Rubble Mound Breakwater

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1. Abstract

The paper deals with the design of a rubble mound breakwater. The design is carried out according to the Coastal Engineering Manual, 2006, and contains design of the height of the rubble mound breakwater, the stone size in each layer and bearing capacity of the soil.

2. Introduction

Design of rubble mound breakwater is a very complex matter. This is due to all the different parameters that affect the design e.g. wave height, varying water depth, variation of stone size, slope angle, damage level and so on. The design is often based on empirical expressions developed by several experiments.

In the follow, the design is given and after that the height and stone sizes are determined. Finally, the bearing capacity of the soil is determined.

3. Design Conditions

In the following section, the design conditions are described. The rubble mound breakwater must fulfill the conditions given in Table 3.1.

Allowable overtopping, q:	0.4 m^3/sec/m
Armor unit:	rough quarry stone
Armor and under layer material is quarry stone, γ_a:	2.5 t/m^3
Structure slope:	1:2
Shape:	Symmetric

Table 3.1: *Design conditions for the rubble mound breakwater.*

The water depth, h, varies between 5.5m and at high water up to 7.2m. It is therefore necessary to determine the wave length, L, for both cases. The wave length is determined by iterating (.1), where T is the wave period.

$$L=\left(\frac{gT^2}{2\pi}\right)\tanh\left(\frac{2\pi h}{L}\right) \qquad (.1)$$

The design conditions for the water are given in Table 3.2.

Water depth, SWL:	5.5 m
Beach slope:	1:20
Design high water:	1.7 m
Hs	2 m
$H_{1/10}$	2.5 m
T_m or T_{om}	8 sec
L_o	100 m
$L_{h=5.5}$	55.4
$L_{h=7.2}$	62.2

Table 3.2: *Design condition for the water.*

The design conditions for the soil are given in Table 3.3, where it is assumed that it is not possible to have a failure or settlements in the Limestone layer at the depth of 21.5m.

0 m		
	Sand	$\gamma = 17$ kN/m^3
	fine to medium loose	$\varphi = 30°$
		c = 0
5.5 m		
	Clay	$\gamma = 14$ kN/m^3
	Soft	$\varphi = 0°$
	Over-consolidated	c = 50 kPa
		$e_o = 2.2$
		k = 10-5 cm/s
		$a_v = 3\text{x}10^{-3}$ m^2/kN
		$C_c = 0.3$
21.5 m		
	Limestone	

Table 3.3: *Design conditions for the soil.*

Furthermore, a toe, to protect the armor layer, must be constructed but the height is still unknown. The rubble mound breakwater is designed with to under layers beneath the armor layer. Beneath these layers there is a core. The top width of the rubble mound breakwater must a least be 3 times the stone diameter of the armor stones. The breaking condition for wares are given by

$$\frac{H_b}{h_b} = 0.78 \qquad (.2)$$

The minimal water depth for non breaking waves is determined to 3.2m using $H_{1/10}$. This means that the toe must be lower than 2.3m or else the waves will break.

The rubble mound breakwater is illustrated in Figure 3-1. All the relevant parameters for the figure are given in Table 3.1 to Table 3.3.

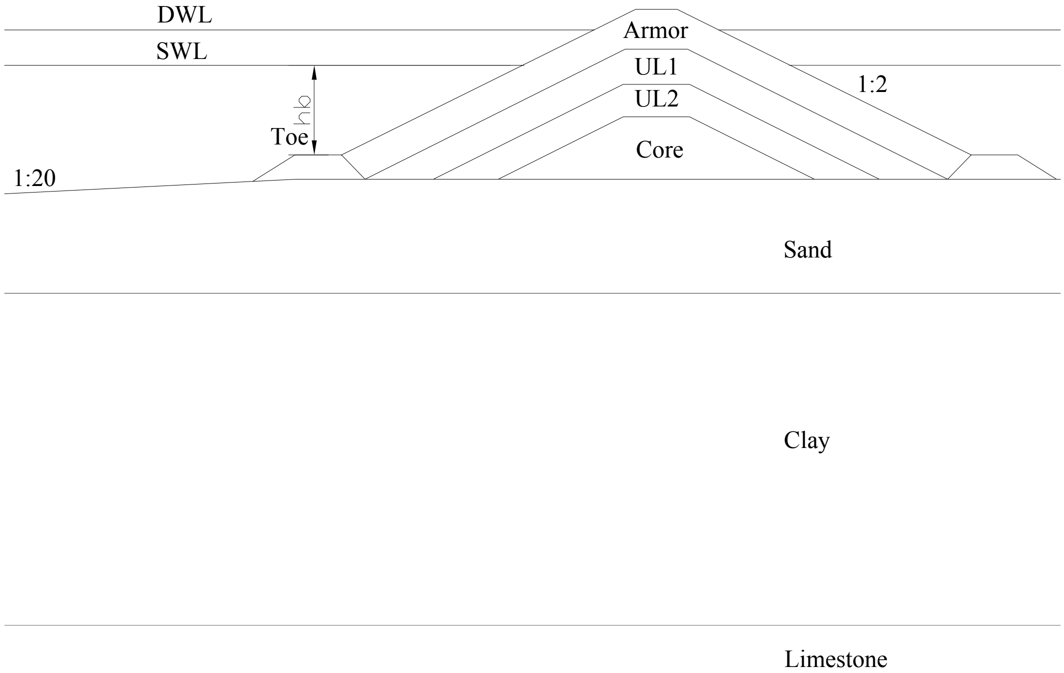

Figure 3-1: *Illustration of rubble mound breakwater.*

4. Design of Rubble Mound Breakwater

In the following chapter, the rubble mound breakwater is designed. The design is done according to Coastal Engineering Manual, 2006. Before the layers can be designed, the elevation is determined. Finally, the toe is designed.

4.1. Design Elevation

The design elevation consist of contributions from a wave run up on the slope, the wave it self, the settlements of the rubble mound and a freeboard which provides safety against overtopping. In the following, these parameters are determined.

Freeboard

The height of the freeboard is determined according to Owen (1980, 1982), Table VI-5-8 in Coastal Engineering Manual. The equation to determine the height is given by

$$\frac{q}{gH_s T_{om}} = a \cdot exp\left(-b \frac{R_c}{H_s} \sqrt{\frac{s_{om}}{H_s}} \frac{1}{\gamma_r} \right) \qquad (.3)$$

where

g is the gravitational acceleration, 9.82m/s^2

R_c is the freeboard or height of elevation

s_{om} $= \frac{H_s}{L_{om}} = \frac{2m}{62.2m} = 0.032$, equation VI-5-2

a is a coefficient that is read to 0.013 (for straight smooth slopes)

b is a coefficient that is read to 22 (for straight smooth slopes)

γ_r is a coefficient that is 0.5 - 0.6, here 0.6 since this yields the biggest freeboard

Note that the wave length for the depth of 7.2m is used since this is the design wave length. The height of the freeboard is then calculated to be 1.24m, which is illustrated in Figure 4-1.

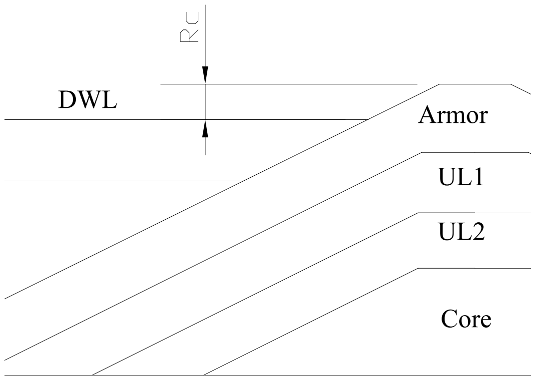

Figure 4-1: *Illustration of freeboard, R_c.*

Wave Run up

Wave rum up is a phenomenon caused by the breaking waves on a slope, cf. Figure 4-2.

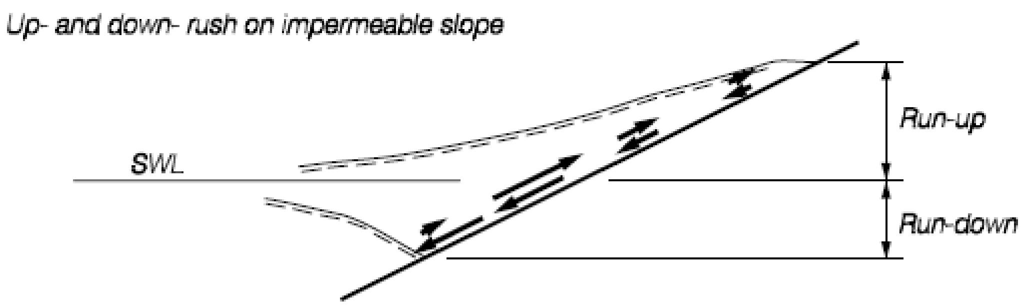

Figure 4-2: *Wave run up and run down. [CEM, 2006]*

The exceedance level is chosen as 2% and for rack armored slopes with irregular waves the run up can be calculated by equation VI-5-13 given by

$$\frac{R_{ui\%}}{H_s} = B\left(\xi_{om}\right)^C \quad for \quad 1.5 \le \xi_{om} \le \left(D/B\right)^{1/C} \tag{.4}$$

where

D is coefficient which is 1.97, c.f. Table VI-5-5

B is coefficient which is 1.17, c.f. Table VI-5-5

C is coefficient which is 0.46, c.f. Table VI-5-5

The surf-similarity parameter ξ_{om} is given by equation VI-5-2

$$\xi_{om} = \frac{tan\,\alpha}{\sqrt{s_{om}}} = 2.79 \tag{.5}$$

The wave run up is then determined according to (.4)

$$\frac{R_{ui\%}}{H_s} = 1.17 \cdot 2.79^{0.46} \quad for \quad 1.5 \le 2.79 \le \left(1.97/1.17\right)^{1/0.46} = 3.1$$
$$\Downarrow \tag{.6}$$
$$R_{ui\%} = 3.75m$$

The design elevation con be determined according to

$$R_{design} = h + \eta + \sum R + \rho_{total} \tag{.7}$$

where

ρ_{total} is the total settlements which here are set to 0.1m

η is the wave setup and equal to 0

The design elevation is determined to

$$R_{design} = 7.2 + 1.24 + 3.75 + 0.1 = 12.3m \tag{.8}$$

4.2. Design of Layers

The mass of the armor layer is determined according to Table VI-5-22. This is for irregular head on waves were the slope is between 1.5<cotα<3 and for non-overtopping. The damage level is chosen to be between 0-5%. The equation is given by

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1 \right)^3 \cot\alpha} \tag{.9}$$

where

ρ_s	is the mass density of rocks, 2.5t/m^3
ρ_w	is the mass density of water, 1t/m^3
H	is the wave height, here H_s
K_D	is the stability coefficient read to 2.4 for non breaking waves

The result is given by

$$M_{50} = \frac{2.5 \cdot 2^3}{2.4 \left(\frac{2.5}{1} - 1 \right)^3 \cdot 2} = 1.23 \frac{t}{m^3} \tag{.10}$$

It is now possible to determine the equivalent cubic length of the median rock

$$D_{n50} = \sqrt[3]{\frac{M_{50}}{\rho_s}} = \sqrt[3]{\frac{1.23}{2.5}} = 0.8m \tag{.11}$$

The width of the crest is determined by equation VI-5-116 given by

$$B = nk_{\Delta} \left(\frac{W}{w_a} \right)^{1/3} \tag{.12}$$

where

n	is the number of stone (3 is recommended as a minimum number)
k_Δ	is the layer coefficient given in Table VI-5-51 and is 1.02 for quarrystones (smooth)
W	is the primary armor unit weight, here equal to M_{50}

w_a is the specific weight of armor unit material

The width is then calculated to

$$B = 3 \cdot 1.02 \left(\frac{1.23}{2.5} \right)^{1/3} = 2.4m \tag{.13}$$

The average thickness of the armor and underlayers (r) are determined by equation VI-5-117 given by (.14) where it is typical that n is 2 for all layers.

$$r = nk_{\Delta} \left(\frac{W}{w_a} \right)^{1/3} \tag{.14}$$

The placing density, also known as the number of armor units per area, is given by equation VI-5-118

$$\frac{N_a}{A} = nk_{\Delta} \left(1 - \frac{P}{100} \right) \left(\frac{w_a}{W} \right)^{2/3} \tag{.15}$$

Where A is the surface area and P is the cover layer average porosity given in Table VI-5-51 and is 38 for quarrystones (smooth). The thickness and the number of armor units per area are then determined

$$r = 2 \cdot 1.02 \left(\frac{1.23}{2.3} \right)^{1/3} = 1.6m \tag{.16}$$

$$\frac{N_a}{1} = 2 \cdot 1.02 \left(1 - \frac{38}{100} \right) \left(\frac{2.5}{1.23} \right)^{2/3} = 4.5 stones / m^2 \tag{.17}$$

The design of the underlayers, the core, and the toe is done according to Figure VI-5-55 shown in Figure 4-3.

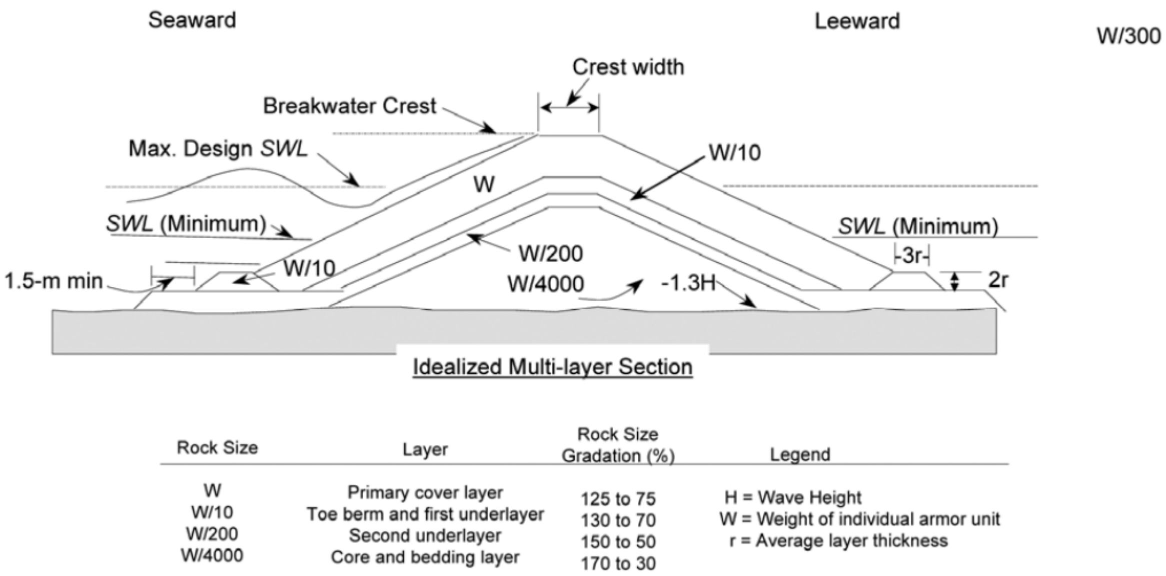

Figure 4-3: *Design of layers. [CEM, 2006]*

Using Figure 4-3 and equation (.14) and (.15) the unit weight, layer thickness and amount of units per area are determined and the results are given in Table 4.1 where the volume of stones for unit length is calculated using Figure 4-4

Layer	W [t/m^3]	D_{n50} [m]	r [m]	N_a [stones/m^2]	Vol. of stones for unit length [m^3/m]
Armor	1.23	0.8	1.6	4.5	98
UL 1	0.12	0.37	0.8	21	46
UL 2	0.006	0.14	0.3	154	17
Toe	0.12	0.37	1.6	21	12
Core	0.003	0.11	-	-	251

Table 4.1: *dimensions for the rubble mound breakwater.*

The core volume of stones pr unit length is found by using a porosity of 64% cf. Table VI-5-51. The volume of all the stones for 1m of rubble mound breakwater is

$$\frac{Vol_{total}}{1m} = 98 + 46 + 17 + 12 + 251 = 424 m^3 / m \qquad (.18)$$

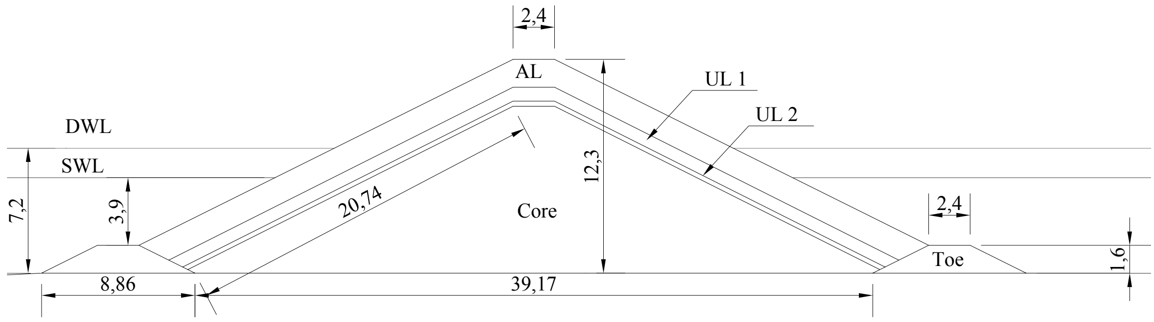

Figure 4-4: *Dimensions for the rubble mound breakwater.*

The total load of the rubble mound breakwater without buoyancy is given by

$$\frac{W}{1m} = 424\, {}^{m^3}\!/\!_{m} \cdot 25\, {}^{kN}\!/\!_{m^3} = 10600\, {}^{kN}\!/\!_{m} \qquad (.19)$$

The load acting on the seabed is affected of the buoyancy and is therefore lower than (.19) suggest. The worst case is for SWL because this gives the smallest buoyancy force. It is assumed due to area considering that 75% of the structure is exposed for buoyancy and the load on the seabed is determined by

$$\frac{W_{seabed}}{1m} = 10600 \cdot 25\% + 10600 \cdot 75\% \cdot \frac{10kN/m^3}{25kN/m^3} = 5830\, {}^{kN}\!/\!_{m} \qquad (.20)$$

5. Bearing Capacity

In the following, a static possible force distribution is used to determine the bearing capacity. When using a static possible force distribution the bearing capacity will be safe. Since there does not exist a simple solution for the bearing capacity of a foundation on two layer soil (sand and clay), it is assumed that the failure will occur in the clay. The force from the rubble mound breakwater is distributed trough the layer of sand as shown in Figure 5-1.

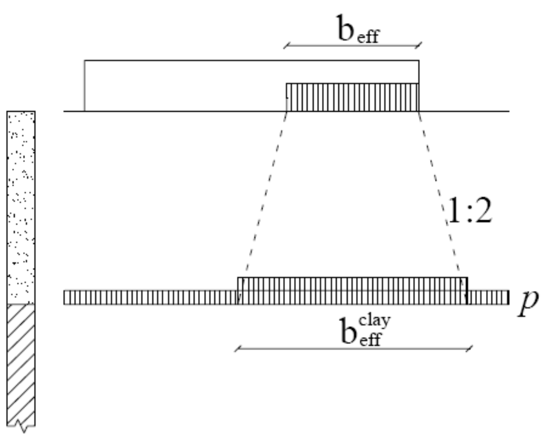

Figure 5-1: *Stress distribution.*

The stress on the clay is then given by

$$\sigma_{clay} = \frac{5830kN/m}{8.86m\cdot 2 + 39.17m + \frac{5.5m}{2}\cdot 2} = 93kPa \qquad (.21)$$

One stress bands is introduced below the stress on the clay as illustrated in Figure 5-2. By using simple static and assuming a Mohr-Coulomb failure function it is possible to determine the bearing capacity of the clay.

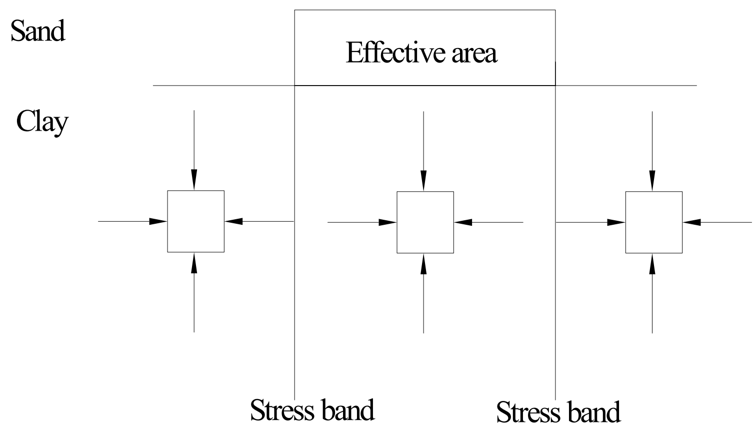

Figure 5-2: *Illustration of one stress band.*

Since the sand gives the same force on both sides of the stress band it can be neglected. The Mohr-Coulomb solution is illustrated in Figure 5-3

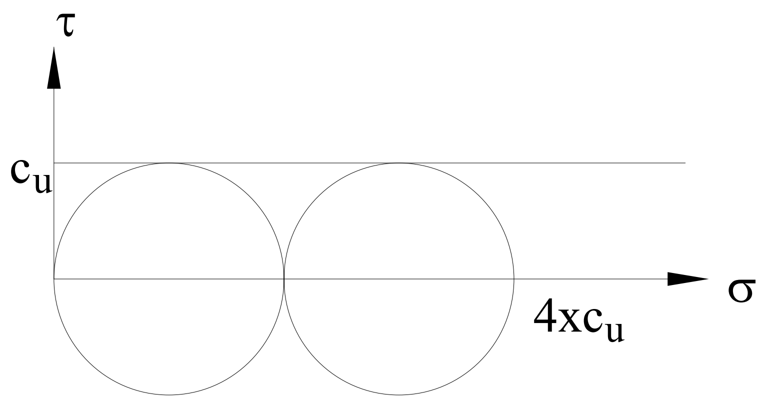

Figure 5-3: *Mohr-Coulombs circles for one stress band.*

As the two Mohr-Coulomb circles suggest, the bearing capacity of the foundation is equal to 4 times c_u and with a c_u of 50kPa, cf. Table 3.3, the bearing capacity is 200kPa and the layer of clay will not fail since the load calculated in (.21) is 93kPa. A more fine static solution can be made by using a infinite numbers of stress bands, but it is not necessary since it only makes the bearing capacity better.

6. Structural Design Summery

The main results are summarized in Table 6.1. The dimension of the rubble mound breakwater is given in Figure 4-4.

$L_{h=5.5}$	55.4
$L_{h=7.2}$	62.2
Freeboard	1.24m
Exceedance level for run-up	2%
Run up	3.75
Design elevation	12.3m
$M_{50armor}$	1.23t/m^3
M_{50ul1}	0.12t/m^3
M_{50ul2}	0.006t/m^3
M_{50core}	0.003t/m^3
Crest width, B	2.4m
r_{armor}	1.6m
r_{ul1}	0.8m
r_{ul2}	0.3m
$N_{a,armor}$	4.5stones/m^2
$N_{a,ul1}$	21stones/m^2
$N_{a,ul2}$	154stones/m^2
$N_{a,toe}$	21stones/m^2
Weight reduced of buoyancy	5830kN/m

Table 6.1: *Main results.*

7. References

[CEM, 2006]
Coastal Engineering Manual, 2006.