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Bed, Bank and Shore Protection 2

Breakwaters and Closure Dams
Part two

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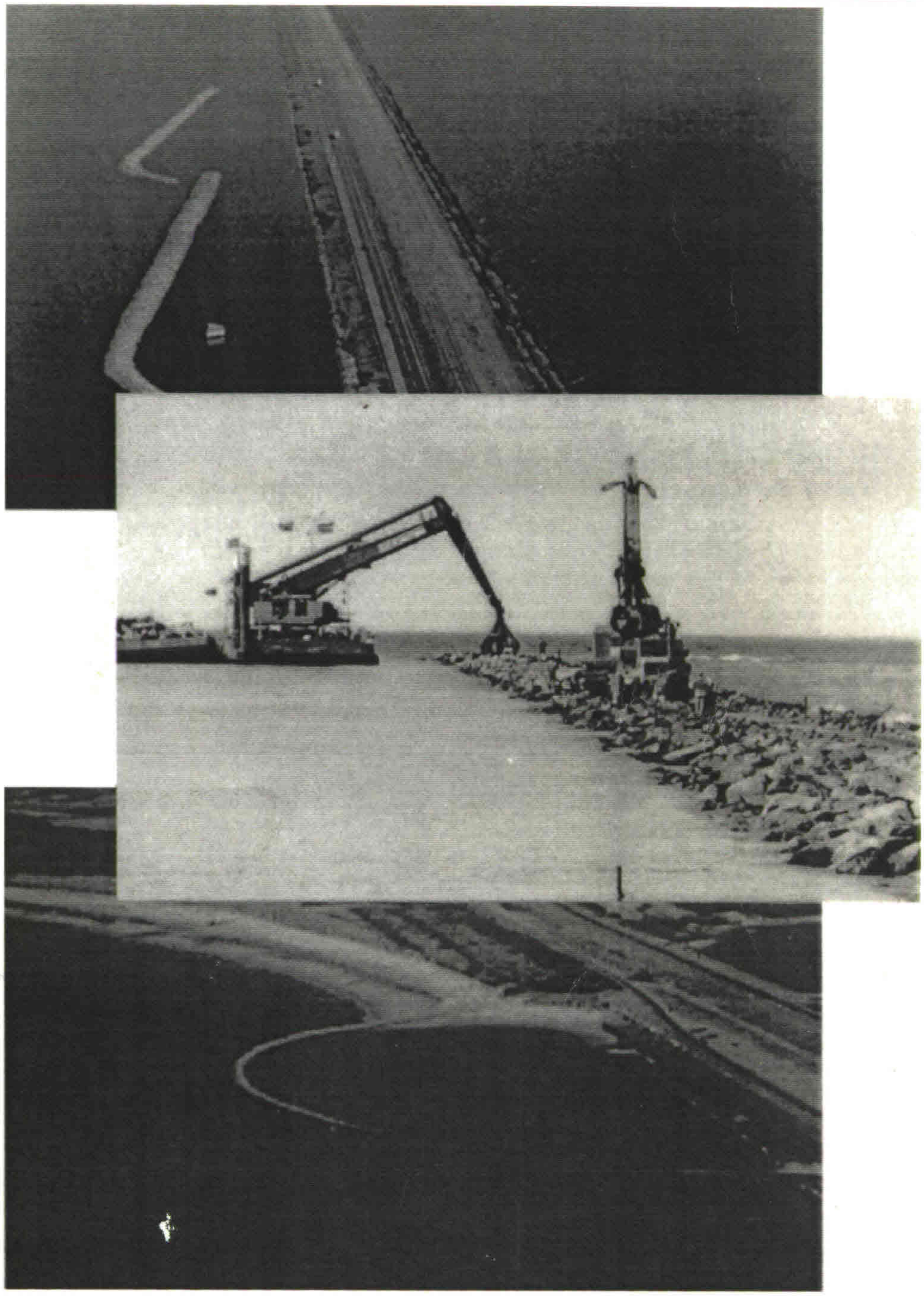

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6. Data Collection

6.1 General

In chapter 5, we have refreshed some of the theory that we need for designing structures like breakwaters and closure dams. In practice, however, it is not only important to master the theory, but it is equally important to assess the physical presence of certain phenomena that are known from theory. For each subject, attention is paid in this chapter to the availability and if necessary the collection of relevant data on the prevailing hydraulic, geotechnical and other conditions.

For design purposes, we are sometimes interested in extreme events, specifically, when we try to establish the loading conditions for the Ultimate Limit State. In many cases it is not possible to use direct observations since our records are too short to make a sensible assessment of extreme events. In that case we have to rely upon an indirect approach, in which we use data that have been recorded over a sufficiently long period. In this respect specific reference is made to series of meteorological data (wind direction, wind speed, barometric pressure, temperature, rainfall, visibility, humidity) that have been collected even in remote areas over a much longer span of time than wave heights, tidal currents and river discharges. Meteorological stations, airports, hospitals and even missionary outposts may come up with a surprising wealth of data. Calculation and calibration can then transform such data into the required data.

6.2 Hydrographic Data

6.2.1 Bathymetry

Before starting a design job, it is essential to have proper maps of the seabed or riverbed available. In most cases, hydrographic charts are useful but their scale is such that the information is not detailed enough. The same applies for standard river navigation charts. Another problem is that the datum of those charts is often related to tidal levels (MLWS or similar) and that the datum needs not be the same for the whole chart. This can pose serious problems when quantities of dredge or fill material have to be determined on the basis of such charts. The advantage is that for most regions in the world such reliable charts have been available for at least 200 years. The hydrographic departments in several countries preserve those old charts. They can provide very valuable information about long-term morphological developments in the region.

In practice, it means that most projects require specific maps to be made. Nowadays a hydrographic survey poses not too many problems. One needs a survey launch, equipped with echosounder, (D)GPS positioning, a reliable radio tide gauge and a data logger. When the sea is rough and the survey launch makes considerable movements due to waves, one may add a swell compensatory as well. With the aid of modern software, a plotter without much human input can draw the maps. Because the data are available in a digital form, it is also possible to use them for all kinds of arithmetical exercises, such as calculating volumes to be dredged or filled, but also to asses erosion or accretion in between subsequent surveys.

It is wise to pay attention to the type of echosounder used. When a high frequency (210 kHz) is used for the measuring beam, the depth indicated is the top of a soft layer. When lower frequencies (30 kHz) are used, the beam penetrates into the soft mudlayers. By using a dual frequency instrument, one can obtain an impression of the thickness of such layers of soft mud.

6.2.2 Tides (See also chapter 5.2.1.)

a Vertical tides

The tidal constants for most important harbours in the world are known. They are published annually either by a national hydrographic office or by the British Admiralty[1].
For minor ports, one has to rely on national or local authorities, and the reliability of data provided may not always be as good as required.
Setting up a local observation point and performing hourly observations of the water level during a period of one month can obtain a provisional insight. Application of harmonic analysis techniques easily leads to a reasonable estimate of the most important tidal constants. Only when one is interested in the long periodic components a longer observation period is required.

b Horizontal tides

Tidal currents are sometimes indicated on hydrographic maps. The accuracy is mostly insufficient for design or planning purposes. Some hydrographic departments issue flow atlases[2, 3] with more comprehensive information. Dedicated flow measurements can generally not be avoided. They are however time consuming since they have to be continued for at least 13 hours. It is therefore advisable to analyse flow phenomena by mathematical model and to use field measurements mainly to calibrate the model.

6.2.3 Storm surges

Tidal water level variations can be predicted accurately. In excess of the tidal variations, there may be meteorological effects that influence the water levels. Since meteorological effects can not be predicted long in advance, one must take them into account on a statistical basis. If no direct observations are available, one may use recorded wind velocities, barometric pressures and hurricane or cyclone paths to estimate the probability of extreme water levels.

6.2.4 Waves

There are few places in the world where long series of wave observations are available. This is simply the case because reliable instruments for wave recording did not exist until recently (say 1970). The oldest observations of waves were carried out on board of ships that had a voluntary arrangement with a meteorological office to carry out certain observations. Although the observations of waves were visual observations, the accuracy is acceptable since the officers were well trained and could compare the observed state of the sea with standard pictures provided by the met office. These observations were collected and sorted according to locations spread over the oceans. In this way, the first wave atlases were edited[4]. More recently, a large collection of similar data has been assembled and edited[5, 6]. A disadvantage of these data sets is that the oceans have been divided into relatively large areas, so that detailed information close to the shore is still not readily available.

Although direct measurement of wave heights in the preparation phase of a project will never provide the required long term data, it is still useful to have such direct observations, were it only to calibrate the calculation methods used to transform indirect observations into local wave data.

Modern methods for wave measurement are

- electric (resistance or capacitor type) wave gauges, mounted on a platform or a pile,
- acceleration type gauges, mounted in a floating buoy;
- pressure gauges, mounted on the seabed
- inverted echosounder, measuring the distance from sea bed to water surface;
- remote sensing techniques (from satellite).

A problem is that one wants to measure wave heights in relatively deep water, so that shoaling or breaking does not (yet) affect the measured heights. This makes all pile or platform mounted gauges relatively expensive, unless use can be made of an existing facility. Pressure gauges are not recommended because the actual wave pressure at bottom level is not only a function of the wave height, but also of the ratio between wave length and water depth. One of the most popular instruments is the (Dutch) Waverider Buoy, fabricated by Datawell, Haarlem. This device measures the vertical acceleration of the water surface, and transforms this by double integration into a vertical motion. This makes the observation of swell (T> 20 s) difficult and not fully reliable. Remote sensing techniques are still in an experimental stage, but the prospect is good, certainly because observation of remote wave fields can result in timely warning for bad weather.

6.3 Meteorological data

Although meteorological phenomena do not play a direct role in the design of hydraulic structures, they are important in an indirect way. In the previous paragraphs, we have discussed the importance of barometric pressures and wind data as generators of surges and waves. In a similar way, precipitation plays a major role in the generation of river discharges. Wind plays a direct role, when one considers forces on ships and structures or the effect on spray of breaking waves.

Other factors may be important as well, though their role is not so obvious. In this sector, we must mention visibility, which is important because any marine operation is seriously hampered by fog. Temperature and humidity are important to equipment (cooling, corrosion), but also for hardening concrete.

Data are generally available in the national meteorological offices; measuring instruments are easily available in the market.

6.4 Geotechnical data

Required geotechnical data for a breakwater or closure dam certainly include all data required to assess the bearing capacity of the subsoil, both during construction and later on. Stability of the works shall be ascertained during all phases. Further, one wants to predict settlements as a function of time; in order to ensure that the required crest level is available permanently. In many cases, the works will be accompanied by substantial dredging, so that soil properties for this purpose must also be known. In case erosion or scour is expected to occur, it is necessary to establish the resistance of the existing seabed against this threat. *Table 6-1, Required soil data for the evaluation of the geotechnical limit states (after the CUR/RWS Manual)*[7] gives a good impression of the geotechnical failure mechanisms and their relation with basic geotechnical data.

Geotechnical limit states					Geotechnical information	
Macro instability			Macro-failure	Micro-instability		
Slip failure	Lique-faction	Dynamic failure	Settle-ments	Filter-erosion	Name	Symbol
A	A	A	A	A	Soil profile	-
A	A	A	A	A	Classification/grain size	D
A	A	A	B	A	Piezometric pressure	p
B	B	B	A	A	Permeability	k
A	B	B	A	B	Dry/wet density	$\rho_d, \rho_{wt}, \rho_{wk}, \gamma=\rho g$
-	A	B	-	-	Relative density, porosity	n, n_e
A	B	B	-	C	Drained shear strength	c, ϕ
A	-	-	-	C	Undrained shear strength	s_u
B	-	-	A	-	Compressibility	C_c, C_s
A	-	-	A	-	Consolidation coefficient	c_v
B	B	A	A	-	Moduli of elasticity	G, E
B	A	A	A	-	*In situ* stress	σ
-	A	B	A	-	Stress history	OCR
B	A	A	B	-	Stress/strain curve	G, E

A: Very important B: Important C: Less important

Table 6-1, Required soil data for the evaluation of the geotechnical limit states

This means that for a major project always a general geological analysis must be carried out to obtain a proper understanding of the geophysical and hydro-geological conditions. The most important aspects that require attention are:

- Geological stratification and history
- Groundwater regime
- Risk of seismic activities.

Basic data can be obtained from the national geological services and in more general terms from scientific libraries and universities. Most of the available information will refer to land, and little to estuaries and sea. Such basic geological data will provide a general insight in what can be expected in the area of interest. Mostly it is insufficient for engineering purposes, so that in any case additional soil investigations have to be carried out. These investigations may comprise of the following methods:

- Penetration tests (CPT or SPT) to establish in-situ soil properties;
- Borings to take samples at various depths for further analysis in the Laboratory;
- Geophysical observations.

Those specific investigations are expensive and difficult, because they must be done at sea, under the direct influence of tides, waves and currents. This makes any penetration test or boring to a time consuming and risky affair. Therefor there is a tendency to limit the number of such local tests. This imposes the risk that discontinuities in between the measuring locations will not be recognised.
That is the reason to combine such local observation with a geophysical survey. The geophysical survey uses electro-resistivity, electro-magnetic and seismic techniques to obtain a continuous image of the soil conditions in the tracks sailed with a survey vessel. The disadvantage is that there is mostly no direct link between measured data and the geotechnical soil properties. Combination of geophysical survey and point measurements eliminates the disadvantages of both. The geophysical data ensure that no discontinuities are overlooked, while the boreholes and penetration tests provide the link with the actual engineering properties of the soil.

Table 6-2, In situ test methods and their perceived applicability (after CUR/RWS publication 169), gives a complete view of the available in-situ test methods and their applicability.

	Site-investigation methods												
	Geophysical methods (Section C.1)				Penetration methods (Section C.1.2)						Borings (Section C.1.3)		
	Seismic	Electr. resist.	Electro-magnetic	nuclear	Cone penetr. test (CPT)	Piezo cone test (CPTU)	Stand. penetr. test (SPT)	Field vane test (VST)	Press. meter test (PMT)	Dilato meter test (DMT)	Dist. samples	Undist. samples + Lab. tests	Moni-ring wells
Soil profile	C	C	C	-	A	A	A	B	B	A	A	A	-
Classification	-	-	-	-	B	B	B	B	B	B	A	A	-
Piezometric pressure	-	-	-	-	-	A	-	-	B	-	-	-	A
Permeability	-	-	-	-	-	B	-	-	B	-	C	A	C
Dry/wet density	-	-	-	A	C	C	C	-	-	-	C	A	-
Relative/density	-	-	-	-	B	B	B	-	C	C	-	A	-
Friction angle	-	-	-	-	B	B	B	C	C	C	-	A	-
Undr. shear strength	-	-	-	-	B	B	C	A	B	B	-	A	-
Compressibility	-	-	-	-	C	C	-	-	C	C	-	A	-
Rate of consolidation	-	-	-	-	-	A	-	-	A	-	-	A	C
Moduli of elasticity	A	-	-	-	B	B	B	B	B	B	-	A	-
In-situ stress	-	-	-	-	C	C	-	C	B	B	-	A	-
Stress history OCR	-	-	-	-	C	C	C	B	B	B	-	A	-
Stress/strain curve	-	-	-	-	-	C	-	B	B	C	-	A	-
Ground conditions													
Hard rock	A	-	A	A	-	-	-	-	A	-	A	A	C
Soft rock-till, etc.	A	-	A	A	C	C	C	-	A	C	A	A	A
Gravel	A	B	A	A	C	C	B	-	B	-	A	C	A
Sand	A	A	A	A	A	A	A	-	B	A	A	C	A
Silt	A	A	A	A	A	A	B	B	B	A	A	A	A
Clay	A	A	A	A	A	A	C	A	A	A	A	A	A
Peat-organics	C	A	A	A	A	A	C	B	B	A	A	A	A

A: High applicability B: moderate applicability C: Limited applicability

Table 6-2, In situ test methods and their perceived applicability

6.5 Construction Materials, Equipment, Labour

6.5.1 Construction Materials

The most important construction materials for closure dams and breakwaters are quarry stone and concrete.

A Quarry stone

Quarry stone is natural rock, obtained from quarries. Basically, there are three or four different types of quarries:

- Commercial quarries for the production of ornamental tiles (marble, etc.)
- Commercial quarries for the production of fine aggregates for concrete, road construction, etc.
- Commercial quarries for the production of large size rock for hydraulic engineering
- Dedicated quarries for the same

It is evident that the quarries for ornamental stone are not relevant for our purpose. The quarries for the production of aggregates are in general not suited to supply the size of stone required for large hydraulic engineering projects. The fine material can be used, however, as filter material.

In some parts of the world, where a regular demand exists for larger size stone, some quarries have specialised in this field. They offer stone in standard weight categories, mostly according to the national standards of their customers. Such quarries exist for instance in Belgium, Germany, Norway, Sweden and Scotland.

The relevant properties are generally known and listed in catalogues.

The situation is different if a large project is to be executed in an area where no such quarries exist. In that case, a rock formation has to be found that can be used to open a dedicated quarry, specifically for the project. The following data should in any case be obtained:

- Specific weight and density of the material
- Durability in air and in (sea) water
- Resistance against abrasion
- Strength (tensile and compressive)
- Maximum size that can be obtained and distribution curve.

In general, these data are so important that it is worthwhile to employ a specialised group of geologists to find a suitable location for a quarry. It is even recommended to carry out one or more test blasts before a final decision is taken to open a quarry. Apart from the technical data on the rock, it is necessary to be sure that the quarry operation can be started from a social, environmental and legal point of view. Since quarry stone and quarries are quite essential for any major hydraulic engineering project, more details are provided in Annex 1.

B Concrete

Specifically when a large project is to be executed in a remote area, it is essential to be sure of the quality and availability of other construction materials as well. For closure dams and breakwaters, it is hardly possible to avoid the use of concrete. It is therefore recommended to collect data on the availability and quality of cement, aggregates, water and reinforcing steel. It is also essential to study the climatological conditions to see if special measures are required for curing the fresh concrete.

In this respect it is also important to know of any local codes and standards, and if the obligatory sections thereof are not interfering with our own quality assurance.

6.5.2 Equipment

There is a large mutual influence between the design of a breakwater or closure dam and the construction method. In the same way, the equipment to be used depends largely on the construction method and vice versa.

In a similar way, during the design stage, questions of maintenance and repair shall be discussed. Do we rely upon regular inspection and maintenance, or do we opt for a more or less maintenance free structure?

This leads all to the main question whether we will try to use locally available equipment, or whether we mobilise the required heavy equipment from elsewhere? In case we want to use local equipment it is necessary to obtain a detailed insight in quality, capacity and cost of such equipment. In case we decide to mobilise the equipment, the questions are how we can get the equipment to the location, and whether there are any restrictions on (temporary) import. Local conditions play a role like temperature (cooling of engines), dust (capacity of air filters), quality of fuel and lubricants, etc.

6.5.3 Labour

When planning a large project, it is also essential to know whether there is a skilled local labour force, and whether it is allowed to employ skilled and partly skilled expatriate labour. In many cases special facilities are to be provided for the accommodation of personnel. Such facilities shall be available right from the start of the actual construction. Poor working and living conditions will have a strong negative influence on the quality of the work.

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7. Stability of randomly placed Rock Mounds

7.1 Introduction

Although the stability of individual stones on a slope under wave attack is certainly not the only criterion for the proper functioning of a rubble mound breakwater, it is a subject that deserves a lot of attention. This is because many breakwaters failed due to a defective design in this respect.

7.2 Historic Review

7.2.1 General

As indicated in chapter 3, breakwater design was for many years a question of trial and error. It was short before WW II that Iribarren in an attempt to understand the influence of rock density developed a theoretical model for the stability of stone on a slope under wave attack. Iribarren continued his efforts throughout the years until his final publication on the subject at the PIANC conference of 1965 in Stockholm.

In the mean time, in the USA, the US Army Corps of Engineers had developed a keen interest in the stability of breakwaters, and long series of experiments were carried out by Hudson at the Waterways Experiment Station in Vicksburg.

Where Iribarren focussed on a theoretical approach, assisted by some experiments, Hudson concentrated on collecting a large data set from hydraulic model experiments to derive conclusions from an analysis of those data. In both cases, experiments were carried out using the then standard techniques, i.e. by subjecting the models to regular, monochromatic waves.

The experiments comprised the construction of an infinitely high slope, covered with stones of a particular weight and density. The slope was then exposed to a wave train with waves of a particular height and period, starting with low waves and increasing the height in steps, until loss of stability of the stones was observed. It must be kept in mind that loss of stability is not a clearly defined phenomenon. There is an amount of subjectivity involved, in particular because the loss of the first stones may not de attributed completely to the wave action, but at least partly to the accidental position of the stone after construction. In the following, the work of Iribarren and Hudson will be explained in more detail.

7.2.2 Iribarren

Iribarren[1, 2, 3, 4, 5, 6] considered the equilibrium of forces acting on a block placed on a slope. Since the considerations of Iribarren referred to forces, the weight of the block W is introduced as a force, and thus expressed in Newton. (It is good to realise that in literature, one finds the block size indicated by weight or by mass.)

The forces acting on a unit, positioned on a slope under an angle α, are (*Figure 7-1 , Equilibrium of forces after Iribarren*):

- Weight of the unit (vertical downward)
- Buoyancy of the unit (vertical upward)
- Wave force (parallel tot the slope, either upward or downward)
- Frictional resistance (parallel tot the slope, either upward or downward, but contrary to the direction of the wave force)

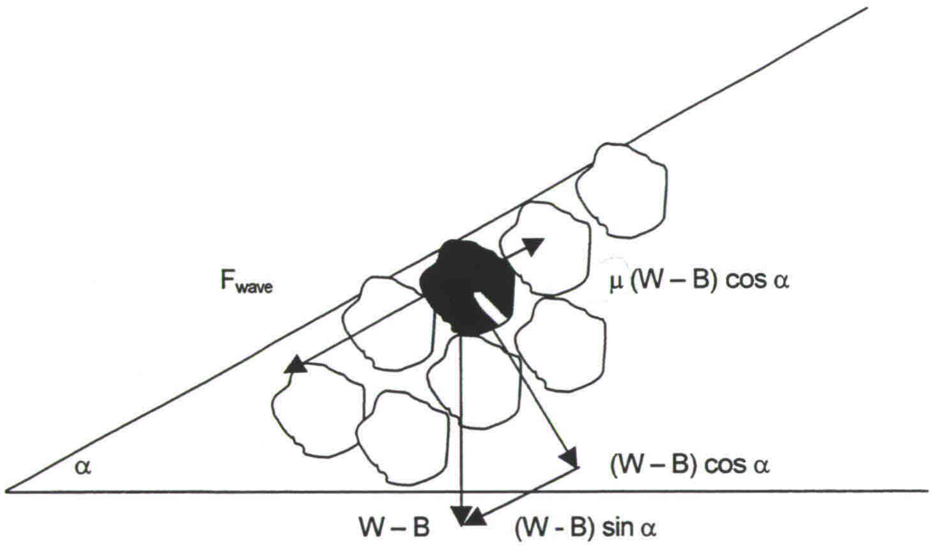


Figure 7-1, Equilibrium of forces after Iribarren

Iribarren resolved these forces into vectors normal and parallel to the slope. Loss of stability occurs if the friction is insufficient to neutralise the other forces parallel to the slope.

The parameters are:

W	= weight of block	[N]
B	= buoyancy of block	[N]
W-B	= submerged weight of block	[N]
V	= volume of block	[m^3]
α	= angle of slope	[-]
μ	= friction coefficient	[-]
ρ_r	= density of block (rock)	[km/m^3]
ρ_w	= density of (sea) water	[km/m^3]
Δ	= $(\rho_r - \rho_w)/\rho_w$	[-]
H	= wave height	[m]
D_n	= characteristic size of stone = $V^{1/3}$	[m]
F_{wave}	= wave force	[N]

Iribarren assumed a set of simple relations between F_{wave}, D_n, H, ρ and g as follows:

$$F_{wave} = \rho_w . g . D_n^{\ 2} . H \qquad (7.1)$$

$$W - B = (\rho_r - \rho_w) D_n^{\ 3} \qquad (7.2)$$

and

$$W = \rho_r D_n^{\ 3} \qquad (7.3)$$

It must be mentioned here that these relations are subject to criticism, since they are too simple. It must be expected that the shape of the block and the period of the wave play a role. Further, the relation between the wave force and the wave height and stone size point at a dominance of drag forces, whereas acceleration forces are neglected.

Nevertheless considering the equilibrium for downrush along the slope, this leads to a requirement for the block weight:

$$W \geq \frac{N\rho_r g H^3}{\Delta^3(\mu\cos\alpha - \sin\alpha)^3} \quad \text{(7.4)}$$

For uprush, the formula changes into:

$$W \geq \frac{N\rho_r g H^3}{\Delta^3(\mu\cos\alpha + \sin\alpha)^3} \quad \text{(7.5)}$$

N is a coefficient that depends (a.o.) on the shape of the block, the value must be derived from model experiments. The friction factor μ can be measured by tilting a container filled with blocks and determining the angle of internal friction.

In Iribarren(1965), recommendations are given for values of N and μ. The most important values are given in *Table 7-1 , Coefficients fot Iribarren formula.*

Type of block	Downward Stability ($\mu\cos\alpha - \sin\alpha)^3$		Upward Stability ($\mu\cos\alpha + \sin\alpha)^3$		Transition slope between upward and downward stability
	μ	N	μ	N	cot α
Rough angular quarry stone	2.38	0.430	2.38	0.849	3.64
Cubes	2.83	0.430	2.83	0.918	2.80
Tetrapods	3.47	0.656	3.47	1.743	1.77

Table 7-1 , Coefficients fot Iribarren formula

It must be kept in mind that the coefficient N represents many different influences. At first, it is a function of the damage level defined as “loss of stability”. It further includes the effect of the shape of the blocks, but not the internal friction, because this is accounted for in the separate friction coefficient. Finally, it covers all other influences not accounted for in the formula.

7.2.3 Hudson

Since 1942, systematic investigations into the stability of rubble slopes have been performed at the Waterways Experiment Station in Vicksburg, USA. On the basis of these experiments, Hudson[7, 8, 9] proposed the following expression as the best fit for the complete set of experiments:

$$W \geq \frac{\rho_r . g . H^3}{\Delta^3 . K_D . \cot\alpha} \quad \text{(7.6)}$$

The formula is applicable for slopes not steeper than 1 : 1½ and not flatter than 1:4.

The coefficient K_D represents many different influences just like the coefficient N in the formula of Iribarren. At first, it is a function of the damage level defined as “loss of stability”. It further includes the effect of the shape of the blocks and the internal friction. Finally, it covers all other influences not accounted for in the formula.
Recommended values for K_D have regularly been published and updated by the Corps of Engineers in the Shore Protection Manual. Subsequent editions of this manual thus reflect the changing insight over the years.
In the 1977 edition[10], the wave height H is defined as the significant wave height H_s, and the values for the most common types of blocks are given in *Table 7-2, K_D Values recommended given in SPM 1977*:

Type of block	Number Of Layers (N)	Structure Trunk K_D		Structure Head K_D	
		Breaking Wave	Non breaking wave	Breaking wave	Non breaking wave
Rough angular quarry stone	1	**	2.9	**	2.3
Rough angular quarry stone	2	3.5	4.0	2.5*	2.8*
Rough angular quarry stone	3	3.9	4.5	3.7*	4.2*
Tetrapod	2	7.2	8.3	5.5*	6.1*
Dolos	2	22.0	25.0	15.0	16.5*
Cube	2	6.8	7.8		5.0

*There is a slight variation of recommended K_D value for different slopes
**Use of single layer is not recommended under breaking waves

Table 7-2, K_D Values recommended given in SPM 1977

In the 1984 edition[11], following a number of dramatic failures of rubble mound breakwaters, it is recommended to use H_{10}, the average of the highest 10 percent of all waves. This is equal to 1.27 H_s.

Type of block	Number Of Layers (N)	Structure Trunk K_D		Structure Head K_D	
		Breaking Wave	Non breaking wave	Breaking wave	Non breaking wave
Rough angular quarry stone	1	**	2.9	**	2.2
Rough angular quarry stone	2	2.0	4.0	1.6*	2.8*
Rough angular quarry stone	3	2.2	4.5	2.1*	4.2*
Tetrapod	2	7.0	8.0	4.5*	5.5*
Dolos	2	15.8	31.8	8.0	16.0*
Cube	2	6.5	7.5		5.0

*There is a slight variation of recommended K_D value for different slopes
**Use of single layer is not recommended under breaking waves

Table 7-3, K_D Values recommended given in SPM 1984

Be careful: probably too conservative!

A comparison between *Table 7-2, K_D Values recommended given in SPM 1977* and *Table 7-3, K_D Values recommended given in SPM 1984* shows a much more conservative design recommendation in 1984. Not only have the values of K_D been changed, but also the replacement of Hs by H10 is quite a dramatic change, certainly if one realises that the wave height appears with a third power in the Hudson formula. This results in the opinion of many designers in a too conservative approach!

Hudson defines the K_D value for initial damage: 0-5% of the blocks in the armour layer. He counts the number of blocks from the centre of the crest down the outer slope to a level equal to the "no-damage wave height", $H_{D=0}$, below still water level. It is important, however, to know what happens when the wave height is larger than the zero damage wave height, in other words, when the structure is overloaded. The Shore Protection Manual gives data for various types of armour units and various levels of over-loading. These data are summarised in *Table 7-4, Damage due to over-loading.*

Unit		Damage (D) in percent						
		0-5	5-10	10-15	15-20	20-30	30-40	40-50
Quarry stone smooth	$H/H_{D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarry stone rough	$H/H_{D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	1.56
Tetrapod	$H/H_{D=0}$	1.00	1.09	1.17	1.24	1.32	1.41	1.50
Dolos	$H/H_{D=0}$	1.00	1.10	1.14	1.17	1.20	1.24	1.27

Table 7-4, Damage due to over-loading

Be careful, damage due to breaking of units not included
Damage percentage ≥ 30-40 often means failure

7.2.4 Comparison of Hudson and Iribarren formulae

When comparing the formulae of Iribarren and Hudson, the difference appears to be greater than it in fact is. The influence of wave height, rock density and relative density is equal. The coefficients are different, but can easily be compared. The main difference occurs in the influence of the slope. A comparison of the two expressions within the validity area of the Hudson formula ($1.5 < \cot\alpha < 4$) reveals that a proper choice of coefficients leads to a minor difference between the two formulae only. It is evident that for very steep slopes (close to the angle of natural repose) Hudson can not give a reliable result. It is also likely that for very flat slopes waves will tend to transport material up the slope, which was not considered by Hudson at all.

This becomes clearer when one takes the third root from both formulae. The stability expression changes then to:

Hudson: Fout?   Iribarren: Fout?

$$\frac{H}{\Delta D} = \frac{K_D^{-1/3}}{\sqrt[3]{\cot\alpha}} \qquad \frac{H}{\Delta D} = \frac{N^{1/3}}{\mu\cos\alpha \pm \sin\alpha} \quad \text{.............................(7.7)}$$

The coefficients in the formulae are a sort of dustbins for all kind of unknown variables and unaccounted irregularities in the model investigations. Variables brought together in the Coefficients K_D and N are:

- Shape of the blocks
- Layer thickness of the outer ("armour") layer
- Manner of placing the blocks
- Roughness and interlocking of the blocks
- Type of wave attack
- Head or trunk section of the breakwater
- Angle of incidence of wave attack
- Size and porosity of the underlying material
- Crest level (overtopping)
- Crest type
- Wave period
- Shape of the foreshore
- Accuracy of wave height measurement (reflection!)
- Scale effects if any

In view of this, one can not expect a large consistency in reported values of K_D. In fact, there is a tremendous scatter in the results, and this is no surprise. For the designer it means that he shall be extremely careful when applying the formulae. In the application, one must realise what influence uncertainties have on the final result. This applies for the selection of the coefficients, but also for the choice of wave height and relative density. Small changes have a large influence on the required block weight.

Since there is no basic difference between the two formulae (as long as one applies the Hudson formula within limits for the slope), one can work with either formula. Many designers prefer the Hudson formula because it is a little simpler to use and because there are far more experimental data on the coefficient K_D than on the Iribarren coefficients.

7.2.5 Application of Hudson formula

The Hudson formula gives the designer a clear picture of available means to improve the stability of the armour layer, and their effectivity:

Increase of ρ_r
This can be done by selecting rock from a different quarry, or by producing concrete with heavy aggregates. Because of the influence of Δ, it is very effective.

Increase W
When natural rock is used there is generally a maximum block size that can be obtained from a quarry. When concrete units are used, one must take into account that very large units pose problems of structural strength.

Decrease slope
Be careful, the volume of material increases rapidly. Too flat slopes may lead to upward loss of stability, not covered by the Hudson formula. The method is only used for armour layers consisting of quarry stone. By flattening the slope, it may be possible to use cheaper local material, so that the extra volume of material is compensated. When concrete armour units are used, it can easily be demonstrated that it is always more economic to apply the steepest possible slope (1:1.5).

Grout smaller blocks together with asphalt
This method has been used at the breakwater in IJmuiden, and at several locations in the Netherlands. Special care is required to avoid uplifting of grouted layers due to pressures building up under the armour layer.[12]

Increase K_D by using specially shaped interlocking blocks
The most effective blocks shapes are the very slender blocks like Dolos. Because of breakage, their use is limited to smaller sizes. This reduces the applicability. It must be realised that special block shapes are costly because the higher cost of the moulds, the labour intensive use of the moulds, the difficulties when handling and stacking the blocks.

The use of all kinds of specially shaped concrete armour units has become quite popular. There has been a period that no self-respecting laboratory or consultant could do without his own armour unit.

This started with the development of Tetrapods by Sogreah; it was followed by (a.o.) Akmon, Dolos, and many others. The merit of the Tetrapod was that it demonstrated that by interlocking a K_D value could be obtained that was about twice as high as the values for quarry stone, thus leading to half the weight. The disadvantage was the complicated shape, requiring an expensive mould. Since a patent protected the shape, royalties levied by the inventor consumed part of the potential saving. This resulted in a Dutch initiative to develop a similar unit, and to levee no patent rights. This resulted in the Akmon unit, yielding about equal K_D factors as the Tetrapod. Complexity of the mould was similar, however.

Following the development of the Akmon, Zwamborn in S. Africa attempted to maintain the basic shape of the Akmon, but to further increase the porosity by making the legs more slender. Initially, this was very promising, yielding K_D values of 20 and higher. At the same time, the sizes of vessels were growing, requiring longer breakwaters, extending in deeper water with higher waves. This resulted in the design of some breakwaters (Sines, Portugal is the most striking example) with very large unreinforced units. In a very short period thereafter, a large number of breakwaters failed. It appeared that the mechanical strength of the concrete was insufficient to resist the forces, specifically during rocking of the units against each other. This had never been investigated in a model, and if it had done, it would have had no result, because of scale effects.

It has been recommended in the mean time to adapt a more conservative approach, and to prevent rocking for slender units larger than 20 to 25 tons. Another development was to avoid the use of slender units and to rely upon simple cubes. Although the required weight is larger than for the more sophisticated shapes, considerable savings are achieved on the cost of moulds, the cost of casting, the storage and the handling. Most recently, Sogreah developed a massive block, the Accropod®, which can be used in a single layer, provided it is carefully placed in a certain pattern.

An impression of the best known blocks can be obtained from *Figure 7-2, Tetrapod, Figure 7-3, Akmon, Figure 7-4, Dolos and Figure 7-5, Accropod.*

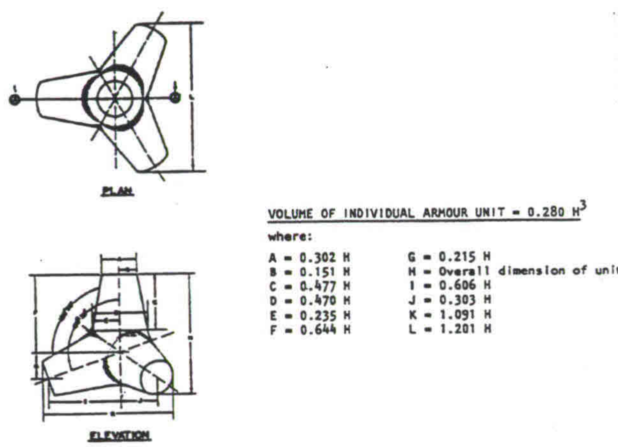
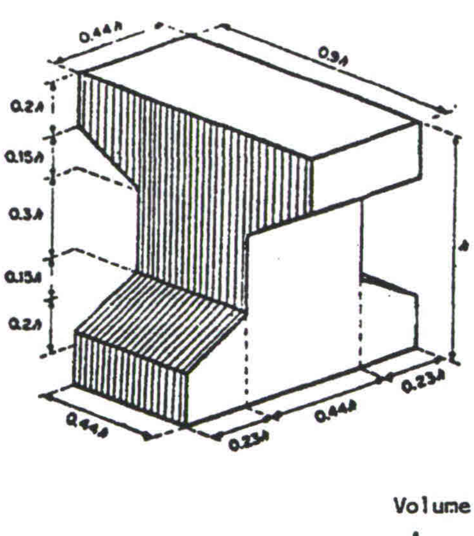


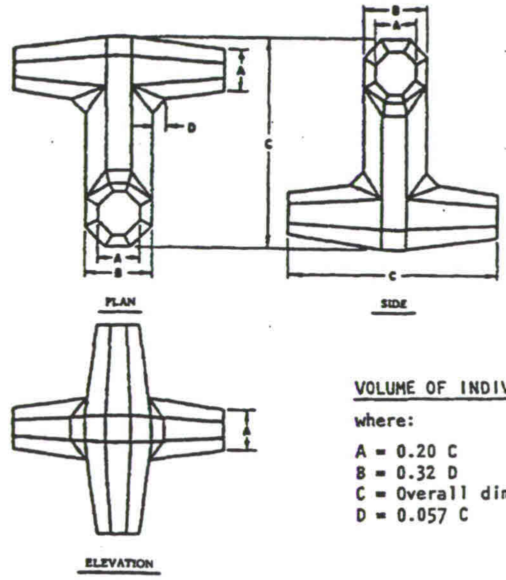
Figure 7-2, Tetrapod



Volume of block : 0.280 h3

[illegible]

Figure 7-~~3~~4, Akmon

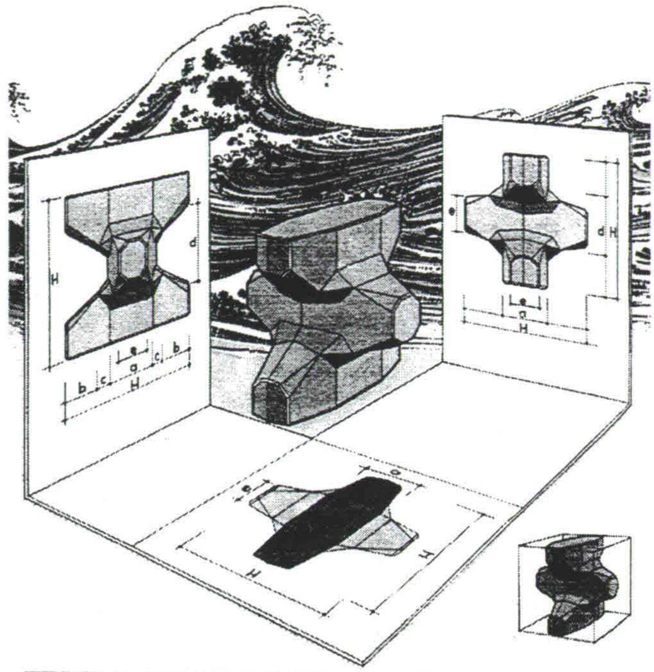


VOLUME OF INDIVIDUAL ARMOUR UNIT = $0.16\ C^3$

where:

A = 0.20 C
B = 0.32 D
C = Overall dimension
D = 0.057 C

Figure 7-~~4~~ 3, Dolos



Dimensions	a	b	c	d	e	H
X by H to obtain dimension	0.376	0.222	0.093	0.555	0.259	1

Total envelope surface of block = $3.46 H^2$

Figure ~~7-5~~ 4, Accropod®

7.3 Irregular waves, Approach of Van der Meer

7.3.1 General

Between 1965 and 1970, the first wave generators were developed that could generate irregular waves according to a certain predefined spectrum. Model tests in the first years were aimed at ad-hoc investigations. Several researchers attempted to overcome the shortcomings of the Hudson approach by introducing more variables. Initially, their results did not point in the same direction. In his PhD thesis at Delft University in 1988, van der Meer[13] succeeded in presenting an approach based on irregular waves that has gradually been accepted throughout the engineering community.

In the first place, he used a clear and measurable definition of damage. Initially, this was expressed by the parameter

$$S = A \Big/ {D^2}_{n50} \qquad \text{.......(7.8)}$$

in which

A	= the erosion area in a cross section in	[m²]
$D_{n50} = \left(W_{50} \big/ \rho_r \right)^{1/3}$		[m]
W_{50}	= average mass of armour stones	[kg]
ρ_r	= density of armour stone	[kg/m³]

For a definition sketch one is referred to *Figure 7-6, Damage(S) based on ~~ersion~~ Erosion area (A).* The area A is often measured using a rod with a half sphere of a certain size attached to it.

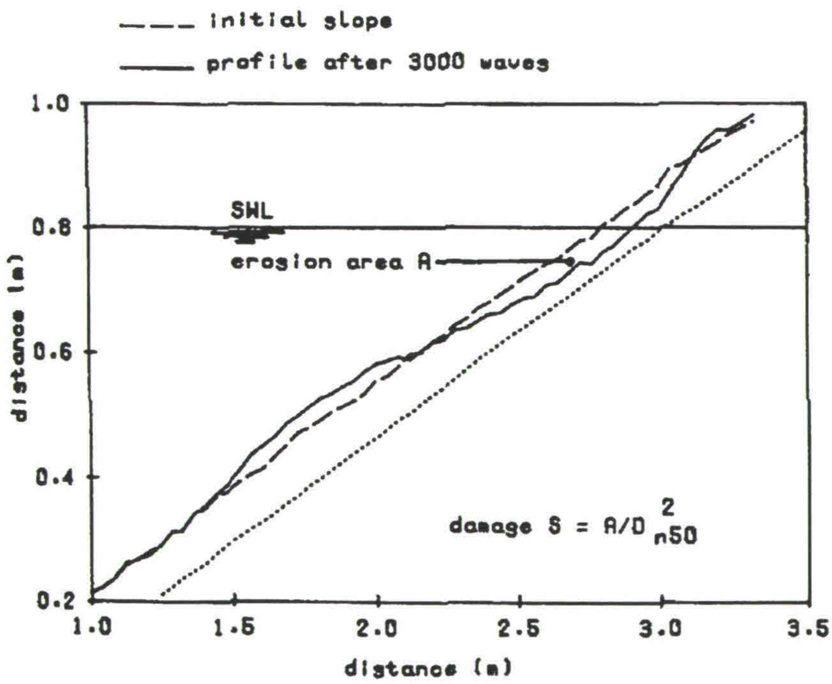


Figure 7-~~6~~ 7, Damage(S) based on ~~ersion~~ E ~~a~~ A rea (A)

The erosion in the area A is partly caused by settlement of the rock profile, partly by removal of stones that lost stability
Since the erosion area is divided by the area of the armour stone, the damage S represents the number of stones removed from the cross section, at least when porosity and shape are not taken into account. In practice, the actual number of stones removed form a strip with a width of Dn50 is between 0.7 and 1.0 times S.

If the armour layer consists of two layers of armour units, one can define limits for acceptable damage and failure. These limits are more liberal for flatter slopes, since in that case, the damage is distributed over a larger area. Critical values for S are given in *Table 7-5, Classification of damage levels S for quarry stone*.

Slope	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4	3	8 – 12	17
1:6	3	8 – 12	17

Table 7-5, Classification of damage levels S for quarry stone

At a later stage, the definition of damage has been adapted slightly. A value N is defined, which is the number of units displaced from one strip of the breakwater with a width of D_{n50}. The relation with S is established via the porosity. When the number of displaced units is counted, the settlement of the mound is omitted from the considerations of damage. The number N is often used when studying stability of armour layers consisting of concrete units.

Van der Meer choose to express the stability in terms of $H_s/\Delta D_{n50}$, and then investigated the influence of several parameters that he considered relevant. We discuss these parameters briefly.

Wave period
Van der Meer assumed the effect of the wave period to be connected with the shape and intensity of breaking waves. He therefor used the Iribarren parameter

$$\xi = \tan\alpha / \sqrt{s} \qquad (7.9)$$

in which

$$s = 2\pi H/gT^2 \qquad (7.10)$$

Using the characteristic values for irregular waves at deep water H_s and T_p or T_m, this leads to the use of ξ_{s0p} and ξ_{s0m} respectively.

It must be noted that the value of H_s in the expression $H_s/\Delta D$ is measured at the location of the toe of the structure, after elimination of any wave reflection.

Contrary to Hudson and Iribarren, Van der Meer found a clear influence of the storm duration. The longer the storm, the more damage. This can easily be explained by the model technique. Hudson and Iribarren used regular waves. A longer duration of the test series did not change the wave attack on the structure. In an irregular wave field, a longer storm duration leads to a higher probability of occurrence of extremely high waves. Apparently these extremely high waves are responsible for an ongoing damage.

Van der Meer finds further a certain influence of the permeability of the breakwater structure as a whole. He expresses this permeability in a factor P, for which he indicates values based on a global impression of the stone size in subsequent layers *(Figure 7-7, Permeability* *coëfficients for various structures*).

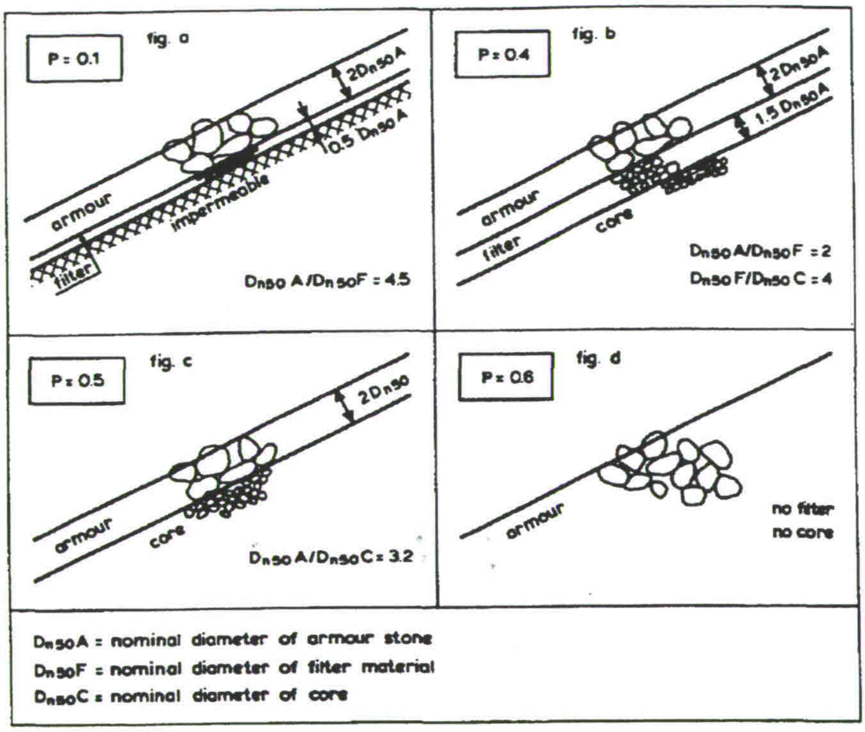


Figure 7.7, Permeability coëfficients for various structures

6

7.3.2 Quarry Stone

After extensive curve fitting, van der Meer concludes than that for quarry stone, the stability is ruled by:

For plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(S \Big/ \sqrt{N} \right)^{0.2} 1 \Big/ \sqrt{\xi_m}$$

For surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(S \Big/ \sqrt{N} \right)^{0.2} \sqrt{\cot\alpha}\,.\xi_m^P$$

The transition between plunging and surging waves can be derived by intersecting the two stability curves, which yields:

$$\xi_{mcrit} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} \quad \text{...(7.12)}$$

Depending on slope and permeability, the transition lays between ξ_{s0m} =2.5 and 4.
The reliability of the formula can be expressed by giving the relative standard deviation σ/μ (in percent) for the coefficients 6.2 and 1.0. These relative standard deviations are respectively 6.5% and 8%, as compared to a reliability of the Hudson formula of 18%.

7.3.3 Concrete Blocks

When testing armour layers of artificial material, like concrete, it makes no sense to vary the slope of the breakwater. Since the block weight is not so strictly limited as for quarry stone (the quarry has a clear maximum block size), it is much more effective to increase the block weight than to flatten the slope. This makes using the Iribarren number ξ in a formula less realistic, since this expresses the influence of both, wavelength or period and slope.

Since the mechanical strength of the concrete blocks may play a role, it is useful to distinguish damage due to actually displaced units (their number is indicated by N_{od}, and damage due to blocks that might break because they are rocking against each other (their number is indicated by N_{or}).
The total number of moving units, $N_{omov} = N_{od} + N_{or}$.
The value of N_{od} is compatible with the value of S, apart from the influence of porosity and settlement. S is about double the value of N_{od}.

For various frequently used blocks, Van der Meer gives the stability[14]. He makes a distinction between displaced blocks and moving blocks. The difference appears to be a reduction of the stability number with 0.5. The scatter of data for cubes and Tetrapods is normally distributed with a relative standard deviation $\sigma/\mu = 0.1$.

Note that D_n is the nominal diameter of the unit, or the cubic root of the volume. For various blocks this leads to:

Cubes	D_n = equal to the side of the cube
Tetrapods	D_n = 0.65 D id D is the height of the unit
Dolos	D_n = 0.54 D id D is the height of the unit (waist ratio 0.32)
Accropod	D_n = 0.7 D id D is the height of the unit

Similar to the damage levels for quarry stone, damage levels can also be classified for concrete units as in *Table 7-6, Classification of damage levels N_{od} and N_{omov} for quarry stone.*

Block Type	Slope	Relevant N-value	Start of Damage	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
Cube	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod < 25 ton	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod >25 ton	1:1.5	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos < 20 ton	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos > 20 ton	1:1.5	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Accropod	1:1.33					> 0.5

Table 7-6, Classification of damage levels N_{od} and N_{omov} for quarry Stone

Cubes

$$\frac{H_s}{\Delta D} = \left(6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \qquad \frac{H_s}{\Delta D} = \left(6.7\frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} - 0.5 \quad \text{....(7.13)}$$

Tetrapods

$$\frac{H_s}{\Delta D} = \left(3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} \qquad \frac{H_s}{\Delta D} = \left(3.75\frac{N_{omov}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} - 0.5 \quad \text{....(7.14)}$$

Dolos

The stability of Dolos was investigated by Holtzhausen and Zwamborn (1992)[15], with the following result:

$$N_{od} = 6250\left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{5.26} s_{op}^{3} w_r^{20 s_{op}^{0.45}} + E \qquad (7.15)$$

in which

w_r	= Waist ratio of the dolos
E	= Error term

The waist ratio has been made a variable in the dolos design to enable the choice of a less slender shape with less chance of breaking. Waist ratios are between 0.33 and 0.4. The error term E represents the reliability of the formula. It is normally distributed with a mean value equal to zero, and a standard deviation σ(E):

$$\sigma(E) = 0.01936\left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{3.32} \qquad (7.16)$$

Accropode

The Accropode unit is applied in a single layer at a slope 1:1.33, according to the recommendations of SOGREAH. The recommended placing method is given in Annex 3, on the basis of documents provided by Sogreah.

Van der Meer finds no influence of storm duration and wave period for these units. Instead, he defines:

Start of damage, $N_{od} = 0$ *at*

$$\frac{H_s}{\Delta D_n} = 3.7 \qquad (7.17)$$

Failure, $N_{od} > 0.5$ at

$$\frac{H_s}{\Delta D_n} = 4.1 \qquad (7.18)$$

The values 3.7 and 4.1 may be considered as stochastic variables with a standard deviation of 0.2.
It is clear that failure occurs at a wave height that is only slightly higher that the wave height which is connected to "start of damage". In this way, a built-in safety coefficient that applies to all rubble mound breakwaters is not valid for the single layer Accropod. Van der Meer recommends therefore to include a safety coefficient and to use as a design value:

Design:

$$\frac{H_s}{\Delta D_n} = 2.5 \qquad (7.19)$$

7.4 Special Subjects

7.4.1 General

With respect to the stability of stone on a slope under wave attack, the basic elements have been discussed in the previous paragraphs. There are, however still some subjects that need further attention in order to fine-tune a structural design.
These subjects will be treated here in the subsequent sections.

7.4.2 Shallow water conditions

It has been indicated in chapter 5 that in shallow water, when waves are breaking, the Rayleigh distribution is no longer valid. In deep water, the wave height exceeded by 2% of the waves is about 40% larger than the significant wave height: $H_{2\%}$ = 1.4 H_s. In shallow water, this ration may drop down to a value between 1.2 and 1.3.

When considering the effect of shallow water depth on the stability of armour units on a slope, one can assume that the highest waves yield the most important contribution to the damage. One could therefore re-write the Van der Meer formulae for quarry stone by using $H_{2\%}$ instead of H_s. The formulae then become:

For plunging waves: (7.20) For surging waves: (7.21)

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7P^{0.18}\left(S/\sqrt{N} \right)^{0.2} 1/\sqrt{\xi_m} \qquad \frac{H_{2\%}}{\Delta D_{n50}} = 1.4P^{-0.13}\left(S/\sqrt{N} \right)^{0.2} \sqrt{\cot\alpha}.\xi_m^P$$

Note that ξ is not changing, since the wave steepness is always measured at deep water!

If one is sure that $H_{2\%}$ is reduced due to breaking, it is possible to account for this fact by using the re-written formulae and substitute the actual value of $H_{2\%}$. It is stressed, however that even if extensive breaking has been observed at the location of the breakwater, the conditions may be different during the design storm as a result of higher water levels (storm surge) or a lower bed level (erosion and scour).

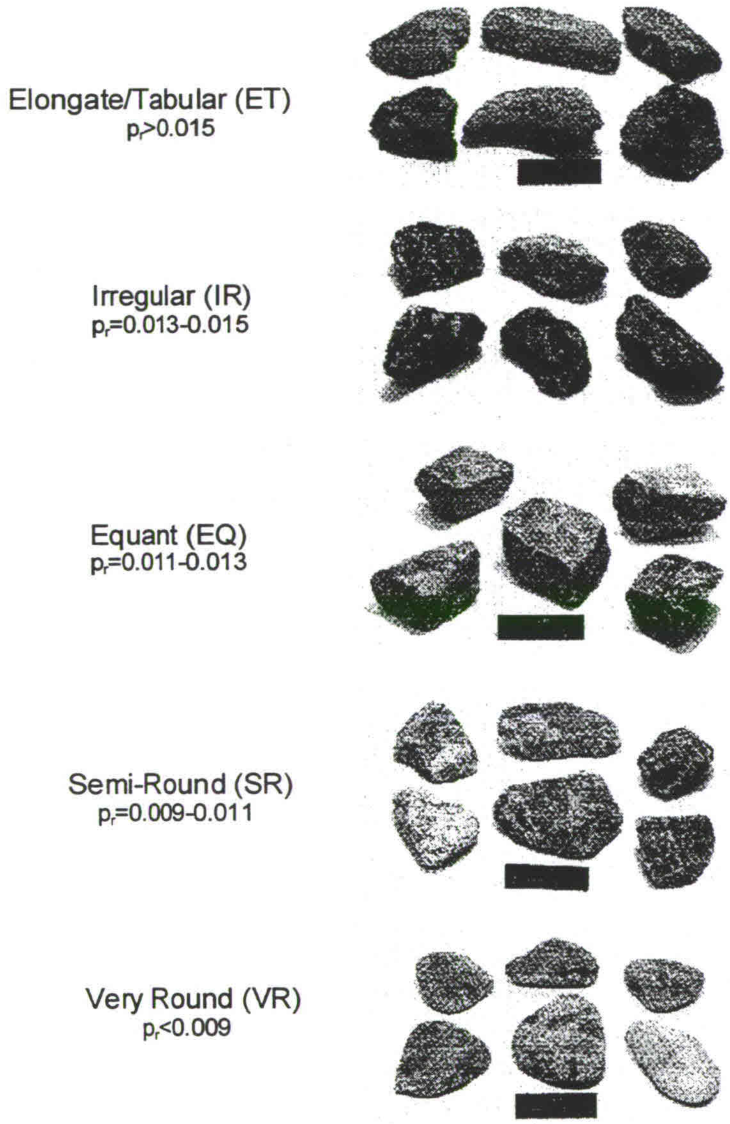
7.4.3 Shape of Quarry Stone

Hudson indicated already by varying values of K_D, that the angularity of quarry stone has an influence on stability. Latham et al. (1988)[16] investigated the influence of the shape of individual stones on their stability. They used designations like "fresh", "equant", "semi-round", "very round", and "tabular". As compared with "standard" quarry stone, the coefficient in the van der Meer formula changes slightly as shown in *Table 7-7, Effect of stone shape on stability.*

Rock shape	Plunging waves	Surging waves
Elongate/Tabular	6.59	1.28
Irregular	6.38	1.16
Equant	6.24	1.08
Standard v.d.Meer	*6.2*	*1.0*
Semi-round	6.10	1.00
Very round	5.75	0.80

Table 7-7, Effect of Stone Shape on Stability

For a visual impression of block shapes, one is referred to *Figure 7-8, Visual comparison of Block shapes (from CUR/CIRIA Manual[1])*.



meest duidelijk !

7-7

Figure 7-8, Visual Comparison of Block Shapes (from CUR/CIRIA Manual[1])

Similar investigations have been carried out by G.Burger[17] in his master's thesis. He indicates a relation between stability and l/d ratio of the quarry material.

Tabellen : ook in standaard format.

7.4.4 Grading of Quarry Stone

When quarry stone is purchased from a commercial block stone quarry, gradation is usually according to national standards

Converting mass into diameter is done on the basis of the well-known D_n (nominal diameter) method:

$$D_n = \sqrt[3]{M/\rho} \quad \text{.......................(7.21)}$$ 2

In the Netherlands, such standardised grading are:

Mass	D_n
10-60 kg	0.16-0.30 m
10-200 kg	0.16-0.43 m
60-300 kg	0.30-0.49 m
300-1000 kg	0.49-0.72 m
1000-3000 kg	0.72-1.04 m
3000-6000 kg	1.04-1.31 m
6000-10 000 kg	1.31-1.55 m

If the stone is classified according to sieve diameter, one can determine D_s. Although sieving is not a practical method for the larger stones, one can establish a general relation:

$$D_n = 0.8\, D_s \quad \text{.......................} \cancel{(7.22)}$$ 7.23

The grading of a stone class is often defined as D_{85}/D_{15}. Common values are:

Type of grading	D_{85}/D_{15}
Narrow	<1.5
Wide	1.5 – 2.5
Very wide (quarry run or riprap)	2.5 – 5 and more

Stability is usually investigated for normal wide grades. Very wide grades will result in slightly more damage than narrow and wide gradings. The very wide gradings can, however easily lead to demixing, so that it is difficult to effectively control the quality of stone delivered.

7.4.5 Stability of Toe

It is certainly not necessary to extend the armour layer over the full water depth right to the seabed. The Shore Protection Manual gives some rules of the thumb, indicating that the armour layer shall extend to about a wave height below still water level. The armour layer shall than be supported by a toe, for which the same Manual gives an indicative stone weight of 10% of the weight of the regular armour.

This approach is not very satisfactory, since one can imagine that the choice of a heavier stone in the toe allows a higher position of the toe, whereas a smaller stone in the toe will lead to a lower positioned toe.

Gerding[18] did useful work on this subject in his master's thesis. He investigated the relation between unit weight of toe elements, toe level, and damage (Nod). His findings were confirmed by the thesis work of Ms. Linda Docters van Leeuwen[19] who also varied the rock density ρ_r.

The final result was: volgende pagina,

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$$\frac{H_s}{\Delta D_{n50}} = \left(0.24\frac{h_t}{D_{n50}} + 1.6\right)N_{od}^{0.15} \quad \text{.................................} (\cancel{7.23})\ 24$$

Critical values for N_{od} are:

N_{od}	Character of damage
0.5	Start of Damage
1.0	Acceptable Damage
4.0	Failure

These values apply for a standard toe, with a height of 2 to 3 D and a width of 3-5 D. The validity range is further:

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

A definition sketch is given in *Figure 7-9 , Definition sketch toe stability*.

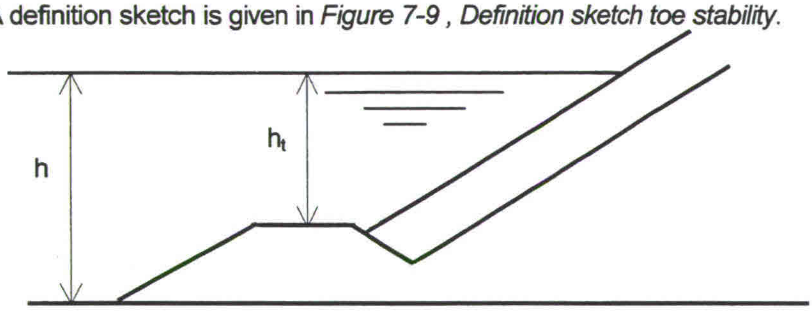

Figure 7-~~9~~, Definition sketch toe stability

7.8

7.4.6 Breakwater Head

The head of a breakwater is relatively vulnerable since the curvature causes the armour units to be less supported and/or less interlocking. In general, damage occurs on the inner quadrants, which is understandable if one looks at the 3D shape (*Figure 7-10, Typical damage Pattern Breakwaterhead*).

Therefore, the head of a breakwater is often reinforced either by using heavier armour units or by reducing the slope. An idea of the required measures can be obtained on the basis of recommended K_D values from the Shore Protection Manual.

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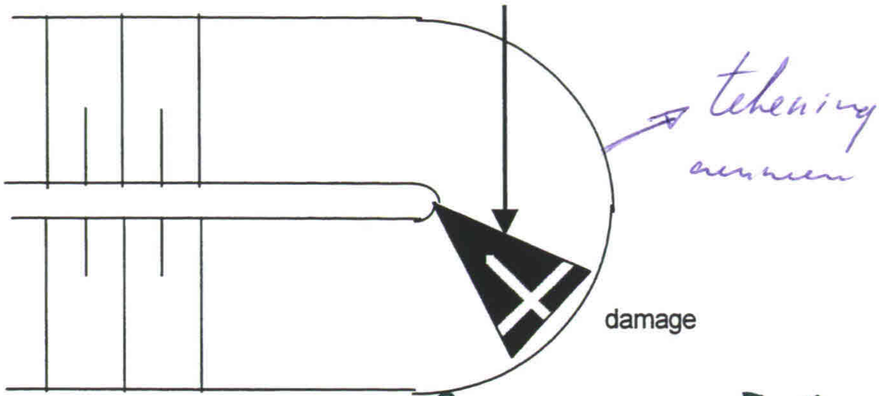

Figure 7-10, Typical Damage Pattern Breakwaterhead 7-9

Neither of the structural solutions is ideal: heavier blocks are posing construction problems; a flatter slope may cause a hazard to navigation.

7.4.7 Stability of Crest and Rear Armour

As long as the crest of the structure is so high that it prevents considerable overtopping, the armour units on the crest and the rear slope can be much smaller than the armour on the front slope. The size is often determined by waves generated in the harbour basin by wind or passing ships. Only in the vicinity of the harbour entrance, one must take into account waves penetrating through the entrance.

In many cases, however, the crest will not be that high, and in design conditions considerable overtopping will take place. In some cases, the crest will be even below still water level under such conditions.

This means a reduction in the direct attack on the front-side armour, and at the same time a more severe attack on the crest and the inner slope. In this way, there is a relation between the choice of crest level and the material on the crest and the inner slope.

There are insufficient data available in literature to indicate what is the critical crest level that requires continuation of the armour layer over the crest along the inner slope to a level that is well below still water level. This is certainly the case for submerged breakwaters, it is certainly not required if the crest level exceeds the significant run-up level. In between engineering judgement and specific model tests shall bring the answer. In a limited study for a breakwater with tetrapod armour layer, de Jong[20] concludes that the worst condition for the rear armour exists when the freeboard R_c divided by the nominal diameter is between 0 and 1. ($0<R_c/D_n<1$).

7.4.8 Stability of Low and Submerged Breakwaters

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One can discern basically two cases: a crest above still water level and a submerged crest.

Low Crest

Van der Meer derived a reduction factor for the armour size D_n. This reduction factor is:

$$\frac{1}{1.25-4.8R_p^*}\text{, for } 0<R_p^*<0.052 \qquad (7.24) \quad 25$$

With:

$$R_p^* = \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}} \qquad (7.25) \quad 26$$

Application of this formula leads to a reduction in block size up to 80% of the original value if the crest is at still water level. This is equivalent with a weight reduction of about 50%.

There is still research going on to distinguish damage for low crested breakwaters for front slope, crest and rear slope. This work is not sufficiently conclusive to be included in a textbook.

An exception is made for De Jong [20]. He gives a special stability formula for a Tetrapod armour layer on a low-crested breakwater. This formula is valid for the armour units on crest and front slope:

$$\frac{H_s}{\Delta d_n} = s_{om}^{0.2}\left\{(2.64k_\Delta + 1.25) + 8.6\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5}\right\} * \left(1 + 0.17e^{-0.61\frac{R_c}{D_n}}\right) \quad \text{........(7.26)}$$

27

Submerged Crest

When the crest is submerged, the wave attack is no longer concentrated on the slope, but rather on the crest itself. Van der meer re-analysed the data of Givler and Sorensen(1986) with the following result:

$$\frac{h_c}{h} = (2.1 + 0{,}1S)e^{-0.14N_s^*} \quad \text{..(7.27)}$$

28

in which

h$_c$ = height of the crest, measured from the bottom
h = water depth, measured from the bottom
S =damage level as defined earlier

$$N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3}$$

and

s$_p$ = local wave steepness

7.5 Future Developments

In view of the relatively large scatter in the results of the stability formulae, and because of the large influence of factors that are not present in the formulae (shape of foreshore and others) it is still good engineering practice to test a final design in a physical model. In such a model test, the structure must be exposed to wave loads higher than the design wave. Preferably, the test shall be continued till failure of the structure, so that the safety margins can be judged. Such model tests are quite labour intensive, and therefore quite costly.

To overcome this problem it is possible to continue the kind of work that van der Meer started. By generalising model investigations it must be possible to further improve and refine the design formulae. This will certainly be done.

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Another development is a change in the type of model studies. By the increasing capacity of computers it will become feasible to solve the Navier Stokes equations and calculate the water movement in front of and in rubble mound structures. At the moment this is already feasible in a 1D approach, by solving the long wave equations. This method is described by van Gent[21] in his PhD thesis. The model ODIFLOCS is available at Delft University of Technology. Van Gent has calculated the reshaping of berm breakwaters using this model.

The next step will be a 2D approach, and progress in this direction is being made. This opens the possibility of studying the behaviour of rubblemound structures with the aid of a pc or workstation.

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Alle litteraturlyster i karakter:

8. Dynamic Stability

8.1 Introduction

In chapter 7, stability of blocks on a slope was studied under the condition that the units would be stable. In principle, no movements were permitted. We have seen that this requires heavy blocks. Obtaining such heavy blocks in quarry stone is not always possible because of geological limitations of the quarry. Casting the blocks in concrete is complicated because very large units are rather sensitive to breaking.

It has therefore been studied whether it would be possible to allow slight movements of the armour stone, so that the shape of the outer slope can adapt itself to the prevailing wave conditions. It is evident that a steep slope will tend to become more gentle. To maintain the overall function of the breakwater, including the required crest level, one will have to provide extra material. That is the reason that this type of breakwater is often called Berm Breakwater, since the extra material is placed in a berm on the front side of the structure.

Application of extra material is only feasible when the cost of this material is not too high. This is the case when the quarry is not too far away from the construction site of the breakwater. Concrete units are never used in berm breakwaters, they are too costly and their sensitivity to abrasion is too large.

An added advantage of a berm breakwater is the fact that a wider gradation of material can be used. It prevents expensive sorting operations in the quarry. The wider gradation plus the fact that the maximum stone size is limited makes it easier to bring the stone demand for the design in accordance with the yield curve of the quarry. In the most simple form, the quarry yield is split into maximum three categories: filter material, core material and berm material.

Allowing the waves to reshape the outer slope eliminates the need to bring this slope under a specific angle. The contractor can dump the stone by truck and level it with a bulldozer, leaving the slope under the angle of internal repose. This again represents an important saving on construction cost. One must ascertain, however that a sufficient volume of material is used per running metre of cross section.

8.2 Seaward profiles

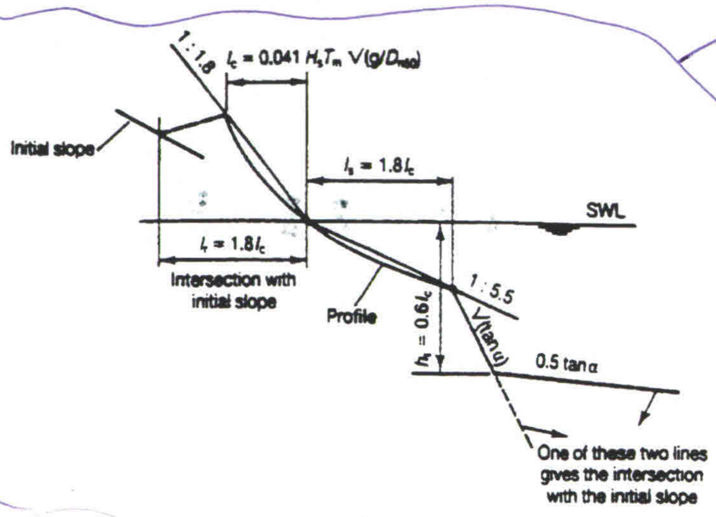

Figure 8-1, Schematised profile for sand gravel beaches

Model tests on slopes, which are not statically stable, have indicated that a typical profile is formed according to *Figure 8-1, Schematised profile for and gravel beaches (From: van der Meer[1] 1990).*

The characteristic dimensions can be expressed in terms of wave parameters:

$$l_c = 0.041 H_s T_m \sqrt{\frac{g}{D_{n50}}} \quad (8.1)$$

$$l_s = l_r = 1.8\, l_c \quad (8.2)$$

$$h_t = 0.6\, l_c \quad (8.3)$$

As can be seen from *Figure 8-1, Schematised profile for sand gravel beaches*, the intersection point of the profile with the still water level determines the position of the newly formed slope. From this point, an upper slope is drawn under 1:1.8 and a lower slope under 1:5.5. The horizontal distance l_c determines the position of the crest on the upper slope, the distance l_s determines a transition point on the lower slope. The actual slope in the zone of wave attack is a curved line through the three points. Below the (lower) transition point, a very steep slope develops under the angle of natural repose φ. If the original slope was already steep, the steep lower slope continues until the bottom. If the original slope was gentle, the steep part continues until a level h_t below SWL. From the newly formed crest, the equilibrium profile connects to the original slope at a distance l_r (the run-up length).

The position of the intersection point with SWL is not known beforehand, but can be found easily when one realises that the volume of erosion should be equal to the volume of accretion.

An example of slope development based on the formulae is shown in *Figure 8-2, Influence of wave climate on a berm breakwater profile*.

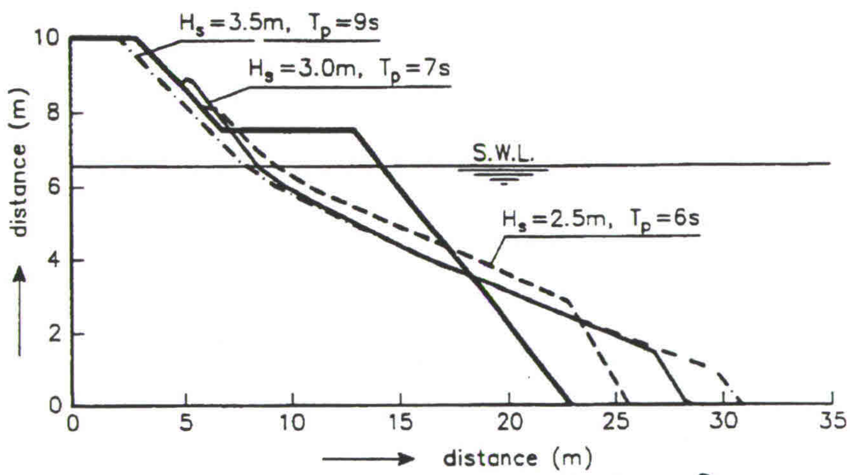

Figure 8-2, Influence of Wave Climate on a Berm Breakwater Profile

When designing a berm breakwater, the designer can by trial and error change the width and height of the berm in such a way that the core is always protected by at least a double or triple layer of armour material. The trial and error work is made easier by the use of the software package "BREAKWAT" of WL | Delft Hydraulics.

In principle, two types of initial cross sections are used, one with a berm at crest level, and one with a berm slightly above MSL. In the latter case, the berm level is chosen at such a height that trucks can safely drive over the berm.

8.3 Longshore transport of stone

When a statically stable breakwater loses armour units from the cross section, it is not relevant in which direction the units are moving since they are already accounted for as damage. When armour units of a berm breakwater are moving out of their place, it is assumed that they find another, more stable position within the same cross section. This assumption is not correct when the wave approaches the structure under an angle. In that case, the armour unit may be transported over a certain distance along the breakwater. Its place may be filled by another unit, originating from a profile a little further upstream. In the end, this process can not continue, since there will be a section which is eroded continuously. In that range, no equilibrium profile is possible anymore.

That is the reason that one should not accept considerable longshore transport along a berm breakwater. Burcharth and Frigaard[2, 3] did some research in this respect, and they state that longshore transport remains within reasonable limits if the armour size for berm breakwaters is not taken too small. They recommend the following limits:

Trunks exposed to steep waves: $$\frac{H_s}{\Delta D_{n50}} \leq 4.5 \qquad (9.4)$$

Trunks exposed to oblique waves: $$\frac{H_s}{\Delta D_{n50}} \leq 3.5 \qquad (9.5)$$

Breakwater Heads: $$\frac{H_s}{\Delta D_{n50}} \leq 3 \qquad (9.6)$$

Van der Meer has carried out similar tests and concludes that the number of stones transported per wave along the breakwater S(x) is maximum for wave angles between 15 and 35 degrees. The transport is (according to van der Meer[4]):

$$S(x) = 0 \qquad \text{for } H_oT_{op} < 105 \qquad (9.7)$$

and

$$S(x) = 0.00005(H_oT_{op} - 105)^2 \qquad (9.10)$$

in which:

$$H_o = \frac{H_s}{\Delta D_{n50}}, \text{ and } T_{op} = T_p\sqrt{\frac{g}{D_{n50}}} \qquad (9.11)$$

8.4 Crest and rear slope

One of the design principles of a berm breakwater is simplification of the cross section. Therefore, the armour on the crest and the rear slope is the same as on the front slope. Since we have seen in the previous paragraphs that $H_s/\Delta D_{n50}$ should be in the order of 3 to 3.5, this applies for the armour on the rear slope as well.

From model investigations by van der Meer and Veldman[4], it appeared that the crest level determines the damage to the inner slope, with a slight influence of the wave steepness as well.

Start of damage: $$\frac{R_c}{H_s} s_{op}^{1/3} = 0.25 \qquad (9.12)$$

Moderate damage: $$\frac{R_c}{H_s} s_{op}^{1/3} = 0.21 \qquad (9.13)$$

Severe damage: $$\frac{R_c}{H_s} s_{op}^{1/3} = 0.17 \qquad (9.14)$$

8.5 Head of berm breakwater

It is evident that wave attack on the roundhead of a berm breakwater shows similarities with wave attack by oblique waves. As stated in 9.3, it is recommended to keep the value of H/ΔD limited to less than 3.

It is further a wise measure to supply an extra buffer of armour stone at the roundhead when longshore transport is expected. The extra quantity in the buffer can either be created by increasing the height or the width of the berm. The buffer must, however, not become an obstruction to safe navigation.

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9. Stability of Monolithic Breakwaters

9.1 Introduction

The problem of the stability of monolithic breakwaters has not been solved in a satisfactory and generally accepted way yet. Research efforts are under way, but have not resulted in a generally applicable theory. Nevertheless, monolithic breakwaters are being built, and designers do use practical formulae. In this chapter, we will discuss a theoretical approach and a practical method developed in Japan. As the stability is a joint effect of wave load and subsoil resistance, some soil mechanics will be discussed as well. Next to the stability of the monolithic breakwater, also some other aspects of wave structure interaction will be discussed.

Because of the intense interest in many countries, a rapid development of the knowledge of monolithic breakwaters must be expected, compatible with the evolution around rubble mound breakwaters between 1988 and 1993. For the reader it means that always the most recent sources of literature shall be consulted next to this study book.

9.2 Wave forces and their effect

9.2.1 Quasi static forces

In Chapter 5, the formula for the pressure distribution under a wave according to the linear wave theory has been given. On the basis of this formula, Sainflou[1] (1928) developed a method to calculate pressures on a vertical wall by non-breaking waves. Rundgren[2] (1958) carried out a series of model experiments and concluded that Sainflou's method overestimates the wave force for steep waves. Rundgren then used and modified the higher order approach as proposed by Miche[3] (1944). This Miche-Rundgren method gives satisfactory results for steep waves, whereas the original Sainflou-method is best suited for long and less steep waves.

The main and important aspect of the Miche-Rundgren approach is the definition of a parameter h_0, which is a measure for the asymmetry of the standing wave around SWL. This leads to pressure diagrams as shown schematically in *Figure 9-1, Schematic Pressure Distribution for non-breaking waves.*

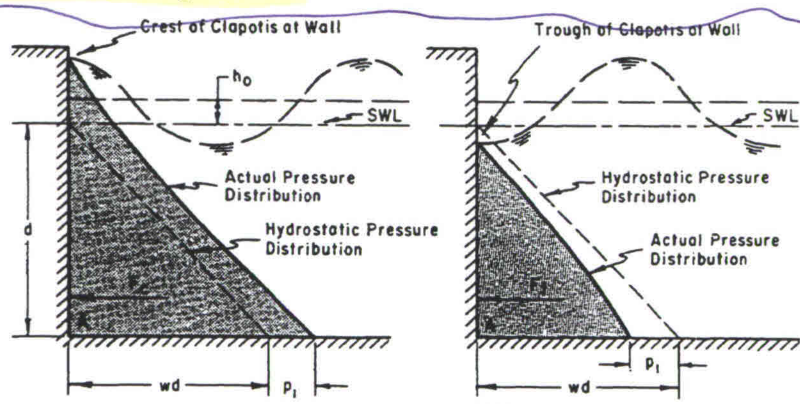

Figure 9-1, Schematic Pressure Distribution for non-breaking waves

In this figure wd and p_1 are given by:

$$wd = \rho g h \text{, and}$$

$$p_1 = \left(\frac{1+r}{2}\right)\frac{\rho g H_i}{\cosh\left(2\pi h/L\right)} \qquad (9.1)$$

The Shore Protection Manual[4] gives design graphs for the calculation of h_0 as a function of wave steepness, relative wave height (H/h) and reflection coefficient. It also gives graphs to calculate integrated pressures and resulting turning moments for crest and trough of the wave.

This leads to a relatively simple load diagram (*Figure 9-2, Load and Equilibrium Diagram*), in which the horizontal hydrostatic forces on the front and rear wall have been omitted because they eliminate each other. For stability, one must consider the resistance against translation and the resistance against rotation. It is stressed here that the resistance against rotation can not be taken simply as the sum of the moments around point A. Long before the structure starts rotating, the pressure under point A has reached a value that leads to failure of the subsoil or failure of the corner of the caisson.

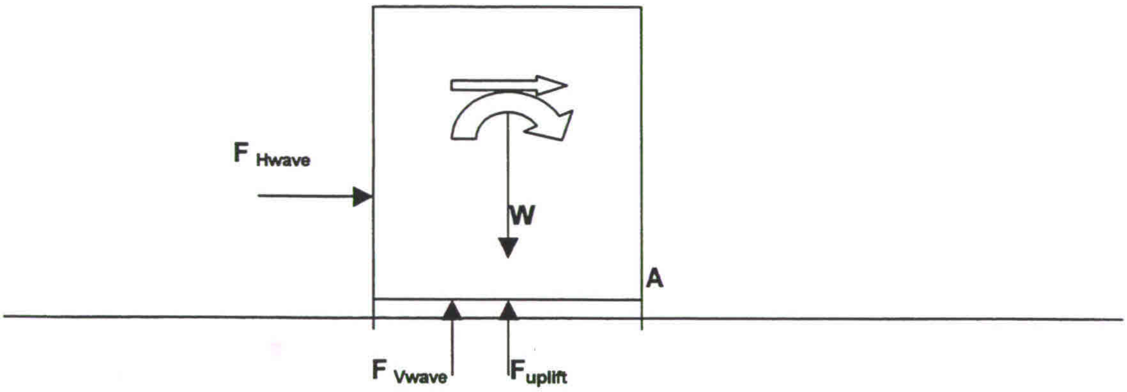

Figure 9-2, Load and Equilibrium Diagram

Since these formulae have been derived for regular monochromatic waves, it is necessary to combine them with spectral theory and arrive at a statistical distribution of wave farces and overturning moments. It can then be decided what frequency of exceedance is accepted during the lifetime of the structure. In this way, the design loads can be established.

The loads defined so far are called quasi-static forces, because they fluctuate with the wave period of several seconds and do not cause any (direct) dynamic effects.

9.2.2 Dynamic forces

In 9.2.1, we restricted ourselves to the forces by non-breaking waves. When waves are breaking, we know, however, that impact or shock pressures occur in the vicinity of the water surface. The duration of those pressures is very short, but the (local) magnitude is very large. The quasi static pressures are always in the order of ρgH, but the impact pressures can be 5 to 10 times higher. An example of a pressure record is given in *Figure 9-3, Example of pressure record*.

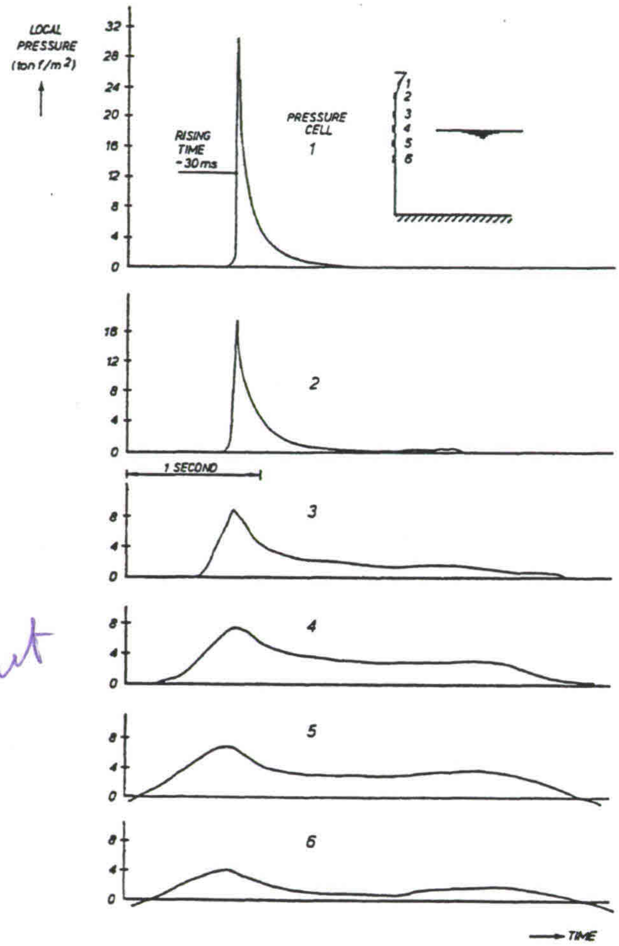

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Figure 9-3, Example of pressure record

Many researchers have studied the phenomenon in the laboratory, and none have come with a satisfactory explanation that can predict the occurrence and the magnitude of a wave impacts as a function of external parameters. Bagnold[5] was the first of those researchers. He found that the impact pressure occurs at the moment that the vertical front face of the breaking wave hits the wall, and mainly when a plunging wave entraps a cushion of air against the wall.

Apparently, the deceleration of the mass of water in the wave crest, combined with the magnifying effect of the air cushion, causes the high pressures. Two models can be used to describe and calculate this effect:

- The continuous water jet hitting a plane yields a pressure:
  $p = \frac{1}{2} \rho u^2$ (u is the water velocity in the jet)
- The water hammer effect, resulting in:
  $p = \rho uc$
  in which:
  u = the water velocity in the conduct
  c = the celerity of sound in water (1543 m/s)

The water velocity in the crest of the breaking wave is equal to the wave celerity (in shallow water: $\sqrt{gh}$)
Substitution of reasonable figures leads to a water velocity in the order of 10 m/s and impact pressures:
Continuous jet: 55 kPa (5.5 mwc)
Water hammer: 16,000 kPa (1600 mwc)

In reality, we know that the impact pressures reach values between 50 and 150 mwc.

Measurement of the impact pressures in a model is complicated because the short duration of the load requires a very stiff measuring system to provide proper data. Moreover, the compressibility of the water (influenced by entrained air) is an important factor because it determines the celerity of the compression wave in water. Uncertainties about model conditions endanger upscaling into prototype figures.

Minikin[6, 7] has given a method to calculate wave impact pressures, but his method overestimates impact pressures and does not lead to satisfactory results.

From all experiments it has become clear, however, that the duration of the wave impact is short, and the area where the impact takes place at the same time is small.

This means that the wave impact forces can not be used for a static equilibrium calculation. The dynamic effects must be taken into account, inclusive mass and acceleration of the breakwater in conjunction with its elastic foundation and the added mass of water and soil around it. Preliminary analysis has shown that it is specifically the momentum connected with the breaking wave that determines the stability or loss of stability of the breakwater. Care must also been taken of potential resonance phenomena, when the loading frequency coincides with the resonance frequency of the structure as a whole or for some individual members of the structure.

It would be a sound method of design to establish a physical relation between the impact pressure, the hydraulic parameters and the structural parameters. On the basis thereof, one should establish the exceedance curves of certain loads during the lifetime. Taking into account the response of the structure, one can then determine the probability of failure of the structure during its lifetime. Unfortunately, the physical description of wave impacts is insufficient to start this approach.

The most important lesson that can be learned from this paragraph is the uncertainty that is connected with wave impact forces as such and their effect on the stability of monolithic breakwaters. It is therefore good engineering practice to try and avoid exposure of monolithic breakwaters to breaking waves. In this context it is good to point at the fact that even if no breaking waves are expected at the location of the breakwater, they mat be induced by the breakwater itself, specifically when the monolith is place on a high mound of stone.

It can further be concluded that the risk of local impact pressures increases for structural elements that entrap breaking waves. If water can escape sideways from the impact area, the pressures remain low (compare free jet),if water can not escape, the local pressures may become quite high (compare water hammer). In this way, certain details of monolithic breakwaters are relatively vulnerable. (*Figure 9-4, Risk of local impact forces*)

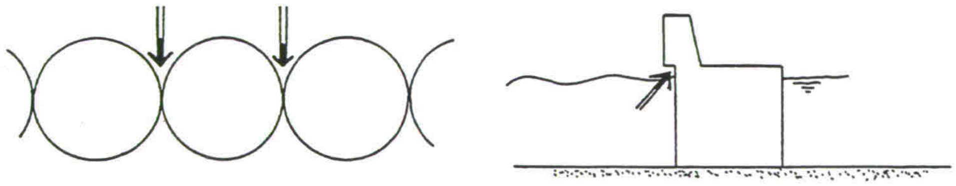

Figure 9-4, Risk of local impact forces

9.2.3 A working compromise: the Goda formula

Where the uncertainties around the design of vertical breakwaters have reduced the number of such breakwaters in Europe and the USA, in Japan, construction continued with varying satisfaction. Goda[8] analysed many of the successful and unsuccessful structures and came up with a practical formula that can be used to analyse the stability of a monolithic breakwater. From a theoretical point of view, one can object that the Goda is not consistent in his definition of design load and risk. In practice, the safety factors he proposes are apparently adequate, as long as one realises that conditions with breaking waves should be avoided as much as possible. If this is not possible, extensive model investigations are to be carried out, followed by a dynamic analysis of structure and foundation.

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Goda[9] has summarised his work in an article published in 1992 at the short course on design and reliability of coastal structures. This article is added to this book as Annex 3. Pending further theoretically based developments, the Goda formula can help to establish a first idea about stability of a monolithic breakwater.

9.2.4 Influencing the forces

It has been shown that the quasi-static forces and the dynamic forces have a trend to translate and rotate the structure, resulting in displacement of the structure and/or damage to the foundation and the bottom corners.

The effect of the external forces can be reduced by changing the direction of the horizontal force, or by spreading the force in space and in time.

The first effect is easily understood if one realises that the water pressure is always acting along the normal on a plane. When the front wall of the monolith is tilted, it means that the wave force is no longer horizontal, but directed towards the foundation. This reduces the horizontal component and strengthens the vertical component of the force. Altogether, the likelihood of sliding reduces and the overturning moment is also reduced (*Figure 9-5, Hanstholm Caisson*)

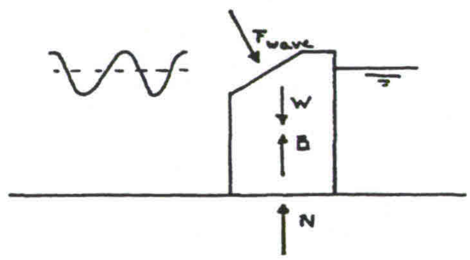

Figure 9-5, Hanstholm Caisson

Another method is the creation of a chamber in front or on top of the structure, so that the point of application of the force is spread over two walls, and a time lap is created between the two forces. This reduces the maximum instantaneous force, although the duration is elongated. Jarlan[10] first applied such idea (*Figure 9-6, Jarlan Caisson*), partly to reduce forces, partly to reduce the reflection. In Japan, a large number of similar ideas has been developed and brought into practice. In a number of cases, the idea is combined with power generation. Many of these designs have been described by Tanimoto and Takahashi[11] (*Figure 9-7, Possible cross section of semi-circular caisson breakwater for extremely high breakers* and *Figure 9-8, Cross-section of curved-slit caisson breakwater at Funakawa Port*)

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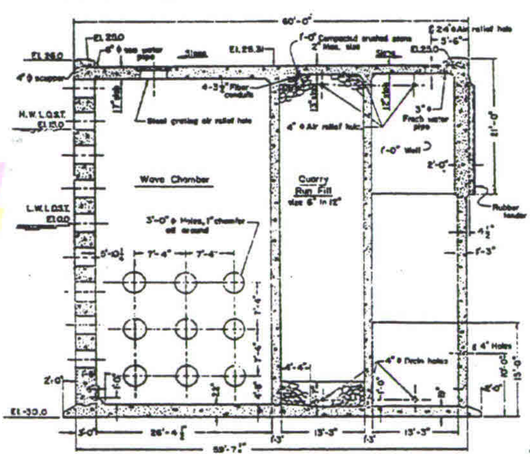

Figure 9-6, Jarlan Caisson

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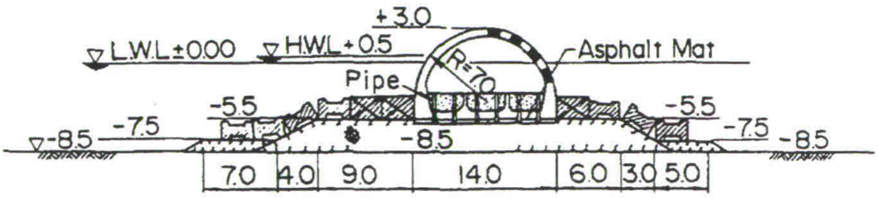

Figure 9-7, Possible cross section of semi-circular caisson breakwater for extremely high breakers

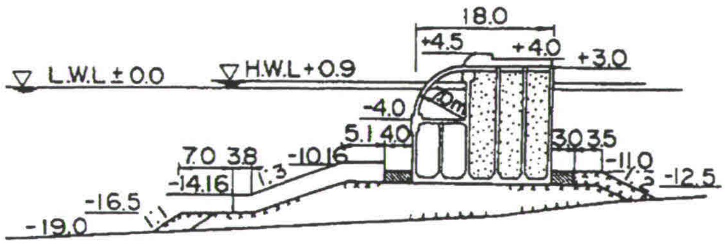

Figure 9-8, Cross section of curved-slit caisson breakwater at Funakawa Port

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9.3 Overtopping and transmission

Transmission of vertical wall breakwaters placed on a rubble berm has also been investigated by Goda[8]. He relates the relative freeboard R_c/H to the transmission coefficient and finds a slight influence of the height of the berm. (See Figure 9.9)

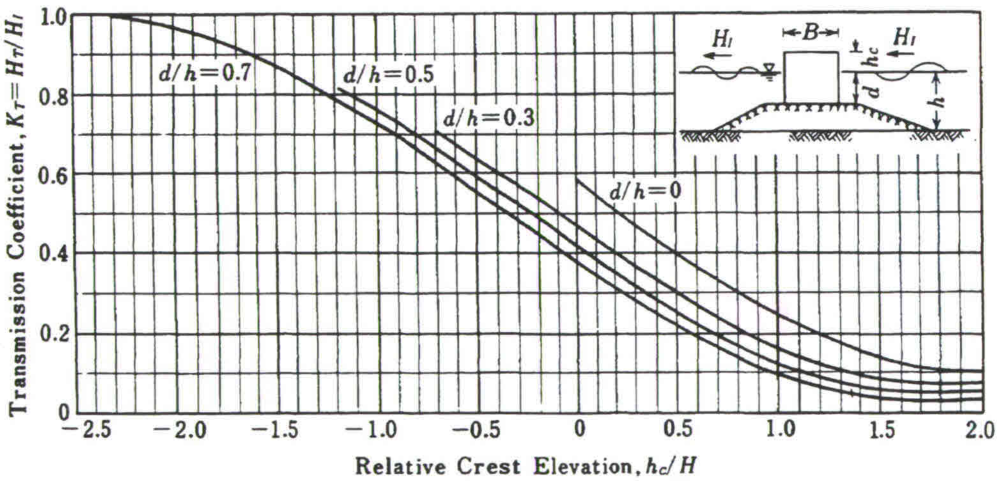

Figure 9-9, Wave transmission for a vertical breakwater

Overtopping has been investigated by Franco et al[12] (1994). They describe the unit discharge q as:

$$\frac{q}{\sqrt{gH_s^3}} = a * \exp(-b\frac{R_c}{\gamma H_s}) \qquad (9.2)$$

with
q = unit discharge ($m^3/m/s$)
a, b = experimental coefficients
γ = geometrical parameter

For a rectangular shape
a = 0.192
b = 4,3
γ = 1

It must be kept in mind that a vertical face breakwater causes a lot of spray when hit by a wave. The spray may also be blown over the breakwater. This effect is not included in the above formula.

9.4 Foundation

The hydraulic forces exerted on the caisson plus the weight determine what will be the local pressures in the interface between the caisson and the foundation. It will be clear that these pressures must not lead to (soil mechanical) failure. Because the foundation is flexible to a certain extent, it must be verified whether the mass-spring system formed by caisson (mass) and foundation (spring) gives rise to resonance phenomena. Depending on the outcome of that investigation, one may decide that a static stability analysis is sufficient (as is often the case). Soil-mechanical failure is nevertheless one of the most likely failure modes.

Even if it is decided after analysis that a quasi-static approach is justified, the cyclic effect of the load may not be overlooked. The load will anyway cause an increase of the total stress level (σ) and initiate a compression of the subsoil. In first instance this will lead to a higher stress level in the ground water (p). Depending on the permeability of the soil, the excess water will drain and gradually, the effective stress (σ') will increase. This all in accordance with one of the basic laws from soil mechanics:

$$\sigma = p + \sigma' \qquad (9.3)$$

Because of the cyclic character of the load, it is possible that drainage of excess water is not complete when the next loading cycle starts. In this way, the water pressure may gradually increase. Eventually, this will lead to a condition that the effective stress becomes very low or even zero. A low effective stress will greatly reduce the resistance against sliding; an effective stress equal to zero leads to liquefaction or the formation of quick-sand. This is the main reason that care is recommended when designing monolithic breakwaters in areas that are sensitive for liquefaction: soil consisting of fine, loosely packed sand as in the SW part of the Netherlands.

Preventive methods against liquefaction are possible, but expensive. Soil replacement and compaction of the subsoil are the most common methods.

Because of the possibility that high ground water pressures occur under the corners of the monolith, also large vertical gradients are likely. It is therefore necessary to cover a (fine) grained subsoil with an adequate filter. Because of the large gradients, it is recommended that the filter be designed as a geometrically impervious filter.

A granular foundation layer may also be required if the structure is placed on an uneven hard seabed. In that case, it is the function of the foundation layer to flatten the seabed and to avoid pressure concentrations and an unpredictable support pattern of the structure.

Alternatively, one may create pre-designed contact areas in the bottom of the structure, so that the bending moments in the floor plate can be calculated.

To create a perfect and homogenous contact plane between the foundation and the structure, sometimes a grout mortar is injected. This technique has been developed in the offshore industry for the foundation of gravity platforms, but the use has spread to regular coastal engineering projects as well. To avoid loss of grout, a skirt is provided along the circumference of the bottom of the caisson. This skirt (mostly a steel sheet) penetrates into the foundation and creates a chamber that can be filled with the grout mortar.

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10. Wave – Structure Interaction (Rubble Mounds)

10.1 Introduction

Even if a rubble mound structure is stable under the action of waves, there is interaction between the structure and the wave field near the structure. We discern various phenomena that lead to a different wave pattern in the vicinity of the structure:

- Wave reflection
- Wave run-up
- Overtopping
- Wave transmission

Wave reflection plays a role in front of the structure, wave run-up takes place at the structure, overtopping and transmission are important for the area behind the structure. Before using any of the expressions given in this chapter, it is useful to analyse which phenomenon influences the design problem in question. Too often formulae for run-up or overtopping are used when the designer wants to address wave transmission, etc.

10.2 Reflection

In chapter 5 we have already seen the formula for a standing or partially standing wave. The wave motion in front of a reflecting structure is mainly determined by the reflection coefficient r.

If 100% of the incoming wave energy is reflected, one can safely assume that the reflection coefficient $K_r = H_r/H_i = 1$. This is generally valid for a rigid vertical wall of infinite height.

The reflection coefficient reduces for sloping structures, for rough or permeable structures, and for structures with a limited crest level.

Postma (1989)[1] has investigated the reflection from infinitely high rock slopes. He found a clear influence of the breaker parameter ξ, and of the permeability P as defined by van der Meer.

For a first estimate, he proposes to use a simple formula:

$$K_r = 0.140\xi_{op}^{0.73} \quad \text{......(10.1)}$$

For a more accurate approach, he gives the formula:

$$K_r = 0.081P^{-0.14}\cot\alpha^{-0.78}s_{op}^{-0.44} \quad \text{......(10.2)}$$

This formula can only be used within the validity range of the various parameters as given below:

$0.1 < P < 0.6$
$1.5 < \cot\alpha < 6$
$0.004 < s_{op} < 0.06$
$0.7 < \xi_{op} < 8$
$0.1 < K_r < 0.8$
$0.03 < h/L_{op} < 0.3$
$0.09 < H_{si}/h < 0.23$
$2 < H_{si}/D_{n50} < 6$

→ dit staat schots en scheef.

10.3 Run-up

Wave run-up is the phenomenon that an incoming wave crest runs up along the slope till a level that may be higher than the original wave crest. The vertical distance between still water level SWL and the highest point reached by the wave tongue is called the run-up "z". From the definition it is clear that we can only speak of run-up when the crest level of the structure is higher than the highest level of the run-up (*Figure 10-1, Defenition of wave run-up*). Run-up figures are mainly used to determine the probability that certain elements of the structure are reached by the waves. In an indirect way, it can be used to estimate the risk of damage to the inner slope of the structure.

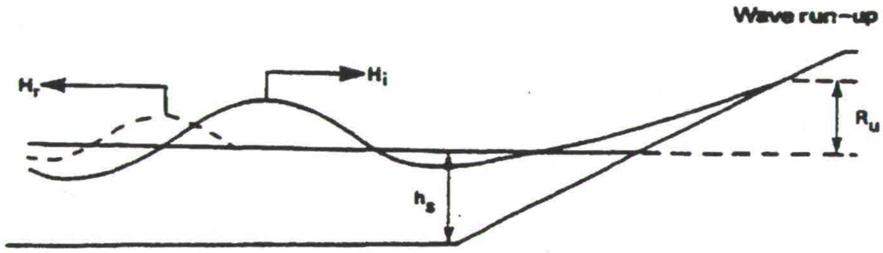

Figure 10-1, Defenition of Wave Run-up

In the Netherlands, research on run-up has always attracted a lot of attention[2, 3]. This was mainly to assess the required crest level of dikes and sea walls. Most of the data have been collected and published by the "Technische Adviescommissie voor de Waterkeringen, TAW" (Technical Committee for sea defences). Since most of the research efforts were directed to run-up on dikes with a slope protection of asphalt or stone revetment, most results are valid for smooth, impermeable cover layers.

Since the inner slopes of many dikes in the country are covered with grass, it is not acceptable that a large percentage of the incoming waves will reach the crest and subsequently cause damage to the inner slope. Therefore, in most cases, the 2% run-up is given: the run-up level that is exceeded by 2% of the incoming waves. It is assumed that the grass on the inner slope of a sea dike can withstand this condition.

The run-up on a smooth impermeable slope is then expressed as:

$$z_{2\%}/H_s = 1.6\xi_{op} \quad \text{..........} (10.3)$$

with a maximum of 3.2.

$z_{2\%}$ = run-up level exceeded by 2% of the waves
H_s = significant wave height
ξ_{op} = breaker parameter for deep water and peak period

greater!

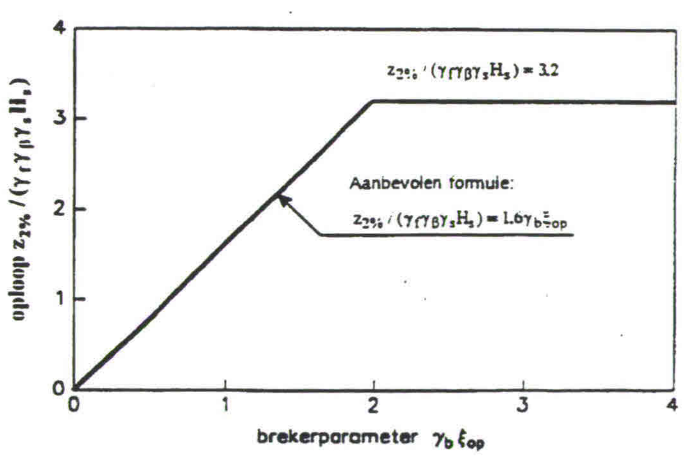

Figure 10-2, 2% Wave Run-up on a smooth Impermeable Slope

The run-up level can effectively be reduced by designing a berm at still water level, by increasing the roughness of the surface, or by increasing the permeability of the structure. Waves approaching the structure under an angle will also lead to reduced run-up levels. This reduction is expressed in terms of reduction factors γ. For the design of breakwaters, the reduction factor γ_f for the roughness is the most important one. The most effective artificial roughness is the application of ripples, which can reduce the run-up to about 65% of the original value.

Special investigations have been done for slopes covered with a double layer of quarry stone. The results can be summarised in two formulae:

$$z_{2\%}/H_s = 0.88\xi_{op} \quad \text{...(10.4)}$$

for $\xi_{op} < 1.5$, and

$$z_{2\%}/H_s = 1.1\xi_{op}^{0.46} \quad \text{...(10.5)}$$

for $\xi_{op} > 1.5$.

In *Figure 10-3, 2% wave run-up on rubble covered slopes*, these formulae are indicated, inclusive the actually observed points.

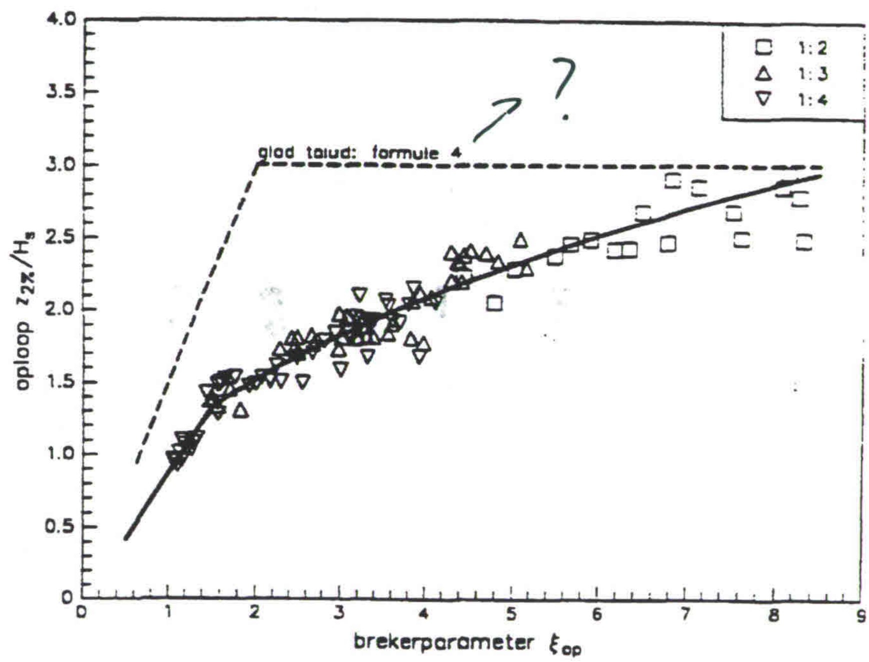

Figure 10-3, 2% wave run-up on rubble covered slopes

These data and figures have been taken from the draft technical report "Golfoploop en Golfoverslag" of TAW.

Van der Meer and Stam[4] have interpreted the (same) original model data in more detail. They propose a set of more general formulae:

$$z_{u\%}/H_s = a\xi_m \quad (10.6)$$

for $\xi_m < 1.5$ and

$$z_{u\%}/H_s = b\xi_m^c \quad (10.7)$$

for $\xi_m > 1.5$.

These formulae are valid for breakwaters with an (almost) impermeable core (P = 0.1). For structures with a permeable core (P = 0.4), the relative run-up is limited to a value:

gelykheta

$$z_{u\%}/H_s = d \quad (10.8)$$

$z_{u\%}$ = run-up level exceeded by u % of the incoming waves
a, b, c, d parameters (*Table 10-1 , run-up parameters for rubble covered and permeable slopes*)
ξ_m = breaker parameter for deep water, mean period

run-up level u (%)	a	b	c	d
0.1	1.12	1.34	0.55	2.58
1	1.01	1.24	0.48	2.15
2	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
Sign.	0.72	0.88	0.41	1.35
Mean	0.47	0.6	0.34	0.82

Table 10-1 , Run-up parameters for rubble covered and permeable slopes

No separate data are available for run-up on slopes covered with concrete armour units.

10.4 Overtopping

Overtopping is defined as the quantity of water passing over the crest of a structure per unit of time. It has therefore the same dimensions as the discharge Q [m^3/s]. Because this quantity of water is often a linear function of the length of the structure, it is expressed as a specific discharge per unit length [m^3/s/m].

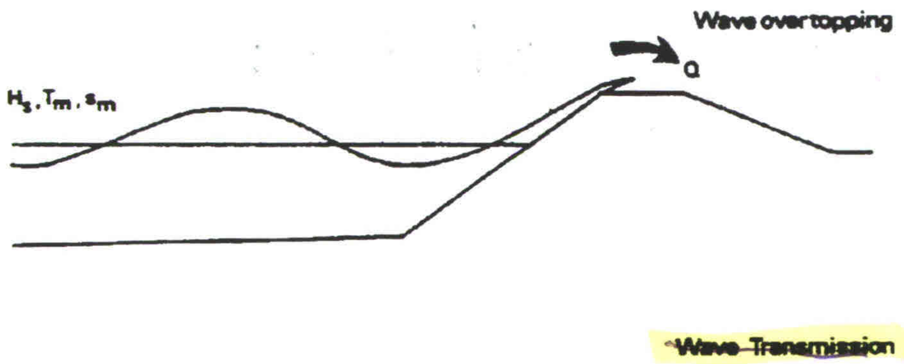

Figure 10-4, Typical wave overtopping

The study of overtopping quantities is again a subject that was initiated in relation to the stability of inner slopes of grass covered dikes. When designing breakwaters, the quantity of overtopping may be important to determine the capacity of drainage facilities for port areas directly protected by the breakwater, or to assess the risk for people or installations on the crest of the breakwater. Although the assessment of those (sometimes-subjective) risks on the basis of model experiments is difficult, it is possible to derive a trend. Earlier investigations from Japan and Italy[5] have been collected and interpreted by van der Meer[6]. These results are given in Table *10-2, Overtopping discharges and their effects.*

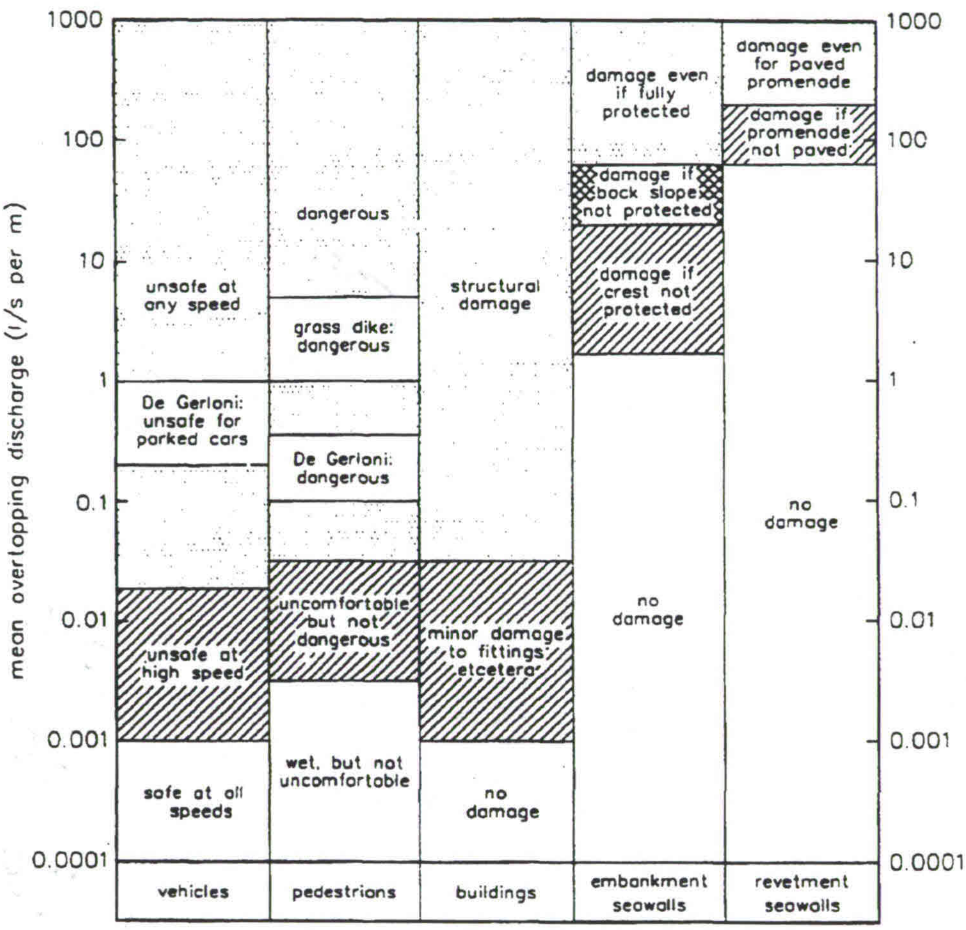

Table 10-2, Overtopping discharges and their effects

Similar to the conditions governing wave run-up, overtopping is also largely influenced by the nature of the outer slope of the structure. An added effect is the influence of the shape and nature of the crest (presence of a crown wall). Unfortunately, various model investigations[7, 4, 8, 9, 10] have been carried out with varying structures. It is not well possible to derive a generally applicable formula for overtopping. The reader is advised to study the available literature in detail and to select the most promising approach for his specific problem. For a first approximation, one may use the formula of Bradbury et al (1988), valid for a structure without crown wall, and similar to *Figure 10-4, Typical wave overtopping*:

$$Q = a\,R^{-b} \qquad (10.9)$$

with:

$$Q = \frac{Q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{om}}{2\pi}} \qquad (10.10)$$

and

$$R = \left(\frac{R_c}{H_s}\right)^2 \sqrt{\frac{s_{om}}{2\pi}} \qquad (10.11)$$

in which:

R_c = Crest freeboard relative to SWL
H_s = Significant wave height
s_{om} = deep water wave steepness, based on mean period
Q = specific discharge in $m^3/s/m$

3

Relevant values of the coefficients a and b depend on the structural details as given in *Figure 10.5*. The values for a and b are given in *Table 10-2, Overtopping coefficients*.

Type of structure	A	B
Section A	$3.7 * 10^{-10}$	2.92
Section B	$1.3 * 10^{-9}$	3.82

Table 10-3, Overtopping coefficients

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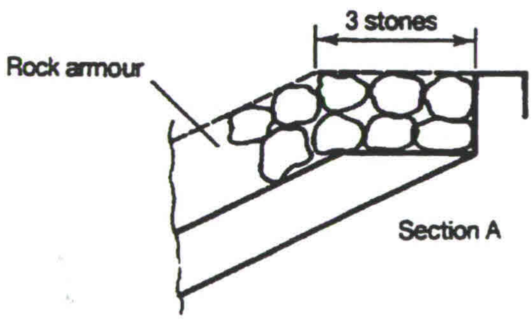

Figure 10-5, Typical cross section for overtopping (no crown wall)

Low volgens verdelen !

10.5 Transmission

Wave transmission is the phenomenon that wave energy passing over and through a breakwater creates a (reduced) wave action in the lee of the structure (*Figure 10-6, Typical wave transmission*). This will certainly happen when considerable amounts of water are overtopping the structure. Wave transmission is also possible, however, when the core of the structure is very permeable and the wave period is relatively long. It is specifically the influence of these two factors that has for a long time prevented the derivation of an acceptable formula for wave transmission.

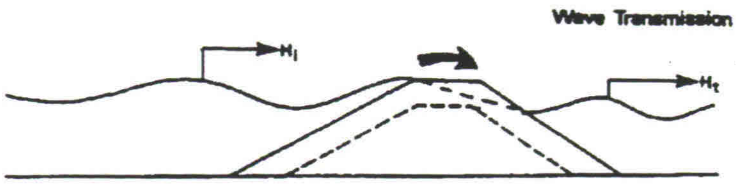

Figure 10-6, Typical wave transmission

groter

The effects of wave transmission has been investigated by many authors[11, 12, 13, 14]. This resulted in a diagram presented in *Figure 10.7, Wave transmission for low crested structures*.
It must be noted that the transmission coefficient can never be smaller than 0 or larger than 1. In practice, limits of about 0.1 and 0.9 are found (*Figure 10-7, Wave transmission for low-crested structures*).

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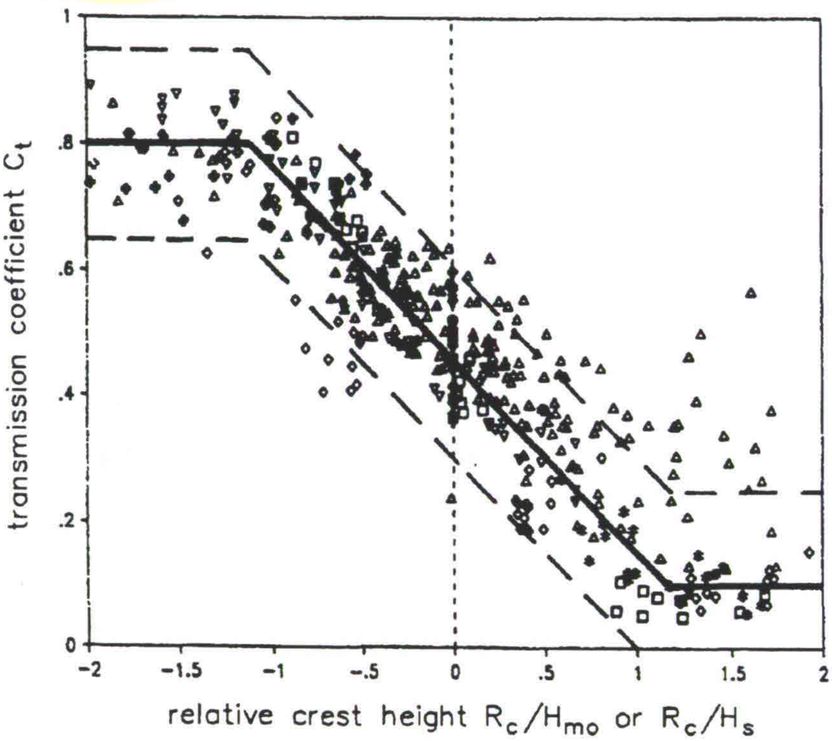

Figure 10-7, Wave transmission for low-crested structures

It is remarkable that for R_c=0, which represents a structure with the crest at SWL, the transmission coefficient is in the order of 0.5. This means that a relatively low structure is already rather effective in protecting the harbour area behind the breakwater. In combination with the requirements for tranquillity in the harbour, the designer can decide on the minimum required crest level.

Eventually, Daemen[15, 16] in his MSc thesis has been able to produce an acceptable formula that relates the transmission coefficient to a number of structural parameters of the breakwater. To account for the effect of permeability, Daemen has decided to make the freeboard R_c of the breakwater dimensionless dividing it by the armour stone diameter. This eliminates a lot of the scatter that was present in previous approaches. The Daemen formula reads (for traditional low crested breakwaters) as follows:

$$K_t = a\frac{R_c}{D_{n50}} + b \qquad (10.12)$$

with:

$$a = 0.031\frac{H_i}{D_{n50}} - 0.24 \quad (10.13)$$

$$b = -5.42 s_{op} + 0.0323\frac{H_i}{D_{n50}} - 0.0017\left(\frac{B}{D_{n50}}\right)^{1.84} + 0.51 \quad (10.14)$$

K_t = H_{st}/H_{si} = transmission coefficient
H_{si} = incoming significant wave height
H_{st} = transmitted significant wave height
R_c = crest freeboard relative to SWL
D_{n50} = nominal diameter armour stone
B = crest width
s_{op} = wave steepness

Use of the Daemen formula is complicated in case it is decided to use a solid crown block, or to grout armour stones with asphalt into a solid mass. Therefore, another MSc student, R.J. de Jong[17, 18], reanalysed the data and came up with a different expression. He choose to make the freeboard dimensionless in relation with the incoming wave height:

$$K_t = a\frac{R_c}{H_{si}} + b \quad (10.15)$$

with

a = 0.4, and

$$b = 0.64\left(\frac{B}{H_{si}}\right)^{-0.31} * \left(1 - e^{-0.5\xi}\right) \quad (10.16)$$

The factor 0.64 is valid for permeable structures; it changes into 0.80 for impermeable structures.

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11. Design Practice of Cross Sections

11.1 Introduction

In the foregoing chapters we have used terms as armour layer and core, we have discussed the stability of concrete armour units, and we have seen the principles of a berm breakwater. It is now time to discuss some practical rules for the design of cross sections. Many of these rules are just rules of the thumb, and only gradually an experimental basis is created to accept or refuse these rules. The present chapter is thus a mix of research, experience and plain engineering judgement.

It must be kept in mind that natural rock is mostly obtained by blasting, and that the size of the stone obtained (yield curve) can only be influenced to a limited extent. It is much easier to increase the percentage of fine material than the percentage of coarse material. Any material that is blasted must be handled in the quarry, whether it is used in the breakwater or not. If it is not used, it must be left somewhere, and in many countries deposition of waste material requires special licenses that are difficult to obtain. It is therefore an element of good design to try and use all materials produced by the quarry.

In principle, we can follow two solutions. The **first** one is to split the quarry production in two or three categories (filter material, core and armour). This leads almost automatically to a berm breakwater. The gradation of the stone categories is rather wide, (D_{n85}/D_{n15} up to 2.5 or even 3). The **second** solution includes classification of the quarry output into a larger number of categories, each with a narrower gradation (D_{n85}/D_{n15} up to 1.5). In this way it is possible to select the proper stone size for a specific function. It leads to a more economic use of material. The berm breakwater with its larger volume is in the advantage when the production cost is low and the quarry is located near the site of the breakwater. When quarry stone is more costly and the quarry is at great distance, it is more economic to build a multi-layered breakwater. Also in this case, however, it is an advantage to keep the design of the cross section as simple as possible.

11.2 Porosity and Layer thickness

11.2.1 Porosity

When rock or concrete blocks are placed in the cross sections of a breakwater, it is important to have an idea about the porosity and the layer thickness. The porosity is important because it determines at least part of the hydraulic response of the structure, and it influences the stability (P in the van der Meer formula). During construction it is important because the porosity determines the bulk density. Quarry stone is often paid for per ton of material to the quarry operator. When the contractor is paid per m^3 for placing the material in the cross section, as is often the case, it is essential for a proper cost estimate to know the bulk density. The volumetric porosity n_v is defined as follows:

$$n_v = 1 - \left(\rho_b / \rho_r \right) \qquad (11.1)$$

in which

ρ_b = bulk density as laid

ρ_r = density of rock

Determination of the bulk density is not simple because of the errors made at the boundaries of the measured volume. Preliminary data can be found in the Shore protection Manual[1], the CUR/CIRIA Manual[2], or in the more recent MSc thesis of Bregman[3].

Table 11-1, Thickness and porosity in narrow gradation armour layers (with data from Shore Protection Manual and the CUR/CIRIA Manual) indicates porosity levels between 35 and 40% for placed quarry stone in thin layers. Bulk handling may lead to a porosity that is up to 5% higher than the values in *Table 11-1, Thickness and porosity in narrow gradation armour*

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layers. A wider gradation, however, may lead to lower porosity. Because of the uncertainties in the determination of the porosity and the bulk density it is recommended to carry out some in situ tests to ascertain the actual values. It is emphasised that special placement of quarry stone (with the longest dimension either parallel or perpendicular to the slope) has a large effect on layer thickness, but also on stability.

In *Table 11-1, Thickness and porosity in narrow gradation armour layers*, also the porosity of concrete units is given. Here again, the method of placement may cause large differences in porosity and in stability.

Type and shape of units	Layer thickness n	Placement	Layer coefficient k_t	Porosity N_v	Source
Smooth quarrystone	2	random	1.02	0.38	SPM
Very round quarrystone		random	0.80	0.36	Cur/Ciria
Very round quarrystone		special	1.05 – 1.20	0.35	Cur/Ciria
Semi-round quarrystone		random	0.75	0.37	Cur/Ciria
Semi-round quarrystone		special	1.10 – 1.25	0.36	Cur/Ciria
Rough quarrystone	2	random	1.00	0.37	SPM
Rough quarrystone	>3	random	1.00	0.40	SPM
Irregular quarrystone		random	0.75	0.40	Cur/Ciria
Irregular quarrystone		special	1.05 – 1.20	0.39	Cur/Ciria
Graded quarrystone		random	-	0.37	SPM
Cubes	2	random	1.10	0.47	SPM
Tetrapods	2	random	1.04	0.50	SPM
Dolosse	2	random	0.94	0.56	SPM
Accropode	2				Sogreah
Akmon	2			?	WL

Table 11-1, Thickness and porosity in narrow gradation armour layers

11.2.2 Layer thickness and number of units

For armour layers it is important to know what the effective layer thickness is for a single, double or triple layer of material. This is essential in designing and constructing a cross section. The crest level is determined on the basis of the required protection at the lee side of the breakwater. Given this required crest level, one must know at what level the core shall be finished. If the crest of the core is too high, it means that too much of material has been used, which will probably not be paid for by the client. If the crest of the core is too low, and the client still wants the given crest level, it means that extra armour stone has to be used. Since armour stone is generally more expensive than core material, this again will cause a financial loss to the contractor.

The effective layer thickness is discussed extensively in the Shore Protection Manual(1984). The thickness of a layer t (in m) is calculated as:

$$t = n * k_t D_{n50} \qquad (11.2)$$

in which n is the number of stones across the layer.
In the American literature, k_Δ is often used instead of k_t, and r instead of t.

Values for k_t are also given in *Table 11-1, Thickness and porosity in narrow gradation armour layers*.

It is also important to know the number of armour units N required to cover a certain area A. This number is:

$$N = nk_t(1-n_v)D_{n50}^{-2} \quad (11.3)$$

Although *Table 11-1, Thickness and porosity in narrow gradation armour layers* gives values of k_t and n_v for concrete armour units as well, it is emphasised that those values can fluctuate considerably. This depends on the interpretation of the qualification "random placement". It is possible to place cubes much denser than indicated in the Table. This will improve the stability. Therefore, care is required when data from inexperienced researchers are used.

11.3 Berm breakwater

The cross section of a berm breakwater consists in principle of two materials, core material and armour material. The armour material is the coarsest fraction of the quarry yield, the core material is the finer fraction. The armour material is located in a berm along the outer slope of the breakwater. The quantity of armour material is chosen in such a way that after a series of storms envisaged during the lifetime of the breakwater, the core is always covered with at least a double layer of armour material. This applies to the front slope, the crest, and the exposed part of the inner slope, i.e. to a little below low water level.

Crest level
It has been discussed in chapter 8 (dynamic stability) that the crest level determines the stability of the inner slope. Given a reasonable ratio between wave height and nominal stone size, the minimum crest level follows from the accepted level of damage. When designing the cross section. It is good engineering practice to create a safe level where trucks, bulldozers and cranes can work without much interference from waves.

Filter layer
At the front slope, the incident and reflected waves create a complicated pattern of orbital velocities and pressure fluctuation. This will cause larger stones to sink slowly into the seabed, unless the latter consists of rock. The same hydraulic conditions will enhance the risk of scour and erosion of the seabed just in front of the breakwater. Eventually, both phenomena together will lead to loss of material and potentially to loss of stability of the entire breakwater. It is therefore good engineering practice to provide a filter layer under the toe of the breakwater, and to extend this filter over some distance in front of the toe to prevent forming of a scour hole close to the structure. In case of a berm breakwater it is wise to apply such filter also in the area that will be covered by armour stone after reshaping of the seaward slope.

Protection of the seabed may consist of a granular filter, or a combination of a filter mattress and a cover of quarry stone. Filter rules are beyond the scope of this book; the reader is referred to the CUR Manual[4] on the Use of rock in hydraulic engineering or to Schiereck (1998)[5].

Slopes
Since wave action will reshape the profile anyway, it makes no sense to bring the cross section under a particular slope. It is generally accepted to leave the material under the natural angle as it is deposited by barge, dump truck and/or bulldozer.

This leads to two frequently used basic cross sections, one with the berm at crest level, one with the berm just above MSL. These two cross sections are presented in *Figure 11-1, Berm breakwater with high berm* and *Figure 11-2, Berm breakwater with berm at MSL.*

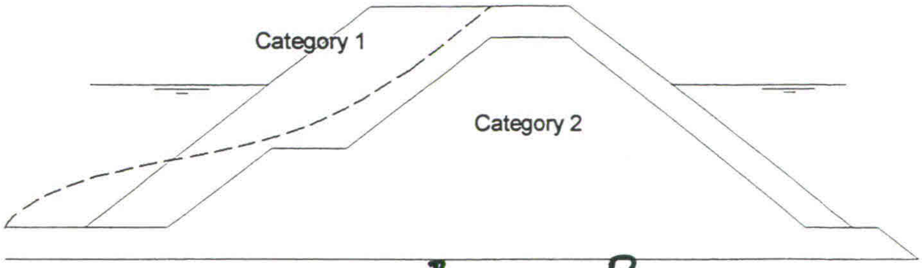

Figure 11-1, Berm breakwater with high berm

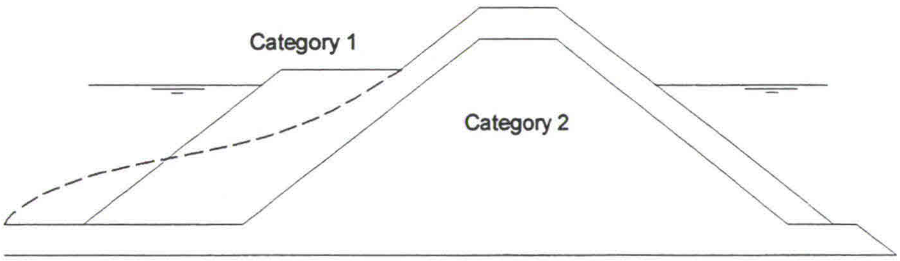

Figure 11-2, Berm breakwater with berm at MSL

11.4 Traditional multi layer breakwater

11.4.1 Classification

Although there are some standard cross sections for traditional multi-layered breakwaters, numerous variations can be made, which makes classification quite difficult. The first, and most logical classification is done by crest level.

High / low crest
In this context, a high crest is considered to be so high that the inner slope is not severely attacked by the action of overtopping waves. The inner slope is designed for waves by passing ships, or waves locally generated in the harbour basin. A low crest is that low that the inner slope is severely attacked by waves passing over the crest. To protect the inner slope, generally the same armour is used on the inner slope as on the seaward slope.

Crest design
A second type of classification is possible with respect to the nature of the breakwater crest. This can consist of armour units or of a solid cap block. If the crest consists of armour units, the crest is mostly not accessible, neither for human beings, nor for equipment. It means that maintenance is only possible by using floating equipment. If the crest is formed by a solid block, it is common practice to design it in such a way that it can be used as a road, both during construction and subsequently for maintenance or other purposes.

Rock or concrete armour units
A third type of classification is possible on the basis of the type of armour material. Since the maximum size of quarry stone is limited, it is not uncommon to reduce the seaward slope to obtain sufficient stability. (Note: the inner slope can be steeper!). In case of the use of concrete armour units, it can easily be demonstrated that a steep slope of say $1{:}1^1/_2$ ($\cot \alpha = 1.5$) leads to the most economic design.

11.4.2 General design rules

To explain the various design rules, a definition sketch of a multi-layer cross section is given in *Figure 11-3, Definition sketch of a multi-layer cross section*. It indicates the main elements of such breakwater and their respective names.

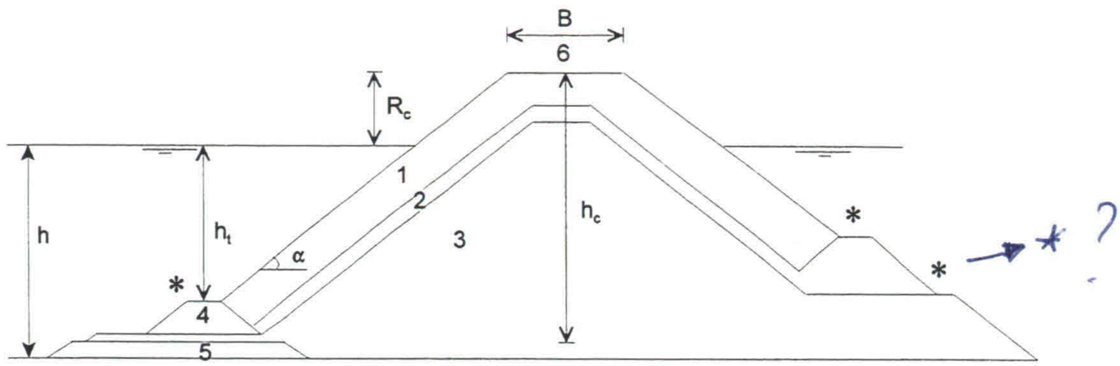

Figure 11-3, Definition sketch of a multi-layer cross section

1. Armour Layer
2. First underlayer
3. Core
4. Toe or Toe Berm
5. Filter
6. Crest

Tolerances
Irrespective of the construction element under consideration, it must be clear that there are considerable size deviations to be expected. Size deviations between design and reality, but also size deviations from one location to another location in the breakwater. Constructing mounds of stones with units up to 1 or 2 m diameter has an inherent inaccuracy in the order of magnitude of the stone size. The design of the structure shall take into account these tolerances. At different levels and locations, berms shall be provided where deviations in the actual measurements can be taken into account.

Armour layer
It is evident that the armour layer shall be able to withstand the wave attack during design conditions. The severity of these conditions follows from economic considerations. In general, the armour is placed in a double layer (n=2), since this allows a few armour units to be displaced before the underlying material is exposed.

The armour layer consists of concrete units or quarry stone. In case of quarry stone, it is generally the heaviest fraction of the quarry yield curve. It has a narrow grading (D_{85}/D_{15} < 1.5).

Crest
If the crest is consisting of loose armour units, its width shall be at least 3 stones, or in the form of a formula:

$$B = nk_t D_{n50} \qquad (11.4)$$

where n=3.

If the crest is formed by a concrete block, it shall be ascertained that the layer on which the block is placed or cast in situ is wider than cap block. It is never possible to fill voids under the cap block after it has been placed. To protect the cap block, it is recommended that armour material is placed at the seaward side to the full height of the breakwater. Parapet walls extending above the level of the armour units will be loaded heavily. In many cases such walls have suffered extensive damage. To ensure that the armour layer properly shields the cap block, it is recommended that a horizontal berm is maintained in front of the cap with a width of at least the size of one armour unit.

First underlayer
The layer direct under the armour layer is called the first underlayer. It is obvious that the units from this layer shall not pass through the voids in the armour layer. In the literature, one finds a rule that the weight of units in the underlayer should not be less than 1/10 of the weight of the armour proper. This is a very strict rule if compared with the filter rules of Terzaghi. These rules allow a ratio of 4 to 5 in diameter between two subsequent filter layers. One must remain a bit on the conservative side, however, because of the large consequences of failure of the filter mechanism. The filter shall therefore be "geometrically impermeable". It is recommended that the weight ratio of subsequent layers of quarry stone be kept between 1/10 and 1/25. (D_{n50} ratio between 2 and 3). It is noted in this context that choosing finer material for the first underlayer influences the notional permeability parameter P in the van der Meer formula. This leads to heavier armour material.

A second consideration for the selection of a certain size for the material in the first underlayer may be the stability during construction. Depending on the construction sequence, the first underlayer may be exposed to a moderate summer storm.

In case the armour units are concrete blocks, the secondary underlayer is the heaviest fraction of the quarry yield curve. When the armour units are quarry stone, the first underlayer is composed of an intermediate fraction of the quarry yield. It is generally a narrow grading.

Toe berm
The toe berm is the lower support for the armour layer. In traditional literature, one finds a weight recommended equal to the weight of the first underlayer. With the most recent data as presented in the chapter on stability formulae, the designer can find a balance between the level of the toe berm and the size of the stone in the berm.

Core
In most cases, the material of the first underlayer is such that the core can be situated directly under it. Assuming again a weight ratio between the first underlayer and the core between 1/10 and 1/25, it means that the core material is a factor 100 to 625 lighter than the armour material. This means that generally it is not necessary to apply a second underlayer between the core and the first underlayer.

For the core, generally a material called "quarry run" or "tout venant" is used, indicating that it is meant to represent the finer fractions of the quarry yield curve. It shall be noted that under no circumstances overburden (degraded or weathered rock) can be mixed with the quarry run. Mostly, quarry run has a wide ($1.5 < D_{85} / D_{15} < 2.5$) to very wide ($D_{85} / D_{15} > 2.5$) grading.

Filter
Specifically under the seaward toe, large pressure gradients may exist, that tend to wash out material from the seabed through the structure. Extension of the core material under the toe berm may even not guarantee the integrity of the structure as a whole. Loss of material in this region is an important threat to the stability of the armour layer. There are ample examples in literature that substandard filters have initiated failure of a breakwater.

It is therefore recommended to apply a geometrically impermeable filter under the seaward part of the breakwater. This filter may consist of a number of layers of granular material, or of a geotextile or other mattress. The pressure gradients under the centre of the structure and under the inner toe are generally much less. Here, often the quarry run may act as a filter of sufficient quality. Care must be taken, however, when land is reclaimed directly behind the breakwater. Internal reflection may than again cause filter problems at the inner boundary of the breakwater. In such case, special investigations are required to determine a satisfactory solution.

Since the layers of a granular filter are constructed at a considerable depth under water, it is necessary to give any separate layer a thickness that guarantees the presence of that particular material at any location. It will also be useful if the presence of the required material can be ascertained by inspection. In practice, it means that no layers thinner than 0.5m shall be designed. This may lead to a relatively thick filter bed if a granular filter is applied.

Scour protection
Just in front of the breakwater, the seabed may be eroded due to a concentration of currents, or due to a partially standing wave. Since loss of bed material directly in front of the toe may cause a soil mechanical stability problem, it is recommended to apply a blanket in front of the breakwater as scour protection. The width is to be determined on the basis of local conditions, but should not be less than 5 to 10 m.

11.4.3 Standard cross sections

In *Figure 11-4, Rubble mound breakwater - light overtopping (with cap block), , Figure 11-5, Rubble mound breakwater - light overtopping, Figure 11-6, Rubble mound breakwater - moderate overtopping (with cap block), Figure 11-7, Rubble mound breakwater - moderate overtopping and Figure 11-8, Rubble mound breakwater - severe overtopping* examples are given of standard cross sections based on the considerations given in 11.4.2. These cross sections show the changes of the leeward slope for increasing crest level and thus for decreasing overtopping and transmission. Examples are given as well of cross sections with a concrete cap block. This feature is rarely seen in low-crested or submerged breakwaters, probably because of the difficulty to place the block.

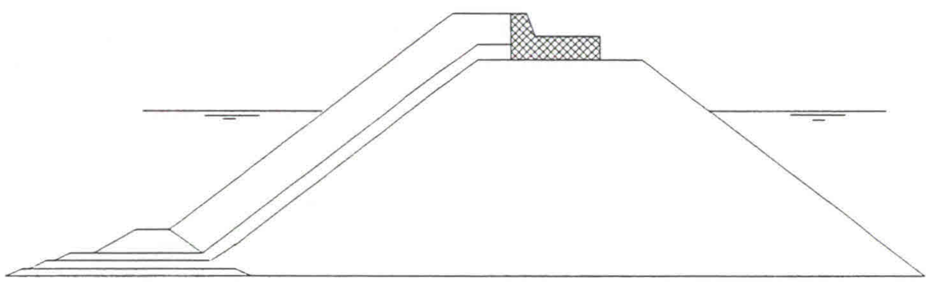

Figure 11-4, Rubble mound breakwater - light overtopping (with cap block)

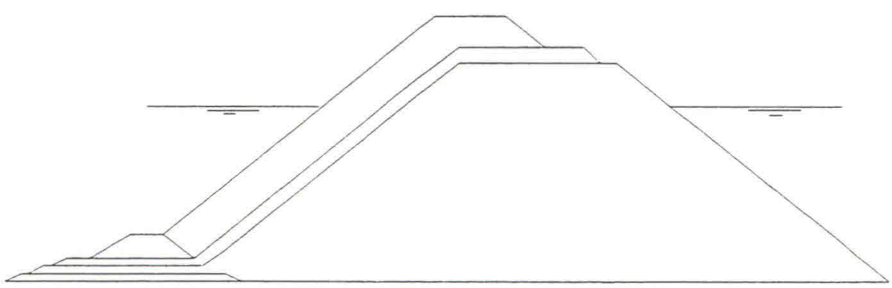

Figure 11-5, Rubble mound breakwater - light overtopping

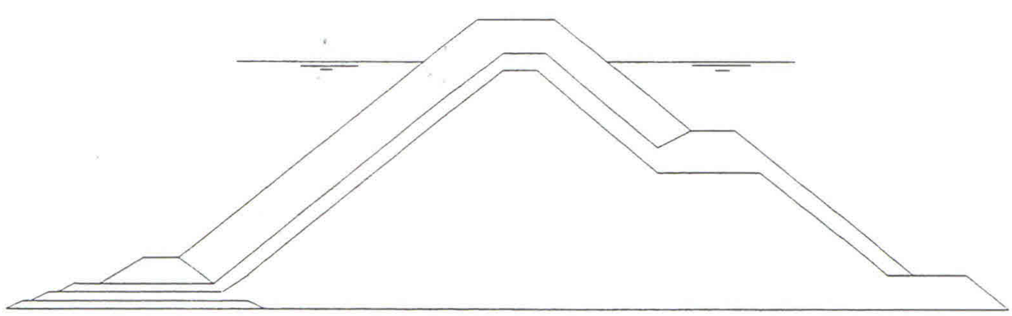

Figure 11-6, Rubble mound breakwater - moderate overtopping

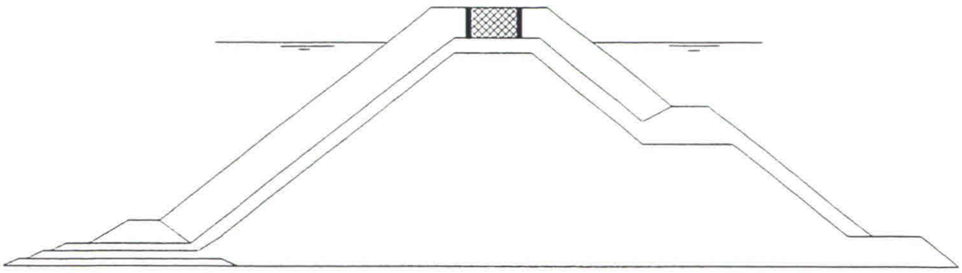

Figure 11-7, Rubble mound breakwater - moderate overtopping (with cap block)

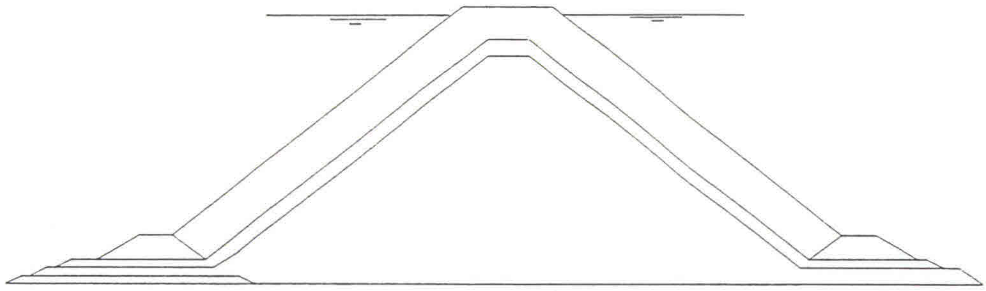

Figure 11-8, Rubble mound breakwater - severe overtopping

It must be noted that due to the relation between local water depth and local significant wave height, the cross section (including the size of the armour units) will vary considerably along the alignment of the breakwater. This gives the designer an added opportunity to match the quarry output with the over all demand of the project.

In the shallow water close to the shore, the standard design with a granular filter under the toe, will be difficult to construct. Due to the thick filter bed, the level of the toe berm becomes too high. The problem can be solved by dredging a trench for the toe, by replacing the granular filter by geotextile, or by modifying the toe berm. These four solutions are sketched in *Figure 11-9, Shallow water, dredged trench, Figure 11-10, Shallow water (5m) dredged trench gravel filter, Figure 11-11, Shallow water, dredged trench filter cloth, Figure 11-12, Shallow water (no excavation, filter cloth, increased berm).*

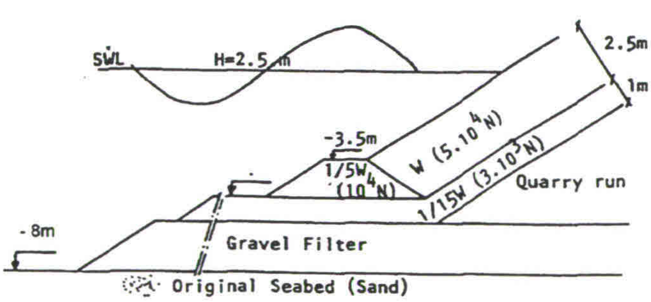

Figure 11-9, Shallow water, dredged trench groter

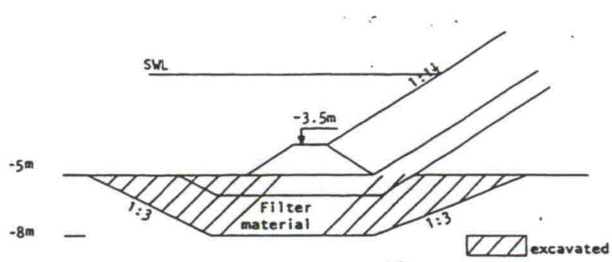

Figure 11-10, Shallow water (5m) dredged trench gravel filter

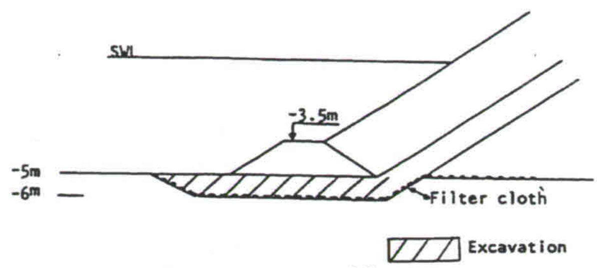

Figure 11-11, Shallow water, dredged trench filter cloth

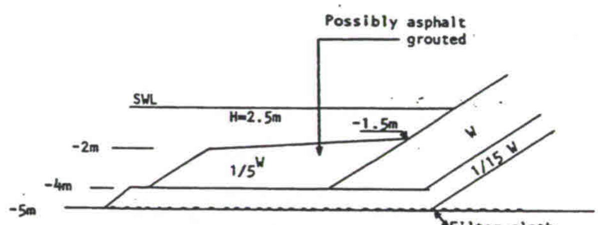

Figure 11-12, Shallow water (no excavation, filter cloth, increased berm)

Although the dredging of a trench seems expensive and a bit of an academic solution, it is not. In many cases, the bearing capacity of the subsoil is insufficient to create a safe foundation for the heavy load presented by the breakwater. This will be demonstrated by low safety coefficients in slip circle calculations, specifically when the soil is compressible and impermeable (consolidation time). In such cases, it is good engineering practice to apply a soil improvement. Placing the toe in the dredged trench creates the intended soil improvement.

klopt ook.

Although the figures presented in this chapter give a good impression of possible cross sections, the reader is recommended to study cross sections of actually constructed breakwaters as well. This applies to both successful designs, that have survived, but also to the unsuccessful examples that failed. When studying cross sections in handbooks, it must be kept in mind that for the sake of simplicity, sometimes essential features are not shown. In Annex 2, a review is given of all alternative cross sections that were generated during the design process of the Europoort breakwaters. In the same annex, sketches are given of the breakwaters built in IJmuiden and Scheveningen. → niet waar staan er niet in.

11.5 Monolithic breakwaters

In spite of the complexity of calculating the stability and the structural strength of monolithic breakwaters, the international commission on the study of waves[6] (formed by PIANC) has established a number of design recommendations. It is stressed here that these data are relatively old, and that a PIANC working group is currently preparing a new report on vertical wall breakwaters.

The Commission on the Study of Waves distinguishes two limit states:

- A limit state for use of the breakwater characterised by a wave height H_u with a reasonable return period,
- A limit state for rupture of the breakwater characterised by a wave height H_r that is an extreme wave height.

For establishing a preliminary design of a vertical breakwater the commission recommends a first approximation for the cross section as follows *(Figure 11-13, Recommended cross section according to PIANC (1973)):*

- Wall presenting a free height of at least $1.5H_r$ below low water (Follows a recommendation by the XVIIIth International Navigation Congress, Rome 1953);
- Wall having a thickness of at least 0.8 times the free height;
- A toe protection against undermining with a thickness of at least 0.15 times the free height. (This places the seabed at least $1.72H_r$ under LW;
- Crest rising to an elevation of 1.3 to 1.5 times H_u above HW on the sea side and 0.5 times H_u on the harbour side;
- Parapet wall with a thickness of about 0.75 H_u;
- Scour protection extending at least $2.5H_u$ in front of the wall.

This leads to a basic cross section as sketched in *Figure 11-13, Recommended cross section according to PIANC (1973).* This sketch can only be used as a first approximation in a design, its stability shall be verified thoroughly for every new application.

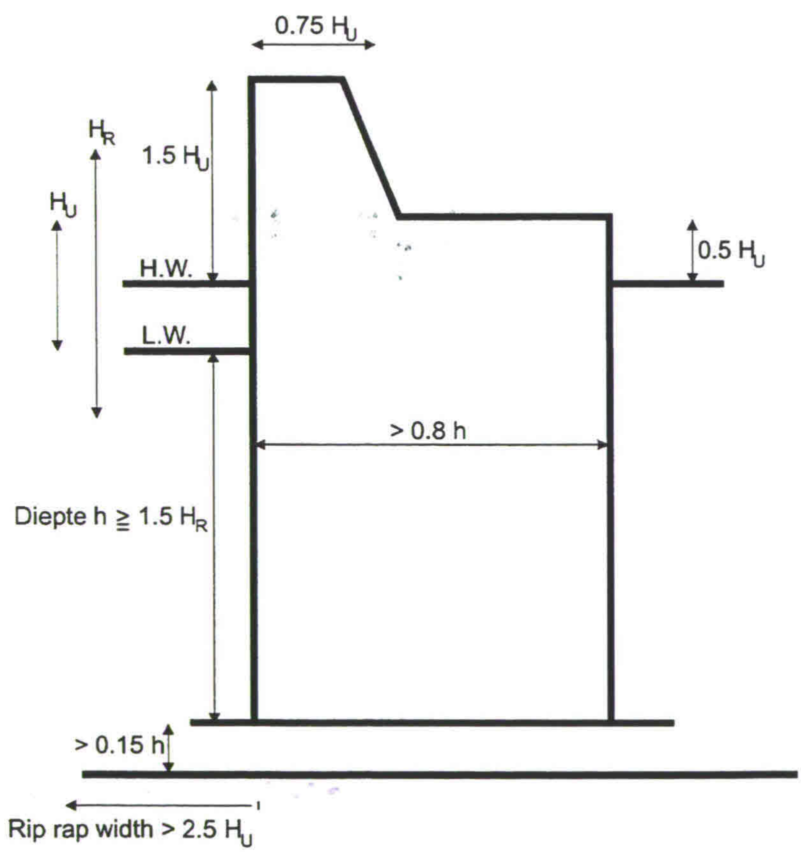

Figure 11-13, Recommended cross section according to PIANC (1973)

List of references chapter 11

[1] Anonymous (1984) "Shore Protection Manual" Coastal Engineering Research Center, US Army Waterways Experiment Station, Vicksburg, Miss. USA.
[2] Anonymous (1991) "Manual on the use of rock in coastal and shoreline engineering" CUR/CIRIA Report 154/83, CUR, Gouda, the Netherlands.
[3] Bregman, M. (1998) "Porositeit in geplaatste steenlagen, onderzoek naar factoren die de hoeveelheid geplaatst steenmateriaal beïnvloeden", Masters Thesis, Delft University of Technology, Delft and Royal Boskalis Westminster, Papendrecht, The Netherlands.
[4] Anonymous (1995) "Manual on the use of rock in hydraulic engineering" CUR/RWS Report no. 169, CUR, Gouda, The Netherlands.
[5] Schiereck, G.J. (1998) "Introduction to bed, bank and shore protection" Lecture notes Ctwa4310, Delft University of Technology, Faculty of Civil Engineering, Delft, The Netherlands.
[6] Anonymous, (1976) "Final Report International Commission on the study of Waves" Part 2, PIANC, Brussels, Belgium.

d'Angremond:
materiaal -> materieel -> de wijze v storten,
daarna nog een stukje over caissons.

van Roode:
manier v. lossen hangt af van het afte sluiten gat ->
quarry stone of zand ~~of~~ caissons.

tussen beide delen zit geen interactie

er staat dat het ook voor closure dams geldt maar die worden helemaal niet genoemd. Methoden zijn totaal verschillend, dan lijn ook Raitsnemen.

12. Construction Methods

12.1 Introduction

It is common for closure dams and breakwaters that the design of the cross sections is not only determined by theoretical or analytical considerations. There is a strong interaction between the construction method and the shape of the cross section. This does not mean that sound design principles should be neglected, it means that the feasibility of a construction method is equally important as a theoretical consideration in the design. Whereas we often start designing a cross section from top to bottom, i.e. by fixing the crest level, the outer slope and the weight of the armour units, we must start considerations about the construction from bottom to top. In doing attention must be paid to the stability (or vulnerability) of each construction phase, and to the accessibility of the work front for the anticipated construction equipment.

Closure dams and breakwaters have one further aspect in common: the massive character. Realisation of the structure requires a tremendous amount of material that is to be acquired, for instance from a quarry, to be transported to the location of the structure, and then placed within the profile along the alignment. This is a logistic problem that covers much more than the handling of the material only. It concerns opening of the various working sites, mobilising and maintaining equipment, mobilisation and accommodation of personnel, and last but not least the actual handling of millions of tons of material. Optimising the solution of this logistic problem may create savings that are much larger than a little extra cost for a sub-optimal design of the cross section.

It is impossible to make a complete description of all possibilities for the construction of closure dams and breakwaters in a book like this. In practice, consultants and contractors have a special and very well documented department to do this kind of work in the tender and pretender stage. In this book it is attempted to give students an idea of possibilities and problems. Reference is made to the CUR/RWS Manual no.169 (1995)[1], where more details are given, and from which some information is copied here.

In this chapter, we attempt to follow material and construction elements from their source to their final destination. This applies specifically for quarry stone and large concrete elements, aspects of using sand and other dredged material will largely be left to more specific books on dredging. Since the possibilities of handling material depend largely on available equipment, first characteristics of the most common equipment must be given. For this, the reader is referred to Annex 4. klopt niet.

EQuipment Annex 1

12.2 Quarry

It has been indicated earlier which data must be collected before it is decided to open a quarry in a particular rock formation. To start the actual quarry operations, some requirements must be met:

- Access shall be ascertained
- Required permits shall be available
- Protection measures against damage to human interests and ecology shall be operational
- Accommodation for personnel shall be available
- Maintenance facilities for equipment shall be available
- Supply of fuel, spare parts, explosives, etc. shall be organised

The planning of the quarry operation is mainly based on the expected fragmentation curve. The blasting and quarrying shall be done in a systematic way, following a pre-determined mining plan. During the blasting, benchfloors are created that can be used for sorting the material, loading and transport. The width of the benchfloor shall provide adequate working space for these purposes.

In most cases the larger fractions will be difficult to obtain. In the beginning, this does not appear to be a problem, since in the first phases of the project, only finer material for filters and core is used. The larger fractions are required in the later phases of the project, when armour layers and breakwater heads are under construction. Then, however, it is too late to modify the blasting scheme and to obtain the required percentage of armour stone. It is therefore recommended to produce the larger fractions from the start of the operation, and not to postpone the efforts until the last stage of quarrying. The consequence is that all stone gradations shall be sorted out and stored separately, right from the beginning, even if some stone classes are not yet used.

This leads automatically to the need for a large stockpile area where stone can be stored until it is used.

It is recommended that in the quarry the classification of stone is facilitated by providing sample stones per category and by frequently using a weighbridge to check the weight of stones.

12.3 Transport

Transport of material from the quarry to the work site can be done in three different ways:

- By road;
- By rail;
- By water (either inland or sea);
- Combination of methods.

It is impossible to indicate a preferable mode of transport. The choice depends on local conditions, available facilities and required extra investments. In general, transport by water is far cheaper (4 to 5 times) per tonkilometer than transport by road or rail. This is valid only if a waterway of sufficient width and depth is available, or can be made available at little extra cost.

It is not certain that delays in the transport chain coincide with delays in the quarry operation. This is a second reason to provide a stockpile area at the quarry side with sufficient capacity to cope with irregularities in production and delivery of stone.

12.4 Actual breakwater or dam construction

At the location of the dam or the breakwater, a relatively large construction yard is required. Place has to be provided for offices, accommodation, workshops, etc. In general, also a stockpile for quarry stone and other construction materials is required to act as a buffer when supply and discharge are not in balance. When concrete armour units are used, a concrete mixing plant is required as well as a block casting area and a storage area for the armour units.

- In principle, there are three methods to bring the material into the designed profile:
- By floating equipment
- By rolling equipment
- By a combination of both.
- By Rail.

It is evident that for detached breakwaters or for dam sections not (yet) connected to the mainland floating equipment is the most logical choice.

12.4.1 Rolling equipment (welke zyn dat dan)

If rolling equipment is used, a dam is built out with a work front in several phases, for filters, core material, first underlayer, toe, etc. The crest of the dam is used as main supply road. It has a minimum width of 4m for one lane traffic. For two-lane traffic the crest width shall be at least 7m. As an alternative, one can create passing places. Since it is virtually impossible to drive over the armour units (they are too large), the access road to the work front is often created at the crest of the core or at the crest of the first sublayer. The level of this crest shall be high enough above HW to guarantee the safety of equipment and personnel working there.

In this way, the full length can be constructed according to the design, except for the armour units on the crest. These units can be placed in the final stage, working backwards from head to mainland (See Fig. 12.1).

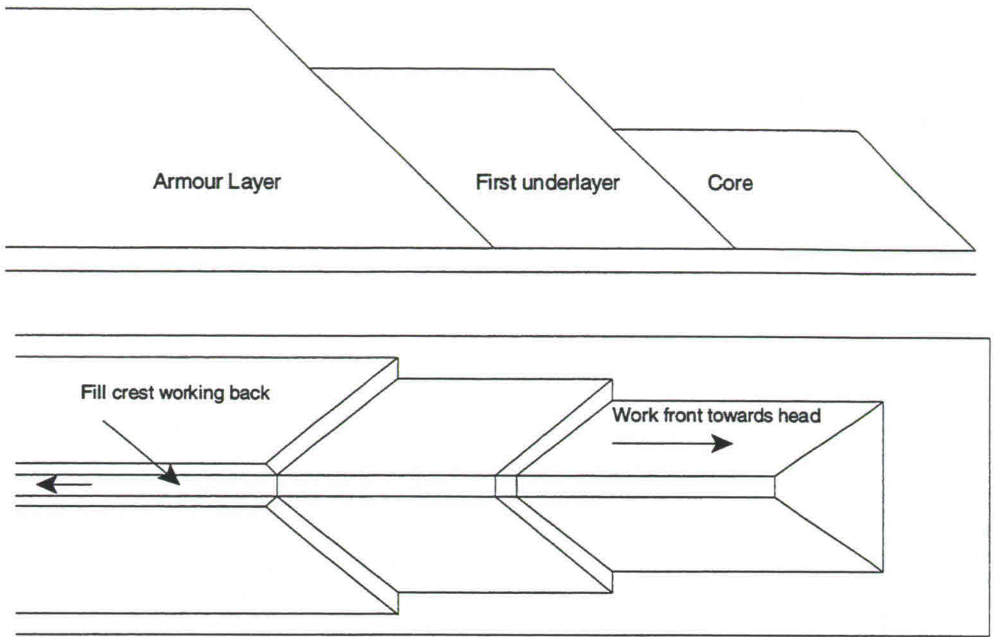

Figure 12-1, Subsequent Work Fronts

In case a concrete crest block is used instead of a crest of loose armour units, one may bring this cap block in place when building out the dam. The cap block can than be used advantageously to provide a better quality access road to the work front.

The filters and the core are often placed by bulk dumping. The armour units are always placed individually by crane to avoid the risk of breakage or misplacement. The method to place the first underlayer depends on the size and the local conditions. For the material that is placed individually, the crane is nowadays fitted with electronic positioning equipment to place the units in a pattern as prescribed in the specifications.

A disadvantage of the dry construction method is the fact that all material must be transported over a rather primitive road with limited capacity. This becomes ever more important as the length of the dam increases. When the crane at the work front prevents direct dumping, one may consider the use of a gantry crane. The required width and height of the workroad over the crest may lead to a much bulkier design.

The major advantage of the dry construction method is the potential use of cheap local equipment and the independence of working conditions at sea (fog, waves, swell, currents).

12.4.2 Floating equipment

For the use of floating equipment, a work harbour is required from the start of the operations. In this harbour, the barges can be loaded.

For placing filter layers of a limited thickness, split barges or preferably a side-dumping vessel may be used. For construction of the core, bulk dumping can be applied with bottom door barges, split barges, tilt barges or flat deck barges with bulldozers pushing the material over board. Intermediate layers along the slopes may be applied with side unloading barges.

As soon as the structure reaches a level higher than HW -4m to -3m, the use of these barges and vessels becomes impossible. If one continues to use floating equipment, floating cranes or crane platforms are needed to finish the upper part of the profile. The cranes may also be necessary to trim the slopes of the core that are constructed in bulk dumping operations.

The main advantage of the "wet" construction method is the possibility to start construction at more than one work front, or to build detached breakwaters. When material is transported by barge anyway, it is an advantage to directly use the supplied material, without stockpiling.

A disadvantage is the dependence on working conditions at sea and the need for a working harbour.

12.4.3 Combined method

A combination of the wet and dry construction method is often preferred since the wet working method for the bulk of the material in the lower part of the cross section reduces the tonnage to be transported over the crest considerably.

One must realise, however, that the combined method brings not only the advantages of both methods, but also the disadvantages.

12.5 Minimizing risks during construction

It has been mentioned a few times that attention shall be paid to the stability of the cross section during construction. For closure dams, this may be evident, because the closure dam in itself is a temporary structure that shall be replaced and protected by the final structure as soon as possible. Nevertheless, one must take into account that the closure dam must withstand the hydraulic loads that are expected during the period of exposure.

In a similar way, one must consider the various construction phases of a breakwater. Considering Figure 12.1, it will be clear that the work front can not withstand the design storm. Therefore, one must consider what risks are threatening the structure during the construction phases and find ways to reduce those risks. Common methods are:

- Select a specific construction period
- Reduce the exposed length of the vulnerable part of the structure
- Construct protective bunds

Specific construction period
Sometimes it is possible to reduce the risks by limiting the construction period to the summer season (or to a particular monsoon season). If the complete structure can be completed in this calm period, the risk can be much less. Otherwise it may be possible to interrupt construction during the rough season. When the work fronts are protected well, it will be possible to resume construction next year.

Reduce exposed length
In other cases, it is an option to keep the distance between the three work fronts as small as possible. If damage occurs, it will be restricted to a small stretch of the structure. Since the contractor is still present with all his equipment, repair of the short damaged section needs not to pose great difficulties.

Protective bunds
Instead of the work sequence as sketched in Figure 12.1, it is possible to dump the first underlayer before the core material. The method can be compared with the construction of reclamation bunds around a confined reclamation area in dredging. Disadvantage of the method is that more material is required for the first underlayer than strictly necessary on the basis of the theoretical two-layer design. In Figure 12.2, the traditional method is compared with the alternative.

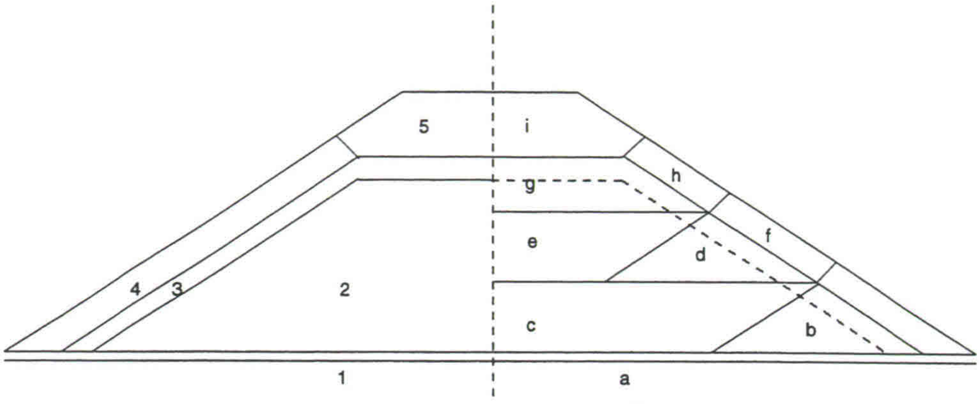

Figure 12-2, Varying construction sequence

Construction sequence as in figure 12.1
1. Filters
2. Core
3. First underlayer
4. Armour layer
5. Crest (working back)

Construction sequence providing protective bunds
a) Filters
b) First underlayer (part)
c) Core
d) First underlayer (part)
e) Core
f) Armour layer (part)
g) First underlayer (part)
h) Armour layer (part)
i) Crest (working back)

It is clear that the alternative method (in the right hand side of Fig. 12.2) provides a better protection of the core material during construction phases. It is clear at the same time that it requires more (expensive) first underlayer material, and that the construction method is a little more complicated. Depending on the availability and cost of material versus the cost of handling, one can save some of the extra material by double handling.

12.6 Monolithic breakwaters

12.6.1 General

Monolithic breakwater structures can be composed in several ways:
- Assembly of small units, connected in situ
- Construction of large units in situ
- Use of large prefabricated units

Prefabricated large units can be transported to the site in various ways: self floating (caisson), floating with the aid of rigs or lift vessels (rings) or over the crest of the existing breakwater.

12.6.2 Assembly of small units

The oldest vertical wall breakwaters were composed of rectangular blocks of natural stone. These blocks were sawn in the quarry and placed in the breakwater according to a pattern compatible with the present brickwork techniques. The blocks were connected with dowels to ensure the monolithic behaviour of the structure.

This technique is still used, although the blocks are often casted in concrete nowadays, and steel reinforcement and cement mortar are used to connect the blocks. (Figures 12.3 and 12.4)

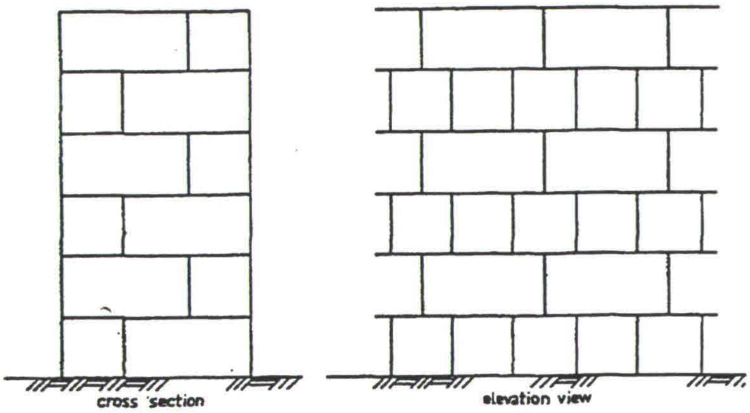

Figure 12-3, Typical block wall

grote

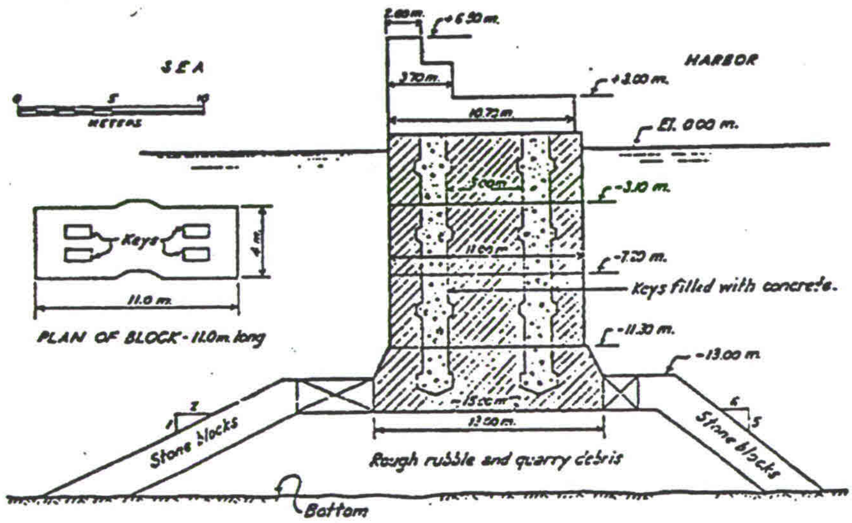

Figure 12-4, Breakwater Algiers, Morocco

12.6.3 Construction of large units in situ

The most common example of in situ construction of large monolithic units is the construction of sheet pile cells. The main problem of this type of structure is the closure of the slots between the individual piles. Also the workability during pile driving may cause problems.

The cells are filled with soil or stones. It must be assumed that due to overtopping and spray, the fill material is saturated with water over its full height. Depending on the type of fill material, cyclic loading and wave impact forces may cause liquefaction of the fill material, which results in relatively high ground pressures on the sheet piles. Poor connections between the piles are a serious complication in that case.

12.6.4 Prefabricated large units

As mentioned in 12.6.1, prefabricated large units can be transported in different ways. The most elementary way is to use the buoyancy of the elements. In that case, we speak of caisson type structures. Because of their specific importance for both dams and breakwaters, they are treated in a separate paragraph: 12.7.

It is not necessary, however, that the prefabricated unit is fitted out with a watertight bottom. It is possible to place circular or rectangular rings on a foundation bed and fill them with quarry run to act as a monolithic breakwater. The units can then be brought in place using separate floats or lift barges. It is also possible to roll them out over the crest of the placed units and lower them with a gantry crane into position. This method was used in Hanstholm (Denmark) and Brighton (See Figure 12.5).

groter.

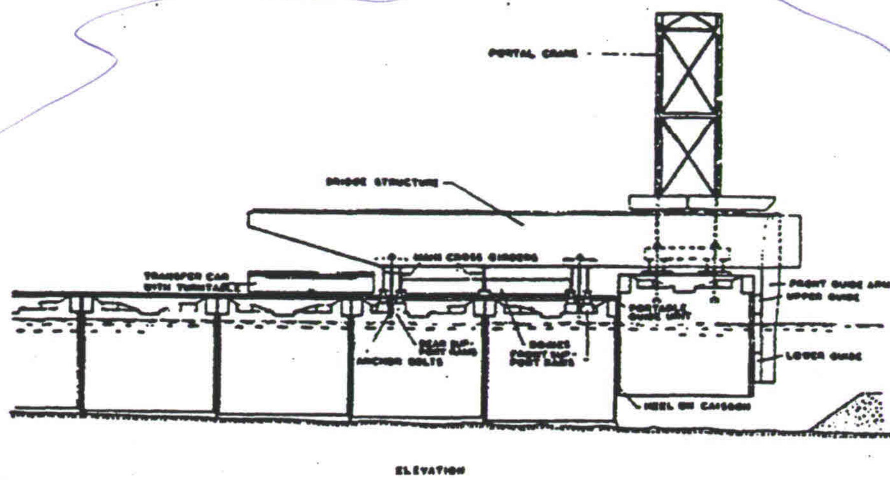

Figure 12-5, Construction method Dover Breakwater

12.7 Use of Caissons

12.7.1 Introduction

Caissons have been widely used in both, closure dams and monolithic breakwaters. Although there can be some structural and operational differences, the basic principles are quite identical. The structural differences may be due to the different load pattern or to the fact that in closure dams, the caissons are designed to allow discharge until gates are closed. The operational differences may be due to the fact that in closure dams the current is much more of a constraint during placing the caissons than in breakwaters.

Because of the small differences, no distinction is made here when discussing the construction aspects of caissons for the two types of application.

12.7.2 Building yard

It is possible to construct caissons in rather different ways. The main difference is the questions whether construction is completed in the dry, or that just the base of the caissons is constructed in the dry, until the buoyancy is sufficient to launch them and complete construction in floating condition.

Whether only the lower part is constructed in the dry or the whole caisson makes not much difference for the initial stage of construction. The construction yard must be kept dry until the structures are ready to float. For this purpose one may use the following facilities:

- Dry-dock at a shipyard
- Slipway
- Lift deck
- Dredged (special purpose) dock

The first three facilities are common features at a shipyard. They can be used if available at an affordable cost. Disadvantage may be that the space is too small to construct the required number of caissons in a limited period. Then, one may consider to float out the caissons long before they are completed and to finish the upper part and the superstructure in floating condition.

The last option, a specially dredged large dock is a common solution in the Netherlands. The dock is kept dry by deep wells, and the closing dike can easily be breached by a dredge when the construction is completed. All caissons for the closure in the Delta project have been constructed this way (see Fig. 12.6).

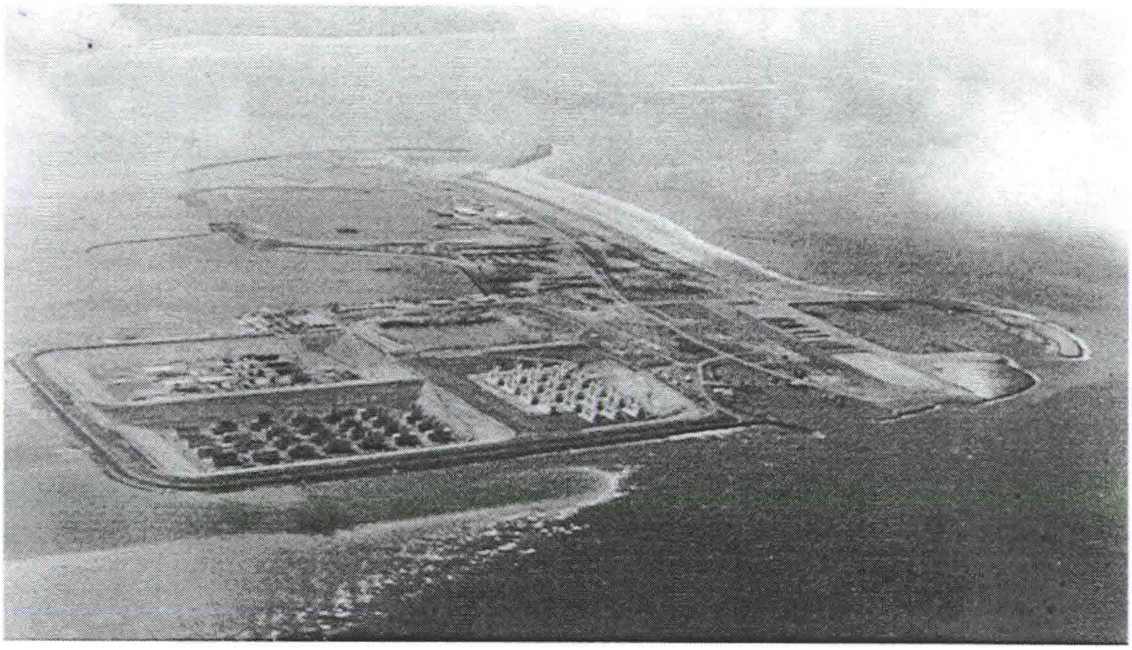

Figure 12-6, Dredged Dock for Construction of Caissons at the Neeltje Jans Construction Island

Advantage of such special purpose dock is that it can be built close to the site where the dam is to be constructed. Also the size of the dock can be chosen to comply with the specific demand.

The construction site for the building of caissons is further fitted out like any construction site for a large concrete structure.

12.7.3 Transport

After completion of the caissons they have to be transported to the site of the dam or breakwater. It is essential that sufficient depth and keel clearance is available throughout the route from the dock to the site. Tugboats with sufficient power to overcome currents and to maintain a reasonable speed tow the caissons.

Proper attention shall also be paid to the stability of the caissons. This means that adequate calculations of the metacentric height must be carried out.

12.7.4 Placing

When on site, the caissons must be placed in the required position at the seabed. Adequate tugboat assistance is required to tow the units in position and to keep them in position during sinking. This is generally done at slack water. The question whether the placing operation is carried out during HW slack or during LW slack depends on the draft of the caisson and the available water depth. Advantage of the LW slack is in many places a reduced wave action.

It is not easy to keep the unit in position during the sinking operation. It is recommended to have at least one or two winches available to make a connection with the shore or with previously placed caissons. Specifically short before landing on the foundation, the unit has a tendency to move horizontally out of control. This is due to the overpressure in the thin layer of water between the seabed and the bottom of the caisson. The problem can be solved by increasing the permeability of the foundation layer or by fixing a skirt or some steel rods in the bottom of the caisson.

? caissonstuk van Roode ?

12.8 Construction of a closure

12.8.1 Closing operations

A closing operation is a struggle with nature. Flowing water on an erodible bed has to be controlled. Every human action to obstruct the flow will immediately be counteracted in some way or another by nature itself. Of course this happens within the laws of nature, of which many (but not all) are known. This knowledge gained in the past by (bad) experience, is supplemented these days by advanced research and experiment. Nevertheless, the changes in conditions during the progress of the closing operation are sometimes difficult to predict.

In the figure, an example of an unpredictable change in topography of a closure is illustrated. During the closure of the River Feni Estuary in Bangla Desh in 1984/'85 the longitudinal profile of the alignment enlarged considerably in a month time. This was caused by a meandering secondary gulley and did not change the river's discharge, but a lot more material was needed for the closure.

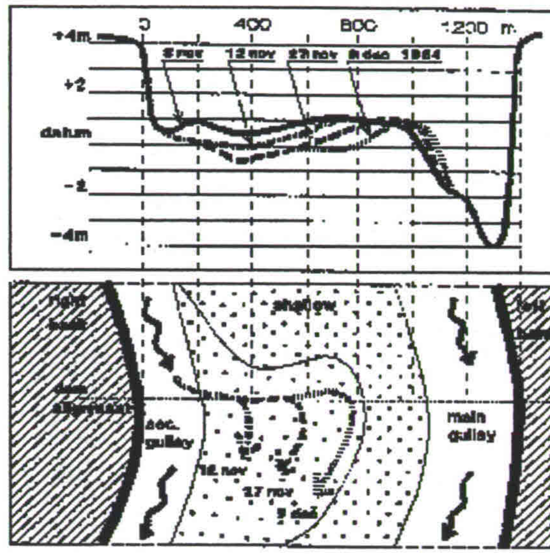

Figure 12-7, Profile and plan view of the river Feni

Every closure design needs a full description of the existing situation in the first place. The hydrology, topography and soil structure of the area and climatological conditions have to be assessed. Then, a calculation is required to establish the change of conditions to be expected after the planned closure is realised. Lastly the intermediate phases of the construction process have to be detailed. Very important is to conclude which stages are critical and determining in the ever changing situation.

Data is but seldom complete and not always reliable. Besides, nature provides unpredictable conditions but sometimes they can be described statistically. Theory is an approximation of practice. Consequently there is always a rather high level of risk involved that things go different than expected. The historic cases clearly show the correctness of this statement. Much attention has to be paid to "what if" aspects of the design without typically over-dimensioning. However, the execution of a closure will always require a lot of improvisation in order to act immediately on nature's reactions.

12.8.2 Closure by hydraulic filling with sand only

In a number of cases, a tidal basin can be closed by pumping sand only. The principle is simple. As long as more sand is pumped into the closure gap than the flow erodes away, the gap narrows. Due to the development of dredgers with very high capacities (5000 to 8000 m^3 sand per hour), this has become realistic. Bulldozers are needed to spread the sand-water mixture over the fill and shape the desired profile of the dam. Besides, they prevent the erosion of gulleys on the fill slope and densify the deposited sand. This equipment, used to control the fill process has improved likewise.

The main question thus is how much capacity is needed. A very high capacity can be attained by using many delivery lines from various dredgers. However, to keep the fill under control, every delivery line needs a certain width for these operations. More capacity means a much wider fill-profile, by which progress is not improved. So, there is a practical limit to the supply. Of course, the gap can be approached from two sides.

A sand closure is a horizontal closure, with a progressively narrowing gap, in which the flow increases until the very last gap can be blocked. The flow in the gap has a sand transporting capacity which is related to the flow velocity to the power of the order of four. This means that when the flow increases from 2 m/s to 2.5 m/s the transport of sand multiplies by a factor 2.5 and when it goes to 3 m/s by a factor 5.

The normal process of building a sand fill dam is as follows. A sand water mixture is pumped by the dredger via a delivery line to the fill, sufficiently high above H.W. (point A1 in the figure). The mixture runs down the slope to the water line (A2) and into the water, while the sand settles. The sand creates a slope above water (A1-A2) which is much flatter than the submerged slope (A2-A3). In the ebb period, going from H.W. to L.W., the progress is as shown in the figure from A1-A2-A3 to B1-B2-B3. Progress seems small, as the delivery point hardly shifts (A1-B1).

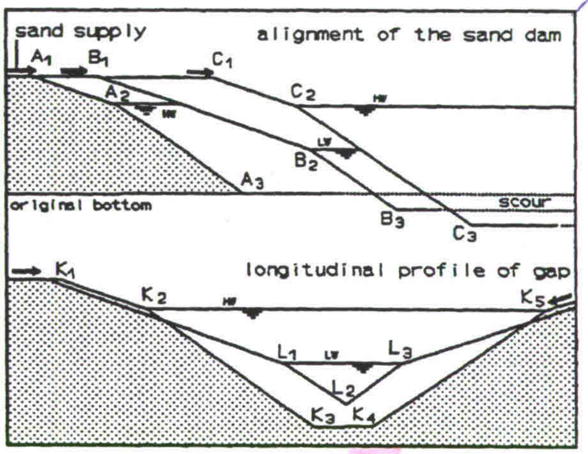

Figure 12-8, Closure by pumping sand

All the sand goes into the toe circle. During flood however, the line B1-B2-B3 shifts to C1-C2-C3. The nett progress per tidal cycle is A1-C1. Scour will erode the original bottom in front of the dam and enlarge the profile to be made. Besides, part of the supplied sand will be taken by the flow also and carried outside the alignment area. The further the dam extends, the higher the flow in front of it and the smaller the nett progress per tidal cycle.

When hardly any progress is made, the final blocking has to be enforced. This is a special operation, which starts at the moment of H.W., shown in the figure by the line K1-K2-K3-K4-K5 for two fills approaching the gap from both sides. Progress during the ebb may seem negative even, as the line K1-K2 goes backward but the gap size at L.W. is virtually small (L1-L2-L3). Then, just before slack water, the profile has to be blocked. This requires the temporary interruption of hydraulic transport with recourse to earth moving plant thus obviating the erosive action of the hydraulic transport water. Bulldozers and cranes will have to shift as much sand as possible into this L.W.-gap to shape a tiny ridge into the triangular profile (L1-L2-L3). This sand is taken from the above water slope (K2-L1). The ridge being ready, the flow is blocked and all the sand supplied by resumed pumping will settle in the profile. The ridge, protruding above L.W. has to be heightened and widened to stay ahead of the rising outer water. The volume required increases tremendously with the rising level, as the ridge length also increases. To create a stable profile over the full length (K2-K5), able to stand the head difference of the full tidal range, in a couple of hours, requires a very skilled fill procedure and a sufficient sand delivery capacity as well.

In total, the principal questions are: how large a gap can be closed in the last tidal cycle and what will be the sand transporting capacity of the flow in that gap? The last gap's operation takes place in one tidal cycle, so for a semi-diurnal tide in about twelve hours. The twelve hours before that is the last tidal cycle in the normal fill procedure, in which some narrowing progress is still to be made. Consequently, the possibility to close is fully determined by the operations during the last day. The capacity required thus is determined by:

- the normal process to attain the size of gap that can be closed in one tidal cycle,
- the ebb-phase to shape the tiny ridge at the L.W.-slack period,
- the flood-phase to build a stable profile before the next H.W.

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With that capacity laid down, the progress in the days (weeks) before can be calculated by phase wise determining the nett progress per cycle. Summing-up gives a reasonable approximation of the total time needed for the construction of the dam.

In practice several sand closures have been realised. It appeared that the maximum flow velocity which could be accommodated was in the order of 2 to 2.5 m/s, dependent on the grading of the sand . These velocities occur for a head difference of about 0.30 m, according to the flow formulae in gaps (section 3.2), for rounded (sand-)dam heads. The gap size for which this boundary condition exists can be calculated using a mathematic model of the closure procedure.

For instance in the mathematic example "case 1" of section 3.3, the flow reaches about 6 m/s in the final stages (4 and 5), when the head difference is slightly more than half of the tidal range (1.5 m). The 2.5 m/s is reached already at stage 2, which is for a gap dimension of 4000 m^2. This gap size is far too much for the final day's operation. Nevertheless, the sand-closure of the "Krammer" (in the rear of the Eastern Scheldt basin) in 1987, closed a basin of about 55 km^2 in which a tidal range of 2.70 m existed. However, by that time, the storm surge barrier in the entrance of the estuary was operational and the tidal range was artificially reduced to 0.60 m during the closure. Thus, flow velocities were kept under 2.5 m/s to enable the closure. This shows that if the tidal range is smaller than 0.6 m, even very large basins can be closed by pumping sand.

Basins with larger tidal ranges can only be closed by sand pumping if the basin's storage area is much smaller. An example of that is the sand closure of the Wohrdener Loch in northern Germany near Meldorf in 1978. The tidal range during the neap tide on closure day was 3.20 m. The storage area was 10 km^2, the grading of the sand about 350 micron, the total installed dredge capacity 8000 m^3 per hour and 14 bulldozers and 8 hydraulic excavating machines were busy at the fills. The length of the gap at the water line during H.W. (K2-K5) in that case was 120 m. The flood phase capacity, to strengthen the ridge, appeared to be the determining factor.

A few conditions determine the possibility to close with sand only:

- the tidal range or the storage area of the basin has to be sufficiently small,
- large quantities of good quality sand must be available nearby,
- high-capacity dredgers have to be used,
- a well-organized fill-procedure by cranes, back-hoes and bulldozers is required.

As long as the original bottom in the gap has a resistance against erosion comparable to the sand used for closing, a scour protection is not relevant. Scour is acceptable unless it endangers stability of structures in the proximity. A considerable volume of sand is carried by the flow outside the desired profile. This reduces the progress but is part of the method. The lost sand is not considered a loss. Actually, instead of providing an expensive bottom protection, scour is compensated by using an extra quantity of sand. The many machines operating at the fill require a good passable subsoil. Sand closures with very fine or silty fine sand are hardly or not possible.

12.8.3 Scour prevention by mattresses or filter layers

Every closure of a watercourse leads to a situation in which the flow accelerates, circles around dam heads or crosses over materials with different hydraulic roughness. Each of these results in increase of the capacity to erode. In the sand closure process the scour is accepted since its magnitude is limited because of the restrictions to the flow velocities which offer the possibility to apply the method. For all other methods and dependent on the resistance of the bottom material against erosive action, scour holes may develop, which can endanger the stability of the closure dam. This has to be prevented by the placing of bottom protection means at all relevant locations. These do not prevent the scour completely but shift its bearing towards less vulnerable locations and may reduce the scour depth. Scour prevention therefore is part of most closure processes and generally one of the first actions in practice.

Generally speaking the scour resistance of the bottom material is difficult to predict. Rock and stiff clay will be very resistant, soft clay is rather resistant, peat may stand the attack quite long and then suddenly break out in large lumps. The behaviour of sand has been investigated intensively and several formulae have been derived to predict the scour hole development. Since in practice a sandy subsoil is but seldom homogeneous, the actual scour may still deviate from the predicted values.

In short, scour holes can be expected at places where:
1. the flow velocity increases in course of time
2. the flow distribution over the vertical changes
3. the flow is not saturated with sediment
4. the turbulence intensity increases

These aspects occur in closure processes for instance:
- when diminishing the gaps profile (item i)
- at the end of a stone protection as consequence of change in roughness (item ii)
- due to reduction of the discharge quantity in the approach gulley (item iii),
- around dam heads, structures and obstacles (item iv)

Of course combinations of these four often occur.

In one-directional flow, the scouring process creates a hole which is characterised by its steep slope at the upstream side, its depth and its gentle downstream slope. In tidal areas, where the flow changes direction in every tidal cycle, the shape of the hole will be different. The reversing flow smooths the hole out slightly by which the slopes become more gentle. The development of the hole goes quickly in the beginning but gradually slows down. By creation of the hole, the bottom topography adapts itself to the flow's eroding capacity and in the end an equilibrium state is reached. The depth of the scour hole develops in an exponential relation with time. In many cases the equilibrium state is reached so quickly that the intermediate stages are of no importance. However, if a number of caissons are placed one after the other in a couple of weeks time, the flow pattern changes stepwise in short periods and so does the scouring capacity. The scour hole then develops as a summation of intermediate successive stages.

The development of scour holes in itself is not dangerous. Only in cases where they come too close to either the closure dam or adjoining dams or structures, will they endanger stability of these structures. Then, uncontrolled scour should be prevented. A scour protection by a bottom mattress or a filter layer will be required. Since the costs of these protections in closure works generally are considerable, minimizing the dimension is important.

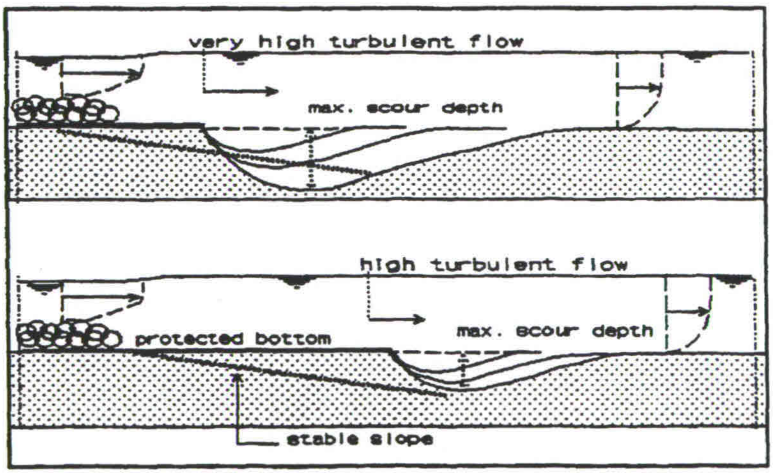

Figure 12-9, Development of scour hole

However, the installation has to be done in advance of the determining situation (construction phasing) and a too short protection may give a large risk. The longer the protected area, the further away the hole develops and since on that spot the attack will be less, for instance because of spreading of the flow or diminishing of the turbulence intensity, the equilibrium depth of the hole will

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be less. Both aspects, further away and lesser depth, improve the stability consideration of the endangered structure. Usually, as a first approximation, a protected length of about 10 times the water depth is considered safe. For detailed engineering in case large costs are involved, the optimization requires physical model investigation in a hydraulic laboratory.

For the stability considerations the dam in the closure gap and the joining bottom protection act as a total structure. Consequently groundwater flows and potential head differences will build up over this protection. Therefore the protection has to:

- be flexible to follow changes in bottom topography
- be well connected to the bottom, leaving no room for piping,
- be sufficiently heavy to prevent flapping in the flow,
- be extra ballasted at its end to prevent turning up when the tide turns,
- be impermeable for the soil material underneath.
- be stable in all flow conditions in all relevant construction phases.
- be permeable for water to prevent high pressures underneath, (sometimes the requirement for an impermeable part is combined with the scour protection, in spite of the pressure increase; see figure on page 4-8).

The present-day bottom protection is made of a geotextile, reinforced with a grating of strings made of willow or bamboo, which is ballasted with quarry stone. Specially near scour holes and on gulley sides, the protection may overlay quite steep slopes. Therefore the textile needs strength and the ballast needs stability against rolling-down. The geotextile has the main function, to be soil tight. For very fine subsoils sometimes a two-layer geotextile is used of which one layer provides the strength (a woven) and the other one (a felt), the tightness. The purpose of the grating is to keep the mattrass stretched during the laying operation, and to give lateral support to the ballast.

Installing the protection is done by various methods. Usually, the area to be protected is divided into rectangular plots. Each plot is covered by one mat. Overlaps of 2 meter in the width and 5 meter in length allow for inaccuracies of placement. The method which is generally used is to bring the mat to the plot in floating condition, position and stretch it between anchored pontoons and then lower it to the bottom by ballasting. Forces by the flow on the mat are considerable due to the large surface area of the mat and oblique flow is a hindrance. Therefore, the maximum practicable dimensions of these plots are taken about 30 meters wide and 80 to 120 meters long (in flow direction).

Another method is to wind the mat onto a cylinder, and unroll the cylinder either at the water surface or close to the bottom. For this method the grating is omitted, while some ballast has to be tightly fixed to the geotextile in order to keep it down on the bottom straight after unrolling. Generally, in all methods, the protection of the bottom is laid in an early stage of the closing works as flow velocities during lay-operation are still limited. Besides, flow attack on overlaps and erosion along the mat during the lay operation will also be less and there is less risk for an upturned rim or a piping channel underneath the mat.

A granular filter, consisting of various layers of material of increasing coarseness, from sand via gravel to quarry stone, also serves the purpose. The advantages of this protection are:

- they can be laid much easier and quicker then the geotextiles,
- they are self correcting for small damages (for instance for a dropped anchor), which makes them less vulnerable,
- there are no structural joints,
- they are relatively simple to remove (dredging),
- towards the end of the area to be protected, they can gradually be faded out.

The disadvantages are:

- there is no structural coherence, they disintegrate on steep slopes,
- a major damage escalates, as it exposes more fine material underneath,
- the construction height is large as each layer needs to be quite thick to allow for inaccuracies during the placing.

A granular filter may be a good solution if scour develops a larger hole than expected. Then, well in time, the steep gradient of the hole can be stabilized by quickly dropping a cover of gravel or quarried stone.

Sometimes a design option is considered in which the protection has to be watertight. In practice this is very difficult to realise. The laying of large membranes in flowing water is a difficult operation and to obtain watertight joints is nearly impossible.

12.8.4 Quarry stone dams, dumped or tipped

Quarry stone is frequently used for closure operations. Various methods apply and the method, the equipment and the conditions are interrelated. A major distinction is the difference between a horizontal and a vertical closure which results in completely different flow- and water level parameters. Different construction methods or equipment may result in dam profiles with different slopes and thus influence stability calculations. The grading of the stone depends on the flow- and stability criteria.

During a horizontal closure the dam heads are build out by tipping stone, for instance with dump trucks. Then, the dam head slopes are steep natural slopes of 35 to 45 degrees. The flow will attack the dam at the dam head, where the flow turns around the spearhead, contracts and looses contact, starting a line of eddies. The attack is most severe at the upstream side in the gap where the eddies start. The dam heads can proceed with a dump capacity of 250 to 300 m^3 per hour per dam head. The final gap, which is the gap during the last tidal cycle, is much smaller than with sand closures. A tidal basin closed with quarry stone will therefore have a larger head difference over the final gap than with sand closure (if possible) for the same basin.

stabiliteit apart hoofdstuk. dus shields vertellen!

For stone stability calculations, the formula of Shields is used. According to the formula, for flow over a stone bed, there is a relation between the critical flow velocity U_{kr} and the submerged density (Δ) and diameter of the stone (D). A distinction has to be made for the stones in front of the dam head and on the slope of dam head. In front of the head the stone bed may be part of the bottom protection, which gets the determining flow condition at the moment the damhead approaches. To account for the three dimensional flow conditions Shields formula has to be amplified by a factor K:

$$\overline{U_{kr}} = \frac{1}{K} * C * \sqrt{\frac{\Psi}{g}} * \sqrt{\Delta g D} \quad \text{...(12.1)}$$

For the stability of the stones on the front slope of the dam head, another correction factor is used. The formula then is:

$$\overline{U_{kr}} = \log(3 * \frac{h}{D}) * \sqrt{\Delta g D} \quad \text{...(12.2)}$$

An advantage of the gradual progress of the dam head is that the line of eddies also shifts and so does the flow attack on the bottom. Time for scour downstream of the protected bottom, is short as long as dam head construction proceeds without interruption. While progressing with the closure, the potential head over the dam increases and seepage flow will increase. Down the side slope of the dam at the side of low water level slope stability may then be critical. If this is a determining condition, timely reshaping of the initially made dam profile has to be done.

When closing vertically, the flow pattern will be completely different. Building up layer after layer the flow will reach a stage in which critical flow starts. Stability of the profile depends on its shape which depends on the way the dam is built.

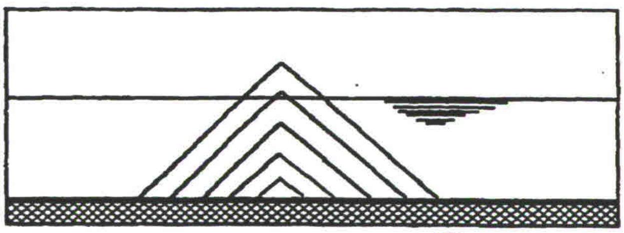


Figure 12-10, Profiles in line-dump

There are two different ways, dumping in a line or dumping in wide layers. Dumping in a line along the dam axis, for instance from a temporary bridge or a cableway, creates a steep triangular profile, gradually increasing in height. The stones on the top are the least stable and may tumble down the slope locally. Since the number of stones in the top is small, the level of the top will be very irregular. This is a negative aspect as long as the top of the dam is submerged. This is particularly true in case of critical flow, since peak flow velocities are higher in these depressions in the dam. Later, when the dam has reached a more elevated level, the downstream slope will be the most vulnerable for instability. Seepage flow together with the overflow of water over the top can take the stones out of the steep profile. This method requires a high initial investment in the construction of a temporary bridge or cableway but has the advantage of being able to reach any place along the alignment at any time to proceed with construction or to repair the profile.

A different profile can be obtained by dropping the stones over a broad band instead of in a line. Then, a trapezium-shaped sill can be heightened layer by layer. This has many advantages. First, in contrast with the line-dump, every successive layer contains less material for the same rise. Creating the top thus takes little time, shortening the critical period. Second, the slopes of the dam sides can be created in a flatter profile, dependent on the width of every individual layer. Thus stability of the profile will be much higher. Third, a few dislodged stones will not create a depression in the sill over the full cross-section.

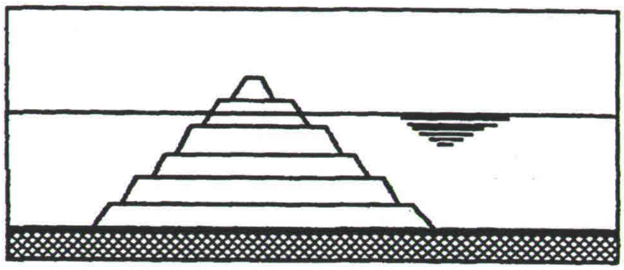


Figure 12-11, Profiles in horizontal layers

The operational side of the latter method, is more difficult, however. It requires specialized equipment, regular surveying and good positioning equipment. At first it may be possible to drop stones using dump barges or split barges. The barges can be used head-on in the flow, as the width of the band will fit the length of the hopper. As the sill gets higher, the keel clearance of the vessels reduces and finally the process cannot proceed any more. The last couple of meters has to be done in a different manner. Cranes, conveyors or the like have to be used to reach outboard and that limits the capacity very much. Besides, the operation with vessels is increasingly difficult as result of the worsening flow conditions.

For stability calculations of the stones on the sill, the formula of Shields is also used. However, the relation has to be adapted for the typical phases, mentioned above, of a vertically built stone dam under construction. The triangular or trapezoid profile and the broad sill are not in accordance with Shields test conditions. Therefore the formula can be adapted into:

$$\overline{U_{kr}} = (f + 1.4 * \log\frac{a}{D}) * \sqrt{\Delta g D} \quad \text{.......................................(12.3)}$$

in which a is the water depth above the sill and the factor f goes from 1 for the very first horizontal layer via 0.75 for a broad sill to 0.25 for a very sharp crested sill.

In a vertical closure the most critical situation occurs if, in a rather advanced phase, a local instability leads to a depression of some extent in the sill's crest. The vertical closure method is used generally to limit the increase in flow velocity, usually in order to use smaller size stones. In a major depression the flow concentrates and reaches much higher values than accounted for. Thus, the stone underneath in the depression is not stable either and the dip will soon deepen considerably. The vertical closure is changed into a horizontal gap. Most likely, the bottom protection will not be designed for those conditions and the failure will become a major disaster.

Due to the difficulty to construct the toplayers of the dam, generally, the last phase of closure will be a horizontally built top on the vertically built trapezium. Then, three-dimensional flow patterns develop. This has to be considered in the stability calculations and some extra safety in the dimensions should be taken.

misschien eerder bij 12.7.4.

12.8.5 Caissons, closed or provided with sluice gates

Caissons, in closure design, are large, artificially made structures or vessels used to block a final closure gap in one effort or in a few major steps. In emergency cases existing ships, pontoons or the like have been used, sometimes after adaption to fit the gap dimensions. In normal circumstances caissons are specifically designed for the purpose. Generally they are made of concrete, have a box shape and are self floating. Three typically different systems can be distinguished:

- The final gap is closed in a single operation by placing one or a few caissons simultaneously.
- Several identical units are made, which together fit into the gap. They are placed one after the other in period of several days.
- Several units are used of which a number (or all) are provided with sluice gates. Every unit is placed with its gates closed, but after stabilising of the caisson, the gates are opened. As soon as all caissons are rigidly in place, all the gates are closed at a suitable moment.

Which system is used depends on circumstances and conditions. They are progressively more expensive.

The caisson is intended to block (part of) the gap and thus will be positioned transverse in the flow. Since dimensions are generally considerable, even small flow velocities will result in high forces for manipulating and positioning of the caisson. Therefore placing will always be done during slack water when the tide turns. In practice the moment with flow velocity zero does not exist. Generally the tide starts turning near the bottom first and later at the surface and usually not over the full gap length at the very same moment. Slack water therefore is the period (time window) in which velocities are smaller than about 0.5 m/s in either direction. In a tidal cycle there are two of these periods, during high-water when flood changes into ebb and during low-water from ebb into flood. A number of considerations determines which of the two is selected:

- duration of that time window, which is not the same for the two slack periods.
- available keel clearance in the approach route of the caisson; sailing during high water may be preferred.
- draft and stability of the floating caisson, the ballasting operation and the sinking depth.
- the desired waterlevel in the basin after closure.
- the way of placing and the equipment used.

The last item relates to the fact that caissons are preferably positioned by pushing (or pulling) them against the current flow direction. The advantage is that if something goes wrong, the caisson is pushed back by the flow in the free space, while in the opposite case the caisson may float into the gap and get damaged or cause damage. The procedure thus starts by bringing the caisson into the gap against the diminishing flow well before the moment of still water. The most commonly used way of bringing the caisson into the gap is to position it, in advance, head-on on one side of the gap, to connect a corner to a fixed point on the shore (as a hinge) and to swing it around that point like a door into the gap. Then, by ballasting, the caisson is lowered and put down just before (or at) slack. Further ballasting will stabilize its position while the flow direction, for flow underneath and around the caisson, turns.

For an operation at high water slack, when the tide turns from flood into ebb, the positioning needs to be done opposite the flood flow, thus from the basin side towards the open water. Therefore, the caisson has to be sailed into the basin via the gap during a preceding high water period (assuming the fabrication dock to be outside the basin). If the (last or the only) caisson is pushed into the gap by tugs, they become trapped in the basin.

After the caisson has been put onto the foundation bed, four aspects are important:

- the load of the caisson onto the bed should be well-spread, which defines tolerance of the bed level and the structural strength of the caisson.
- flow underneath the caisson will soon reach high values but piping (with scour of bed material) should be prevented.

- the gap size needs to be longer than the length of the caisson to allow for tolerances and diagonal length during the swing motion; however, outflanking of the flow along the sides has to be blocked immediately.
- the caisson will soon be subjected to a high potential head, which will try to either shift or overturn the caisson; generally, a linear decline of the potential head underneath the caisson is assumed. However, if permeability of the bed is lowest at the downstream side, the upward pressure is higher than average. Besides, seepage flow in the bed material concentrates along the lower edge.

An example of an unusual concept of closure in which these aspects can be demonstrated clearly, is the closure in 1978 of the Miele, a main gulley in the tidal flat area near Meldorf in the north-west of Germany. The originally planned closure method failed and left a closure gap with limited possibilities to enforce a closure before the next winter. The gap was 320 m wide and the bottom elevation was 3.60 m below MSL. The tide had a range of 3.5 to 4 meter. It was decided to try a closure by caissons, adapting five identical sand transport barges. A new bottom protection had to replace the distorted one.

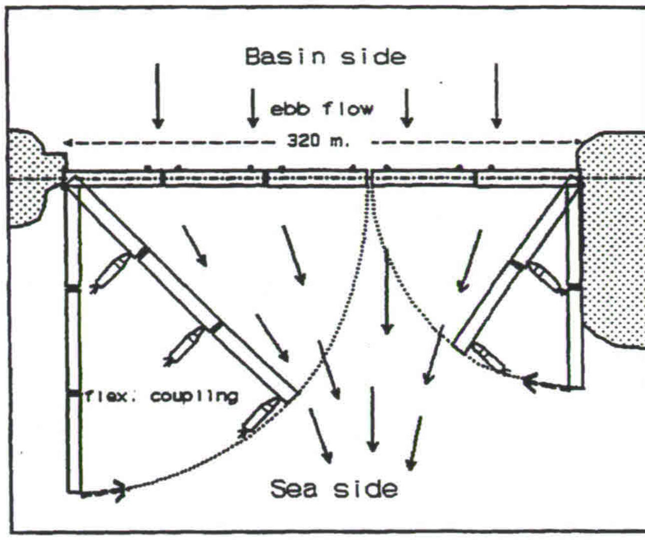


Figure 12-12, Caisson closure in the Miele (Meldorf)

A sill had to be created as foundation for the barges. The limited water depth did not allow a high sill, not even large stones. Therefore flow velocities had to be kept low and the five barges could not be placed one after the other. The problem was how to put one composite caisson, consisting of five rather fragile steel barges onto a 320 m long sill without risk for breaking, nor for piping underneath, nor for piping at the ends. The solution found was as follows:

The barges were provided with heavy steel H-profiles underneath, along the two sides, to improve longitudinal strength, to improve penetration into the bed material and reduce free-spanning along uneven bed levels. For stability calculations it was assumed that the bed underneath the downstream H-profile would be the most impermeable and determine the upward water pressure. The barges were assembled into two composite caissons, one consisting of two and one of three barges. The connection between the barges was made by flexible material which allowed every barge to settle independently (within reasonable limits). Much attention is given to smoothen the sill to avoid high spots which would pierce the barge bottoms. The two caissons were positioned near the gap at the two shore ends, where they were connected to a hinge (pole) by steel wire. Both caissons were swung into the gap simultaneously at low water slack. For ease of positioning, to prevent the "doors" to swing too far, steel tubes had been piled in the alignment of the gap. Tugs on the seaward side had to gently push the caissons against these tubes. Being in line, the caissons showed a wide slit where they met. By pushing from the shore-ends and releasing the wire hinges, the slit was closed, while the space was divided over the two shore connections. Ballasting was done by pumping water and sand into the barges. Stones were dumped in the shore end slits and via a floating pipeline sand was pumped at high capacity along the full length of the caissons to prevent piping. As soon as the rising water allowed dump vessels to sail, stone was dumped alongside the barges also. (After a sand dike was provided at the basin side of the caissons, the barges were emptied, refloated, refitted and taken back in normal operation again.)

Generally several caissons will be placed one after the other in a period of several days. Every caisson blocks part of the gap's profile and the next caisson will be more difficult to position. Flow velocities increase, time window diminishes and the turbulent eddies in front of the caisson will be more severe. Although the program will try to work from spring tides to neaps, the last caisson to be placed will be determining for the design of caisson dimension and foundation bed material. The advantage of this method is that the operational phase in which flow velocities are very high is relatively short. This means that in areas with limited workable period, for instance because of weather or river discharges, the progress is within schedule. Besides, the duration of exposure to high flows with large eroding capacity is small.

For large closures the last caisson may need unrealistic dimensions. Then, the use of caissons provided with sluice gates is a good option. In that case, every caisson blocks a small part of the gap profile only viz. side walls, diaphragm walls, bottom structure and ballast hold. The gates will provide an opening of 80 to 85% of the submerged section of the caisson, which is to be multiplied by a discharge coefficient in the hydraulic calculations. Again the determining conditions are those during placing of the last caisson. This flow condition determines the dimension of the total opening provided by the gates. Multiplication by 1.3 gives the total gap size to be blocked by caissons with gates.

List Of references Chapter 12

1. Anonymous, (1995) "manual on the use of rock in hydraulic engineering" CUR/RWS Publication no. 169, June 1995, CUR Gouda, The Netherlands.

13. Failure modes and optimisation

13.1 Introduction

For a long time, the design process of rubble mound breakwaters was not very analytical. Often, a design wave height was selected on the basis of a limited number of field data. The final design was then tested in a hydraulic model. In general, a geotechnical study completed the efforts.

In the hydraulic model study, the hydraulic stability of the cross section was verified when exposed to the design wave. Although it was evident that this design wave height could be exceeded, seldom waves higher than the design wave were used in the model. Scatter in the model results and inconsistencies in the model procedures were hardly accounted for. It was also not common to use safety coefficients to cope with uncertainties in load or resistance of the structure.

In this way, it could happen that new armour units like the Dolos were developed with a very good hydraulic performance. It was not recognised, however, that the mechanical strength of such units was insufficient to withstand the impact forces under design conditions. In the same way, it was not always recognised that the margin between initial damage and failure of the armour was different for armour layers consisting of traditional quarry stone and the newer artificial armour units. This caused an unnoticed reduction of the inherent safety factor of the traditional structure.

The failure of a number of large rubble mound breakwaters triggered a more analytical approach to the design of such structures. One of the first systematic works on this subject is the CIAD report[1] on the reliability of breakwater design. It was soon followed by a more comprehensive study of PIANC PTC II Working Group 12[2]. The PIANC study did not only present methods for probabilistic analysis at level I, II and III; it also gave recommendations for the values of partial safety coefficients based on a general probabilistic analysis.

In the mean time, PIANC started a new working group to carry out a similar study for monolithic breakwaters. The report of this working group is expected in 1999.

For closure dams a systematic probabilistic analysis has been published, as Annex A in the CUR/RWS Manual on the Use of Rock in Hydraulic Engineering[3]. This is not remarkable because at least a few failures have occurred in the Dutch design practice:

- during the closure of dike breaches at Walcheren (1945),
- during the closure of dike breaches in 1953 in the SW part of the Netherlands, and
- during the closure of the Markiezaatskade as part of the Delta Project.

13.2 Failure mechanisms

For a proper insight in the behaviour and reliability of a structure under design conditions (and excess of design conditions), it is necessary to have a more or less complete idea of potential failure modes or failure mechanisms. A failure is defined as a condition when the structure loses its specified functionality. This can either be connected with a serviceability limit state or with an ultimate limit state.

For breakwaters, the protective function is the most important one in most cases. Failure is thus related to any damage that leads to unwanted wave penetration into the harbour, followed by further structural and/or operational damage.

The CUR/RWS Manual[3] gives a general overview of failure modes of rock structures (*Figure 13-1, Failure modes of Rock Structures*). Burcharth[4] presents a slightly different review (*Figure 13-2, Failure modes for a rubble mound breakwater according to Burcharth*).

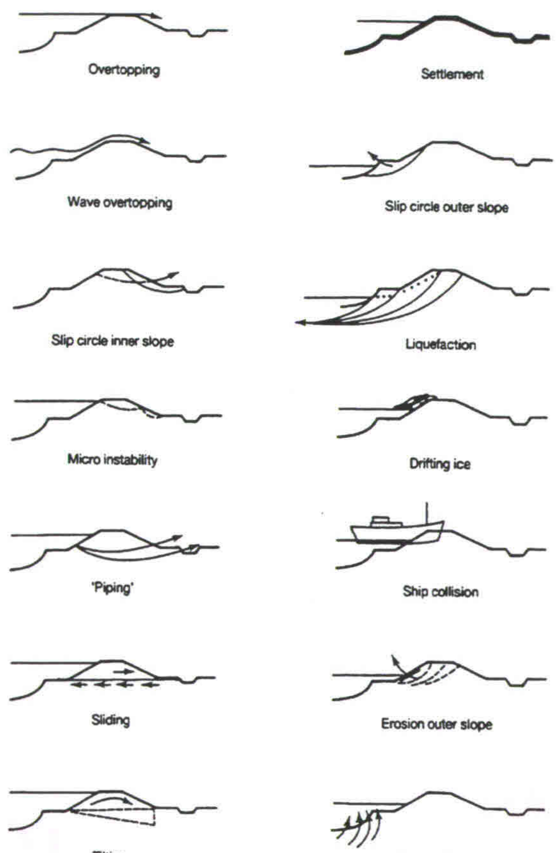


Figure 13-1, Failure modes of Rock Structures

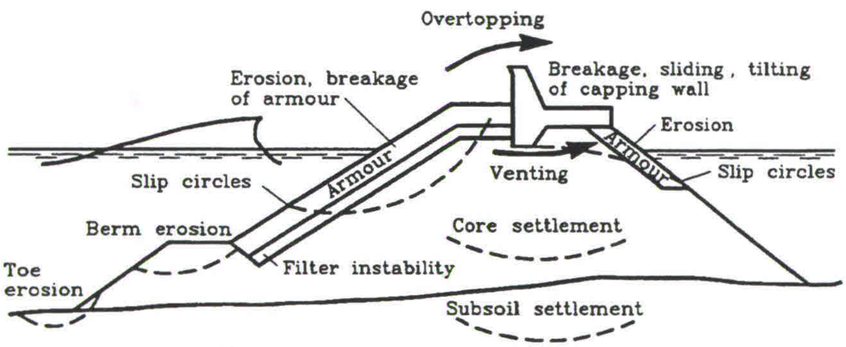
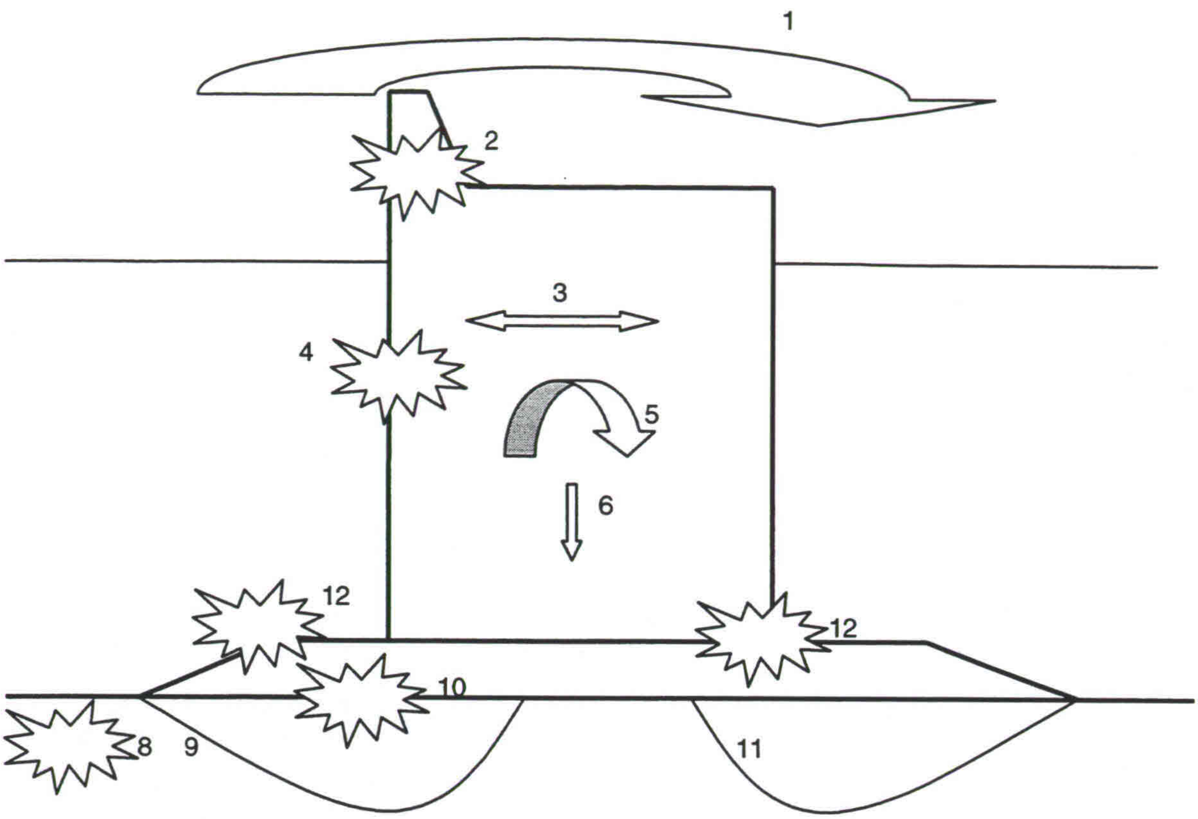


Figure 13-2, Failure modes for a rubble mound breakwater according to Burcharth

These overviews are given with some reluctance because it is not yet feasible to give properly defined limit states for each of the failure modes separately in terms of load, resistance and scatter of results.

The same applies for monolithic breakwaters. For this type of breakwaters some failure mechanisms have been assembled in *Figure 13-3, Some Failure Mechanisms for Monolithic Breakwaters.*



1. Overtopping
2. Structural Failure of Parapet
3. Translation
4. Structural Failure of Front Wall
5. Rotation
6. Settlement
7. Erosion of Filter Layer
8. Scour
9. Forward failure of foundation
10. Washing out of fines
11. Backward failure of foundation
12. Failure of Filter Layer due to rocking

(Figuur is CUR op zand naar afsluitdammen)

Figure 13-~~3~~2, Some Failure Mechanisms for Monolithic Breakwaters

13.3 Fault trees

A structure can be schematised as a complex system, consisting of many components, which may function or fail. A fault tree sketches the systematic relations between failing or functioning of all components in their mutual, interactive relation. Failure of a component may or may not trigger the malfunctioning of a component at a higher level, until eventually the structure as a whole does not perform the functions for which it was meant. Failure of the structure as a whole may also occur if two uncorrelated events happen at the same time. Considering these options, one can point at "AND and "OR" gates, indicating parallel and serial relationships. By quantifying the probability of failure of each component, and by combining the various causes of failure, it is possible to assess the overall probability of failure of the system, be it a breakwater or a closure dam. It is beyond the scope of this book to enter deep into the theory of fault tree analysis. The reader is referred to more specialised books on reliability theories. Just for illustration, a simplified fault tree and the related calculation of the probability of failure are given in *Figure 13-4, Fault tree and probability of failure after CUR/RWS.*

A complete fault tree analysis reveals the contribution of each failure mechanism to the overall probability of failure for the complete structure.

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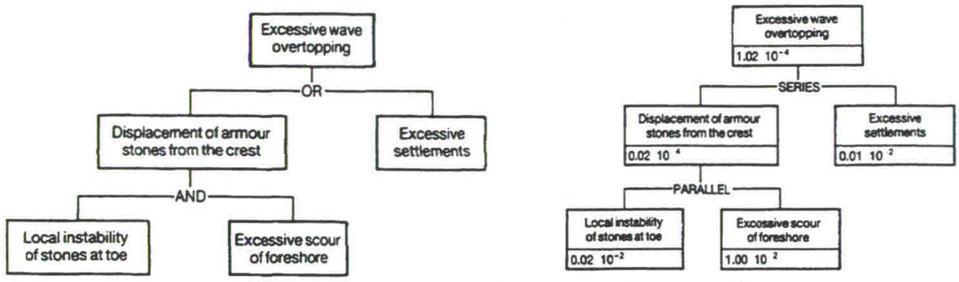

Figure 13-4, Fault tree and probability of failure
after CUR/RWS

The probability of failure for each component of the system can be determined by making a preliminary design and assessing the uncertainties in load and resistance (strength) via a reliability parameter Z. This can be carried out at different levels of sophistication. At present, "Level II" methods are the most commonly used. Apart from the probability of failure, the Level II calculation specifies the relative contribution of each load and strength parameter. For breakwaters, the uncertainty of the wave climate is often the most important contribution to the probability of failure on the loading side; on the structural side it is the scatter in the stability formulae and the inaccuracy in the nominal diameter of the armour stone (D_{n50}). In this way it is possible to analyse what are the most promising measures to improve the probability of failure if necessary.

The next question is then whether the calculated probability of failure for the system is acceptable or not. After a lengthy study to quantify the probability of failure it is highly unsatisfactory to make this decision on an irrational basis.

It is then wise to quantify the risk of failure in terms of the product of probability (of failure) and its consequences (damage). These consequences are not limited to the cost of the failing structure, but include the consequential damage as well. For a fully destroyed breakwater the damage thus represents the cost of reconstruction of the breakwater plus the delays in the port operations as a result of it. The risk (being the product of probability and cost of damage) is expressed in terms of cost per unit of time (generally per annum).

It is possible to reduce the risk by strengthening the structure. In general this can only be done at (some) extra cost. In this way, the construction cost of the structure increases, but the risk reduces, mainly due to a lower probability of failure. Since the construction cost is expressed in actual (money) value at the moment of construction, it is necessary to capitalise the annual risk due to failure over the lifetime of the structure, and calculate its present day value. Then, the extra construction cost can be compared with the savings on the capitalised risk.

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In practice, the situation is more complicated, because it is not only the risk of failure that has to be accounted for, but also the risk of partial damage, resulting in the need for maintenance and repair. A second complication is the fact that often there are several ways to improve the strength of a structure, and it is not always clear what is the best (most economic) way to do so. This macro and micro optimisation process is discussed in 13.4.

13.4 Optimisation

13.4.1 Micro level

Optimisation at micro level can best be explained in the deterministic design process. It aims at a design that leads to the minimum total cost for a given strength level. To achieve this goal, it is necessary that all material in the structure fulfils its function, and is used in the optimum way.

This can be compared with designing a frame. It is then attempted to select the members such that all are exposed to a stress level close to the maximum admissible stress. In the same way, it can be attempted that all elements in a closure dam or a breakwater are close to (partial) failure when exposed to the design wave height.

In a probabilistic design process, it means that one should avoid a very large contribution to overall failure by a single partial failure mechanism while other mechanisms do not at all contribute to the probability of failure. It is wise to distribute the contribution to overall failure over a number of failure mechanisms. In fact, one should base this distribution on considerations of marginal cost. If a construction element is relatively cheap it is not so much of a problem if it is over-designed. If it is relatively expensive, over-designing in comparison with other elements leads to too high cost.

It means that the designer shall attempt to make a balanced design. This can easily be explained when considering the cross section of a rubble mound breakwater. If the crest level is designed so high that no overtopping occurs even under severe conditions, it makes no sense to protect the inner slope with heavy armour stone. For a low crested breakwater on the other hand it is essential to carefully protect the inner slope.

13.4.2 Macro level

Also optimisation at macro level can best be explained in the deterministic design process, when only one failure mechanism with simple load and strength parameters is considered. When more mechanisms and parameters play a role, the calculations become rapidly more complicated, and one should be careful not to make mistakes that lead to completely false conclusions.

The method was developed by Paape and van de Kreeke[5] for rubble mound breakwaters as early as 1964. The method is discussed in the following, and a sample calculation is given in Annex 5 References to Tables and Figures refer to that Annex.

klopt.

The method starts with the assumption that there is a direct relation between one load parameter (the no damage wave height, H_{nd}) and a strength parameter (the weight of the armour units, W). It is further assumed that the wave climate is known and available in the form of a long-term distribution of wave heights (Table 1). The interaction between load and strength is determined on the basis of laboratory experiments, which indicate that damage starts when a threshold value (H_{nd}) is exceeded. The damage to the armour layer increases with increasing wave height until the armour layer is severely damaged and the core of the breakwater is exposed. This occurs at an actual wave height $H = 1.45\ H_{nd}$. It is assumed that damage is then so far extended that no repair is possible, and that the structure must be rebuilt completely. For intermediate wave heights, a gradual increase of damage is assumed, expressed in a percentage of the number of armour units to be replaced (Table 2).

The breakwater is then designed for a number of design wave heights, where a higher design wave causes a heavier and more costly armour layer, whereas the core remains unchanged. The cost of construction is I. The cost of rebuilding the breakwater is assumed to be equal to the estimated construction cost, the cost of repairing damage to double the unit price of the armour units. It is then possible to list the construction cost and the anticipated cost of repair, still split over the three categories of damage (4%, 8% and collapse). Adding up the three categories of damage for a particular design wave height yields the average annual risk anticipated for that design if all damage is repaired in the year the damage took place. If it is decided not to repair the breakwater except in case of collapse, the risk is just the risk caused by the category collapse.

Since the risk is still expressed in a value per annum, it must be ascertained what amount of money shall be reserved at the moment of construction to be able to pay the average annual repair cost during the lifetime of the structure. Although money is regularly spent from this repair fund, it still accrues interest at a rate of δ% per annum.

If the annual expense is s, the interest rate δ%, and the lifetime of the structure T, it can easily be derived that the fund to be reserved (S) is:

$$S = \int_0^T e^{-\frac{\delta}{100}t} dt = s\frac{100}{\delta}\left(1 - e^{-\frac{\delta T}{100}}\right) \qquad (13.1)$$

for T = 100 years S = s.100/δ, and
for T = 10 years S = 0.35s. 100/δ.

The interest rate is generally set in the order of 3.5%.
By adding the initial construction cost I and the capitalised risk S, one arrives at the total cost of the structure. When this total cost (I + S) is plotted as a function of the design wave height, it appears that there is an optimum design wave height or an optimum strength of the structure.

List of references Chapter 13

[1] Anonymous (1985), "Computer Aided Evaluation of the Reliability of a Breakwater Design" Final Report of the CIAD Project Group Breakwaters, ISBN 90-6818-019-3 CIAD, Zoetermeer, the Netherlands.

8 [2] Anonymous (1992), "Safety of Rubble Mound Breakwaters" Report of PTC II Working Group 12, PIANC, Brussels, Belgium.

8 [3] Anonymous (1995), "Manual on the Use of Rock in Hydraulic Engineering" CUR/RWS Report 169, CUR, Gouda, the Netherlands.

[4] Burcharth, H.F. ,(1992), "Reliability Evaluation of Structures at Sea" Proc. Of the Short Course on Design and Reliability of Coastal Structures, Venice 1992, ed. Tecnoprint Bologna, Italy.

[5] Kreeke, J. van de , and Paape, A. ,(1964) "On Optimum Breakwater Design" Proc. 9th ICCE, ASCE, New York, USA.

14. Examples, Alternatives and Cases.

14.1 Closing an Estuary, creating Final Gaps in the Tidal Channels.

In the foregoing chapters various details of examples and cases have been given in relation to the subject discussed. However, a plan showing the construction phases of the closure of an estuary entrance, comprising several channels and tidal flats was not detailed yet. In this chapter, a few examples of a hypothetical closure will demonstrate various possibilities. A number of alternatives will be outlined and the relation with some historic cases will be discussed. Data on flow velocities and discharges is taken from mathematical calculations, which are not detailed. However, data relevant for the motivation is presented.

The example assumes a tidal estuary which has to be closed along a fixed alignment. The longitudinal profile of the total closure consists of (see Figure 14-1: phase 0):

- a foreshore, 250 m wide, 0.5 m lower than mean sea level.
- a secondary gulley of 200 m width and an average depth of 4 m below mean sea level (MSL),
- a tidal flat 300 m wide, with a level of about MSL,
- the main gulley 250 m wide, with an average depth of 6.5 m below MSL and the largest depth along the bank.

The longitudinal profile of the closure gap thus is 4000 m^2 at high and 1800 m^2 at low water. The tide is a semi-diurnal sine wave with a range of 3 m. In all calculations the tidal range is taken constant; neap-spring variation is ignored.

The storage area of the basin is 20 km^2 at high and 5 km^2 at low water.

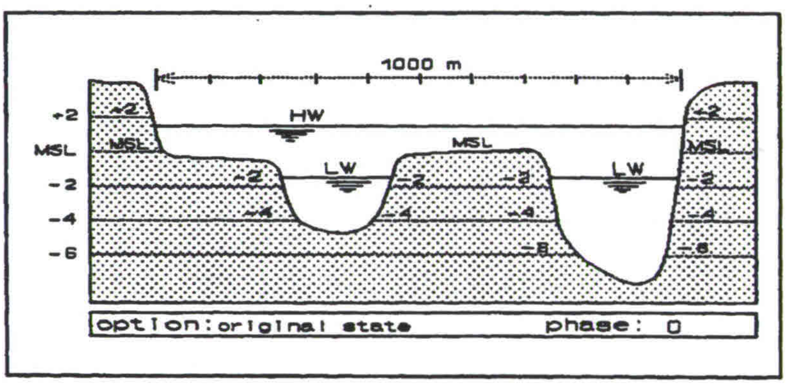


Figure 14-1, Definition sketch of case

Three main options will be studied:

a) dam sections across the shallows first, next closing the gulleys. (in this section)
b) dealing with the gulleys first and closing the shallows last. (in section 7.2)
c) all simultaneously. (in section 7.3)
Each option may have several alternatives.

The option "shallows first", is detailed below.

Dam sections across the shallows will create two gaps. Then, many possibilities exist, but a few alternatives are unattractive. The secondary gap for instance is very shallow for using caissons. And for a vertical closure, the total length of the (two) sills, 450 meters only, is rather short (this will be clarified when detailing the options b and c). A mathematic model is needed to get the required data for a well-considered decision. Horizontal closure by tipping quarry stone in both gaps is a very good possibility, but for the purpose of this example, a combination of placing caissons and tipping quarry stone will be detailed.

In the next three figures some construction phases are presented. The program reads:

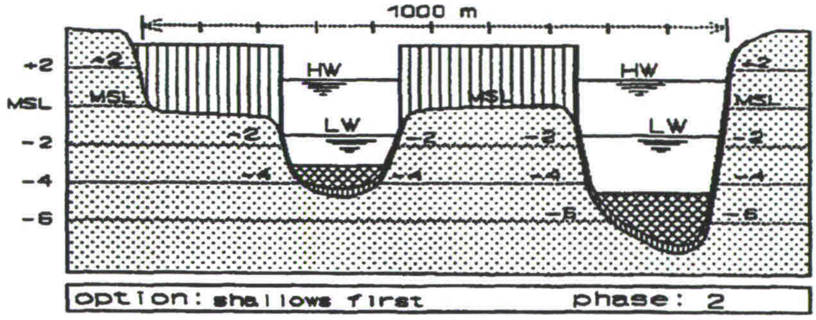


Figure 14-2, Phase 2 of the shallow first option

Two gaps are created by damming the shallows. Bottom protection is provided for in the remaining gaps. Then, both closure gaps are slightly diminished in sectional profile by creating sills. For a caisson closure abutments are also made. These are concrete structures or sheetpile walls which shape the vertical sides of the closure gap. Next, caissons are positioned in the main gap and finally the secondary gap is closed by tipping quarry stone.

cijfers weergeven in de figuur
tabellen zelf maken

Expressed in phases:

phase and action	foreshore	sec. gulley	tidal flat	main gulley
0 original state	250m; -0.5	200m; -4	300m; MSL	250m; -6.5
1 bott.prot. + shallows	dammed	200m; -3.5	dammed	250m; -6
2 partial sills in both	dammed	200m; -3	dammed	250m; -4.5
3 *final sill, abutments*	*dammed*	*200m; -2.5*	*dammed*	190m; -4.5
4 first caisson in place	dammed	200m; -2.5	dammed	128m; -4.5
5 sec. caisson in place	dammed	200m; -2.5	dammed	66m; -4.5
6 third caisson in place	dammed	200m; -2.5	dammed	closed
7 narrowing on sec. sill	dammed	100m; -2.5	dammed	closed
8 further narrowing	dammed	50m; -2.5	dammed	closed
9 very last gap	dammed	10m: -2.5	dammed	closed

Table 14-1, Different phases in closure process

As the two gaps are blocked virtually one after the other, there may be quite some imbalance in the tidal system between the gaps. This may require extra measures. To prevent this, the tipping into the secondary gap has to run together with the placing of the caissons. The risk then is that the flow conditions during the sinking of the last caisson are too high. In fact, this is already questionable in the above phasing.

Checking on these conditions gives the data in the next table. At the moment that two caissons are placed (phase 5) the maximum discharge via the secondary gap has doubled and reaches about the same magnitude as the main gap had originally. To the contrary, the main gap halved its discharge. The secondary channel will have to accommodate the doubled quantities, for which its profile will be quite small. A scouring of a gulley across the shallow from the main to the secondary channel therefore is the likely consequence of the imbalance.

(The values in the table have been calculated for the same tide variation at sea; possibilities of using spring/neap variation is discarded. Maximum flow does not necessarily coincide with maximum discharge, neither do the maxima of the two gaps always coincide)

phase	situation (m/s,m³/s)	secondary gap during ebb Umax	Qmax	secondary gap during flood Umax	Qmax	main gap during ebb Umax	Qmax	main gap during flood Umax	Qmax
0	orig.	1.09	915	1.07	940	1.09	1810	1.07	1825
1	bp+dams	1.33	1010	1.27	1045	1.33	2070	1.27	2085
2	sills	1.67	1065	1.57	1135	1.67	1935	1.57	1995
3	abutm	2.12	1090	1.94	1215	2.12	1790	1.94	1865
4	1 placed	2.71	1305	2.39	1505	2.57	1385	2.26	1470
5	2 placed	3.57	1550	3.00	1875	3.19	820	2.69	895

Table 14-2 , Velocities during different phases

(phase 2 is pictured in Figure 14-2)

The positioning of the last caisson takes place in the situation of phase 5 during HW-slack, as at LW there is insufficient water depth. At the end of the flood period the flow diminishes as follows:

- 30 min. before slack: u = 1.50 m/s,
- 20 min. before slack: u = 1.20 m/s,
- 10 min. before slack; u = 0.70 m/s.

For a safe sinking operation, these values are far too high. Consequently, the program has to be adapted. Instead of using ordinary caissons, they can be equipped with sluice gates. The program then reads:

phase and action	foreshore	sec. gulley	tidal flat	main gulley	sluice gate
4 first placed, opened	dammed	200m; -2.5	dammed	128m;-4.5	56m; -3.5
5 sec. placed, opened	dammed	200m; -2.5	dammed	66m; -4.5	112; -3.5
6 third caisson placed	dammed	200m; -2.5	dammed	0m;	112; -3.5
7 narrowing on sill	dammed	100m; -2.5	dammed	0m;	112; -3.5
8 further narrowing	dammed	50m; -2.5	dammed	0m;	112; -3.5
9 very last gap in sec.	dammed	10m; -2.5	dammed	0m;	112; -3.5
10 close sluice gates	dammed	dammed	dammed	0m:	closed

Table 14-3, Phases with sluicegates in caissons

(phase 4 is pictured in Figure 14-3)

This time, the positioning of the last caisson takes place in phase 5 with the sluice gates of the two other caissons opened (assumed to have 56m effective width each and a floor thickness of 1m).

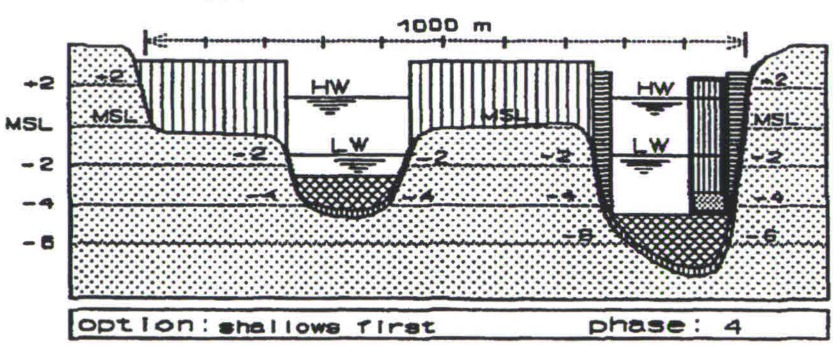


Figure 14-3 , Shallow first option with sluice gates

Then, at the end of the flood period the flow conditions appear to be acceptable:

- 30 min. before slack: u = 0.75 m/s,
- 20 min. before slack: u = 0.50 m/s,
- 10 min. before slack; u = 0.20 m/s.

At first, the balance between the gaps will be better than in the former schema, as the gates provide a flow profile while tipping starts in the other gap. Although the secondary gap is closed before the gates close, the main channel does not exceed its original discharge. The balance can still be improved by closing a number of gates simultaneously with the tipping. However, that worsens the conditions for the tipping.

The flow conditions are:

		secondary gap				main gap **			
phase	situation	during ebb		during flood		during ebb		during flood	
	(m/s,m³/s)	Umax	Qmax	Umax	Qmax	Umax	Qmax	Umax	Qmax
5	1+2, open	2.60	1260	2.30	1445	2.32	1460	2.06	1580
6	3 placed	3.35	1480	2.85	1775	2.82	965	2.40	1095
7	100m gap	3.87 *	830	3.40	1040	3.67	1155	3.03	1360
8	50m gap	3.78 *	410	3.57	535	3.95 *	1220	3.36	1485
9	10m gap	3.62 *	80	3.58	105	4.05 *	1245	3.58	1560

Table 14-4 , Flow conditions in new construction scheme

grote
letter
titels.

* means critical flow.
** via the sluice gates
(phase 7 is pictured in Figure 14-4)

Critical flow occurs during the ebb, also in the sluices. Per tidal cycle it lasts for nearly 2 hours in phase 8 and for 2.5 hours in phase 9. Probably, this can be prevented by providing the third caisson with sluice gates as well (which was not required for the sinking operation). If so, the conditions for the tipping of the stone are better, but the imbalance between the gaps increases. Whether it is worth the extra expense, depends on the savings on stone tipping and bottom protection.

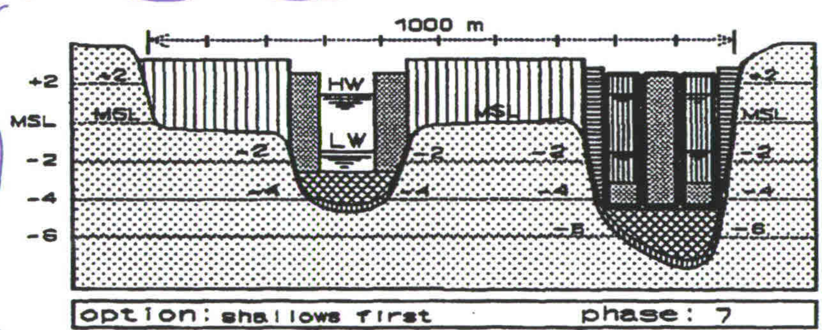


Figure 14-4 , Phase 7 of the closure process

The flow and discharge conditions for three sluice caissons are:

		secondary gap				main gap **			
phase	situation	during ebb		during flood		during ebb		during flood	
	(m/s,m³/s)	Umax	Qmax	Umax	Qmax	Umax	Qmax	Umax	Qmax
7	100 m gap	3.35 *	720	2.80	875	3.14	1570	2.63	1805
8	50m gap	3.55 *	385	3.09	480	3.51	1695	2.91	1980
9	10m gap	3.49 *	80	3.15	100	3.81 *	1780	3.15	2120

Table 14-5, Flow for various caiissons

Critical flow in the sluices now occurs during the very last operation and lasts for half an hour only. Maximum flow velocities in the secondary gap reduce by about 10%.

A historic case of the above system of closing a tidal basin is the construction in 1965 up to 1972 of the "Brouwersdam", damming the "Brouwershavensche Gat" by which the "Lake Grevelingen" was created. Dimensions of the channels and the basin were much larger. The total dam alignment had a length of 6 km. In that case the minor gulley was closed by sinking 12 sluice-caissons, each 68 m long, on a sill levelled at 10 m below MSL. The main gulley was closed by gradual closure with concrete cubes of 50 kN each. The profile of this gap was about 13000 m^2. Contrary to the example, the gradual closure was a vertical closure, dropping the cubes by means of a pre-installed cableway. As in the example, a limiting factor for the progress of the gradual closure was the flow condition during positioning and sinking of the last caisson.

14.2 Blocking the Main Channel first.

In this section the same estuary as above is closed by reducing the profiles of the gulleys first. Then the main gulley will be blocked completely. Next, the secondary gulley will be further reduced and finally the total rest profile will be blocked. This option is worth consideration if it leads to a cheaper closure operation. The obvious disadvantage, when the channels are blocked first, is that the bottom across the shallows has to be protected against scour also. Cost savings on the other items need to compensate for this expensive extra. Such savings may result from a possibly reduced dimension of the protection in the gulley and from cheaper caisson design. A major saving would result by using caissons without sluice gates, while another saving could be obtained by using two caissons only.

A determining factor for the decision to omit sluice gates is the positioning of the last caisson. The flow conditions will be best when the flow profile is as large as possible at that moment. This is the case when there is no sill in the secondary gap. Assuming the same dimensions of the caissons, the determining total flow profile is the original profile, diminished by bottom protection over the full length, by the abutments on both sides of the main gulley, by the foundation sill and by the caisson(s) already placed. The HW-slack period then is characterized by:

- 30 min. before slack: u = 0.80 m/s,
- 20 min. before slack: u = 0.60 m/s,
- 10 min. before slack; u = 0.25 m/s.

Positioning will not be a problem. There is however, a substancial imbalance between the two gulleys. The maximum flood via the secondary gulley is 1975 m^3/s, which is more than twice the original. A sill in this gulley, up to the level -3m, brings the discharge down to 1860 m^3/s. The effect is small and flow velocities in the main gulley increase up to:

- 30 min. before slack: u = 0.95 m/s,
- 20 min. before slack: u = 0.65 m/s,
- 10 min. before slack; u = 0.35 m/s.

The sinking is possible, but further raising the sill is not acceptable. The construction of the sill up to -3m in the secondary gulley can best be done simultaneously with the foundation sill in the main gulley. After that, caissons can be placed at short intervals to limit the duration of the imbalance.

Saving on the number of caissons depends on the flow conditions for creating the smaller gap for the (two) caissons, as the narrowing of the gulley has to be done by pumping sand or the like. The flow profile available is the original profile diminished by bottom protection and by the island for the abutment on the shallows. The longitudinal profile then consists of 250 m foreshore, 200 m secondary gulley and 250 m shallows, all provided with bottom protection, an island section on the shallows of 50 m length and adjoining island in the main gulley along 75 m, leaving a gap length for two abutments of 25 m each, two caissons of 60 m each and 5 m extra (see figure). In the secondary gap, bottom protection will be present and some of the sill construction may exist. The calculations show that the maximum flow velocities in the gap are 1.70 m/s during ebb and 1.60 m/s during flood, which is no problem for the construction of the island.

The conclusion is that blocking the gulleys first can be done by closing the main gap with two simple caissons, while a restricted sill is present in the secondary gulley. The remaining flow profile consist of two 250 m long shallow sections and a partly blocked gap in between.

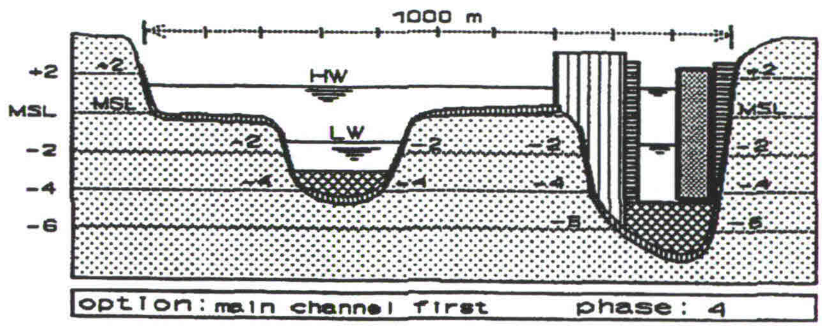


Figure 14-5, Phase 4 or of main channel first option

The construction phasing up to this moment thus reads:

phase and action	foreshore	sec. gulley	tidal flat	island	main gulley
0 original state	250m; -0.5	200m; -4	300m; MSL	none	250m;-6.5
1 bott.prot+island	250m; MSL	200m; -3.5	250m; +0.5	125m	175m;-6
2 sills in both	250m; MSL	200m; -3	250m; +0.5	125m	175m;-4.5
3 sill, abutments	250m; MSL	200m; -3	250m; +0.5	150m	125m;-4.5
4 first caisson in pl	250m; MSL	200m; -3	250m; +0.5	150m	65m;-4.5
5 sec. caisson in pl	250m; MSL	200m; -3	250m; +0.5	-	closed -

Table 14-6, Phasing of main channel first option

Flow velocities and discharges are as follows:

		secondary gap **				main gap			
phase	situation	during ebb		during flood		during ebb		during flood	
	(m/s,m³/s)	Umax	Qmax	Umax	Qmax	Umax	Qmax	Umax	Qmax
0	orig.	1.09	915	1.07	940	1.09	1810	1.07	1825
1	botpr+isle	1.61	1155	1.52	1215	1.61	1695	1.52	1720
2	sills	2.01	1175	1.85	1295	2.01	1525	1.85	1600
3	abutm	2.42	1355	2.19	1525	2.29	1205	2.07	1285
4	after 1st	3.06	1585	2.68	1860	2.73	705	2.40	770
5	after 2nd	3.98 *	1860	3.37	2310	closed	0	closed	0

Table 14-7, Flow velocities during the various phases

** the central 200 m section only (the shallows falling dry during low tide).

The next step could be a horizontal closure by tipping quarry stone, from either one side or from both sides. It creates high flow velocities in the secondary gap. The situation is comparable with the former option, except for the sluice gates. The flow velocities will rise in this case to about 4.5 m/s. Therefore, it is more appropriate to try to reduce the profile of the secondary gap, maintaining the flow across the shallows. Dumping quarry stone by dump-vessels will be impossible because of draught restriction. However, vertical closure is possible by means of a temporary bridge (to be installed in the previous period) or a cableway (ditto). The length is considerable (700 m) but 500 m of the bridge crosses shallow water and so, the foundation cost is limited.

Another method, with a difficult operational procedure, is to recognise the fact that during LW the dam section across the shallows falls dry for several hours. The first step is to bring the sill level up in two layers to above LW, so from -3 to -2 and then to -1. The water level in the basin will not follow the sea level and the relation between the levels of the sea, the basin and the sill determine the operational possibilities. The equipment to be used is a shallow-draught crane vessel and dump trucks with hydraulic cranes, approaching via the drying dam sections. The operational period per tidal cycle, the work-window, is small and the production is low, but the equipment is available on the market and investment in bridge or cableway can be avoided. Gradually, layer by layer, the sill will be raised. For every layer, the determining moment exists when the last 10 m has to be made. That missing part of the layer is a dip in the sill level which is subjected to higher (critical) flow.

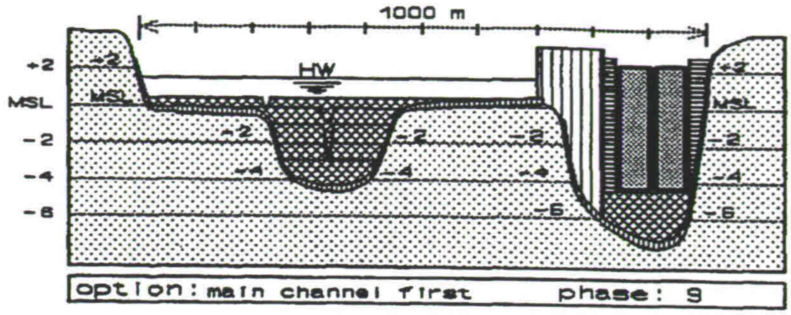


Figure 14-6 , Construction method with shallow sections above LW

The crane vessel can operate when anchored near the gap during the periods that flow velocities are smaller than 2 m/s. For the layer from -3m to -2m (phase 5 to 6) this is 2 hours during HW and 1 hour during LW on average. For the next layer (phase 6 to 7), up to -1m, this is 1.5 hours at HW and three quarters of an hour during LW. After that, dump trucks may start to assist during the LW-period, as the water level at the seaward side will fall lower than the sill level. At the start of the next layer up to MSL (phase 7 to 8), the basin level falls below that level during about 1 hour (b-b in the figure). When finishing the layer, the water levels are less than 0.5 meter above the sill's level for two hours. Though risky, trucks can operate in water depth of less than 0.5 meter.

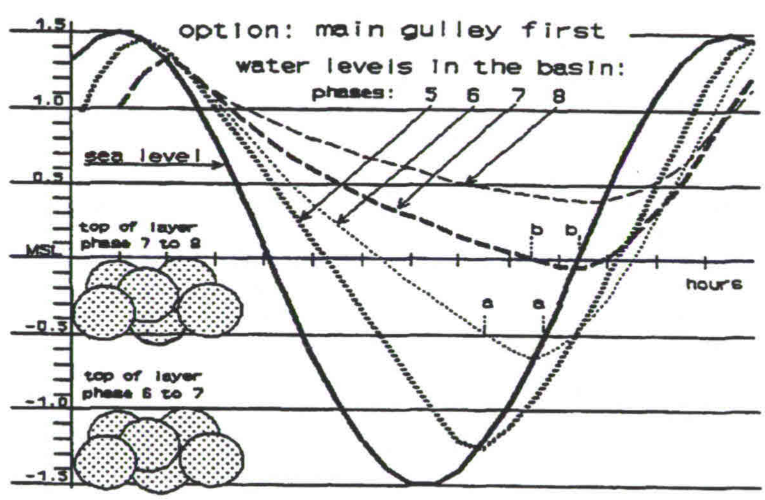


Figure 14-7, Waterlevels during closure

The construction phasing thus continues for the 700 m section vertically: verwijzen naar fig 14-6!

The construction phasing thus continues for the 700 m section vertically:

phase and action	foreshore		sec. gulley		tidal flat
6 first layer	250m; MSL	97m;-2	6m;-3	97m;-2	250m; +0.5
7 first layer	250m; MSL	97m;-1	6m;-2	97m;-1	250m; +0.5
8 level foreshore	250m; MSL	97m;MSL	6m;-1	97m;MSL	250m; +0.5
9 level tidal flat	222m;+0.5		6m;MSL	222m;+0.5	250m; +0.5
10 level +1	347m; +1		6m; +0.5		347m; +1
11 final layer	dammed		6m: +1	dammed	

Table 14-8, Construction phases with dry falling sill

zelfde opzet aanhouden.

The flow velocities for the various levels of the sill are:

Umax. in m/s:		deepest part		deepest but one		deepest but two		deepest but three	
phase	situation	ebb	flood	ebb	flood	ebb	flood	ebb	flood
5	after 2nd	3.98 *	3.37	2.34 *	2.85 *	1.80 *	2.50 *	not applicable	
6	up to -2	4.22 *	3.43	3.81 *	3.84	2.28 *	3.03 *	1.94 *	2.32 *
7	up to -1	3.82 *	3.38	3.28 *	3.68 *	2.38 *	3.15 *	2.02 *	2.32 *
8	up to MSL	3.27 *	2.92	2.50 *	2.91 *	2.06 *	2.32 *	not applicable	
9	up to 0.5	2.32 *	2.67	1.98 *	2.32 *	not applicable		–	
10	up to +1	1.86 *	2.18 *	1.05 *	1.55 *	–			
11	up to HW	0.88 *	1.55 *	high water free					

Table 14-9, Flow velocities over the sill

* means limited by critical flow condition.

In the table the deepest part represents the dip in the sill mentioned above. Although in all sections of the sill critical flow limits the flow velocity, in the dip this is only true for the ebb. Besides, ebb is determining for sill levels up to about MSL, above that flood flows are higher.

Considering the above results, it appears that the maximum flow velocity in the secondary gap during the raising of the sill is not very much less than in case a horizontal closure had been designed (4.22 m/s instead of about 4.50 m/s). This is due to the fact that the limiting critical flow condition for the -2m sill level under these circumstances is about the same as the normal flow condition for a narrow gap with a 3 m tidal range. The determining condition occurs for the dip in the low sill in the 200 m gap. In that stage of the process, the 500 m shallow sections are too elevated to be useful. This example proves by its exception that the general rule that vertical closure leads to smaller flow velocities than horizontal closure is (not always) true. Neither the difficult operational procedure and small production capacity, nor the investment for a bridge or cableway, can compete (money wise) with the dump trucks operating from two sides for horizontal closure.

A rather critical point in the closure phasing is the situation near the island after the caissons have been placed. Then, the main gulley is blocked and the secondary gulley more than doubles its discharge. An easy way for the water to pass through the gap is to follow the main gulley, to circle across the shallows around the island and to return into the main gulley. Scouring a short-cut like that is a typical example of the consequence of the imbalance. It would be a disaster and has to be prevented.

A historic case of a closure in which firstly gulleys were blocked by caissons and then the shallows were closed, is the closure of the Schelphoek, one of the major dike breaches in the south west of the Netherlands (1953). The situation of the breaches and the closure alignment are pictured in section 2.3 showing the development of erosion gulleys. The picture giving the situation after 20 weeks shows the two gaps that had been shaped, typically suited for caisson placing, while the long overland stretches had been protected by mattresses. After the caissons had been placed the overland sections followed by horizontal closure. A large number of shallow concrete units was placed every tide, in such a short sequence that the overland flow could not scour a short-cut.

The vertical closure procedure, using dump trucks, driving on the sill and using hydraulic cranes to level the cobbles, was executed during the closure of the Markiezaatskade (1983), one of the secondary dams required in the Deltaworks. The dam closed a shallow tidal basin of about 20 km^2 with a tidal range of about 4 meters. The final gap was 800 m long and had a basic sill level of 2 m below MSL. That time the vertical closure was an advantage as the full 800 m had the same low sill level. The quarry stone dam was constructed in layers of 1 or 0.5 m thickness.

14.3 Closure over the Full Dam Length.

The option to close over the full length of the alignment is logically a vertical closure or a combined closure. The principle is to block the gulleys partly by sills first, until one level exists over the full length. Then, either vertically or horizontally, the 1000 m long gap above that sill is closed. The difference with the option to close (one of) the gulleys first is that there are no caissons, that the imbalance will be smaller and that flow conditions are more favourable due to the longer weir. The first two phases of construction are:

- a bottom protection along the full alignment,
- sills to be dumped up to the level of -2.5m (vessels' draught permitting).

Then, horizontal or vertical closure has to be selected. With a horizontal closure one very final gap is created. While proceeding to that gap the flow conditions on the lowest part of the sill will be determining. As that sill level is -2.5m, the situation of the former options is created again. The same final stage is reached than at the cost of a bottom protection over the full length. The bottom protection in the shallow areas therefore is superfluous, so the full length option is meaningful only in case vertical closure is taken. The next phases therefore are:

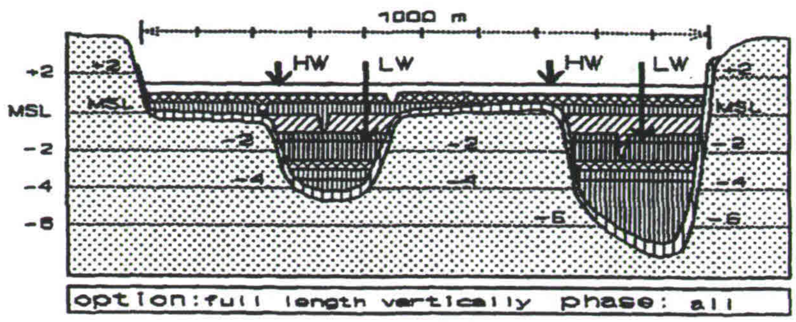


Figure 14-8 , Full length closure

- a next layer bringing the level of the sills up to -1m,
- further layers of 0.5 m thickness up to HW.

The procedure is identically to the last phases of the former option. The difference is the weir length which is 1000 m instead of 700 m.

The phasing of the closure thus reads:

phase and action	foreshore	sec. gulley	tidal flat	main gulley
0 original state	250m; -0.5	200m; -4	300m; MSL	250m; -6.5
1 bot.prot+sill (-3.5)	250m; MSL	200m; -3.5	300m; +0.5	250m; -3.5
2 sills dumped (-3)	250m; MSL	200m; -3	300m; +0.5	250m; -3
3 sills dumped (-2.5)	250m; MSL	200m; -2.5	300m; +0.5	250m; -2.5
4 sill by trucks (-1)	250m; MSL	200m; -1	300m; +0.5	245m; -1 5m;-2.5
5 up to MSL	445m; MSL	5m; -1	300m; +0.5	250m; MSL
6 up to +0.5	445m; +0.5	5m; MSL	300m; +0.5	250m; +0.5
7 up to +1	445m; +1	5m; +0.5	300m; +1	250m; +1
8 up to HW	445m; +1.5	5m; +1	300m; +1.5	250m; +1.5

Table 14-10 , Phasing of full dam length closure

The influence on the flow conditions appears from the lists below (to be compared with the table in the former option for the 700 m long weir):

flow and discharge		secondary gap				main gap			
phase	situation	during ebb		during flood		during ebb		during flood	
	(m/s,m³/s)	Umax	Qmax	Umax	Qmax	Umax	Qmax	Umax	Qmax
0	orig.	1.09	915	1.07	940	1.09	1810	1.07	1825
1	botpr+sill	1.78	1230	1.66	1310	1.78	1535	1.66	1635
2	sills -3	2.06	1180	1.90	1310	2.06	1475	1.90	1635
3	sills -2.5	2.48	1110	2.23	1305	2.48	1385	2.23	1630
4	sill -1	2.99 *	710	3.49 *	1135	2.99 *	870	3.49 *	1390

Table 14-11, Flow during full dam length closure

The discharge quantities in the remaining gap diminish with the progressing construction of the sill. The determining flow conditions are those in the narrow dip of every layer. The flow velocities in the various parts of the 1000 m gap are listed in the table on the next page. The phase with the determining flow conditions is phase 4, shown in Figure 14-9.

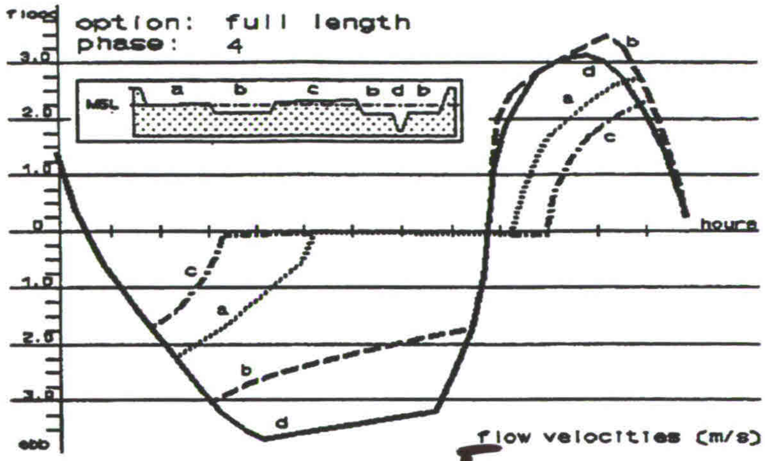


Figure 14-9, Determining flow conditions

Comparing the phases 4 to 8 of this option, with phases 7 to 11 of the former option, it appears that the maximum flow velocity is considerably less (3.62 m/s and 4.22 m/s) as consequence of the longer gap length. All flow conditions reach the critical flow stage, except for in the last dip in the sill, which exists as consequence of the construction of the layer up to the level of -1 m. In the dip the flood flow is still sub-critical. The magnitude is slightly less than the critical flow above the -1 m elevated sill, which is due to the low discharge-coefficient used for this narrow gap.

Umax. in m/s:		deepest part		deepest but one		deepest but two		deepest but three	
phase	situation	ebb	flood	ebb	flood	ebb	flood	ebb	flood
4	sill -1	3.62 *	3.09	2.99 *	3.49 *	2.24 *	2.74 *	1.68 *	2.27 *
5	up to MSL	2.97 *	2.87	2.33 *	3.05 *	1.98 *	2.32 *	not applicable	
6	up to 0.5	2.32 *	2.64 *	1.95 *	2.41 *			–	
7	up to +1	1.90 *	2.05	1.11 *	1.55 *	not applicable		–	
8	up to HW	0.88 *	1.55 *	high water free			–		

Table 14-12, Flow velocities during closure

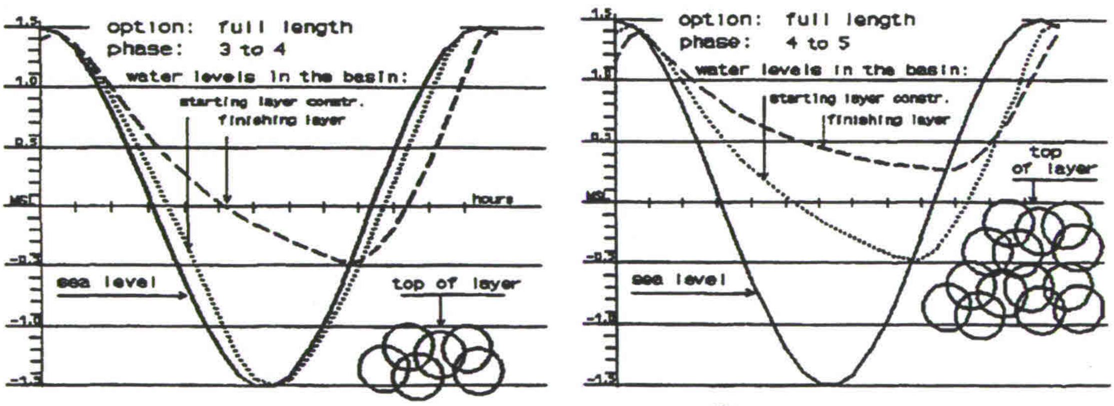


Figure 14-10, Waterlevel fluctuation during closure

Even more important is the fact that the time window for the equipment to operate on the sill is much longer than in the former option. This is due to the lower low-water level in the basin for the comparable level of the sill. On the other hand, an operational difficulty of the present option is that the layer construction has to advance over 500 m from each side instead of 350 m.

An alternative to avoid this problem of 500 m driving distance, is to prepare an approach-road towards to centre of the sill via an artificial island on the tidal flat. The construction can then advance from four sides along 250 m each at the cost of a major transport to the island and of installing transhipment facilities. The total sill length reduces with the width of the road connection only. If the island is situated in the alignment, the advantage of the long sill length decreases. At certain dimensions the method even changes into the first option with the final gaps in the channels, using vertical closure instead of caissons.

An example of a closure by constructing sills in the gulleys and providing one level over the full length of the alignment is the damming of the River Feni in Bangla Desh in 1984/85 (see also page 1-1). The tides were very variable, due to shallow water effects. Spring ranges doubled the neap ranges while low water levels were always about the same (see section 3.1, influence of MSf). During spring tides the tidal wave entered the estuary as a tidal bore. Therefore, conditions during neaps were a lot more favourable than during springs and the last layers of the vertical closure had to be forced in a major effort over the neap tide period. During that closure day a neap-tide-safe profile had to be constructed on top of the sill, which had to be heightened up to a spring-tide-safe profile within a week's time.

The tide on closure day rose from -0.5 m to +2.65 m (range 3.15 m) while the start level of the sill was +0.70 m. On closure day an embankment was constructed up to the level +3 m by piling up jute bags filled with clay. In total 1,000,000 bags were positioned by 12,000 Bangla Deshi people, all by hand, in five hours time. In order to minimize the hauling distance the bags had been stored in 12 stockpiles along the alignment of the dam, which reduced the total gap's length to about 1000 meter. The enlargement of the profile to the spring tide safe profile was done by trucks, tipping clay.

14.4 Final remarks.

The calculations and considerations on the closure options for the above example demonstrate that the change in conditions during the closure operations depends on the method adopted. Furthermore, it shows the correctness of the statement, made in the preface, that there is not one single correct solution for this design problem. On the basis of the above calculations, without further details about availability and costs of materials and equipment, final conclusions can not be drawn.

Besides, various other considerations may influence the decisions, such as: "Is there a possibility to build a large dry-dock for the construction, immersion and float-out of the caissons? Is there any social-economic reason why labour intensive low-level technology is preferred above a high-skilled approach? How skilled is the labour force available and in what sort of operations are they experienced? Are there any restrictions to import and use equipment or materials from outside the country and what is the taxation?"

But even the technical arguments have not all fully been considered. What about operational conditions and time periods. Are there seasonal changes in sea level or tidal amplitudes, periods with storm surges or cyclones, monsoons giving restrictions to operations because of waves and swell, limiting the execution of part of the works to specific work-windows? What happens when, due to unforeseen set-backs, the critical operation will exceed that time window?

No attention has been paid yet to the impact of extreme conditions on the design? Is sufficient data available to make an analysis of the probability of the occurrence of such an event? In the considerations made in the above example, not even the tide has been varied for springs and neaps.

Some operations are more vulnerable for extreme conditions then others and so consequently the measures to be incorporated in the design may take different proportions of the costs. These have to be included in the estimation of the various options for making a proper selection. Besides, in the event of a failure, some are rather simply overcome but others are disastrous. These considerations are difficult to weigh, sometimes it is hardly possible and yet it is all part of the design process.

It is outside the ambit of this book to offer a finite and predetermined formula for any dam closure. The objective has been to illustrate the basic technical problems likely to be encountered and the application of scientific principles to their solution, substantiated by the wealth of practical experience involving successes as well as failures.

15. Review

15.1 Breakwaters

The main choice for the designer of a breakwater lies in the choice between a rubble mound type of structure and a monolithic one. Advantages and disadvantages are therefore repeated here. Some of them are site specific and some are valid for the present moment only. The designer must therefore carefully judge in which direction he will move.

Advantages of the rubble mounds are:

- Simple construction
- Withstands unequal settlements
- Large ratio between initial damege and collapse
- Many guidelines available for the designer

Disadvantages of the rubble mound are:

- Depends on availability of adequate quarry
- Quantity of material large in deeper water
- Large space requirement
- Difficult to use as quay wall

en de gecombineerde ?

Advantages of the monolithic breakwater are:

- Short construction time on location
- Can function as quay wall
- Economic use of material in deeper water

Disadvantages of the monolithic breakwater are:

- Sensitivity for poor foundation conditions (settlements and liquefaction)
- Complete and sudden failure when overloaded
- Reflection against vertical wall
- Limited support for designer from guidelines and literature

15.2 Closure dams

For closure dams there are a few main directions the designer can follow (See chapter 2.3 and 12.8.4.). The first one is the choice in basics methods, the second one the optimal use of the natural conditions and boundary conditions and the third, the selection of materials and equipment.

Opsomming van verschillen bijv:
* ULS bij BW uitvoerd werk, bij dam uitvoering
* bij dam staking maatgevend, bij BW golf
* verschillende methoden van uitvoering.
* meer logistiek ontwerpen en stroombeelden!

Annex 1 Construction Equipment

A 1.1 Land-based equipment

A 1.2.1 Material in bulk

The land-based equipment must be split in highway and off-highway equipment. The off-highway equipment is designed to work on rough, uneven surfaces, where traditional vehicles can hardly move. In most countries, use of this equipment on public roads is not permitted since it causes excessive damage to the pavement.

If, nevertheless, material must be transported over long distances, there are a few options left:

- Construct special roads or tracks;
- Use high capacity highway trucks;
- Use existing or special rail connections.

The most widely used off-highway equipment is shown in Figure 1. It shows a mix of tyre and track based vehicles. Use of tyre-fitted equipment in quarry stone operations leads to excessive wear and tear, although it can not always be avoided. It is often necessary to spread finer material over the larger size stone (with a bulldozer) to create an accessible surface for tyre mounted equipment.

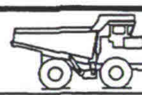
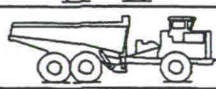
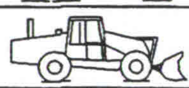
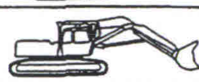
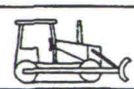
type		capacity (m^3)	weight (ton)	wheel load (ton) ground pressure	width (m)
(off higway) dump truck		20 - 90	empty: 30 - 110 loaded: 60 - 270	front/rear (ton) empty: 15/15 - 50/60 loaded: 20/40 - 90/180	wheel base 3.7 - 5.7
articulated dump truck		12 - 27	empty: 20 - 40 loaded: 40 - 90	front/rear (ton) empty: 10/10 - 20/20 loaded: 14/26 - 30/60	wheel base 5.7 - 6.8
wheel loader		2.5 - 9	15 - 86		bucket width 2.7 - 4.7
track loader		2.5 - 3	25	60 - 90 kPa	bucket width 2.7
backhoe crane		0.5 - 15	15 - 200	40 - 150 kPa	track gauge 2 - 5
front shovel		2 - 15	40 - 200	70 - 190 kPa	track gauge 2 - 5
bulldozer		blade width 2.5 - 5 m	10 - 80	50 - 100 kPa	track gauge 2 - 3

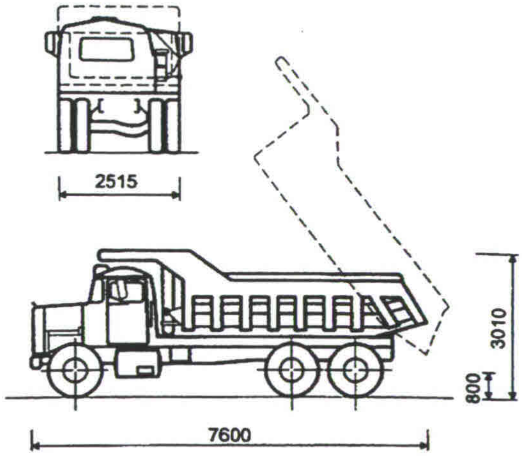
Figure A1-1, Review of land-based equipment

A comparison between a highway and an off-highway dumper is given in Figure 2.

hier ook "landen" verhaal van Roode lij?

MACK DM 686 SX(6 x 4)

nett carrying capacity	25,000 kg
gross vehicle weight	
front	8,200 kg
rear	29,480 kg
total	37,680 kg
nett weight	12,680 kg
engine	em 6-285; 210 kw at 2,100 rpm
fuel tank capacity	340 ltr
tyres	12.00 x 24
rock body	12 m^3



WABCO 35C (4 X 4)

nett carrying capacity	31,750 kg
gross vehicle weight	
front	18,865 kg
rear	39,358 kg
total	58,223 kg
nett weight	
front	13,399 kg
rear	13,073 kg
total	26,472 kg
engine: detroit 12v - 71n	320 kW at 2,100 rpm
max. speed	
forward	66 km/hr
backward	8.8 km/hr
turning circle	14.9 m
fuel tank capacity	454 ltr
body contents	
struck	17.6 m^3
heaped 1:1	26 m^3
tyres	18.00 x 33 24 PLY

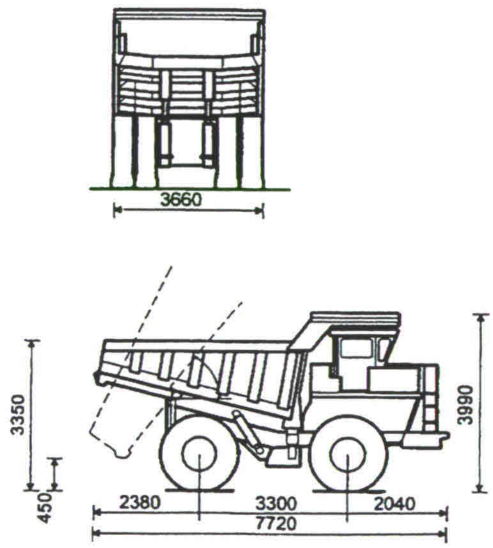


Figure A1-2, Tipper truck (highway) versus dump truck (off-highway)

Use of this heavy equipment requires a considerable space in the quarry, on the road, or on the crest of a breakwater. (See Figure 3) If space is insufficient to provide two lanes, passing places must be created at a practical distance. Since backing up reduces the speed considerably, also turning places or even turntables must be provided sometimes.

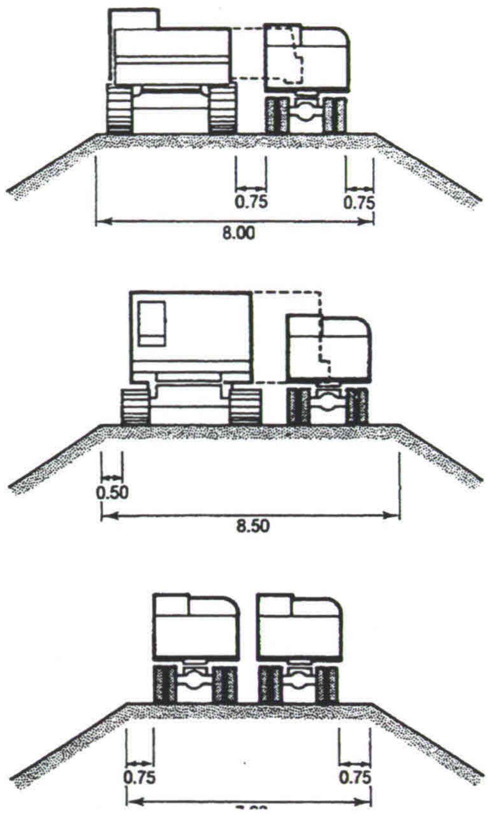


Figure A1-3, Space requirements for heavy vehicles

If the construction material can not be placed by direct dumping methods, bulk handling is still possible by using skips or containers. These skips can be loaded at the stockpile and transported to the work front on trucks or trailers. They can also be filled by dump trucks at the work front. At the work front, they are handled by crane and emptied at the spot that was not accessible for the direct dumping procedure.

1.1.2 Special placement

The larger size quarry stones and the concrete armour units are not placed in bulk but individually by crane. For this purpose, heavy cranes are used as indicated in Figure 4. These cranes can either be wheel mounted or tyre mounted. The lifting capacity decreases with the distance. It means that placing armour units near the toe of the structure is the most critical load condition for the crane. It is possible, however, to make use of the buoyancy of the elements by keeping the load just submerged when the crane is reaching out.

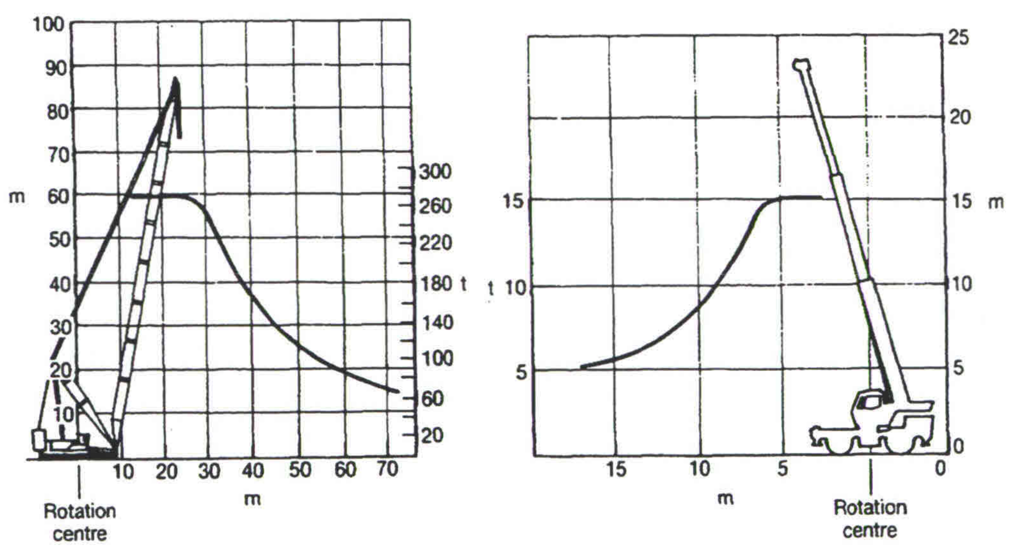
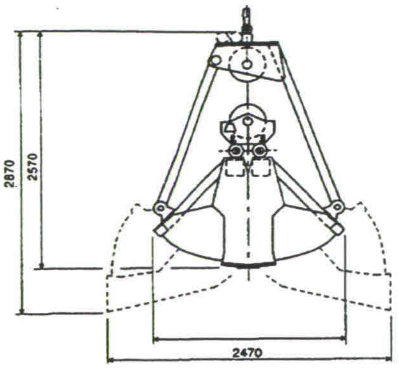
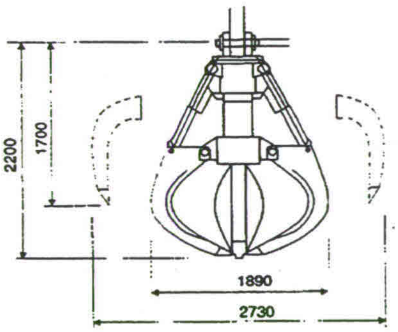


Figure A1-4, Lifting capacity of two typical heavy cranes



ROPE CLAMSHELL

capacity	1000 ltrs
type	2 ropes, digging
dead weight	1550 kg
width	1200mm



HYDR. GRAB

capacity	1000 ltrs
type	hydraulic grab with orange peel shells with mechanical swivel 360
no. of shells	5
max. load	8 tons
dead weight	1890 kg.

Figure A1-5, Grab types

The cranes handle individual units with the aid of a clamshell grab or an orange peel grab. (Fig. 5)

A crane as indicated in Figure 4 obstructs the work front, for instance at the tipping end of a breakwater under construction. Although the crane is necessary to place armour units and heavier stone, it prevents direct dumping of core material by dump trucks. It has been indicated that providing a skip or container can solve this, so that the crane can do the bulk handling as well. This method, however reduces the construction speed considerably. Therefore, sometimes the heavy crane is placed on a gantry, so that trucks can pass under the frame of the crane.

A 1.2 Waterborne equipment

A 1.2.1 Material in bulk

For the handling of material in bulk by waterborne equipment, we must make a distinction between very fine-grained material that can be handled by dredging equipment and the coarser material.

Pipeline transport
Fine-grained material like sand and gravel can still be handled by dredging equipment using hydraulic transport modes (pumps and pipelines). For transport over limited distances, pipeline transport is very common. In most cases, the pipeline is laid over already reclaimed land, and extended as the material pumped into the water reaches the required level. Under water natural slopes are formed, generally with a rather gentle slope, so that large volumes of material are required per running meter of dam. The coarser the material, the steeper

become the slopes. This leads eventually to much smaller quantities per running meter and thus to a faster forward movement of the work front.

Pipelines can also be laid over water by using floats or pontoons to carry the weight. In this way it is possible to apply sand or gravel in layers over the seabed. In order to prevent uncontrolled spreading of material, the end of the pipeline is often submerged and fitted with a diffuser.

Pipeline transport is not feasible in very rough seas, and not economic over large distances. Pipeline transport is than replaced by transport in barges or seagoing vessels like trailing suction hopper dredges. These vessels discharge the material either through openings in the bottom, or in the case of hopper dredges by pumping the material overboard via the suction pipe. In this way, a similar effect is achieved as in the case of a fixed pipeline with a diffuser.

Floating transport
By floating transport, we mean transport by barge or vessel. Some of these vessels can be used for both, the finer material like sand and gravel and the medium sized material like quarry stone to weights of say, 1 ton.

Amongst the barges and vessels we can distinguish the following types:
- Flat deck barges;
- Bottom door barges;
- Split barges;
- Tilt barges;
- Side unloading vessels.

All barges can be either push or pull barges or self-propelled vessels, with a varying sophistication of propulsion. The more sophisticated the propulsion system, the better is the accuracy of working in waves and currents.

- Flat deck barges exist in a wide range of carrying capacity, the largest meant for overseas transport of quarry stone in batches up to 30,000 tonnes! Loading and unloading is mostly done by crane or by wheelloader. Smaller size barges have a draught up to 2.5m, the large seaworthy barges may have a larger draught up to 5 m.

- Bottom door barges are used for fine-grained material and quarry stone as well. Their loaded draught is about 2.5 m. The load is dumped by opening bottom doors. Since the doors are opening all at once, the dumping is very uncontrolled. Care must be taken that the doors are not damaged when they are opened and may hit the mass of fresh dumped stone. In some cases, the barges are constructed such that the bottom doors do not stick out below the keel of the vessel, so as to prevent damage to the doors. The vessels can not dump to a higher level than MSL –3m. The capacity is in the order of 600tonnes.

- Split barges are again used for both fine-grained material and medium sized quarry stone. The barges unload by splitting the two halves of the hull in a longitudinal direction. Some barges can maintain a relatively small cleft. By moving the vessel sideways it is possible to dump a curtain of material that covers the seabed like a carpet. The capacity is in the order of 600 to 1000 tonnes

- Tilt barges are in fact flat top barges that are unloaded by flooding a ballast compartment along the side of the vessel. In this way the vessel is listing so much that the load slides down over the side into the water. The dumping is rather uncontrolled. Tilt barges are gradually replaced by more modem equipment. The capacity is in the order of 600 tonnes.

- Side unloading vessels are again flat deck vessels. Unloading is realised by pushing the material overboard by mechanical means. In this way accurate dumping is possible, specifically if the vessel is equipped with a sophisticated propulsion system. The capacity ranges from 600 to 2000 tonnes.

The various types of vessels are shown in Figure 6.

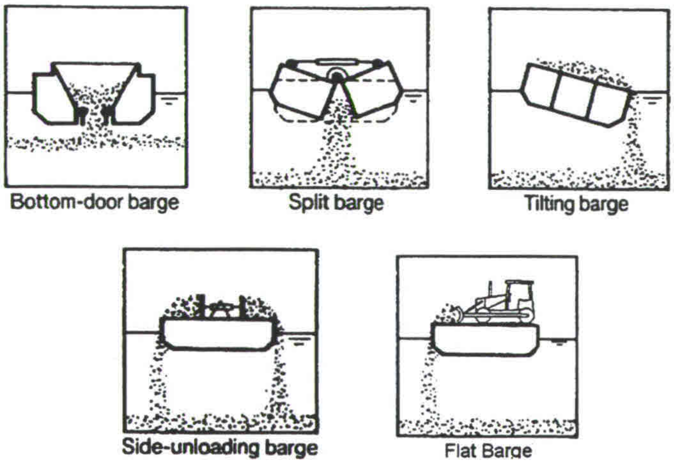


Figure A1-6, Barges for dumping material

A 1.2.2 Special placement

Special placement of material with waterborne equipment is defined here as placing with a crane. This is a narrow definition because it has been indicated in paragraph A1.2.1 that pipelines and some barges can also be used for (more or less) accurate placement.

Using a crane to places sometimes heavy material at sea is complicated because of the disturbance by waves and currents.

If the crane is fixed on a pontoon or vessel, proper care must be taken of the anchoring, so that the position can be maintained in spite of the prevailing currents. The action of the waves, however, is more difficult to compensate. There are always relative movements between the crane and the structure, and between the crane and a transport barge if the material is supplied from a vessel different from the crane pontoon. These relative movements make the handling (especially of heavy) material complicated and sensitive for weather delays. Therefore, it is often tried to reduce the relative movements between two vessels by combining the transport barge with the crane pontoon. (Fig. 7)

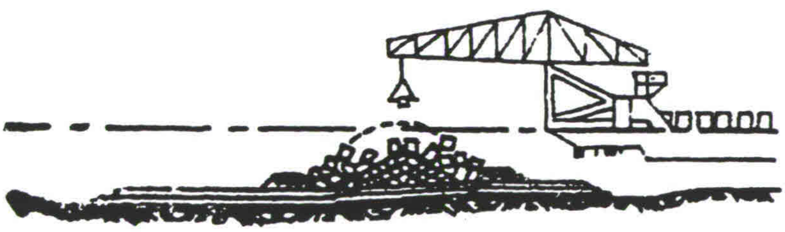


Figure A1-7, Example of combined transport and crane vessel

Another solution is to make the crane platform completely independent of the water motion. This can be done by methods developed in the offshore industry: the self-elevating platform. Such platforms have been used during the construction of the IJmuiden breakwater. They are floating pontoons during transport. The spuds or legs are lowered when the pontoon arrives at the right position. By hydraulic jacks, the pontoon is then lifted out of the water along the spuds and forms eventually a stable working platform for the crane.

A 1.3 Tolerances

When considering construction equipment, it is impossible to neglect the accuracy of placing material under varying conditions. It makes a tremendous difference whether material is placed in the dry, where it is possible to visually inspect and control the operations, or the material is placed under water, where one must rely on instruments for observation and control. Fortunately, position fixing and other measuring instruments have improved over the last years, so that the difference between working above and under water has been reduced. Still, however, there is a considerable difference. As a rough indication, one is referred to Table 1.

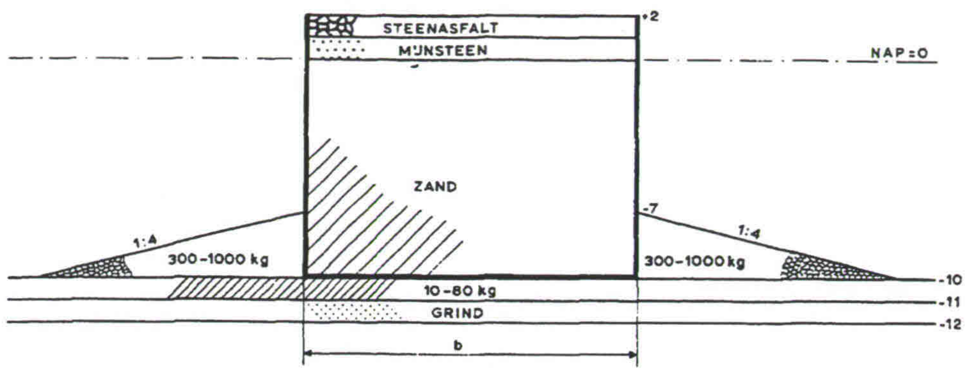
Type of Material	Place of application			
	Above water	Less than 5m below LW	5 to 15 m below LW	More than 15m below LW
Gravel (on or from land)	0.05m	0.10 to 0.15m	0.10 to 0.15m	0.10 to 0.15m
Gravel waterborne (standard)	n.a.	0.3 to 0.5m	0.3 to 0.5m	0.3 to 0.5m
Gravel Waterborne (special)	n.a.	0.1 to 0.2m	0.1 to 0.2m	0.1 to 0.2m
Quarry stone (bulk) W < 300kg	0.25 to 0.5 D_{50} absolute ±0.2m	+0.5 to – 0.3m	+0.5 to – 0.3m	+0.5 to – 0.3m
Quarry stone (bulk) W > 300 kg	+0.4 to– 0.2m	+0.8 to – 0.3m	+0.8 to – 0.3m	+0.8 to – 0.3m
Quarry stone (individual) W > 300 kg	± 0.3 D_{50}	± 0.5 D_{50}	± 0.5 D_{50}	± 0.5 D_{50}
Armour layer Design profile versus actual profile	+0.35 to– 0.25m	+0.6m to – 0.4m	+0.6m to –0.4m	+0.6m to –0.4m

Table A1-1, Vertical placing tolerances

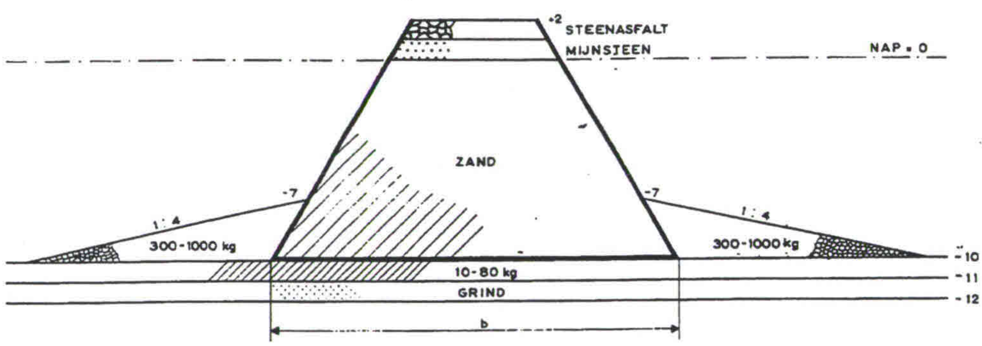
Annex 2 Alternatives for Hoek van Holland Breakwaters

1. Caisson Breakwaters

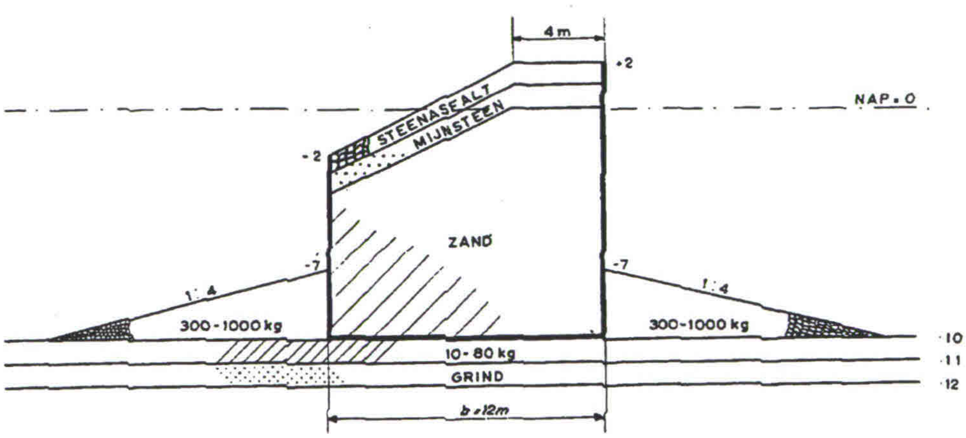
90° CAISSON



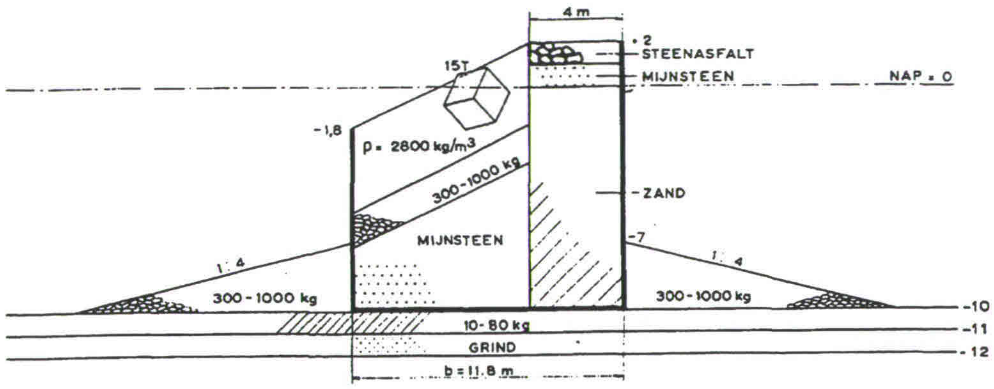
60° CAISSON



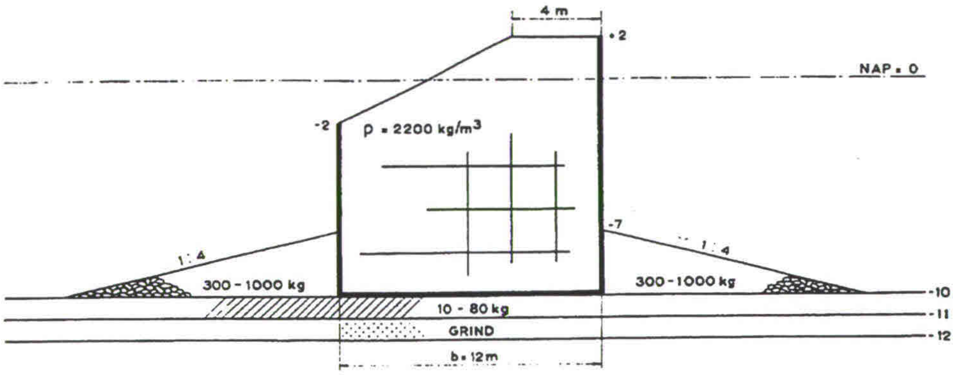
„HANSTHOLM" CAISSON



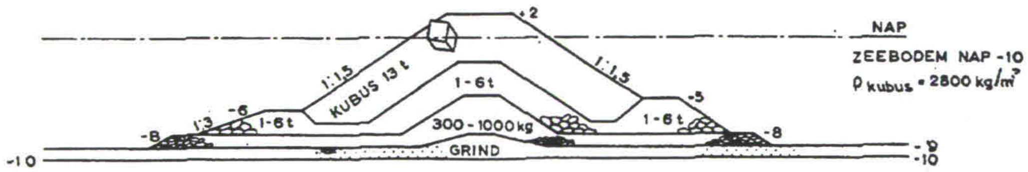
„HANSTHOLM" CAISSON MET BLOKKEN

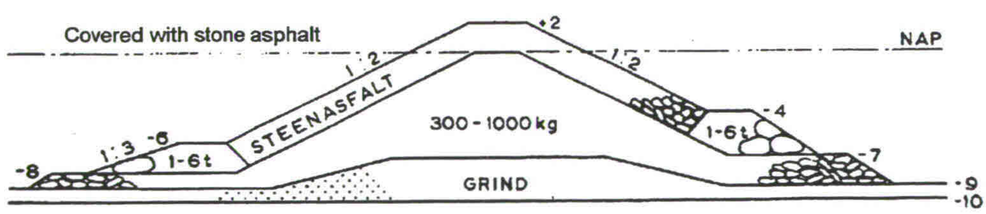


„HANSTHOLM" BLOKKENMUUR



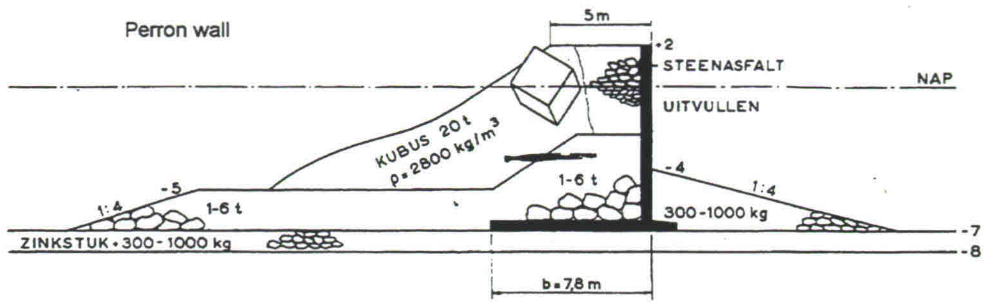
2. Stone Breakwaters



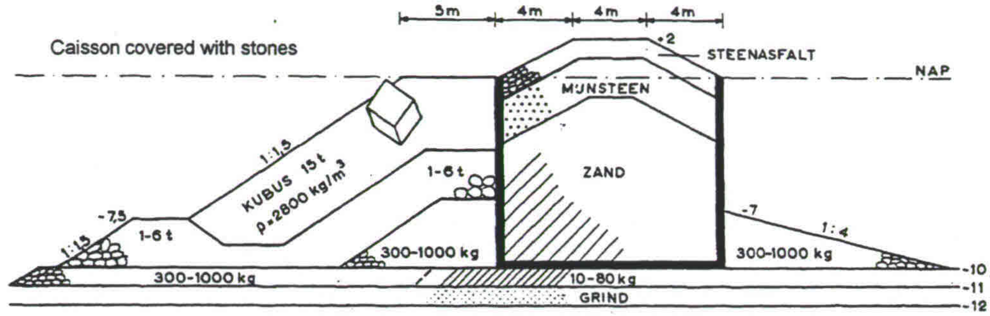


3. Combinations

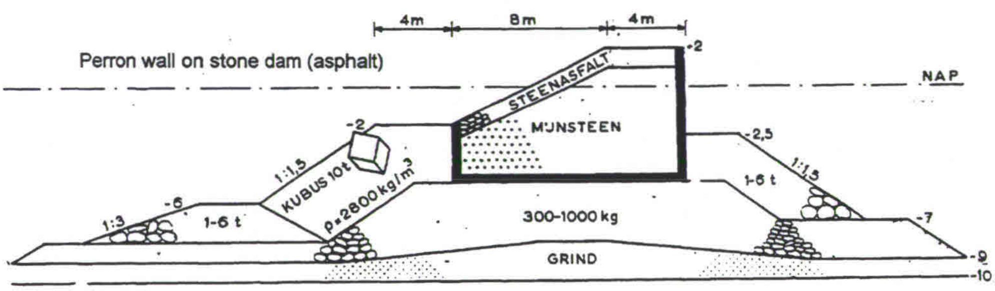
Perron wall



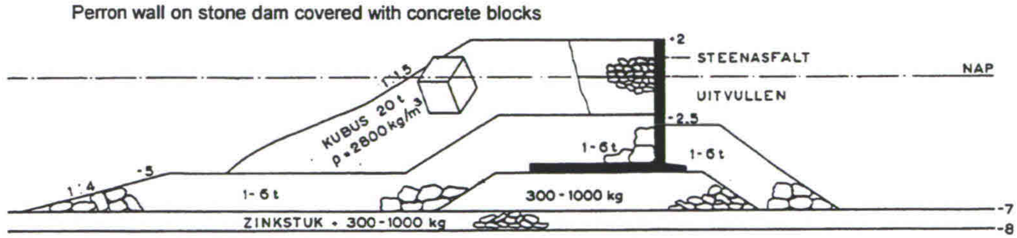
Caisson covered with stones



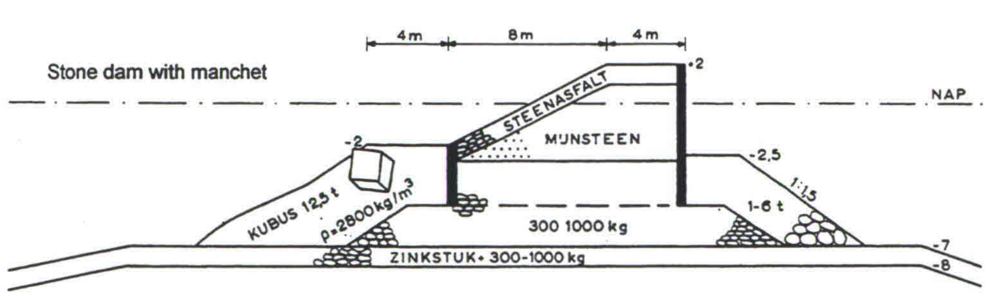
Perron wall on stone dam (asphalt)



Perron wall on stone dam covered with concrete blocks



Stone dam with manchet



THE DESIGN OF UPRIGHT BREAKWATERS

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ABSTRACT

The historical development of upright breakwaters in Japan is briefly reviewed as an introduction. Various wave pressure formulas for vertical walls are discussed, and then the design formulas currently employed in Japan are presented with an example of calculation. Several design factors are also discussed.

TABLE OF CONTENTS

1. INTRODUCTION

An upright breakwater is defined here as a structure having an upright section rested upon a foundation It is often called a vertical breakwater or composite breakwater. The former is sometimes referred to a structure directly built on the rock foundation without layers of rubble stones. The latter on the other hand means a breakwater functioning as a sloping-type structure when the tide level is low but as a vertical-wall structure when the tide level is high. Because the terminology may vary from person to person, the definition above is given here in order to avoid further confusion.

Upright breakwaters are of quite old structural type. Old ports in the Roman Empire or ports in even older periods had been provided with breakwaters with upright structures. The upright breakwaters of recent construction have the origin in the 19th century. Italian ports have many upright breakwaters as discussed in the following lecture by Dr. L. Franco. British ports also have a tradition of upright breakwater construction as exemplified in Dover Port. The British tradition can be observed in old breakwaters of Indian ports such as Karachi, Bombay, and Madras. Japanese ports owes this tradition of upright breakwaters to British ports, because the modern breakwater construction began at Yokohama Port in 1890 under supervision of British army engineer, retired Major General H. S. Palmer. Since then Japan has built a large number of upright breakwaters along her long coastline extending over 34,000 km. The total length of upright breakwaters in Japan would exceed several hundred kilometers, as the total extension of breakwaters is more than 1,000 km.

The present note is intended to introduce the engineering practice of upright breakwater design to coastal and harbor engineers in the world, based on the experience of Japanese engineers.

2. HISTORICAL DEVELOPMENT OF UPRIGHT BREAKWATERS IN JAPAN

2.1 Examples of Upright Breakwaters in Modern History of Japanese Ports

Figure 1 illustrates typical cross sections of upright breakwaters in Japan in time sequences, which is taken from Goda [1985]. The east breakwater of Yokohama Port in Fig. 1 (a) utilized the local material of soft clayey stones for rubble foundation and minimized the use of concrete blocks in the upright section. The stone-filled middle section was replaced by concrete blocks during reconstruction after the storm damage in 1902. The wave condition in Yokohama was not severe with the design height of 3 m.

The structural type of upright breakwaters was adopted at a more exposed location of Otaru Port as shown in Fig. 1 (b) by I. Hiroi in 1897, who was the chief engineer of regional government, later became a professor of the Tokyo Imperial University, and established the framework of Japanese harbor engineering. The first reinforced-concrete caisson breakwater in Japan was built at Kobe in 1911, based on the successful construction of caisson-type quaywall

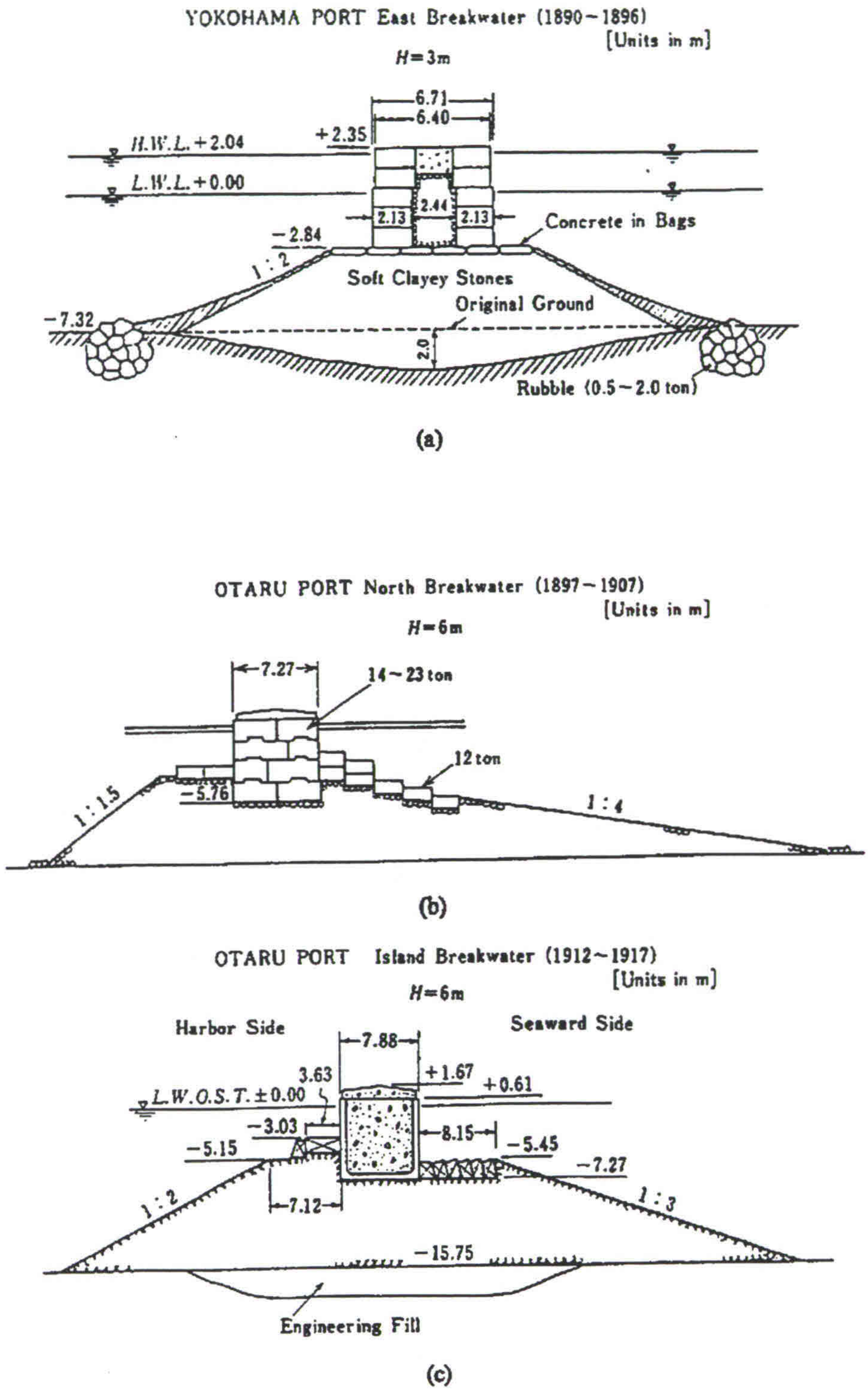
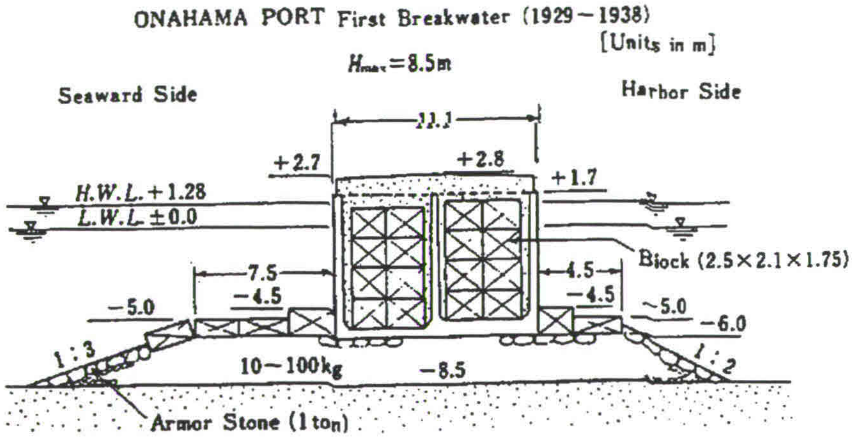
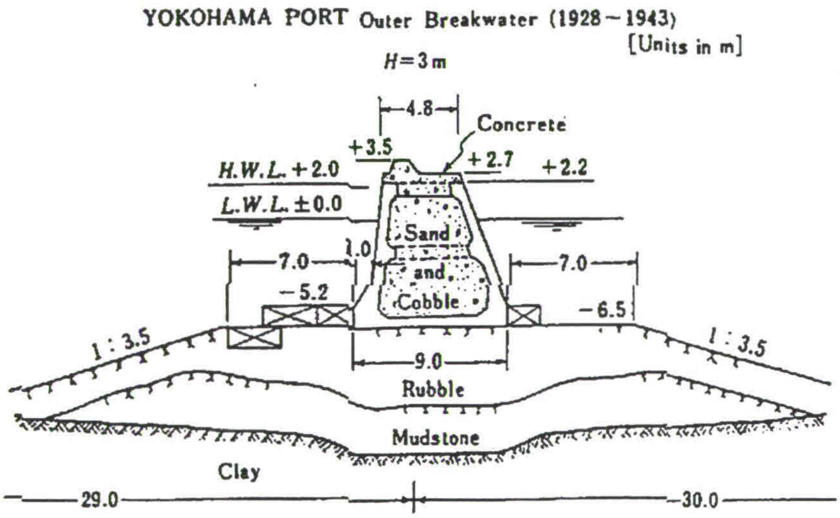


Fig. 1 (a-c) Historical development of upright breakwater in Japan after Goda [1985].



(d)



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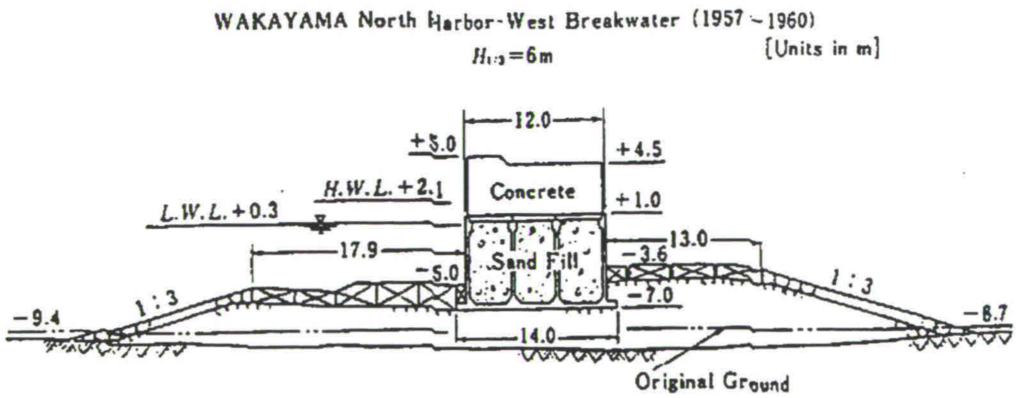


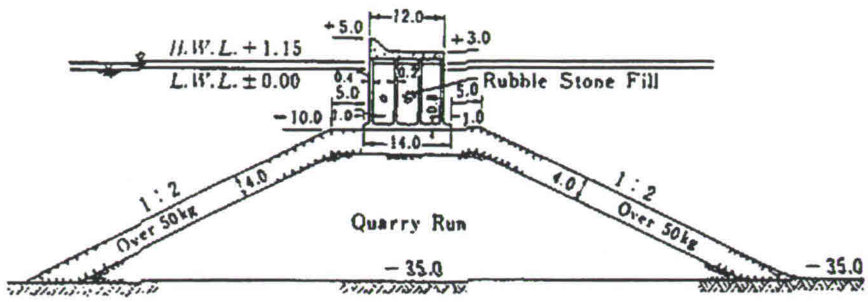
(f)

Fig. 1 (d-f) Historical development of upright breakwater in Japan (continued) after Goda [1985].

OFUNATO PORT Tsunami Breakwater (1962–1968)

[Units in m]

Tsunami : H=6 m, T=15~40 min
Wind Waves : $H_{1/3}$=4 m, $T_{1/3}$=9 s

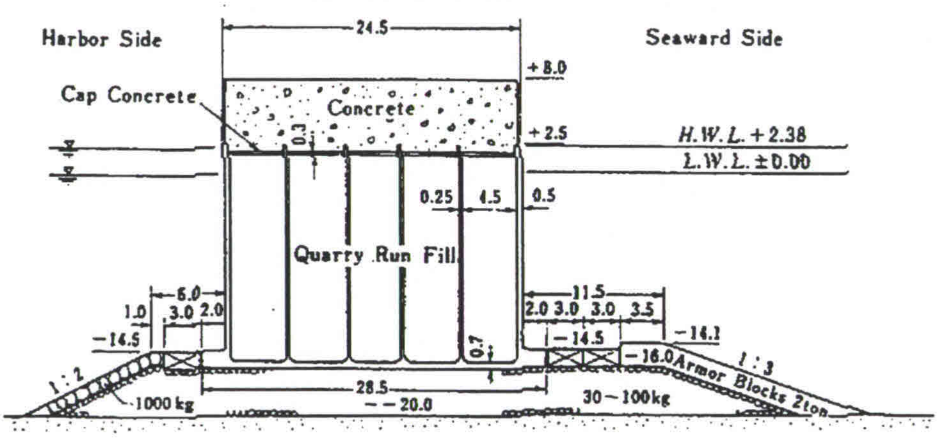


(g)

HOSOJIMA PORT Breakwater (1974~1985)

[Unit in m]

$H_{1/3}$=8.3 m, $T_{1/3}$=14.0 s

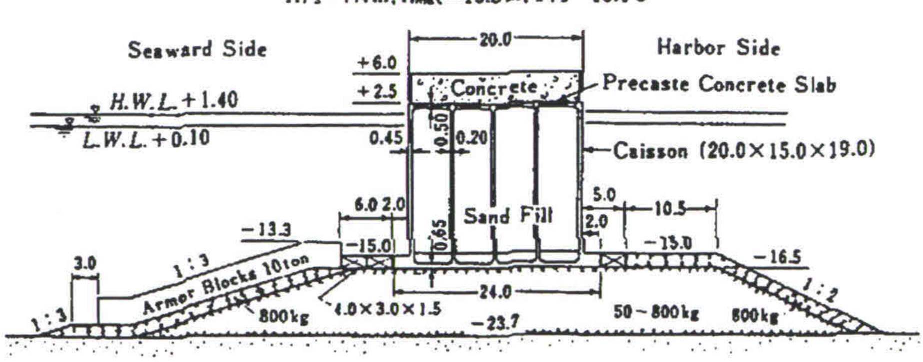


(h)

ONAHAMA PORT Offshore Breakwater (1980~)

[Unit in m]

$H_{1/3}$=7.4 m, H_{max}=13.3 m, $T_{1/3}$=13.0 s



(i)

Fig. 1 (g-i) Historical development of upright breakwater in Japan (continued) after Goda [1985].

at Rotterdam in 1905. Then Hiroi, immediately seeing the bright future of caisson breakwaters, employed the concept to an island breakwater of Otaru Port shown in Fig. 1 (c), where the design wave was 6 m high. He carried out various field measurements, including wave pressures on a vertical wall, for his finalization of breakwater design. Through these efforts, he came to *propose the wave pressure formula* for breakwater design, which is to be discussed in the next section.

Hiroi's breakwater caissons were filled with concrete for durability and stability. The work time for concrete placement was sometimes saved by the use of precast blocks as in the example of Onahama Port in Fig. 1 (d). Concrete filling of breakwater caisson had been a tradition before the end of World War II, but a pioneering construction of reinforced concrete caisson breakwater with sand filling was carried out in Yokohama Port during the period of 1928 to 1943: Fig. 1 (e) shows its cross section. After World War II the use of sand as the filler material of caisson cells gradually became a common practice in Japan.

The breakwater of Wakayama Port shown in Fig. 1 (f) was built upon a quite soft ground so that it was provided with a wide foundation for the purpose of counter-balancing the weight of upright section. The breakwater of Ofunato Port in Fig. 1 (g) was built to reduce the inflow of tsunami waves into the bay. The water depth of 35 m below the datum level was the deepest one at the time of construction in 1962, but the present record of the deepest breakwater in Japan is held at Kamaishi Port with the depth of 60 m. Some design features and wave pressures on this breakwater have been discussed by *Tanimoto* and Goda [1991b]. One of the widest breakwaters is that of Hosojima Port shown in Fig. 1 (h): the widest at present is found at Hedono Port in a remote island with the width 38m (see Tanimoto and Goda 1991a). The breakwater of Onahama Port shown in Fig. 1 (i) is of recent design using Goda's wave pressure formulas to be discussed later.

2.2 Some Features of Japanese Upright Breakwaters

As seen in these examples, Japanese breakwaters of upright type have a few common features. One is the relatively low crest elevation above the high water level. Presently, the recommendation for ordinary breakwaters is the crest height of 0.6 $H_{1/3}$ above the high water level for the design condition. For the design storm condition, this elevation is certainly insufficient to prevent wave agitations by the overtopped waves. However, it is a way of thinking of harbor engineers in Japan that the design waves are accompanied by strong gale and storm winds in any case and safe mooring of large vessels within a limited area of harbor basin cannot be guaranteed even if wave agitations are reduced minimum. As the storm waves with the return period of one year or less are much lower than the design wave, the above crest elevation is thought to be sufficient for maintaining a harbor basin calm at the ordinary stormy conditions.

Another feature of Japanese upright breakwaters is a relatively wide berm of rubble foundation and provision of two to three rows of large foot (toe) protection blocks. There is no fixed rule for selection of the berm width and engineers always consult with the examples of existing breakwaters in the neighborhood or those at the location of similar wave conditions. It is somewhat proportional to the size of concrete caisson itself, but the final decision must await good judgment of the engineer in charge. The foot protection concrete blocks have the size ranging from 2 to 4 m in one direction and the height of 1.5 to 2 m, weighing 15 to 50 tf. Though these blocks used to be solid ones, recent blocks are provided with several vertical holes to reduce the uplift force and thus to increase the stability against wave action.

A new development in upright breakwaters of Japan is the employment of various modifications to the shape of concrete caissons, such as perforated walls, vertical slits, curved slits with circular arc members, dual cylindrical walls and others (see Tanimoto and Goda 1991a). These new caisson shapes have been developed to actively dissipate wave energy and thus to reduce wave reflection and wave pressures. A number of these breakwaters have been built and functioning as expected.

3. REVIEW OF WAVE PRESSURE FORMULAS FOR VERTICAL WALL

3.1 Hiroi's Formula

Prof. Hiroi published the wave pressure formula for breakwater design in 1919. It is a quite simple formula with the uniform pressure distribution of the following intensity:

$$p = 1.5\ w_0\ H \tag{1}$$

where w_0 denotes the specific weight of sea water and H the incident wave height. This pressure distribution extends to the elevation of 1.25 H above the design water level or the crest of breakwater if the latter is lower, as shown in Fig. 2.

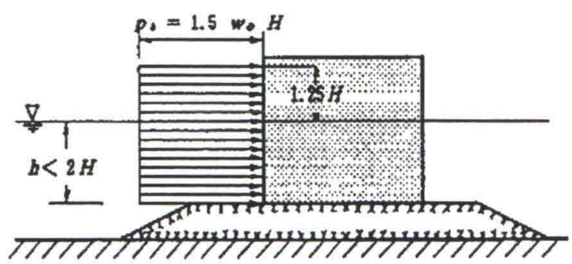


Fig. 2 Wave pressure distribution by Hiroi's formula.

Prof. Hiroi explained the phenomenon of wave pressure exerted upon a vertical wall as the momentum force of impinging jet flow of breaking waves and gave the reasoning for its quantitative evaluation. However, he must have had some good judgment on the magnitude of wave pressure from his long experience of harbor construction and several efforts of pressure measurements in situ. He states that he obtained the records of wave pressure exceeding 50 tf/m² by the pressure gauges set at a concrete wall in water of several meters deep. Nevertheless, he did not incorporate such high pressures into the formula of breakwater design, by saying that the high wave pressure must have lasted for only a short duration and are ineffective to cause appreciable damage to breakwaters.

Hiroi's wave pressure formula was intended for use in relatively shallow water where breaking waves are the governing factor. He also recommended to assume the wave height being 90% of water depth if no reliable information is available on the design wave condition. Hiroi's wave pressure formula was soon accepted by harbor engineers in Japan, and almost all breakwaters in Japan had been designed by this formula till the mid-1980s.

The reliability of Hiroi's formula had been challenged thrice at least. The first challenge was the introduction of Sainflou's formula in 1928 for standing wave pressures. Differentiation of two formulas was made, by referring to the recommendation of PIANC in 1935, in such a way that Hiroi's formula was for the case of the water depth above the rubble foundation being less than twice the incident wave height, while Sainflou's formula was for the water depth equal to or greater than twice the wave height. The second challenge was raised when the concept of significant wave was introduced in early 1950s. Which one of H_{max}, $H_{1/10}$, or $H_{1/3}$ is to be used in Hiroi's formula was the question. A consensus was soon formed as the recommendation for the use of $H_{1/3}$ based on the examination of existing breakwater designs and wave conditions. The third challenge was made by Goda [1973] against the insensitivity of the estimated pressure intensity to the variations in wave period and other factors. Hiroi's formula could not meet this challenge and is not used presently for the design of major breakwaters.

Though the pressure formula by Hiroi was so simple, the total wave force thus estimated was quite reliable on the average. Thanks to this characteristic, Japanese breakwaters had rarely experienced catastrophic damage despite the very long extension around the country.

3.2 Sainflou's Formula

As well known, Saiflou published a theory of trochoidal waves in front of a vertical wall in 1928 and presented a simplified formula for pressure estimation. The pressure distribution is sketched as in Fig. 3, and the pressure intensities and the quantity of water level rise δ_0 are given as

$$\left.\begin{aligned} p_1 &= (p_2 + w_0 h)\ (H+\delta_0)/(h+H+\delta_0) \\ p_2 &= w_0 H/\cosh kh \\ \delta_0 &= (\pi H^2/L)\coth kh \end{aligned}\right\} \qquad (2)$$

where L is the wavelength and k is the wavenumber of $2\pi/L$.

Sainflou [1928] presented the above formula for standing wave pressures of nonbreaking type and the formula has been so utilized. The formula was derived for the purpose of practical application from the standpoint of a civil engineer and it has served its objective quite well. Just like the case of Hiroi's formula, it was born when the concept of wave irregularity was unknown. There seems to exist no established rule for the choice of representative wave height to be used with Sainflou's formula. Some advocates the use of $H_{1/3}$, some favors $H_{1/10}$, and the other prefers the selection of $H_{1\%}$.

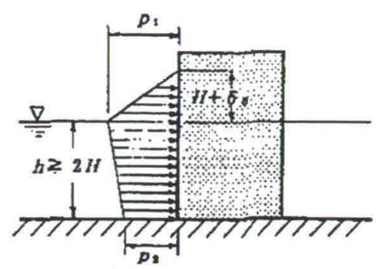


Fig. 3 Wave pressure distribution by Sainflou's formula.

It was customarily in Japan to use $H_{1/3}$ with Sainflou's formula but in a modified form. Through examinations of several minor damage of breakwaters, it had been revealed that a simple application of Sainflou's formula had yielded underestimation of wave pressures under storm conditions. For the zone extending $\pm H/2$ around the design water level, the wave pressure by Sainflou's foumula was replaced with that by Hiroi's formula. The modified formula was sometimes called the partial breaking wave pressure formula in Japan, because it was aimed to introduce the effect of partial wave breaking in relatively deep water. The dual system of Hiroi's wave pressure formula for breaking waves and of modified Sainflou's formula for standing waves had been the recommended engineering practice of breakwater design in Japan for the period from around 1940 to the early 1980s.

3.3 Minikin's Formula and Others

Although Hiroi's formula had been regarded as the most dependable formula for breaking wave pressures in Japan, it remained unknown in Europe and America. As the field measurement at Dieppe revealed the existence of very high pressures caused by impinging breaking waves and the phenomenon was confirmed by laboratory experiments by Bagnold [1939], harbor engineers in western countries began to worry about the impact breaking wave pressures. Then in 1950, Minikin proposed the following formula for breaking wave pressures, which consisted of the dynamic pressure p_m and the hydrostatic pressure p_s as sketched in Fig. 4:

Dynamic pressure :

$$\left.\begin{aligned} p_m &= p_{max}\,(1-2\,|\,z\,|\,/H)^2 & &: |\,z\,| \leqq H/2 \\ p_{max} &= 101\, w_0\, d\,(1+d/h)\, H/L & & \end{aligned}\right\} \tag{3}$$

Hydrostatic pressure :

$$p_s = \begin{cases} 0.5\, w_0\, H\,(1-2z/H) & : 0 \leqq z < H/2 \\ 0.5\, w_0\, H & : z < 0 \end{cases} \tag{4}$$

Because it was the first descriptive formula for breaking wave pressures, it was immediately accredited as the design formula and listed in many text-book and engineering manuals. Even in present days, technical papers based on Minikin's formula are published in professional journals from time to time.

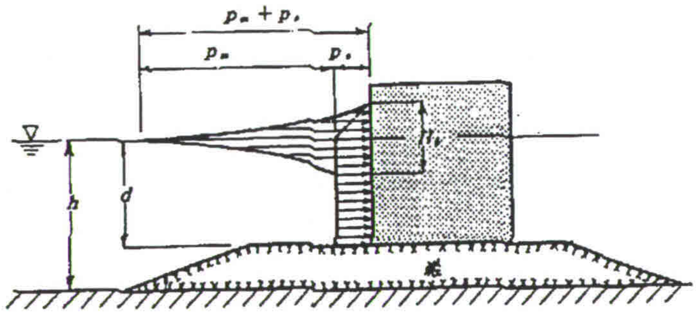


Fig. 4 Wave pressure distribution by Minikin's formula.

Minikin [1950] did not give any explanation how he derived the above formulation except for citing the experiments of Bagnold. In the light of present knowledge on the nature of impact breaking wave pressures, the formula has several contradictory characteristics. First, the maximum intensity of wave pressure increases as the wave steepness increases, but the laboratory data indicates that waves with long periodicity tends to generate well developed plunging breakers and produce the impact pressure of high intensity. In fact, Bagnold carried out his experiments using a solitary wave.

Second, Eq. 3 yields the highest p_{max} when d is equal to h or when no rubble foundation is present. It is harbor engineers' experience that a breakwater with a high rubble mound has a larger possibility of being hitten by strong breaking wave pressures than a breakwater with a low rubble mound.

Third, Minikin's formula yields excessively large wave force against which no rational upright breakwater could be designed. To the author's knowledge, no prototype breakwater has ever been constructed with the wave pressures estimated by Minikin's formula. Reanalysis of the stability of prototype breakwaters in Japan which experienced storm waves of high intensity, some undamaged and others having been displaced over a few meters, has shown that the safety factor against sliding widely varies in the range between 0.09 and 0.63 [Goda 1973b and 1974]. The safety factors of undamaged and displaced breakwaters were totally mixed together and no separation was possible. Thus the applicability of Minikin's formula on prototype breakwater design has been denied definitely.

There have been several proposals of wave pressure formulas for breakwater design. Among them, those by Nagai [1968, 1969]] and Nagai and Otusbo [1968] are most exhaustive. Nagai classified the various patterns of wave pressures according to the wave conditions and the geometry of breakwater, and presented several sets of design formulas based on many laboratory data. However, his system of wave pressure formulas was quite complicated and these formulas gave different prediction of wave pressures at the boundaries between the zones of their applications. Another problem in the use of Nagai's method was the lack of specification for represtantaive wave height for irregular waves. There was only a few cases of verification of the applicability of his method for break-water design using the performance data of prototype breakwaters. Because of these reasons, the method is not used in Japan presently.

The Miche-Rundgren formula for standing wave pressure [CERC 1984] represents an effort to improve the accuracy of Sainflou's formula for engineering application. Certainly, the formula would give better agreement with the laboratory data than Sainflou's one. However, it has not been verified with any field data and its applicability for brekwater design is not confirmed yet.

4. DESIGN FORMULAS OF WAVE PRESSURES FOR UPRIGHT BREAKWATERS

4.1 Proposal of Universal Wave Pressure Formulas

It is a traditional approach in wave pressure calculation to treat the phenomena of the standing wave pressures and those by breaking waves separately. Casual observations of wave forms in front of a vertical wall could lead to a belief that breaking wave pressures are much more intensive than nonbreaking wave pressures and they should be calculated differently. The previous practice of wave pressure calculation with the dual formulas of Hiroi's and Sainflou's in Japan was based on such belief. The popularity of Minikin's formula prevailing in western countries seems to be owing to the concept of separation of breaking and nonbreaking wave pressures.

The difference between the magnitudes of breaking and nonbreaking wave pressures is a misleading one. The absolute magnitude of breaking wave pressures is certainly much larger than that of nonbreaking one. The height of waves which break in front of a vertical wall, however, is also greater than that of nonbreaking waves. The dimensionless pressure intensity, $p / w_0 H$, therefore, increases only gradually with the increase of incident wave height beyond the wave breaking limit, as demonstrated in the extensive laboratory data by Goda [1972].

A practical inconvenience in breakwater design with the dual pressure formula system is evident when a breakwater is extended offshoreward over a long distance from the shoreline. While the site of construction is in shallow water, the wave pressures are evaluated with the breaking wave pressure formula. In the deeper portion, the breakwater would be subject to nonbreaking waves. Somewhere in between, the wave pressure formula must be switched from that of breaking to nonbreaking one. At the switching section, the estimated wave pressures jump from one level to another. With the Japanese system of the combined formulas of Hiroi's and modified Sainflou's, the jump was about 30%. To be exact with the pressure calculation, the width of upright section must be changed also. However, it is against the intuition of harbor engineers who believe in smooth variation of the design section. The location of switching section is also variable, dependent on the design wave height. If the design wave height is modified by a review of storm wave conditions after an experience of some damage on the breakwater, then an appreciable length of breakwater section would have to be redesigned and reconstructed.

The first proposal of universal wave pressure formula for upright breakwater was made by *Ito et al. [1966] based on the sliding test of a model section* of breakwaters under irregular wave actions. Then Goda [1973b, 1974] presented another set of formulas based on extensive laboratory data and being supported by verification with 21 cases of breakwater displacement and 13 cases of no damage under severe storm conditions. The proposed formulas were critically reviewed by the corps of engineers in charge of port and harbor construction in Japan, and they were finally adopted as the recommended formulas for upright breakwater design in Japan in 1980, instead of the previous dual formulas of Hiroi's and modified Sainflou's.

4.2 Design Wave

The upright breakwater should be designed against the greatest force of *single wave expected during its service life. The greatest force would be* exerted by the highest wave among a train of random waves corresponding to the design condition on the average. Thus the wave pressure formulas presented herein are to be used together with the highest wave to be discussed below.

(1) Wave height

$$H_{max} = \begin{cases} 1.8\, H_{1/3} & : h/L_0 \geqq 0.2 \\ \min\{(\beta_0^* H_0' + \beta_1^* h),\ \beta_{max}^* H_0',\ 1.8\, H_{1/3}\} & : h/L_0 < 0.2 \end{cases} \tag{5}$$

$$H_{1/3} = \begin{cases} K_s H_0' & : h/L_0 \geqq 0.2 \\ \min\{(\beta_0 H_0' + \beta_1 h),\ \beta_{max} H_0',\ K_s H_0'\} & : h/L_0 < 0.2 \end{cases} \tag{6}$$

in which the symbol $\min\{a, b, c\}$ stands for the minimum value among a, b and c, and H_0' denotes the equivalent deepwater significant height. The coefficients β_0 and others have empirically been formulated from the numerical calculation data of random wave breaking in shallow water as follows, after Goda [1975]:

$$\left.\begin{aligned} \beta_0 &= 0.028\,(H_0'/L_0)^{-0.38} \exp[20\tan^{1.5}\theta] \\ \beta_1 &= 0.52 \exp[4.2\tan\theta] \\ \beta_{max} &= \max\{0.92,\ 0.32\,(H_0'/L_0)^{-0.29} \exp[2.4\tan\theta]\} \end{aligned}\right\} \tag{7}$$

$$\left.\begin{aligned} \beta_0^* &= 0.052\,(H_0'/L_0)^{-0.38} \exp[20\tan^{1.5}\theta] \\ \beta_1^* &= 0.63 \exp[3.8\tan\theta] \\ \beta_{max}^* &= \max\{1.65,\ 0.53\,(H_0'/L_0)^{-0.29} \exp[2.4\tan\theta]\} \end{aligned}\right\} \tag{8}$$

in which the symbol $\max\{a, b\}$ stands for the larger of a or b, and $\tan\theta$ denotes the inclination of sea bottom.

The shoaling coefficient K_s is evaluated by taking the finite amplitude effect into consideration. Figure 5 has been prepared for this purpose based on the theory of Shuto [1974].

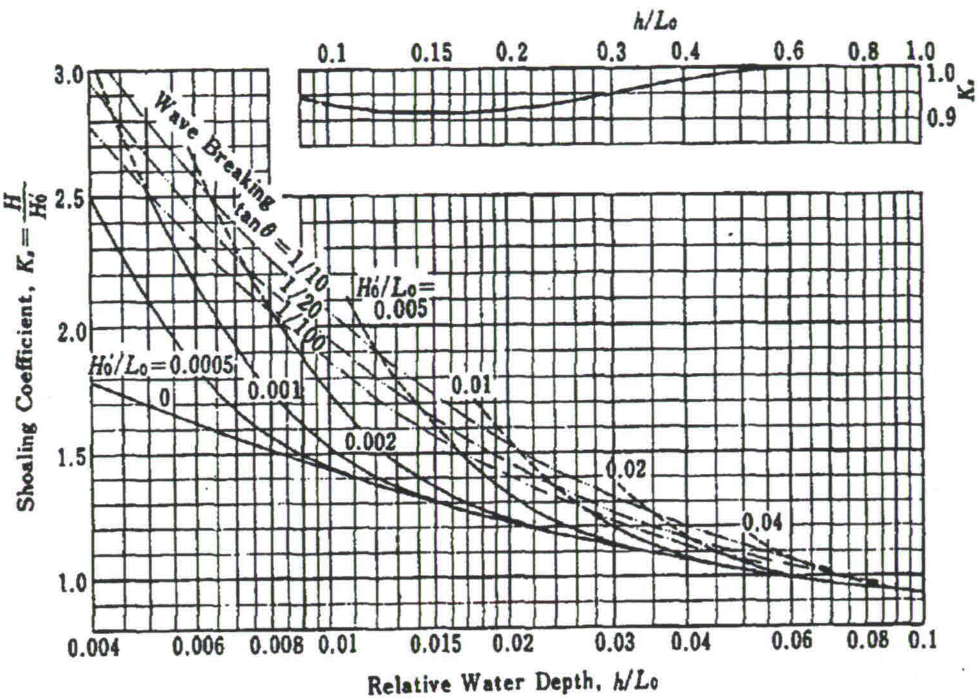


Fig. 5 Diagram of nonlinear wave shoaling coefficient K_s.

The selection of the fixed relation $H_{max} = 1.8\ H_{1/3}$ outside the surf zone was based on three factors of reasoning. First, the fixed ratio was preferred to an introduction of duration-dependent relation based on the Rayleigh distribution of wave heights, because such variability in the design wave height would cause some confusion in design procedures. Second, the examination of prototype breakwater performance under severe storm wave actions yielded reasonable results of safety factor against sliding by using the above fixed relation. Third, a possible deviation of the ratio $H_{max}\ /H_{1/3}$ from 1.8 to 2.0, say, corresponds to an increase of 11% and it can be covered within the margin of safety factor which is customarily taken at 1.2. However, it is a recommendation and an engineer in charge of breakwater design can use other criterion by his own judgment.

For evaluation of H_{max} by the second part of Eq. 7 or within the surf zone, the water depth at a distance 5 $H_{1/3}$ seaward of the breakwater should be employed. This adjustment of water depth has been introduced to simulate the nature of breaking wave force which becomes the greatest at some distance shoreward of the breaking point. For a breakwater to be built at the site of steep sea bottom, the location shift for wave height evaluation by the distance 5 $H_{1/3}$ produces an appreciable increase in the magnitude of wave force and the resultant widening of upright section.

(2) Wave Period

The period of the highest wave is taken as the same with the significant wave period of design wave, *i.e.*,

$$T_{max} = T_{1/3} \tag{9}$$

The relation of Eq. 9 is valid as the ensemble mean of irregular waves. Though individual wave records exhibit quite large deviations from this relation, the use of Eq. 9 is recommended for breakwater design for the sake of simplicity.

(3) Angle of Wave Incidence to Breakwater

Waves of oblique incidence to a breakwater exert the wave pressure smaller than that by waves of normal incidence, especially when waves are breaking. The incidence angle β is measured as that between the direction of wave approach and a line normal to the breakwater. It is recommended to rotate the wave direction by an amount of up to 15° toward the line normal to the breakwater from the principal wave direction. The recommendation was originally given by Prof. Hiroi together with his wave pressure formula, in consideration of the uncertainty in the estimation of wave direction, which is essentially based on the 16 points-bearing of wind direction.

4.3 Wave Pressure, Buoyancy, and Uplift Pressure

(1) Elevation to which the the wave pressure is exerted

The exact elevation of wave crest along a vertical wall is difficult to assess because it varies considerably from $1.0H$ to more than $2.0H$, depending on the wave steepness and the relative water depth. In order to provide a consistency in wave pressure calculation, however, it was set as in the following simple formula:

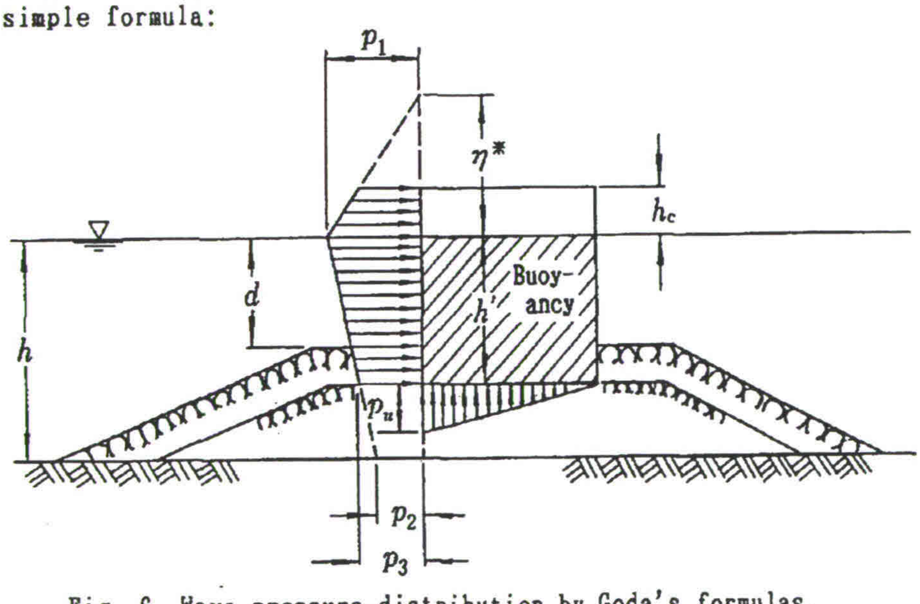


Fig. 6 Wave pressure distribution by Goda's formulas.

$$\eta^* = 0.75\,(1 + \cos\beta)\,H_{max} \qquad (10)$$

For waves of normal incidence, Eq. 10 gives the elevation of $\eta^* = 1.5\,H_{max}$.

(2) Wave pressure exerted upon the front face of a vertical wall

The distribution of wave pressure on an upright section is sketched in Fig. 6. The wave pressure takes the largest intensity p_1 at the design water level and decreases linearly towards the elevation η^* and the sea bottom, at which the wave pressure intensity is designated as p_2.

The intensities of wave pressures are calculated by the following:

$$\left.\begin{aligned} p_1 &= 0.5\,(1 + \cos\beta)\,(\alpha_1 + \alpha_2 \cos^2\beta)\, w_0 H_{max} \\ p_2 &= p_1/\cosh kh \\ p_3 &= \alpha_3\, p_1 \end{aligned}\right\} \qquad (11)$$

in which

$$\left.\begin{aligned} \alpha_1 &= 0.6 + 0.5\,[2kh/\sinh 2kh]^2 \\ \alpha_2 &= \min\{[(h_b - d)/3h_b](H_{max}/d)^2,\ 2d/H_{max}\} \\ \alpha_3 &= 1 - (h'/h)\,[1 - 1/\cosh kh] \end{aligned}\right\} \qquad (12)$$

where h_b denotes the water depth at the location at a distance $5H_{1/3}$ seaward of the breakwater.

The coefficient α_1 takes the minimum value 0.6 for deepwater waves and the maximum value 1.1 for waves in very shallow water. It represents the effect of wave period on wave pressure intensities. The coefficient α_2 is introduced to express an increase of wave pressure intensities by the presence of rubble mound foundation. Both coefficients α_1 and α_2 have empirically been formulated, based on the data of laboratory experiments on wave pressures. The coefficient α_3 is derived by the relation of linear pressure distribution. The above pressure intensities are assumed to remain the same even if wave overtopping takes place.

The effect of the incident wave angle on wave pressures is incorporated in η^* and p_1 with the factor of $0.5\,(1 + \cos\beta)$ and a modification to the term of α_2 with the factor of $\cos^2\beta$.

(3) Buoyancy and uplift pressure

The upright section is subject to the buoyancy corresponding to its displacement volume in still water below the design water level. The uplift pressure acts at the bottom of the upright section, and its distribution is assumed to have a triangular distribution with the toe pressure p_u given by Eq. 13.

$$p_u = 0.5\,(1 + \cos\beta)\,\alpha_1\,\alpha_3\,w_0 H_{max} \qquad (13)$$

The toe pressure p_u is set smaller than the wave pressure p_3 at the lowest point of the front wall. This artifice has been introduced to improve the accuracy of the prediction of breakwater stability, because the verification with the data of prototype breakwater performance indicated some overestimation of wave force if p_u were taken the same with p_3.

When the crest elevation of breakwater h_c is lower than η^*, waves are regarded to overtop the breakwater. Both the buoyancy and the uplift pressure, however, are assumed to be unaffected by wave overtopping.

4.4 Stability Analysis

The stability of an upright breakwater against wave action is examined for the three modes of failure: *i.e.*, sliding, overturning, and collapse of foundation. For the first two modes, the calculation of safety factor is a common practice of examination. The safety factors against sliding and overturning are defined by the following:

$$\text{Against sliding :} \quad S.F. = \mu\,(W - U)\,/\,P \tag{14}$$

$$\text{Against overturning :} \quad S.F. = (Wt - M_U)\,/\,M_P \tag{15}$$

The notations in the above equations are defined as follows:

- M_P : moment of total wave pressure around the heel of upright section
- M_U : moment of total uplift pressure around the heel of upright section
- P : total thrust of wave pressure per unit extension of upright section
- t : horizontal distance between the center of gravity and the heel of upright section
- U : total uplift pressure per unit extension of upright section
- W : weight of upright section per unit extension in still water
- μ : coefficient of friction between the upright section and the rubble mound

The safety factors against sliding and overturning are dictated to be equal to or greater than 1.2 in Japan. The friction coefficient between concrete and rubble stones is usually taken as 0.6. The coefficient seems to have a smaller value in the initial phase of breakwater installment, but it gradually rises to the value around 0.6 through consolidation of the rubble mound by the oscillations of the upright section under wave actions. The fact that most of breakwater displacements by storm waves occur during the construction period or within a few years after construction supports the above conjecture.

The bearing capacity of the rubble mound and the sea bottom foundation was used to be examined with the bearing pressures at the heel of upright section and at the interface between the rubble mound and the foundation. However, a recent practice in Japan is to make analysis of circular slips passing through the rubble mound and the foundation, by utilizing the simplified Bishop method (see Kobayashi et al. 1987). For the rubble mound, the apparent cohesion of $c = 2\ \mathrm{tf/m^2}$ and the angle of internal friction of $\phi = 35°$ are recommended.

4.5 Example of Wave Pressure Calculation

An example of calculation is given here in order to facilitate the understanding of the breakwater design procedure. The design wave and site conditions are set as in the following:

Waves: $H_0' = 7.0$ m, $T_{1/3} = 11$ s, $\beta = 10°$
Depth etc.: $h = 18$ m, $d = 10$ m, $h' = 11.5$ m, $h_c = 4.5$ m
Bottom slope: $\tan\theta = 1/50$

The incident wave angle is the value after rotation by the amount up to 15°.
The geometry of upright breakwater is illustrated in Fig. 7.

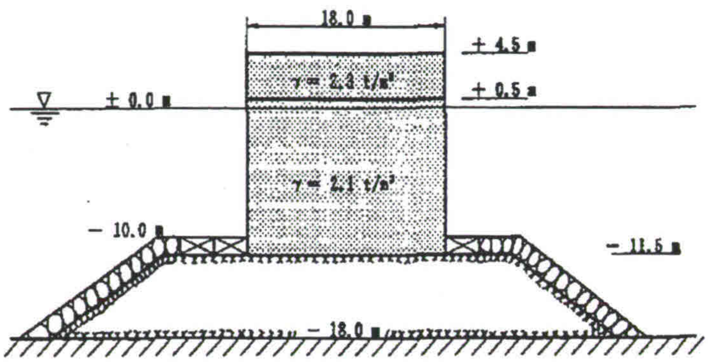


Fig.7 Sketch of upright breakwater for stability analysis.

i) Design wave height H_{max} and the maximum elevation of wave pressure η^*

The coefficients for wave height calculation are evaluated as

$$L_0 = 188.8\text{ m},\quad H_0'/L_0 = 0.0371,\quad h/L_0 = 0.0953,\quad K_s = 0.94$$
$$\beta_0 = 0.1036,\quad \beta_1 = 0.566,\quad \beta_{max} = \min\{0.92,\ 0.84\} = 0.92$$
$$\beta_0^* = 0.1924,\quad \beta_1^* = 0.680,\quad \beta_{max}^* = \min\{1.65,\ 1.39\} = 1.65$$

Then, the wave heights and the maximum elevation are obtained as

$$H_{1/3} = \min\{10.91,\ 6.44,\ 6.58\} = 6.44\text{ m}$$
$$h_b = 18.0 + 5\times 6.44/50 = 18.64\text{ m}$$
$$H_{max} = \min\{14.02,\ 11.55,\ 11.84\} = 11.55\text{ m}$$
$$\eta^* = 0.75\times(1 + \cos 10°)\times 11.55 = 17.19\text{ m}$$

ii) Pressure components

The wavelength at the depth 18 m is $L = 131.5$ m. The coefficients for wave pressure are evaluated as

$$kh \doteq 2\pi\times 18/131.5 = 0.860$$
$$\alpha_1 = 0.6 + 0.5\times[2\times 0.860/\sinh(2\times 0.860)]^2 = 0.802$$

$$\alpha_2 = \min\{[(18.64-10.0)/(3\times 18.64)]\times(11.55/10)^2,\ 2\times 10/11.55\}$$
$$= \min\{0.206,\ 1.732\} = 0.206$$
$$\alpha_3 = 1-11.5/18.0\times[1 - 1/\cosh(0.860)] = 0.820$$

Then, the intensities of wave pressure and uplift pressure are calculated as

$$p_1 = 0.5\times(1+0.9848)\times[0.802+0.206\times(0.9848)^2]\times 1.03\times 11.55$$
$$= 11.83\ \mathrm{tf/m^2}$$
$$p_2 = 11.83/\cosh(0.860) = 8.49\ \mathrm{tf/m^2}$$
$$p_3 = 0.820\times 11.83 = 9.70\ \mathrm{tf/m^2}$$
$$p_4 = 11.83\times(1 - 4.5/17.19) = 8.73\ \mathrm{tf/m^2}$$
$$p_u = 0.5\times(1+0.9848)\times 0.802\times 0.820\times 1.03\times 11.55 = 7.76\ \mathrm{tf/m^2}$$

The symbol p_4 denotes the pressure intensity at the top of upright section.

iii) Total pressure and uplift, and their moments

$$P = 0.5\times(11.83+9.70)\times 11.5+0.5\times(11.83+7.76)\times 4.5 = 167.9\ \mathrm{tf/m}$$
$$M_P = 1366.2\ \mathrm{tf\text{-}m/m}$$

$$U = 0.5\times 18.0\times 7.76 = 69.8\ \mathrm{tf/m}$$
$$M_U = (2/3)\times 69.8\times 18 = 837.6\ \mathrm{tf\text{-}m/m}$$

iv) Stability of upright section against wave action

The specific weight of upright section is assumed as in the following:

The portion above the elevation +0.5 m : $\gamma_c = 2.3\ \mathrm{tf/m^3}$
The portion below the elevation +0.5 m : $\gamma_c' = 2.1\ \mathrm{tf/m^3}$

The difference in the specific weight reflects a current practice of sand filling in the cells of concrete caisson. The weight of upright section is calculated for the dry and in situ conditions, respectively, as

$$W_0 = 2.1\times(11.5+0.5)\times 18.0+2.3\times(4.5-0.5)\times 18.0 = 619.2\ \mathrm{tf/m}$$
$$W = 619.2-1.03\times 11.5\times 18.0 = 406.0\ \mathrm{tf/m}$$

The safety factors against sliding and overturning of the upright section are calculated as in the following:

Against sliding: $S.F. = 0.6\times(406.0-69.8)/167.9 = 1.20$
Against overturning: $S.F. = (406.0\times 9.0-837.6)/1366.2 = 2.06$

Therefore, the upright breakwater with the uniform width of B = 18.0 m sketched in Fig. 7 is considered stable against the design wave of $H_0' = 7.0$ m and $T_{1/3}$ = 11.0 s.

5. DISCUSSION OF SEVERAL DESIGN FACTORS

5.1 Precautions against Impulsive Breaking Wave Pressure

The universal wave pressure formulas described hereinbefore do not address to the problem of impulsive breaking wave pressure in a direct manner. The coefficient α_2, however, has the characteristic of rapid increase with the decrease of the ratio d/H_{max}. This increase roughly reflects the generation of impulsive breaking wave pressure.

Though the impact pressure of breaking waves exerted upon a vertical wall is much feared by coastal and harbor engineers, it occurs under the limited conditions only. If waves are obliquely incident to a breakwater, the possibility of impact pressure generation is slim. If a rubble mound is low, the sea bottom should be steep and waves be of swell type for the impact pressure to be generated. A most probable situation under which the impact pressure is exerted upon an upright breakwater is the case with a high rubble mound with an appreciable berm width (see Tanimoto et al. 1987). Most of breakwater failures attributed to the action of the impulsive breaking wave pressure are due to the wave forces of normal magnitude, which could be estimated by the universal wave pressure formulas described in the present lecture note.

The impact pressure of breaking waves last for a very short time duration, which is inversely proportional to the peak pressure intensity. In other words, the impulse of impact pressure is finite and equal to the forward momentum of advancing wave crest which is lost by the contact with the vertical wall. The author has given an estimate of the average value of the impact pressure effective in causing sliding of an upright section, by taking into account the elastic nature of a rubble mound and foundation [Goda 1973a]. Because the major part of impact is absorbed by the horizontal oscillations and rotational motion of the upright section, the impact pressure effective for sliding is evaluated as $(2\sim3)\ w_0 H_{max}$.

Nevertheless, the pressure intensity of the above order is too great to be taken into the design of upright breakwaters: the mean intensity of wave pressure employed for the stability analysis of the breakwater sketched in Fig. 7 is only $0.91\ w_0 H_{max}$. Engineers in charge of breakwater design should arrange the layout and the cross section of breakwater in such way to avoid the danger of impact pressure generation. If the exertion of impulsive breaking wave pressure on the upright section seems inevitable, a change in the type of breakwater structure, such as a sloping-type breakwater or a vertical breakwater protected by a mound of concrete blocks, should be considered.

5.2 Structural Aspects of Reinforced Concrete Caisson

The upright section of vertical breakwater is nowadays made by reinforced concrete caisson. The width is determined by the stability condition against wave action. The height of caisson or the base elevation is so chosen to yield the minimum sum of the construction cost of rubble mound and upright section.

The length of caisson is governed by the capacity of manufacturing yard. In March 1992, Kochi Port facing the Pacific in Shikoku, Japan, set a breakwater caisson with the length 100 m in position. It is of hybrid structure with steel frames and prestressed concrete.

A concrete caisson is divided into a number of inner cells. The size of inner cells is limited to 5 m or less in ordinary design. The outer wall is 40 to 50 cm thick, the partition wall 20 to 25 cm thick, and the bottom slab 50 to 70 cm thick. These dimensions are subject to the stress analysis of reinforced concrete. As the upright breakwater withstands the wave force mainly with its own weight, the use of prestressed concrete for breakwater caisson is not advantageous in the ordinary situations. For the caisson of special shapes for enhancing wave dissipation such as the caisson with circular arc members, prestressed concrete is utilized.

5.3 Armor Units for Rubble Mound

The berm and slope of a rubble mound needs to be protected with armor units against the scouring by wave action. Foot-protection blocks weighing from 15 to 50 tf are placed in front of an upright section. The rest of the berm and slope are covered by heavy stones and/or specially-shaped concrete blocks. The selection of armor units is left to the judgment of engineers, with the aid of hydraulic model tests if necessary.

A formula for the weight of armor stones on the berm of rubble mound has been proposed by Tanimoto et al. [1982] as the results of systematic model tests with irregular waves. The minimum weight of armor stones can be calculated by a formula of the Hudson type:

$$W = \gamma_r H_{1/3}^3 \,/\, [N_s^3 \; (S_r - 1)^3] \tag{16}$$

in which W is the weight of armor stones, γ_r the specific weight of armor stones, S_r the ratio of γ_r to the specific weight of seawater, and N_s the stability number, the value of which depends on the wave conditions and mound dimensions.

For waves of normal incidence, Tanimoto et al. [1982] gave the following function for armor stones:

$$N_s = \max \{1.8,\ (1.3 \frac{1-\kappa}{\kappa^{1/3}} \frac{h'}{H_{1/3}} + 1.8 \exp [-1.5 \frac{(1-\kappa)^2}{\kappa^{1/3}} \frac{h'}{H_{1/3}}])\} \tag{17}$$

in which the parameter κ is calculated by

$$\kappa = [2kh'/\sinh 2kh'] \sin^2 (2\pi B_M \,/\, L') \tag{18}$$

and where h' denotes the water depth at which armor stones are placed, L' the wavelength at the depth h', and B_M the berm width.

Though the stability number for concrete blocks has not been formulated, a similar approach to the data of hydraulic model tests on concrete blocks will enable the formulation of the stability number for respective types of concrete blocks.

6. CONCLUDING REMARKS

The design and construction of upright breakwaters is a well established, engineering practice, at least in Japan, Korea, and Taiwan. A large number of these breakwaters have been built and will be built to protect ports and harbors. In these countries, the problem of impulsive breaking wave pressure is rather lightly dealt with. The tradition owes to Prof. Hiroi, who established the most reliable wave pressure formula in shallow water and showed the upright breakwaters could be successfully constructed against breaking waves.

This is not to say that no breakwaters have failed by the attack of storm waves. Whenever a big storm hits the coastal area, several reports of breakwater damage are heard. However, the number of damaged caissons is very small compared with the total number of breakwater caissons installed along the whole coastline. Probably the average rate per year would be less than 1%, though no exact statistic is available. Most cases of breakwater damage are attributed to the underestimation of the storm wave condition when they were designed.

In the past, the majority of breakwaters were constructed in relatively shallow water with the depth up to 15 m, for example, because the vessels calling ports were relatively small. In such shallow water, the storm wave height is controlled by the breaking limit of the water depth. One reason for the low rate of breakwater failure in the past could be this wave height limitation at the locations of breakwaters.

The site of breakwater construction is moving into the deeper water in these days. Reliable evaluation of the extreme wave condition is becoming the most important task in harbor engineering, probably much more than the improvement of the accuracy of wave pressure prediction.

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Annex 4 Glossary

Abutment (*Bruggenhoofd*) That part of the valley side against which the dam is constructed, or the approach embankment in case of bridges which may intrude some distance into the waterway.
Accretion (*Aanslibbing, Aanzanding, Sedimentatie*) Build up of material solely by the action of the forces of nature through the deposition of waterborne or airborne material.
Aggradation (*Aanzanding*) A build up or raising of the channel bed due to sediment deposition.
Alongshore See LONGSHORE
Apron (*Stortebed*) Layer of stone, concrete or other material to protect the toe of a structure against scour.
Armour layer (*Bekledingslaag*) Protective layer on rubble mound breakwater composed of armour units.
Armour unit (*Bekledingselement*) Large quarrystone or special concrete shape used as primary protection.
Articifal nourishment, beach replenishment, beach feeding (*Kunstmatige Zandsuppletie, Kustvoeding*) Supplementing the natural supply of beach material to a BEACH, using imported material.
Axis of stream (*Stroomas*) Line joining the mid points of the surface of the stream at successive cross-sections.
Back rush (*Terugloop*) The seaward return of the water following RUN-UP.
Backwater curve The longitudinal profile of the water surface in an open channel where the depth of flow has been increased by an obstruction such as a WEIR or DAM across the channel, by increase in channel roughness, by decrease in channel width or by a decrease of the bed gradient.
Barrage (*Stuwdam*) A barrage built across a river, comprising a series of gates which when fully open allow the flood to pass without appreciably increasing the flood level upstream of the barrage.
Barrier (*Stuw*) The function of a barrier is to control the water level. It consists of a combination of a concrete or a steel structure with or without adjacent ROCKFILL DAMS.
Bastion (*Bastion, Vooruitgeschoven Punt*) A massive GROYNE, or projecting section of seawall normally constructed with its crest above water level.
Bathymetry (*Diepteligging*) Topography of sea/estuary/lake bed.
Beach (*Strand*) By common usage the zone of BEACH MATERIAL that extends landward from the lowest water line to the place beyond the high water line where there is a marked change in material or physiographic form, or to the line of permanent vegetation.
Beach material (*Strandmateriaal*) Granular sediments usually sand or shingle moved by the sea.
Bed load (*Bodemtransport*) The quantity of sediment moving along the bed by rolling, jumping or sliding with at least intermittent contact.
Bed protection (*Bodembescherming*) A (rock) structure on the sea bed or the bed of a river or estuary in order to protect the underlying bed against erosion due to current and/or wave action.
Bend scour (*Bochtuitschuring*) EROSION in (the other part of) a river bed
Berm 1) Relative small mound to support or key-in an ARMOUR LAYER.
2) A horizontal step in the sloping profile of an EMBANKMENT.
Berm breakwater Rubble mound with horizontal BERM of ARMOUR STONES at about sea side water level, which is allowed to be (re)shaped by the waves.
Bifurcation (*Watersplitsing*) Location where a river separates in two or more reaches or branches.
Blanket (*Filter*) A layer or layers of graded fine stones underlaying a breakwater, GROYNE or rock EMBANKMENT to prevent the natural bed material being washed away.
Braided river (*Vlechtende Rivier*) A river type with multiple channels separated by shoals, bars and islands.
Braiding belt Area extending on both sides along a BRAIDING RIVER out to the extreme historic alignments of the river banks.

Breastwork (*Beschoeiing*) Timber structure generally parallel to coast.
Bull nose Substantial lip or protuberance at the top of the seaward face of a wall, to deflect waves seaward.
Bypassing (*Omleiden van sediment*) Moving of BEACH MATERIAL from the accumulating updrift side to the eroding downdrift side of an obstruction to LONGSHORE TRANSPORT, e.g. in inlet or harbour.
Canal (*Kanaal/Gracht*) A large artificial channel, generally of trapezoidal cross-section, designed for low velocity flow.
Caisson (*Caisson*) Concrete box-type structure.
Catchment area (*Stroomgebied*) The area which drains naturally to a particular point on a river, thus contributing to its natural discharge.
Channel (Vaargeul) A general term for any natural or artificial bed for running water having a free surface.
Coastal defences, coastal works (*Kustverdedigingswerken*) Collective terms covering protection provided to the coastline. These include COAST PROTECTION and SEA DEFENCES.
Coastal processes (*Kustprocessen*) Collective term covering the action of natural forces on the coastline and adjoining sea bed.
Coastal regime (*Kustregime*) The overall system resulting from the interaction on the coast and sea bed of the various COASTAL PROCESSES.
Coast protection (*Kustverdediging*) Works to protect land against EROSION or encroachment by the sea.
Cofferdam (*Dam rondom Bouwkuip*) A temporary structure enclosing all or part of the construction area so that construction can proceed in the dry.
Combined closure method (*Gecombineerde Sluitmethode*) Construction of a DAM by means of partly the HORIZONTAL CLOSURE method and partly the VERTICAL CLOSURE method.
Confluence (*Samenvloeiing*) The junction of two or more river reaches or branches.
Confluence scour (*Erosie ter plaatse van Samenvloeiing*) Erosion at the CONFLUENCE of rivers.
Cover layer (*Deklaag*) The outer layer used in a revetment system as protection against external hydraulic loads.
Crenulate An indented or wavy shoreline beach form, with the regular, seaward pointing parts rounded rather than sharp as in the CUSPATE type.
Crest (*Top van een Golfbreker*) Highest part of a breakwater sea wall, SILL or DAM.
Crown-wall (*Kroonstuk*) Concrete superstructure on a RUBBLE MOUND.
Cuspate Form of beach shoreline involving sharp seaward pointing cusps (normally at regular intervals) between which the shoreline follows a smooth arc.
Dam (*Dam*) Structure built in rivers of estuaries, basically to separate water at both sides and/or to retain water at one side.
Deep water (*Diep Water*) Water so deep that waves are little affected by the bed. Generally, water deeper than one half the surface wave length is considered to be deep water.
Degradation or erosion (*Erosie*) A lowering of the channel bed due to SCOUR.
Design storm (*Maatgevende Storm*) Sea walls will often be designed to withstand wave attack by the extreme DESIGN STORM. The severity of the storm (i.e. RETURN PERIOD) is chosen in view of the acceptable level of risk of damage or failure.
Diffraction (*Diffractie*) Process by which energy is transmitted laterally along a wave crest. Propagation of waves into the sheltered region behind a BARRIER such as a breakwater.
Dike (*Dijk*) A long, low EMBANKMENT with a height usually less than four to five metres and a length more than ten or fifteen times the maximum height. Ususally applied to DAMS built to protect land from flooding.
Discontinuity (*Discontinuiteit*) Any actual or incipient fracture plane in a rock mass including bedding planes, laminations, foliation planes, joints and fault planes.
Diversion channel (*Omleidingkanaal*) A WATERWAY used to divert water from its natural course. The term is generally applied to a temporary arrangement e.g. to by-pass water round a DAM site during construction.
Downdrift (*Benedenstrooms*) The direction of predominant movement of LITTORAL DRIFT along the shore.

Drowned flow (*Subkritische Stroming*) see SUBCRITICAL FLOW.
Durability (Duurzaamheid) The ability of a rock to retain its physical and mechanical properties (i.e. resist DEGRADATION) in engineering service.
Duricrust (*Zoutkorst*) A hard layer formed at a present or past desert surface where salts carried in solution by capillary action have precipitated and cemented the surface layer sediments.
Dynamic equilibrium (*Dynamisch Evenwicht*) Short term morphological changes that do not affect the MORPHOLOGY over a long period.
Eddy (Wervel) A vortex-type motion of fluid flowing partly opposite to the main current.
ELCL, LCL, UCL, EUCL Extreme lower, lower, upper and extreme upper class limits used to define standard grading classes. Each class limit is a particular weight for which the cumulative percentage passing by weight must fall within a specified range, e.g. for ELCL, between 0 to 2%.
Embankment (*Dijk, Dam*) Fill material, usually earth or rock, placed with sloping sides and with a length greater than its height. An embankment is generally higher than a DIKE.
Energy Any device constructed in a WATERWAY to reduce or destroy the energy of fast flowing water.
Erosion (*Erosie*) The wearing away of material by the action of natural forces. Or: Loss of e.g. beach material due to natural forces.
Facing (*Afwerklaag*) A coating of a different material, masonry or brick, for architectural or protection purposes e.g. stonework facing, brickwork facing (concrete dam) or an impervious coating on the upstream slope of the DAM.
Fetch (length) (*Strijklengte*) Relative to a particular point (on the sea), the area of sea over which the wind can blow to generate waves at the point. The fetch length depends on the shape and dimensions of the fetch area, and upon the relative wind direction.
Filter (*Filter*) Intermediate layer, preventing fine materials of an underlayer from being washed through the voids of an upper layer.
Flaws (*Scheurtjes/Discontinuiteiten*) Discontinuities and voids within a piece of rock.
Flood plain (*Uiterwaarden*) The area within the flood EMBANKMENTS.
Flood routing (*Hoogwaterberekening*) The attenuating effect of storage on a flood passing through a valley, a CHANNEL or RESERVOIR by reason of a feature acting as a control e.g. a reservoir with a spillway capacity less than the flood inflow or the widening or narrowing of a valley.
Flood wall, splash wall (*Spatmuur*) Wall, retired from the seaward edge of the sea wall crest, to prevent water from flowing on to the land behind.
Flow regime (*Stroomregime*) Combinations of river discharge and corresponding water levels and their respective (yearly or seasonally) averaged values and characteristic fluctuations around these values.
Foreshore (*Nat Strand*) The part of the shore lying between Mean High Water (Spring) and Mean Low Water level (Spring).
Fracture toughness (*Scheurweerstand*) The characteristic level of stress intensity ahead of a crack tip that is required to propagate the new crack catastrophically through the mineral fabric of the rock.
Freeboard (*Vrijboord*) The height of a structure above STILL WATER LEVEL.
Physical model (*Fysisch Model*) See SCALE MODEL.
Geotextile (*Geotextiel*) A synthetic fabric which may be woven or non-woven used as a FILTER or separation layer.
Gradings (*Gradering*): Distribution, with regard to size or weight, of individual stones within a bulk volume. Heavy, light and fine gradings are distinguished.
Gradual closure method (*Geleidelijke Sluitingsmethode*) Method in which the final gap is closed gradually either by the VERTICAL or the HORIZONTAL CLOSURE method or a combination of both methods. This method includes the use of large, massive CAISSONS to be placed on a SILL.
Granular filter (*Opgebouwd met Korrelvormig Materiaal*) A band of granular material which is incorporated in an EMBANKMENT dam and is graded so as to allow SEEPAGE to flow across or down the filter zone without causing the migration of the material from zones adjacent to the FILTER.
Groyne (*Strandhoofd*) A structure generally perpendicular to the shoreline built to control the movement of BEACH MATERIAL.

Hard defences (*Ondoorlatende/Harde Verdediging*) In common usage, normally taken to describe concrete, timber, steel, asphalt or RUBBLE shoreline structures. Rubble or rock structures are often considered SOFT DEFENCES because of their ability to absorb wave energy.
Head (*Kop van de Dam*) End of BREAKWATER or DAM.
Headwater level (*Bovenstroomse Waterstand*)The level of the water in the RESERVOIR.
Horizontal closure method (*Horizontale Sluitmethode*) Construction of a DAM by dumping the materials from one or both banks, thus constricting the WATERWAY progressively laterally until the gap is closed. The method is also known as end dumping, VERTICAL CLOSURE (both ICOLD definitions) and point tipping.
Hydraulics (*Vloeistofmechanica*) Science of water motion/flow/mass behaviour
Hydrology (*Hydrologie*)Science of the hydrological cycle (including precipitation, run-off, fluvial flooding).
Igneous rocks (*Vulcanisch Gesteente*) Formed by the crystallisation and solidification of a molten silicate magma.
In-situ block (*Stuk rots in Groeve*) A piece of rock bounded by discontinuities located within the rock mass prior to excavation.
Intact fabric strength (*Verwachte Sterkte door Samenstelling*) Strength of rock as a consequence of strength and fabric of the rock's minerals.
Integrity (*Samenhang*) The degree of wholeness of a rock block as reflected by the degree to which its strength against impacts is reduced by the presence of flaws.
Internal erosion (*Interne Erosie*)The formation of voids within soil or soft rock caused by the mechanical or chemical removal of material by SEEPAGE.
Irregular waves (*Onregelmatige golven*) Waves with random wave periods (and in practice, also heights), which are typical for natural wind-induced waves.
Levee (*Rivierdijk/Zomerdijk*) Flood EMBANKMENT less than one metre in height.
Lining (*Tussenlaag*) A coating of asphaltic concrete, concrete, reinforced concrete to provide water-tightness, to prevent EROSION or to reduce friction of a canal, tunnel or shaft.
Littoral drift, littoral transport (*Kusttransport*)The movement of BEACH MATERIAL in the LITTORAL ZONE by waves and currents. Includes movement parallel (LONGSHORE TRANSPORT) and perpendicular (onshore - offshore transport) to the shore.
Littoral zone (*Kustzone*) BEACH and SURF ZONE.
Longshore (*Parallel aan de kust*) Along the shore.
Longshore scour (*Ontgronding*) Local EROSION near fixed objects, including (rock) structures. **Longshore transport** (*Langstransport*) Wave-induced movement of sediment, rock or gravel along a beach (but also along sloping rock structures).
Mach-stem wave Higher-than-normal wave generated when waves strike a structure at an oblique angle.
Maintenance (*Onderhoud*) Repair or replacement of components of a structure whose life is less than that of the overall structure, or of a localised area which has failed.
Mattres (*Zinkstuk*) A blanket of brush, poles, plastic, fibres or other material lashed together to protect the EMBANKMENT or river channel from EROSION.
Maximum water level (*Maximale Waterstand*); The maximum water level, including flood surcharge, which the DAM has been designed to withstand.
Mean (*Gemiddeld*) The average value of a parameter.
Meandering (*Meanderend*) A single channel having a pattern of successive deviations in alignment which result in a more or less sinusoidal course.
Mean wave period (*Gemiddelde Golfperiode*) The mean period of the wave defined by zero-crossing.
Metamorphic rocks (Metamorfisch Gesteente) Formed by the effect of heat and pressure on IGNEOUS or SEDIMENTARY rocks for geological periods of time with the consequent development of new minerals and textures within the pre-existing rock.
Modular flow see SUPERCRITICAL FLOW.
Morphology (*Morfologie*) The transport of sediment and the consequential changes with time of the river or sea bed bed and river banks.
Numerical model (*Numeriek Model*) A description of the reality by means of mathematical equations which allow to predict the behaviour of flows, sediment and structures.

Monochromatic waves (*Monochromatische/Regelmatige Golven*) A series of waves, each of which has the same wave period.
Offshore breakwater (*Offshore Golfbreker*) A breakwater built towards the seaward limit of the LITTORAL ZONE, parallel (or near parallel) to the shore.
One-dimensional (1-D) model (*1-Dimensionaal Model*) A NUMERICAL MODEL in which all the flow parameters are assumed to be constant over the cross-section normal to the flow. There is only a velocity gradient in the flow direction.
Orthogonal (Wave Ray) In a wave refraction/diffraction diagram, a line drawn perpendicular to the wave crest.
Outlet (*Doorvoer Opening*) An opening through which water can be freely discharged from a RESERVOIR to the river for a particular purpose.
Overtopping (*Wateroverslag*) Water passing over the top of the SEA WALL.
Parapet (*Kopmuur*) Solid wall at crest of SEA WALL projecting above deck level.
Parapet-wall (*Golfkeermuurt*) See CROWN-WALL.
Peak period (*Piekperiode*) The wave period determined by the inverse of the frequency at which the wave ENERGY SPECTRUM reaches a maximum.
Pitching (*Bekleding*) Squared masonry or precast blocks or embedded stones laid in regular fashion with dry or filled joints on the upstream slope of an EMBANKMENT dam or on a RESERVOIR shore or on the sides of a channel as a protection against wave and ice action.
Pore pressure (*Poriëndruk*) The interstitial pressure of fluid (air or water) within a mass of soil, rock or concrete.
Porosity (*Porositeit, Gedeelte van Volume dat niet door Zand/Gesteente wordt ingenomen*) Laboratory measured property of the rock indicating its ability to retain fluids or gasses.
Porous (*Poreus*) In terms of REVETMENTS and ARMOUR, cladding that allows rapid movement of water through it such as during wave action (many GEOTEXTILES and sand asphalt can be non-porous under the action of waves but porous in soil mechanics terms).
Prototype (*Prototype*) The actual structure or condition being simulated in a model.
Quasi three-dimensional (3-D) model (*Quasi 3-Dimensionaal Model*) A NUMERICAL MODEL in which the flow parameters vary in two dimensions, but which allows to determine the flow parameter in the third dimension.
Quarry (*Groeve*) Site where natural rock stone is mined.
Quarry run (*Restmateriaal*) Waste of generally small size material, in a QUARRY, left after selection of larger GRADINGS.
Random waves (*Onregelmatige Golven*) The laboratory simulation of irregular sea states that occur in nature.
Reach (*Rivierpand*) Part of a river channel in longitudinal direction.
RDI_d, RDI_s, Rock DURABILITY indicators of Fookes et al (1988).
Reef breakwater (*Flexibele Golfbreker*) RUBBLE MOUND of single sized stones with a crest at or below sea level which is allowed to be (re)shaped by the waves.
Reflected wave (*Teruggekaatste Golf*) That part of an incident wave that is returned seaward when a wave impinges on a BEACH, sea wall or other reflecting surface.
Refraction (of Water Waves) (*Breking/Refractie*) The process by which the direction of a wave moving in SHALLOW WATER at an angle to the contours is changed so that the wave crests tend to become more aligned with those contours.
Refurbishment, renovation (*Reparatie/Renovatie*) Restoring the sea wall to its original function and level of protection.
Regime theory (*Regime Theorie*) Empirical method for predicting river characteristics.
Regular waves or Monochromatic waves (*Regelmatige Golven*) Fully periodic waves with constant period, which are practically not found in nature.
Regulating reservoir (*Beheersings-Reservoir*) A RESERVOIR from which water is released so as to regulate the flow in the river.
Rehabilitation (*Verbetering*) Renovation or upgrading.
Replacement (*Vervanging*) Process of demolition and reconstruction.
Reservoir (*Reservoir*) An artificial lake, basin or tank in which a large quantity of water can be stored.
Retention water level (*Aflaatniveau waarbij Aflaatmechanisme in werking treedt*) For a RESERVOIR with a fixed overflow SILL it is the lowest crest level of that sill. For the outflow from which is controlled wholly or partly by movable gates, syphons or by other means, it is

the maximum level at the DAM to which water at the dam may rise under normal operating conditions, exclusive of any provision for flood surcharge.
Return period (*Herhalingstijd*) In statistical analysis an event with a return period of *N* years is likely, on average, to be exceeded only once every *N* years.
Revetment (*Bekleding*) A cladding of stone, concrete or other material used to protect the sloping surface of an EMBANKMENT, natural coast or shoreline against EROSION.
Rip rap (*Stortsteen/Breuksteen*) Wide graded quarry stone normally used as a protective layer to prevent EROSION of the sea and/or river bed, river banks or other slopes (possibly including the adjoining crest) due to current and/or wave action.
River regime (*Rivier Regime*) Combinations of river discharge and water levels, characteristic for a prescribed period (usually a year or a season) and determining for the overall MORPHOLOGY of the river.
River training structure (*Constructie t.b.v. de Normalisatie van een Rivier*) Any configuration constructed in a stream or placed on, adjacent to or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce SCOUR, or in some other way alter the flow and sediment REGIMES of a river.
Rock degradation model (armourstone) A model under research and development, which attempts to predict yearly weight losses from the ARMOUR, taking account of rock properties and site conditions.
Rockfill dam (*Stortsteendam*) An EMBANKMENT dam in which more than 50 % of the total volume comprises compacted or dumped pervious natural or crushed stone.
Rock weathering (*Verwering van een Rots*) Physical and mineralogical decay processes in rock brought about by exposure to climatic conditions either at the present time or in the geological past.
Rubble mound structure (*Stortstenen Constructie*) A mound of random-shaped and random-placed stones.
Run-up (*Oploop*) The uprush of water onto a structure or BEACH as a result of wave action.
Run-up, run down (*Oploop/Afloop*) The upper and lower levels reached by a wave on a structure, expressed relative to still water level.
Scale or physical model (*Schaal/Fysisch Model*) Simulation of a structure and/or its (hydraulic) environment in usually much smaller dimensions in order to predict the consequences of future changes. The model can be built with a fixed bed or a movable bed.
S-Slope breakwater (*S-vormige Golfbreker*) RUBBLE MOUND with gentle slope around still water level and steeper slopes above and below.
Scour (*Ontgronding*) Local EROSION near some fixed object.
Scour protection (*Verdediging tegen locale Ontgronding*) Protection against EROSION of the sea bed in front of the TOE.
Sea defences (*Zeeweringen*) Works to prevent or alleviate flooding by the sea.
Secular changes (*Veranderingen op lange Termijn*) Long-term changes in sea level.
Sediment load (*Sedimenttransport*) The sediment carried through a CHANNEL by streamflow or by waves and currents at sea.
Sedimentary rocks (*Afzettingsgesteente*) Formed by the sedimentation and subsequent lithification of mineral grains, either under water or more rarely on an ancient land surface.
Seepage (*Kwel*) The interstitial movement of water that may take place through a DAM, its foundation or ABUTMENTS.
Sill (*Drempel*) a) A submerged structure across a river to control the water level upstream;
b) The crest of a SPILLWAY.
Shallow water (*Ondiep Water*) Commonly water of such depth that surface waves are noticeably affected by bottom topography.
Shoulder (*Schouder*) Horizontal transition to layer of larger size stones which is placed at higher elevation.
Significant wave height (*Significante Golfhoogte*) The average height of the highest of one third of the waves in a given sea state.
Significant wave period (*Significante Golfperiode*) An arbitrary period generally taken as the period of one of the highest waves within a given sea state.
Slope (*Talud/Helling*) The inclined face of a cutting or canal or EMBANKMENT
Slope protection (*Taludverdediging*) The protection of EMBANKMENT slope against wave action or EROSION.

Soft defences (*Energie Absorberende Verdediging*) Usually refers to BEACHES (natural or designed) but may also refer to energy absorbing structures including those constructed of rock, considered as HARD DEFENCES because of their stability.
Spillway (*Overlaat*) A structure over or through which flood flows are discharged.
Spur (-dike) or Groyne (*Krib in Rivier*) A structure extending from a bank into a channel that is designed usually to protect the banks or to provide enough water depth for navigation purposes.
Stationary process (*Stationair Proces*) A process in which the mean statistical properties do not vary with time.
Still water level (*Stil Waterniveau*) Water level which would exist in the absence of waves.
Stilling bassin (*Woelbekken*) A basin constructed so as to dissipate the energy of fast flowing water e.g. from a SPILLWAY or bottom outlet and to protect the river bed from erosion.
Stochastic (*Stochastisch*) Having random variation in statistics.
Storage reservoir (*Spaarbekken*) A RESERVOIR which is operated with changing water level for the purpose of storing and releasing water.
Storm surge (*Stormvloed*) A rise in water level in the open coast due to the action of wind stress as well as atmospheric pressure on the sea surface.
Streambed (*Rivierbed*) Low water channel.
Subcritical (*Subkritisch*) The flow condition above a dam by which the TAILWATER level influences the upstream head. The discharge is a function of upstream and downstream head. Also called submerged flow, submodular flow or DROWNED FLOW.
Supercritical (*Superkritisch*) The flow condition above a DAM by which the upstream head is independent of the TAILWATER level. The discharge is a function of the upstream head only. Also called free flow, rapid flow or MODULAR FLOW.
Surfzone (*Brandingszone*) The area between the outer most breaker and the limit of the wave RUN-UP.
Suspended load (*Zwevend Materiaal*) The material moving in suspension in a fluid, kept up by the upward components of the turbulent currents or by the colloidal suspension.
Swell (Waves) (*Deining*) Wind generated waves that have travelled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their FETCH.
Tailwater level (*Benedenwaterniveau*) The water level downstream of a DAM or SILL.
Thalweg (*Thalweg*) The locus of the deepest points in a valley or river at successive cross-sections.
Two/three-dimensional (2/3-D) model (*(2/3-D) Model*) A mathematical model in which the flow parameters vary in two/three dimensions.
Tides (*Getijden*) Water movements, basically due to global astronomic response of Oceans and besides, on the continental shelves and in coastal waters -and particularly estuaries and bays-strongly affected (amplified) by shallow water and coastal planforms. Typical specific definitions of associated local water levels, in decreasing order, are HAT or HHW, MHWS, MHW, MLW, MLWS, LAT or LLW.
Toe (*Teen*) Lowest part of seaward and port-side breakwater slope, generally forming the transition to the sea bed.
Total load (*Totaal Transport*) The sum of BED LOAD and SUSPENDED LOAD in the river.
Toe banket See APRON.
Training wall (*Strekdam*) A wall built to confine or guide the flow of water over the downstream face of an overflow DAM or in a CHANNEL.
Type 1 breakage (*Breukmechanisme 1*) Breakage of rock blocks into major pieces along flaws.
Type 2 breakage (*Breukmechanisme 2*) Breakage of rock blocks along new fractures.
Upgrading (*Verbetering*) Improved performance against some or other criteria.
Uplift (*Opwaartse Kracht*) The upward pressure in the pores of a material (interstitial pressure) or on the base of a structure.
Up-rush, down-rush (*Golfoploop/Golfafloop*) The flow of water up or down the face of a structure.
Vertical closure (*Verticale Afsluiting*) Construction of a DAM by dumping the materials over the full width during which operation the dam crest is raised more or less uniformly over the

entire gap until the channel is completely blocked. The method is also known as frontal dumping, HORIZONTAL CLOSURE (both ICOLD definitions) and traverse dumping.
Vesicular (*Holten bevattend*) Used to describe basalt and other volcanic rocks containing many spherical or ellipsoidal cavities produced by bubbles of gas trapped during solidification.
Wandering See MEANDERING.
Waterway (*Vaargeul*) A navigable CHANNEL.
Weir (*Overlaat*) A low dam or wall across a stream to raise the upstream water level. Termed fixed crest weir when uncontrolled.
Wave return face (*Golfombuigende Kruinconstructie*) The face of a CROWN WALL designed to throw back the waves.

Annex 5 Optimum breakwater design

Wave Height H {m)	Probability of Exceedance (times per annum)
4	1.11
5	$1.58*10^{-1}$
5.2	$8.4*10^{-2}$
5.5	$7.62*10^{-2}$
5.8	$3.8*10^{-2}$
6	$2.47*10^{-2}$
6.5	$7.35*10^{-3}$
7.15	$3.0*10^{-3}$
7.25	$2.63*10^{-3}$
7.8	$9.0*10^{-4}$
7.98	$8.0*10^{-4}$
8.7	$1.5*10^{-4}$

Table A5-1, Long-term wave climate

Actual Wave Height H	Damage in % of armour layer
$H < H_{nd}$	0
$H_{nd} < H < 1.3H_{nd}$	4
$1.3H_{nd} < H < 1.45\ H_{nd}$	8
$H > 1{,}45\ H_{nd}$	Collapse

Table A5-2, Development of damage

The initial construction cost I of the breakwater is estimated to be:
\$ 8620 for the core and \$ $1320.H_{nd}$ for the armour layer.

For design wave heights of 4, 5, 5.5 and 6 m this results in initial construction cost as per Table 3

Design wave height H_{nd}	Initial cost breakwater "C"	Initial cost Amour Layer "A"
(m)	(\$) per running meter	(\$) per running meter
4	13900	5280
5	15220	6600
5.5	15900	7280
6	16540	7920

Table A5-3, Initial Construction cost per running meter

H_{nd}	$1 < H < 1.3\ H_{nd}$ n = 4% damage			$1.3\ H_{nd} < H < 1.45\ H_{nd}$ n = 8% damage			$H > 1.45\ H_{nd}$ Collapse		
	Δp	Δw	$\Delta p.\Delta w$	Δp	Δw	$\Delta p.\Delta w$	Δp	Δw	$\Delta p.\Delta w$
(m)	(1/year)	(\$)	(\$/year)	(1/year)	(\$)	(\$/year)	(1/year)	(\$)	(\$/year)
4	1.02	420	430	$4.6\ 10^{-2}$	860	40	$3.8\ 10^{-2}$	13900	530
5	$1.5\ 10^{-1}$	530	80	$4.7\ 10^{-3}$	1060	5	$2.6\ 10^{-3}$	15220	40
5.5	$7.4\ 10^{-2}$	580	40	$2.2\ 10^{-3}$	1160	-	$8\ 10^{-4}$	15900	10
6	$2.4\ 10^{-2}$	630	15	$7.5\ 10^{-4}$	1260	-	$1.5\ 10^{-4}$	16540	3

Table A5-4, Annual risk for various values of H_{nd} per category of damage level

Note:
$\Delta p = p_l - p_{l+1}$ probability of occurrence of the wave height in the indicated interval
p_l = probability of exceedance of the wave height at the lower limit of the interval
p_{l+1} = probability of exceedance of the wave height at the upper limit of the interval
Δw = cost of repair of the armour layer (2*n*A) respectively cost of replacement (C)

This leads to the values of average annual risk $s = \Sigma(\Delta p.\Delta w)$ as per Table 5.m,m,m,m,m,m,,,

H_{nd}	$s = \Sigma(\Delta p.\Delta w)$		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	($ per year)	($ per year)	($ per year)
4	1000	570	530
5	125	45	40
5.5	50	10	10
6	18	3	3

Table A5-5, Average annual maintenance cost for various maintenance strategies

For a lifetime of 100 years, which is a reasonable assumption for a breakwater, capitalisation on an interest rate of 3.33% leads to the figures as given in Table 6.

H_{nd}	Capitalised risk S		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	($)	($)	($)
4	30000	17100	15900
5	3750	1350	1200
5.5	1500	300	300
6	540	90	90

TableA5-6, Capitalised maintenance cost for various maintenance strategies

It is now a simple exercise to add the initial cost I and the capitalised maintenance cost S as in Table 7.

H_{nd}	Total cost I + S		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	($)	($)	($)
4	43900	31000	29800
5	18970	16570	16420
5.5	17400	**16200**	**16200**
6	**17080**	16630	16630
6.5	17300		

Table A5-7, Total cost for various maintenance strategies

The optimum values are printed bold.

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