CHAPTER 9

Geometrical design of coastal structures

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1 INTRODUCTION

The main contours of a coastal structure are determined by the choice of the type of structure and by the functional requirements. These functional requirements determine, for instance, the layout of the structure and also the required crest height. During the conceptual design phase and later on during the final design, the geometry of the structure will be established. This includes the shape of the structure with the various materials used with their dimensions and layer thickness.

This chapter deals first with the functional design of coastal structures and the various types that exist. The geometrical design of seawalls/dikes and of breakwaters is then treated separately and more in depth. Various design aspects are treated in other chapters. Wave run-up and wave overtopping has been described in chapter 8 and the formulae given there can be used to determine the crest height of dikes and seawalls. Filter structures and design of armour layers are treated in chapters 10 and 11, respectively. Other types of protection of the seaward side of seawalls and dikes are described in chapters 12-18.

References for geometrical, functional and conceptual design are CUR/RWS (1995), CIRIA/CUR (1991), Pilarczyk, ed. (1990) and van der Meer (1993).

2 FUNCTIONAL DESIGN

Design of coastal structures should be based upon the functional requirements taking into account the environmental conditions in the project area and giving due regard to constructional aspects, operation and maintenance. The function of a flood protecting coastal structure is mainly to protect the hinterland against the adverse effect of high water and waves. If high water protection is required the structure should have a height well above the maximum level of wave run-up during storm surges. This normally calls for high crest elevations.

If, however, some overtopping is allowed in view of the character of the hinterland, the design requirement is formulated in terms of the allowable amount of overtopping. Average overtopping discharge values of 1-10 liters per second per running meter of dike may be accepted for instance. Obviously crest elevations can be reduced considerably in this case.

For structures, such as breakwaters, where wave reduction is the main objective, a further reduction in crest height can be applied. Wave heights due to transmission and overtopping should be negligible during operational conditions, but may reach values of the transmitted wave height of 0.3 m to 1 m in extreme design conditions.

Finally, training walls are mainly used to direct flow. The crest elevation is mainly determined by constructional aspects which implies that a minimum level of 2 m above mean high water should be applied to guarantee an uninterrupted progress of work (van der Weide, 1989). Wave overtopping during operational and extreme conditions is of less concern in this case.

3 TYPES OF STRUCTURES

3.1 General

Generation of design concepts is based on both the functional requirements and the experience and creative thinking of the designer (CUR/RWS, 1995). An important criterion in selecting alternatives for further development into well-defined structural concepts is the failure risk involved in the various alternatives, and the relation of this risk to their corresponding benefits. CUR/RWS (1995) gives the following categories of structures where rock is the basic material:

- seawalls and dikes
- breakwaters
- groynes and shore protection breakwaters
- gravel beaches
- offshore bed or scour protection
- closure dams
- barriers, weirs and sills
- bank protection
- river training works, including spur dikes
- bridge priers and abutments
- spillways and outlets

Only the first two categories, seawalls/dikes and breakwaters will be treated in this chapter.

3.2 Seawalls and dikes

Common characteristics for all coastal and shoreline defence structures are in close relation to the land, both in relation to functions and for construction. Seawalls and dikes usually border on shallow water with the corresponding hydraulic loadings.

Seawalls have been constructed with a wide variety of materials and cross-sections. The most common types of seawall cross-sections are shown in Figure 1 (CUR/RWS, 1995). These are:

- slope protection (with or without berm)
- reclamation bund
- rehabilitation mound of an existing vertical wall
- anti-scour mat in front of an existing vertical wall

Dikes usually have a rather mild slope, mostly of the order of 1:2 or milder. A dike consists of a toe construction, an outer slope, often with a berm, a crest of a certain height and an inner slope, see Figure 1 of chapter 8. Figure 2 shows a schematisation of a dike on a distorted scale. The outer slope may consist of various materials such as asphalt, a revetment of concrete blocks, or grass on a clay cover layer. Combinations of these are also possible. Slopes are not always straight; the upper and lower parts do not always have a similar gradient if a berm has been applied. Figure 3 gives another example of the seaward side of a dike.

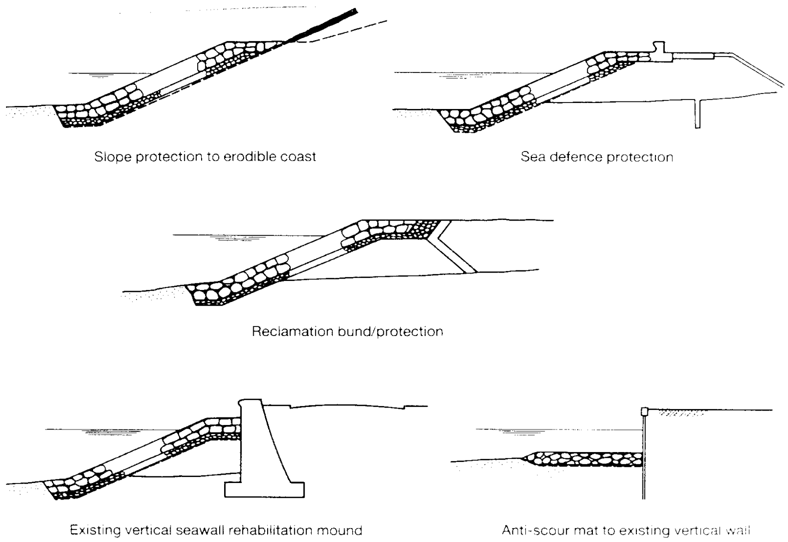

Figure 1 Basic seawall concepts (from CUR/CIRIA, 1995)

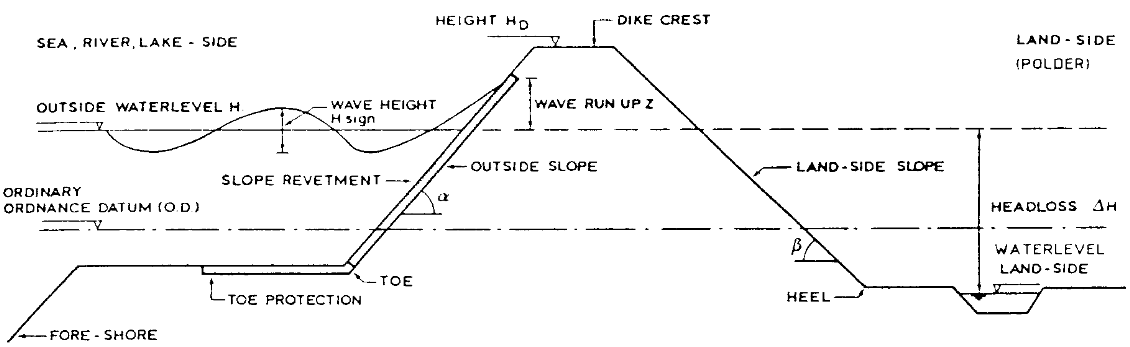

Figure 2 Schematisation of a dike (distorted scale)

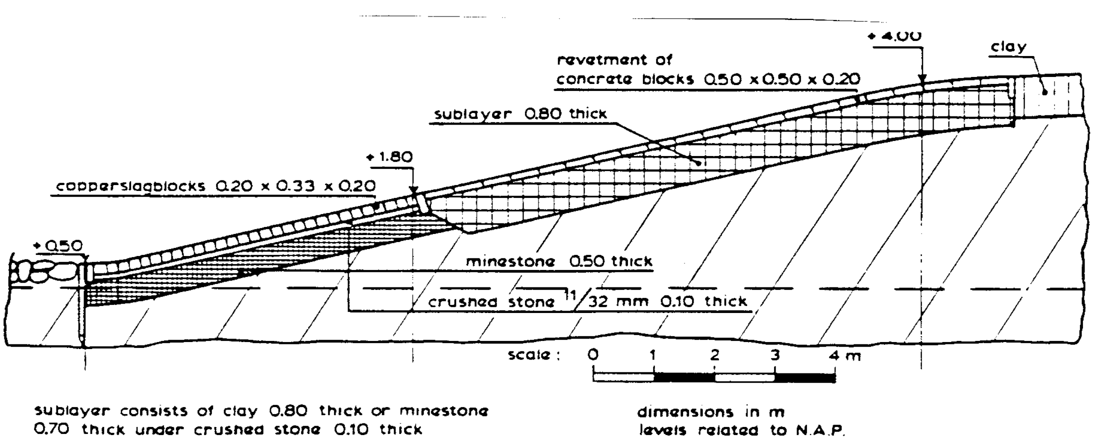

Figure 3 Example of dike protection (from Pilarczyk, ed., 1990)

3.3 Breakwaters

Both the alignment and the cross-section of a breakwater affect -to a certain extent- the hydraulic loading of the armour or cover layer. Moreover, the bulk volume of a breakwater is mainly determined by these geometrical characteristics. In many cases breakwaters are exposed to relatively heavy wave loadings because of their protruding situation.

Given the strong dependency of the required armour strength on the wave height, often high demands must be made upon the armour elements, construction techniques and equipment. Depending upon the specific function to the breakwater, overtopping may be allowed or not, a choice which has important consequences for the design of the structure. The most common breakwater concepts are shown in Figure 4 and given by CUR/CIRIA (1995):

- conventional rubble mound breakwater
- berm breakwater
- reef type structure
- low-crested/submerged breakwater
- caisson breakwater on rock foundation
- composite caisson/rubble mound breakwater

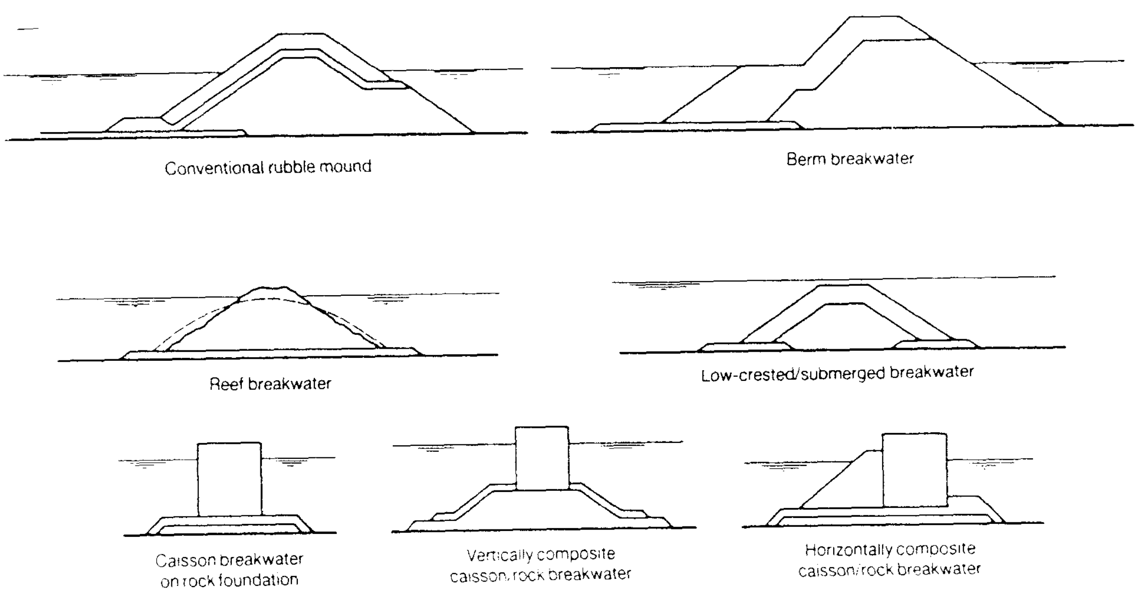

Figure 4 Basic breakwater concepts (from CUR/CIRIA, 1995)

4 GEOMETRICAL DESIGN OF DIKES AND SEAWALLS

4.1 Loading zones

The degree of wave attack on a dike or seawall during a storm surge depends on the orientation in relation to the direction of the storm, the duration and strength of the wind, the extend of the water surface fronting the seawall and the bottom topography of the area involved. For coastal areas there is mostly a certain correlation between the water level (tide plus wind set-up) and the height of the waves, because wind set-up and waves are both caused by wind. Therefore, the joined frequency distribution of water levels and waves seems to be the most appropriate for the design purposes (Pilarczyk, ed., 1990).

For seawalls and dikes in the tidal region, fronting deep water, the following approximate zones can be distinguished:

- the zone permanently submerged. This zone is not present in the case of a high level foreshore
- the zone between MLW and MHW; the ever-present wave loading of low intensity is of importance for the long-term behaviour of the structure
- the zone between MHW and the design level; this zone can be heavily attacked by waves, but the frequency of such attack reduces as one goes higher up the slope
- the zone above design level, where there should only be wave run-up.

A bank slope revetment in principle functions no differently under normal circumstances than under extreme conditions. The accent is, however, more on the persistent character of the wave attack rather than on its size. The quality of the seaward slope can, prior to the occurrence of the extreme situation, already be damaged during relatively normal conditions to such a degree that its strength is no longer sufficient to provide protection during the extreme storm.

The division of the slope into loading zones has not only direct connection with the safety against failure of the revetment and the dike as a whole, but also with different application of materials and execution and maintenance methods for each zone, see for instances Figure 3. It is emphasized that for each design phase these alternatives should be elaborated at a comparable level of detail. The same applies to the construction alternatives which may have a great influence on the total structure cost.

4.2 Dike or seawall shape

The average slope angle of the bank may not be so steep that the whole slope or revetment can loose stability through sliding. This criterion gives the maximum slope angle.

Gentler (flatter) slopes lead to a reduced wave force on the revetment and less wave run-up; wave energy is dissipated over a greater length. By using the wave run-up or wave overtopping approach (chapter 8) for calculations of the crest height of a trapezoidal profile of a dike for different slope angles, the minimum volume of the embankment can be obtained.

However, this does not necessarily imply that minimum earth volume coincides with minimum costs. An expensive part of the embankment comprises the revetment of the seaward side slope and the slope surface increases as the slope angle decreases. The optimum cross-section, based on costs, can be determined if the costs of earth works per m^3 and those of the revetment per m^2 are known. Careful attention is needed, however, because the revetment costs are not always independent of the slope angle. For example, for steep slopes heavy protection is required while for mild slopes the cheaper grass mat can often provide a sufficient protection.

Another point of economic optimization can be the available space for dike construction or improvement.

Common Dutch practice for a dike is to apply a slope of 1:3 on the inner slope and between 1:3 and 1:5 on the seaward slope. The minimum crest width is 2 m. The seaward side berm is a common element in Dutch dike construction. It could in the past lead to a reduction in the expenditure on stone revetments as on a very gentle sloping berm a good grass mat can be maintained and it produced an appreciable reduction in wave run-up.

Present practice, in order to obtain a substantial reduction in wave run-up or wave overtopping, is to place the outer berm at or close to the water level of the design storm flood. If the berm lies too much below that level the highest storm flood waves would not break beneath or on the berm and the run-up will be inadequately affected, the grass mat on the upper slope too heavily loaded by waves which may lead to possible erosion. For the storm flood berm at the high design levels as in the Netherlands (design return period 10,000 years) there are in general no problems with the growth of grass on the berm and the upper slope.

However, there can be circumstances which require also the application of a hard revetment on the berm and even on a part of the upper slope. This is the case when high water levels frequently occur, leading to more frequent run-up on the upper part by salt water. A common grass mat can only survive a few salty events a year.

An important function of the berm can be its use as an access road for dike maintenance. In general care should be taken to prevent erosion of the grass mat at the junction with the revetment. The abrupt change in roughness may lead to more local erosion. It is advised to create a transition zone by applying cell-blocks, geogrids or other systems allowing vegetation.

The influence of slope angle and the application of a berm is shown in Figure 5. Three cross-sections have been drawn, all with the same wave run-up level. The steepest slope 1:3 gives the highest crest height. A gentler slope 1:4 reduces the crest height and even the volume required for the dike. A berm gives another reduction in dike height and volume.

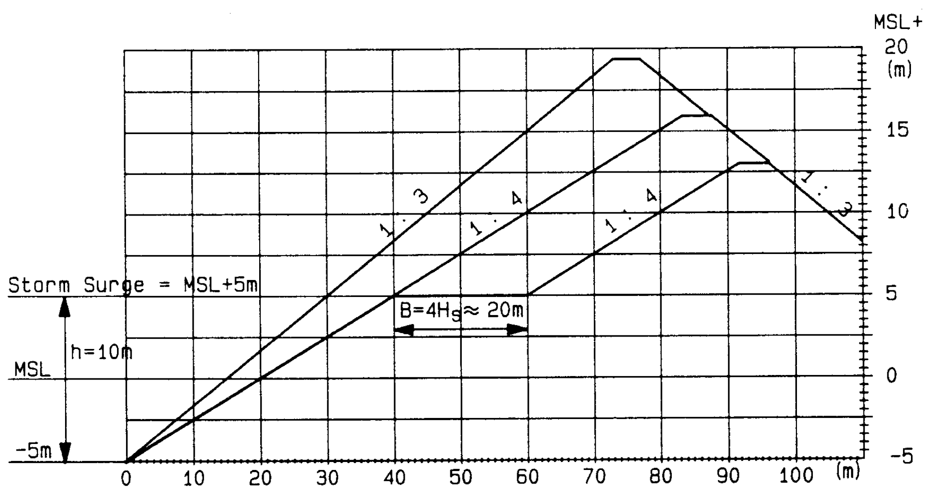

Figure 5 Example of different dike shapes and height with the same 2%-wave run-up level (from Pilarczyk, ed., 1990)

4.3 Dike or seawall height

The height of a dike for many centuries has been based on the highest known flood level that could be remembered. It is evident that in this way the real risk of damage or the probability of flooding was unknown. Little was known about the relation between the cost to prevent flooding and the cost of damage that might result from flooding.

In the twentieth century it was found that the occurrence of extremely high water levels and wave heights could adequately be described by probability distributions. However, the extreme distributions, often based on relatively short periods of observations, mostly have to be extrapolated into regions far beyond the field of observations.

After the 1953 disaster the probability of flooding was studied in the Netherlands in relation to the economical aspects. Finally, it was decided to base the design of most of the sea dikes on a storm surge level with a return period of 10,000 years. The main reason for this large return period is the enormous economical damage that occurs if dikes in the low laying areas of Holland breach. It is much cheaper to build higher dikes than to bear the costs of a flooding.

This may be different in other areas, for example in the UK, where only small areas will be affected and where the inundation depth may be smaller than in the Netherlands.

In the Netherlands the wind set-up is mostly incorporated in the estimated storm surge level. If it is not the case, the wind set-up should be calculated separately and added to the design water level. Besides the design flood level various other elements play a role in determining the design crest level, see Figure 6:

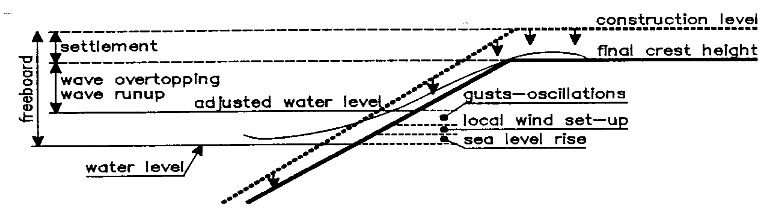

Figure 6 Important aspects when computing the dike height

- wave run-up or overtopping height. Depends on wave height and period, wave angle of approach, roughness and permeability of the slope, and the profile shape. See chapter 8.
- an extra margin to the dike height to take into account seiches (oscillations) and gust bumps (single waves resulting form a sudden violent rush of wind); this margin in the Netherlands varies from 0-0.3 m for the seiches and 0-0.5 m for the gust bumps, depending on the location.
- a change in bottom level or a rise of the mean sea level (the forecast for the estimated life time of the structure).
- settlement of the subsoil and the dike body during its life time.

The combination of all these factors mentioned above defines the crest freeboard of the dike and the dike height for construction.

5 GEOMETRICAL DESIGN OF BREAKWATERS

5.1 Crest height

Wave run-up

The crest height of seawalls with rubble mound protection or armouring may well be determined by an allowable overtopping percentage. This means that under design conditions only a few percentage of the waves may reach the crest and inner side of the structure. A formula for the 2%-wave run-up has been given in chapter 8. Figure 7 gives the formula in a graph and gives a comparison with a smooth slope. In many situations a rock structure can have a much lower crest height than a smooth structure like a dike.

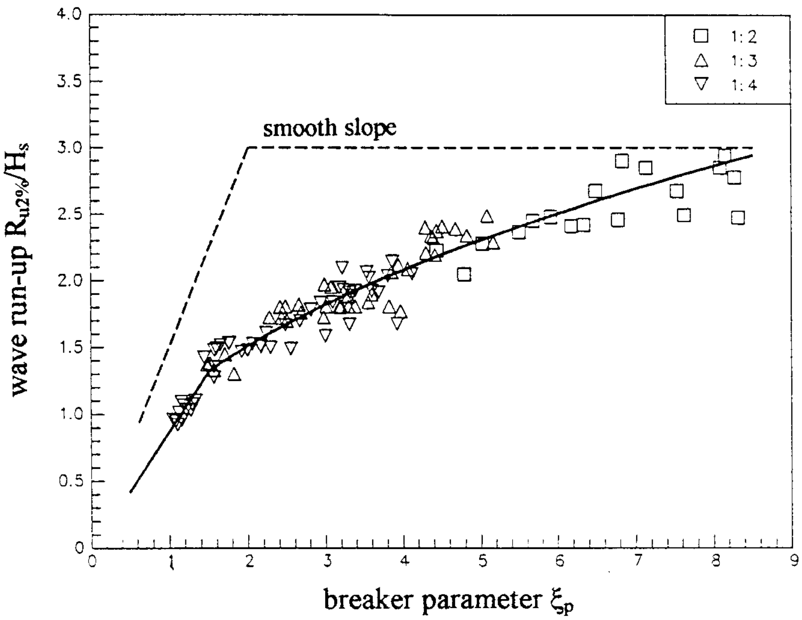

Figure 7 Wave run-up (2%) on rock slopes

It is also possible to allow a larger overtopping percentage under design conditions, but for percentages larger than about 10-15 the overtopping waves will generate a transmitted wave height behind the structure which may become about 10% of the incident wave height. Van der Meer (1993) gives a formula for the distribution of the run-up levels on a rock slope. Based on that formula it is possible to determine the crest height of a rock structure for any desired overtopping percentage.

Simple formula for wave transmission

In most cases the crest height is determined by a limited allowable wave height behind the structure, the so-called transmitted wave height. The transmission coefficient C_t is the ratio of transmitted and incident wave height. An overall view of available data on wave transmission is given in Figure 8 (van der Meer, 1993). The most simple relationship can be found if the transmission coefficient is related to the relative crest freeboard, i.e. the difference between the design water level (see Figure 6) and the crest freeboard R_c/H_i. A value of 1 means that the crest height is one wave height above the water level, a value of 0 gives a structure with the crest level at the water level.

The fitted relationships in Figure 8 may be described as follows:

$$
\begin{array}{lll}
\text{for } -2 < R_c/H_i < -1.13 & C_t = 0.8 & \\
\text{for } -1.13 < R_c/H_i < 1.2 & C_t = 0.46 - 0.3\, R_c/H_i & (1) \\
\text{for } 1.2 < R_c/H_i < 2 & C_t = 0.1 &
\end{array}
$$

The relationships give a simplistic description of the data available, but will often be sufficient for preliminary design. The upper and lower bounds of the data considered are given by the 90% confidence bands. The standard deviation, measured vertically, is $\sigma_{Ct} = 0.09$.

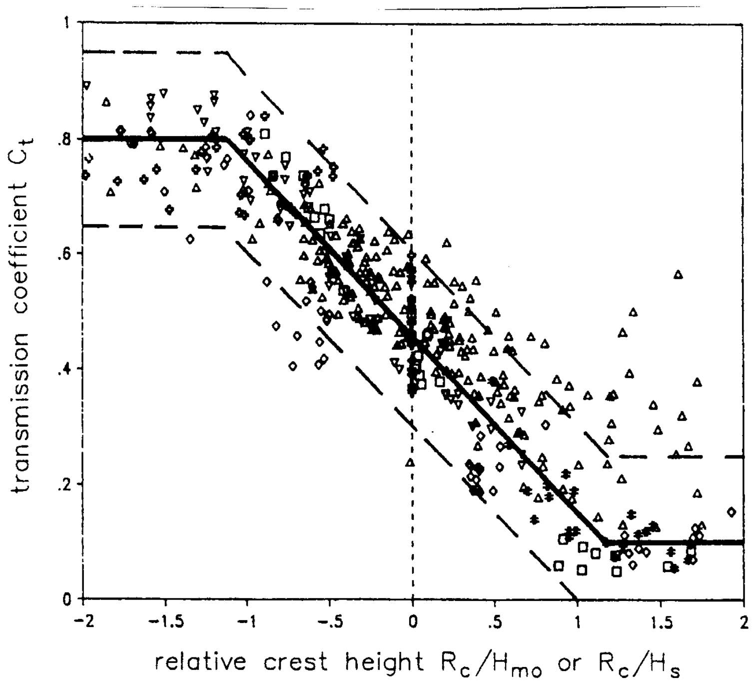

Figure 8 Simple relationship for wave transmission over rubble mound structures (van der Meer, 1993)

Sophisticated approach on wave transmission

More recent research by de Jong (1996) and d'Angremond et al. (1996) has given the influence of wave steepness, slope angle and the crest width on wave transmission. The principle equation is similar to formula 1, i.e. a straight decreasing line from large to small wave transmission with R_c/H_i as parameter (see Figure 8):

$$C_t = a - 0.4\, R_c/H_i \qquad \text{with a maximum of } C_t = 0.8 \text{ and a minimum of } C_t = 0.075 \qquad (2)$$

The parameter "a" describes all the other relevant influences:

$$a = (B/H_i)^{-0.31} * (1 - e^{-0.5\xi}) * A_{str} \qquad (3)$$

with: B = crest width

ξ = breaker parameter, see formula 1 in chapter 8

A_{str} = a coefficient depending on the type of structure:

rock slopes and concrete units:	$A_{str} = 0.64$
smooth impermeable dam (asphalt)	$A_{str} = 0.80$
impermeable smooth block revetment	$A_{str} = 0.80$
block mattresses	$A_{str} = 0.75$
gabion matresses	$A_{str} = 0.70$

The standard deviation around formula (2) is given by $\sigma_{Ct} = 0.06$, resulting in a 90% confidence band of $C_t \pm 0.10$, a considerable improvement with respect to formula 1.

Percentage of overtopping waves related to wave transmission

Not all incident waves will overtop a low-crested structure. The lower the crest of the structure the more waves will overtop and increase wave transmission. Project-related scale model tests at Delft Hydraulics have given the relationship between the percentage of overtopping waves and the wave transmission. The tests were related to conventional breakwater cross-sections, armoured with tetrapods or accropode and the crest had a (low) concrete superstructure.

Overtopping was defined as wave passing the front wall of the superstructure and this was measured with a wave gauge. An overtopping wave hit the gauge and gave a peak on the signal. All the peaks (overtopping waves) were counted and related to the total of incident waves. This gave the overtopping percentage.

Figure 9 gives the percentage of overtopping waves as a function of the relative crest freeboard. It appeared that the armour size had influence on the percentage of overtopping waves. The relative crest freeboard, therefore, was defined by R_c*D_n/H_i^2. The nominal diameter D_n is the cubical size of a unit and is described in chapter 11. The figure gives no difference between tetrapods and accropode.

Figure 10 gives the corresponding wave transmission coefficients in a similar way as in Figure 8. Wave transmission was not measured in all the tests given in Figure 9. Figure 10 shows that for high crests, say $R_c/H_i > 1$, always some wave transmission can be expected. This wave transmission goes through the breakwater and is not generated by overtopping waves.

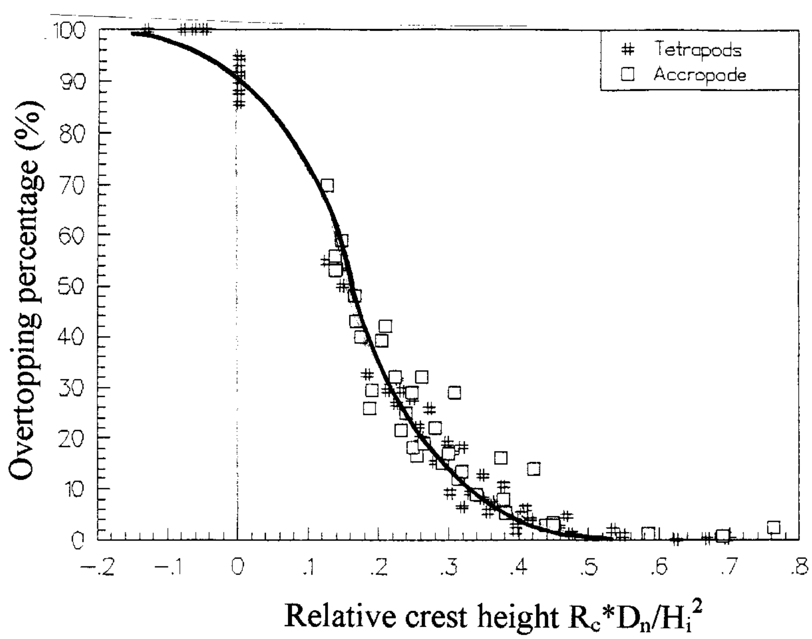

Figure 9 Percentage of overtopping waves as a function of crest height (conventional breakwater)

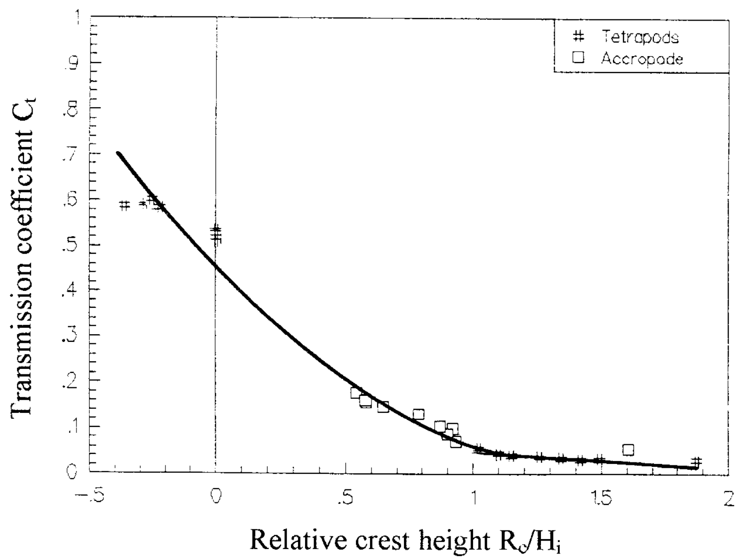

Figure 10 Wave transmission as a function of relative crest height
(conventional breakwaters)

5.2 Cross-section

A breakwater consists of various parts as can be seen in Figure 11. The core and toe may be based on filter layers which protect sand coming through the rock. In order to have a smooth transition from core to bigger rock or artificial units one or more under layers may be designed. The armour layer may be continued over the crest (low-crested structures) or a superstructure may be designed. A superstructure gives easy access to the breakwater.

The design of armour layers has been given in chapter 11. Scouring and toe protections have been described in chapters 7 and 18 respectively.

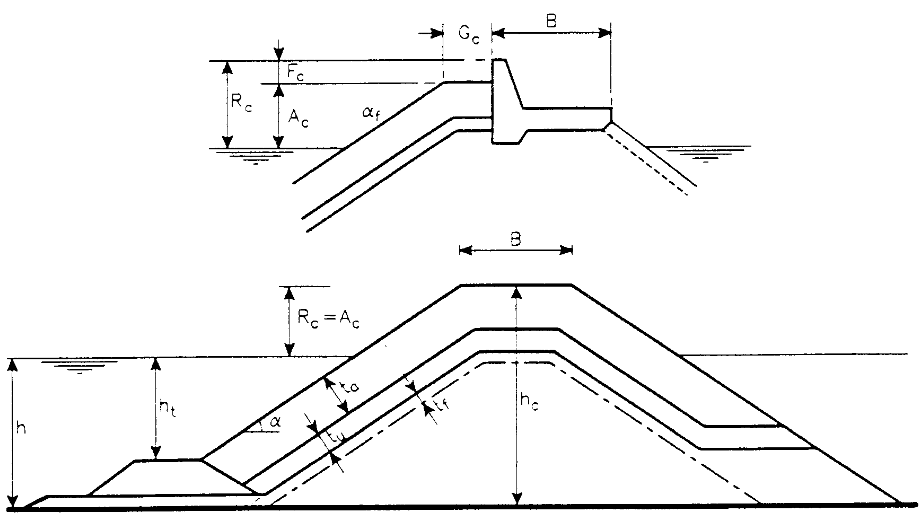

Figure 11 Cross-section of a conventional breakwater

Some general points and design rules for the geometrical design of the cross-section will be given here. These are:

- the minimum crest width
- the thickness of (armour) layers
- the number of units or rocks per surface area
- the grading of rock
- the bottom elevation of the armour layer
- the supporting toe
- the crown wall

The crest width is often determined by construction methods used (access on the core by trucks or crane) or by functional requirements (road/crown wall on the top). Where the width of the crest can be small a required minimum width B_{min} should be provided, where (SPM, 1984):

$$B_{min} = (3 \text{ to } 4)\, D_{n50} \tag{4}$$

The thickness of layers is given by:

$$t_a = t_u = t_f = n\, k_t\, D_{n50} \tag{5}$$

The number of units per m^2 is given by:

$$N_a = n\, k_t\, (1 - n_v)/D_{n50}^2 \tag{6}$$

where:

t_a, t_u, t_f = thickness of armour, under layer or filter
n = number of layers
k_t = layer thickness coefficient
n_v = volumetric porosity

Values of k_t and n_v as given in the SPM (1984) are summarised in Table 1:

Table 1 Values of k_t and n_v (SPM, 1984)

	k_t	n_v
smooth rock, n = 2	1.02	0.38
rough rock, n = 2	1.00	0.37
rough rock, n > 3	1.00	0.40
graded rock	-	0.37
cubes	1.10	0.47
tetrapods	1.04	0.50
dolosse	0.94	0.56

The number of units in a rock layer depends on the grading of the rock. The values of k_t that are given above describe a rather narrow grading (uniform rock). For riprap and even wider graded material the number of rocks cannot easily be estimated. In that case the volume of the rock on the structure can be used.

The grading of the rock can be given by D_{85}/D_{15}, where D_{85} and D_{15} are the 85% and 15% values of the sieve curves, or by D_{n85}/D_{n15}, based on the mass distribution curves. Examples of gradings are shown in Table 2 showing the relationship between class of stone and D_{85}/D_{15}. Further details of recommended methods of specifying gradings and of suggested gradings are given in CUR/CIRIA (1991).

Table 2 Examples of rock gradings

Narrow grading		Wide grading		Very wide grading	
$D_{85}/D_{15} < 1.5$		$1.5 < D_{85}/D_{15} < 2.5$		D_{85}/D_{15}	
Class	D_{85}/D_{15}	Class	D_{85}/D_{15}	Class	D_{85}/D_{15}
15 – 20 t	1.10	1 – 9 t	2.00	50- 1000 kg	2.71
10 – 15 t	1.14	1 – 6 t	1.82	20 -1000 kg	3.68
5 – 10 t	1.26	100 – 1000 kg	2.15	10 – 1000 kg	4.64
3 – 7 t	1.33	100 – 500kg	1.71	10 – 500 kg	3.68
1 – 3 t	1.44	10 – 80 kg	2.00	10 – 300 kg	3.10
300 – 1000 kg	1.49	10 – 60 kg	1.82	20 – 300 kg	2.46

The bottom elevation of the armour should be extended down slope to an elevation below minimum still-water level of at least one (significant) wave height, if the wave height is not limited by the water depth. Under depth limited conditions the armour layer should be extended to the bottom as shown in Figure 11 and supported by a toe.

5.3 Supporting toe

In most cases the armour layer on the seaside near the bottom is protected by a supporting toe, see Figure 11. If the rock in the toe has the same dimensions as the armour, the toe will be stable. In most cases, however, one wants to reduce the rock size in the toe. The most simple relationship is assumed if the stability number $H_s/\Delta D_{n50}$ (see chapter 11) is related to the relative depth h_t/h, where h_t is the depth of the toe below still-water level and h is the water depth just in front of the toe. CIRIA/CUR (1991) and van der Meer (1993) gave this relationship with a small number of tests from Delft Hydraulics (DH) and the Danish Hydraulic Institute (DHI). Later Gerding (1993) conducted small scale model tests specially on toe stability, see also van der Meer et al. (1995).

First of all the damage level was better defined. The damage level N_{od} was used (see also chapter 11). This is the actual number of displaced stones related to a width, along the longitudinal axis of the structure, of one nominal diameter D_{n50}. N_{od} = 0.5 means start of damage (a safe figure for design), N_{od} = 2 gives some flattening out and N_{od} = 4 means complete flattening out of the toe. This applies to a “standard” toe size of about 3 – 5 stones wide and 2 – 3 stones high. For wider toe structures a higher damage level can be applied before flattening out occurs.

One of the conclusions of the research was that wave steepness had no influence on stability. Van der Meer et al. (1995) gave an improved formula with respect to toe stability, where the toe depth was given as h_t/D_{n50}. As D_{n50} appeared also in the stability number $H_s/\Delta D_{n50}$ it was found later on that for low toe structures unrealistic (even negative!) required toe diameters could be calculated. Therefore, Gerding’s work has been re-analysed. Figure 12 gives all his data with a design formula. This formula is almost similar to the original simple formula of van der Meer (1993), but based on more data points. The formula on toe stability can be written as follows:

$$H_s/\Delta D_{n50} * N_{od}^{-0.15} = 2 + 6.2\ (h_t/h)^{2.7} \qquad (7)$$

A high toe, say $h_t/h < 0.4$ comes close to a berm and, therefore, close to the stability of the armour layer. Armour layers have stability numbers close to $H_s/\Delta D_{n50} = 2$. This is the reason that formula 7 does not start in the origin, but at $H_s/\Delta D_{n50} * N_{od}^{-0.15} = 2$ for $h_t/h = 0$. Formula 7 can be used in the range:

$0.4 < h_t/h < 0.9$
$3 \quad < h_t/D_{n50} < 25$

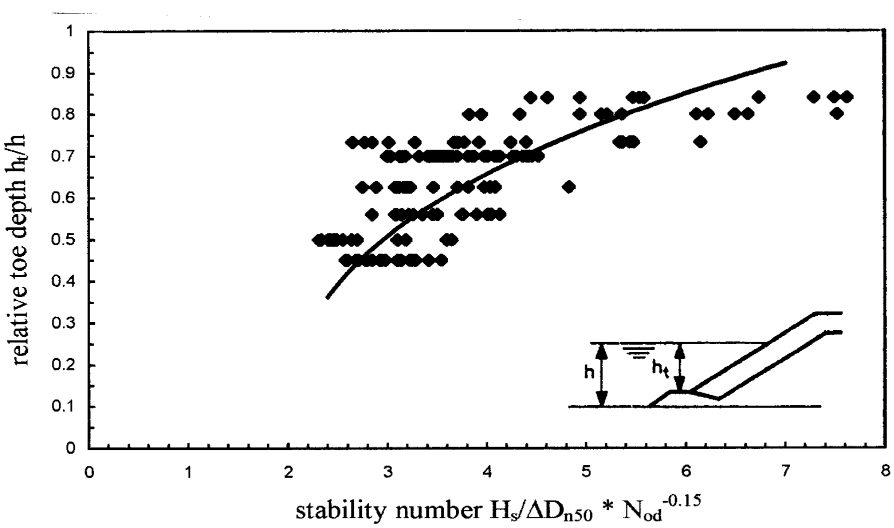

Figure 12 Toe stability as a function of relative depth h_t/h and damage level N_{od}

5.4 Crown walls

For a number of reasons the introduction of a crown wall on top of a breakwater can be considered (CUR/RWS, 1995):

- rubble mound breakwaters are often designed to sustain some damage and access across the breakwater is needed for repairs
- a crown wall with parapet may lead to a substantial reduction in the amount of stone which would otherwise be needed for a comparable conventional design (see Figure 11)
- with overtopping the crown wall may limit the width of the mound and by its shape protect the lee side slope.

There are also certain disadvantages related to a crown wall which should be taken into account in selection and design:

- the crown wall represents a rigid element in a structure which is flexible by nature. Uneven settlements may lead to great problems for the elements of the crown wall and even more to transport facilities

- increase of the parapet wall, in order to reduce the volume of stones, leads to very large wave impact forces on this wall. Such a design should be avoided.
- overtopping water becomes concentrated more into a jet and is a potential danger for the lee side armour

The design of crown walls should commence with an assessment of their stability. CUR/RWS, 1995, gives some basic rules and more detailed information can be found in Pedersen, 1996. In summary, apart from the weight of the crown wall, wave forces form the only other significant load. Other practical details can be found in CUR/RWS, 1995.

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