate to sandy flat with little mud, hight yearly salinity;

- Second (secondary) belt, including plants on dense silt, with prop-typed roots: *R. apiculata*, *Kandelia candel*, *trifoliata* etc.

- Third belt, including plants above mean tidal level, with knee-typed roots: *Hibiscus tiliaceus*, *paviflora*, *Phoenix paludosa* etc.

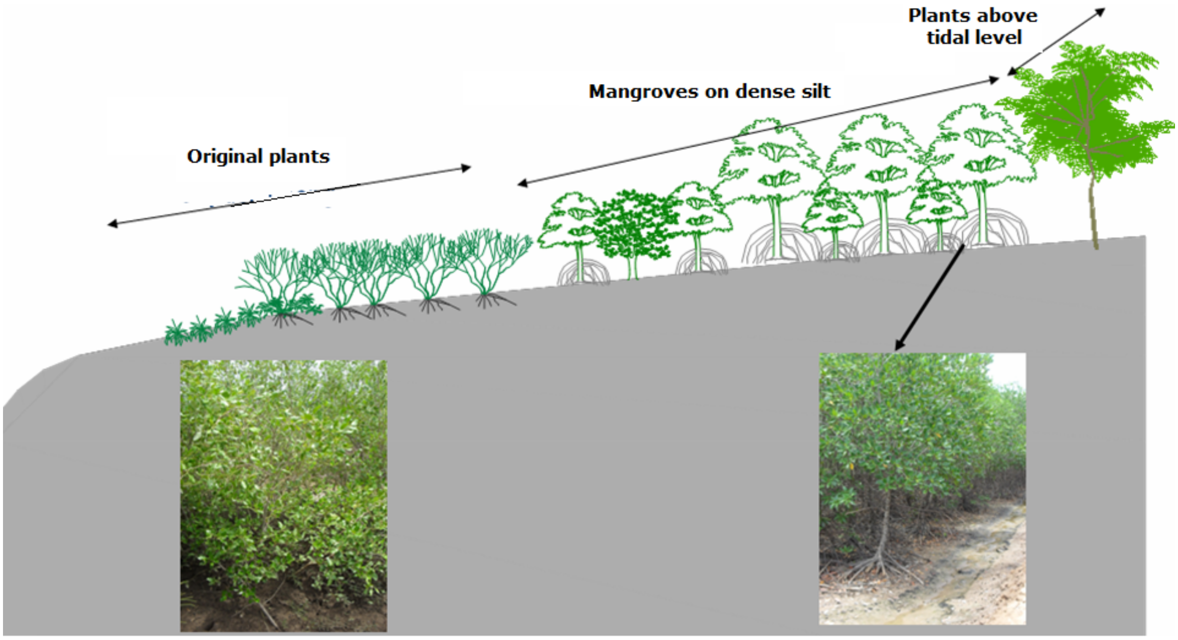


Figure 8-2. *Natural ecological succession of mangroves*

In case of seawards gentle foreshore and normal weather, climate and nearshore hydrodynamic conditions, the beaches are aggraded annually and the trend of mangrove succession is towards the degradation of old groups, giving places to new groups on built-up and more tightly adhesive bed. It results in the trend of above-mentioned forest belts towards the gradual seaward shifting. However, when natural conditions change (bed, salinity, sediment) or due to human activities (chopping, aquaculture, fishing etc.), some types of mangrove are degraded and vanish completely, and there exist only simple mixed forests or homogeneous forests. In addition, the regenerating capacity of mangrove forest will be limited or cannot occur. If the average life cyle of a mangrove is about 20-25 years, after this period all of the original mangroves will grow older, be degraded and the forest belts become sparser and thinner.

In case old mangrove is being degraded, primary foliage is above high tidal level, the roots are also degraded and become unstable, the volume of plants submerged in tidal water is inconsiderable. Therefore, the wave-attenuating capacity of old and stunted plants is limited and they are likely to collapse. On the other hands, the beds with their age of several ten years in the mangrove forests will be aggraded and much more elevated (normally at high tidal level), and consequently they are severely attacked and seriously eroded, many mangroves are pushed over. In such areas, high tides and waves will erode the soil layer aggraded during the formation of mangrove forest, resulting in the deeper settlement of the foreshore, which is similar to that in initial state without mangrove forests.

Similarly, in case of mixed forests or homogeneous forests without regenerating capacity and no annual supplementary planting, the forest belts will be degraded and vanish after only a period of several ten years, so does the erosion.

Therefore, appropriate solutions must be adopted for each section of mangrove forests:

- *In case of natural forests:* establishing a corridor for regenerating plants at the seaward edge of mangrove forests by means of stabilizing and protecting the foreshore, with no permission for fishing in these areas, especially in regenerating season of original plants. On the other hands, chopping and trimming some old and stunted plants in alternate forest edges and admitting new appropriate plants in those forest edges.

- *In case of planted mangrove forests:* continuing to grow these types of mangrove in order to enlarge the forest belt seawards, and also supplementing the forests with other types of plants.

- *In case of alluvial deposit being accreted without mangrove forests:* planting mangroves and establishing forest belts, which are appropriate to specific conditions of the foreshore.

- *In case of foreshore being eroded*: Revetments, groynes and sea walls must be constructed for beach nourishment and formation as per design parameters. After that, appropriate types of original plants are selected, grown in soil pots meeting the necessary technical standards (dimensions of soil pots: height of 20-40 cm, diameter of 10-30 cm, depending on types of plant) and then planted on the newly-formed foreshore.

Criteria for mangroves to be planted are as follows:

+ *Avicennia*: age of 10-12 months, height of 0.7-1m, stump diameter of 1,0-1,2cm;

+ *Caseolaris*: age of 22 - 24 months, height of 1,2 - 1,5m, stump diameter of 1-2cm;

+ *Kandelia candel*: age of 22 - 24 months, height of 0,7 - 1,05 m, stump diameter of 0,8-1,5 cm;

+ *R. apiculata*: age of 10-12 months, height of 0.7-1 m, stump diameter of 1-1,2cm.

8.1.5 Design of mangrove lines

Design mangrove lines are designed parallel to the coastline and in alternate patterns in order to have optimal effect in wave attenuation. Original plants are grown in the first year. In the following years, plants in the next forest belt are grown. Appropriate densities of plants depend on the requirements of wave attenuation and forest states (density of some primary mangroves: *Sonneratia caseolaris*, *Sonneratia alba* 1.600 plants/ha; *A. alba*, *A. officinalis*, *A. marina* 10.000 plants/ha; *R. apiculata* 10.000 plant/ha).

Waterways for boats should be included in the design of mangrove lines (common width is $50 \div 100$ m) with sufficient distance from shore (in the range of $10 \div 50$ m) in order to prevent cattle.

8.1.6. Determination of wave parameters in case of mangrove forests

Design wave height at the location of structure (see *Section 5.5.4*) considering the influence of mangrove forests is calculated by the following formula:

$$H_s = K_t.\,(1+\sigma).H_{s,k}$$

where,

H_s - Design wave height at the location of structure including the influence of mangrove forests (m).

$H_{s,k}$ - Design wave height at the location of structure without the influence of mangrove forests (m).

K_t - Wave-attenuating coefficient (see Fig. 8-1);

σ - Modelling error coefficient; $\sigma = 0,1 \div 0,3$.

*(Guidelines on the design of mangrove forest planting – see **Appendix G**)*

8.2. Sea groins and Breakwaters

8.2.1. Functions

In case the coast is eroded and mangrove forest cannot be planted, the groyne systems can be applied: conventional groins, T-shaped groins or conventional groins in combination with detached breakwater in order to facilitate the accretion and to prevent the coastal erosion.

8.2.1.1. Sea groins

- Groins are usually applied at the beaches where the nearshore and longshore currents are dominant to reduce those currents.

- Preventing the longshore sediment transport, retaining the sediment for aggrading in the eroded areas.

- Adjusting the coast line, making the direction of nearshore currents adaptive to the incident wave direction, reducing the quantity of drifting sediment.

- Shielding the beach under the impact of oblique waves, creating the tranquil areas where suspended sediments are settled.

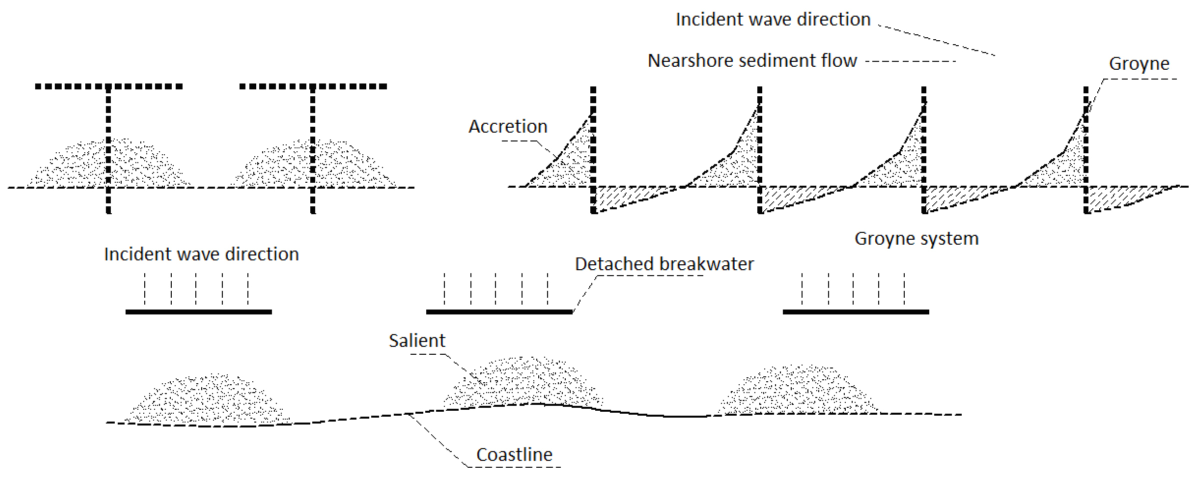


Figure 8.3 *Protective solutions for sea dikes by sand-retaining and wave-damping structures*

8.2.1.2. Detached breakwaters

Shielding the area behind the wall from waves, reducing the wave impacts on coastal areas, and resisting the erosion.

Collecting the drifting sediments in order to form the accreted strip between the wall and the coast, reducing the nearshore currents.

8.2.2. Design of sea groines and detached breakwaters

8.2.2.1. Sea groin system

- Planning for the outer boundary of the groin system needs to be performed, creating a smooth line, well-connected coastline at both sides. The length of a groyne is determined by the breaker zone and the sediment characteristics at the construction site.

- Groine axis is perpendicular to the coastline, or at an angle of $100^0 \div 110^0$ with reference to the incident wave direction.

- Groine foot needs to be well-connected to the stable coastal area, at a level with no impacts of waves and currents.

- Average elevation of groine crest is the same as mean tidal water level, and the crest inclination is the same as foreshore slope.

8.2.2.2. Detached breakwater system

- Depending on the wave-damping requirements, the crest elevation is determined through the relating formula between the wave height and relative height between the wall and the depth.

- Detached breakwaters are usually placed discontinuously with the length in the order of 1,5÷2,5 times of the distance between the wall and the dike toe, the width of the gaps is in the order of 0,4÷0,6 times of the length of a wall segment;

- In complex hydrological and oceanographic conditions, there should be a combination of alongshore and cross shore structures in order to combine the functions of sand-retaining and wave-damping. T-shaped structures are formed by the combination of groin system and detached breakwaters in discontinuous or closely continuous patterns.

8.2.2.3. Configuration and layout of groines

a. Configuration of a groyne: head, trunk and foot (see ***Fig. 8.4****).*

The groynes are extended seawards to reduce the impacts of waves and currents on the coastline, to retain the longshore sediment, to aggrade the area between the two groins, to enlarge and build up the foreshore and to improve the groynes and the coast.

b. Arrangement of groine system

- *Layout routing:* Planning for the new shoreline need to be made for the protected coastal section. This new coastline should be smooth and well-connected with the coastal section with no groynes. The groyne length should not be too small, extending to the breaker zone and the zone of strong nearshore currents.

- *Groyne axis:* Groyne axis is perpendicular to the shoreline. If the wave direction is stable, the groyne axis should be selected as per that direction, which is the most favourable condition for the accretion between the groynes.

Design angle between the wave direction and the groyne axis should be $\delta = 100^0 \div 110^0$, the value $\delta \geq 120^0$ should not be used. α is chosen so that the area of the triangle ABC (see ***Fig. 8.4***) is maximum: α and θ must satisfy the following relation:

$$\alpha = \frac{\pi + \theta}{2} \tag{8.4}$$

With $\theta = 30^0 \div 35^o$, $\alpha = 110^0$;

With $\theta = 60^0 \div 90^o$, $\alpha = 90^0$;

T-shaped groynes can be used in order to improve the wave-damping and accretion effect (see ***Fig. 8.5***).

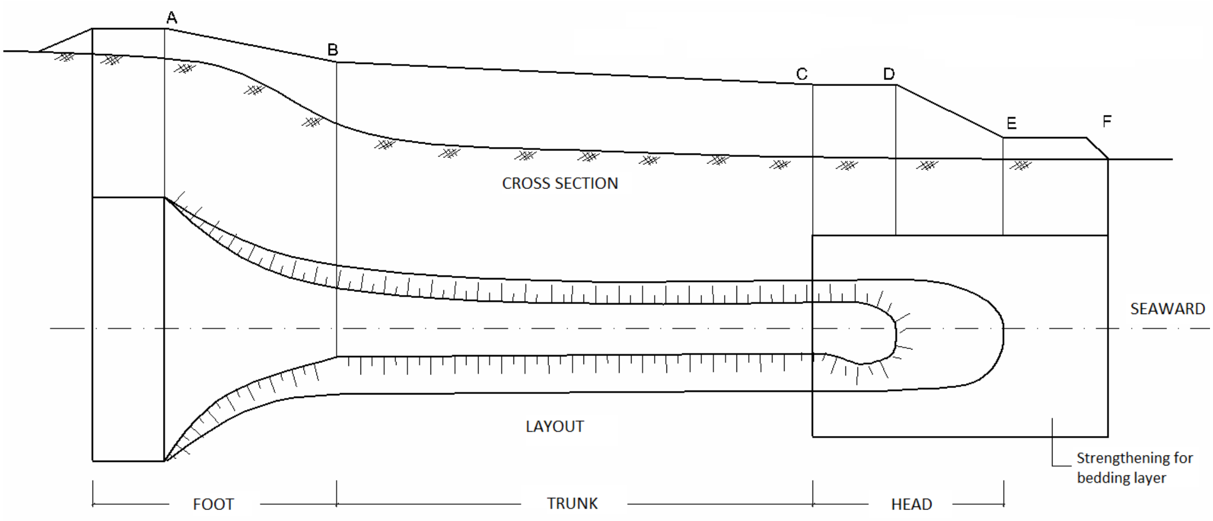


Figure 8.4 *Elements of a groyne*

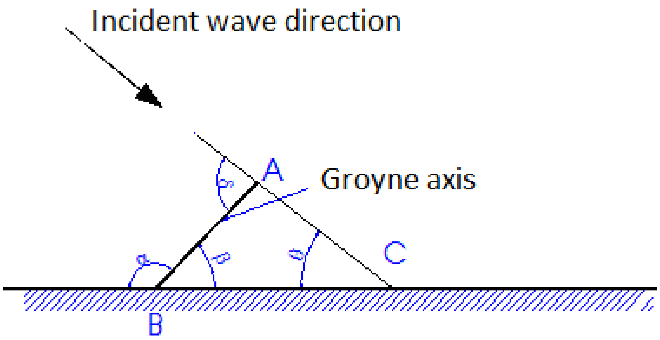


Figure 8.5. *Layout of a groyne*

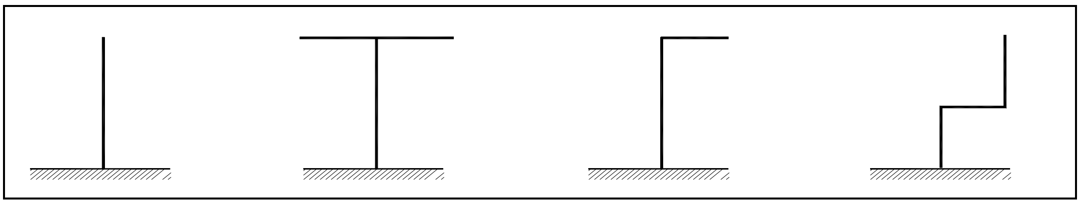


Figure 8.6. *Layout of some types of groyne*

- *Groyne length:* Groynes need to be laid in a system with the length covering the longshore sediment transport zone. The groyne length can be equal to the range of the foreshore to be protected, plus 1/5 of the distance between the two groynes. It is usually 40 ÷ 60m for the small-sized gravel beaches, 100 ÷ 150m for sand beaches.

- *Groyne height:* In general, the higher the groyne is, the more sediment is retained by the groyne system. In fact, however, if the groyne is too high, the wave reflection is greater, which cause more erosion at the toe. For sand beaches, the height of groynes should be 0.5-1.0m larger than the shore surface level. For gravel beaches, the height can be larger.

- *Distance between the groins:* usually 1,5 ÷ 2,0 times of the groyne length for gravel beaches, and 1,0 ÷ 1,5 times of the groyne length for sand beaches. For large-scale projects, testing and surveying must be performed in order to adjust the design approparïately.

- *Protection structures for groin toe and slopes:* determined in similar way as the outer protection of sea dikes (see *Section 6.2*).

8.2.2.4. Layout and configuration of detached breakwaters

a. Configuration of detached breakwaters

Detached breakwaters should be submerged and placed at a certain distance from and usually parallel to the shoreline. Based on the purpose of services, usage of the foreshore area to be protected, compare the economical-technical efficiency of the alternatives in order to decide. The distance between the coast and the walls should be 1,0 ÷ 1,5 times of deep water wave length.

The wall body has uniform cross-section along its length and two sides bearing different loads: seaward side and shoreward side (see ***Fig. 8.7***).

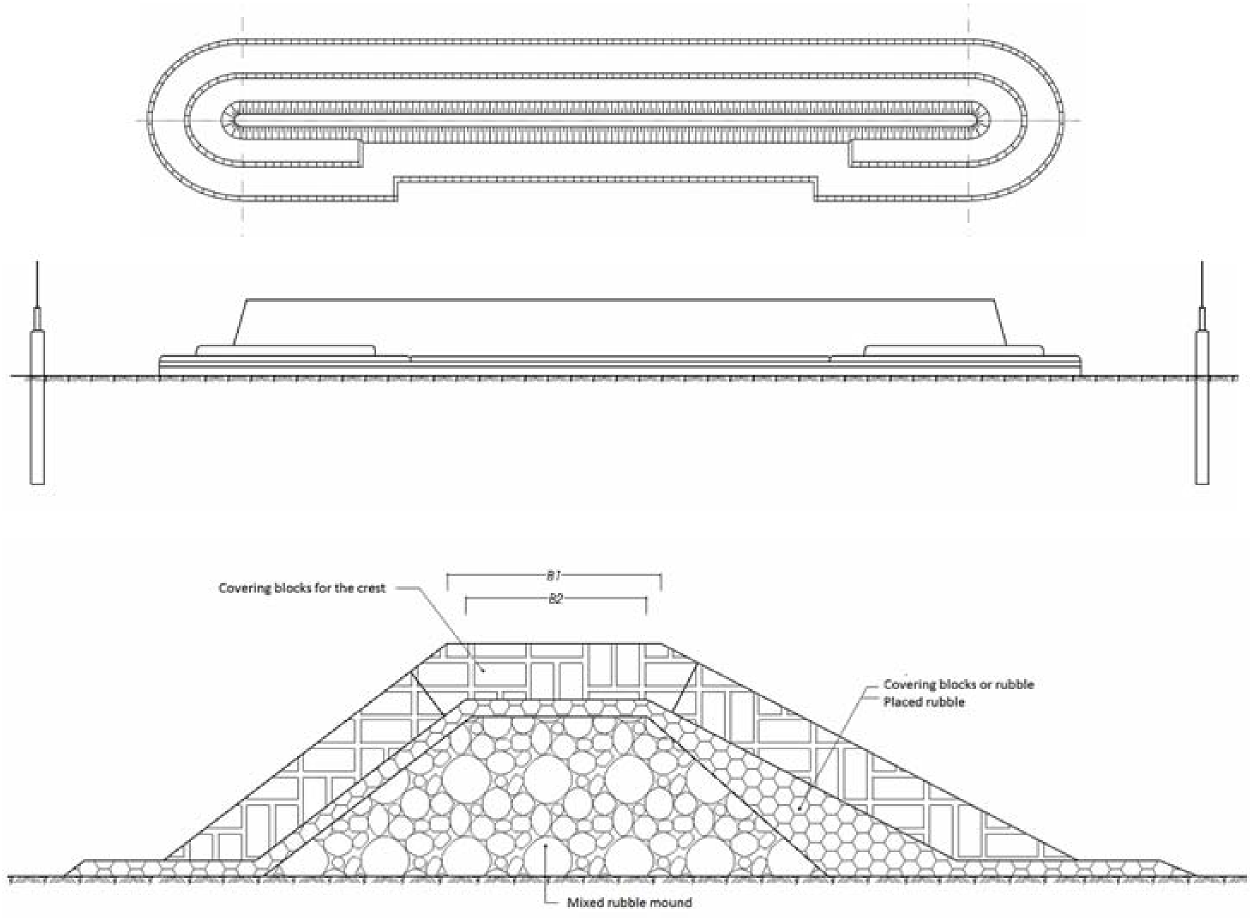


Figure 8.7 *Layout and configuration of detached breakwater*

(a) Layout; (b) Front view from the shore; (c) Cross section

b) Layout of detached breakwater

Detached breakwater can be arranged discontinuously along the protected shoreline, creating the gaps for sediment exchange outside and inside the breakwaters.

Length of the breakwater segment is 1,5 ÷2,5 times of the distance between the breakwater and the shoreline, the gap width is 0,4 ÷0,6 of the length of a breakwater segment and 2 times of the wave length.

- Level of emerged breakwater crest is:

$$H_{Tp} + 1/2\ H_{S\ \text{at the structure location}} + \text{settlement};$$

- Level of the submerged breakwater crest is:

$$H_{Tp} - 1/2\ H_{S\ \text{at the structure location}} + \text{settlement}$$

(usually 0,5 m lower than Design Water Level).

- Crest width of detached breakwaters: determined by means of stability calculation (normally greater than the water depth at the location of structure);
- Slope and toe protection: see ***Appendix H***.

8.3. Beach nourishment

8.3.1. Definition, formation and evolution

Artificial beach-nourishing structures are measures that adopt artificial methods in order to transport the sediments from other areas to fill up the coastal sections to be protected to maintain and improve the beach stability or to create a desired beach and to recover the natural state of the landscape.

8.3.2. Design procedure

(1). Evaluating the present and historical conditions of the coast;

(2). Determining the wave regimes and influence factors;

(3). Verifying the sources, amount and quality of nourishing material;

(4). Evaluating the present structures and proposing the solutions;

(5). Designing the sections for the purposes of protecting the economic and residential structures under the impact of storms;

(6). Planning the monitoring, supervising, maintaining, repairing and re-nourishing processes after the construction.

8.3.3. Main parameters of beach nourishment

+ Khối lượng nuôi bãi lần đầu và khối lượng tăng cường theo định kỳ.

+ Chu kỳ nuôi bãi.

The following main parameters of beach-nourishing structures that need to be determined are as follows (see ***Fig. 8.8***):

+ Shape of cross section: before (actual state) and after nourishment;

+ Nourishing range;

+ Design level and slopes;

+ Elevation, width and slope of foreshore;

+ Initial nourishing volumes and periodical supplementary volumes;

+ Beach nourishment periods.

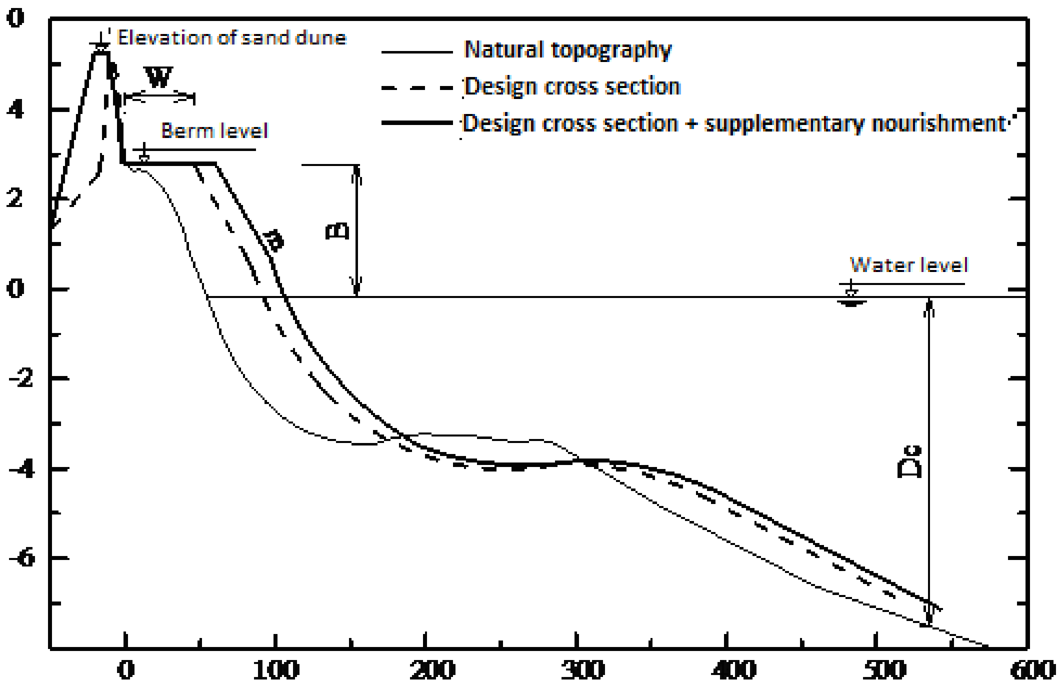


Figure 8.8 *Main parameters of beach nourishment*

9. MANAGEMENT AND PROTECTION OF NATURAL SAND DUNES

9.1 General definitions

9.1.1 Formation and development of coastal sand dunes

Dry sand is blown landward, and accumulated in the area above highest water level and retained by the vegetation layers. These layers grow by means of continuous deposit of sand layers. This process continues and forms the coastal sand dunes. Therefore, the formation and development of coastal sand dunes depend on the following factors: source of sand from the sea; width of dry beach between the high tides; impacts of tides and prevailing landward wind direction.

Sand dune system is a part of the whole section of coastal area and beach which can be distinguished as: coastline, beach and sand dunes. The shape of sand dunes is determined by the following factors: shape of coastline, beach; currents and waves; prevailing wind direction; return period of storms and the coastal sand grading. The displacing rate of each sand dune or the whole sand dune system depend mainly on the following factors:

- Accretion/erosion in the adjoining area between beach and sand dune (reflecting the stored sand in the coastal areas and sand dune areas);

- Equilibrium between vegetation layers and wind stress all over the sand dune system.

9.1.2 Main components of coastal sand dunes

Coastal sand dunes include sand dune area and the beach (main components are given in ***Fig. 9.1***).

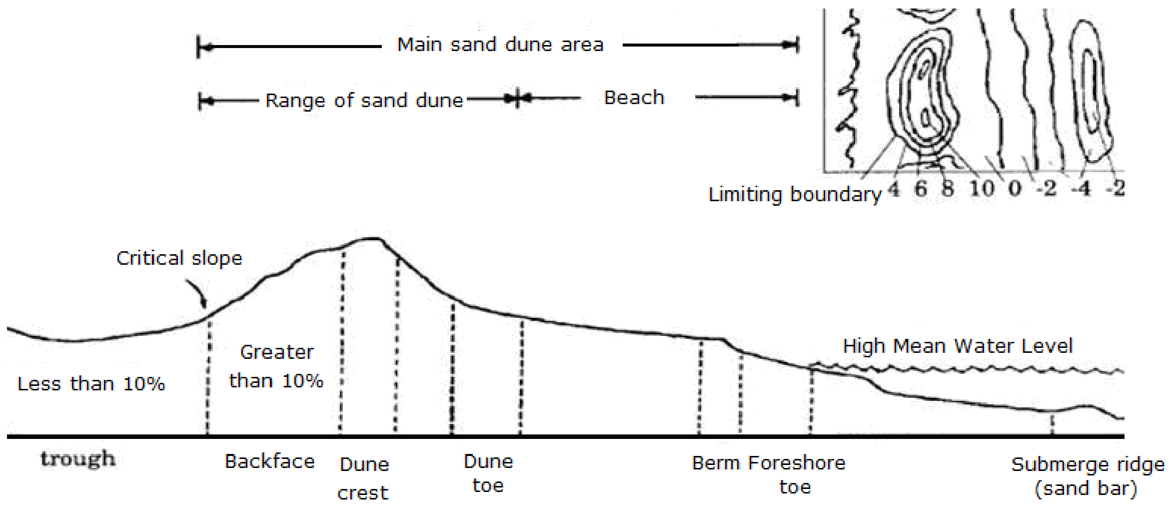


Figure 9.1 *Description of coasta sand dune components*

Main components of coastal sand dunes include:

- *Berm*: a section of the beach in the upper limit of wave loading, formed by the wave-induced sand deposit.

- *Backshore*: the limit between the highest mean water level and the dune toe;

- *Dune toe*: the area at the upper limit of backshore and determined relatively by the considerable variation of slope;

- *Dune crest*: it is the location where the elevation is greatest for a single sand dune. In case of a system of continuous sand dunes, it is the line connecting highest points of the sand dunes along the long axis (normally it is almost parallel to the coastline).

- *Backface*: the area from dune crest to the point where the topographical slope changes and become less than 10%.

- *Dune line*: the line established by some adjacent and continuous sand dunes;

- *Lateral limit*: the end limit of sand dunes which is perpendicular to the coastline or to the dune line;

- *Landward limit*: the circular boundaries of sand dune above the mainland.

9.2 Role of coastal sand dunes

9.2.1 Sand dunes as a coastal protection

9.2.1.1 Protective function

Sand dunes system is considered coastal natural protection structures. The main functions of sand dune system are: alleviating the impacts of storms and high waves; preventing or restraining the landward salt intrusion; retaining the sand for the supplemention in the coastal eroded areas after the storms; protecting the rearward area, functioning as a buffer area against the impacts of erosion.

The most important attribute of sand dunes in terms of coastal protection is the dune crest (above the mean water level and the storm surge level), geometric dimensions and dune width. The ultimate protective function of sand dune system also takes the height and the width into consideration, in addition to the intactness of sand dunes along the coastline. The continuous sand dune system normally has the greatest protection function.

The rate of vegetation layers on the sand dune also play an important part in the protection function of sand dune, as they create the resistance for sand dunes against the impacts of wind and oceanographic factors. Sand dunes are then natural stable sea dike withstanding the wind and waves. These natural dikes can be strengthened by means of height and stability improvement using some structural measures.

9.2.1.2 Assessment level for the protective function of sand dunes

Depending on the geometric characteristics and structure of sand layers, artificial or natural vegetation layers on sand dunes as well as the extent, location of infrastructures (houses, inhabitant etc.) relative to the locations of sand dunes, the assessment level can be proposed for the protective role of coastal sand dunes in terms of natural sea dike system.

The following levels can be considered:

- *Sand dunes with minor protective function*: in this case, the infrastructure and inhabitable areas are located on the rear side, with the average elevation greater than that of dune crest and separated from the sand dunes by the lagoons, small bays, riverine canals – see ***Fig. 9.2(a)***.

- *Sand dunes with medium protective function*: in this case, the infrastructure and inhabitable areas are located on the rear side of sand dunes with the average elevation greater than the highest mean water level and less than the storm surge level – see ***Fig. 9.2(b)***.

- *Sand dunes with major protective function*: in this case, the infrastructure and inhabitable areas are located on the rear side of sand dunes with the average elevation less than or equal to the highest mean water level and much less than the storm surge level – see ***Fig. 9.2(c)***.

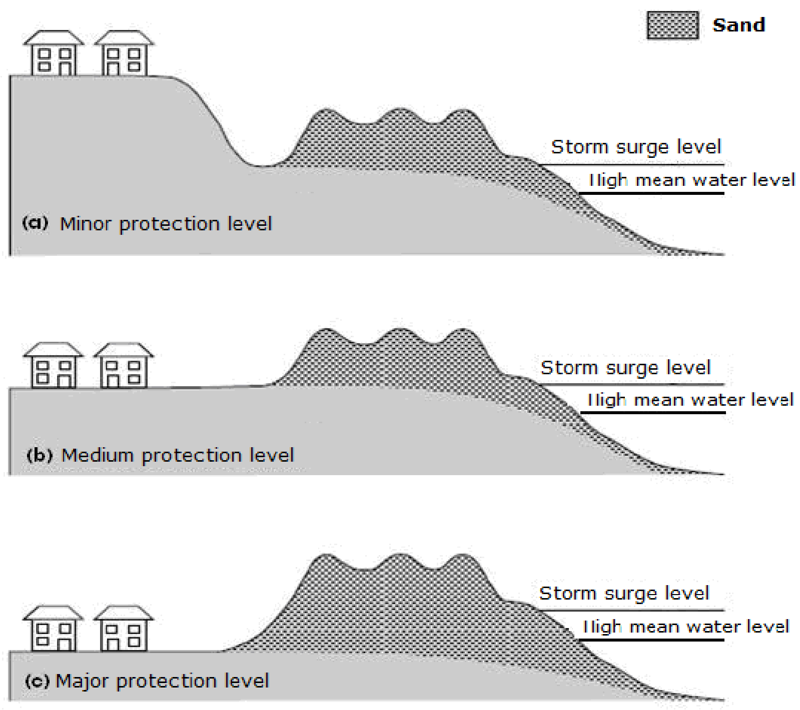


Figure 9.2. *Description of protective role of coastal sand dunes*

9.2.2 Distinction between sea dikes and sand dunes

9.2.2.1 Sea dikes

Sea dike is an artificial structures, built for a specific purpose of protecting human life, property, infrastructures in the area behind it from unfavourable impacts from the sea. The basic operating rule of a sea dike system is that the shape of dike route remain unchanged before and after the exertion of impacts from storms, high water level and wave height etc. However, this is only ensured in case the impacts of seaward parameters are lower or close to the design conditions of sea dikes.

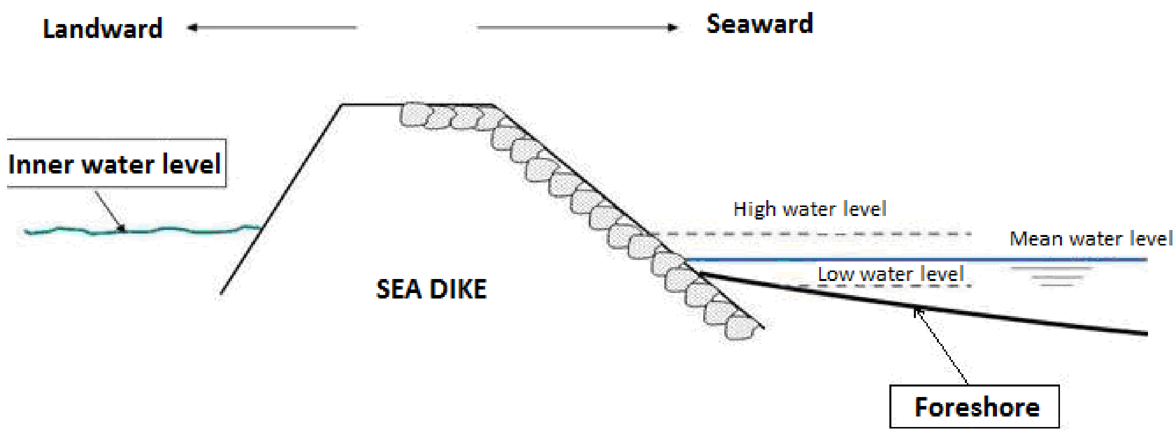


Figure 9.3. *Description of sea dike cross-section*

9.2.2.2 Sand dunes

The coastal sand dune system is a flexible protective solution in coastal areas. In specific cases, with the requirement of inhabitant and infrastructure protection on the rear side and on the sand dunes, the damage and destructiveness are not allowed. In fact, a natural sand dune can be destroyed or seriously eroded if no protective measures are adopted or the available solutions cannot meet the requirements of safety level.

However, similar to sea dike, a sand dune system can be an appropriate coastal protecttion structures. In case sand dunes function as a protection structure (with adequate stability, height and width), they can be considered a section of the whole coastal protective solutions.

9.3 Types of sand dune erosion

There are two types of sand dune erosion: wind-induced erosion and wave-induced erosion.

9.3.1 Wind-induced erosion of sand dunes

Wind-induced erosion causes the movement of sand. In this mode, sand from sand dunes is moved and forms the adjacent dune or spreads over the ground, surrounding roads, plants and houses.

9.3.2 Wave-induced erosion of sand dunes

Wave energy plays an important role in the formation of beach, accretion and transport sand on the beach periodically. Wave swashing deposits sand onto the beach while wave run-down pushes sand away. During calm duration, sand forms the beach, and during stormy duration the beach can be eroded by the impacts of waves.

Sand dune erosion is a typical example of cross-shore sediment transport. In a storm, dune erosion is related to the erosion of sand from the dune and the deposit in the deeper area within a shore duration; it is a typical cross-shore sediment transport. Normally, a storm is accompanied by the higher water levels and higher wave heights. Under stormy conditions, the cross section of sand dune can be much different from the initial equilibrium profile. During the storm, the regenerating process of dune profile will occur, in which onshore sand and sand from the dunes are eroded and deposited in deeper area (see ***Fig. 9.4***).

After the storm, the restoration of dune profile will normally occur due to the sediment transport processes under normal meteorological and oceanographic conditions.

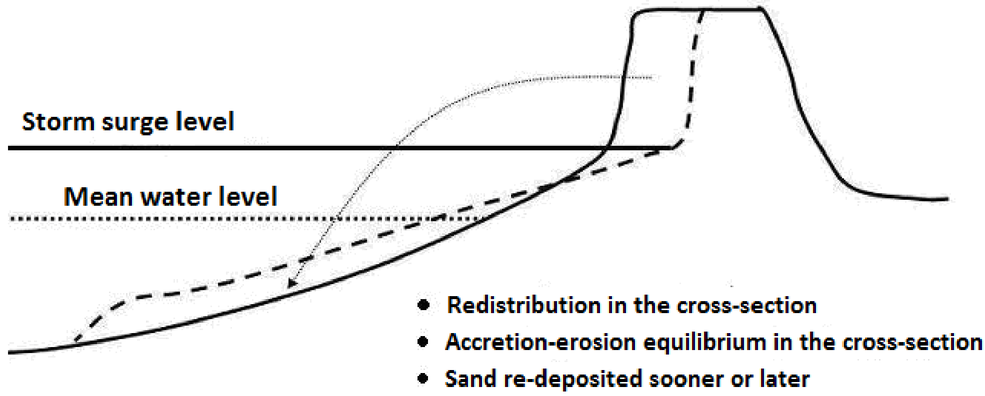


Figure 9.4. *Description of wave-induced erosion of sand dunes*

9.4 Evaluation of safety level and erosion potential of coastal sand dunes

9.4.1 Evaluation of safety level of the infrastructure and inhabitants on the rear side and on coastal sand dunes

9.4.1.1 Safety within large extent

In case of small, narrow sand dunes being eroded and destroyed during storms, the breakage of sand dunes will cause flooding and human loss, also the serious damage to the infrastructure and inhabitants on the rear side of sand dune. The damage levels depend on the location, elevation of infrastructure and inhabitable areas behind the sand dunes relative to the dune crest elevation, highest mean water level and highest storm surge level (as described in Section 9.2). In this case, the safety in large extent is mentioned (see ***Fig. 9.5***)

Figure 9.5 *Description of safety within large extent*

9.4.1.2 Safety within small extent

Safety withing small extent is considered in the following cases:

a) Width of sand dunes strip is relatively large (or at least large enough not to be destroyed by erosion). The safety of infrastructure and inhabitants on the rear side is then not a significant problem (see ***Fig. 9.6***).

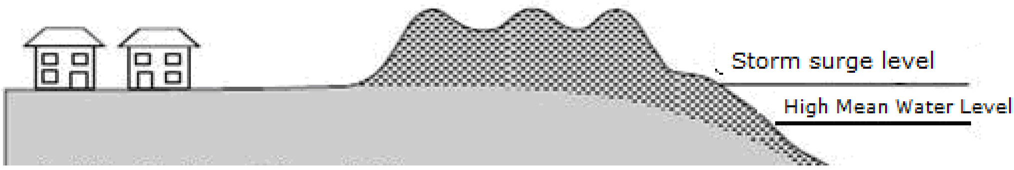


Figure 9.6 *Description of safety within small extent*

b) Infrastructure and inhabitants are located on coastal sand dunes. Even if these sand dunes are large, the safety of infrastructure and inhabitants is still a problem which needs careful care as the sand dunes are eroded in process or during storm (see ***Fig. 9.7***).

Figure 9.7 *Description of safety within small extent*

9.4.2 Assessment of erosion potential of coastal sand dunes

9.4.2.1 Description of sand dune erosion during storm

The erosion of sand dunes can be considered a problem of cross-shore sediment transport. On a cross section of coastal sand dunes, sand is moved to deeper area and becomes stable there. The first approaching method is to assume the equilibrium of sand limited in the cross-section extent. Total quantity of eroded sand from the dunes and the top part of dune profile (in m^3/m) and the quantity of sand deposited somewhere on the cross section are the same.

During storm, together with rapid increase of water level and wave height, the quantity of sand from the dunes is moved seaward. As dune erosion normally occurs within very short duration, only computation methods taking only cross-shore sediment transport into consideration are adopted in calculating dune erosion, and a definition of "eroded section" is used. With those methods and the above-mentioned definition, the shape of eroded section in the upper part is considered a known parameter. This shape during the most severe storm depends on the wave height and wave period, also the sand grading of the dunes. In case the characteristics of eroded section are given, the eroded length of sand dunes can be determined easily using sand equilibrium method within a closed section.

9.5 Evaluating the stability of sand dunes in case of erosion

The calculations of sand dune erosion in many cross sections are not reliable enough to evaluate the stability level of sand dunes. When a certain sand dune is eroded, there are many possibilities: sand dune is destroyed or it is still stable. Therefore, the criteria for the assessment of dune stability must be considered thoroughly. These criteria must be based on the actural conditions and depend on the following factors:

- Characterisitics of sand dunes themselves: range, location, extent and dimensions;
- Factors related to the protected areas by sand dunes, including: infrastructure, inhabitants on the rear side or on sand dunes;
- Oceanographic factors: water level, waves;
- Correlation in terms of function, location etc. between the above-mentioned factors.

Based on the actual situation of coastal sand dunes in Vietnam, the following cases must be distinguished in order to evaluate the stability level of sand dunes:

- Evaluating the stability level of sand dunes in case sand dunes function as protective solutions for the infrastructure and inhabitants on the rear side at a safety level within large extent;

- Evaluating the stability level of sand dunes in case sand dunes function as protective solutions for the infrastructure and inhabitants on the rear side at a safety level within small extent;

Evaluation of sand dune stability level in this section is described by the two main parameters as follows:

- Calculated frequency allowing the evaluation of erosion potential at different levels, or the relations between frequencies and different depths of sand dune erosion.

- Design standards in the calculation of protective solutions for sand dunes.

9.6 Management and protection of coastal sand dunes

9.6.1 Determining the extent of coastal sand dunes

From the calculation results and forecasts in Section 9.4, the safety extent and safety range for erosion and conditions to plant the vegetation layers withstanding wind impacts in order to determine the extent of coastal sand dunes.

9.6.1.1 Requirement of documentation

- Map of coastal area with scales of 1/10.000 to 1/1000 (depending on the safety range for inhabitants and sand dunes for each area, and the requirements of management and exploitation).

- Oceanographic data: lowest and highest mean water level, height of swashing waves at high tides.

9.6.1.2 Determining the extent of coastal sand dunes

The detailed drawings are made on the basis of sand dune definition, the descriptive diagram of limited range of sand dunes (see Section 9.1), above-mentioned topographic and oceanographic data. These drawings are the bases for the determination of coastal sand dunes, including:

- Topographs of coastal and sand dune areas;

- Longitudinal sections and some typical cross sections of coastal sand dunes and the beaches.

9.6.2 Solutions for the management of coastal sand dune erosion

9.6.2.1 No intervention to sand dune system

- Adaptive management;

- Raising the public awareness;

- Management of travelling in the area of sand dunes.

9.6.2.2 Intervention with small extent and short duration

- Planting on sand dunes (see the attached Appendix);

- Protecting the dune face by artificial materials (branches, timbers etc.);

- Protective fencing;

- Rehabilitating and restoring the beach and sand dune profiles;

- Protecting sand dunes by sand bags;

- Protecting sand dunges by placed gabions;

9.6.2.3 Protection of sand dune face

- Artificial headlands using rocks and gabions in front of sand dunes;

- Sea groynes;

- Near shore breakwaters;

- Artificial reefs in front of sand dunes;

9.6.2.4 Protection structures with large extent and long duration

- Revetments using placed rocks;
- Revetments using placed timber blocks;
- Impermeable revetments and sea walls/bulkheads.

(Bases of sand dune erosion computation and DUROS-PLUS Model are given in Appendix I).

10. TECHNICAL REQUIREMENTS IN SEA DIKE CONSTRUCTION

General requirements in sea dike construction conform to the current stipulations in the construction of earth dam and hydraulic structures. In addition, there are some other following specific requirements:

10.1 Construction technology and embankment quality control

10.1.1 Technical requirements of dike embankment

10.1.1.1 Profiling as per design cross section

Profiling piles and strings are used at site as per design dike cross section, at a distance of not more than 50 m.

10.1.1.2 Measurement

The volume of dike embankment is determined on the basis of measuring the dike cross section before and after the embankment (entirely or partly). The dimensions and locations of the structures must be measured, ensuring the accuracy as per the approved design documents.

10.1.1.3 Preparation of dike foundation base (newly-built and upgraded dikes)

The dike foundation must be treated well in order to ensure the stability of the dike including the layout cleaning, the removal of cover layer as per the design documents. If case of dike upgrading, the foundation base must be cleaned; also, the cavities and hollows must be filled up; the weathered topsoil layer must be removed; the grading must be performed with the largest height for each grade is twice of compacting layer thickness (about 30 cm).

10.1.1.4 Embankment material

The filling soil must ensure the physico-mechanical properties and reserves as per the design requirements, which have been determined in the design stage of construction drawings; all the roots, rubbish and waste are completely removed prior to the exploitation of soil for the embankment. In case the filling soil is taken at site, apart from the aforementioned requirements, the exploiting location must be at least 20m (beyond the protecting range of the dike) from the dike toe with no impacts on the mangrove forests.

10.1.1.5 Compacting works

The embankment is performed in layers with the inclination of 2% - 5% for the drainage of rain water, getting higher with the dike elevation. The thickness of filling layers must be compatible with the functions of compacting machine, but not greater than 50cm. Compacting equipment must be compatible with the physico-mechanical properties of the filling soil as per the design, ensuring the link between the compacted layers into a homogeneous mass of soil.

The dike sections, which have been divided according to corresponding contractors, must be filled uninterruptedly ensuring the quality of filling soil at the adjoining locations.

In case the construction area is under the impacts of tides, appropriate construction methods and progress should be adopted; the water content of the filling soil must be maintained as per the stipulation, ensuring the design compaction.

10.1.1.6 Embankment of the testing dike

a. For low-cohesive soil:

In order to define the number of compaction to obtain the design dry density of soil, the testing dike must be built as follows:

- Dimensions of testing dike: 6 x 2 x 60 (m);

. - Compaction must be performed in layers with a thickness of 15 cm.

Using the soil with natural moisture content, divided into 3 parts as follows:

+ Part 1: area 400 cm^2, 6 times of compaction;

+ Part 2: area 400 cm^2, 8 times of compaction;

+ Part 3: area 400 cm^2, 12 times of compaction;

- Some samples must be taken right after the compaction in order to inspect the in situ dry density of soil.

Grading characteristics must be analysed in order to determine the elastic limit and plastic limit of the same type of soil in use.

- Using the best result as a standard for the subsequent quality inspection.

Select the soil samples with maximum dry density determined by the method of embanking the testing dike in order to analyse the permeability.

The dry density of filling material must be equal to 95% of standard dry density after embanking the testing dike.

- For each type of soil, embanking the testing dike, and the corresponding results will be used as technical standards of design, and appropriate type of soil for construction can also be selected.

b. For cohesive soil:

With the above-mentioned compacting method for wet clay as presented in Section 10.1.1.6, prepare a dike section for testing, determine the dry soil density at different locations with appropriate methods, using the average value as the standard dry density.

10.1.2 Regulations on quality inspection

10.1.2.1 Inspection of dike cross section

After the construction, the geometrical dimensions of the dike cross section must be inspected every 100m as per the current standards.

10.1.2.2 Inspection of compaction quality

Testing samples must be taken at regular distances (1 sample/300m^3 of filling soil) with dedicated equipments in order to analyze the moisture content and dry density at site and in the laboratory.

(1) Density and moisture content of soil at site:

In case of loose soil, the tests with at least 06 samples should be conducted and avarage results are obtained for each location. In case of cohesive soil, after the compaction presented in Section 10.1.1.6, some samples must be taken in order to inspect the in situ density using the cutting ring on the cubic clay sample with smooth surface (appropriate length of the perpendicular side is 30 cm or 40 cm).

(2) Analysis of grading characteristics:

Taking an appropriate amount of soil samples in order to analyze the grading characteristics.

(3) Determining the elastic limit and plastic limit as per the current standards.

10.2 Technical requirements of the construction of dike slope protection structures

10.2.1 Rockfill revetment

10.2.1.1 Technical process of rockfill revetment construction

(1) Thickness of revetment

Ensuring the design thickness.

(2) Rock quality

Ensuring the technical standards as per 14TCN 12-2002[1] *"Hydraulic structures - Rock construction - Technical requirements of construction and acceptance"* and other current technical standards and codes; the mortar used in the construction must conform with the Technical Code 14TCN 80-2001[1] *"Hydraulic construction mortar – Technical requirements"*

10.2.1.2 Quality inspection of rockfill revetment after construction

(1) Quality of stones

Visual inspection is necessary and compression strength test in the laboratory must be conducted for rocks with different sizes if there is any difference during the in situ inspection.

(2) Thickness and placing method

The discrepancy of the revetment thickness must not exceed 5% of the design value.

(3) Grading characteristics of revetment rock

- Select an area of 50m^2, then measure the outer diameter of each stone and mark it using paint or chalk.

- Place the stones with the same sizes in a group (as per ***Table 10.1***), then determine the percentage for each group.

Table 10.1 *Rock classification*

No.	Group	No.	Group
1	0,80÷1,0 m	5	0,30÷0,40 m
2	0,60÷0,70 m	6	0,20÷0,30 m
3	0,50÷0,60 m	7	0,10÷0,20 m
4	0,40÷0,50 m	8	0,05÷0,10 m

From the measured diameters of the stones, the area of each stone is determined, which is then multiplied by the thickness of rockfill revetment and the specific density of the rock in each group. By this way, the distribution of the stones with different sizes on the rockfill revetment surface can be determined. The existence of 50% of the stones having average weight (W50) must be ensured with an allowable error of 10%.

(4) Quality of mortar used for the construction of rock revetment

Ensuring a slump of 3 ÷ 9cm. Six (06) samples must be taken for every $30m^3$ of mortar for the purposes of inspection in the laboratory.

10.2.2 Concrete revetment

10.2.2.1 Material requirements

Sand, gravel, water, cement are used for manufacturing the concrete of the revetment as per 14TCN66-2002 to 14TCN 73-2002[1] *"Hydraulic Construction Material – Technical Requirements and Testing Methods"*.

10.2.2.2. Inspection of concrete quality

As per 14TCN 63-2002, 14TCN 64-2002 and 14TCN 65-2002[1].

10.3 Technical requirements of construction and quality inspection of sand and gravel filter layers

10.3.1 Thickness and placing method

The placement must be performed at the correct location, with correct grading characteristics and thickness as per the design drawings. The compaction of the filter layers is not necessary, however the construction must ensure the design grading characteristics and thickness.

10.3.2 Grading characteristics of fiter layers

Given in ***Table 10.2*** and ***10.3***

Table 10.2. *Appropriate sand grading for filter layer (as per American standards)*

Sieve mesh size	Sieve N^0	Percentage of weight retained on the sieve
4,67mm	$N^0$4	0
2,83mm	$N^0$8	5÷15
1,19mm	$N^0$16	10÷25
0,59mm	$N^0$30	10÷30
0,297mm	$N^0$50	15÷35
0,149mm	$N^0$100	12÷20
Pan		3÷7

Table 10.3: *Appropriate gravel grading for filter layer*

Sieve mesh size	**Sieve N^0**	**Percentage of weight retained on the sieve**
19-38,1	3/4- 11/2	40÷55
9,51-19	3/8- 3/4	30÷35
1,19 mm	$N^0$4- 3/8	15÷25

10.3.3 Inspecting the quality of filter layer

Ensuring the design thickness with allowable discrepancy of 10%:

- Material used for the filter layer must meet the requirements of quality limited in terms of grading as given in ***Table 10.2*** and ***10.3***.

- Samples of gravel and sand, which used for the filter layers, are taken every 20 m along the length of the dike in order to analyze the grading characteristics.

10.4 Technical process of the construction and quality inspection of filtering geotextile

10.4.1 Placement of filter geotextile

- Keep the area where the filtering geotextile is placed dry or perform the placing works at low tides.

- Preparing the layout for filtering geotextile spreading: cleaning, levelling works on the slope.

- At the area with no water, dig the cut-off to the design level and place the filtering geotextile, pinned to the cut-off and slope as per the design guidelines.

- At the area with water, filtering geotextile is placed into the cut-off grove and pinned. The geotextile must be spread from the toe up to the slope in the submerged condition; the pinning at the submerged toe and slope must be performed carefully in order to prevent the uplifting out of the acting point due to water and wave.

- The abutting location between the two overlapped geotextile is 30÷50 cm. If the two geotextile are stitched together, the strength of the joint must be at least 80% of filtering geotextile strength. The top part of the filtering geotextile must be stabilized without water running under and prevent the damage during long period outdoors (not more than 5 days), not exposed to hot sunlight.

10.4.2 Quality inspection of filtering geotextile construction

To be performed at site and at the same time, inspect the cut-off dimensions and rock placement of the revetment. The geotextile quality must meet the design and inspection requirements as per 14 TCN 91- 1996 to 14TCN 99- 1996: *"Geotextile – technical requirements and testing methods".*

10.5 Technical process and inspection of grass quality on the inner dike slope

10.5.1 Technical process

Pieces of fresh grass with dimensions of about 30x30cm and thickness of 5÷10 cm are anchored by bamboo piles on the slope.

10.5.2 Quality control

Visual inspection of quality, ensuring the covering of grass on dike slope as per design requirements.

*10.*6 Technical process of growing mangrove forest

- Selecting the appropriate types of mangrove forest corresponding to the natural conditions of each area, such as geology, pedology, climatology, hydrology, etc. with good capacity of wave attenuation;

- Reclaiming the soil in order to facilitate the growth of the trees during the initial stage in the areas where the soil is infertile and the alluvial sediment deposition is low, not capable of satisfying the requirements of tree growth.

- Appropriate techniques of growing and facilitate the growth of the trees during the initial stage;

- Selecting the appropriate crops for the growth of the trees.

10.6.1 Technical process

Mangrove plants such as Bruguiera are grown on the foreshore ensuring a distance of 1x1 m between the plants, crossed-planting, with a density of 10.000 trees/ha, about 1 m from the dike toe. The width of planting band is equal to 3÷5 times of wave length with wind velocity of 32m/s, which is about 30÷50m on average.

10.6.2 Quality inspection

Visual inspection of quality in terms of extent and density. The inspecting duration is stipulated by the design regulation.

10.7 Technical requirements in the construction of sloping sea groynes

10.7.1 Dumping sand for the treatment of foundation

- The influence factors such as depth, currents and waves must be studied in order to analyse and overcome the sand drifting phenomenon. Tests should be conducted to determine the anchoring locations. If the depth is great and current velocity is high, the methods of pouring sand using funnel, injection pumping etc. can be adopted.

- Dumping sand: the construction procedure must be divided into many segments, and the rock covering must be performed in time after the sand dumping. The segment length depends on the natural conditions, construction capacity. The requirements of sand dumping quality are as follows:

+ The top of sand layer must not be lower than 0,2 m; not higher than 0,5 m with reference to the design elevation.

+ Width of the dumped sand top layer must not be smaller than the design width and not exceed 3 m on each side.

10.7.2 Dumping of rock and cubic concrete blocks

10.7.2.1 Segment division in construction

The construction procedures with division and bedding must be determined based on the design, construction capacity and the impact level of tides, waves and currents at different locations on the dike body.

10.7.2.2 Dumping location

Based on the depth, current velocity and waves in order to determine the anchoring location of rock-dumping barges.

10.7.2.3 Sequence of rock dumping on soft soil foundation

- In case the loading quarry-stone layer is placed, the loading part must be released and then the rocks for dike body are dumped;

- In case the foundation extrusion is needed, the stones are dumped starting from the middle and extending gradually to both sides.

10.7.2.4 Dumping of rock covering the slope and buffer layers

The design thickness must be ensured, and the covering rock layer must not be steeper than the design slope.

10.7.2.5 Allowable error

The allowable error for the dumped rock that forms the boundary line of the design dike profile is given in ***Table 10.4***.

Table 10.4. *Allowable error for dumped rock at the boundary line of design cross section*

Weight of dumped rock (kg)	10÷100	100÷200	20÷0300	300÷500	500÷700	700÷1000
Allowable difference in level (cm)	±40	±50	±60	±70	±80	±90

10.7.2.6 Levelling the rock-dumped surface and rock placement

The difference in level between the design boundary line and the actual profile is given in ***Table 10.5***.

Table 10.5. *Allowable level difference between actual and design boundary line*

Work	Weight of rock (kg)	Allowable difference in level (cm)
Levelling	10÷100	±20
	100÷200	±30
Placing	200÷300	±40
	300÷500	±50
	500÷700	±60
	700÷1000	±70

10.7.2.7 Restraining the boundary line

Prior to casting the cubic concrete blocks, the blocks should be placed at the front boundary line in order to restrain. The discrepancy between the actual and design boundary line must not exceed 30cm.

10.7.3 Fabricating and laying the covering blocks

10.7.3.1 Tight formwork for the fabrication of concrete structural elements.

The formwork for the fabrication of concrete structural elements must make of steel, ensuring the stiffness and facilitation in fabricating and dismantling.

Upon pouring the concrete into the mould: if the top surface gets air bubbles, before the concrete set, use the mortar to plaster once, and screed many times in order to ensure smoothing. Dimension errors and surface defects for precast structural elements must not exceed the values given in ***Table 10.6***.

Table 10.6. *Dimension errors and surface defects for precast structural elements*

Items		Allowable error (cm)	Notes
Dimensions	- Length of sides - Diagonal - Height - Position of openings	±1,0 ±2,0 ±1,0 ±2,0	Applied for the structural elements with standard geometrical dimensions
Surface defects	- Side chip - Blistering depth - Discrepancy at the formwork joining	≤5,0 ≤0,5 ≤2,0	Applied for different types of concrete structureal elements

10.7.3.2. Transportation

The strength of concrete must meet the requirements of hoisting before being transported.

10.7.3.3 Installation

Impacts of waves and construction progress must be taken into consideration, ensuring the complete covering of the underlying stones before being eroded. Prior to the installation, the inspection of additional maintenance of the slope inclination and the actual conditions of underlying stone layer surface must be performed, levelling by means of small-sized stones scattered in order to fill the large openings. Allowable error must not exceed ± 5cm for the construction works above the water, and ± 10cm under water.

10.7.3.4 Covering blocks at the end of the slope

Ensuring the tight contact with the rock-dumping prism at the dike toe.

10.7.3.5 Using Dolos or Tetrapod blocks for slope covering

Ensuring a uniform density all over the slope.

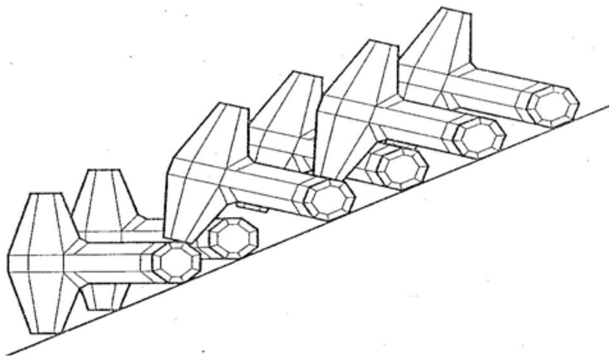


Figure 10.1. *Placing diagram of Dolos blocks on the slope*

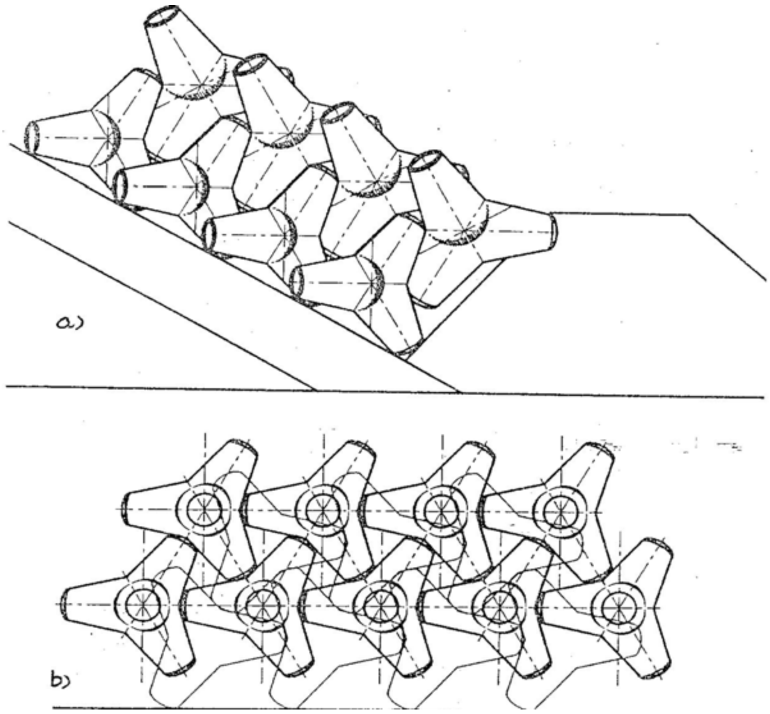


Figure 10.2. *Placing method of tetrapod blocks on the slope*

a. Cross-section; b. Layout

- Method of placing Dolos blocks: vertical placement at the lower part of the slope, borne by the horizontal ribs of the underlying blocks; horizontal placement, borne by the rock layer of the dike slope. Connecting bars cross the horizontal rib of the adjacent blocks so that the underlying stones are not exposed (see ***Fig.10.2***).

10.7.3.6 Installation error of structural elements

- For Dolos and Tetrapod blocks: The discrepancy between the actual installation quantity and the design quantity must not exceed ± 5%.

- For square block-typed slabs: the level difference with the adjacent blocks must not exceed 15 cm, the jointing opening between the two blocks must not exceed 10cm.

10.7.3.7 In case of the slope covered by hand-placed quarry-stones

The shapes of the stones should be relatively close to the prism, with the length not smaller than the design thickness. The stones are placed vertically, their weight must be greater than the design weight. Covering layer made of quarry-stones must meet the following requirements :

- More than 90% of the area ensuring the design thickness;

- The opening between two placed stones must be less than 2/3 of the smallest diameter of the underlying stones, with no inter-connecting openings perpendicular to the surface of covering layer. The dimensions of the openings are defined as follows:

+ Allowable width joining openings: 3 cm;

+ Allowable width of triangular openings: 7 cm;

+ Allowable roughness of slope surface: 3 cm;

- Paved stones must be tightly packed with smaller stones locked underneath (in case the crowbar is used to lift a big stone out of the slope, then 2 or 3 adjacent stones are also prised up).

10.7.3.9 Slit-pushing, tightly joint sealing mortar

The allowable requirements of rock construction are as follows:

- Joint : 4 cm
- Width of triangular openings: 8 cm
- Roughness of slope surface: 3 cm

10.8 Environmental aspects

The environmental impact assessment must be performed in the planning phase of the construction investment project. During the construction stage, the environmental impacts must be limited, especially in case of the mangrove forests; solutions for reducing noise and dust must be adopted when transporting the material and during the construction; the layout and traffic roads which are under the impacts of the construction must be restored.

11. DIKE MANAGEMENT, REPAIR AND MAINTENANCE

11.1. General stipulations

- The protected range of sea dike must comply with the stipulations in the Dike Code.

- Sea dikes must be handed over to the managing, exploiting and operating authorities after completing. The management, repair and maintenance of sea dike are performed as per current stipulations.

- Inspection, monitoring during and after the strike of storms, and timely treatment of damage to sea dike system are required.

11.2. Dike Repair and Maintenance

The repair and maintenance of sea dike must be performed periodically in order to enhance the structural stability and durability of the structure. The main aspects that should be focused are as follows:

11.2.1. Inspecting and analysing the working status of sea dike and the variations of hydrodynamic conditions

11.2.1.1 Periodic inspection:

The inspecition must be performed once or twice a year based on the following factors:

- Density and intensity of the storms that are likely to strike (seasonal forecast in each area).

- General importance and strategic location of sea dike.

- Location and importance of the structural elements.

+ Concentrated mainly at the locations with permanent load impacts (such as revetment, transition locations etc.);

+ Inspecting the abrasion, cracking, displacement or breaking of sea dike under the impacts of waves and currents;

- The following items of sea dikes and revetments should be inspected and monitored:

+ Dike height or crest elevation; settlement of dike body;

+ Quality of dike slope and dike body protection (slope stability, burrows and holes etc.);

+ Geometric dimensions of revetment (cross and longitudinal sections, thickness).

+ Physico-mechanical properties of the revetment;

+ Quality of transition structures (revetment toe, filter layers etc.);

+ Scour development in front of the dike toe (if any);

- The duration of periodic inspection of the structural elements located above the low tidal level is given in ***Table 11.1.***

Table 11.1. *Duration of periodic inspection*

Item	Longest period
Geometry of revetment	12 months
Position of the stones on the revetment	12 months
Physico-mechanical properties of the revetment	12 months
Scour development	6 months

- In case of the structural elements that are completely submerged, the inspecting period should be at least once a year.

11.2.1.2. Situational inspection

Based on the situation of storms, the inspection of the status of sea dikes and revetments should be performed prior to the strike of the forecast storms in order to prepare for the confrontation with probable situations. After the storms, the damage of sea dike and revetment must be inspected in order to formulate plans for the timely repair.

11.2.1.3. Impacts of hydrodynamic changes on the structures

Equipments for monitoring the variations of hydrological and oceanographic conditions are required as the bases for the analysis and evaluation of varying trend (positive or negative) of the loads acting on the structures. Based on these documents, the plan for upgrading and rehabilitating sea dikes and revetments in the future can be proposed.

11.2.2. Main items requiring repair and maintenance

- Repairing, rehabilitating and strengthening the dike crest, dike slope, crown wall, path along dike toe, stairs on dike slope;

- Planting and nourishing the mangroves, and also the plants protecting sand dunes;

- Repairing the sluices cutting through the sea dike, and also the revetments;

- Urgent handle of sea dike failure.

- Detecting and handling the hidden threats in the dike body; clearing the dike crest surface, dike slope and dike toe;

- Constructing and repairing the structures for dike management: milestones; water gauging rods; signs; observing stations and equipments; stores, material stockpiles, managerial buildings, border crossings, observation posts.

- Topographic and geological survey of dike body and dike foundation; analyzing and evaluating the evolution of the foreshore, the revetment, the mangrove forests, the tides in important areas; supplementing the database for dike management.

- Suppplementing, collecting and maintaining the material stockpiles for flooding prevention and protection.

APPENDICES

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