Chapter

Design of Berm Breakwaters

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Berm breakwaters may be a good alternative for rock armoured slopes and even concrete armoured slopes or breakwaters. Guidance on berm breakwater design, including large rock quarrying, was lacking as this type of breakwater does not belong to the conventional designs of rock and concrete armour. Some guidance by the authors became available in recent years in conference papers and that all has resulted in a book on berm breakwaters. This chapter considers the actual geometrical design of three cross-sections of berm breakwaters, depending on available rock sizes as well as on the design wave climate. Three wave climates are taken, 3 m, 5 m and 7 m. For each wave climate various maximum rock classes are considered, which in reality will depend on availability of such rock. One design has been presented for each wave climate and one is referred to the book for a number of other possibilities. For the 3 m and 5 m wave climate the designs have been compared with a conventional two-layer design. All the designs together give a good impression of what can be achieved by a proper berm breakwater design.

1. Introduction

The design of modern berm breakwaters started more or less in 1983 by Baird & Associates in Canada. The original design consisted of mass armoured berms that were reshaped to statically stable S-shaped slopes, see Figure. 1. The design was adopted in Iceland and eventually led to a development with more stable structures by utilizing available rock sizes, large rock and more gradings, see Figure 2. This more stable and only partly reshaping structure is called the

Icelandic-type berm breakwater. Real guidance on design and construction of berm breakwaters was lacking, but the new book of both authors may be seen as an improvement on this, [Van der Meer and Sigurdarson, 2016].

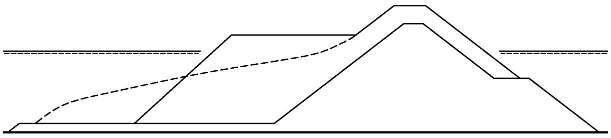


Fig. 1. Original design of modern berm breakwaters.

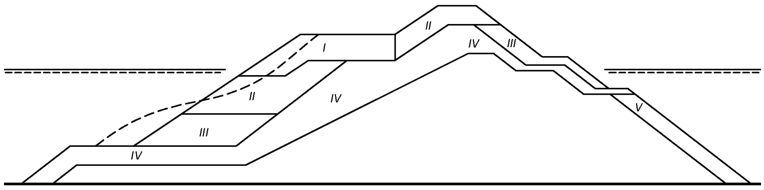


Fig. 2. Icelandic-type berm breakwater; more rock classes, less reshaping.

Aspects of this book were presented at various conferences, [Sigurdarson and Van der Meer, 2012], [Sigurdarson and Van der Meer, 2013], [Van der Meer and Sigurdarson, 2014], [Sigurdarson *et al.*, 2014] and [Sigurdarson and Van der Meer, 2015]. This chapter summarises the design formulae on recession of the berm and wave overtopping over the crest, as well as the geometrical design formulae. Then these formulae have been used to give three example designs of berm breakwaters.

Berm breakwaters can be divided into hardly reshaping (HR), partly reshaping (PR) and fully reshaping (FR), all depending on the stability number for the (100-years) design condition, H_{sD}. A berm breakwater can be designed as a mass armoured berm breakwater (MA) or an Icelandic-type berm breakwater. The classification is given in Table 1, where S_d is the damage number and Rec the expected recession of the berm.

Table 1. Classification of berm breakwaters, given for the 100-years condition.

Breakwater type	*Abbrevation*	$H_{sD}/\Delta D_{n50}$	S_d	Rec/D_{n50}
Hardly reshaping berm breakwater (Icelandic-type)	**HR-IC**	1.7-2.0	2-8	0.5-2
Partly reshaping Icelandic-type berm breakwater	**PR-IC**	2.0-2.5	10-20	1-5
Partly reshaping mass-armoured berm breakwater	**PR-MA**	2.0-2.5	10-20	1-5
Fully reshaping mass-armoured berm breakwater	**FR-MA**	2.5-3.0	--	3-10

2. Berm recession and wave overtopping

Recession of the berm, Rec, of a berm breakwater depends mainly on the stability number $H_s/\Delta D_{n50}$. Recession up to the design condition (100-years event) is given by:

$$Rec/D_{n50} = 1.6\ (H_s/\Delta D_{n50} - 1.0)^{2.5} \quad (1)$$

A smaller increase in recession can be expected during overload situations. A practical method would be:

> Use Equation 1 up to design conditions; Calculate the recession with Equation 1 for overload conditions; Determine the final recession for these overload conditions *by taking only half of the calculated increase in recession in the previous step*. (2)

Wave overtopping at a berm breakwater is given by the general formula in EurOtop [2016], with an adapted influence factor γ_{BB} for berm breakwaters:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp\left[-\left(1.5\frac{R_c}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right)^{1.3}\right] \quad (3)$$

with:

$$\gamma_{BB} = 0.68 - 4.5 s_{op} - 0.05 B/H_{sD} \qquad \text{for HR and PR} \quad (4)$$

$$\gamma_{BB} = 0.70 - 9.0\ s_{op} \qquad \text{for FR} \quad (5)$$

and B/H_{sD} is given by the design wave height. Here s_{op} is the fictitious deep water wave length, using the peak period, T_p, and the significant wave height, H_{m0}, at the toe of the structure. B = berm width.

Equations 3-5 give the *mean value approach* [EurOtop, 2016], and this method should be used to compare with measurements and it can be used in a probabilistic approach if the equations are rewritten to a reliability function, taking into account the given standard deviations. For a *design or assessment approach*, however, it is better to take a little safety into account as the equations give quite some scatter. A good way is to include one standard deviation extra into this design or assessment approach, which gives the following design formulae instead of Equation 3:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.1035 \cdot \exp\left[-\left(1.35\frac{R_c}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right)^{1.3}\right] \quad (6)$$

3. Geometrical design guidance

3.1. *Description of the cross-section*

In both Figures 1 and 2 similar geometrical parameters have to be established, where for the Icelandic-type a few more parameters are needed. Figure 3 shows the geometrical design parameters of an Icelandic-type of berm breakwater with four classes of rock. A berm breakwater has an upper slope and a lower slope, with in between a berm at a certain level, d_b, with respect to the design water level DWL, and with a certain width, B.

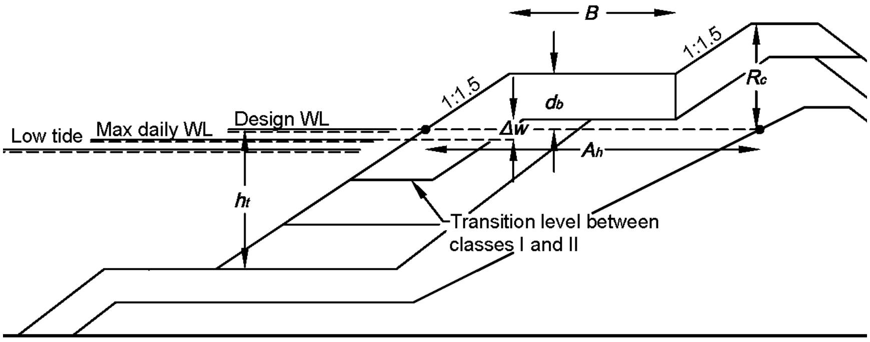


Figure 3. Principal cross-section of an Icelandic-type berm breakwater with the main geometrical design parameters.

For a first design of the cross-section the lower slope as well as the upper slope are taken as $\cot\alpha = 1.5$. Only for the mass-armoured breakwater a lower slope of $\cot\alpha = 1.2$ is taken as often these type of berm breakwaters have a steeper lower slope. After this first design the slopes may be modified, depending on actual situations.

The crest height, R_c, should be designed to a certain level, depending on allowable wave overtopping at the design water level DWL. The level of a toe berm (if any) or apron is given by the water depth, h_t, above it. Also the total volume of large rock (Classes I to III or IV for an Icelandic-type) should be sufficient, leading to a "horizontal armour width, A_h", described later in this chapter. Crest width and toe berm width (if any) are in first instance taken as 1 H_{sD}. Depending on actual predicted recession or demands from required space behind the crest, these values may be changed later on.

Tide and daily wave conditions may influence the design of the cross-section, as a safe working level has to be chosen for construction. This level is given as Δw about the daily maximum water level. In the case of an Icelandic-type this level may influence the final berm level as one may use this level as a working platform.

A main parameter is the design water level, DWL, combined with the design wave conditions. The design wave conditions and rock availability may lead to a certain classification of structure (see Table 1) to be designed. Structural (recession) and functional behaviour (wave overtopping) may be assessed for lower conditions as well as for higher or overload conditions, but the main design guidance given here on determining the cross-section is based on the 100-years wave height. The maximum water level during this design event is taken as the design water level, DWL, and may influence the choice of the berm level.

A good performance-based design, however, should also look at conditions beyond the 100-years design conditions. These overload conditions, sometimes given as a certain percentage more than the design conditions, or given as a condition for a much larger return period, should be related to the wanted performance. For overload conditions one may accept more recession on the berm and may accept larger wave overtopping, but these conditions should not lead to failure of (parts of) the structure. Even under overload conditions there should be some part of the berm left and wave overtopping may not lead to severe damage or failure of the crest and or rear of the structure. The geometrical design rules for berm size and crest height will therefore also be applied in the example designs for overload conditions and acceptable recession and overtopping damage.

3.2. *Berm width, B, as function of recession and resiliency*

Depending on the classification the expected structural behaviour is given by more or less recession: hardly, partly or fully reshaping. This has influence on the choice of berm width. It is clear that the lower the stability number is, the more stable the structure will be, with less recession. But less recession should not be the only objective of designing the berm width. The berm width should be (much) larger than expected small recession. With smaller recession there is a larger capability to cope with extremes, called the resiliency of the structure. This resiliency should play a role in designing the berm width, although it has never been formulated explicitly in design rules.

It is proposed to consider the following guidelines on resiliency, given in Equations 7-9, connecting expected recession, Rec, and required berm width, B. The resiliency is given as a percentage, $P_{\%}$, of the berm width that may erode under the design condition H_{sD}. For a fully reshaping structure the percentage should not be 100% as the percentage is given for the design conditions only.

Overload conditions larger than the 100-years condition may take more from the berm and for these type of structures one should also consider possible maintenance aspects.

Very resilient, hardly reshaping, IC HR $P_{\%}$ = 10-20% (7)
Good resiliency, partly reshaping, IC PR or MA PR $P_{\%}$ = 20-40% (8)
Minimum resiliency, fully reshaping, MA FR $P_{\%} \leq$ 70% (9)

The choice of $P_{\%}$ has to be made by designer and client, where a lower limit can be taken for more safety, but also for a more costly berm. If the wanted resiliency has been chosen, the berm width follows from:

$$B = Rec/(P_{\%}/100) \tag{10}$$

As example, if the expected recession under the design wave height is expected to be 4 m and the wanted resiliency has been chosen with $P_{\%}$ = 30% (a partly reshaping berm breakwater), then the berm width becomes B = 4/0.3 = 13.3 m.

And then there is a geometrical criterion for the berm width. As Class I rock of an Icelandic-type of berm breakwater may be quite large, the berm width is very often close to 3 to 4 D_{n50}. A rule for a minimum berm width could be:

$$B_{min} = Rec + 1\ D_{n50} \quad \text{with a minimum of at least } 3\ D_{n50} \tag{11}$$

3.3. *Crest level, R_c*

Severe wave overtopping may damage the rear side of a breakwater and if this rear is not protected well, the classical failure of a breakwater may occur, i.e. lowering of the structure to about mean sea level. Such severe wave overtopping may occur under overload situations, passing the 100-years condition. It is also for this reason that allowable wave overtopping for the design wave height should not be taken too high. If the rear is not protected by large rock, wave overtopping discharges between q = 10-30 l/s per m may easily destroy the crest of the breakwater.

Design data revealed that most berm breakwater structures have a relative freeboard of:

$$R_c/H_{sD} = 1.0 – 1.2 \tag{12}$$

Equation 12 could be used for comparison and if no allowable overtopping has been given. An easy method to check the stability of the rear side is given by Van der Meer and Veldman [1992]. It only applies for fully reshaping mass-armoured berm breakwaters and with similar rock in the berm and over the crest and rear. The method gives a stability number, including significant wave height, crest freeboard and wave steepness (using the peak period).

$$R_c/H_s * s_{op}^{1/3} = A \tag{13}$$

with A = 0.25 for start of damage; A = 0.21 for moderate damage and A = 0.17 for severe damage.

For the design wave height (100-years condition) one should stay below start of damage (say A = 0.25-0.30), where for the overload situation between start of damage and moderate damage could be taken as the maximum allowable situation (A = 0.21-0.25 in Equation 13).

The crest width of a berm breakwater may be limited to a certain number of nominal diameters of the rock that is present as cover layer on the crest. A good design choice for a minimum crest width is:

$$\text{Crest width} = 4\ D_{n50}\ \text{(crest rock)} \tag{14}$$

3.4. *Horizontal armour width, A_h*

An important design parameter in Figure 3 is the "horizontal armour width, A_h". It is the horizontal distance at design water level from the seaward slope of the armour to the transition of sorted rock class to the core. The good structural behaviour of berm breakwaters is, for a large part, due to the large capacity of dissipating wave energy in the large berm. A class of sorted rock gives large voids between the stones and this causes the dissipating capacity. For this reason the horizontal width of the armour at design water level should not become too small. It has always been an implicit but important parameter in design and development of the Icelandic-type berm breakwater in projects in Iceland and Norway.

Based on detailed analysis of a large number of constructed berm breakwaters, see [Van der Meer and Sigurdarson, 2016], the following design formula was established:

$$A_h/H_{sD} = 2\ H_{sD}/\Delta D_{n50} \tag{15}$$

The horizontal armour width A_h depends linearly on the stability number. Hardly reshaping structures would give A_h = 3.4 to 4.0 H_{sD}, partly reshaping structures to A_h = 4.0 to 5.0 H_{sD} and fully reshaping berm breakwaters to A_h = 5.0 to 6.0 H_{sD}, taking into account the classification given in Table 1.

3.5. *Proposal for optimised mass armoured berm breakwater*

In the original design a berm would consist of, for example, a wide grading of 1-9 t rock. It is relatively easy to divide this class into two gradings of 1-4 t and 4-9 t. Then the smallest rock is used for the lower part of the berm and the larger rock at the upper part, see Figure 4.

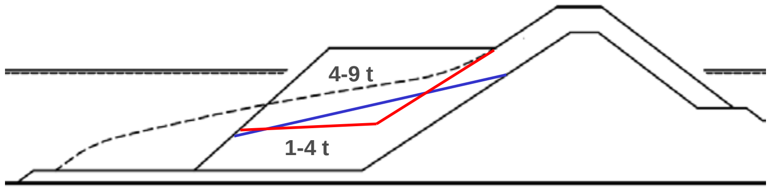


Figure 4. Proposal for a new “mass-armoured” berm breakwater: two more narrow graded rock classes instead of one wide grading.

It is the designer’s decision to choose between the configuration with the kinked or with the straight line. Both options must be seen as comparable with respect to reshaping. With this modification the design moves a little towards an Icelandic-type of berm breakwater, but still has no rock layers, just two volumes of rock: a lower part and an upper part. Construction costs of a design like Figure 4 will be comparable to the original design with one wide grading and exactly the same rock is used. But the design of Figure 4 is more stable, giving less recession, as the stability number increased due to a larger D_{n50} for the 4-9 t rock than for the 1-9 t rock.

If easy construction is wanted or required, a mass-armoured berm breakwater may be an option. But it is strongly proposed to decrease recession and increase resiliency, without increasing costs, to use two classes of rock as given as an example in Figure 4.

3.6. *Berm level, d_b*

The berm width B as defined in Figure 3 is given by wanted resiliency, described in Section 3.2 and Equations 7-10. The berm level d_b often depends on two aspects. The first and easiest is that the berm level should be at least 0.6 H_{sD} above design water level. Such a high berm increases stability, as it is able to dissipate more energy from the up-rushing waves. Of course the berm may be designed at a lower level, but one should expect then more recession.

$$d_b \geq 0.6\, H_{sD} \tag{16}$$

The other aspect on berm level might be the level for construction work. Often the top level of the Class II, III and IV, which gives the transition to Class I rock at the berm determines with 2 D_{n50} of Class I rock on top the final level of the berm, see Figure 3. This level is recommended as a working level for the excavators placing the Class I rock.

The minimum working level depends on the local situation with tides and daily waves and assessment of such a level would not differ much from procedures given in the Rock Manual [2007]. For example one could take mean high water spring (MHWS) with extra safety for daily waves. This extra safety, given as Δw in Figure 3, depends on the actual wave climate expected during construction to ensure safe working conditions. For low wave activity Δw has often been chosen between 0.5 m and 1 m and for more moderate wave activity between 1 m and 1.5 m.

In conclusion, the lower level of the two diameters thick Class I is based on Equation 16 or on local tide levels (for example mean high water spring) and a safe level for daily wave conditions. Then two diameters D_{n50} of the Class I rock have to be added to come to the final berm level.

3.7. *Apron*

Figures 1 and 3 show that the large rock of a fully reshaping berm breakwater is laid on a foundation layer with an apron in front of the berm. Such a layer is really required if the structure is founded on sand or other movable material (and not on rock). This is a common rule for conventional breakwaters, where a geometrical tight, or sometimes geometrically open, filter layer has to be designed, or where a geotextile is used. Also berm breakwaters need such a foundation layer, where sand cannot escape from beneath the structure. For a reshaping berm breakwater large rock from the berm will fall down the slope and form a longer S-shaped profile, as given in Figures 1 and 2. Therefore an apron or foundation layer has to be designed in front of the berm to provide a foundation for the reshaping berm.

3.8. *Transition from Class I to Class II rock*

If the transition from Class I to Class II, see Figure 3, is at a too high level, waves may attack and damage the smaller rock of Class II, that may then lead to larger recession of Class I. Based on earlier designed structures it is proposed to use 0.4 H_{sD} below DWL as a limit for transition for Icelandic-type berm breakwaters.

Using this 0.4 H_{sD}-limit should also consider lower water levels with adjusted design wave heights. Structural behaviour (recession) and wave overtopping are often considered at the design water level, DWL, which is the highest water level that can occur under design conditions (100-years event). But it is also possible that the design wave height is present for some time at lower levels, for example during low tide, but including surge. For relatively deep situations the wave height will remain more or less the same for lower water levels, but in depth-limited situations the wave height will decrease with lower water levels considered. The 0.4 H_{sD}-limit for the transition from Class I to Class II rock has to be measured from the lowest water level that can occur with more or less the design conditions (within 90% of H_{sD}).

For a mass-armoured berm breakwater, with two classes of rock as proposed in Figure 4, a good transition might be 0.6 H_s below the considered water level.

3.9. *Possible toe berm*

Figure 3 also shows a foundation level for the large rock classes of an Icelandic-type berm breakwater, well above the level of the foreshore. In the graph this is given as Class IV on a horizontal layer. For relatively deep water as well as for depth-limited conditions it is favourable for the stability of the berm if this foundation level is as high as possible. In principle one could consider this structure as a toe berm structure for a conventional breakwater and use toe stability formulae to assess the stability. One such formula is given amongst others in the Rock Manual [2007, Equation 5.188].

4. Design method and examples of design

4.1. *Geometrical design method in a spreadsheet*

The design formulae in Section 2, together with the geometrical design rules given in Section 3, lead to a quite straight forward method to design berm breakwaters. The behaviour of the berm breakwater under design conditions depends very much on the type of berm breakwater that is considered: hardly, partly or fully reshaping.

Then there is a division in Icelandic-type berm breakwaters, with often 3-4 large rock classes in the berm, and the mass-armoured berm breakwaters. These have originally only one wide rock class for the berm, but as stated in Section 3 and Figure 4 it is strongly advised to divide the berm in two narrower rock classes. That means that such a mass-armoured berm breakwater comes close to an Icelandic-type berm breakwater with only two classes.

The geometrical design means that the parameters in Figure 3 have to be established. As most of them have been given in formulae it is quite easy to make a spreadsheet and calculate the parameters automatically. Then design choices can be made and a conceptual cross-section can be drawn, also in the same spreadsheet. The outcome may need further modification and will finally result in the design drawing of the breakwater cross-section.

The developed spreadsheet (available at wwww.vdm-c.nl) will be described here in depth and will then be used for a number of examples, without further explanation of the spreadsheet. The first part gives the general design conditions as given in Table 2. A grey cell means that it is a requirement to give this item a (design) value. Design wave heights have to be given, the H_{sD} for the 100-years return period and the overload condition, which could be about 20% higher, or based on a much longer return period. In this way a performance-based design is achieved. Design water levels have to be given, as well as the wave height associated with a low water level. Finally, allowable overtopping has to be given for both design conditions, as well as mass densities of rock and water.

Table 2. General conditions in the design spread sheet. Grey cells are inputs required by the user.

General conditions			*Remarks*
Design wave height H_{sD}	5	m	100-years return period
Peak period T_p	10.3	s	
Overload H_s	6	m	About 1.2 H_{sD}
Design water level DWL	2	m CD	
Lowest water level with H_{sD}	1	m CD	
Lowest storm level	0	m CD	
H_s at lowest storm level	4.5	m	
Mean High Water Spring	1	m CD	
Bottom level of foreshore at toe of structure	-10	m CD	
Allowable overtopping q for H_{sD}	1	l/s per m	
Allowable overtopping q for overload	10	l/s per m	
Mass density water	1025	kg/m^3	
Mass density rock	2700	kg/m^3	

The next item is the specification of rock classes that are foreseen for the berm of the breakwater, see Table 3. After first calculations or a changed quarry yield prediction or output, it might be possible that this input will change and lead to a slightly adjusted cross-section. Two rock classes should be specified for a mass-armoured berm breakwater, leaving a blank in the two lowest cells, and three classes for an Icelandic-type berm breakwater. One may give the 0% and 100% class limits, as often done for the Icelandic-type designs, but also the limits as given for the standard heavy gradings in the Rock Manual [2007]. These are NLL < 10% and NUL > 70%, where NLL and NUL describe the class boundaries.

Table 3. Rock classes to be specified in the design spreadsheet

Choice of rock classes		
Rock Class I: minimum mass (0-10%)	5	t
Rock Class I: maximum mass (70-100%)	10	t
Rock Class II: minimum mass (0-10%)	1	t
Rock Class II: maximum mass (70-100%)	5	t
Rock Class III: M_{min} (leave open for MA)	0.3	t
Rock Class III: M_{max} (leave open for MA)	1	t

Table 4. Outcome of main parameters in the design spreadsheet

Outcome of main parameters			*Remarks*
Wave steepness s_{op}	0.030	-	
Relative mass density Δ	1.63	-	
Median mass Class I M_{50}	7.5	t	Middle of the class limits
Nominal diameter Class I D_{n50}	1.41	m	
Stability number $H_{sD}/\Delta D_{n50}$	2.18	-	
Type of berm breakwater	Partly reshaping		Table 1
Number of rock classes for berm	3		
Basic recession for H_{sD} (no adaptation)	3.38	m	Equation 1
Recession for overload (no adaptation)	6.08	m	Equation 2
Nominal diameter Class II, D_{n50}	1.04	m	Middle of the class limits
Nominal diameter Class III, D_{n50}	0.62	m	Middle of the class limits

All data specified in Tables 2 and 3 lead to the first outcome, mainly on related parameters and the type of berm breakwater. An overall view is given in Table 4. The wave steepness has been calculated, as well as M_{50}'s or D_{n50}'s, where these are based on the middle of the class limits. For example the Class I 5-10 t armourstone has M_{50} = 7.5 t, with an associated D_{n50} = 1.41 m. This leads to a stability number $H_s/\Delta D_{n50}$ for design conditions and consequently to the characteristics of the berm breakwater: hardly, partly or fully reshaping (based on Table 1).

The basic recession has been calculated, based on Equation 1 for the design condition and on Equation 2 for the overload condition. This is a basic recession as not all possible influences on recession, positive as well as negative, as described in Van der Meer and Sigurdarson [2016] have yet been taken into account.

The berm width of a berm breakwater is closely related to the expected recession, but even more on the wanted resiliency, see Table 5. Small expected recession gives a large resiliency and vice versa. It is a designer's explicit choice how resilient he or she wants to design the berm breakwater. There is a relationship between the expected recession and resiliency, as in Equations 7-9, but the resiliency is given as a range. This is 10-20% for a hardly reshaping berm breakwater, 20-40% for a partly reshaping and less than 70% for a fully reshaping berm breakwater. The percentage is the part of the berm that might be eroded by the design wave height H_{sD}. A measure for the berm width is then given by Equation 10, where Equation 11 gives a minimum berm width based on a required minimum number of stones (geometry).

Table 5. Berm width and level, based on resiliency, as in the design spreadsheet.

Resiliency, berm width and level			*Remarks*
Wanted resiliency	30	%	Equations 7-9
Resulting Berm width B from resiliency	11.26	m	Equation 10
Minimum berm width B_{min} from geometry	4.78	m	Equation 11
Berm level 0.6 H_{sD}	5	m CD	Equation 16
Δw for waves during construction	1	m	Safety measure
MHWS plus Δw = working level	2	m CD	
Minimum berm level from construction	4.81	m CD	Above level + 2 D_{n50} Class I
Design choice of berm width	12.00	m	
Design choice of berm level	5.00	m CD	

The berm level is a free choice, but a berm level above a certain value ($\geq$ 0.6 H_{sD}, see Equation 16) increases stability and reduces recession. Therefore this level is given to base a final design choice on. In Table 5 this comes to a level of +5 m CD. A minimum berm width follows from the construction procedure if one wants to construct Class I rock by working on the top level of the Class II rock and take into account some safety with regard to frequent waves. This safety is given by Δw and must be related to expected frequent wave conditions during construction. This safety is put on top of the level for MHWS. This all results in a minimum berm level, in the example in Table 5 this is +4.81 m CD.

Based on the outcome in Table 5, the designer has to make two design choices: the berm width and the berm level. The final berm width may also depend on the application of positive and negative influences, see Van der Meer and Sigurdarson [2106] and is chosen here as 12.0 m. The final berm level can be based on the two calculated levels and here the highest level is chosen to have the positive influence of a high berm level on recession: 5.0 m CD.

The horizontal armour width, A_h, determines more or less the volume of large rock in the berm, see Figure 3. The minimum horizontal armour width is given by Equation 15 and has been calculated in Table 6. The designer's choice in the table is quite close to this value.

Table 6. The horizontal armour with A_h in the design spreadsheet.

Required horizontal armour width A_h	21.8	m	Equation 15
Design choice of A_h	22.0	m	

The transition from Class I to Class II rock at the seaward slope should not be too high, as the Class II rock will result in more recession to the structure if this rock is attacked by waves. Section 3.8 gives some guidance. One could consider the lowest possible water level with the design wave height H_{sD}, as well as a lower water level with a little smaller wave height. For an Icelandic-type berm breakwater with at least three rock classes in the berm, the highest level of transition is 0.4 H_s below the water level considered and for a mass-armoured berm breakwater with two classes this may be 0.6 H_s. Table 7 gives the calculation and the designer has to make a final choice on the level.

Table 7. Transition level from Class I to Class II as in the design spreadsheet

Minimum transition level to Class II			*Remarks*
For H_{sD} at lowest level	-1.0	m CD	Section 3.8
For lowest level with according H_s	-1.8	m CD	Section 3.8
Design choice of transition for IC (3 rock classes)	-1.8	m CD	
Transition lower class for MA (2 rock classes)	-2.7	m CD	Section 3.8

The crest level design of a berm breakwater depends very much on what wave overtopping should be allowed. Based on the analysis of tested berm breakwaters [Van der Meer and Sigurdarson, 2016] most berm breakwaters had a crest level 1.2 to 1.4 H_{sD} above design water level (Equation 12). These limits have first been calculated in Table 8. But if an allowable overtopping discharge has been given, the required influence factor γ_{BB} can be calculated and subsequently the required crest level by the design equation on wave overtopping, Equation 6, together with Equations 4 and 5. Perpendicular wave attack has been assumed.

Table 8. Crest level calculations as in the design spreadsheet

Crest level (γ_β = 1)			*Remarks*
If no overtopping criteria, $R_{c\,min}$	8.0	m CD	Equation 12
If no overtopping criteria, $R_{c\,max}$	9.0	m CD	Equation 12
For given allowable overtopping, q, γ_{BB}	0.42		Equations 4 and 5
Required crest level for design conditions	9.92	m CD	Equation 6
Required crest level for overload	9.64	m CD	Equation 6
Design choice of crest level	10.00	m CD	

Equation 6 gives a *design approach* with some safety on the outcome (about one standard deviation more). This safe design is proposed as prediction of wave overtopping is quite uncertain. The influence factor γ_{BB} is different for partly and hardly reshaping berm breakwaters (Equation 4) and fully reshaping berm breakwaters (Equation 5). The official transition between the types is at $H_s/\Delta D_{n50} = 2.5$ and by using the two equations, gives a discontinuity at this transition. In order to avoid a discontinuity, the crest levels in Table 8 have been calculated with Equation 4 if $H_s/\Delta D_{n50} < 2.3$ and with Equation 5 if $H_s/\Delta D_{n50} > 2.6$. For $2.3 \leq H_s/\Delta D_{n50} \leq 2.6$ both equations have been used and the crest levels have been interpolated. At the lowest row in Table 8, the designer has to make a choice on the crest level.

Figure 3 shows a foundation level for the large rock classes of an Icelandic-type berm breakwater, well above the level of the seabed. In the graph this is given as Class IV on a horizontal layer. For relatively deep water as well as for depth-limited conditions it is favourable for the stability of the berm if this foundation level is as high as possible. In principle one could consider this structure as a toe berm structure for a conventional breakwater and use the toe stability formula to assess the stability, Rock Manual [2007, Equation 5.188]. In Table 9 this equation has been used and the validity ranges are checked.

A check should be made whether the level of the designed toe can indeed be constructed, see also Figure 3. It is assumed that the core extends seaward with at least a thickness of 1.5 m. Then on top of this core the rock layer of the toe berm will be constructed, which has a thickness of at least 2 D_{n50}. The lowest level of the toe berm is then 1.5 m + 2 D_{n50} above the foreshore. These kinds of calculations have been performed in Table 9, for the design wave height as well as for the overload condition, where the designer has to give the allowable damage level for the toe for both conditions.

Finally, the designer has to make a choice on whether a toe is feasible and what the level should be. Another choice to be made is whether the slope angle of the core should be 1:1.5 or a little gentler in order to save some of the large material in the berm.

Table 9. Check on possibility of a toe berm as in the design spreadsheet.

Check possibility of toe berm at level h_t		*Remarks*
Lowest possible toe level (two layers)	-7.26	m CD
Design conditions		
Allowable damage level for H_{sD}, N_{od}	2	-
Highest level of toe for H_{sD} with chosen N_{od}	-6.78	m CD
Check validity range h_t/D_{n50}	12.5	ok
Check validity range h_t/h	0.71	ok
Overload conditions		
Allowable damage level for overload, N_{od}	4	-
Highest level of toe for overload with chosen N_{od}	-7.19	m CD
Check validity range h_t/D_{n50}	13.2	ok
Check validity range h_t/h	0.74	ok
Design choice of toe berm level (0 if no berm)	-7.2	m CD
Design choice cotα core below A_h	2	-

The final outcome of the design spreadsheet is a draft cross-section with a summary of the design choices. For the calculations and choices made above, this information is given in Figure 5. It shows the predicted recession as well as the horizontal armour width A_h and a division between the three classes.

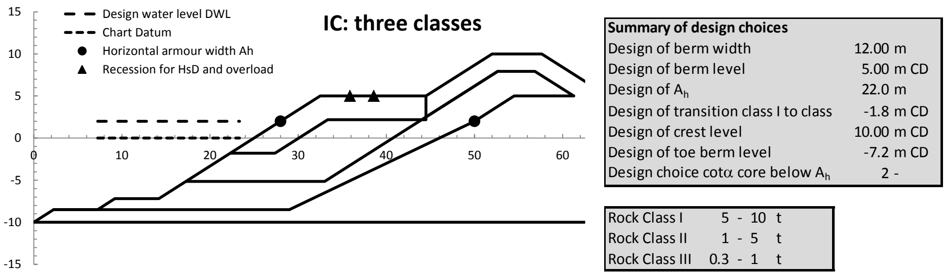


Figure 5. Draft cross-section and summary of design choices, based on the design spreadsheet.

The Class I rock in Figure 5, as given as output of the spreadsheet, has always a thickness of $2D_{n50}$. In the final design stage it may change, for example if the recession is quite large and could reach the underlying Class II rock. In that case it is possible to extend the seaward layer thickness to $3D_{n50}$ or more This will be the case when $H_s/\Delta D_{n50}$ will be close to or larger than 2.2.

4.2. *Design wave climate and other conditions for examples*

The geometrical design method described in Section 4.1 can be used to describe a number of examples. Examples of hardly, partly and fully reshaping will be given, as well as Icelandic-type berm breakwaters (at least three rock classes in the berm) and mass-armoured berm breakwaters (two classes of rock). There are differences between rock coming from a dedicated quarry and standard gradings as described in the Rock Manual [2007], see Van der Meer and Sigurdarson [2016]. The maximum standard grading is 10-15 t, whereas it is often possible to get much larger rock, even classes like 20-35 t, from dedicated quarries.

The type of berm breakwater with its expected behaviour has been described in Table 1 and is mainly depending on the stability number. The kind of berm breakwater that can be designed depends further on the available (maximum) rock class and of course on the design wave height, H_{sD}. In order to come to useful examples some elementary data have been gathered in Table 10.

Table 10. Stability numbers for chosen rock classes and design waves. Grey cells give the examples also described in Van der Meer and Sigurdarson [2016], black ones only this chapter.

		Stability number $H_{sD}/\Delta D_{n50}$		
Dedicated quarry	**M_{50} (t)**	**H_{sD} = 3 m**	**H_{sD} = 5 m**	**H_{sD} = 7 m**
Class 20-35 t	25.0	0.87	1.46	**2.04**
Class 10-20 t	15.0	1.04	**1.73**	**2.42**
Class 4-10 t	7.0	1.34	**2.23**	3.12
Class 1-4 t	2.5	**1.88**	3.14	4.39
Class 0.2-1 t	underlayer			
Class 2-6 t	4.0	1.61	**2.68**	3.76
Class 0.5-2 t	1.2	**2.41**	4.01	5.61
Standard gradings				
Class 10-15 t	12.5	1.10	**1.84**	**2.57**
Class 6-10 t	8.0	1.28	**2.13**	**2.98**
Class 3-6 t	4.5	1.55	**2.58**	3.61
Class 1-3 t	2.0	**2.03**	3.38	4.73
Class 0.3-1 t	underlayer			

First possible rock classes have been given for a dedicated quarry, based on earlier experience and then the heavy gradings as given in the Rock Manual [2007]. Then three wave climates have been considered, a very moderate wave climate with H_{sD} = 3 m, a quite normal wave climate with H_{sD} = 5 m and an extreme wave climate with H_{sD} = 7 m. Table 10 gives the calculated stability numbers for each chosen rock class (except for underlayers) and for each wave climate, considering a mass density of the rock of 2700 kg/m^3 and of sea water 1025 kg/m^3. Actual stability numbers may change with other mass densities.

Stability numbers smaller than $H_{sD}/\Delta D_{n50} < 1.7$ mean that a hardly reshaping berm breakwater can (easily) be made. That is the case for most of the heavy rock classes and a design wave height of 3 m. For a wave height of 5 m, such small stability numbers can only be reached with a Class I rock of 20-35 t (giving $H_{sD}/\Delta D_{n50}$ = 1.46). This low stability number cannot be reached for a design wave height of 7 m. Stability numbers $H_{sD}/\Delta D_{n50} > 3.0$ mean that the structure would become dynamically stable and this is not acceptable for a berm breakwater. Mainly the smaller gradings and for design wave heights of 5 m and more show these large stability numbers.

The most interesting cases in Table 10 are stability numbers between $H_{sD}/\Delta D_{n50}$ 1.7 – 3.0. They give the area of the design of berm breakwaters. These numbers are bold figures in the table. But some stability numbers for the same design wave height are quite similar, mainly because some of the gradings for a dedicated quarry are quite similar to the heavy standard gradings in the Rock Manual [2007]. For this reason examples have been chosen from Table 10 in such a way that, if possible, all three types (hardly, partly and fully reshaping) are present for each design wave height and they are distributed over the rock classes for dedicated quarries as well as the heavy standard gradings. The cells with a gray colour are the examples that all have been be described in Van der Meer and Sigurdarson [2016].

For a design wave height of only 3 m, it is not necessary to design a fully reshaping mass-armoured berm breakwater. Already with a Class I of 0.5 – 2 t it is possible to design a partly reshaping berm breakwater. In total eight examples have been chosen from Table 10. For sake of space only 3 examples have been described in the following sections, one for each design wave height, given in black in Table 10.

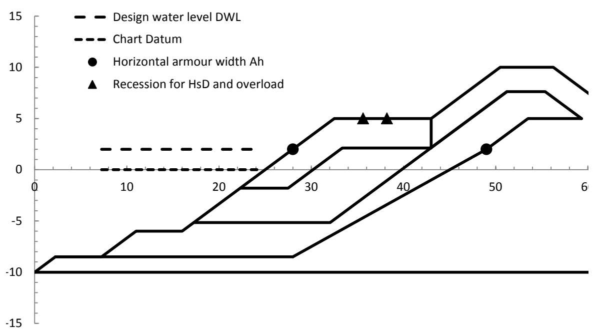
4.3. *Design wave height of 5 m*

4.3.1. *Berm breakwater PR IC standard gradings, Class I 6-10 t*

The design condition is a design wave height of H_{sD} = 5.0 m with a wave steepness of s_{op} = 0.03, giving T_p = 10.3 s. The overload condition is considered to be 20% higher than the design condition, giving H_s = 6.0 m at the design water level DWL. Tides range between 0 m CD to + 1 m CD (1 m tidal range). The design water level (100-years condition) = surge + maximum tide = +2 m CD. Daily waves are quite moderate and a safety margin of Δw = 1 m above MHWS will be enough for construction. A more or less flat foreshore is present and at the toe of the structure the bottom is present at -10 m CD. All input and calculated values, including design choices, are given in the spreadsheet in Appendix D of [Van der Meer and Sigurdarson, 2016].

The stability number is calculated as $H_s/\Delta D_{n50}$ = 2.13, for a Class I of 6-10 t, which indeed gives a partly reshaping berm breakwater. For this Class I from standard gradings, one can choose connecting classes also from standard gradings, as Class II = 3-6 t and Class III = 1-3 t, all given in Table 10. The three classes are given in Figure 6.

The wanted resiliency is taken at 30% reshaping, which is the middle of the proposed range. The allowable overtopping q = 1 l/s per m for the 100-years event and q = 10 l/s per m for the overload. Finally, the mass density of seawater is ρ_w = 1025 kg/m^3 and of rock ρ_r = 2700 kg/m^3. Figure 6 shows the outcome of the calculations.



Summary of design choices		
Design of berm width	10.50	m
Design of berm level	5.00	m CD
Design of A_h	21.0	m
Design of transition class I to class I	-1.8	m CD
Design of crest level	10.00	m CD
Design of toe berm level	-6	m CD
Design choice cotα core below A_h	2	-

Rock Class	6 - 10	t
Rock Class	3 - 6	t
Rock Class	1 - 3	t

Figure 6. Calculated cross-section for H_{sD} = 5 m and Class I = 6-10 t.

The Class III rock of 1-3 t is still quite large compared to the wave conditions and this rock can be used to make a higher toe berm. The calculations (Appendix D – Van der Meer and Sigurdarson [2016]) give a highest toe level of

-5.16 m CD and a level of -6 m CD has been chosen. The transition of Class I to Class II rock on the seaward side was chosen at -1.8 m CD.

The expected reshaping in Figure 6 is given by the triangles, the left one by the design conditions, the right one by overload conditions. The erosion profile should start in these points and will show a S-shaped curve, which crosses the original lower slope just below the water level, see for example Figures 1 or 2. The triangles in Figure 6 suggest that the erosion profile might also take a part of the Class II rock below, certainly for the overload conditions. For this reason the final design, as given in Figure 7, shows a seaward layer thickness of three stones, i.e. 4.3 m. Also the Class II layer underneath this layer has a thickness of three stones: 3.6 m.

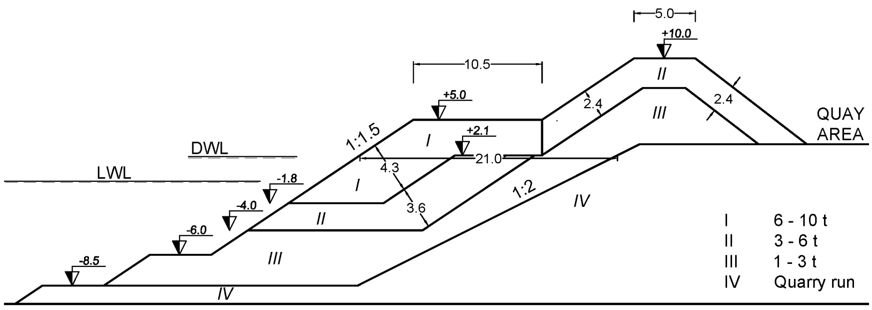


Figure 7. Partly Reshaping Icelandic-type berm breakwater cross-section designed for H_{sD} = 5.0 m, Class I 4-10 t, q_{100y} = 1 l/s per m.

4.3.2. *Conventional rock armour design 6-10 t*

With a design wave height of 5 m, as in the previous section, and a fairly large rock grading like 6-10 t, it is also possible to design a conventional rock armour protection. Such a design can then be compared with the partly reshaping Icelandic-type berm breakwater that is described in Section 4.3.1.

The design conditions are given with a peak period of T_p = 10.3 s. Assuming a relationship of $T_p = 1.2T_m$, gives a mean period of T_m = 8.6 s. The peak of the storm is assumed to give 3000 waves, which is a little over 7 hours. The Van der Meer Equations [Van der Meer, 1988] can be used to calculate the damage level, S_d, for several wave conditions. Figure 8 gives the damage curves for three mean wave periods, as calculated by Breakwat [commercial software from Deltares].

The rock slope that was chosen was 1:2.5. A steeper slope would probably give too much damage. A gentler slope, like 1:3, would give less damage. Figure 8 also shows the design condition (100-years) and the overload condition. The design condition gives S_d = 3.4 and the overload condition gives S_d =6.8.

For a slope of 1:2.5 an allowable damage for a 100-years condition would be between S_d = 2-4. For an overload condition the damage should not exceed S_d = 10. Both conditions are met, which means that a slope 1:2.5 with a 6-10 t armour layer would be able to withstand the given wave conditions.

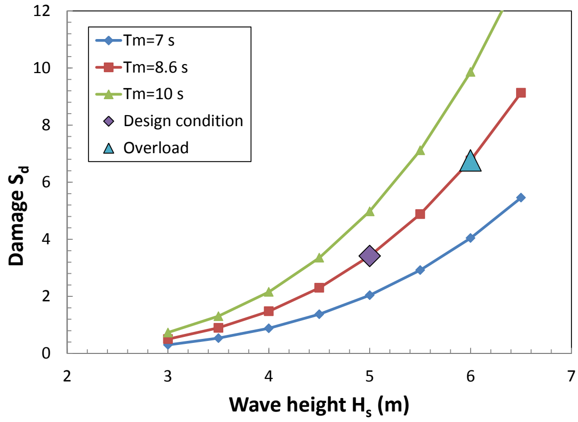


Figure 8. Damage curves for a conventional rock slope with an armour layer of 6-10 t; cotα = 2.5; P = 0.4; N = 3000

Figure 8 also shows the influence of the wave period. This influence is insignificant for berm recession of a berm breakwater, but is significant for a conventional armour layer of rock. A smaller wave period than the design period will give clearly less damage. But the damage increases quite rapidly if the mean period increases from T_m = 8.6 s to 10 s. The damage increases to S_d = 5.0 for the design condition and S_d =9.9 for the overload condition. For a good design of this conventional structure, one should look at the range of wave periods that will be possible. If indeed T_m = 10 s would be possible for design conditions, one should modify the slope angle to 1:3 in order to make the structure more stable. In this example a slope of 1:2.5 is taken for design.

The underlayer is normally 1/10th to 1/15th of the armour mass. This leads to a standard grading of 300-1000 kg. The rock armour layer has a nominal diameter of D_{n50} = 1.44 m, which gives a layer thickness of 2.9 m. The underlayer has a nominal diameter of D_{n50} = 0.62 m, which gives consequently a layer thickness of 1.25 m.

The required crest height can be calculated with the formulae in EurOtop [2016]. A conventional rock armour on an underlayer has a roughness factor of γ_f = 0.40. For the design conditions with H_s = 5.0 m and an allowable overtopping discharge of 1 l/s per m, a crest freeboard is required of 7.45 m. With a design water level of +2 m CD, the required crest height for this condition becomes 9.45 m CD. With the overload condition with H_s =6.0 m and an allowable overtopping discharge of 10 l/s per m, the required crest freeboard becomes R_c = 7.2 m. This is lower than the 7.45 m for the design condition, which means that the crest height can be determined at 9.5 m CD.

The cross-section is given in Figure 9 and can be compared with the partly reshaping berm breakwater in Figure 7. Both structures are fit for purpose. A difference might be that the resiliency of the berm breakwater is larger than for the conventional structure, as after the overload condition still half of the berm is left, where the armour layer of the conventional structure will be close to "underlayer visible". But both structures can cope with such an overload condition.

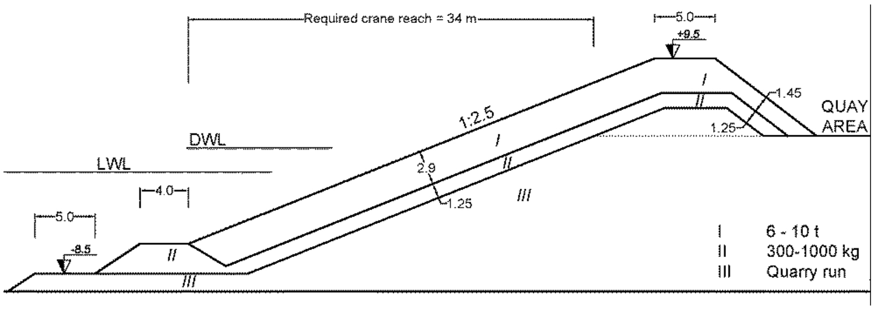


Figure 9. Conventional rock armoured structure with 6-10 t on a slope of 1:2.5. Design for H_{sD} = 5 m and q_{100y} = 1 l/s per m.

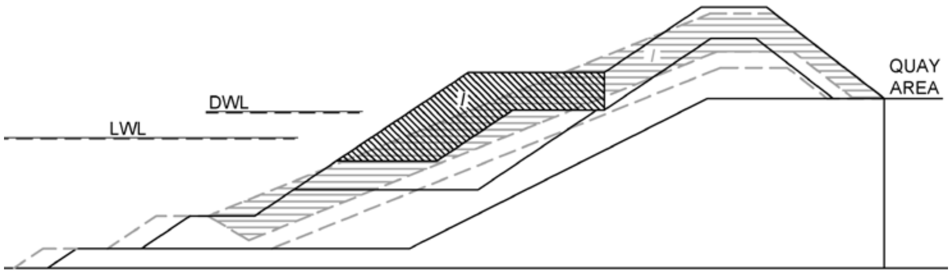


Figure 10. Direct comparison of cross-sections of the conventional breakwater and the berm breakwater

Figure 10 shows direct comparisons of the two cross-sections, where the conventional structure has been given in grey and the berm breakwater in black. The volume of large rock 6-10 t in the conventional design is more than twice of that in the berm breakwater (Class I). Of course the total volume of rock larger than 1 t is larger for the berm breakwater, but the use of the largest rock class is much smaller. In case the largest rock class is not easy to produce, the berm breakwater design gives an advantage. The total volumes of the cross-sections of both designs are quite similar, both around 460-480 m^3 per m length.

All rock on the berm breakwater of Figure 7 can be placed by excavator. This will be much more difficult for the conventional design, as the largest rock near the toe need a crane reach of about 34 m or placement from marine plant.

4.3.3. *Overall conclusions and comparison for a design wave height of 5 m*

Van der Meer and Sigurdarson [2016] give four designs of rock structures that can cope with a 100-years design wave height of 5 m. In this paper only two examples are given: a partly reshaping Icelandic-type berm breakwater with Class I of 6-10 t rock and a conventional design with a similar armour layer. The other two examples, not described here, are a hardly reshaping Icelandic-type berm breakwater with a Class I rock of 10-20 t and a fully reshaping mass armoured berm breakwater with a Class I rock of 3-6 t.

They are all fit for purpose. In all cases the crest level is around 10 m CD, allowing about 1 l/s per m wave overtopping. The main difference is the rock size of Class I on the seaward side and the volume of this largest rock class. A Class I of 10-20 t is of course a very large rock class, but the volume needed with respect to the total volume is quite limited. The largest volume of rock for the breakwater is found for the fully reshaping berm breakwater. But now the largest rock class is only 3-6 t. The difference between the berm breakwater designs is the resiliency. The smallest stability number, or largest rock size for Class I, gives the best resiliency. Even after overload conditions there is a large remaining capacity for severe wave action if the structure is only hardly reshaping. From that point of view one should always try to design for the largest rock class that can be made available.

The conventional structure in Figure 9 has a long 1:2.5 slope with 6-10 t rock. The armour layer cannot be constructed by excavator, but needs a crane with a long reach. Total rock usage is comparable with the berm breakwater with Class I rock of 6-10 t (Figure 7), but the need for the large 6-10 t rock is more than twice for the conventional design. This conventional design is more vulnerable for (longer) wave periods, which is not the case for a berm breakwater.

4.4. *Design wave height of 3 m*

4.4.1. *Berm breakwater HR IC dedicated quarry, Class I 1-4 t*

The design condition is a design wave height of H_{sD} = 3.0 m with a fairly low wave steepness of s_{op} = 0.02, giving T_p = 9.8 s. The overload condition is 20% higher than the design condition, giving H_s = 3.5 m at the design water level DWL. Tides range between 0 m CD to + 1 m CD (1 m tidal range). The design water level (100-years condition) is at the same level as the maximum tide = +1 m CD. Daily waves are quite moderate and a safety margin of Δw = 1 m above MHWS will be enough for construction. A more or less flat foreshore is present and at the toe of the structure the seabed is present at -9 m CD. All input and calculated values, including design choices, are given in the spreadsheet in Appendix D in [Van der Meer and Sigurdarson, 2016].

The allowable overtopping q = 5 l/s per m for the 100-years event and q = 20 l/s per m for the overload. Finally, the mass density seawater is ρ_w = 1025 kg/m^3 and of rock ρ_r = 2600 kg/m^3.

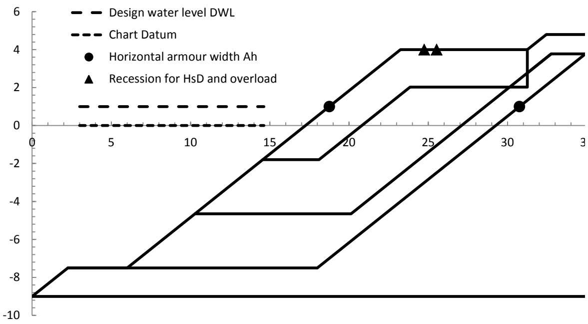
Note that design conditions described above, the allowable overtopping conditions, as well as the mass densities of sea water and rock may differ from the conditions assumed in Section 4.3. On one hand this makes direct comparison between the solutions in that section not straight forward (within the section they are comparable), but on the other hand it shows reality, where these conditions vary.

The wanted resiliency is taken at 20% reshaping, which is the upper value of the proposed range. The stability number is calculated as $H_s/\Delta D_{n50}$ = 1.98, for a class I of 1-4 t, which indeed gives a hardly reshaping berm breakwater. For this class I from a dedicated quarry, one can choose Class II = 0.2-1 t, both classes are given in Table 10. Only two rock classes are needed for this mild wave climate and the calculated cross-section is given in Figure 11.

The “standard” cross-section from the spreadsheet gives a berm breakwater with three rock classes, as in Figure 11. Actually, one should only look at the Class I layer, as the underlying material in the berm will all be Class II 0.2-1 t rock. A high toe berm has no function for a hardly reshaping berm breakwater and therefore a high toe berm has not been designed.

The berm level is higher than 0.6H_{sD}, as the berm level follows from the construction issue that it should be constructed from the underlying Class II rock on a safe working level. But as the Class I rock is not heavy and distances to reach not very far, it may also be possible to construct the Class I rock with an excavator from the core. In that case the berm level can be lowered to roughly

+2.8 m CD, instead of +4.0 m CD as in the final design given in Figure 12. Another option is to raise the crest level a little and have less wave overtopping at marginal costs.



Summary of design choices	
Design of berm width	8.00 m
Design of berm level	4.00 m CD
Design of A_h	12.0 m
Design of transition class I to class I	-1.8 m CD
Design of crest level	4.80 m CD
Design of toe berm level	0.00 m CD
Design choice cotα core below A_h	1.50 -

Rock Class	1 - 4	t
Rock Class	0.2 - 1	t

Figure 11. Calculated cross-section for H_{sD} = 3 m and Class I = 1-4 t.

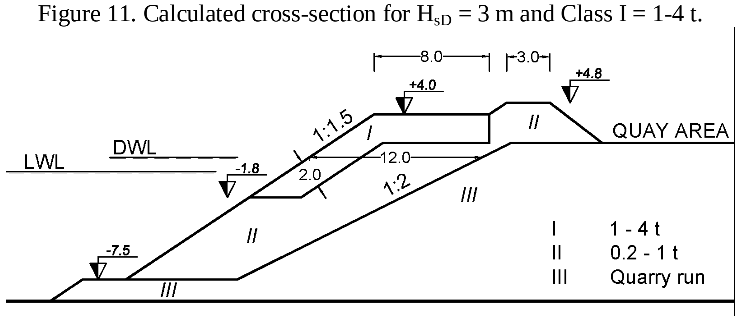


Figure 12. Hardly Reshaping Icelandic-type berm breakwater cross-section designed for H_{sD} = 3.0 m, Class I 1-4 t, q_{100y} = 5 l/s per m.

4.4.2. *Conventional rock armour design*

A design wave height of only 3 m is quite mild and a conventional rock armoured structure can certainly be designed without needing too large rock. Damage curves have been calculated in a similar way as in Section 4.3.2. First a rock armour of 1-4 t has been chosen, similar to the berm breakwater design in Section 4.4.2, see Figure 13. A slope angle of 1:2.5 is needed for a stable design. This would result in a similar cross-section as in Figure 9, although with smaller rock.

A 1-4 t rock grading is not a very large grading. In order to overcome the problem with construction of a gentle slope (long crane or excavator reach) it is also possible to consider a slightly larger rock class, for example 3-6 t, which is also a standard grading.

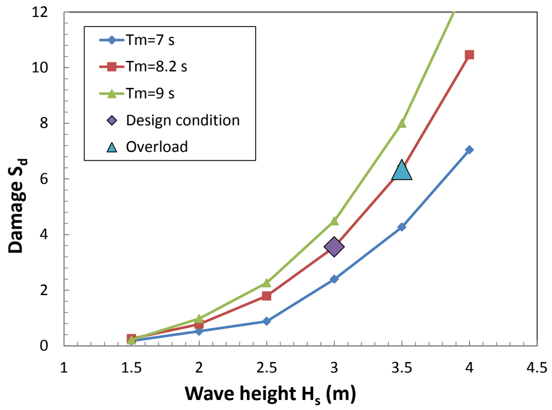


Figure 13. Damage curves for a conventional rock slope with an armour layer of 1-4 t; cotα = 2.5; P = 0.4; N = 3000.

Figure 14 gives similar damage curves as specified in Figure 13, but the rock grading is now 3-6 t and the slope angle required now becomes 1:1.75. This is almost as steep as the seaward slope of the berm breakwaters. An underlayer that will be acceptable, although a little on the small side with respect to the general rule of $1/10^{th}$ to $1/15^{th}$ of the armour layer mass, is 100-300 kg. A standard grading of 300-1000 kg would be too large for an armour layer of 3-6 t. An overtopping discharge of 5 l/s per m is reached for a crest freeboard of 3.45 m. Together with the design water level of +1 m CD, this gives a crest level of +4.5 m CD. This is even a little lower than the +4.8 m CD that is needed for a berm breakwater with 1-4 t rock, see Figure 12. A possible cross-section is shown in Figure 15.

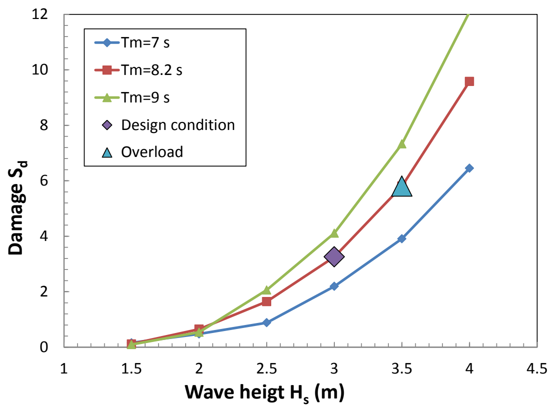


Figure 14. Damage curves for a conventional rock slope with an armour layer of 3-6 t; cotα = 1.75; P = 0.4; N = 3000

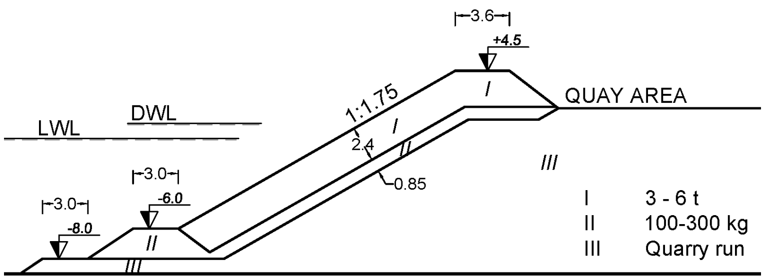


Figure 15. Conventional rock armoured structure with 3-6 t on a slope of 1:1.75. Design for H_{sD} = 3 m and q_{100y} = 5 l/s per m.

The design of the cross-section in Figue 15 can be compared with the berm breakwater design in Figure 12, see Figure 16. The conventional design has a slightly larger armour rock, 3-6 t instead of Class I 1-4 t. The volume of large rock for the conventional design is still significantly larger than for the Class I rock: 63 m^3 per m length against 33 m^3 per m length. But the total volume of rock, taken seaward from the quay area, is larger for the berm breakwater (346 m^3 per m length) than for the conventional design (275 m^3 per m length). The slope of the conventional design is quite steep and the reach needed to place the lowest large rock is not too large. It can possibly be done by a large excavator.

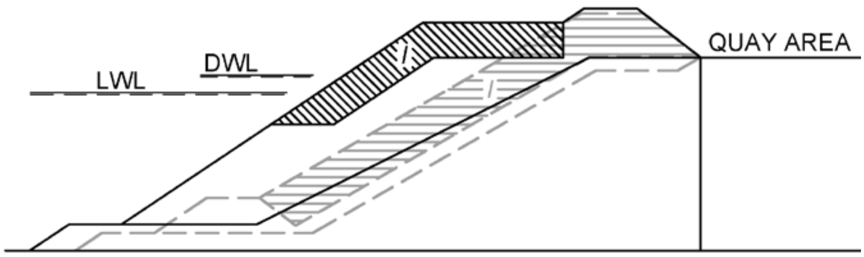


Figure 16. Comparison of conventional cross-section with a berm breakwater design.

Overall, the conventional design needs slightly larger rock, but the total volume of rock is substantially less than for the berm breakwater design. Both structures can easily be constructed.

4.4.3. *Overall conclusions and comparison for a design wave height of 3 m*

A design wave height of H_{sD} = 3 m can be considered as a mild wave climate and a conventional design with a relatively steep slope of 1:1.75 and rock of 3-6 t is well able to resist such a wave climate. If a berm breakwater is designed, quite small rock is sufficient to make a proper design: a hardly reshaping berm breakwater comes to Class I of 1-4 t rock and a partly reshaping berm breakwater to only 0.5-2 t rock. The latter one has not been described here, but is given in Van der Meer and Sigurdarson [2016].

The usage of armour rock of a conventional design, compared to Class I of a berm breakwater, is always significantly larger (roughly a factor of 2). But the total volume of rock in this case of a mild wave climate (and relatively steep slope) is significantly less for the conventional design.

If rock of 3-6 t can be produced, then a conventional design as in Figure 15, may well be cheaper than a berm breakwater design like in Figure 12. Only if this kind of rock cannot be achieved, one can think of a berm breakwater with smaller rock.

This leads to the conclusion that if armour rock is readily available for a conventional design with a steep slope, then a berm breakwater is not a cheaper solution. A berm breakwater may be more attractive if a conventional design leads to too large rock (in large quantities) and with a gentle slope. Such a berm breakwater also needs large rock, but in a much smaller quantity (just Class I rock), and construction will be easier.

4.5. *Design wave height of 7 m*

4.5.1. *Berm breakwater PR IC dedicated quarry, Class I 10-20 t*

The design condition is a quite severe design wave height of H_{sD} = 7.0 m with s_{op} = 0.04, giving T_p = 10.6 s. The overload condition is 15% higher than the design condition, giving H_s = 8.0 m at the design water level DWL. This overload percentage is a little lower than in previous sections and in real design may depend on the steepness of the curve of the extreme wave climate. Tides range between 0 m CD to + 2 m CD (2 m tidal range). The design water level (100-years condition) = surge + maximum tide = +4 m CD. Daily waves are quite moderate and a safety margin of Δw = 1 m above MHWS will be enough for construction. The foreshore is flat at -18 m CD. All input and calculated values, including design choices, are given in the spreadsheet in Appendix D of [Van der Meer and Sigurdarson, 2016].

The wanted resiliency is taken at 30% reshaping, which is the middle value for the range of partly reshaping berm breakwaters. The allowable overtopping is q = 10 l/s per m for the 100-years event and no restriction is given for the overload. The crest level is only calculated for the 100-years condition and becomes 12.5 m CD. Finally, the mass density of seawater is ρ_w = 1,030 kg/m^3 and of rock ρ_r = 2,700 kg/m^3.

A Class I of 10-20 t is a heavy rock grading, beyond standard gradings. It needs a dedicated armourstone production with specific care in quarry yield production and blasting design. But these kinds of rock gradings have been produced in the past. Guidance is given in Van der Meer and Sigurdarson, [2016].

For a Class I of 10-20 t one can choose connecting classes as Class II = 4-10 t and a Class III = 1-4 t, all given in Table 10. The stability number is calculated as $H_s/\Delta D_{n50}$ = 2.44 for a Class I of 10-20 t, which indeed gives a partly reshaping berm breakwater, but quite close to a full reshaping one (the transition is at $H_s/\Delta D_{n50}$ = 2.5). The cross-section as calculated by the spreadsheet is given in Figure 17.

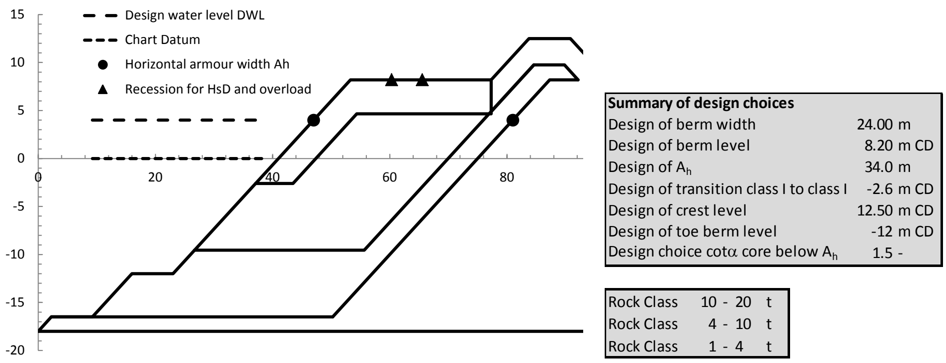


Figure 17. Calculated cross-section for H_{sD} = 7 m and Class I = 10-20 t.

The graph shows the three rock classes. The final design is shown in Figure 18. There is quite some expected recession of the berm, certainly for the overload condition, see Figure 17. It means that reshaping of the berm may well cut into the Class II rock underneath. In order to avoid that the thickness of the seaward side layer of 10-20 t should be increased, see the final design in Figure 17.

A toe berm has been designed at a level of -12 m CD, with Class III of 1-4 t as grading. This will limit the amount of recession a little. It is also possible to design a toe berm at -8 m CD, as in the previous example, but then the toe berm should be constructed of 4-10 t rock, which is Class II rock. In the final design the first choice has been made.

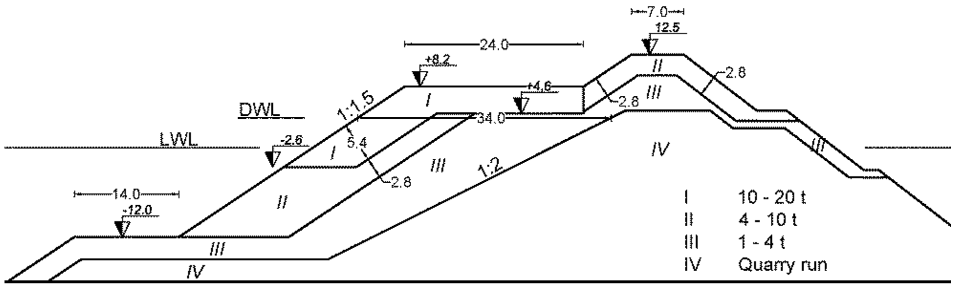


Figure 18. Partly Reshaping Icelandic-type berm breakwater cross-section designed for H_{sD} = 7.0 m, Class I 10-20 t, q_{100y} = 10 l/s per m.

4.5.2. *Overall conclusions for a design wave height of 7 m*

It is hardly possible to design a conventional rock armour layer for a wave climate with a design wave height of 7 m. From a stability point of view, it is only possible with the largest standard grading of 10-15 t in the Rock Manual [2007] and a gentle slope angle of 1:3.5. For a 10-20 t grading from a dedicated quarry, such a design still needs a slope angle of 1:3. For both examples it means that a crane is required with a reach of around 80-100 m that should place stones up to 20 t quite precisely in a double layer deep under water. Another option is to place the armour rock at the toe by barge, but that needs quite calm daily wave conditions to place the rock correctly.

Experience shows that conventional rock armoured structures with these large rock and with such gentle slopes have not been designed and constructed. The main reason is that in such a case one makes the choice to use concrete units on a much steeper slope.

But another option might of course be to design a berm breakwater. Van der Meer and Sigurdarson [2016] give three designs of rock structures (berm breakwaters) that can cope with a 100-years design wave height of 7 m. In this paper only one example is given: a partly reshaping Icelandic-type berm breakwater with Class I of 10-20 t rock. The other two examples, not described here, are a hardly reshaping Icelandic-type berm breakwater with a Class I rock of 20-35 t and a fully reshaping mass armoured berm breakwater with a Class I rock of 6-10 t.

If indeed a Class I of 20-35 t can be obtained from a dedicated quarry, this design is preferred as it gives the smallest total volume of rock and the largest resiliency. But a design with a Class I of 10-20 t (Section 4.9) also gives a good design. A fully reshaping mass-armoured design can be made of rock from a dedicated quarry, but also from standard gradings with Class I of 6-10 t. In this case quite some recession will occur, which means that big rock is moving around from the berm downwards, with possible breakage of rock as a result. For this high design wave height of 7 m, a fully reshaping berm breakwater with only Class I rock of 6-10 t includes some (unknown) risks and if possible one should try to get a larger rock grading for the Class I rock.

"Rock" is the essential word in design of berm breakwaters. Often dedicated quarries can be found and opened to produce the required rock. This is different from designs with rock from existing quarries, where delivery of very large rock classes may be problematic. It has been proven possible to win really large rock in dedicated quarries and this experience has been described in Van der Meer and Sigurdarson [2016]. Quarry and project management as well as blasting and sorting techniques are essential in getting all required rock for an acceptable price. With this experience it is possible to design and construct berm breakwaters for a design wave height of 7 m or even more.

Construction of dedicated Icelandic-type of berm breakwaters is still fairly easy with excavators of maximum 120 t, which can handle rocks up to 35 t. Heavier excavators will probably become available on the market so the future may well be that rock over 35 t can be used in construction.

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