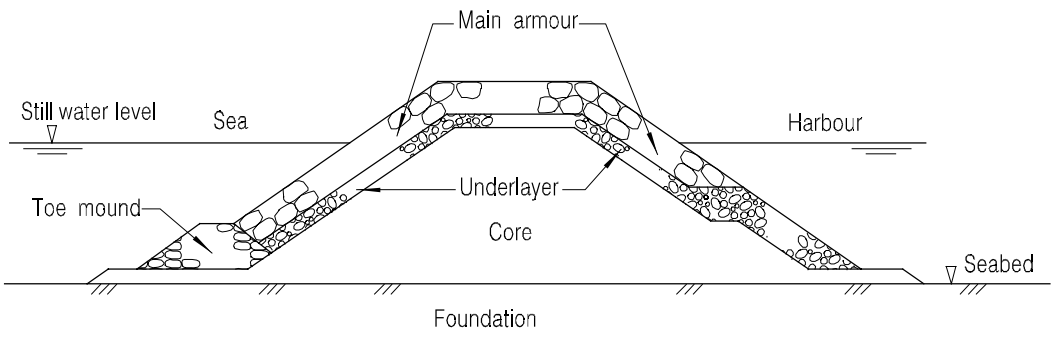
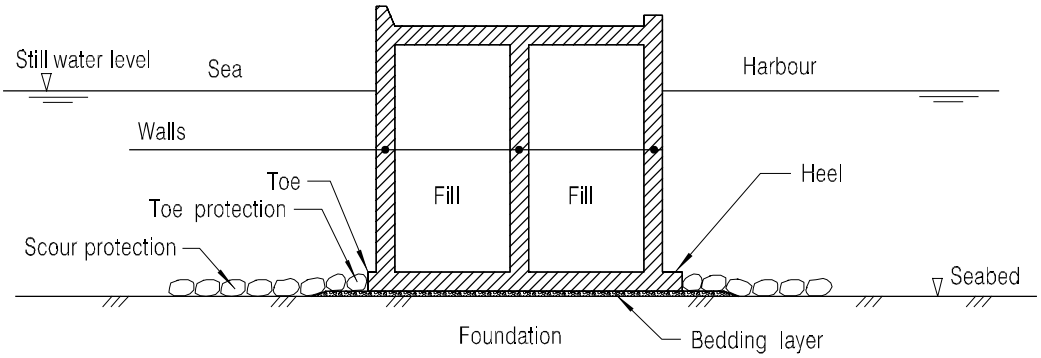
FIGURES

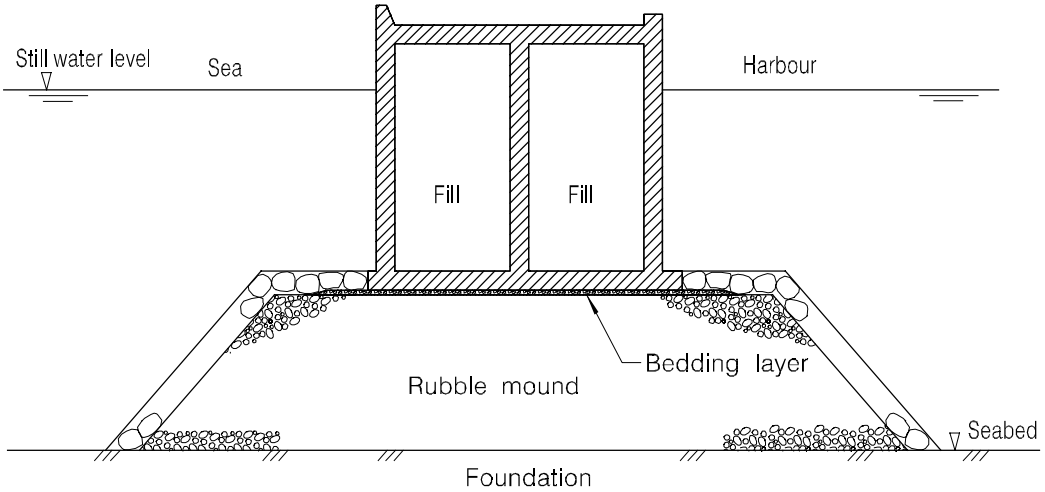
LIST OF FIGURES



(a) Rubble Mound Breakwater



(b) Caisson Type Vertical Breakwater



(c) Composite Breakwater

Note:

1. Caisson type structure can be replaced by blockwork structure.

Figure 1 – Type of Breakwaters

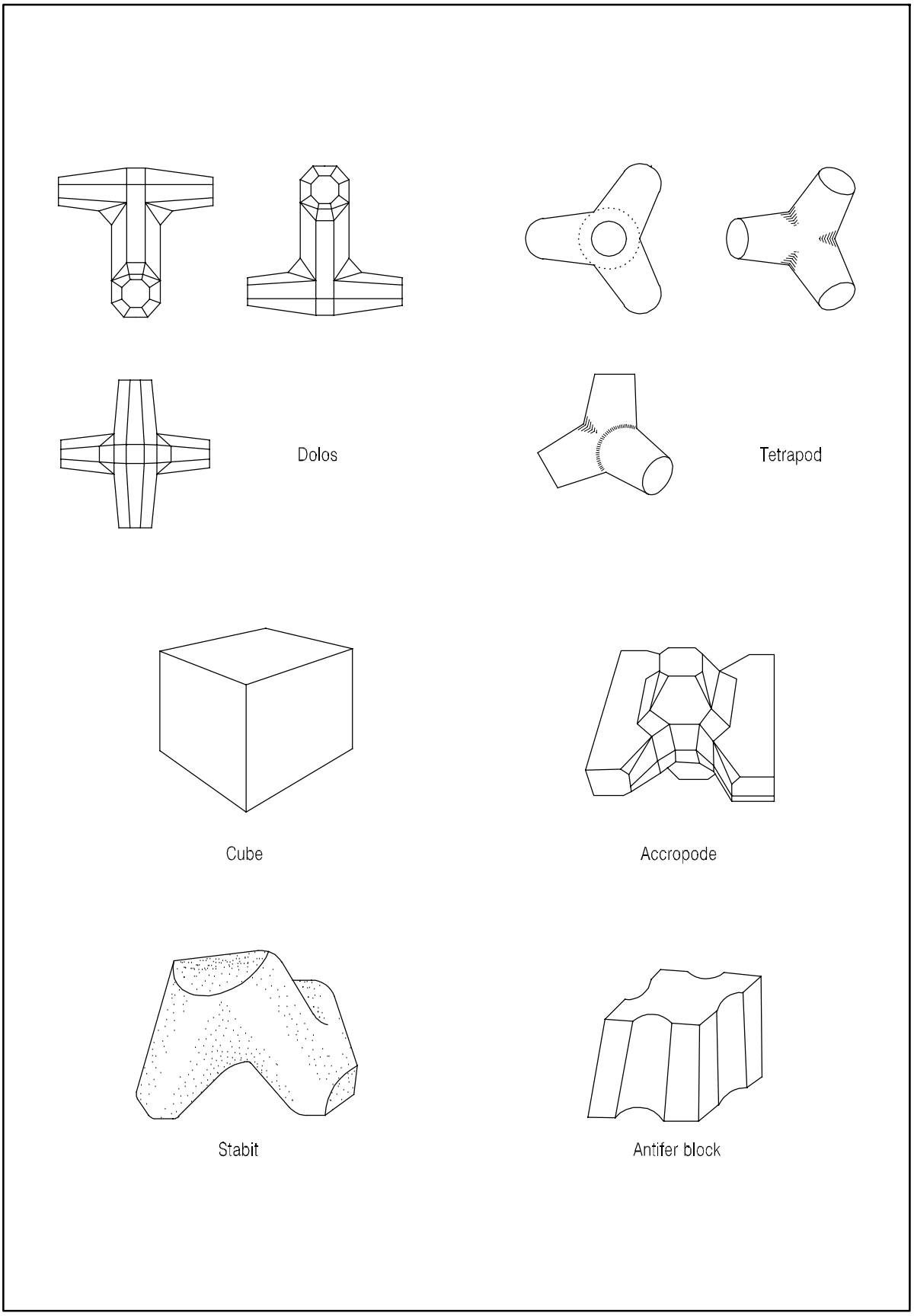
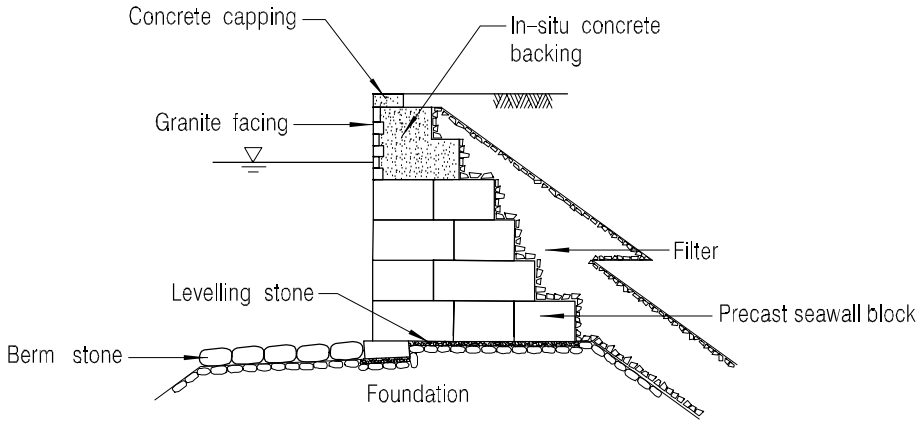
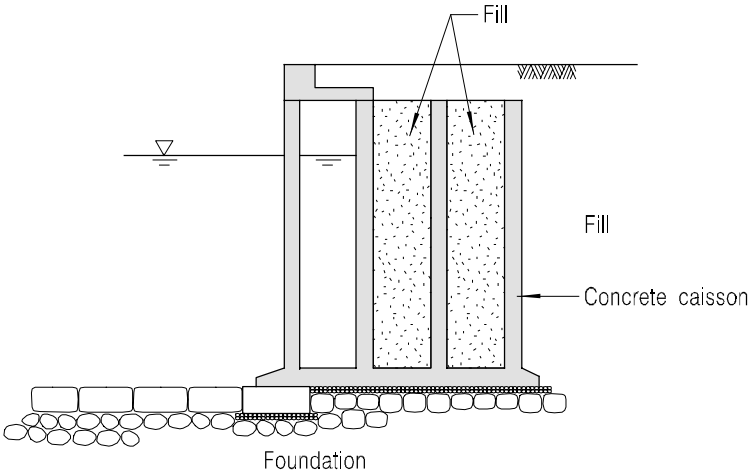


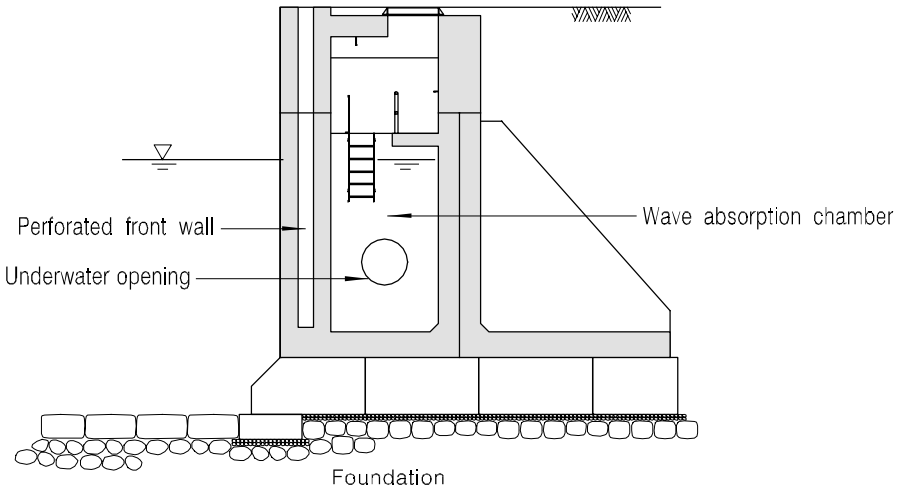
Figure 2 – Precast Concrete Armour Units



(a) Concrete Blockwork Seawall



(b) Caisson Seawall



(c) Wave Absorption Seawall

Figure 3 – Vertical Seawalls

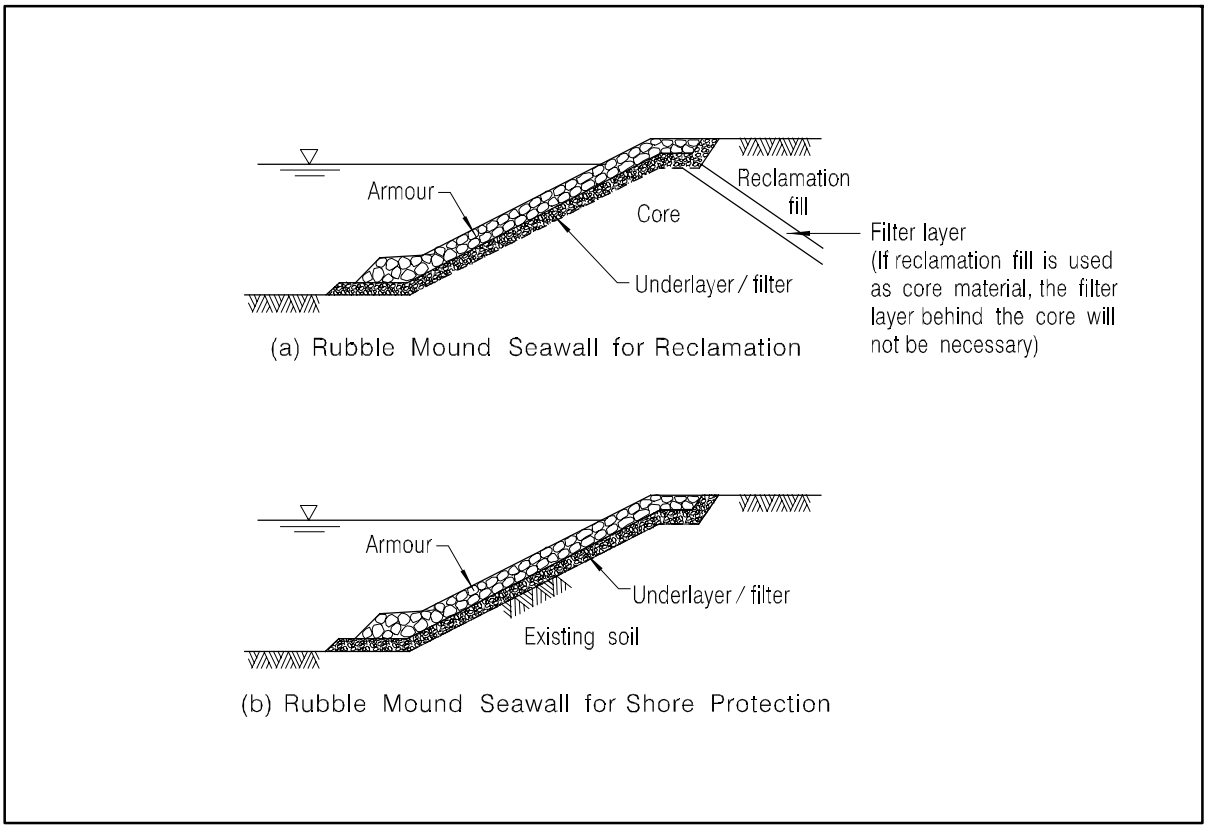


Figure 4 – Rubble Mound Seawalls

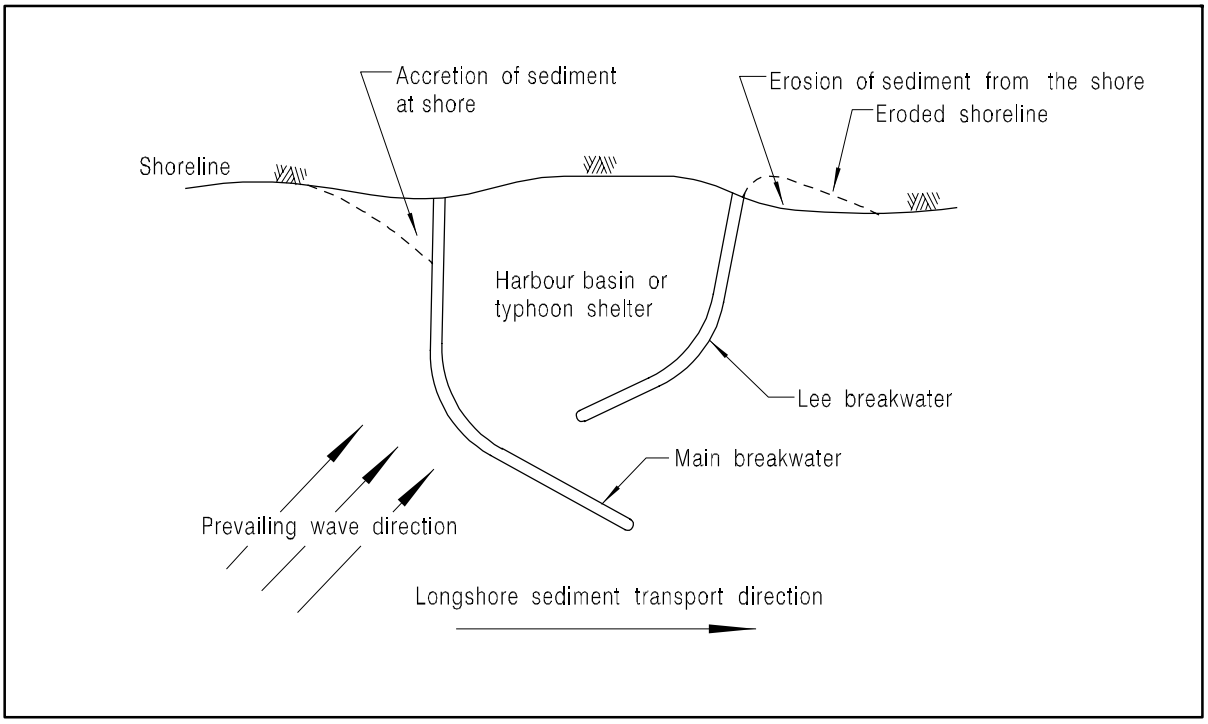
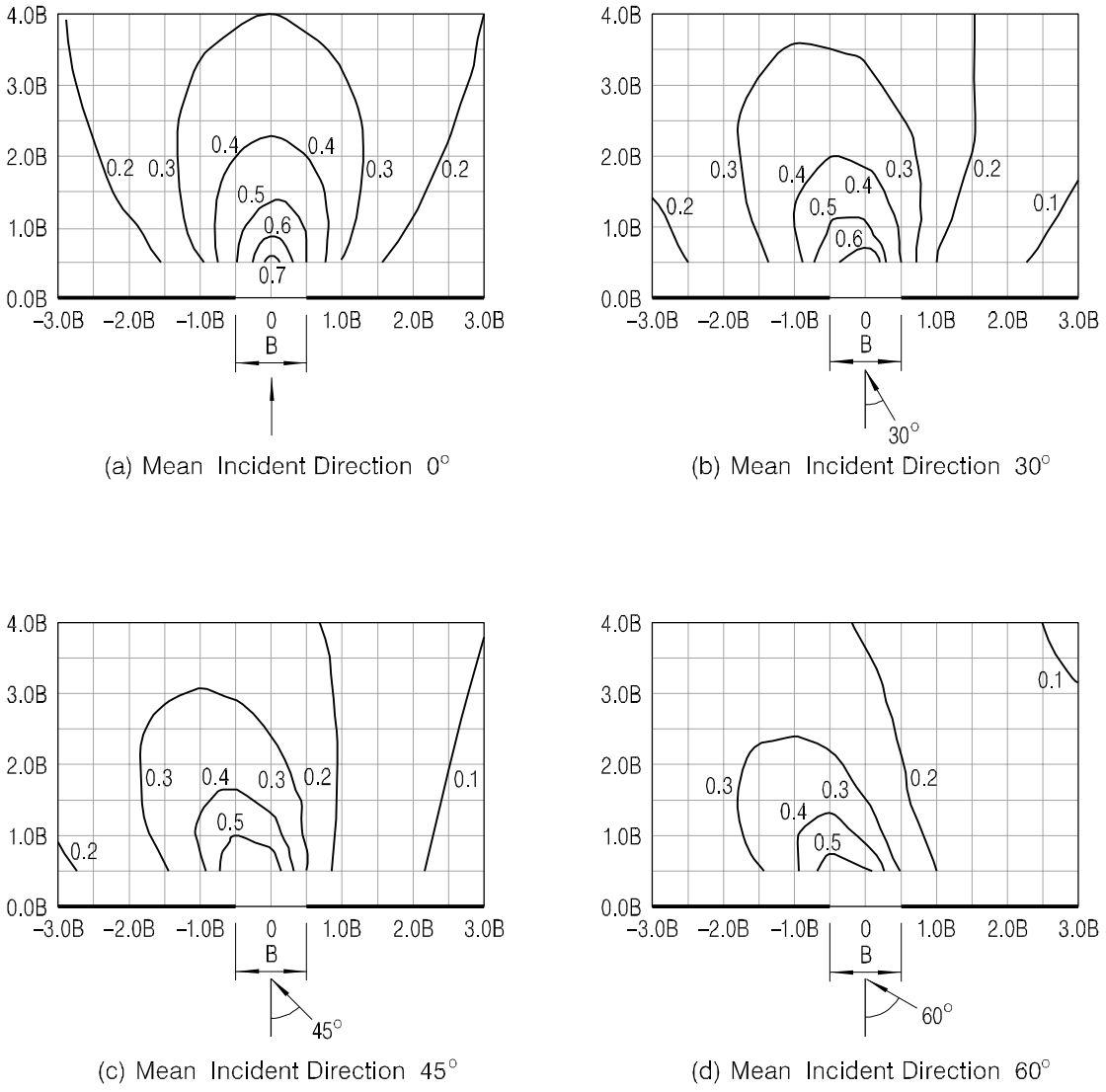


Figure 5 – Breakwater Layout



(a) Mean Incident Direction 0°

(b) Mean Incident Direction 30°

(c) Mean Incident Direction 45°

(d) Mean Incident Direction 60°

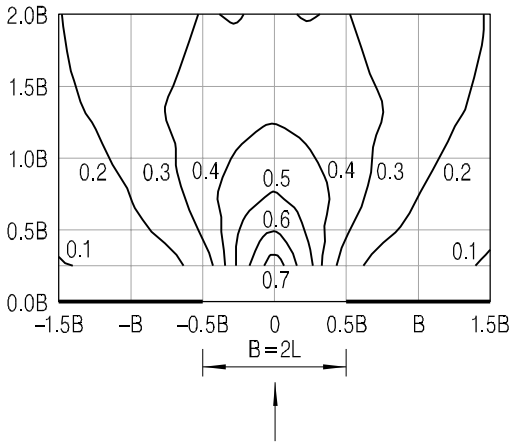
Notes:

1. Wave length corresponds to peak period of wave spectrum.

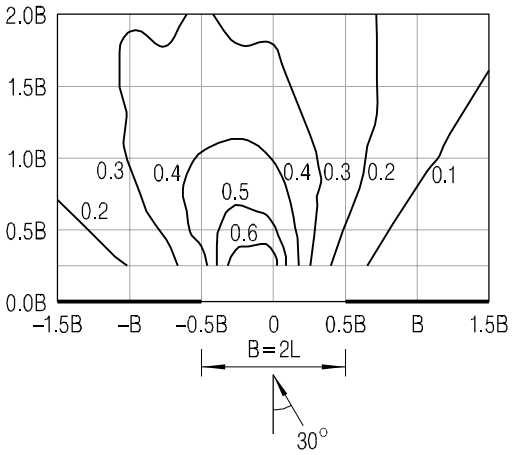
2. Length B = one wave length, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

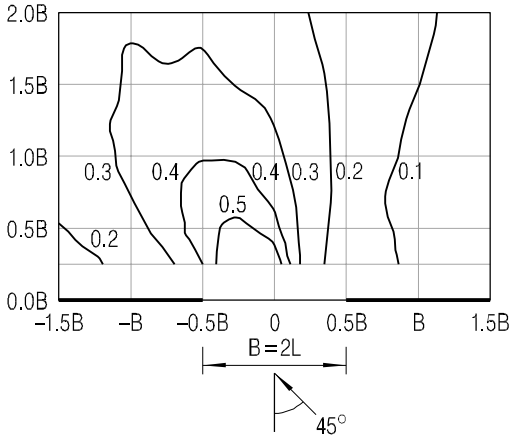
Figure 6 – Diffraction Coefficients for Breakwater Gap (Sheet 1 of 2)



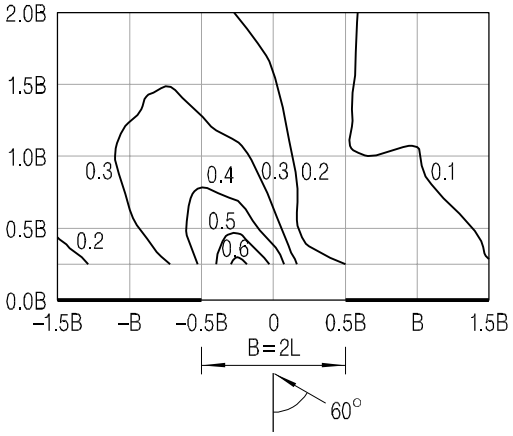
(a) Mean Incident Direction 0°



(b) Mean Incident Direction 30°



(c) Mean Incident Direction 45°



(d) Mean Incident Direction 60°

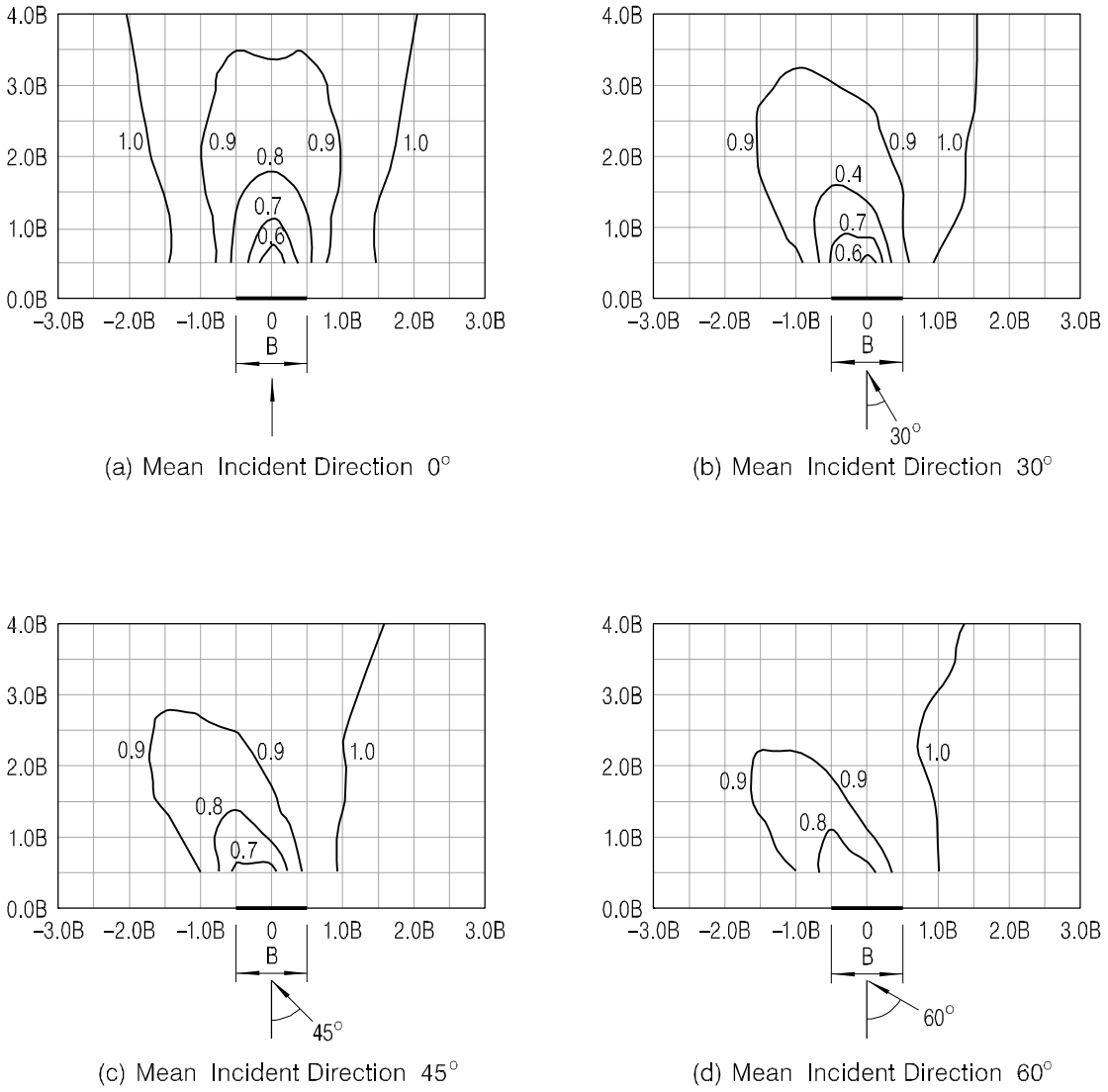
Notes:

1. Wave length corresponds to peak period of wave spectrum.

2. Length B = two wave lengths, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 6 – Diffraction Coefficients for Breakwater Gap (Sheet 2 of 2)



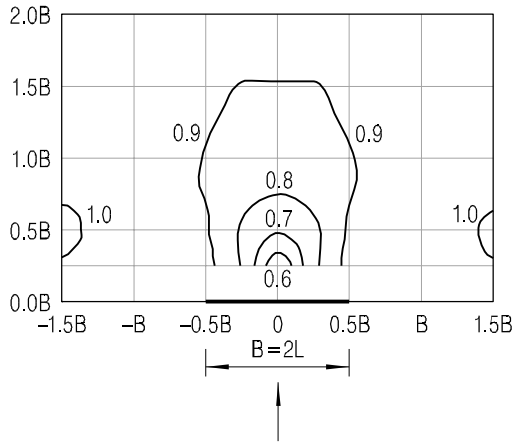
Notes:

1. Wave length corresponds to peak period of wave spectrum.

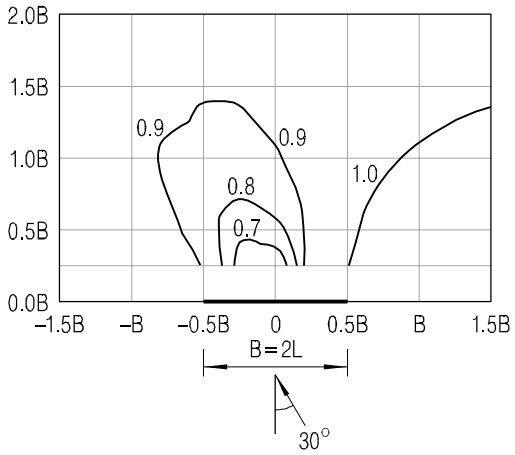
2. Length B = one wave length, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

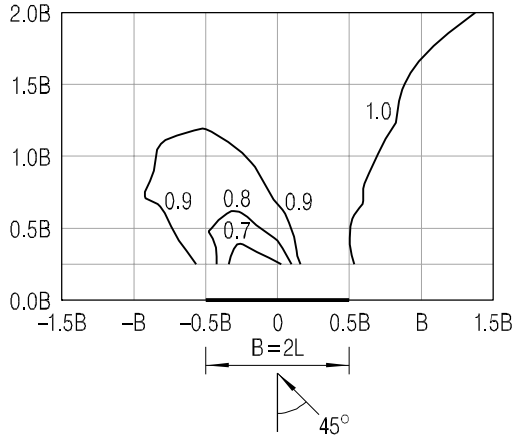
Figure 7 – Diffraction Coefficients for Island Breakwater (Sheet 1 of 2)



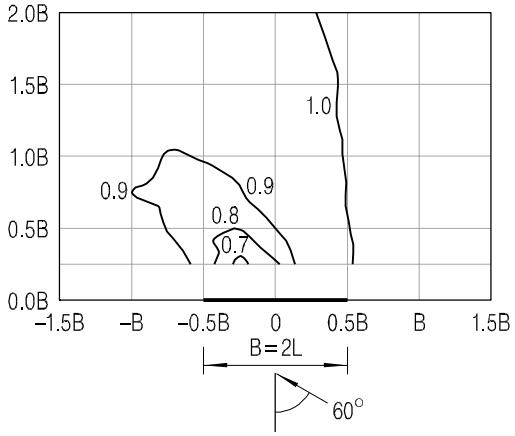
(a) Mean Incident Direction 0°



(b) Mean Incident Direction 30°



(c) Mean Incident Direction 45°



(d) Mean Incident Direction 60°

Notes:

1. Wave length corresponds to peak period of wave spectrum.

2. Length B = two wave lengths, Pierson–Moskowitz spectrum, $\cos^2$ directional spread.

Source: BS 6349 : Part 1 (2000) (Acknowledgement: HR Wallingford)

Figure 7 – Diffraction Coefficients for Island Breakwater (Sheet 2 of 2)

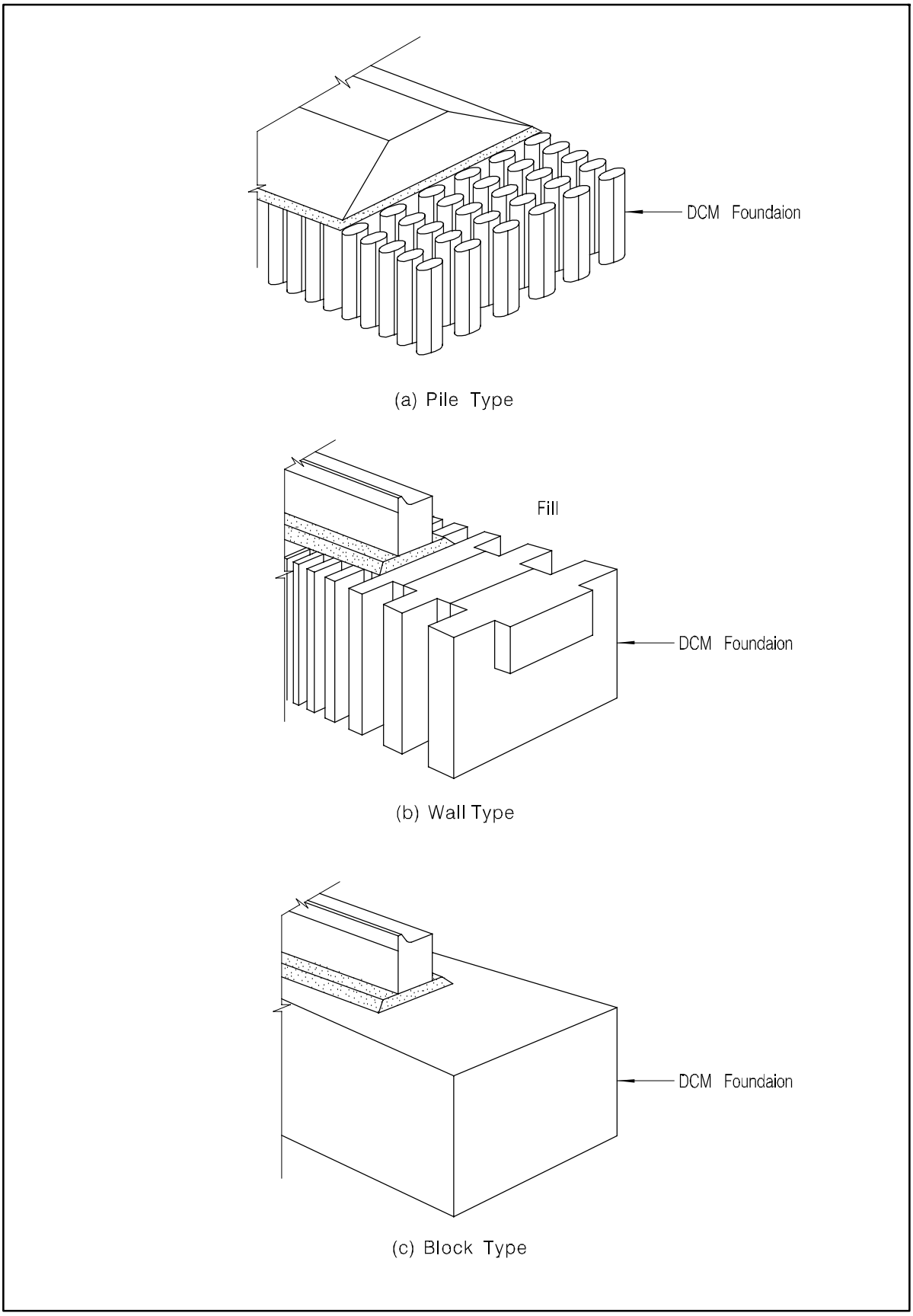
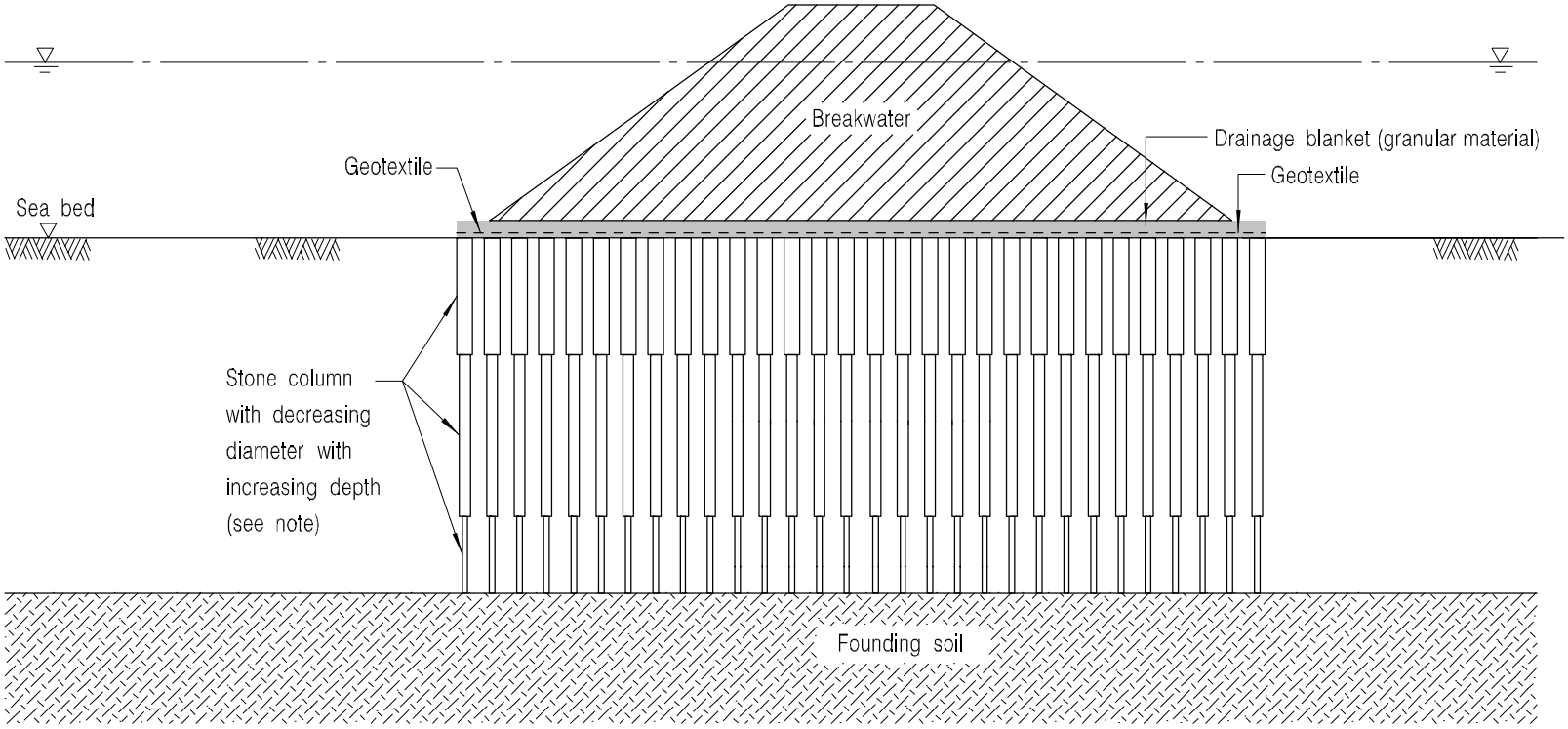


Figure 8 – Layout of Deep Cement Mixing Foundation



Notes :

1. Reduction of diameter with depth as shown, which is due to the increase in confining stress with depth, is for illustration purpose only. The actual amount of reduction will depend on the strength–depth relationship of respective soil stratum.

2. This figure illustrates the general principle of stone columns only; details of the structure such as toe protection are not shown.

Figure 9 – Layout of Stone–Column Foundation

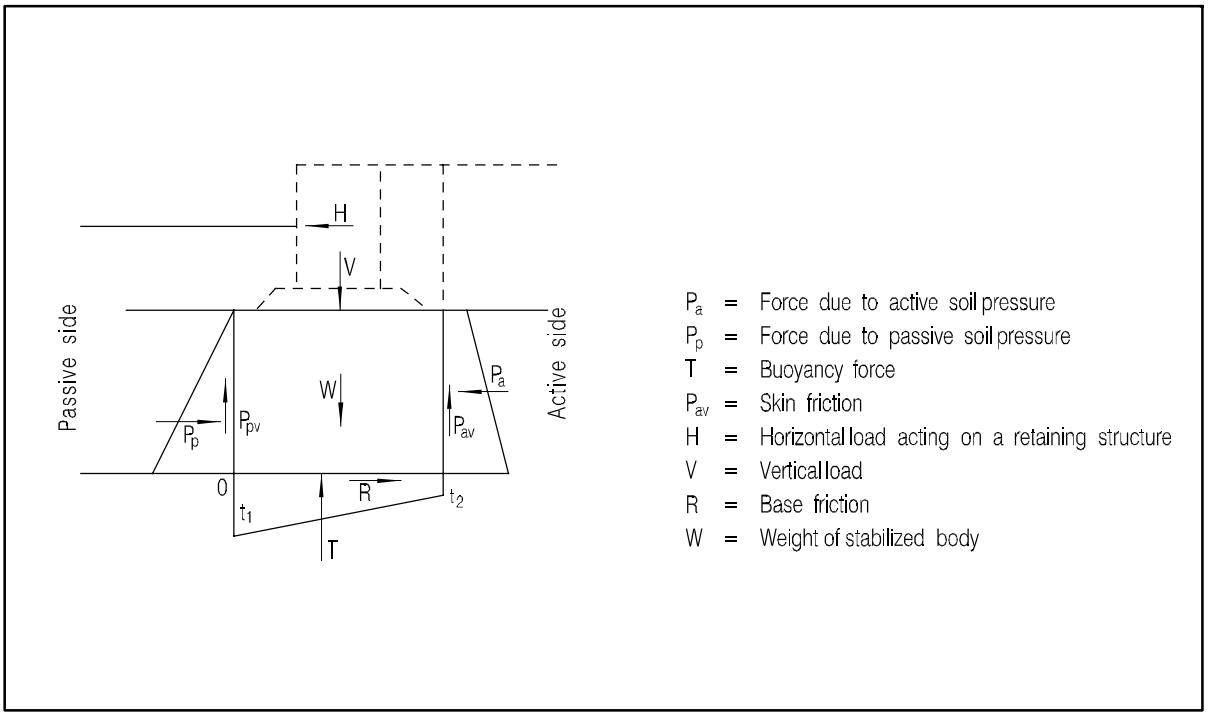


Figure 10 – External Forces on Soil Body Stabilized by Deep Cement Mixing

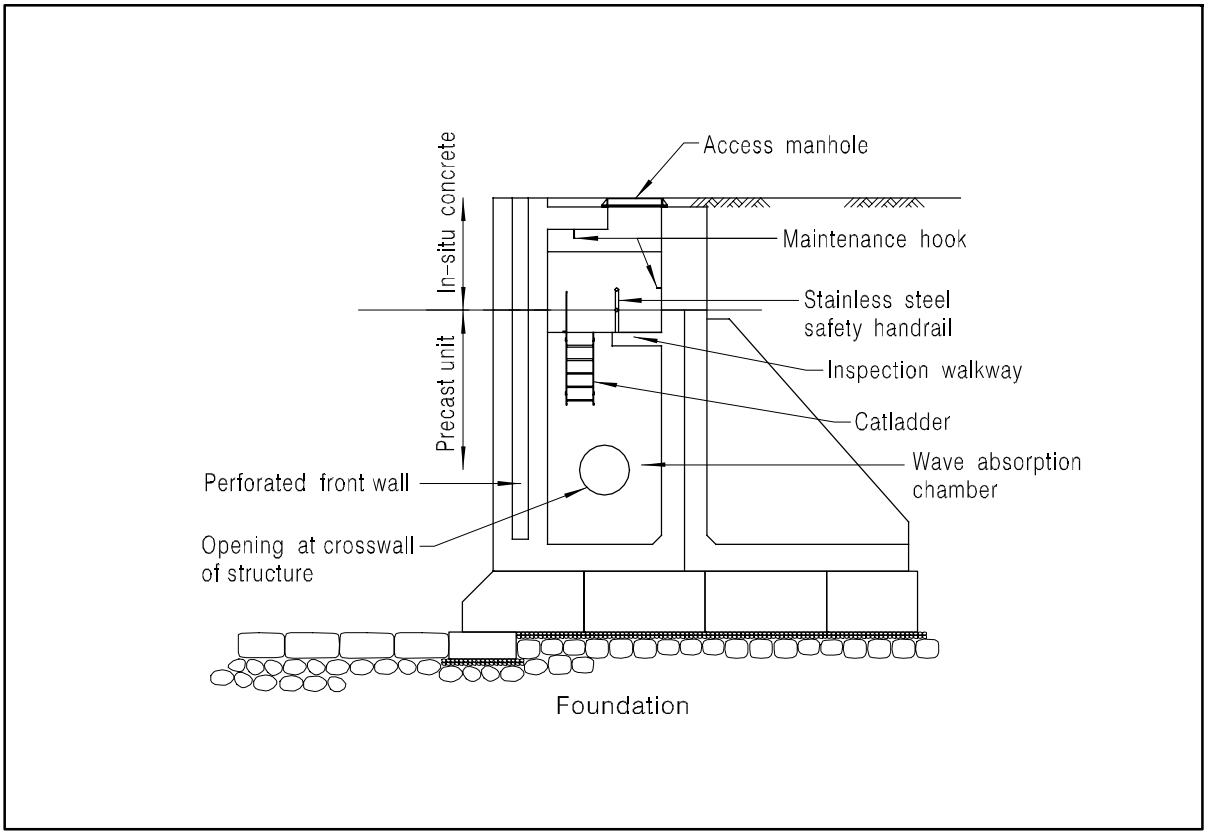
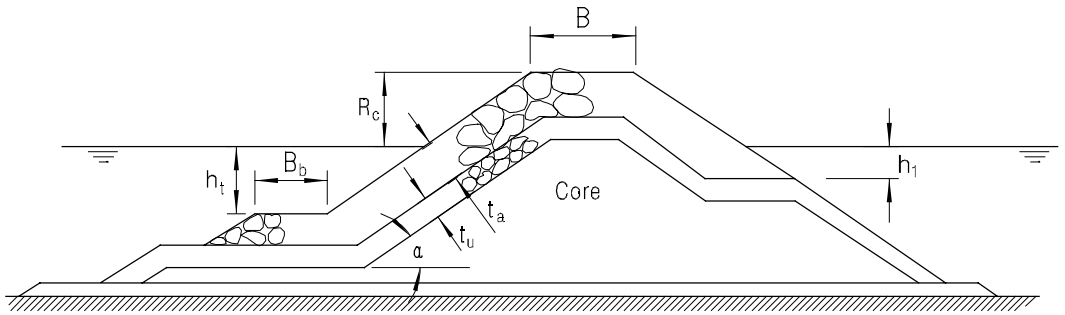
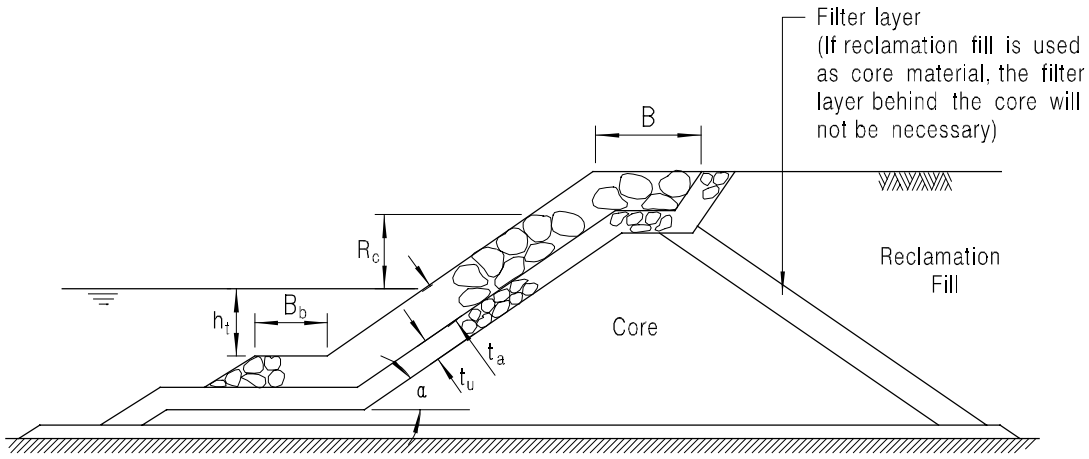


Figure 11 – General Layout of Wave Absorption Seawall



(a) Breakwater



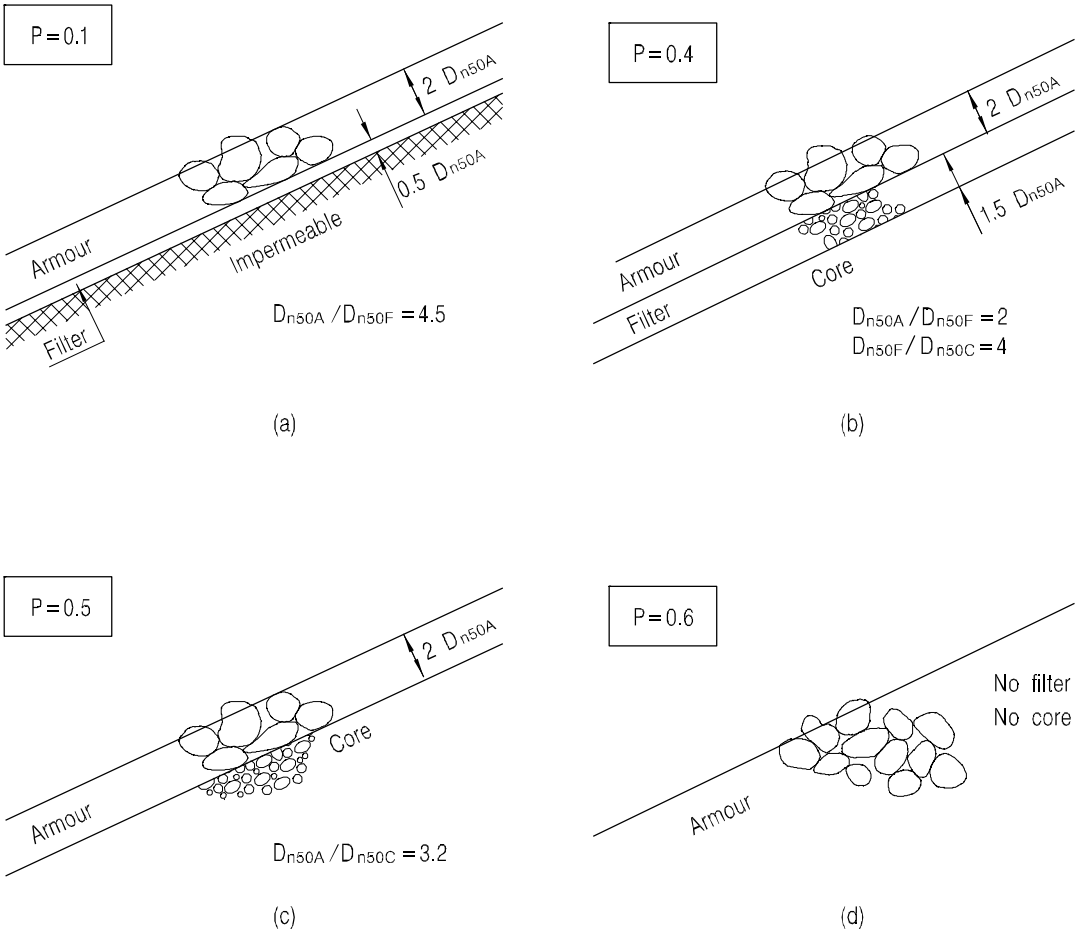
(b) Seawall

B_b = Crest width
B = Top apron width
R_c = Crest free board
t_a = Thickness of armour layer
t_u = Thickness of underlayer
α = Slope of structure
h_t & h_1 = Extent of armour layer on slope

Note:

1. h_t should extend down to the entire slope of the structure in shallow water where waves break.

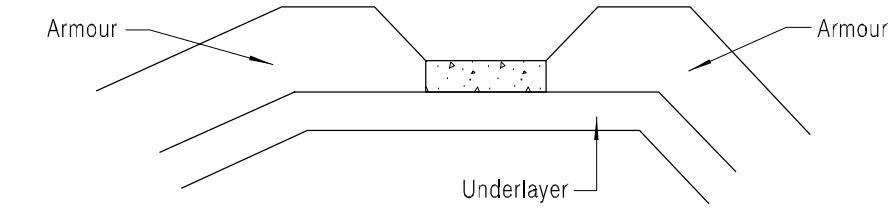
Figure 12 – Definition Sketch for Rubble Mound Breakwaters and Seawalls



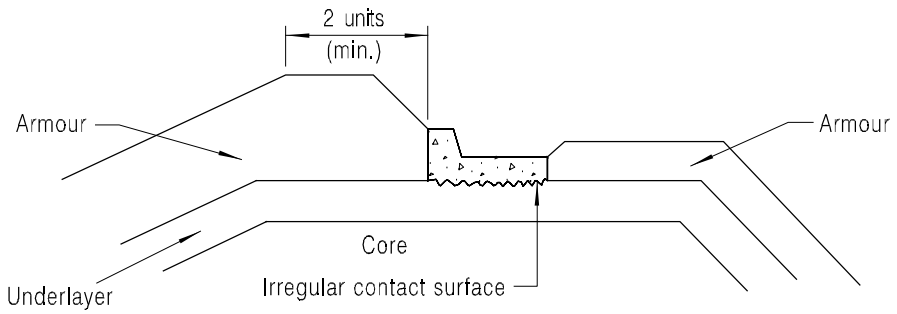
P = Notional permeability factor
D_{n50A} = Nominal diameter armor
D_{n50F} = Nominal diameter filter
D_{n50C} = Nominal diameter core

Source: CIRIA (1991)

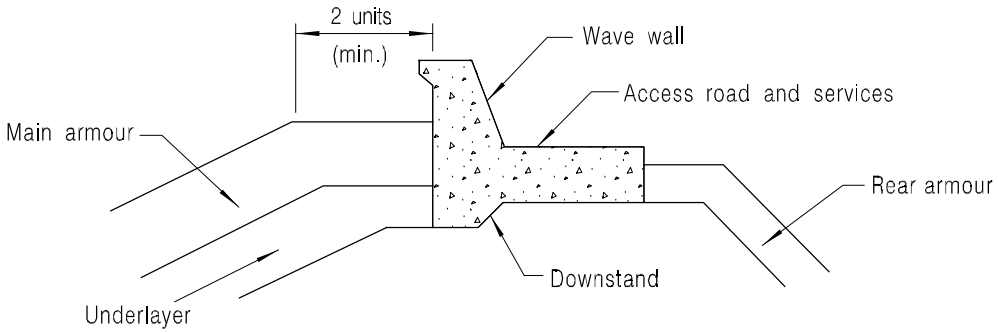
Figure 13 – Notional Permeability Factor



(a) Simple Cap



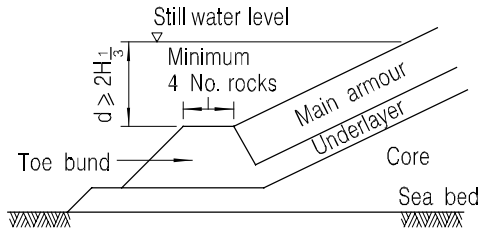
(b) Minimum Crest Wall



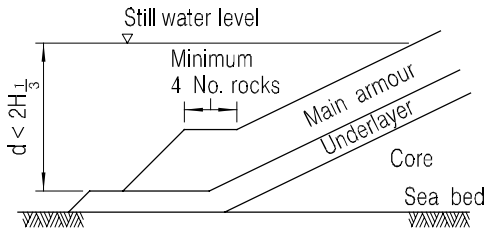
(c) Crest with Wave Wall

Source: BSI (1991)

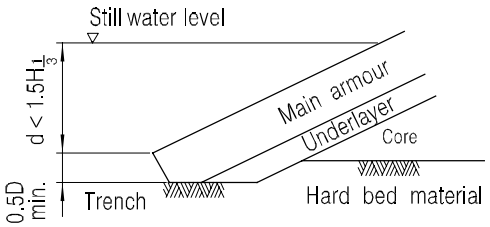
Figure 14 – Typical Crest Structures for Rubble Mound Breakwaters



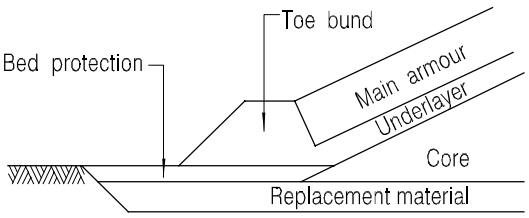
(a) Deep Water Case



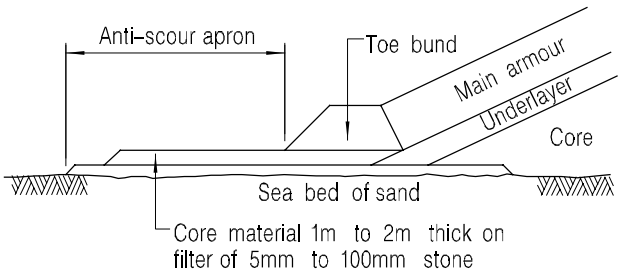
(b) Shallow Water Case



(c) Rock at Bed Level



(d) Soft Material at Bed Level



(e) Protective Toe Apron

d = Still water depth
D = Equivalent cube size of armour

Source: BSI (1991)

Figure 15 – Toe Details for Rubble Mound Structures

$$W = \frac{w_r H^3}{N^3 (s_r - 1)^3}$$

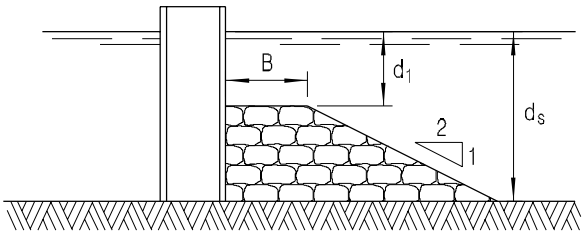
$B \geq 0.4 d_s$

B = 2H or 4 times size of rock whichever is greater

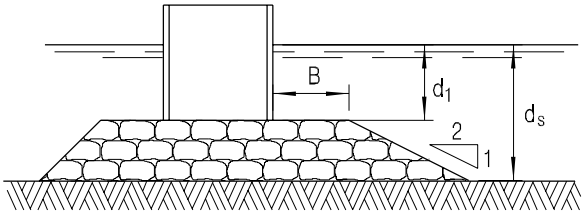
H = Design wave height

w_r = Unit weight of rock

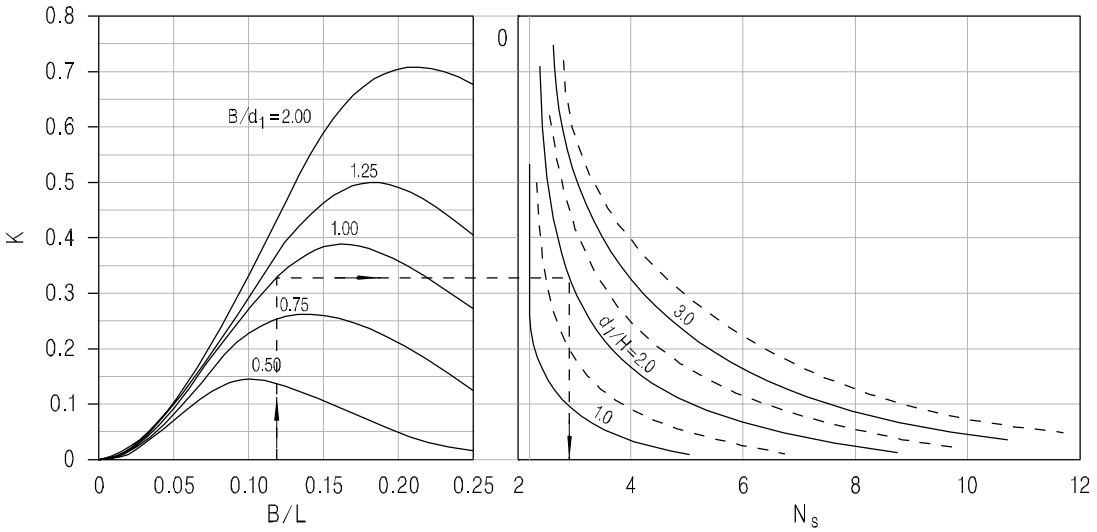
$$s_r = \frac{\rho_{rock}}{\rho_{water}}$$



(a) Rubble Toe Protection



(b) Rubble Foundation



where: K = parameter representing the combined effects of the relative water depth and the relative distance from the vertical wall on the maximum horizontal velocity at the bottom.

H = design wave height associated with depth d_s

L = wavelength associated with the depth d_1

d_s = depth at structure

B = toe apron width

Note:

1. For critical structures at open exposed sites where failure would be disastrous, and in the absence of reliable wave records, the design wave height should be the $H_{1/100}$ during an extreme event at the structure corrected for refraction and shoaling. If breaking might prevent the $H_{1/100}$ wave from reaching the structure, the maximum wave that could reach the structure should be taken for the design value of H. For less critical structures, design wave height could be taken between $H_{1/10}$ and $H_{1/100}$.

Source: CETN (1988) and Tanimoto et al (1982)

Figure 16 – Toe Protection

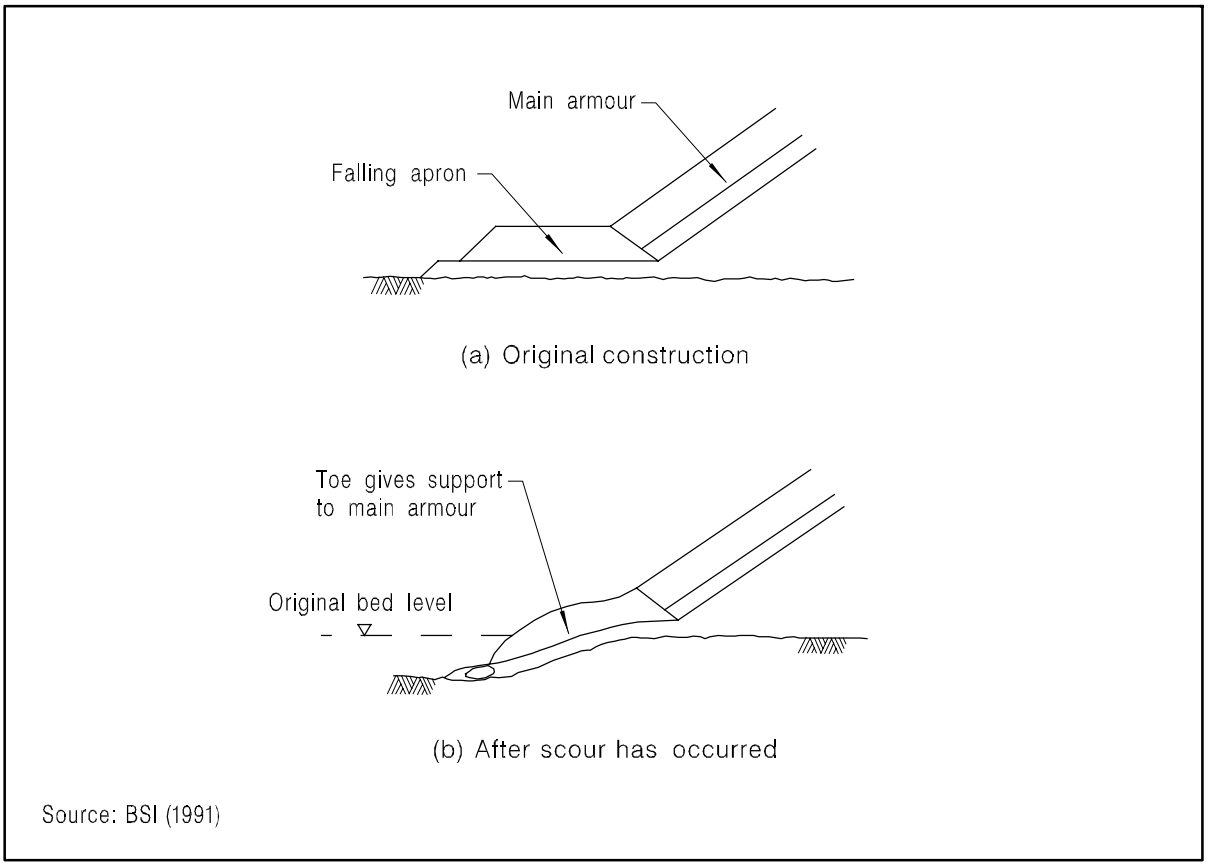


Figure 17 – Falling Apron for Rubble Mound Structures

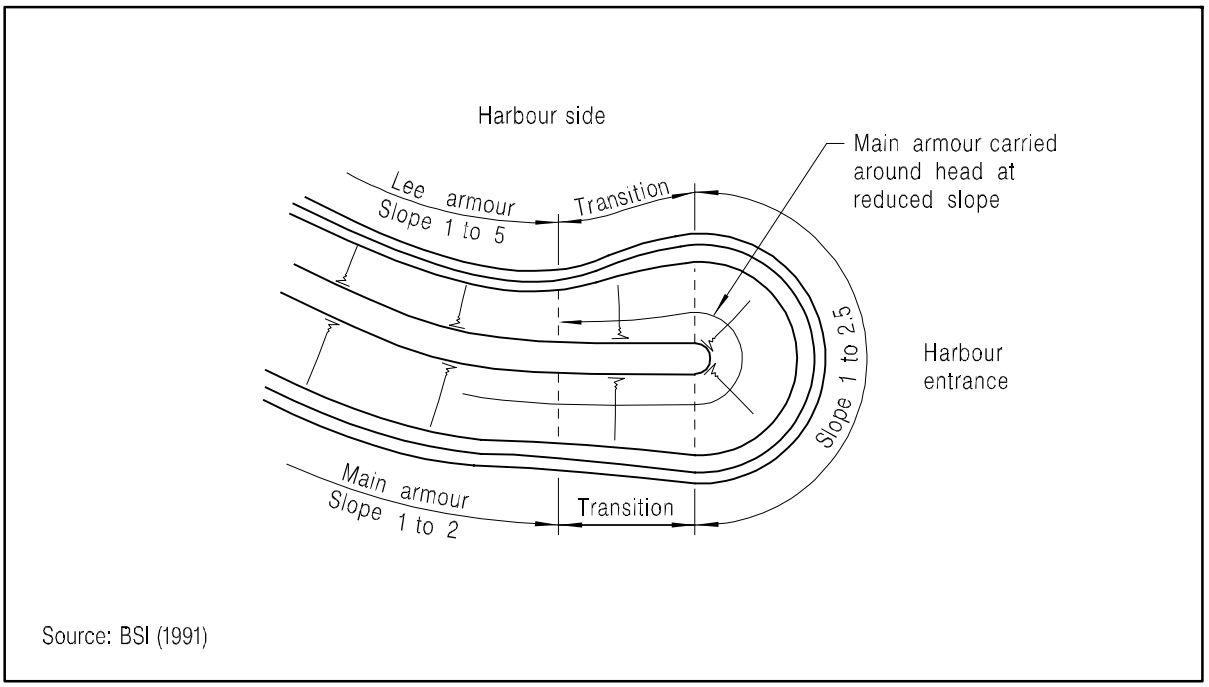
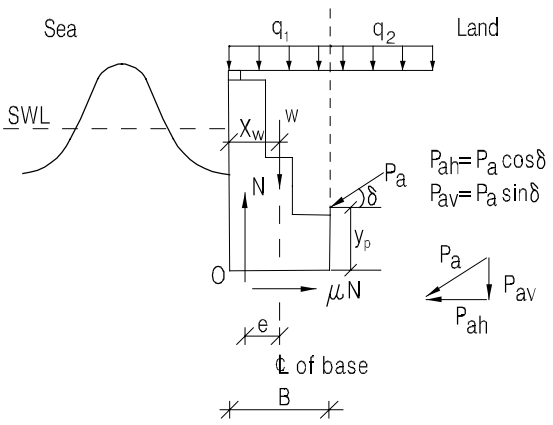
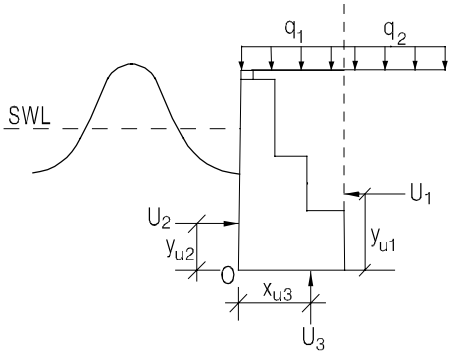


Figure 18 – Typical Breakwater Roundhead Construction



(a) Forces on a Seawall
(Wave and water forces shown in (b))



(b) Wave and Water Forces on a Seawall
(Wave condition with wave trough
at seaward face)

SLIDING FAILURE

Activating force $F_a = P_{ah} + U_1 - U_2$

Resisting force $F_r = \mu N$ where $N = W + P_{av} - U_3$

FOS against sliding = F_r / F_a

OVERTURNING FAILURE

Overturning moment $M_o = P_{ah} y_p - P_{av} B + U_1 y_{u1} - U_2 y_{u2}$

Resisting moment $M_r = W x_w - U_3 x_{u3}$

eccentricity $e = \frac{B}{2} - \frac{M_r - M_o}{N}$

FOS against overturning = M_r / M_o

BEARING CAPACITY FAILURE

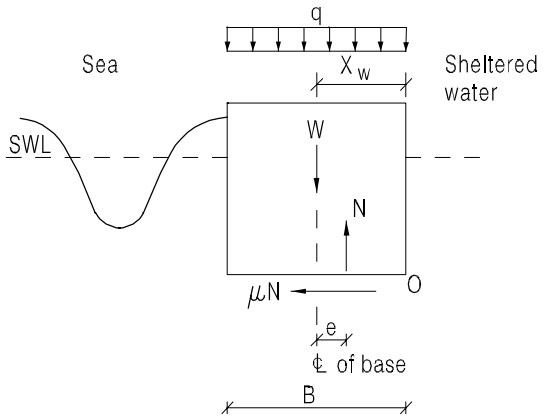
Effective normal load $Q_n = N + q_1 B$

FOS against bearing capacity failure = Q_{ult} / Q_n

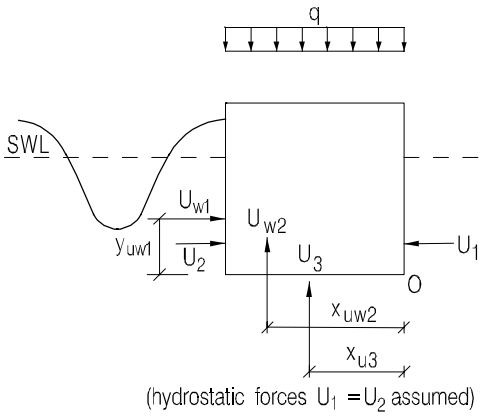
Notes :

1. Other loads not indicated in the figure, if any, should also be taken into account.
2. The total weight W includes the weight of the structure and that of the soil behind the structure up to the virtual back.
3. The soil pressure P_a includes the active pressure behind the virtual back due to soil and the surcharge live load q_2.
4. U_1 refers to the hydrostatic force exerting on the virtual back; U_2 refers to the combined hydrostatic and wave pressure on the seaward face; U_3 refers to the uplift due to combined effect of buoyancy and wave. For details of U_1, U_2 and U_3, see Port Works Design Manual: Part 1 – Figure 17.
5. Live load q_1 to be considered only for checking against bearing capacity failure; otherwise $q_1 = 0$.
6. Coefficient of friction μ may be assumed to be 0.6 for both block /foundation and block / block interfaces.
7. Q_{ult} is the ultimate bearing capacity.

Figure 19 – Stability Calculation for Vertical Seawalls



(a) Forces on a Breakwater
(Wave and Water forces shown in (b))



(b) Wave and Water Forces on a breakwater
(Wave condition with crest at the seaward face and calm sea condition at the sheltered side)

SLIDING FAILURE

Activating force $F_a = U_{w1}$
Resisting force $F_r = \mu N$ where $N = W - U_3 - U_{w2}$
FOS against sliding = F_r / F_a

OVERTURNING FAILURE

Overturning moment $M_o = U_{w1} y_{uw1} + U_{w2} x_{uw2}$
Resisting moment $M_r = W x_w - U_3 x_{u3}$
eccentricity $e = \frac{B}{2} - \frac{M_r - M_o}{N}$
FOS against overturning = M_r / M_o

BEARING CAPACITY FAILURE

Effective normal load $Q_n = N + qB$
FOS against bearing capacity failure = Q_{ult} / Q_n

Notes :

1. Other loads not indicated in the figure, if any, should also be taken into account.
2. The total weight W includes the weight of the structure.
3. U_1 and U_2 refer to the hydrostatic forces exerting on the seaward face and sheltered face respectively ($U_1 = U_2$ assumed); U_3 refers to the buoyancy force; U_{w1} and U_{w2} refer to the horizontal force and uplift due to the waves at the seaward side. For details of U_{w1} and U_{w2}, see Port Works Design Manual: Part 1 – Figures 15 and 16.
4. Live load q to be considered only for checking against bearing capacity failure; otherwise q = 0.
5. Coefficient of friction μ may be assumed to be 0.6 for both block / foundation and block / block interfaces.
6. Q_{ult} is the ultimate bearing capacity.

Figure 20 – Stability Calculation for Vertical Breakwaters