POLYURETHANE BONDED AGGREGATE REVETMENTS
DESIGN MANUAL

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Preface

Purpose of this manual

Recently, polyurethane (PU) has been introduced as a binding material in hydraulic engineering. PU proves to be able to create a durable bonding of mineral aggregate in a marine environment. The introduction of this binding material for revetments leads to the development of new materials for the protection of flood defences against erosion. The PU bonded aggregate revetment (PBA-revetment) has been tested at several pilot sites in the Netherlands and Germany. Further, the number of commercial applications in Europe is growing.

For the PBA-revetment to be considered as a competitive alternative to other types of revetments and, equally important, for the safety assessment over the lifetime of the structure to be performed properly, it is required that the designer has access to a comprehensive set of rules and guidelines for the design, construction and maintenance of new revetments with this material.

Design rules and uncertainties

In this manual design rules and guidelines are presented, based on results from laboratory research, full scale model tests, and experience and field measurements. Research is still ongoing, but from the first results some insight into the structural behaviour of PBA can be obtained.

For the mechanisms that are most relevant to the design of the PBA-revetment, calculation methods are given to determine a minimum required layer thickness. To account for any uncertainties in the material's mechanical properties and behaviour, safe assumptions have been used for material parameters.

Nonetheless, it is expected from the designer to apply these methods with caution, for each case checking if the assumptions are valid and relevant. Chapter 7 of this manual is dedicated to the remaining uncertainties and the assumptions that have been made. Also recommendations are given to minimize the risk in application of new materials.

Development of this manual

Research into failure mechanisms and material properties is performed in a joint effort by universities and the industry. This research is still ongoing and therefore also this manual is still under development; as such it is a *living document*. When additional data comes available, revisions will be made to this manual so that the design rules may become less conservative than at present.

Though the manual is intended to be used worldwide, this first version is written from a western European perspective. International contributions in future revisions will expand the range of applications and situations treated in this manual.

Contents

CHAPTER 1 Introduction

1.1 INTRODUCTION

Polyurethane is an elastomeric material for bonded revetments in coastal engineering. It opens a new range of applications and possibilities. The polyurethane bonded aggregate (PBA) revetment is an innovative plate type revetment consisting of mineral aggregate bonded together by a polyurethane adhesive. Individual crushed stones are fixed together only at their contact points, resulting in a very open structure.

Revetments play a crucial role in the construction of flood defences. Over the years, numerous varieties of revetments have been developed. These materials can be categorized into unbounded and bonded revetments, i.e. loose rock and pitched stones versus asphalt and concrete revetments. In bonded revetments the binding material plays a crucial role in the performance of the revetment.

Recently, polyurethane (PU) as a binding material has been introduced in hydraulic engineering. PU proves to be able to create a durable bonding of mineral aggregate in a marine environment. The introduction of this new binding material for revetments leads to the development of new materials for the protection of flood defences against erosion.

1.2 POLYURETHANE BONDED AGGREGATE REVETMENTS

The PBA revetment is an innovative application which combines high strength PU bonding with the ease of use and hydraulic performance of crushed stones. These stones are completely coated and fixed together permanently at their contact points. The environmentally friendly polyurethane bonds the stones to a monolithic, three-dimensional and stable structure. The low ratio of binding material causes the structure to remain completely porous which, in turn, has a positive effect on the hydraulic properties of the system.

The following properties and modes of action are attributed to PBA revetments:
- high porosity;
- energy dissipation of impacting waves;
- reduction of the wave run-up;
- resistant to UV, cold and heat;
- ecologically compatible;
- easy application and installation;
- large range of applications.

1.3 STRUCTURE OF THIS REPORT

The structure of this report intends to follow the actual steps in the design process of a PBA revetment:

- Chapters 1 and 2: Introduction to the materials and some properties of PBA.
- Chapter 3: System analysis and definition of requirements
- Chapter 4: Conceptual solutions, sizing of the structure and design of details
- Chapter 5: Construction aspects
- Chapter 6: Methods for testing, maintenance and repair, safety assessment
- Chapter 7: Remaining uncertainties and risks in design of PBA revetments

1.4 NOTE TO APPLICATION OF THE DESIGN MANUAL

Starting from 2004, PBA revetments have been constructed on several coastal sites in Germany, The Netherlands, France and Great Britain, as well as in Canada. Although constantly expanding, the experience with polyurethane bonded revetments is still relatively limited, compared to established conventional revetments.

This design manual is based as much as possible on the latest developments in research, laboratory tests and experience from the field. Research into the behaviour and characteristics of the material is still ongoing and the manual will be appended when new insights are gained.

The design methods proposed in this manual have been chosen such, that they will be conservative in most cases. Still it is very important that the reader of this manual takes notice of the remaining uncertainties and risks in the design with PBA in chapter 7.

CHAPTER 2 Polyurethane bonded aggregate (PBA) revetments

2.1 INTRODUCTION

The polyurethane bonded (PBA) revetment is a new type of revetment system for use as a cover layer on dikes. The PBA revetment is a composite of:

- polyurethane adhesive (PU);
- mineral aggregate.

The mineral aggregate is fixed together with a two-component polyurethane adhesive. In the PBA each individual rock is covered with a thin film of polyurethane. When the adhesive is cured, this film fixes the rocks together only on their contact points, creating a highly permeable, open structure.

Figure 2.1

Chemical components and mineral aggregate for PBA (Bijlsma 2008).

2.2 MATERIALS AND PRODUCTION PROCESS

2.2.1 POLYURETHANE ADHESIVE

Chemical components

The polyurethane adhesive consists of two fluid chemical components: the A component *polyol* and the B component: *isocyanate*. When mixed, these components form a strong adhesive. This adhesive cures into a durable solid polyurethane (PU). The polyurethane is chemically inert and inflammable.

Mixing ratio

The ratio of the mixture A:B is approximately 2:1. The exact mixing ratio is prescribed by the supplier of the components.

2.2.2 MINERAL AGGREGATE

Mineral aggregate

For use in a PBA revetment basically any type of coarse graded granular material can be used. The adhesive bonding of PU has been tested successfully on mineral aggregates such as basalt, granite, limestone and iron slag. The granular material can be broken and/or rounded (gravel). Two main requirements of the material are that the rocks are dry when mixed with the PU and that the amount of fines (dust) is not too high (see section 6.2 on quality control).

In practice, the PBA revetment will be constructed from a narrow graded aggregate, with stone sizes ranging from 8/11 mm to 40/60 mm. Table 2.1 shows some examples of materials and grading that have been used in PBA.

Table 2.1

Typical examples of materials and grading that have been used for PBA.

Mineral aggregate	Used grading	Density kg/m^3	Bulk density kg/m^3
Limestone	10/14 mm, 20/40 mm, 30/60 mm	2650-2700	1350-1450
Granite	16/36 mm, 40/60 mm, 50/60 mm	2600-2800	1600-1700

Other suitable materials are basalt (2900-3000 kg/m^3), iron silicate (3100-3400 kg/m^3) and flint stone. Experience with these materials in PBA is limited. The choice of aggregate type and gradation of the aggregate greatly determines the mechanical properties of the PBA revetment. The designer will have to make a trade off between hydraulic properties, mechanical properties and costs:

- use of high quality aggregate (i.e. basalt, granite) increases the bulk weight as well as the strength of the revetment;
- addition of finer fractions, a wide gradation, or combinations of narrow gradations will increase strength, but also the amount of PU needed;
- larger stone sizes increase hydraulic roughness and wave energy dissipation, but the PU bond will be less efficient, resulting in lower strength.

More information on the effect of the aggregate choice on the stiffness and flexural strength of PBA is given in section 2.4. Generally, in coastal structures application of a narrow graded aggregate, with stone sizes in the range of 20/40 mm to 30/60 mm will result in the most efficient construction. These stone sizes result in an optimum number of contact points, applied amount of PU and overall stability.

SECONDARY MATERIALS

The fact that aggregate with a relatively large stone size can be used in a PBA revetment, makes it possible to use (secondary) materials that have not been utilized in coastal structures before. An example is the reuse of railway ballast, which is a mix of broken and rounded aggregate with a grading of 30/60 mm. After the iron deposits have been washed off, the railway ballast can be deployed as any other mineral aggregate.

2.2.3 PREPARATION AND PRODUCTION PROCESS

PBA can be quickly prepared on site. The clean and surface-dry stones are mixed with binding material by a mixer and installed at the location.

Mixing ratio

The granular aggregate is mixed together with approximately 2-3 vol. % of PU. For optimum strength of the composite it is required that each individual rock in the aggregate is completely covered with PU. The exact amount of PU needed for this will be prescribed by the supplier of PU and depends upon:

- grain size distribution;
- shape of the rocks;
- amount of fines.

Mixing process and application

The mixing of the PU with the granular aggregate is not principally different from the mixing of concrete. The process of mixing the PU and the aggregate takes approximately 3 minutes, after which the mixture can be applied. The PU starts curing after 20 minutes (at 23 ºC) and reaches its final strength after 24 to 48 hours.

In Figure 2.2 the principal steps in the manufacturing process of PBA is shown. Further detailed information on construction aspects can be found in chapter 5 of this manual.

Figure 2.2

Steps in the (small scale) production process of a PBA revetment.

2.3 SYSTEM PROPERTIES

2.3.1 GENERAL

In the composite system, each individual rock is covered with a polyurethane coating. At their contact points, this coating forms a durable connection between the rocks. The result is a high strength plate type revetment, with a high open pore volume.

2.3.2 POROSITY

The polyurethane adhesive does not fill the open pore volume of the mineral aggregate. The sticky nature of the adhesive even prevents, to some level, the aggregate from reaching a more dense packing. The porosity of the composite is therefore equal to or somewhat higher than the natural porosity of the bulk granular material, which depends on the choice of stone size and grading. The porosity *p* can be determined by comparing the bulk density of the aggregate to the density of the rock:

$$p = 1 - \frac{\rho_{bulk}}{\rho_s}$$

In which

ρ_{bulk}	= bulk density of mineral aggregate	[kg/m^3]
ρ_s	= density of mineral aggregate	[kg/m^3]

Choice for narrow graded aggregates results in a high porosity of the end-product. Porosities of up to 50% have been achieved for 8/11 mm, 10/14 mm and 20/40 mm grading. This is significantly higher than conventional open revetments such as open stone asphalt. A comparison is given in Figure 2.3.

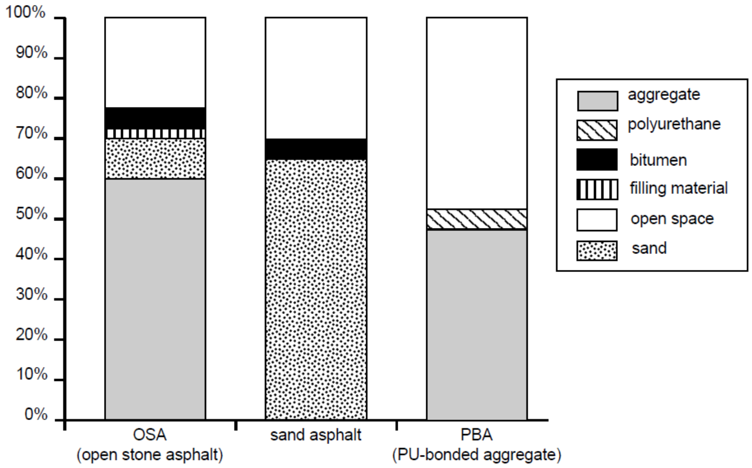


Figure 2.3

Volume percentages of the components in polyurethane bonded aggregate compared with two asphalt-based mixtures (Bijlsma, 2009).

2.3.3 HYDRAULIC CONDUCTIVITY

Standard rules for granular filters apply

The hydraulic conductivity is a measure for the ease with which water can move through the pore spaces of a material. Because the open pore volume of a polyurethane revetment is approximately equal to the natural open pore volume of the used aggregate its hydraulic conductivity is also approximately equal to that of the aggregate. Standard filter rules for granular filters also apply to the PBA revetment. See Annex 5 for calculation rules.

Special case: clogging of the revetment

Under normal conditions the permeability of a PBA revetment is such that overpressures are quickly relieved through the open pore space of the structure. However, it is possible that this open space is filled with fine sediments. This phenomenon of clogging of the PBA revetment has been observed under prototype conditions. Clogging of the structure strongly affects the permeability. See Annex 4 for example calculations with a silted up revetment.

2.4 MECHANICAL PROPERTIES

2.4.1 GENERAL

Once applied on a slope, the PBA revetment acts as an elastic, monolithic and porous plate. The basic mechanical behaviour can be described with the properties:

- stiffness;
- viscoelastic behaviour;
- flexural strength;

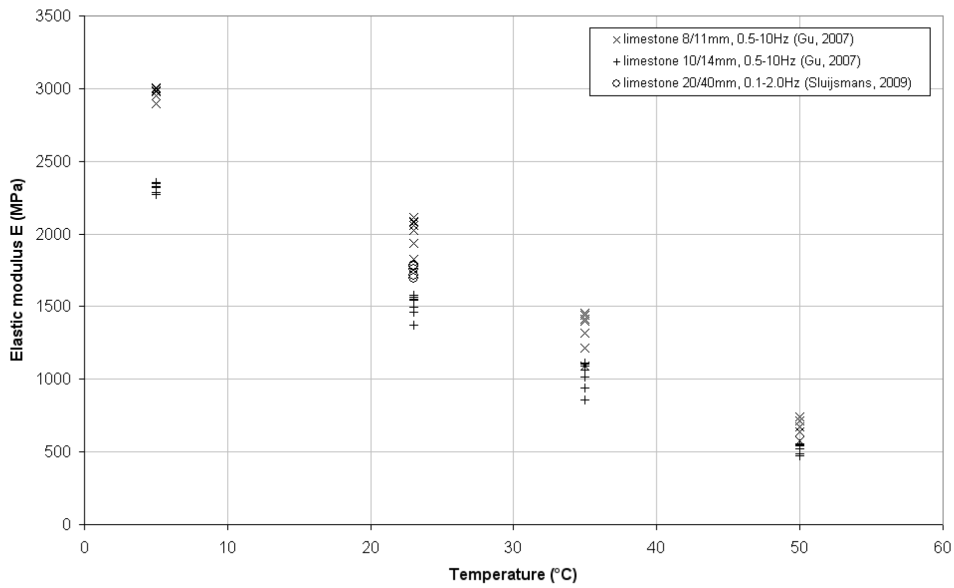
2.4.2 STIFFNESS

The stiffness of the PBA composite can be described with the elastic modulus E . Stiffness is an important design parameter. A higher value results in higher stresses in the material under the same load and with the same deformation. The stiffness is not a constant, but strongly varies with temperature and loading time. At lower temperatures and short (impact) loads the material behaves relatively stiff.

Generally, the temperature of the revetment can be assumed to follow that of the (sea) water temperature. For preliminary design of the PBA revetment in a moderate climate and loaded by wave impact the stiffness can be assumed at E = 3,000 MPa.

Figure 2.4

Stiffness of PBA related to temperature.



Apart from temperature, which has the most significant effect, stiffness is also influenced by:
- type, stone size and grading of the mineral aggregate;
- stone-to-PU ratio;
- mixing ratio of the PU components;
- loading time.

Given the variation of stiffness with abovementioned parameters it is strongly advised to determine the design value, specific to the chosen mixture and conditions, from dynamic bending tests such as the frequency sweep test.

Viscoelastic and time dependent behaviour
The behaviour of PBA is mainly elastic. Creep measurements at the Dutch KOAC/NPC have shown that PBA creeps much less than asphalt, at the relevant lower temperatures [lit. 8]. This means that the plate revetment shows no time dependent (viscous) plastic deformations, which is an important aspect to consider in the structural design (see section 3.3). Therefore, extra attention must be paid to the occurrence of differential settlements in the revetment foundation.

2.4.3 FLEXURAL STRENGTH

The flexural strength describes the maximum bending stress that the PBA can withstand before breaking. The flexural strength of a PBA revetment depends strongly on the grain size and grain size distribution, but also on the type and quality of the aggregate.

Tests on PBA mixtures with narrow graded limestone aggregate up to 30/60 mm show that the flexural strength strongly decreases with increasing stone size. Indicative values for preliminary design are given in the table below.

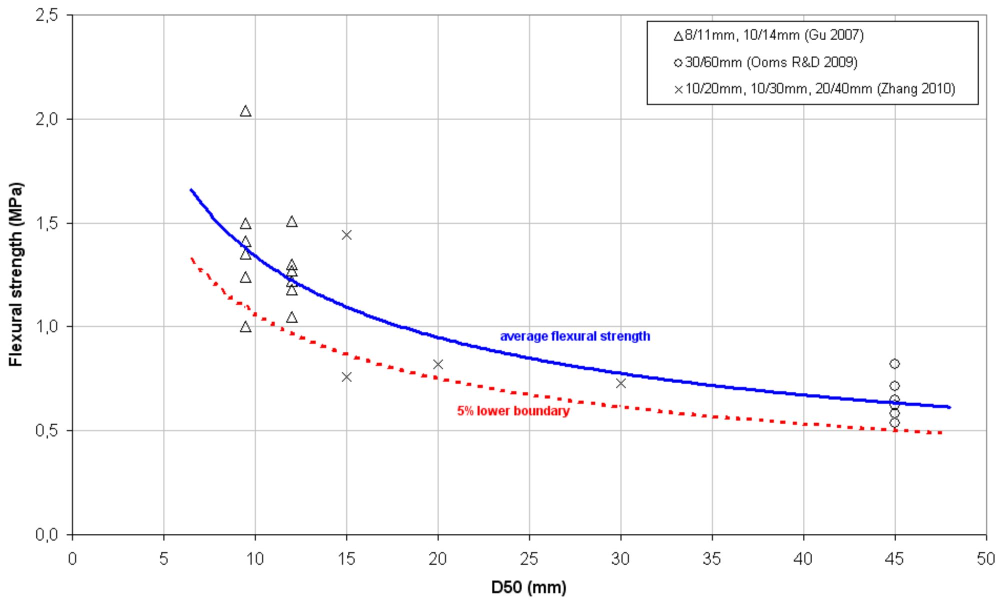
Table 2.2
Flexural strength of several PBA mixtures.

Mixture	Average flexural strength	Flexural strength for preliminary design
Limestone 8/11 mm	1.26 MPa	1.09 MPa
Limestone 10/14 mm	1.42 MPa	0.97 MPa
Limestone 20/40 mm	0.78 MPa	0.62 MPa
Limestone 30/60 mm	0.65 MPa	0.50 MPa

Research into these relations is still ongoing. It is therefore advised to test the flexural strength of the proposed mixture in order to establish the design value for flexural strength in detailed design. Preferably the results of these tests are shared with the manufacturer, so that they can be taken along in future versions of this manual.

Figure 2.5

Flexural strength of PBA related to stone size.



2.5 WAVE-STRUCTURE INTERACTION

2.5.1 GENERAL

The porosity of the PBA revetment influences the behaviour of incoming waves. The open structure allows the wave impact pressures to partly dissipate within the revetment, and partly to be transferred to the subsoil. Also, the hydraulic roughness of PBA strongly reduces the extent of wave run-up and run-down over the revetment, compared to other revetments. When waves do reach the top of the structure and overtopping occurs, the PBA revetment has proved to be more than capable of withstanding overtopping discharges.

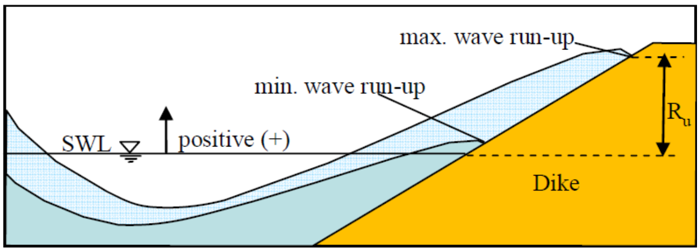
The following section describes some important characteristics of the wave-structure interaction of the PBA revetment. Wave run-up and run-down have a direct relation to the determination of the upper and lower extent of the revetment, as treated in section 4.2.5 of this manual. Wave pressures are important for the design for wave impacts, as described in section 4.3.3 of this manual. Knowledge of the resistance to overtopping waves is related to design for currents (section 4.3.2) and can be very useful in conceptual design of the revetment (allowance of overtopping).

2.5.2 REDUCTION OF WAVE RUN-UP

The wave run-up R_u is defined as the maximum elevation from still water level (SWL) to the point to which the water rises on the seaward face of the revetment. R_u is important in defining the required height of the structure. Generally, the run-up level exceeded by 2% of the incident waves ($R_{u2\%}$) is commonly used for de sign purposes. $R_{u2\%}$ generally depends on the wave height, the so-called surf similarity or breaker parameter, the geometry and surface roughness of the slope as well as on the permeability of the structure.

Figure 2.6

Definition of run-up levels [lit. 11].



Wave run-up on PBA

For highly permeable structures with a rough surface, such as the PBA revetment, most of the energy dissipation takes place at the structure face and within the revetment. This results in different wave run-up behaviour on PBA than with other, less permeable, revetments. The wave run-up can be reduced by 25% and up to 50% with a PBA revetment in comparison to an impermeable slope. The potential reduction of the wave run-up can be taken into account when designing and planning a PBA revetment.

Figure 2.7

Strong reduction of wave run-up over the PBA revetment (upper part of picture) compared to the block revetment in the front. (photo: BASF)

The wave run-up was examined at a slope of 1:3 within the scope of large-scale model tests carried out in the Large Wave Flume in Hannover (LWI 2010) [lit. 11]. Based on the model proposed in EurOtop, 2007 [lit. 16][b] the wave run-up for PBA can be predicted by:

$$\frac{R_{u2\%}}{H_{m0}} = 0.54 \cdot \left[1.65 \cdot \xi_{m-1,0}\right] \qquad \text{for } \xi_{m-1,0} < 2.7$$

With a maximum of

$$\frac{R_{u2\%}}{H_{m0}} = 0.77 \cdot \left[4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right] \quad \text{for } \xi_{m-1,0} \geq 2.7$$

With

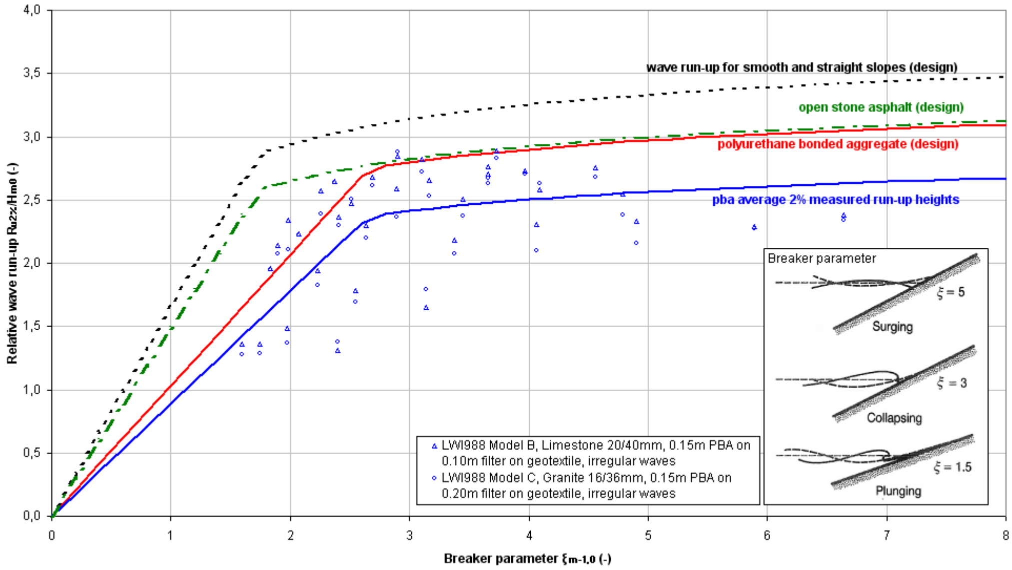
$R_{u2\%}$	= wave run-up level exceeded by 2% of incident waves	[m+SWL]
H_{m0}	= mean spectral wave height	[m]
$\xi_{m-1,0}$	= breaker parameter from spectral wave period $T_{m-1,0} = m_{-1}/m_0$	[-]

and a variation coefficient σ' = 0,159.

For deterministic design and assessment purposes it is recommended not to use the average values but a safer approach. In design practice a safety of one standard deviation above the average is used. Figure 2.8 shows the line for the average relative wave run-up as a function of the breaker parameter as well as the line that can be used for deterministic design. It should be considered that these values are based on a single series of tests at the LWI, with significant wave heights up to 1.2 m.

Figure 2.8

Relative run-up as a function of the breaker parameter for tests performed at the Large Wave Flume [lit. 11] in comparison to an impermeable slope.



INFLUENCE OF WAVE HEIGHT ON RUN-UP REDUCTION

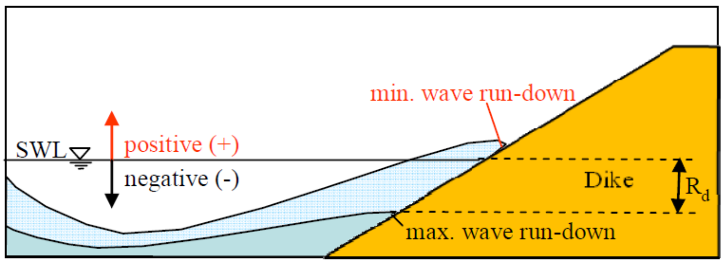
The reducing effect of PBA on the wave run-up is strongest with low wave heights. The wave-flume tests were performed with significant wave heights up to approximately 1.2 m. With the same stone size, larger wave heights will result in a higher relative wave run-up. In turn, choice of a larger stone size for relatively small waves results in a very strong reduction of wave run-up. This effect should especially be taken into account when designing structures subjected to large significant wave heights or when relatively small stone sizes are selected.

2.5.3 REDUCTION OF WAVE RUN-DOWN

The wave run-down R_d is defined as the minimum elevation from still water level (SWL) to the point to which the water surface falls on the seaward face of the revetment (Figure 2.9). R_d is important in defining the required elevation of the revetment under SWL. It is also important for the uplift pressure on the revetment as a result of the internal water level which is generally higher than the external water level during the down rush process. The run-down level exceeded by 2% of the incident waves ($R_{d2\%}$) is commonly used for design purpose. $R_{d2\%}$ generally depends on the wave height, the surf similarity parameter, the geometry and surface roughness of the slope as well as on the permeability of the structure.

Figure 2.9

Definition of run-down levels [lit. 11].



The wave run-down was examined at a slope of 1:3 within the scope of large-scale model tests carried out in the Large Wave Flume in Hannover [lit. 11]. Based on the model proposed in EurOtop, 2007 the wave run-up for PBA can be predicted by:

$$\frac{R_{d2\%}}{H_{m0}} = -0.42 \cdot \xi_{m-1,0} + 0.17 \quad \text{for } \xi < 5.7$$

With a maximum of

$$\frac{R_{d2\%}}{H_{m0}} = -2.25 \text{ for } \xi \geq 5.7$$

With

$R_{d2\%}$	= wave run-down level exceeded by 2% of incident waves	[m+SWL]
H_{m0}	= mean spectral wave height	[m]
$\xi_{m-1,0}$	= breaker parameter from spectral wave period $T_{m-1,0} = m_{-1}/m_0$	[-]

and a variation coefficient σ' = 0,061.

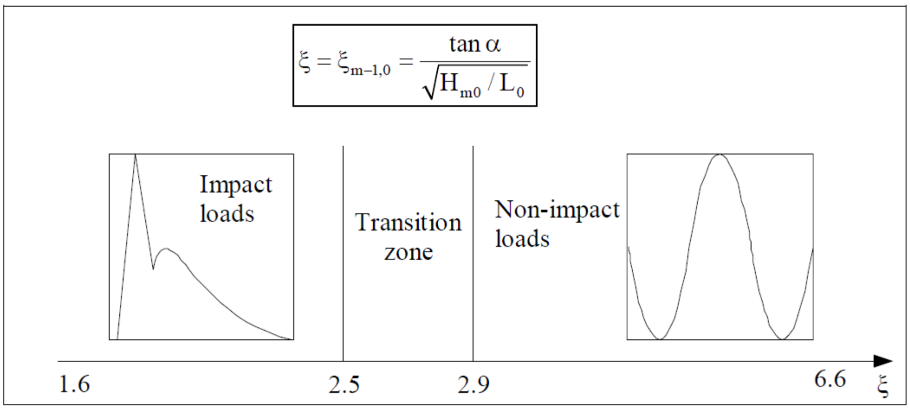
2.5.4 WAVE PRESSURES

Impact and non-impact load

It is necessary to differentiate wave loads (see Figure 2.10) with regard to the breaker types. Collapsing breakers result in impact loads. Surging breakers, on the other hand, tend to non-impact loads.

Figure 2.10

Classification of wave loads on the PBA revetment [lit. 11].



The impact load can be described by the impact factor q. This factor relates the maximum wave (impact) pressure p_{max} to the significant wave height H_{m0} :

$$p_{\max} = q \cdot \rho \cdot g \cdot H_{m0}$$

Where

p_{max}	= maximum wave pressure	[MPa]
q	= impact factor q	[-]
ρ	= specific density of water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]
H_{m0}	= mean spectral wave height	[m]

Results of wave flume tests

Within the scope of the model tests in the Large Wave Flume at Hannover [lit. 11], comprehensive analyses of the wave loads on and beneath a PBA revetment were performed. In these tests the wave impact factor q for PBA was quantified.

Figure 2.11

Waves just before impact at the large wave flume in Hannover [lit. 11].

In the Figure 2.12 the results of the pressure loads on and beneath the PBA revetment are presented for the two load types: impact load and non-impact load.

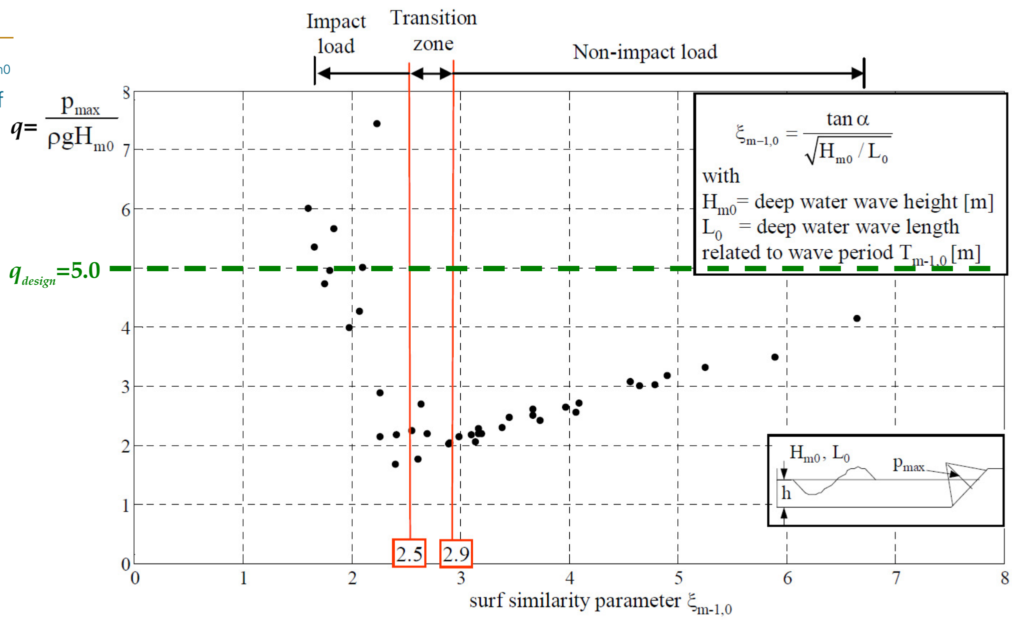


Figure 2.12

Maximum pressure $p_{max}/\rho g H_{m0}$ on the revetment against surf similarity parameter $\xi_{m-1,0}$ for irregular wave tests [lit. 11].

For impact load *q*-values up to 6 were found. For the transition zone q-values in the order of 2 were found. For non-impact loads *q*-values up to 4 were found. Furthermore the following was observed:

- in case of impact loads ($\xi_{m-1,0}$<2.5) part of the impact pressure is damped within the structure. Just beneath the revetment the *q*-values are reduced by a factor of about 0.6;
- in case of the non-impact loads, virtually the same pressure rates appear on and beneath the revetment;
- the largest deformations of the PBA revetment were found as a result of the quasi-static or non-impact part of the loads.

Design

Based on the tests with irregular waves in the large wave flume in Hannover, for design of the PBA revetment a factor $q_{design} = 5.0$ is advised. This value is valid for irregular wave fields with waves up to H_{m0}=1.5 m and takes damping of impact-loads (for $\xi_{m-1,0}$<2.5) into account.

Further information on the wave impact parameter is found in the insertion text on the following page and in Annex 1.

RELATION TO OTHER TESTS

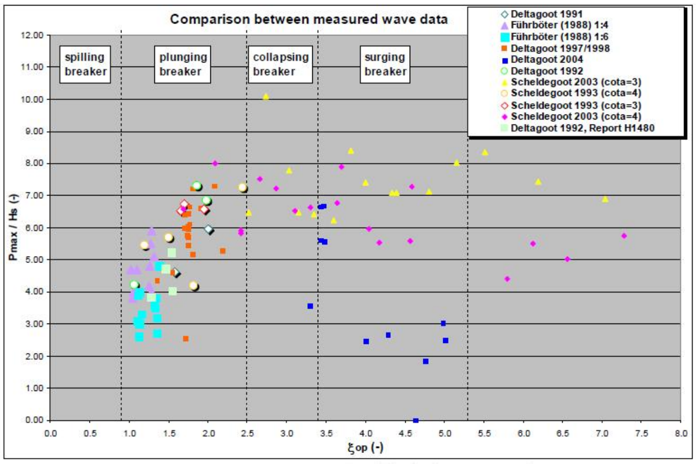
In the Netherlands, wave impact pressures have been subject of research for the past two decades. When the results of the tests at the LWI in Hannover from Figure 2.12 are compared to historic measured data from the Dutch wave flumes in Figure 2.13, at first hand a principle difference can be discerned: the impact pressures have a maximum at approximately ξ_{0p}=2.0-2.5, whereas the LWI tests show a minimum at $\xi_{m-1,0}$=2.5.

However, comparison of these tests is difficult. For the analysis of the LWI data, the deep water wave height H_{s0} and the mean wave period $T_{m-1,0}$ were used instead of H_s and T_p resulting in an offset of the surf similarity parameter. Generally, it can be said that $T_p \approx$ 1.1-1.25 $T_{m-1,0}$ such that also $\xi_{0p} \approx$ 1.1-1.25 $\xi_{m-1,0}$. Therefore, the LWI tests did not cover the surf similarity parameter < 2.0 of comparative data, which might explain the absence of the rising tendency on the left side of the graph.

In order to confirm the abovementioned, in future tests extra attention should be paid to the spilling and plunging breaker regime ($\xi_{m-1,0}$ < 1.6)

Figure 2.13

Overview measured data for Dutch wave flumes [lit. 30].



2.5.5 WAVE OVERTOPPING

In The Netherlands, tests with the PBA revetment were carried out on the inner slope of a dike. In these tests, large quantities of water were discharged over the inner slope from a water tank placed on the crest of the dike (Figure 2.14) in order to analyse the revetments' structural integrity. During these tests, the PBA suffered no damages from overtopping volumes of up to 125 l/s/m. For comparison, it should be noted that overtopping volumes ranging between 0.1 up to 10 l/s/m are taken into account when planning and designing flood control structures. From this it can be inferred that the structural integrity of the revetment can be taken for granted even in case of large overtopping volumes.

Figure 2.14

Tests on the inner slope of a dike.

2.6 DURABILITY

Hydraulic engineering structures are expected to have a life time of 20 years and more. All construction materials on inorganic (cement-bonded) or organic (plastics) basis are subject to the impact of weathering when used outdoor. Weathering is the adverse response of a material to climate, often causing undesirable early product failures. Particularly damage caused by ultraviolet (UV-) radiation, salt water and frost must be considered.

2.6.1 RESISTANCE TO UV-RADIATION

Compressive strength tests under the influence of UV-light have been performed at the laboratories of Atlas Material Testing Technology GmbH, located in Germany. The used Xenon Arc Lamps mimic the natural light source and it is preferred as a light source when the tested material will be exposed to natural sunlight.

For the tests in Atlas 16/32 mm basalt stones were prepared in 10 cm x 10 cm x 10 cm sample moulds with 1.8 wt% PU. The material was irradiated for 12,000 hrs. The stability of these cubes was measured by compression tests. After 12,000 hrs the UV-light has no negative influence on the compressive strength of the PBA samples. The test condition of 12,000 hrs of UV-sun light simulation is comparable to 12 years of real sunlight in middle Europe (ISO 4892-2:2006).

Figure 2.15

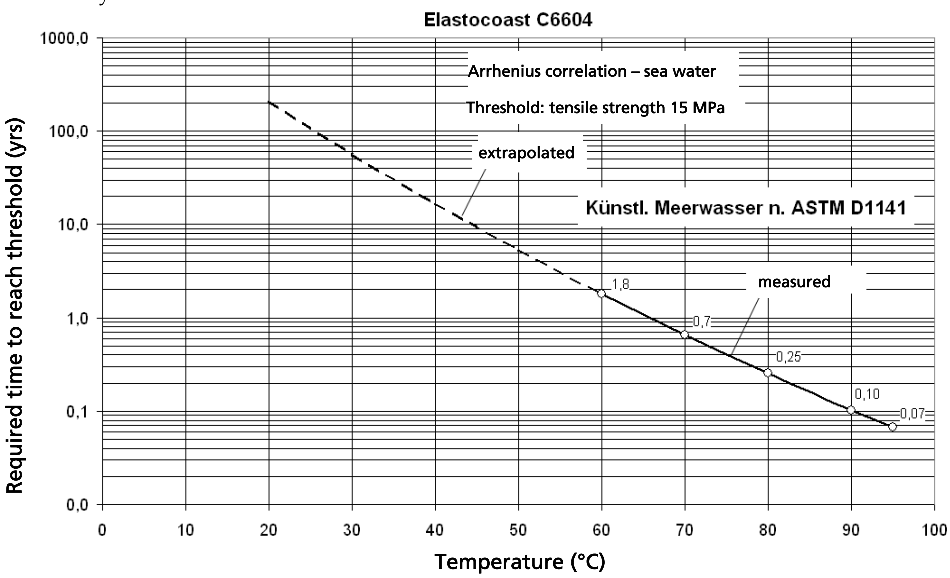
Exposure to UV light in the Atlas laboratory and testing on compressive strength of PBA samples (BASF, 2010).

2.6.2 RESISTANCE TO SALT WATER

The resistance of PBA against salt water was validated by use of the so-called Arrhenius-correlation. In doing so, the degradation process that takes place in sea water was accelerated by increasing the temperature in small steps up to 90 °C. In this way the results can be extrapolated back to 20-30 °C. This extrapolation shows that the PU-material is fully stable in salt or sea water at service temperature of 20-30 °C and it has a lifetime expectation of 80-100 years.

Figure 2.16

Expected lifetime of PBA under influence of salt water (BASF, 2009).



2.6.3 RESISTANCE TO FROST

The STB (Prüfinstitut für Baustoffe und Umwelt GmbH) performed freeze/thaw tests in September 2007 according EN 13 383-2. Their conclusion is that the investigated samples are to be judged as sufficiently resistant against freeze/thaw cycles.

Exposure to high temperatures is not yet tested. The hardened PU mixture is inflammable.

2.7 ENVIRONMENTAL ASPECTS

2.7.1 EFFECTS ON THE AQUATIC ENVIRONMENT

Chemical components and their emission

The PBA revetment uses a two-component adhesive to achieve bonding of mineral aggregate. Apart from the mineral aggregate, the environmental effects of the basic chemical components are well known:

- polyols: these compounds are derived from fatty acids; they pose no threat to the aquatic environment;
- products based on natural castor oil, zeolite and silicon oil: these are non-toxic and naturally degradable;
- P-MDI (*diphenylmethane-4,4'-diisocyanate*): this is a very reactive substance with potential adverse effects to the aquatic environment.

The P-MDI component has the most risk potential and may react with water to the carcinogenic MDA. However, the concentrations of MDA emitted from the polyurethane are so low that they form no risk for the aquatic environment [lit. 14]. After mixing of the chemical components the gross of the P-MDI component reacts with the other components to form chemically inert polyurethane. Emission of MDA from the polyurethane is minimal both during and after the curing phase. The resulting concentration of MDA in the aquatic environment is well below the PNEC (predicted no effect concentration) as established in a risk as established in a risk assessment by the European Union [lit. 29].

Bioassays

Both the short-term and long-term effects of PBA have been assessed in bioassays (toxicity tests). The potential adverse effect of unhardened and hardened PBA have been analysed by the independent Institut Fresenius in Germany and INTRON in the Netherlands. It has been determined that no there are no negative effects on the aquatic environment.

Unhardened composite

Institut Fresenius in Germany analyzed uncured or partially cured samples regarding PH-value, electric conductivity, organic content (total organic carbon) and bioluminescence bacteria [lit. 1]:

- the PH value changes are negligible in alkaline range in comparison to the blank sample without PU;
- the electric conductivity – as a chosen unit of measurement for dissolved inorganic matter – is constant. That means that no inorganic matter is dissolved by use of the polyurethane coating;
- compared to the blank sample (basalt without PU) the organic content decreases significantly. It can be assumed that organic matter sticking to the basalt is bound by the PBA system;
- toxicity to bioluminescence bacteria is not found in any sample.

Hardened composite

The fully cured polyurethane is chemically inert. Once hardened, there are no emissions from the PBA revetment. Therefore, PBA revetment forms no threat to surrounding flora and fauna [lit. 14]. In the Netherlands and in Germany INTRON has taken samples from 5 projects. All samples from the 5 projects have been found to be within the limits from the local resolution about construction material (*Bouwstoffenbesluit*).

APPLICATION OF PBA IN THE NETHERLANDS

In order to achieve approval for application of PBA in the Netherlands the Dutch authorities require that, both from technical and environmental perspective, contact between the unhardened chemicals and the aquatic environment should be prevented. To prevent potential adverse effects it is therefore advised to stop production of the PBA during rainfall and allow at least 1 hour of curing time between application of the PBA revetment on the dike slope and first contact with (rising) water. Also, it is advised to start the (daily) construction of the revetment at the lowest point on the dike slope and work in upward direction. In doing so, maximum curing time is realised.

Safety

The individual components of the PU-adhesive should be handled as being hazardous substances. Therefore, safety precautions must be taken in storage and handling of the components on the construction site (see section 5.3). Once the components are mixed together they react to a non-hazardous adhesive.

2.7.2 ECOLOGICAL POTENTIAL

The PBA revetment can provide ecological benefits by acting as novel habitat for biological colonisation. Research at the University of Amsterdam [lit. 10] has shown that PBA seems to be a material which allows ecological recovery to be fast and according to the typical vegetation of that area:

- field and laboratory research has shown that polyurethane coated rock forms a suitable, hard substrate for colonization by micro-algae;
- the open structure of the composite creates an attractive habitat for vegetation and other organisms.

Experience from the field shows that dike vegetation has returned within 4 months after construction. Moisture availability plays a key role in recovery of the biological community. A strong zonation is present with a concentration of growth around the water line.

A surface treatment by the application of sand on the revetment surface (section 4.2.8) could promote algal attachment. However, it is not necessary for recovery of the biological community on the substrate.

Figure 2.17

Growth of barnacles and mussels on PBA [lit. 4].

2.7.3 SUSTAINABILITY IN DESIGN

In order to achieve a sustainable design, three sustainability criteria can be considered, being:

- eco-efficiency;
- building materials and waste;
- spatial quality.

These criteria are not limited to the construction of the revetment itself, but extend to every step of its lifecycle, starting from the choice and production of the building materials, transport and construction, and finally demolition and disposal of the structure.

Eco-efficiency

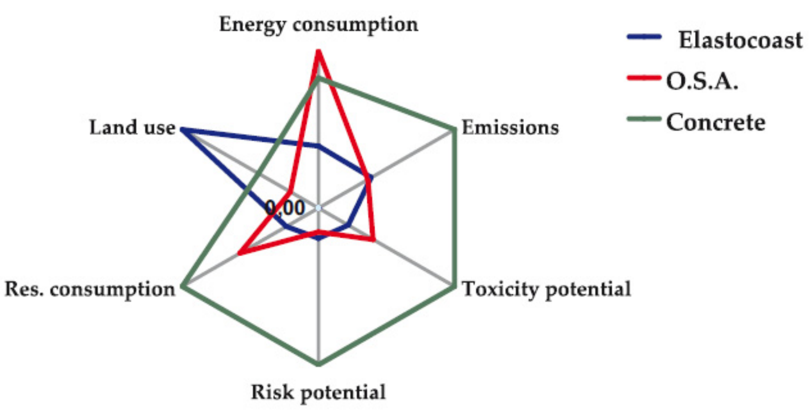
An eco-efficiency analysis has been performed to provide a total cost determination and the calculation of ecological impact over the entire lifecycle of a PBA revetment. Based on a 20,000 m^2 project PBA versus traditional coastal protection methods such as concrete and open stone asphalt were analyzed concerning their eco-efficiency (ISO 14040). The environmental impact of the different alternatives is calculated based on six categories:

- energy consumption;
- emissions;
- toxicity potential;
- risk potential;
- resources consumption;
- land use.

The combination of these individual data provides the total environmental impact of a PBA revetment and allows comparison with two alternative systems: asphalt and concrete (Figure 2.18).

Figure 2.18

Environmental impact of six categories (BASF, 2008).



Compared to open stone asphalt (O.S.A.) and concrete, the PBA revetment (*Elastocoast* ®) has a low impact on environment. The category "land use" is the only category where PBA has a higher impact due to cultivation of the renewable resources (fatty acid esters), which are used to produce PU.

Traditional open stone asphalt has a very high impact in the category “Energy consumption” due to high consumption of fuels and energy for bitumen production, processing and energy content of bitumen itself. In contrast to bitumen, PU does not have to be heated at all during transport and application.

Concrete has the highest impact in most categories. “Resources consumption” is high mainly due to high construction masses of cement and stones extraction. “Risk potential” is originated from the higher amount of stones and cement used and higher number of working hours in the construction site: both aspects have high impact on industrial accidents. The high “toxicity potential” is based on toxicity of materials used in the pre-chain and labelling of the mortar.

Building materials and waste

The PU-adhesive is made out of approximately 50% renewable, re-growing fatty acid esters of natural origin. The bulk of the revetment consists of mineral aggregate. Basically any type of coarse graded aggregate can be used, which creates an opportunity to make use of recycled material such as cleaned railway ballast material[1]. Because of the relatively large stone size of such ballast material, it is unsuitable for use in conventional revetments such as open stone asphalt.

At the end of its lifetime the composite of cured PU and stones can be handled as non-hazardous waste [lit. 1]. According to European Waste Catalogue [lit. 22], separately both cured PU and stones are non hazardous waste. As demolition waste with content of less than 5 vol % PU recycling as building material is possible, otherwise normal waste disposal as building material is possible or recycling after burning of the PU.

Spatial quality

The polyurethane coating is almost transparent, which makes the appearance of a PBA revetment the same as naturally set loose stones. Use of different types of rock give a different appearance (colour) to the revetment and a top layer of sand can be used to roughen the surface if desired. The natural appearance, as well as the material suitability for ecological development, gives a positive contribution to spatial quality.

[1] In 2009 recycled material in the form of cleaned railway ballast (grading 30/60 mm) was successfully applied on a 12,000 m^2 PBA slope revetment in the Bathpolder (NL).

CHAPTER 3 Applications of the PBA revetment

3.1 INTRODUCTION

The design process of revetments starts with a characterization of the revetment and functional requirements that the revetment must fulfil. What are the specific conditions and (hydraulic) loads that are characteristic to the system where the revetment is to be realised? What are the functions of the revetment?

In any project where a revetment is to be applied certain requirements must be met. In general, the following requirements can be distinguished:

- legal requirements;
- primary functional requirements;
- secondary functional requirements;
- specific demands.

Legal requirements include local and/or national laws that ensure the safety of the structure that is to be protected by the revetment. Also legal requirements can restrict the use of construction materials that have adverse effects on the (aquatic) environment, in order to ensure quality of ecological habitats or supplies of drinking water. For environmental aspects of PBA reference is made to section 2.5 of this manual.

Primary functional requirements directly apply to the ability of the revetment to resist hydraulic loads and protect the underlying structure against erosion. This results, for example, in a minimum required layer thickness or in the application of a filter layer. Primary functions vary depending on the type of revetment and the type of hydraulic loads that are relevant to the structure. Secondary functional requirements are additional requirements that allow the revetment to also be used for secondary functions. Examples of secondary functions are: accessibility by vehicles, recreational purposes or ecological development. In sections 3.2 and 3.3 of this manual the characterization of revetments and resulting functional requirements are further elaborated.

Finally, specific demands are aspects of the revetment that do not directly contribute to its primary or secondary functions but can determine certain properties of the revetment or its construction method. For instance, a quick placement of the revetment may be a must if the time window is short (i.e. tidal zone). In chapter 5 of this manual the construction aspects of PBA revetments are discussed.

3.2 CHARACTERIZATION OF THE REVETMENT

3.2.1 GENERAL

In order to determine whether the PBA revetment is a suitable choice for an application area, first insight should be obtained in the local conditions and loading of the structure.

Geographical location
A revetment system is firstly characterized by the geographical location of the planned structure and secondly by the location of the revetment within the profile of the structure. The geographical location or surroundings determine the hydraulic climate that a structure is subjected to. The hydraulic climate embodies the (variation of) water levels, currents and the wave climate. For instance, in a coastal area, a sea dike is often exposed to attack by extreme wind waves, whereas a river dike has to cope with extreme discharges that cause enduring high water levels and currents.

In this manual, the following water defences are analyzed based on Dutch conditions:
- river dikes;
- lake dikes;
- sea dikes;
- dunes;
- breakwaters and groynes;
- upgrading of existing revetments;
- special structures.

Location within the structure
When the hydraulic climate is determined, the nature and severity of the loads on a part of the structure depend on the location within the geometry of the structure. Each water defence structure can be subdivided into sections, according to the location along the:
- longitudinal/horizontal profile;
- transversal/vertical profile.

The longitudinal profile describes the horizontal position and orientation of the water defence within the water system. Lengthways, the nature and severity of loads can vary.

The transversal profile is determined by the size and shape of the water defence. The location within this profile determines the possible load cases. For instance, the lower slope of a structure is subjected to different loads than the crest, and the outer slope different from the inner slope.

3.2.2 DESCRIPTION OF REVETMENTS

For each of the abovementioned systems the following structure is used to present the system characteristics.

Hydraulic climate and characteristic loading
First the characteristic features of the hydraulic climate are summed up. A distinction is made in water levels, waves and currents:

Horizontal location within the profile
A short description is given of the possible spatial variation in hydraulic loads on the structure.

Vertical location within the profile

One or more typical cross profiles are given, in which different zones are distinguished. Each zone has its characteristic loading types. In an overview table the relevant load cases per zone are given together with a reference to the section in this manual in which design rules are provided for this load case.

SYSTEM	Section	Zone I	Zone II	Zone III	Zone IV	Zone V	Zone VI
Load case 1	§4.4.x	0	+	-	-	-	-
Load case 2	§4.4.x	+	+	0	0	-	-
Load case 3	§4.4.x	-	-	-	+	+	+

3.2.3 RIVER DIKE REVETMENT

Hydraulic climate and characteristic loading

Water levels

- Most of the time low discharge occurs and accordingly low water levels. Incidentally high discharges can occur because of excessive rainfall or water originated from ice melting in upstream areas. High river discharges can last for a relatively long period (10 days).
- Water levels can be increased or lowered as a result of wind setup, which depends on wind speed, duration and fetch.
- At the downstream side of the river, where the river flows into the sea, the water level is also influenced by the tidal regime. Depending on the tidal difference this influence can extend over a large distance upstream.

Waves

- Development of wind waves on a river dike is relatively small, because of limited fetch. The size of the wind waves is independent of the water level.
- Waves may carry floating debris (i.e. drift wood).
- The combination of high waves and a high water level may cause overtopping of the structure. This can play a role when during high discharges also a strong wind is present.
- Passing vessels introduce ship-induced waves, which can be significant in size, but in general do not occur simultaneously with extreme discharges or storm conditions.

Currents

- Currents are always present in streaming rivers. Higher currents occur in outside bends and narrow parts.
- High discharges are accompanied by high flow velocities.
- Passing vessels introduce ship-induced currents (return currents) and propeller/thruster jets.
- Currents may carry floating debris (i.e. drift wood).
- In winter, ice can form and be carried along with the flow.

Horizontal location of the profile

- During a storm wind waves are generated with a wave direction approximately equal to the wind direction. The natural path of a river shows bends and turns. Consequently parts the direction of incoming waves relative to the dike differs from location to location.

- Attention should be paid to locations where special load cases may apply, such as ship turning circles or near hydraulic structures.

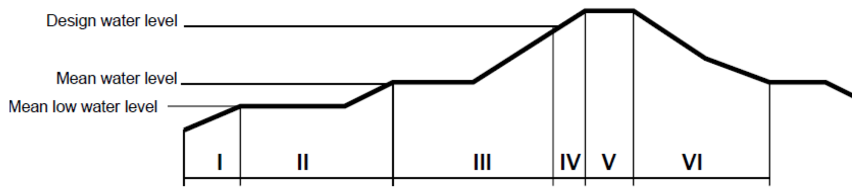
Vertical location within cross profile

Within the cross profile of a river dike, the following zones can be distinguished (see also Figure 3.19 on page 30):

- Zone I – The toe of the dike is permanently submerged, and mostly subjected to currents;
- Zone II – The lower slope of the dike is between mean low water level and mean high water level (only in regions with tidal influence). This zone is frequently subjected to waves and currents. After high water levels, overpressures can develop behind an impermeable revetment;
- Zone III – The upper slope is between mean high water and design water level. During high discharges this zone is loaded by waves and strong currents. In the lower part of this zone overpressures can develop behind an impermeable revetment;
- Zone IV – Above design water level the river dike is loaded by wave run-up. Up to approximately $0.5 \times H_s$ above design water level wave impacts loads are still present;
- Zone V – The crest of the dike is often used for traffic. During extreme water levels and high waves this zone can be loaded by overtopping waves;
- Zone VI – The inner side of the dike is only loaded during wave overtopping and overflow. At the lower part of the inner slope, overpressures may develop behind an impermeable revetment.

Figure 3.19

Typical cross profile and loading zones of a river dike.



The table below shows per zone the load cases that should be considered in design of a revetment and the section of this manual in which the load case is elaborated. The table is indicative; for each specific design situation the designer should decide which load cases are most relevant.

Table 3.3

Load cases per zone.

RIVER DIKE REVETMENT		I	II	III	IV	V	VI
scour / undermining	§4.2.7	+	-	-	-	-	-
ship induced currents	§4.3.2	+	+	-	-	-	-
ship induced waves	§4.3.3	-	+	-	-	-	-
natural currents	§4.3.2	+	+	+	-	-	-
wind wave impact	§4.3.3	-	+	+	+	-	-
wind wave run-up /down	§4.2.6	+	+	+	+	-	-
overpressures	§4.4.4	-	+	0	-	-	-
overtopping	§4.3.2	-	-	-	-	+	+
ice loads	§4.3.5	-	+	+	+	+	-
traffic loads	§4.3.4	-	-	+	-	+	-

Legend:

+	*relevant*	0	*present but usually not dominant*	-	*not relevant*

3.2.4 LAKE DIKE REVETMENT

Hydraulic climate and characteristic loading

Water levels

- In general the water level at lake dikes is fairly constant, i.e. there is no significant tidal influence or discharge variation.
- Wind setup and storm surge cause elevation of water the level, storm duration is usually not longer than 24-48 hours.
- If the lake water body is in contact with the open sea, water levels vary with the sea tide, depending on the size of the inlet.

Waves

- In open water high wind waves can develop. There is a strong correlation between the occurrence of high waves and high water levels.
- Breaking waves can cause severe loading on the dike slope.
- Waves may carry floating debris (i.e. drift wood).
- Waves cause uprush and downrush on slope.
- Overtopping is primarily due to uprush of the most extreme waves, overtopping discharges vary.

Currents

- Wave induced orbital currents (oscillatory)
- In winter, ice can form along lake dikes (fresh water)

Horizontal location of the profile

- Shallow areas before the dike can induce breaking of depth-limited waves, resulting in a reduction of wave loads at the dike.
- The shape of coastline can cause further variation in hydraulic conditions by sheltering or funnelling.

Vertical location within cross profile

Within the cross profile of a lake dike, the following zones can be distinguished:

- Zone I – The zone beneath the mean water level, loaded by currents. Around the still water level this zone is also frequently loaded by waves;
- Zone II – Between mean water level and design water level. During storm conditions this zone is loaded by high waves and strong currents. After occurrence of high water levels static overpressures may develop behind an impermeable revetment.;
- Zone III – Above design water level, the slope is mainly loaded by wave run-up and rundown. Up to approximately $0.5\times H_s$ above design water level wave impacts are still present;
- Zone IV – The crest of the dike is often used for traffic. During extreme water levels and high waves this zone can be loaded by overtopping waves;
- Zone V – During extreme conditions the inner slope may be loaded by overtopping waves.

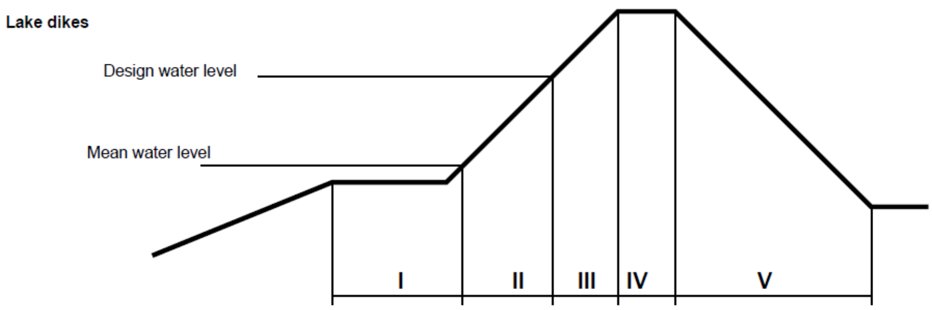


Figure 3.20

Typical cross profile and loading zones on a lake dike.

The table below shows per zone the load cases that should be considered in design of a revetment and the section of this manual in which the load case is elaborated. The table is indicative; for each specific design situation the designer should decide which load cases are relevant.

Table 3.4

Load cases per zone.

LAKE DIKE REVETMENT		I	II	III	IV	IV
scour / undermining	§4.2.7	+	-	-	-	-
ship induced currents	§4.3.2	0	-	-	-	-
ship induced waves	§4.3.3	+	-	-	-	-
natural currents	§4.3.2	0	0	-	-	-
wind wave impact	§4.3.3	+	+	+	-	-
wind wave run-up / down	§4.2.6	-	+	+	+	-
overpressures	§4.4.4	-	+	-	-	-
overtopping	§4.3.2	-	-	-	+	+
ice loads	§4.3.5	+	+	+	+	-
traffic loads	§4.3.4	-	-	-	+	-

Legend:
+ *relevant* 0 *present but usually not dominant* - *not relevant*

3.2.5 SEA DIKE REVETMENT

Hydraulic climate and characteristic loading

Water levels

- Tidal variation (in case of a sea dike or a lake in contact to sea).
- The water level at lake dikes is fairly constant.
- Wind setup and storm surge cause elevation of water the level, storm duration is usually not longer than 24-48 hours.

Waves

- In open water high wind waves can develop. There is a strong correlation between the occurrence of high waves and high water levels.
- Breaking waves can cause severe loading on the dike slope.
- Waves cause uprush and downrush on slope.
- Overtopping is primarily due to uprush of the most extreme waves, overtopping discharges vary.

Currents

- Weak tidal currents
- Wave induced longshore currents
- Wave induced orbital currents (oscillatory)

Horizontal location of the profile

- The shape and orientation of the coastline may cause a spatial variation in the direction of approach of wind waves. Over a long and shallow beach waves approaching under an angle refract and turn towards the coastline.
- Shallow areas before the dike can induce breaking of depth-limited waves, resulting in a reduction of wave loads at the dike.
- The shape of coastline can cause further variation in hydraulic conditions by sheltering, funnelling or contraction of flow lines.

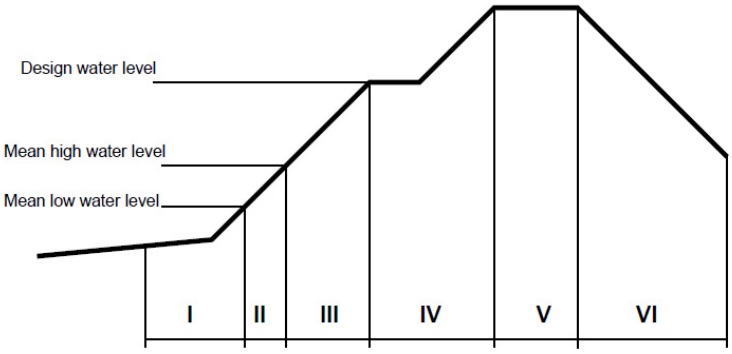
Vertical location within cross profile

Within the cross profile of a lake dike, the following zones can be distinguished:

- Zone I – Constantly submerged. This zone is mainly loaded by currents. Erosion of the foreshore can potentially undermine the revetment structure. Wave action and currents can cause pressure differences over the revetment;
- Zone II – The zone between mean water level and mean high water level is frequently loaded by waves and currents. After occurrence of high water levels, overpressures may develop behind an impermeable revetment;
- Zone III – During extreme conditions the zone between mean high water level and design water level is loaded by high waves and strong uprush and downrush currents. After occurrence of high water levels, overpressures may develop behind an impermeable revetment;
- Zone IV – Above design water level, the revetment is loaded mainly by uprush and downrush of waves. Up to approximately $0.5 \times H_s$ above design water level wave impacts are still present;
- Zone V - The crest of the dike is often used for traffic. During extreme water levels and high waves this zone can be loaded by overtopping waves;
- Zone VI – During extreme conditions the inner slope may be loaded by overtopping waves.

Figure 3.21

Typical cross profile and loading zones of a sea dike.



The table below shows per zone the load cases that should be considered in design of a revetment and the section of this manual in which the load case is elaborated. The table is indicative; for each specific design situation the designer should decide which load cases are relevant.

Table 3.5

Load cases per zone.

SEA DIKE REVETMENT		I	II	III	IV	V	VI
scour / undermining	§4.2.7	+	-	-	-	-	-
ship induced currents	§4.3.2	0	0	-	-	-	-
ship induced waves	§4.3.3	+	+	-	-	-	-
natural currents	§4.3.2	0	0	0	-	-	-
wind wave impact	§4.3.3	-	+	+	+	-	-
wind wave run-up / down	§4.2.6	-	-	+	+	+	-
overpressures	§4.4.4	+	+	+	-	-	-
overtopping	§4.3.2	-	-	-	-	+	+
ice loads	§4.3.5	+	+	+	+	+	-
traffic loads	§4.3.4	-	-	-	+	+	-

Legend:

+ *relevant* 0 *present but usually not dominant* - *not relevant*

3.2.6 TOE PROTECTION OF DUNES

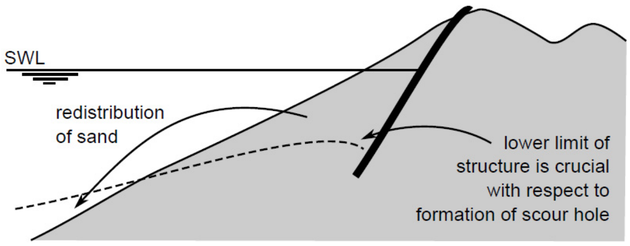
Special considerations for design of dune toe protections

Choice of foundation depth

In front of a 'hard structure' in a sandy beach, which is directly attacked by waves, wave reflection will increase the erosion just in front of the structure. Sediment will be transported in offshore direction until a new equilibrium is reached with a deeper bottom just in front of the seawall (scour hole). This is illustrated in Figure 3.22. For the design of such a structure it is crucial to know beforehand the maximum depth of the possible scour hole during design conditions (to avoid undermining) [lit. 25].

Figure 3.22

Erosion in front of a 'hard' dune foot protection.



Structural erosion

Revetments are so-called 'hard' structures. They might be applied in this special case, provided that the storm (surge) protection problem is the only issue for this stretch of coast. So it refers in fact to a stable part of the coast (stable: seen over a number of years) or an accreting part of the coast. If (also) structural erosion occurs along the stretch of coast under consideration, revetments can in no way be selected as the only protection measure [lit. 25].

Hydraulic climate and characteristic loading

For dunes the same hydraulic conditions apply as for sea dikes. However, dunes are large sand bodies with a dynamic profile in contrast to the fixed profile of dikes. This introduces special load cases:

- sand transport by waves and cross-shore currents may result in scour at the bottom end of the revetment;
- sand transport by wind may result in scour behind the revetment.

Horizontal location of the profile

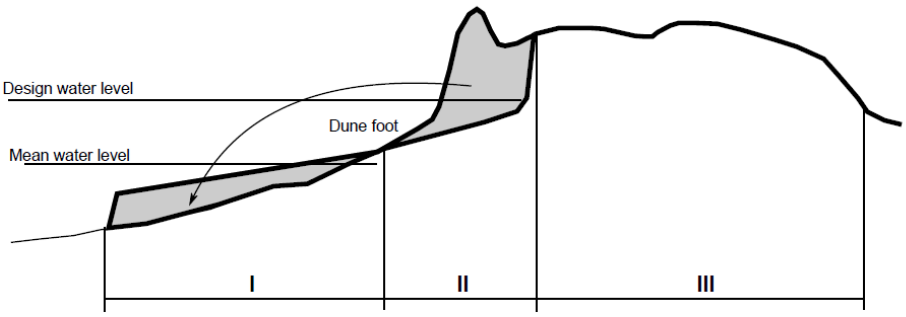
- Dune ridges are parallel to the coastline. The orientation of the coastline is related to the dominant wave direction.
- Transport of sand parallel to the coastline is called longshore transport. Partial obstruction or interruption of longshore transport by presence of a hard structure causes accretion on one side and erosion on the other side of the structure. Often this accretion/erosion pattern is structural.
- The extent of the revetment is to be based upon morphological considerations. See the special considerations at the end of this section.

Vertical location within cross profile

The cross-shore profile of dunes adapts to the yearly variance in hydraulic loads. This results in different profiles for the summer and winter season. During extreme conditions sand is transported from the upper part of the profile into the sea. This sand is deposited on the foreshore resulting in a more gentle slope. A mild sloping and shallow beach reduces wave loads. During mild conditions sand is transported back again from the foreshore to the upper part of the profile by combined wave and wind action.

Figure 3.23

Dynamic cross-shore profile of dunes.



The table below shows per zone the load cases that should be considered in design of a revetment and the section of this manual in which the load case is elaborated. The table is indicative; for each specific design situation the designer should decide which load cases are relevant.

Table 3.6

Load cases per zone.

DUNE FOOT PROTECTION		I	II	III
scour / undermining	§4.2.7	+	+	+
ship induced currents	§4.3.2	-	-	-
ship induced waves	§4.3.3	-	-	-
natural currents	§4.3.2	+	+	-
wind wave impact	§4.3.3	+	+	-
wind wave run-up / down	§4.2.6	+	+	-
overpressures	§4.4.4	-	-	-
overtopping	§4.3.2	-	-	-
ice loads	§4.3.5	-	-	-
traffic loads	§4.3.4	-	-	-

Legend:
+ *relevant*   0 *present but usually not dominant*   - *not relevant*

3.2.7 BREAKWATERS AND GROYNES

Hydraulic climate and characteristic loading

- Wave and current attack from all sides
- Often submerged or overtopped during extreme conditions
- Entrapped air in combination with varying water levels within the breakwater can lead to overpressures over the top part of the structure.

Horizontal location of the profile

Often structures such as breakwaters and groynes are part of a bigger protection structure. Hydraulic conditions can strongly vary depending on the horizontal location within this bigger structure. This is illustrated by Figure 3.24, showing the complex wave interactions at a harbour entrance structure. Important aspects that should be considered include:

- flow contraction and wave reflection around the head of a structure can cause deep scour holes in front of the structure;
- the lee side of the structure can also be subjected to significant loads due to:
  - wave energy being transmitted over and through the breakwater structure;
  - wave diffraction around the ends of the structure;

- wave reflections and possible amplification inside a confined water body.

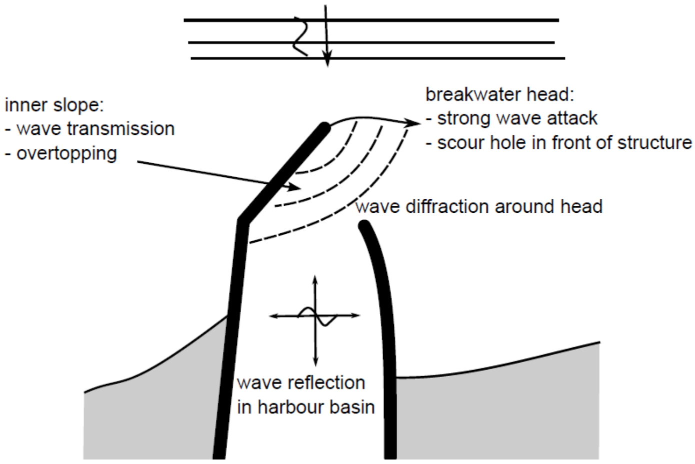


Figure 3.24
Wave interactions with a harbour entrance.

Vertical location within cross profile
Within the cross profile of a breakwater, the following zones can be distinguished:

- Zone I – Constantly submerged. This zone is mainly loaded by currents. Bed erosion in front of the structure can potentially undermine the breakwater. Wave action can cause pressure differences over the revetment;
- Zone II – The region between mean low water level and the crest of the breakwater is constantly loaded by currents and waves. Entrapped air within the breakwater body and wave action can cause overpressures in this region;
- Zone III – During extreme conditions the crest is overtopped by waves, or even submerged. Entrapped air within the breakwater body and wave action can cause overpressures in this region;
- Zone IV – The region between mean low water level and the crest of the breakwater is constantly loaded by currents and waves. Entrapped air within the breakwater body and wave action can cause overpressures in this region;
- Zone V – Constantly submerged. This zone is mainly loaded by currents. Bed erosion behind the structure can potentially undermine the breakwater. Wave action and currents can cause pressure differences over the revetment.

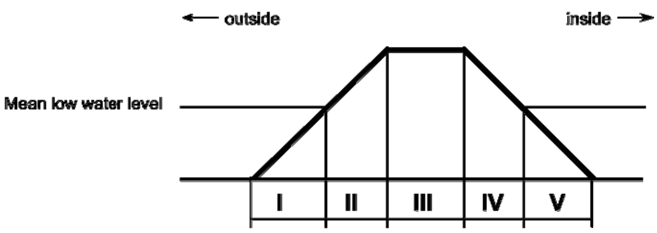


Figure 3.25
Typical cross profile and loading zones of an offshore breakwater.

The table on the next page shows per zone the load cases that should be considered in design of a revetment and the section of this manual in which the load case is elaborated. The table is indicative; for each specific design situation the designer should decide which load cases are relevant.

Table 3.7

Load cases per zone.

BREAKWATERS AND GROYNES		I	II	III	IV	V
scour / undermining	§4.2.7	■				■
ship induced currents	§4.3.2	■				■
ship induced waves	§4.3.3		■	■	■	
natural currents	§4.3.2	■	■	■	■	■
wind wave impact	§4.3.3		■	■	■	
wind wave run-up / down	§4.3.2		■	■	■	
overpressures	§4.3.4		■	■	■	
overtopping	§4.3.2		■	■	■	
ice loads	§4.3.6		■	■	■	
traffic loads	§4.3.5			■		

Legend:
■ *relevant* *present but usually not dominant* *not relevant*

3.2.8 UPGRADING OF EXISTING REVETMENTS

The PBA revetment may be applied as a refurbishment of an existing revetment such as a stone or block pitching in case the pitching itself is not stable under design conditions. Covering of the blocks with a PBA refurbishment provides extra weight to the structure and adds extra coherence. The plate structure of PBA can prevent individual or groups of blocks from being lifted out of the structure by overpressures behind the revetment.

Refurbishment with PBA has the following advantages:
- PBA upgrading improves the stability of the underlying layer. It adds coherence, weight and protection against erosion and is stable on its own;
- because of the stiff foundation and low permeability that an existing revetment often has, the thickness of the upgrading layer is generally not determined by macro scale failure mechanisms.

3.2.9 SPECIAL STRUCTURES

A great variety of hydraulic structures exist, each with case specific hydraulic conditions. When designing a revetment on or near these structures, design loads should be determined from case to case. Examples of special structures are:
- canals (water level and bed shape controlled, ship induced loads);
- sluices (extreme current velocities, ship induced loads);
- spillways (extreme current velocities);
- pipelines (deformation, undermining, anchor impacts);
- etc.

3.3 FUNCTIONAL AND TECHNICAL REQUIREMENTS

An important step in the design of a revetment is to determine the functional and technical requirements that the revetment must fulfil. Some functional requirements determine whether or in what form a PBA revetment is a suitable choice in the given situation and with the given functions. Other requirements are technical and result in a minimum layer thickness which is needed to fulfil the function without the revetment being damaged. The most important requirements for (flood defence) revetments are listed in the table below.

Table 3.8

Functions and requirements of a revetment structure.

	Function*	Requirement
Primary	▪ Protection against erosion	▪ Resistance to hydraulic loads ▪ Adaptation to soil deformations and undermining of the structure ▪ Resistance to ice loading / floating objects ▪ Sand tightness and permeability
Secondary	▪ Traffic	▪ Accessibility and bearing capacity
	▪ Landscape / ecology	▪ Appearance / aesthetics ▪ Suitability for ecological development
	▪ Recreation	▪ Accessibility ▪ Resistance to vandalism

**Durability is also a relevant requirement. Each function is to be fulfilled during the full service life of the structure.*

The primary function of a revetment is the protection of the underlying soil body against erosion. This means that the revetment itself must be capable to endure the erosive forces. Below, the ability of PBA to fulfil the requirements for this primary function is discussed briefly. Also, the accessibility and bearing capacity related to the traffic function is addressed.

Resistance of PBA to hydraulic loads

To provide protection against erosion of the dike body, the PBA revetment must be able to withstand the hydraulic loads that were identified in the system analysis of section 3.2. Both strength and stability are important. The system characteristics and the horizontal and vertical location within the profile determine which mechanisms should be considered (see section 3.2).

Strength of the revetment

- *Erosion of the cover layer.* Under the influence of (wave induced) currents individual rocks can erode from the cover layer (micro scale damage). The revetment must be strong enough to withstand these erosive forces. From prototype and abrasion tests, the PBA revetment proved to be very resistant to erosive forces. In section 4.3.2 the design for currents is treated.
- *Breakage under wave impacts.* Wave impact pressures can cause high peak pressures on the revetment. The severity of these pressures depends on the wave height, breaker shape and the dike slope angle. Under wave impact pressures the PBA revetment acts like a plate structure, loaded perpendicular to its surface. The exceedence of material strength leads to breakage of the material on a macro scale. Determination of the minimum required cover layer thickness under wave impacts is discussed in section 4.3.3.

Incidentally, a revetment can suffer mechanical loads by collision with ships, floating debris or floating ice. Generally mechanical loads by collisions are not taken into account in the design of revetments, mainly because of their low probability of occurrence.

- *Collision with ships or other floating objects.* These mechanical loads are not taken into account in the design of the PBA revetment.

- *Ice loads.* Generally, floating ice does not cause significant damage to revetments with a plane surface. Besides, the probability of the combination of ice formation and storm conditions is negligible in the Dutch climate. Therefore in the Netherlands ice loads are not taken into account in determination of the minimum required layer thickness. Piling ice scraping over the surface of the PBA revetment will cause some superficial damage on a micro scale. Damage by piling ice can be partly avoided by a proper design of the slope. In section 4.3.5 design considerations for ice loading will be further discussed.

Stability of the revetment

- *Subsoil liquefaction*. Due to the open structure of the PBA revetment pressure fluctuations by water level variation and wave impact are easily transferred to the subsoil. When designing and dimensioning the PBA revetment, it has to be ensured that the occurring pore pressures do not result in liquefaction and subsequent damage. This can be achieved by dimensioning the filter layer in consideration of the subsoil. See section 4.4.2.
- *Sliding of (parts of) the revetment by occurrence of a slide plane at the interface between the cover layer and the filter layer*. The mechanism of sliding can occur when the driving force, the gravity force component parallel to the slope, is larger than the friction force between the interface with the underlying soil. This macro scale mechanism applies mainly to revetments on steep slopes. This mechanism is treated in section 4.4.3.
- *Uplift of (parts of) the revetment by static overpressures.* Static overpressures behind the revetment are caused by a difference between water level in front of the revetment and behind the revetment, inside the dike body. The time scale for the development of static overpressures is in the order of several hours. If the upward force by overpressures is larger than the downward force by structure weight (part of) the cover layer can become instable (macro scale). This mechanism is only relevant for revetments with a completely impermeable cover layer. Permeable revetments such as open stone asphalt and pattern grouted rock allow pressure leakage and are not vulnerable to uplift by static overpressures. For the same reasons this mechanism does not have to be taken into account in the design of PBA revetments.
- *Uplift of (parts of) the revetment by dynamic overpressures.* The periodic variation in water level by wave action can also cause overpressures behind the revetment. The time scale of this mechanism is in the order of one wave period. Under normal conditions the permeability of a PBA revetment is such that these pressures are quickly relieved through the open pore space of the structure. However, it is possible that this open space is filled with fine sediments (by natural processes). This clogging of the structure strongly affects the permeability, such that dynamic overpressures can cause uplift of the cover layer (macro scale). See section 4.4.4.

Cope with uneven settlement

The PBA revetment does not show viscous behaviour. Therefore, the cover layer is not able to adapt to large soil deformations without a corresponding increase in internal stresses. Therefore special attention should be paid to situations where substantial differential settlements are expected to occur in the dike body or if scour of the dike toe can lead to undermining of the revetment. The following aspects should be considered:

- *Differential settlement of the dike body.* To some extent, differential settlements can be accepted. The cover layer thickness then should be dimensioned such that the corresponding deformation does not lead to breakage of the structure. See section 4.3.6.
- *Undermining of the revetment by progressive scour from the toe structure.* The toe structure provides support at the lower end of the revetment. If this support is undermined by progressive scour at the toe under the influence of hydraulic loads, the revetment can

become instable and (partly) slide down the slope. A properly designed toe structure should prevent scour from undermining the revetment. The toe structure for a PBA revetment is not principally different from that for asphalt or pitched stone revetments. Some examples and design considerations are discussed in section 4.5.

Functional requirements

Because of the combination of high porosity and aggregate size of the PBA revetment it has a high permeability for both soil and water.

- *Sand tightness.* The PBA itself is not impermeable to sand. In order to prevent washing out of the subsoil an adequate filter layer must be applied on the interface between the PBA and the dike body. In section 4.2.4 the design of the filter layer is treated.
- *Water permeability.* The permeability of the PBA revetment for water makes it less likely that overpressures are developed behind the cover layer. When the open pores of the cover layer are clogged with fine material, the hydraulic conductivity is affected negatively. In section 2.3.3 the hydraulic conductivity of PBA and influence of clogging are quantified. If clogging is expected, a conservative design approach is to assume complete impermeability of the cover layer. See section 4.4.4.

Accessibility and bearing capacity

Generally, a revetment must be accessible for maintenance traffic and sometimes also for recreational purposes. The surface of PBA revetments provides sufficient grip to allow for such traffic. In general, if PBA is applied as a road material the design of the top layer is adapted by using a smaller aggregate. This greatly improves accessibility.

- *Incidental (maintenance) traffic*. If properly designed, the PBA revetment has enough bearing capacity to withstand the axle loads of incidental traffic, such as inspection or maintenance vehicles. In order to prevent damage on a micro scale, for heavy traffic with high axle loads, precautions should be made to distribute the wheel loads (i.e. metal plates). In section 4.3.4 determination of the minimum required layer thickness for traffic loads is discussed (macro scale). In some cases it may be necessary to construct special ‘traffic lanes’ in the revetment, with an adjusted layer thickness or aggregate grading. These lanes should then be well marked out with signs or colours.
- *Regular traffic.* The surface of a PBA revetment is generally to rough to allow for regular traffic. Regular cars can access the revetment only by driving slowly. Accessibility can be improved by applying PBA with a smaller aggregate as top layer.
- *Recreational traffic.* Pedestrians and hikers can walk over the PBA revetment without any problem, the rough surface provides sufficient grip to prevent slipping. For swimmers and surfers PBA with a fine aggregate (16/22 mm) is more comfortable than a rough aggregate (20/40 mm). The surface of the PBA revetment is too rough for cyclists.
- *Animal traffic.* The PBA revetment is quite suitable for ‘ecological traffic’, for instance in the application of a fauna exit location at a river bank. Animals often prefer a rough surface.

CHAPTER 4 Design of the PBA revetment

4.1 INTRODUCTION

After the application area has been characterized and the functional and technical requirements have been specified, a conceptual design is made. In the conceptual design material choices are made, and the structure build-up from the subsoil to the cover layer and the boundaries of the revetment structure are determined.

Structural elements are treated in section 4.2. Then the structure is sized by calculation of the minimum required layer thicknesses of cover and filter layer, taking into account all relevant failure modes. The failure modes concern both the strength of the revetment under bending deformation (section 4.3) and overall stability of the revetment and subsoil (section 4.4). For refurbishment of existing revetments additional mechanisms should be taken into account, which are related to the structural behaviour of the PBA-pitching system (section 4.5). Finally, structure details such as transitions and the toe support are designed. Examples of structure details are given in section 4.6.

The relevant design parameters of a revetment are given in Figure 4.27. Choice of these parameters is related to the soil, the geotechnical and hydraulic boundary conditions as well as other specifications characteristic to geometric and material specific parameters for the revetment.

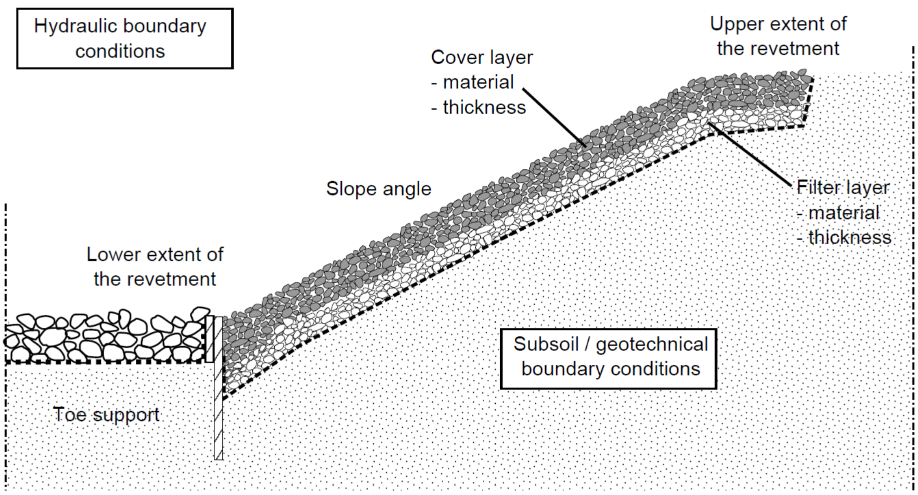


Figure 4.26
Relevant design parameters.

4.2 STRUCTURAL ELEMENTS

4.2.1 COVER LAYER

Material

In principal any type of coarse graded granular material can be used in a PBA revetment. The adhesive bonding of PU has been successfully tested on i.e. basalt stone, granite, limestone and iron slag. The granular material can be broken or rounded (gravel) or a mix of both. Rocks with a nominal diameter typically up to 60 mm are used for PBA revetments.

The choice for an aggregate as base material in a PBA revetment is based on:

- stone type – basalt, granite, limestone, iron slag, etc.;
- size – D_{n50}, D_{max};
- shape – broken, rounded or a mix of both;
- grading – coarse grading 20/40 mm, 30/60 mm, etc.

These aspects determine a great deal of the structural properties of the PBA revetment:

- density – self weight of the revetment;
- permeability – hydraulic conductivity;
- hydraulic roughness – wave run-up;
- minimum layer thickness – practical and functional limits;
- strength – flexural and compressive strength;
- stiffness – bending stiffness, deformation;
- visual appearance – colour of the structure.

The aggregate density and the open pore volume determine the density of the PBA revetment, which is approximately equal to the *bulk* density of the aggregate. The bulk density is a function of the density and the pore volume of the aggregate. Table 4.9 shows some common densities for different types of rock. Depending on the grading of the aggregate, the open pore volume of the composite can be up to 50% (see section 2.3.2). The high porosity results in a relatively light structure.

Table 4.9

Mineral aggregate densities. From: CUR (1999) [lit. 22].

Mineral	Density
Granite	2600-2800 kg/m^3
Basalt	2900-3000 kg/m^3
Limestone	2650-2700 kg/m^3
Iron slag	3100-3400 kg/m^3

The size and grading of the aggregate also influences some of the mechanical properties of the PBA revetment, such as stiffness and flexural strength. More information about the relation between mechanical properties and the choice of aggregate is given in chapter 2 of this manual.

Layer thickness

PBA revetments are generally applied with a cover layer thickness ranging from 0.10 to 0.50 m. From economic point of view, a design goal is to minimize the layer thickness. The minimum layer thickness is determined by the largest of:

- minimum functional layer thickness;
- minimum practically achievable layer thickness;
- minimum required cross section to withstand (hydraulic) loads;

The minimum functional and practical thicknesses are determined by stone size and the construction equipment. The minimum required cross section is determined by the loading of the structure.

Functional limitations

From a functional point of view a minimum layer thickness of approximately 2 × D_{n50} is required to ensure complete coverage of the application area. This is a functional lower limit, applicable for aggregate with a narrow grading. It is commonly applied for dumped rock revetments, and also valid for PBA revetments.

Practical limitations

Due to limitations of construction equipment it is not always practically feasible to place very thin layers of aggregate accurately. These practical limits are determined by stone size, the shape the underground and accuracy of construction equipment. From a practical point of view a minimum layer thickness of approximately 2.5 × D_{max} or 3.5×D_{50} is advised for smaller aggregate sizes. This is based on field-experience with the application of 20/40 mm aggregate and use of a hydraulic crane for profiling. With this combination of aggregate and equipment layers less than 0.10 m thick are hard to achieve.

Figure 4.27

Profiling of the PBA revetment with a hydraulic crane.

Minimum required layer thickness

The minimum required layer thickness is determined by the mechanical properties of the material and by the loading of the revetment, which are discussed in sections 2.4, 3.2 and 3.3 of this manual. In sections 4.3-4.5, design rules for the layer thickness are given. Structure ends and transitions to other structures can introduce extra loads, resulting in a locally larger required layer thickness. For examples see section 4.6.

Coping with uneven settlement

In those cases where strong settlement behaviour of the dike body or subsoil is expected after construction of the revetment, the effect of imposed deformations by uneven settlement must be taken into account in design of the cover layer. The plate revetment is relatively stiff and is not able to adapt to large differential soil deformations.

A principal difference exists between the effects of uneven deformations caused by:
1 spatial variations in the settlement behaviour of the subsoil;
2 inadequate transitions to stiff structures;
3 irregularities in preparation of the subsoil during construction.

Ad 1. Differential soil deformations can be coped with in design by:
- taking into account the effect of extra internal stresses in the cover layer due to imposed deformations. In section 4.3.6 a method is given on how to determine these internal stresses;
- choosing mixture of PU and aggregate with lower stiffness;
- avoid the use of highly compressible soil in new structures.

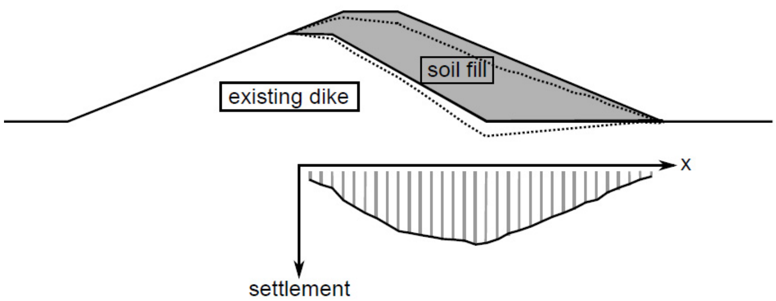
Spatial variations in the subsoil cannot be avoided, since these settlements may occur in all compressible layers, varying from the surface to deep in the subsoil. The weight of the soil structure on the subsoil causes corresponding settlements in the structure itself. In line shaped soil structures, such as levees, largest spatial variations are present in the transverse direction of the structure and underlying subsoil. Therefore most significant differential settlements can be expected in this direction. See also Figure 4.28.

Ad 2. Transitions in stiffness to other structures must be taken into account in design of structure details. Imposed deformations can be introduced by transitions to stiff structures (i.e. walls, staircases, sluices, buildings etcetera). Generally, these structures will not have the same settlement behaviour as the surrounding subsoil. Design of transitions is treated in section 4.6.

Ad 3. Spatial variations also can be introduced during construction, in example by uneven compaction of applied soil. Therefore quality control during construction is very important. The risk of uneven deformations can be minimized by ensuring a sufficient and evenly distributed compaction of the dike slope. During construction extra attention must be paid to avoiding irregularities in the subsoil. Preparation of the slope is treated in section 5.2 of this manual. Quality control is treated in chapter 6.

Figure 4.28

Illustration of differential settlement in cross section of a dike body.



4.2.2 SLOPE ANGLE

The dike slope influences the spatial footprint and the use of resources. Also the dike slope has an influence on the hydraulic loads. The maximum slope angle is determined by:
- geotechnical stability of the dike body;
- internal stability of polyurethane–aggregate composite.

For geotechnical stability of the dike body the same considerations apply as are relevant to all water retaining soil structures. For information about the geotechnical stability of these structures reference is made to literature about water retaining soil structures [lit. 17].

For PBA the internal stability is only of importance during construction. In this phase, the unhardened composite will remain stable on slopes equal to or milder than 1:3. Otherwise, extra measures might be necessary to prevent the unhardened composite from sliding down the slope. In practice a toe support structure is always applied at the lower end of revetments. This makes the application of PBA on slopes up to 1:1.5 possible.

4.2.3 FILTER LAYER

The question whether a PBA revetment will be made with or without a filter layer has to be decided in the course of the individual planning and design stage and is also subject to the respective geometrical, geotechnical and hydraulic boundary conditions.

Like any other revetment that is not completely impermeable, application of a filter layer is necessary to prevent transport of material from the subsoil through the open structure of the PBA revetment. Preferably, the permeability of the revetment to water should not be negatively influenced by the filter layer.

Material

The following figure**Fout! Verwijzingsbron niet gevonden.** shows several design alternatives for the filter layer beneath the PBA revetment:
- Alternative I – granular filter (geometrically open/closed);
- Alternative II – combination of an open granular filter/levelling layer and geotextile;
- Alternative III – geotextile.

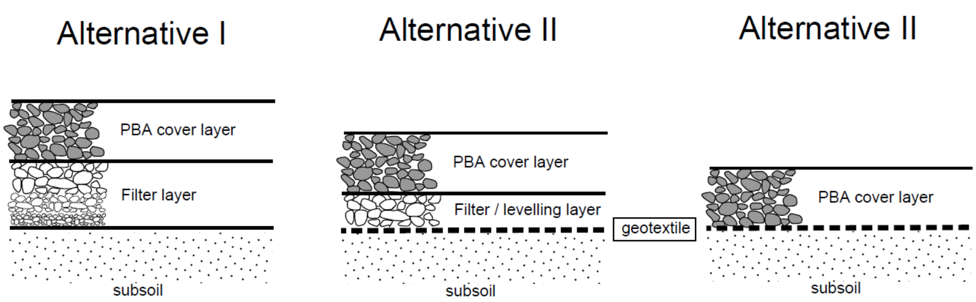


Figure 4.29
Possible design alternatives for the filter layer underneath PBA revetments.

If the filter/levelling layer is insufficient to prevent material loss from the subsoil, a geotextile filter must always be applied at the subsoil interface. The geotextile serves as a filter as well as a separating layer. In some cases it may be essential to prevent bonding between the PBA and the underlying structure (i.e. stone pitching). See also section 4.3.4.

Thickness

The dimensions of the filter layer are determined by several aspects, including:

- filter function;
- dissipation of wave energy;
- prevent excess PU from dripping on underlying geotextile;
- practical limitations.

Filter function

For granular filters a minimum thickness is required to adequately fulfil the filter function. The combination of the PBA revetment and the underlying filter can be treated as a multi layered granular filter. The standard filter rules apply also to the PBA cover layer. The filter may be geometrically open or closed. The required filter material is determined by the aggregate size and grading of the PBA as well as the characteristics and grain size of the subsoil.

Geotextile forms an adequate filter which is easy to apply on the dike surface. The thickness of geotextile ranges from 0.4 mm to 5 mm and therefore does not contribute to the total thickness of the revetment. This enables a slender design of the structure.

For information on the working of (granular) filters and their design, reference is made to literature on the design and application of filters in hydraulic engineering [lit. 18].

Stability of the revetment

Due to the open structure of PBA wave induced pressure fluctuations will penetrate into the subsoil relatively easy. In some cases an increase in pore pressures in the subsoil may pose stability problems such as subsoil liquefaction. Increasing the total thickness of the cover layer and filter layer can be effective in reducing the risk of liquefaction. See section 4.4.2 on design for failure by subsoil liquefaction.

Leakage of excess PU

Dripping of excess unhardened PU from the cover layer onto the underlying layer may be unwanted from a functional point of view. For instance, the permeability of a geotextile can be reduced locally when clogged with PU (Figure 4.30).

When the geotextile is applied onto a hard surface (i.e. refurbishment of stone pitching) the effect of the PU on the permeability of the geotextile is negligible. On other surfaces the effect on permeability may not be neglected. Leakage of excess PU onto the geotextile can be prevented by applying a (thin) layer of loose aggregate on the interface between the PBA and the geotextile (Figure 4.30). This layer should be at least $1.5{\times}D_{n50}$.

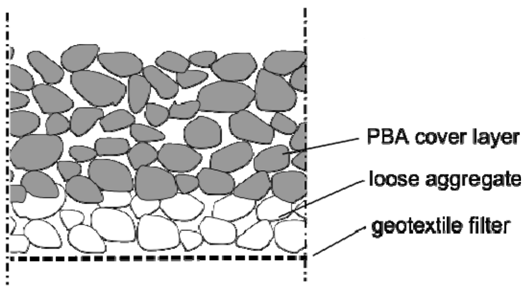


Figure 4.30

Bottom of a cube sample with geotextile filter. Surplus polyurethane dripping down onto the geotextile (Gu, 2007a). Application of a layer of loose aggregate directly underneath the PBA.

4.2.4 SUBSOIL

PBA is suitable to be applied on a variety of undergrounds, such as clay, sand, minestrone or existing revetments. The subsoil beneath the PBA revetment should provide a stable and continuous support for the PBA plate structure.

Subsoil stiffness

The stiffness of the subsoil determines the support reaction of the PBA-revetment under (hydraulic) loads. Subsoil with a high stiffness is favourable. The stiffer the subsoil is, the larger the part of the (hydraulic) load that is directly transferred to the subsoil, relieving the PBA cover layer.

Table 4.10 and Table 4.11 show values for the dynamic stiffness of several subsoil types, described by the soil compression parameter c [MPa/m]. For sand that is well compacted (proctor density at least 95%) a soil compression constant of c = 100 MPa/m can be used. For clay soils c = 30 MPa/m is advised. The soil compression constant c is required as input in the design rules for wave impact loads in section 4.3.3 of this manual.

Table 4.10

Soil compression parameter for several foundation types. From: Verruijt (1999) [lit. 27].

Subsoil	Soil compression constant c (Mpa/m)
Sand - medium compacted (proctor density 87-95%) - well compacted (proctor density 95-100%)	 10 – 100 100 – 300
Sand + clay	30 – 80
Sand + silt	20 – 50
Clay - soft clay - stiff clay	 30 – 60 < 40
Peat	< 50
Gravel	> 70
Sand asphalt	> 500

Table 4.11

Guide values for soil compression constant related to cone resistance. From: CROW (1999) [lit. 23].

Subsoil type	Cone resistance q_c (MN/m^2)	Soil compression constant c (MPa/m)
Peat	0.1-0.3	10-20
Clay	0.2-2.5	20-40
Loam	1.0-3.0	30-60
Sand	3.0-25.0	40-100
Gravel-sand	10.0-30.0	80-130

PBA is a relatively stiff material. For stress distribution over the cover layer and its subsoil it is preferable that the transition in stiffness is minimized, especially when constructing on relatively soft soil such as clay. The foundation stiffness can be improved by application of a fill layer between the subsoil and the PBA cover layer. Common fill materials used in hydraulic engineering include loose aggregate, gravel, iron- or phosphor slag, minestrone, sand asphalt, etc. The effectiveness of the fill material on improving the foundation stiffness depends on the materials used and their degree of compaction.

4.2.5 UPPER AND LOWER EXTENT OF THE REVETMENT

Wave run-up and wave run-down

The movement of the water level over a dike slope is generally greater than the amplitude of the incoming wave height. The maximum and minimum levels that are reached by the wave front are known as wave run-up and run-down, defined as the vertical distance to the design (still) water level. The run-up level can be used in design to determine the upper extent of the revetment and/or overtopping discharges. The run-down level can be used to determine the lower extent of the revetment.

Upper extent of the revetment

The level up to which a slope revetment should extend depends mainly on the hydraulic loads by wave run-up. The uprush and subsequently downrush of water over the slope introduces flow velocities over the revetment. The hydraulic loads caused by wave run-up reduce with increasing height on the slope. At the upper end of a hard revetment the transition is made to a different cover layer, often consisting of grass on clay or another light protection. In this case the revetment must be extended to such a level that both the transition to and the light protection material itself are stable under the hydraulic loads at that level. A method for determination of the 2% run-up level is given in section 2.5.2 of this manual.

Lower extent of the revetment

At the bottom end of a slope revetment a transition is made with either a different type of revetment on the lower slope or with the toe structure. The lower extent of the PBA revetment is mainly determined by the water line. PBA has not yet been tested for application under the water line, and should always be constructed *above* the water line. Thus, the lower extent to which the PBA revetment can be applied depends on the low water level, tidal regime and accessibility for construction equipment. Consequently, the toe structure should be extended up to the lower extent of the PBA revetment, or a transitional revetment structure should be applied in between. A method for determination of the 2% run-down level is given in section 2.5.3 of this manual.

Upper and lower extent of a dune protection

In the design of a dune foot protection, special attention should be paid to the upper and lower extents of the revetment. Due to the dynamic nature of the cross profile shape, the ends of the revetment must be chosen such that undermining of the structure by erosion is prevented. This implies that:

- at the upper end of the revetment a transition is made to a material that is resistant to erosion. This can be a hard structure or vegetation;
- the lower end of the revetment must extend deeper than the largest scour hole that can be expected in front of the revetment. The design engineer should keep in mind that the depth of the scour hole is increased by the presence of the revetment.

For more information about the morphodynamics of sand dunes reference is made to [lit. 20] and [lit. 25].

4.2.6 TOE SUPPORT

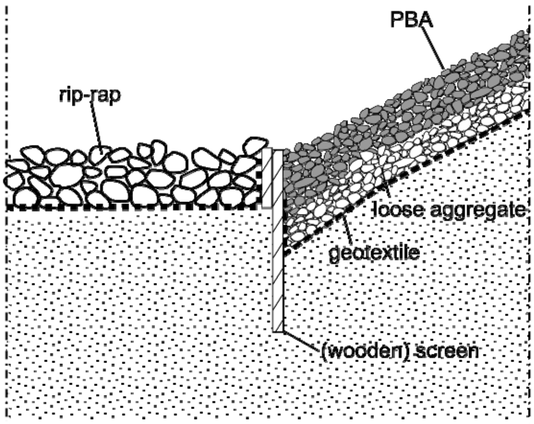
The toe support forms the transition to the foreland at the lower end of the revetment and makes sure the structural integrity of the slope is not jeopardised under extreme conditions. The toe support gives support to the revetment and prevents loss of material from the construction. The toe support generally consists of two structural elements:

1. (rock) apron – provides support to the toe structure and protects against undermining by erosion of the foreland;
2. toe structure – rigid vertical elements and a horizontal line element, against which the revetment is placed.

Ad 1. Rock aprons often consist of dumped rock (rip-rap). The grading depends on the local wave loads. Sand tightness is generally achieved with the application of a geotextile filter.

Ad 2. The toe structure must be sufficiently rigid to form a firm foundation for the revetment. It prevents the revetment from sliding down the slope. Common types are poles, prefab concrete elements and short sheet pile walls. The design of a toe structure for the PBA revetment is not principally different from design for asphalt or pitched stone revetments. The toe structure should prevent sand from eroding from under the PBA. This can be achieved by application of an adequate transition from the PBA to the toe structure. See section 4.6.4.

Figure 4.31
Example of a toe support structure.



4.2.7 SURFACE TREATMENT (OPTIONAL)

Granular material, such as sand, can be applied on the surface of PBA for several functional reasons, such as aesthetics or accessibility.

For attachment to the PBA, the granular material should be dry and be applied preferably within the first 20 minutes after application on the slope.

A rough surface may promote attachment of flora and fauna, although observations in both laboratory and in the field indicate that this is only advantageous during the first months after application. After a longer period there is no significant difference in the ecological development on 'treated' and 'untreated' PBA [lit. 10].

4.3 STRENGTH OF THE REVETMENT

4.3.1 GENERAL

Definition of minimum layer thickness

In this chapter design rules are given to determine the minimum required thickness of the cover layer. The design layer thickness is defined as the minimum cross section present in the considered part of the revetment (Figure 4.32).

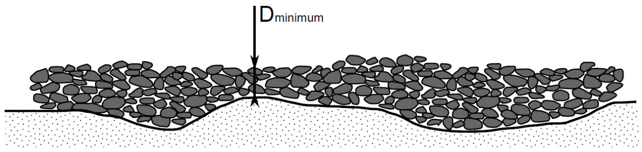


Figure 4.32

Definition of minimum cross section.

In practice irregularities in the application surface and construction inaccuracies result in a spatial variation in actual layer thickness. The thickness adopted for design should account for these construction tolerances, such that the minimum required layer thickness can be guaranteed to be present over the entire revetment. The design layer thickness as given in this manual is always excluding these construction tolerances.

Micro/macro scale level

Generally, for PBA revetment two different scale levels can be discerned on which damage is described, namely:

1. micro scale – on the level of individual rocks;
2. macro scale – on the level of the plate.

Ad 1. On micro scale level, the PBA revetment is treated as a collection of individual rocks, bonded together. Loads typically work on the scale of these individual rocks and their connections to adjacent rocks. The failure mechanism on this scale is erosion, defined by loss of rocks out of the structure. The structural integrity of the PBA revetment is preserved, until damage has progressed to such a level that the effective local layer thickness is significantly changed. In Figure 4.33 the detachment of individual rocks from the cover layer is illustrated. Note that a concentrated damage pattern as shown in the lower panel the figure has *not* been observed in practice and is considered unlikely [lit. 3].

Ad 2. On macro scale level, the PBA revetment is treated as a single plate with corresponding mechanical properties such as stiffness and flexural strength. Loads typically work on a surface area and cause the PBA revetment to respond by deformation or movement of (part of) the revetment as a whole. Macro failure mechanisms include breakage and instability of the cover layer.

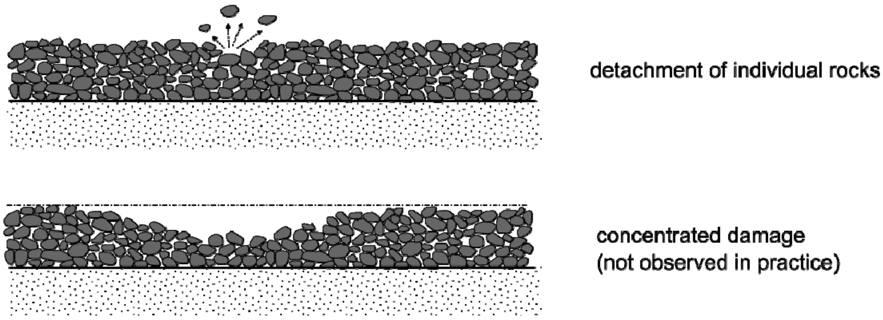


Figure 4.33

Micro scale damage.

4.3.2 DESIGN FOR CURRENTS

General

Currents can lead to erosion of a revetment structure. Flowing water can contain abrasive elements such as loose grains (sand), small rocks or shell fragments. In this section, the resistance of PBA revetments to currents is treated. The most important current loads over a slope protection attacked by waves are the result of wave up- and downrush on the outer slope and wave overtopping volumes on the crest and inner slope. The extent of these loads is partly determined by the roughness of the revetment surface which will also be treated in this section.

Resistance to currents

The PBA revetment is well capable of withstanding high current velocities. Because of the strong bonding between the individual rocks, resistance to erosive forces is very high. The PBA revetment can withstand high current velocities with no form of damage. The presence of fine abrasive material, such as sand and shell fragments, does not cause extra erosion to the PBA revetment. The coating of the most exposed rocks at the surface is weathered, but contact points between the rocks are sheltered and remained undamaged. The strength of the bonding between the rocks is therefore not affected.

Resistance to overtopping discharges

The PBA revetment has been tested up to 125 l/s/m in wave overtopping tests on the inner slope of a dike [lit. 28]. At this overtopping volume the revetment suffered no significant damage. Information about the behaviour of the PBA revetment for overtopping quantities higher than 125 l/s/m is as of yet not available, due to limitations of the testing equipment. For the time being the 125 l/s/m is assumed to be the design limit for PBA revetments.

Figure 4.34

Overtopping tests with a novertopping simulator at Kattendijke.

4.3.3 DESIGN FOR WAVE IMPACT

General

The PBA revetment must be able to withstand the impact pressures of wind-waves and ship-induced waves. Not every part of the revetment suffers from wave impacts and the impacting wave height can vary with the location within the profile. Only those parts of the revetment that are loaded by wave impacts should be dimensioned accordingly (see also section 3.2 of this manual).

Design method and parameters

For design purposes, wave impact is treated as being a static load on a elastically supported plate structure. The wave impact load is schematized as a triangular load (Figure 4.35). The impact load causes a flexural deformation of the plate. The design layer thickness is determined by the occurring bending stresses in the bottom of the plate. These may not exceed the flexural strength of the material (see section 2.4.3). Furthermore, a lower limit is formed by the minimum practically achievable layer thickness, due to limitations of construction equipment (see also section 4.2.1).

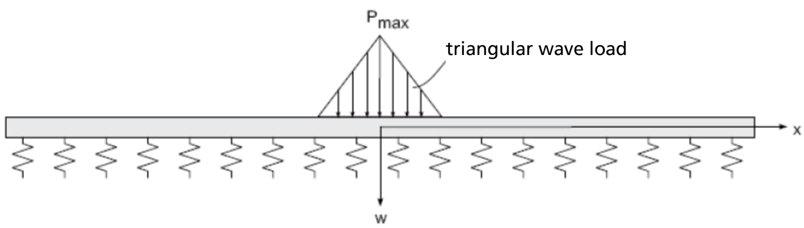


Figure 4.35

Mechanical schematization of a plate type revetment on compressible soil under wave impact (Bijlsma, 2008b).

This schematization is further elaborated in Annex 1 of this manual. Below, a description of the most relevant design parameters is given, as well as several graphs that can be used for preliminary design. Relevant parameters in the design for wave impact are:

- wave load p_{max}:
  - significant wave height $H_{m-1,0}$;
  - wave impact factor q, as function of wave breaker parameter ξ;
- cover layer mechanical properties:
  - elastic modulus, E;
  - flexural strength, σ_{bm};
  - stone size, D_{n50}, D_{50};
- subsoil stiffness, c.

Wave load

A distinction can be made between wave impact loads and non-impact loads (see also section 2.5.4). Both are important in the design of layer thickness since the resulting pressures on the revetment cause deformation of the structure. The wave load can be characterized by the impact factor q. This factor describes the relation between the maximum impact pressure p_{max} and the incident wave height H_{m0}:

$$p_{\max} = q \cdot \rho_w \cdot g \cdot H_{m0}$$

Where

p_{max}	= maximum wave pressure	[MPa]
q	= impact factor	[-]
ρ	= specific density of water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]
H_{m0}	= mean spectral wave height	[m]

The factor *q* is influenced by the wave breaker shape, described by the surf similarity or breaker parameter $\xi_{m-1,0}$, and the wave-structure interaction. In the design graphs the assumption is made for the wave impact factor q = 5.0. For irregular wave fields this is assumed to be conservative (see also section 2.5.4 and Annex 1).

Cover layer mechanical properties
The allowable deformation of the cover layer before breakage occurs depends mainly on the mechanical characteristics described by the elastic modulus *E* and the flexural strength σ_{bm}. A low value for the elastic modulus and high value for the flexural strength are favourable.

The flexural strength of the material is strongly influenced by the stone size of the used aggregate (see section 2.4.3). Apart from this relation, it is assumed that the full bending strength can only be achieved if the plate thickness is not smaller than a predefined number of stone sizes. Full strength is assumed for thicknesses no less than 5 × D_{50} or 6.5 × D_{n50}. For thinner plates a lower strength must be used.

Subsoil
The stiffness of the subsoil determines the support reaction of the PBA-revetment under (hydraulic) loads. Subsoil with a high stiffness is favourable. The stiffer the subsoil is, the larger the part of the (hydraulic) load that is directly transferred to the subsoil, relieving the PBA cover layer.

In case of a very stiff foundation, for instance a sand asphalt bed or an existing block revetment, the required layer thickness is no longer determined by the wave height. Breakage by wave impact is no longer the dominant mechanism. The minimum layer thickness is determined either by practical limitations or other failure mechanisms, see section 4.2.1.

Design graphs
These graphs are valid for the given aggregate sizes and flexural strength. The elastic modulus of the PBA has been assumed to be 3000 MPa, which is a safe value for temperatures above 10°C. A distinction has been made between subsoil's consisting of clay, sand and the case of refurbishment of an existing revetment. For clay a subsoil stiffness of c = 30 MPa/m is assumed. In most cases this will be a safe assumption. For sand a subsoil stiffness of c = 100 MPa/m is used. This value is valid for well compacted sand with a relative proctor density of at least 95%. Further, an irregular wave field is assumed, with a spectral significant wave height H_{m0}.

Figure 4.36

Design layer thickness for PBA limestone 30/60 mm.

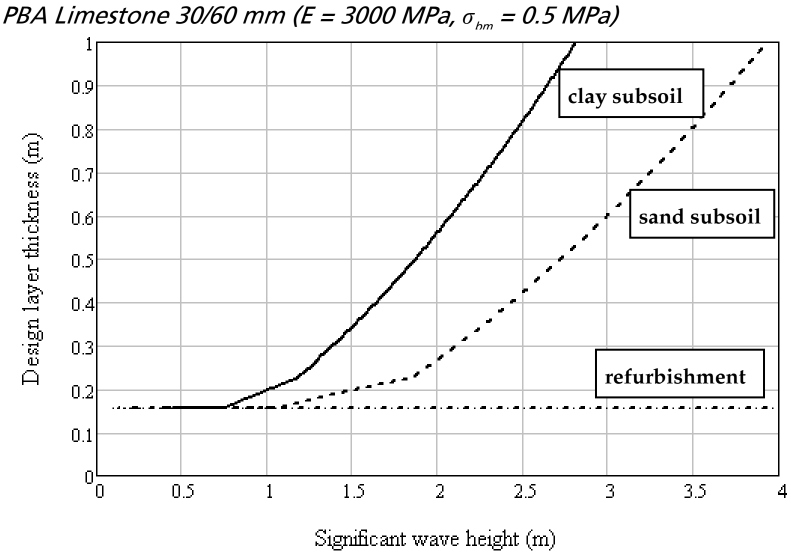


Figure 4.37

Design layer thickness for PBA limestone 20/40 mm.

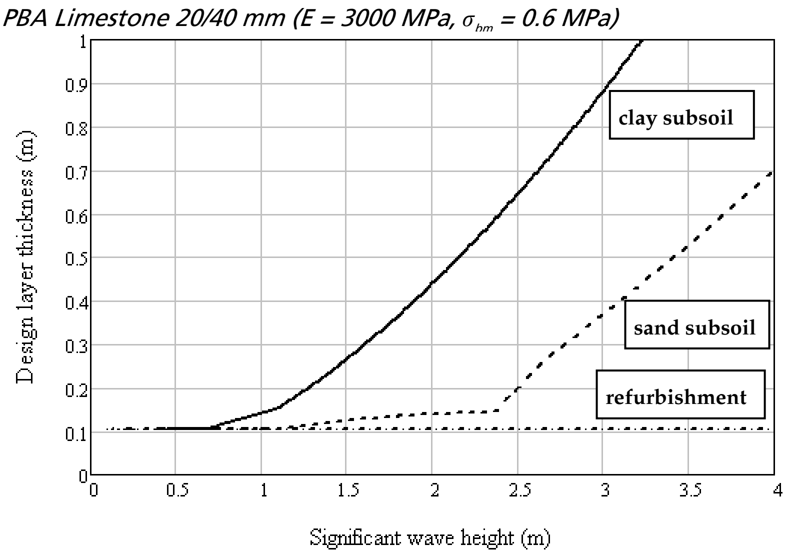
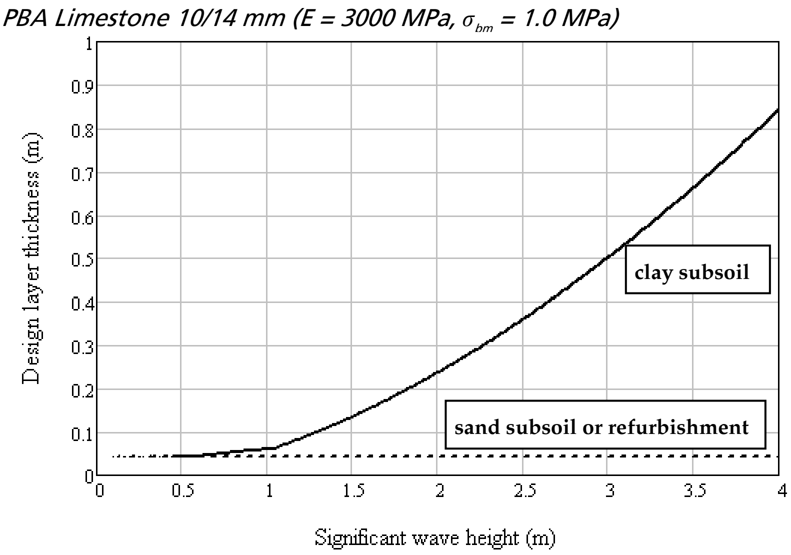


Figure 4.38

Design layer thickness for PBA limestone 10/14 mm.



4.3.4 DESIGN FOR TRAFFIC LOADS

General

The surface of the PBA revetment is only suitable for incidental traffic, such as construction or maintenance traffic. These vehicles can exert high concentrated loads on the plate revetment.

Determination of required layer thickness

For less than 1000 axle loads per year, a preliminary estimate for the minimum required PBA layer thickness is:

- on sand at least 20 cm;
- on clay at least 30 cm.

These values are based on moderate conditions (temperatures not below 10°C) and PBA with crushed limestone 20/40 mm (E = 2,500 MPa, σ_{bm}= 0.78 MPa) and incidental traffic with an axle load of 100 kN (10 ton), approximately 50 kN per tire. If the size of incidental heavy loads are known on beforehand, the minimum required layer thickness can be determined as follows [lit. 21][c]:

$$d = \sqrt[5]{\frac{P^4}{\sigma_{bm}^4} A}$$

With:

$$A = \frac{27E}{16c(1-\nu^2)}$$

In which:

σ_{bm}	= flexural strength	[MPa]
P_{max}	= maximum axle load per tire	[kN/m]
c	= coefficient of compression of subsoil	[MPa/m]
E	= elastic modulus of the cover layer	[MPa]
d	= thickness of the cover layer	[m]
ν	= constant of Poisson (ν =0.35)	[-]

4.3.5 DESIGN FOR ICE LOADS

General

In case of piling or *hummocking* ice, sea or lake ice is forced in such a way that ice sheets are stacked over each other and form a large mass of ice forcing its way up the revetment slope. In general ice loads on a PBA revetment are not a problem. Regular scraping of ice over the revetment may cause some erosion on the revetment surface. Because the surface of the PBA revetment generally does not have edges or any protrusions larger than a single stone size the ice will slide over the revetment without causing high loads, which can be withstood by the structure's behaviour as a single coherent plate.

Design considerations

In design of a PBA revetment in areas where ice loads are to be expected at a regular basis, the following aspects should be considered to minimize damage to the revetment:

- *avoid irregularities in the structure* – any protrusions, ridges or transitions in slope angle can form an attachment point against which ice can stack up;
- *use only mild slopes* – slopes steeper than 1:3 increase risk of damage as a result of ice loads.

4.3.6 COPING WITH UNEVEN SETTLEMENT AND GAPS UNDER THE REVETMENT

General

Uneven settlements in the subsoil beneath a plate revetment result in an increase of internal stresses within the plate. Since the mechanical behaviour of PBA is elastic, not viscous, these stresses will hardly reduce over time. Therefore, the extra internal stresses must be considered in design of the required layer thickness with respect to the strength of the revetment. This can be achieved by adding the extra stresses to those resulting from (hydraulic) loads.

An extreme case of imposed deformations is formed by gaps under the revetment. In practice these gaps are not the result of natural settlement but result from preceding damage (i.e. material loss from subsoil) or irregularities during construction. When detected, cavities under the revetment should always be fixed as soon as possible.

Design method

Two relatively simple methods for estimation of stresses from uneven settlement and gaps are given in Annex 2. These methods are based on an estimation of the bending stresses given a predefined deformation of the revetment.

4.4 STABILITY OF THE REVETMENT

4.4.1 GENERAL

The stability of the revetment is determined by the stability of the cover layer itself, and the (geotechnical) stability of the soil body underneath. The main driving force for instability is excessive water pressure inside the soil core of the structure. Pore pressures in the soil underneath the revetment are particularly important for the structural integrity of the revetment. Excess pore pressures beneath the revetment are caused fluctuations in the water level (nearly static) and sea state conditions, such as wave action (highly dynamic).

In this section design rules are presented for the following mechanisms:

- failure by subsoil liquefaction;
- shear slip failure of the revetment;
- uplifting of the revetment.

Subsoil liquefaction is the least understood of these mechanisms. Observations in the large wave flume have highlighted the significance of this mechanism for the PBA-revetment. Though not observed under natural conditions, it is highly recommended to consider this mechanism in design of the PBA-revetment. Liquefaction is described in section 4.4.2

Shear slip failure of the revetment may occur when the friction force between the cover layer and the soil underneath is minimized by excess pore pressures. Sliding of the revetment down the slope can be prevented by application on mild slopes and providing support with an adequate toe structure. See section 4.4.3.

A high groundwater level inside the dike body can lead to upward pressures, potentially uplifting the revetment. Because of the open structure of PBA uplift by excess overpressures is highly unlikely. In rare cases, such as complete clogging of the revetment, stability can be determined with the help of section 4.4.4.

4.4.2 DESIGN FOR FAILURE BY SUBSOIL LIQUEFACTION

General

Due to the open structure of the PBA revetment pressure fluctuations by water level variation and wave impact are easily transferred to the subsoil. When designing and dimensioning the PBA revetment, it has to be ensured that the occurring pore pressures do not result in liquefaction and subsequent damage. This can be achieved by dimensioning the filter layer in consideration of the subsoil.

As soon as soil liquefaction appears in a soil layer beneath the revetment, damages to the revetment have to be taken into account. In design of a PBA revetment the risk of subsoil liquefaction can be minimized by adequate preparation of the subsoil, and sufficiently dimensioning the cover and filter layer.

Preparation of the subsoil

The failure mechanism of subsoil liquefaction is most likely to occur in:

- non-cohesive, loosely packed soil;
- fine soils mixed with clay.

To reduce the risk of liquefaction, it is of great importance that a good compaction of the subsoil is ensured, prior to construction of the cover layer. Cohesive soils, such as clay, will not be susceptible to subsoil liquefaction. However, when mixed with fine non-cohesive soils, clay intrusions may obstruct the free flow of pore water and thus result in (local) water overpressures.

Application of filter or levelling layer

It is highly recommended to apply a granular filter or levelling layer when constructing the PBA revetment on a sandy subsoil (Alternative I and II in Figure 4.39).

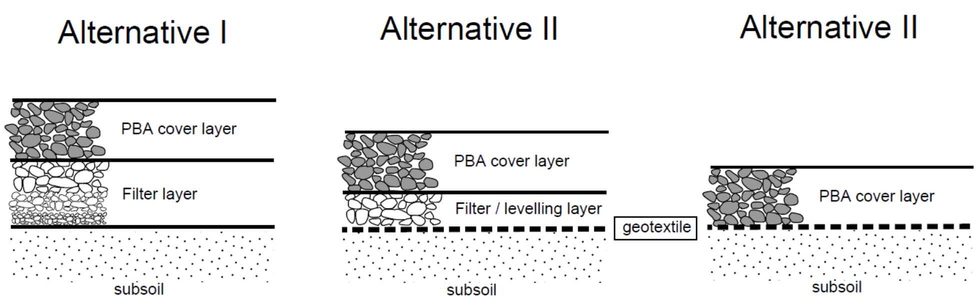


Figure 4.39

Possible design alternatives for a PBA revetment.

A sufficient total thickness of the total of PBA and filter layer will reduce the risk of liquefaction. Wave flume tests with regular waves have shown that a granular levelling layer of 0.10 m under a 0.15 m thick PBA cover layer effectively prevents the subsoil from liquefying for regular waves up to H_{m0} = 1.40 m.

When constructing a PBA revetment on sand the following is advised (regular waves):

- $H_{m0} \leq 1.40$ m → apply granular filter layer with a thickness of at least 0.10 m and a total layer thickness of at least 0.25 m;
- $H_{m0} > 1.40$ m → pore pressures have to be taken into account as design parameter in a detailed analysis. In Annex 3 some backgrounds and references are given.

Liquefaction will primarily occur under repeated wave impact and will be located at a level on the dike slope below still water level, at the level of maximum wave run-down [lit. 11]. Thus, the abovementioned granular filter layer is mainly required in the wave impact zone, down to the level of maximum wave run-down. Lower and upper parts of the structure can suffice with a minimal filter layer thickness.

The same material as used for the PBA can be used in unbound form for the granular layer underneath, combined with a geotextile to prevent loss of material from the subsoil.

OBSERVED CASE OF LIQUEFACTION

Currently, the phenomenon of failure by subsoil liquefaction has only been observed once, and under controlled conditions. In the large wave flume liquefaction occurred with regular waves of H_{m0}=1.4 m on a 0.15 m thick PBA cover layer, placed directly on a geotextile on sand with a slope 1:3. The application of a granular filter layer of 0.10 m thick sufficiently reduced the pore pressures in the subsoil to prevent it from liquefying.

For more information reference is made to LWI Report no. 988 [lit. 11].

4.4.3 DESIGN FOR SHEAR SLIP FAILURE OF THE REVETMENT

General

Generally, in the design of revetments the following shear slip failure modes can be discerned:

- macro-instability of the outer dike slope;
- shear failure over a shear plane at the structure-subsoil interface.

For the first mechanism, macro-instability, reference is made to literature concerning geotechnical stability of (water retaining) soil bodies. The second mechanism is initiated by loss of soil strength as a result of excess static or dynamic water pressures. Similar to the related mechanism of liquefaction (see section 4.4.2) shear slip failure will primarily occur under dynamic wave action at maximum wave-rundown.

Shear failure over a slip plane at the structure-subsoil interface

PBA is very permeable to water. It is therefore unlikely that a slip plane will form on the interface between the cover layer and subsoil. This type of failure is therefore most relevant for revetments that are separated from a sandy dike core by a layer of clay. The slip plan can then form at the clay/sand interface.

The mechanism of shear slip failure for PBA on clay is not principally different from other revetments placed on clay. For more information on and design for shear failure of clay on dike slopes reference is made to design guides on stone pitching revetments, such as the Dutch *Technisch Rapport Steenzettingen* [lit. 21][a].

4.4.4 DESIGN FOR UPLIFTING OF THE REVETMENT

General

In general, the instability of a PBA revetment by water overpressures is unlikely because of its high permeability. However, if the permeability of the revetment is reduced by, for instance, silting up of its pores with fine sediment some overpressures might occur. The PBA revetment can then be subjected to uplift pressures, which must be counteracted by structure weight and coherence of the structure. Water overpressures under a revetment can develop as a result of:

1. static head differences - persistent differences in water level inside and outside of the dike body;
2. dynamic head differences - short term water level variations by wave action.

Ad 1. Uplift by static head differences is *not relevant* to the PBA revetment, simply because PBA is permeable and static pressure differences between the top and bottom side of the cover layer will not occur.

Ad 2. Instability by dynamic head differences is *only relevant in case of complete clogging of the revetment over large areas*. Uplift pressures are then counteracted by both structure weight and flexural bending capacity. Only extreme situations where the siltation is trapped in the PBA structure, even under high upward pressures, will lead to potential uplift of the revetment.

For these special cases, a design method is given below and in Annex 4 of this manual. The design method considers a completely impermeable PBA cover layer, on a permeable filter layer or sandy subsoil. It should be kept in mind that this is a highly unlikely scenario, since when the PBA has been silted up, the underlying filter layer will also be silted up. This has been observed on stone pitchings in the Netherlands [lit. 31]. Therefore it is very unlikely that a hydrostatic pressure will develop under the cover layer.

SPECIAL CASE: COMPLETE CLOGGING OF THE REVETMENT

Under normal conditions, the PBA revetment has a high permeability and overpressures will not occur. When the open pores of the structure are clogged, by for instance trapped sediment, the permeability is strongly reduced and the structure weight increases. The net effect on stability under dynamic overpressures is negative.

The design method for overpressures is based on the conservative assumption that the clogged PBA revetment is completely impermeable to water. The schematization of hydraulic loads in this method is based on a structural analysis of block revetments by Vrijling (2000) [lit. 26]. The water pressure on the revetment at the moment of maximum wave run-down is schematized as hydrostatic. The resulting bending stresses in the PBA revetment are schematized conservatively with an unfavourable mechanical schematization. The complete schematization is included in Annex 4 of this manual.

Below the equations are given that can be used to determine the minimum required layer thickness. The equations can be solved iteratively or with the help of mathematical software.

$$M_{capacity} = \sigma_{bm} \frac{1}{6} d^2$$

$$P = \rho_w g \left(\frac{H_s \xi}{3 \sin \alpha} + \frac{d}{\tan \alpha} \right) \sin \alpha - \rho_{pba} g d \cos \alpha$$

$$x_0 = \frac{H_s \xi}{3 \sin \alpha} - \frac{\Delta d}{\tan \alpha}$$

$$M_{\max} = \frac{\sqrt{3}}{27} x_0 P \qquad (P > 0)$$

In which:

H_s	= significant wave height	[m]
ξ	= surf similarity parameter from peak wave period (ξ_p)	[-]
$M_{capacity}$ (x)	= moment capacity of PBA	[MN·m/m]
Σ_{bm}	= design flexural strength	[MPa]
d	= PBA layer thickness	[m]
x_0	= length over which total loading becomes directed upward	[m]
α	= slope angle of revetment	[°]
Δ	= relative density of (clogged) PBA layer ($= (\rho_s - \rho_w)/\rho_w \approx 1.1$)	[-]
ρ_{pba}	= density of (clogged) PBA (≈2300 kg/m^3)	[kg/m^3]
ρ_w	= density of (sea) water (=1025 kg/m^3)	[kg/m^3]
P	= maximum resultant pressure	[MPa]

To determine the required layer thickness the maximum bending stress in the revetment M_{max} must be equalled with the bending moment capacity $M_{capacity}$.

The graph below can be used for preliminary design of a PBA-revetment for overpressures. It is assumed that the revetment is completely silted up, which is a very conservative assumption.

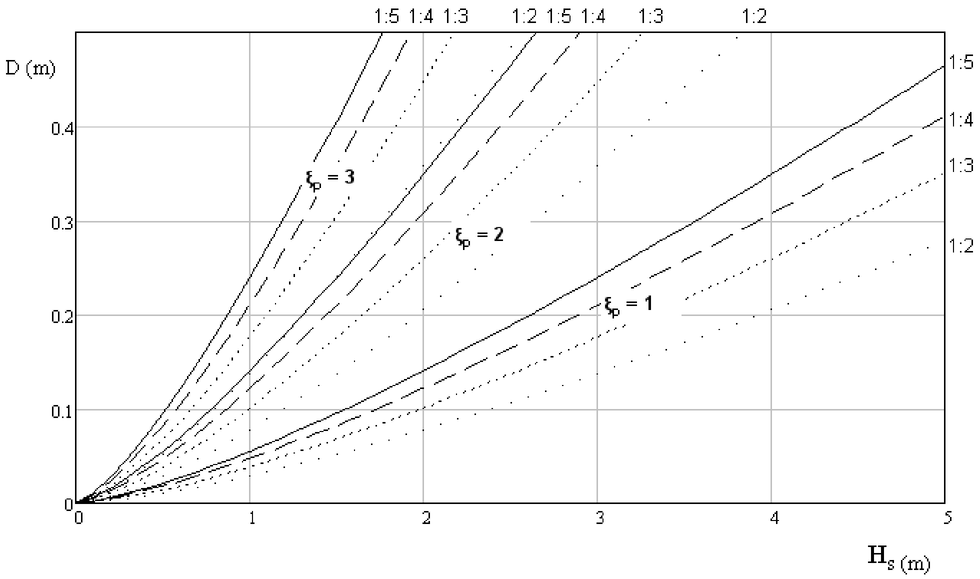


Figure 4.40

Design for overpressures with flexural moment capacity of 0.62 MPa.

4.5 DESIGN OF A STONE PITCHING REFURBISHMENT

General

The PBA revetment may be applied as a refurbishment of an existing revetment such as a stone or block pitching in case the pitching itself is not stable under design conditions. Covering of the blocks with a PBA refurbishment provides extra weight to the structure and adds extra coherence. The plate structure of PBA can prevent individual or groups of blocks from being lifted out of the structure by overpressures behind the revetment.

Design method

For design of a PBA refurbishment the design method for overpressures as discussed in section 4.4.4 can be applied, with some additions. The self weight of the underlying (block) revetment can be added to the total resistance of the PBA to overpressures. In turn, the PBA refurbishment contributes to the stability of the original revetment by adding coherence and bending capacity.

DESIGN METHOD STILL UNDER DEVELOPMENT

> The design method proposed in this section is based on a highly simplified mechanical schematization of forces and reactions in case of a refurbishment with PBA. The resulting dimensions for the cover layer are likely to be very conservative. It is advised to perform a detailed analysis for design of these refurbishments, taking into account the additional considerations at the end of this section. The supplier can be contacted to verify whether an updated version of the design method is available.

The proposed method is valid only under the assumption that there are no interaction forces (shear stresses) between the PBA and the block revetment. In other words, there is no bonding between the two layers. The total stability is then determined by the self weight of the block layer and by the self weight and bending capacity of the PBA. In terms of the equations from section 4.3.4 the equilibrium situation can be determined with the following formulae:

$$M_{capacity} = \sigma_{bm} \frac{1}{6} d_{pba}^2$$

$$M_{\max} = \frac{\sqrt{3}}{27} x_0 P \qquad (P > 0)$$

With:

$$P = \rho_w g \left(\frac{H_s \xi}{3 \sin \alpha} + \frac{d_{pba} + d_b}{\tan \alpha} \right) \sin \alpha - \rho_{pba} g d_{pba} \cos \alpha - \rho_b g d_b \cos \alpha$$

$$x_0 = \frac{H_s \xi}{3 \sin \alpha} - \frac{\Delta_{total} (d_{pba} + d_b)}{\tan \alpha}$$

In which:

H_s	= significant wave height	[m]
ξ	= surf similarity parameter from peak wave period (ξ_p)	[-]
σ_{bm}	= design flexural strength	[MPa]
d_{pba}	= PBA layer thickness	[m]

d_b	= block height	[m]
x_0	= length over which total loading becomes negative	[m]
α	= slope angle of revetment	[°]
ρ_{pba}	= bulk density of PBA	[kg/m^3]
ρ_b	= density of block layer	[kg/m^3]
ρ_w	= density of (sea) water (=1025 kg/m^3)	[kg/m^3]
Δ_{total}	= combined relative density of PBA and block layer:	[-]

$$\Delta_{total} = [(d_{pba}\rho_{pba} + d_b\rho_b)/(d_{pba} + d_b)]/\rho_w$$

The maximum bending stress in the revetment M_{max} must not exceed the bending moment capacity $M_{capacity}$.

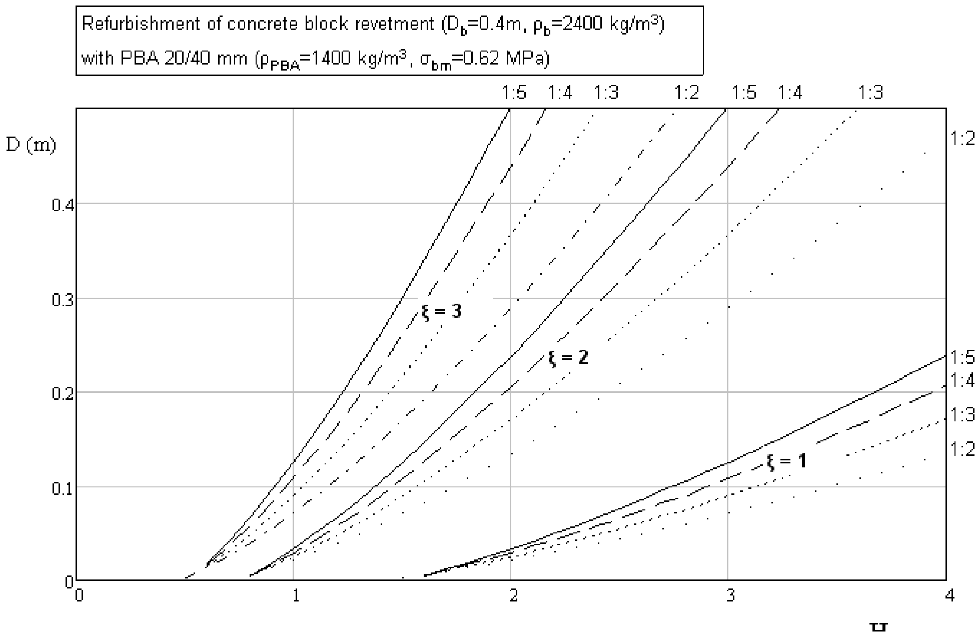
$$M_{\max} < M_{capacity}$$

In contrast to the calculations in section 4.3.4 the relative density of the *unclogged* PBA structure should be taken into account. Any added weight by clogging can be seen as a convenient contribution to stability.

The following graph shows an example of the required layer thickness for refurbishment of a 0.4 m block revetment with ρ_b = 2400 kg/m^3.

Figure 4.41

Required layer thickness for the refurbishment of a 0.4 m thick block revetment (ρ_b = 2400 kg/m^3) with PBA 20/40 mm (ρ_{pba} = 1400 kg/m^3, σ_{bm}=0.62 MPa)



Additional considerations

When constructing a PBA refurbishment on top of an existing block revetment the following mechanisms considerations must be taken into account:

- *wedging* – pure tensile stress occurs in the PBA layer, concentrated at the space between the blocks;
- *punching* – a single block or row of blocks may punch through the cover layer, due to exceedence of shear capacity of the PBA layer.

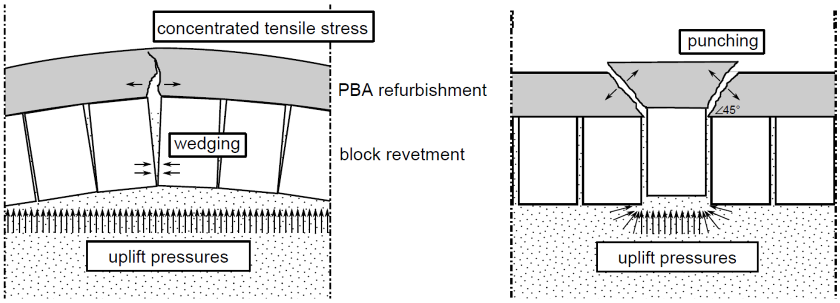


Figure 4.42
Additional mechanisms of wedging and punching in case of PBA refurbishment over a block revetment.

In order to prevent *wedging* it is very important that the PBA does *not* bond directly with the blocks underneath. This would lead to an unfavourable distribution of forces in the PBA layer when single or groups of blocks are lifted by overpressures. When there is bonding between the blocks and the PBA, pure tensile stresses will be concentrated at the joints of the blocks, which is an unfavourable loading condition. In order to prevent bonding between the PBA and the blocks, a geotextile can be used on the interface. A different solution is to make the surface of the block wet before application of the PBA.

At the moment there is no design method for the mechanism of *punching*, since the upward force of a single block is difficult to quantify. Basically, the thickness of the cover layer must be such that sufficient shear strength can be mobilised in the cross section.

4.6 DESIGN OF STRUCTURE DETAILS

4.6.1 GENERAL

Where one part of a structure ends and another part of that same or a different structure begins, a transition is present. Structure transitions require special attention in design, because they are often more vulnerable to damage than adjacent parts of the structure. For this reason the number of transitions should be kept to a minimum.

Internal transitions are transitions from one (part of the) PBA revetment to another. Where the production of one day ends and that of a following day starts, a transition is present. In large works these transitions are always present. The result is that 'fresh' PBA is constructed next to PBA that has already (partially) cured.

In order to provide support to the PBA revetment, the toe support must be a stable and non-moving structure. Further, at all edges of the PBA structure certain boundary effects should be taken into account.

4.6.2 TRANSITIONS TO OTHER MATERIALS

Requirements for transitions to other materials
There are no specific rules for the design of transitions for revetments. Some general requirements that apply to transitions of polyurethane revetments are:
- *Strength:* the transitional structure must have at least the same strength as the polyurethane revetment. Otherwise this could become a weak spot.
- *Hydraulic conductivity*: polyurethane revetments often have a larger hydraulic conductivity than surrounding structures. This should be kept in mind when determining transport patterns of groundwater.

- *Impermeability to sand:* the transition must be impermeable to sand in order to prevent loss of material, which could undermine the structure. An adequate ending or overlapping of filter layers must be part of the design.
- *Flexibility*: the transitional structure must be flexible enough to prevent high stresses in the edges of the relatively stiff polyurethane revetment.
- *Smoothness:* the surface of the transition must be smooth, without protruding edges. These could form an attachment point for wave attack, floating ice or debris.
- *Durability:* the transition must have the same durability as the revetment.
- *Constructability:* a careless design of transitions often leads to practical problems during construction.

Examples of transitions

In the following figures, some examples are given of transitions to other structures.

Figure 4.43

Example transition of PBA to a grass revetment.

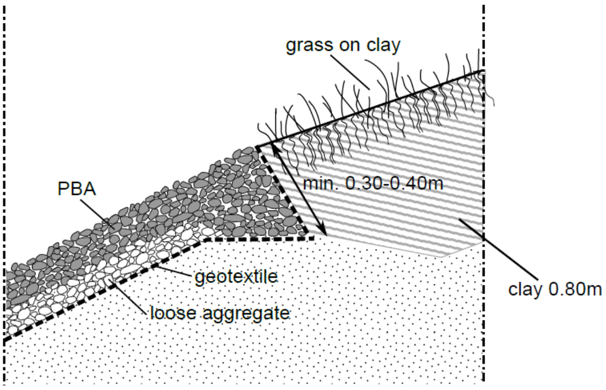


Figure 4.44

Example transition of PBA to a stone pitching.

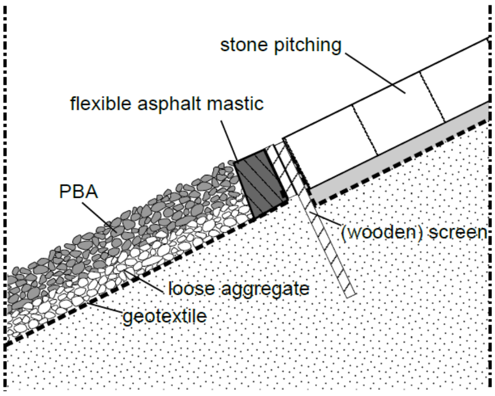
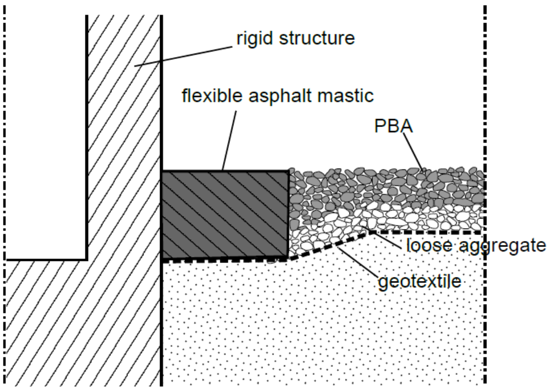


Figure 4.45

Example transition of PBA to a rigid structure.



4.6.3 INTERNAL TRANSITIONS

Requirements for internal transitions

To ensure a good bonding between the old and the new PBA it is important that the interface is kept or made clean and dry before the new PBA is applied. Further, the requirements for internal transitions are no different from those for external transitions treated in section 4.4.1.

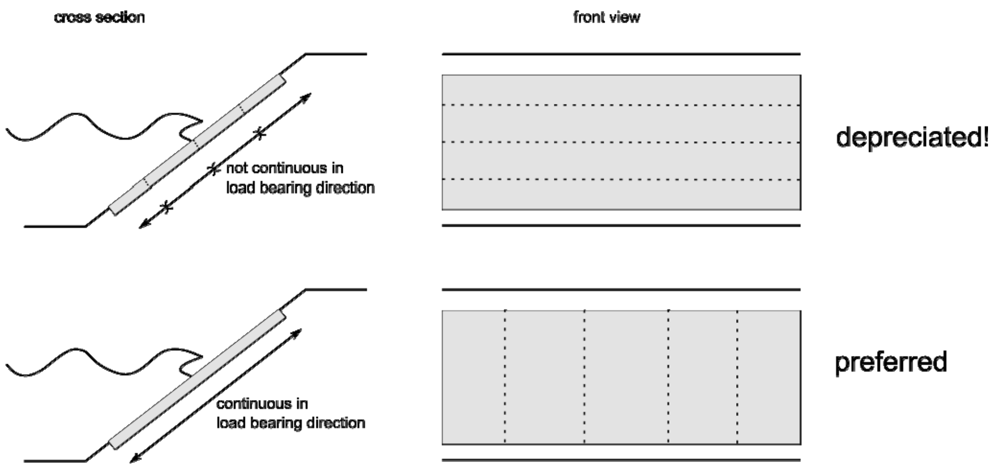
Location of internal transitions

Internal transitions automatically form a potentially weak line in the PBA-revetment. It is important to plan these transitions in such a way that they do not compromise the structural integrity where it is needed most, namely the wave impact zone.

Preferably, the transitions are oriented in the slope direction, so that there are no discontinuities in the main load bearing direction (see Figure 4.46). This way the entire plate length from top to bottom can be utilized to withstand wave impacts.

Figure 4.46

Orientation of internal transitions.



If horizontally oriented transitions cannot be avoided in the wave impact zone, they should be applied in such a way that they do not form a weak line. This can be achieved by for example:

- application of a surplus of polyurethane adhesive at the transition;
- gradually increase of layer thickness towards the transition.

Thermal expansion

All materials expand and contract to a certain extent when subject to temperature changes. For a dike body revetment temperatures variations of 40°C throughout the year are not uncommon. In design of large stretches of PBA, dilatations can be applied in order to allow thermal expansion of the material. However, the thermal expansion coefficient of PBA has not been studied yet and guidelines regarding this subject can not yet be given in this manual.

4.6.4 TOE SUPPORT

In order to provide support to the PBA revetment, the toe support must be a stable and non-moving structure. In section 4.2.6 the structural elements of a toe structure were discussed. The design of a toe structure for the PBA revetment is not principally different from design for asphalt or pitched stone revetments. In Figure 4.47 an example is given of a toe support structure. For more details reference is made to design reports on asphalt or block revetments [lit. 21].

The connection of the PBA to the toe support should be treated as a 'transition to other materials' and must therefore fulfil all requirements mentioned above.

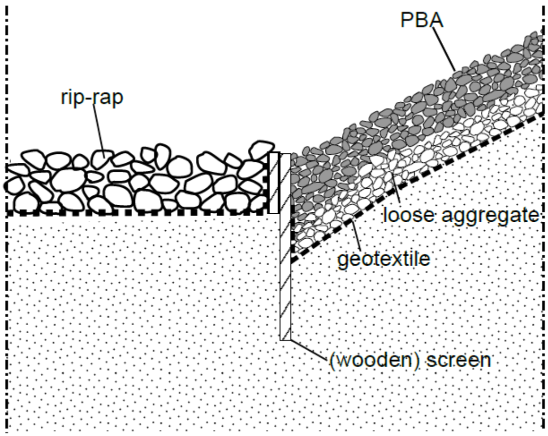


Figure 4.47
Example of toe support structure.

4.6.5 INCREASED LAYER THICKNESS AT STRUCTURE BOUNDARIES

At all edges of the PBA structure boundary effects should be taken into account. Boundary effects can cause significantly higher stresses at the edges than in the centre parts of the structure. Basically any transition in stiffness can induce boundary effects. Typically boundary effects occur at fixed ends, such as the toe support, but also at free ends and transitions to other revetment types.

In order to take into account the boundary effects it is advised to apply a gradual increase in layer thickness of approximately 20% at the ends of the structure.

CHAPTER 5 Construction aspects

5.1 INTRODUCTION

In this chapter several important aspects and attention points are summed up that are relevant for the construction of PBA-revetments.

5.2 PREPARATION OF THE DIKE BODY

Compaction and levelling

- Sufficient and evenly compaction of subsoil is of greatest importance to minimize differential settlement or soil instability. Sand should be compacted up to a relative proctor density of at least 95%.
- Levelling will reduce variability in layer thickness, but is not necessary for structural performance if the minimum required layer thickness is maintained at all places.

Presence of cavities and obstacles

- Special attention should be paid when holes or hard structures are present in the application area and a geotextile is used. Incorrect application of the geotextile may result in cavities under the revetment and reduced structure performance.

5.3 TRANSPORT AND STORAGE OF MATERIALS

PU components

- Polymer components are potentially hazardous chemicals. This means that health and safety precautions should be taken. These safety precautions include personal protection measures, such as glasses and gloves, and measures that protect and confine the chemicals at their storage depot. Reference is made to:
  - Safety instructions from production company, included with chemicals
  - safety regulations for handling chemicals
  - safety regulations for storing chemicals
- The two polymer components are stored separately in closed containers, protected against moist.
- In order to prevent leakage of spilled chemicals into the subsoil, an impermeable HDPE foil can be applied under the storage area. In case the B-component (isocyanate) component leaks, it can be neutralized with the A-component (polyol). The mixture cures and can then be treated as non-hazardous waste. The polyol component itself is not hazardous and should be removed without addition of the B-component.

Mineral aggregate

- The aggregate must remain dry before, during and after transport. This implies that exposed aggregate must be covered on site if it is raining.

- If the aggregate is or becomes moist anyway, it is necessary to dry the aggregate before it can be mixed in the composite. Drying of aggregate can be a time-consuming process.
- Handling operations on aggregate must be kept at a minimum to limit fracturing of rocks, which creates dust and fine fragments. In turn this increases polyurethane consumption (example of dry tumbler).

5.4 PRODUCTION OF PBA

Preparation of the mineral aggregate

- No special preparations are necessary, as long as the aggregate is sufficiently dry and clean. If this is not the case, the aggregate is not suitable for production of PBA.
- If the mineral aggregate is moist, it needs to be dried before mixing with the polyurethane adhesive. Drying is preferably done with (hot) air and a minimum of handling operations.
- The mixing of the PU with the aggregate must take place at an outside temperature between 5°C – 30°C. If the mineral aggregate has a too high temperature (too hot to hold in the hands), for instance due to the drying process, it needs to be cooled down before mixing with the PU.
- After delivery, handling operations that might cause wearing of the rocks must be kept at a minimum. In example: tumbling of the aggregate creates a lot of fine fragments).
- Further reference is made to handbooks on the use of rock in hydraulic engineering [lit. 19].

Mixing the PU components

- The polyurethane components can be mixed mechanically or with a special mixing pump. The result must be a homogeneous substance.
- The curing process starts once the components come in contact with each other.
- The curing process is an exothermic chemical reaction. To limit the increase in temperature of the mixed substance, the mixing amounts need to be kept relatively small (in case of a batch process) or in constant movement (continue process).
- Example: for a batch of 0.5 m^3 of aggregate, an amount of 7.5 l of PU is needed, which takes approximately 2 minutes to mix mechanically.

Mixing the PU with the mineral aggregate

- Mixing of the PU and mineral aggregate is done in a mechanical process, similar to the mixing of concrete.
- The mixing ratio PU-aggregate depends on the size, grading and amount of fine fragments of the aggregate. Presence of fine fragments increase PU consumption. The mixing ratio with PU is based on fairly clean rocks, a high content of fine material results in a higher PU consumption. A small surplus of PU can be used to account for fine fragments in the aggregate.
- The mixing process is continued until every rock is covered with a thin film of PU. This can be assessed by visual observation. If the rocks are not covered fully within a reasonable amount of time (approx 3 minutes), extra PU needs to be added.
- Example: a batch of 0.5 m^3 of aggregate takes about 3 minutes to be fully covered with PU, when mixed with a standard concrete mixer.
- The use of a mobile mixing unit should be considered in order to make efficient use of the time needed for mixing the PU with aggregate. The mixing can then take place during the transport of the materials from the storage to the application area.

5.5 APPLICATION ON THE DIKE SURFACE

- Once the curing process has started (after mixing of the PU components), the processing time is approximately 20 minutes (at 23°C).
- The PU-aggregate mixture must be dumped in the application area and brought to profile within the processing time. After the processing time has been exceeded, any movement of the rocks will result in a reduction of strength. This is a special point of attention when producing relatively thick layers of PBA.
- It is normal that some of the surplus polyurethane leaks down to the filter layer. This does not negatively affect the bonding strength. A layer of loose aggregate can be used to prevent excess PU from blocking the geotextile.

5.6 SURFACE TREATMENTS

- Surface treatments can best be applied within the processing time, when the adhesive is still ‘tacky’.

5.7 LOADING OF THE STRUCTURE AFTER APPLICATION

- After 24 hours, the structure can be loaded and accessed by foot.
- After 2-3 days the structure has reached its full strength.
- If heavy construction material is transported over the PBA revetment, metal slabs can be used to spread the loads.

CHAPTER 6 Quality control

6.1 INTRODUCTION

The quality of a structure depends both on the used materials and execution. During execution of the work the guidelines provided by the supplier of the PU should always be followed in order to achieve a revetment of the quality that it should have. In this manual some attention points are given, that are important for achieving best material quality and should be tested.

6.2 ATTENTION POINTS FOR CONSTRUCTABILITY

Prior to and during construction the following points should regularly be controlled.

- Temperature – The temperature directly influences the curing time of the PU and therefore the available processing time. At high temperatures the processing time is shorter than at low temperatures. If the material is applied or brought to profile after the processing time has elapsed, bonding of the aggregate deteriorates. The processing temperature should be between 5°C – 30°C.
- Moist – The aggregate must be dry (≤ 0.1% humidity) before it is mixed with the PU. Uncured PU reacts with water, creating small gas bubbles. Too much moist on the rocks results in air entrapments in the coating and deterioration of strength.
- Dust and fines – A too high amount of dust and fines results in a higher amount of PU needed to achieve complete coverage of the rocks. If the amount of PU is not adequately adjusted to the amount of fines, the strength of the composite deteriorates.

6.3 MONITORING AND INSPECTION

The quality of the end product should be tested. This should be done visually during construction and after by testing samples that were taken during construction.

- Visual assessment – Discoloration often is a sign of physical changes in polymers. If the coating on the rocks appears to contains small air inclusions or has a milky appearance this may indicate a quality problem. Stripping and loss of bonding may occur. See Annex 4 of this manual for examples.
- Testing by hand – If 24 hours after construction individual rocks can easily be broken from the revetment by hand, this may indicate a quality problem.
- Mechanical tests – During construction on-site cube samples should be taken regularly to be subjected to compression tests. The tests results give an indication of achieved material strength and the continuity of production quality. Sample taking and mechanical tests are highly recommended to check the production quality in case there are irregularities or changes in the production process (i.e. bad weather, large amounts of dust, etcetera).

If before mentioned tests or the age of the structure give reason to be doubtful whether the material still has its original strength, samples can be taken from the revetment to be tested for compressive strength and compared to the strength of the cube samples that were taken during construction. If needed, more comprehensive tests can be performed by testing beams from the revetment in a laboratory for stiffness and flexural strength. PBA is well capable of withstanding the mechanical loads that are introduced by drilling or saw equipment. After removal of a core sample, the bore hole can be filled up with new PBA or asphalt mastic.

Figure 6.48

Obtaining a drilling sample from a PBA revetment.

6.4 MAINTENANCE AND REPAIR

The PBA revetment is relatively easy to repair when damaged or when samples have been taken. By hand or with light equipment small amounts of PBA can be produced on-site to fill any holes that have been formed. When repairing a part of the revetment the following should be considered:

- the surface against which the new PBA is applied must be dry and clean in order to create a good bonding. To ensure a complete connection between the old and the new material an excess of PU adhesive may be applied on the interfaces;
- the sand tightness of the revetment must be maintained. Special attention should be paid to the continuity of geotextile filters.

6.5 SAFETY ASSESSMENT

In time, requirements that are demanded for structures may change, as well as insight in loading or hydraulic conditions. Therefore, during the lifetime of the revetment several safety assessments will be carried out, in order to determine whether the revetment still fulfils these requirements.

As long as the material can be considered to have the same quality as directly after completion of the work, it is sufficient to apply the design rules with the original mechanical properties and based on up-to-date normative hydraulic conditions. Otherwise, appropriate mechanical properties must be determined and used for assessment.

CHAPTER 7 Remaining uncertainties and risks in the design of PBA revetments

7.1 INTRODUCTION

Since PU is a relatively new bonding material in hydraulic engineering applications, the PBA revetment does not have the advantage of extensive field experience and long term observations that conventional revetments have. The still limited knowledge of mechanical properties of some mixtures and of the long-term behaviour of the revetment introduces some uncertainties in the application of PBA revetments.

In this chapter an overview is given of uncertainties and assumptions that remain in the design process. In time, these uncertainties will be resolved. Until then, some measures are proposed that can be taken to minimize the risk of using PBA revetments.

7.2 REMAINING UNCERTAINTIES AND ASSUMPTIONS

The following uncertainties are still to be resolved.

- PBA has been tested under irregular waves of up to Hm0=1.5 m. Extrapolations to larger wave heights should be used with caution.
- The effect of variation in stone size and/or mixing ratio on the stiffness and strength of PBA has not yet been studied. Therefore, the mechanical properties of new mixtures should always be tested prior to construction.
- The design values for flexural strength and stiffness are based on a limited number of bending tests. Conservative values have been assumed, but actual values will vary per mixture.
- The mechanical models in this manual assume a full flexural strength as from a layer thickness of at least 6.5 times D_{n50} and a zero flexural strength at a thickness of zero times D_{n50}. The strength for layer thicknesses between 0-6.5 times D_{n50} is based on linear interpolation. This is assumed to be a conservative approach.
- The design method for wave impacts assumes that PBA is not susceptible to aging or fatigue during the lifetime of the revetment. In other words the initial design strength is assumed irrespective of loading history. The susceptibility has not yet been fully verified in tests.
- No (empirical) relation is yet available to relate flexural bending strength of PBA to the compression strength of cubes. Therefore, on-site cube sample tests cannot yet be translated to equivalent design parameters.

- The relation between susceptibility to subsoil liquefaction and the thickness of the PBA+ granular filter has not been studied. The construction height proposed in this manual is based on only one series of tests at the large wave flume in Hannover.

7.3 RECOMMENDATIONS FOR MINIMIZING RISK

For mixtures for which the mechanical properties are not specified in this manual it is recommended to perform additional mechanical tests to verify the validity of the design values for stiffness and flexural strength. Relevant mechanical tests are the four-point bending test to determine flexural strength and the compression test on cube samples for quality control.

Finally, it is recommended to only apply PBA revetments on locations that are well visible and accessible for inspection.

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ANNEX 1 Design for wave impacts

Schematization method

Analogous to asphaltic revetments the PBA structure will be schematized as a plate on linear-elastic foundation (TAW, 2002) [lit. 15]. The plate is loaded by a wave impact, schematized as a triangular distributed load q(x) with a maximum of P_{max} and a base width equal to *H*.

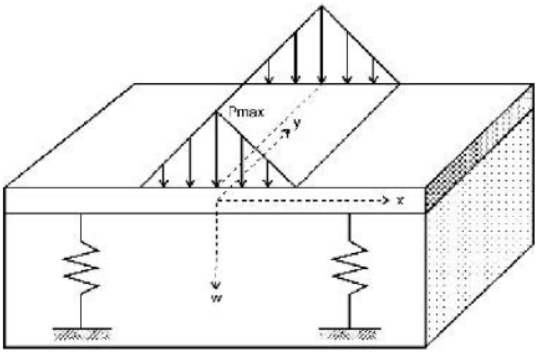


Figure A1.1
Schematization of the wave impact by a triangular load (TAW, 2002).

The determination of design plate thickness is done in three steps:

- Step 1: Determination of design wave impact load
- Step 2: Choice of representative material parameters
- Step 3: Determination of required plate thickness

Below a short description is given on choices and actions taken with each step. This section ends with some pre-calculated graphs that can be used for design.

Step 1: Determination of design wave impact load

Schematized wave impact pressure
For wave impacts the design wave height H_d should be chosen sufficiently high such that the chance of exceedence of this wave height is acceptable. The wave impact load is schematized as a triangular load with a base width equal to H_d. The maximum impact pressure occurs in the middle of the wave load and is estimated with the following equation.

$$P_{\max} = \rho_w \cdot g \cdot q \cdot H_d$$

Where:

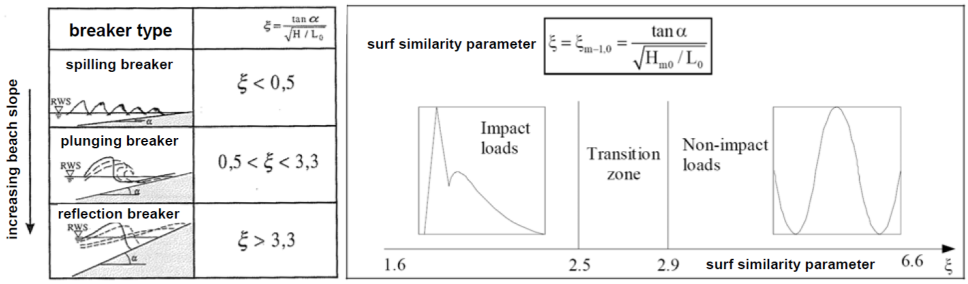
ρ_w	= mass density of water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]
q	= wave impact parameter	[-]
H_d	= design wave height	[m]

Wave impact parameter q

The factor q is influenced by the wave breaker shape, described by the surf similarity or breaker parameter $\xi_{m-1,0}$, and the wave-structure interaction. It is necessary to differentiate wave loads (Figure A1.2) with regard to the different waves, breaker types and loads on and beneath the PBA revetment. Collapsing breakers result in impact loads. Surging breakers, on the other hand, tend to non-impact loads.

Figure A1.2

Classification of wave loads subject to the surf similarity (breaker) parameter [lit. 9].



Within the scope of the model tests in the Large Wave Flume at Hannover [lit. 11], comprehensive analyses of the wave loads on and beneath a PBA revetment were performed. The measured data was analysed on the basis of the rendered parameterisations. In the following figures the results of the pressure loads on and beneath the PBA revetment are presented for the two load types:

- impact load (Figure A1.3) and;
- non-impact load (Figure A1.4).

Figure A1.3

Maximum compressive load on and beneath the revetment due to dynamic impact loads [lit. 9].

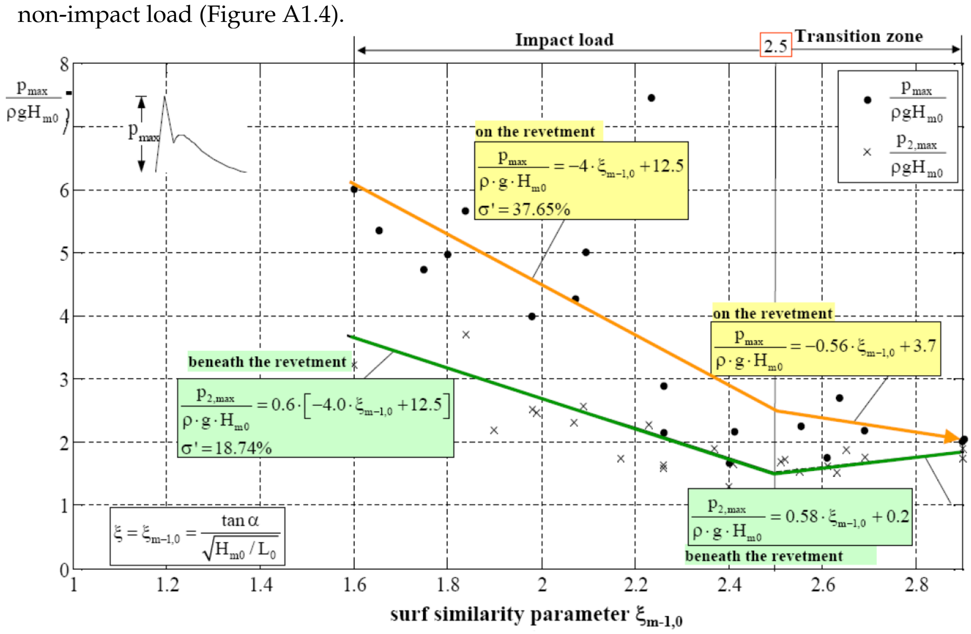
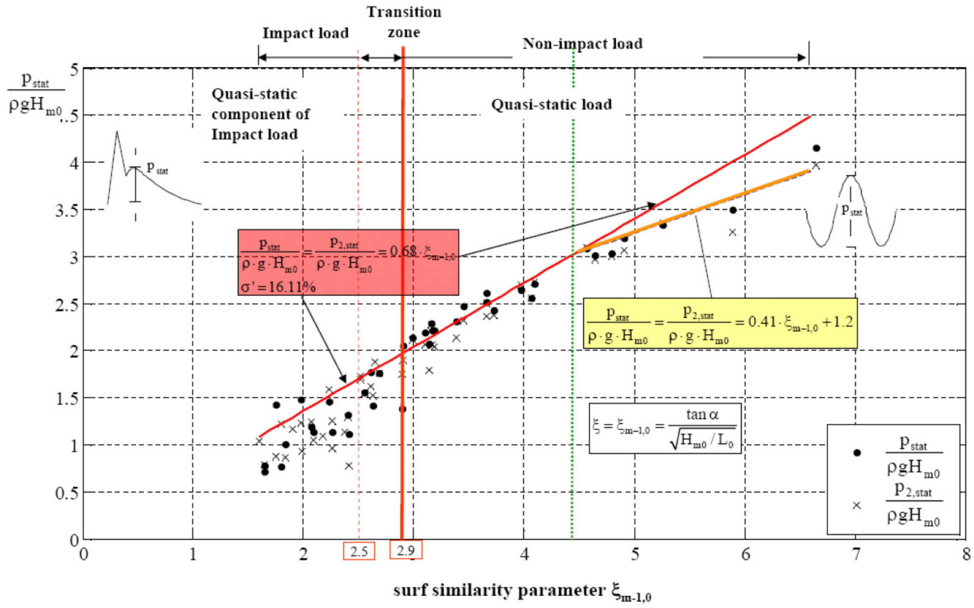


Figure A1.4

Maximum compressive load on and beneath the revetment due to non-impact loads [lit. 9].



For impact load q-values up to 6 were found. For the transition zone q-values in the order of 2 were found. For non-impact loads q-values up to 4 were found.

In case of the wave impact loads, a differentiation between the load levels on and beneath the revetment is discernable (Figure A1.3). Just beneath the revetment the q-values are reduced by a factor of about 0.6. The difference between the two curves in shows the damping of the wave pressure by the revetment. In case of the non-impact loads, virtually the same pressure rates appear on and beneath the revetment (Figure A1.4). Consequently, the largest deformations of the PBA revetment were found as a result of the quasi-static or non-impact part of the loads.

For design of the PBA revetment a factor $q = 5.0$ is advised. For irregular wave fields this is a conservative assumption.

Step 2: Choice of representative material parameters

Foundation stiffness

The stiffness of the foundation has a large influence on the stresses in the PBA cover layer when loaded. A higher soil stiffness results in a larger part of the load being directly transferred to the subsoil, which is a favourable condition for the loading of the PBA cover layer. The stiffness of the foundation can be expressed in the soil compression parameter *c* [MPa/m]. This parameter is related to the dynamic stiffness E_{dyn} of the soil. In section 4.2.4 of this manual values are given for the soil compression parameter for several foundation types.

The soil stiffness depends on many factors, including soil grading and compaction. Because it is difficult to make a reliable estimate of the soil stiffness it is advised to use a conservative value (= low value) for the compression parameter *c* in design calculations.

PBA stiffness

PBA has a relatively high stiffness when compared to open stone asphalt. The stiffness of PBA depends on the type and grading of the aggregate and on temperature (see also section

2.4.2 of this manual). For design calculations a conservative value of E = 3,000 MPa can be used.

PBA flexural strength
PBA has a relatively high flexural strength when compared to open stone asphalt. Flexural strength can be in the order of 0.6-1.0 MPa (see also section 2.4.4 of this manual). The strength is mainly dependent on type and grading of the used aggregate. For design calculations the lower 5% boundary value is advised. The choice of this conservative value also accounts for some uncertainties in the mechanical properties of the material.

Step 3. Determination of required plate thickness
When the wave impact load and the material parameters have been determined, the maximum tensile stress in the bottom of the PBA revetment, directly under the wave load (x=0) can be determined as follows:

$$\sigma_{\max} = \frac{P_{\max}}{4\beta^3 z}\left[1 - e^{(-\beta z)}(\cos(\beta z) + \sin(\beta z))\right]\frac{6}{D^2}$$

with:

$$\beta = \sqrt[4]{\frac{3c(1-\nu^2)}{ED^3}}$$

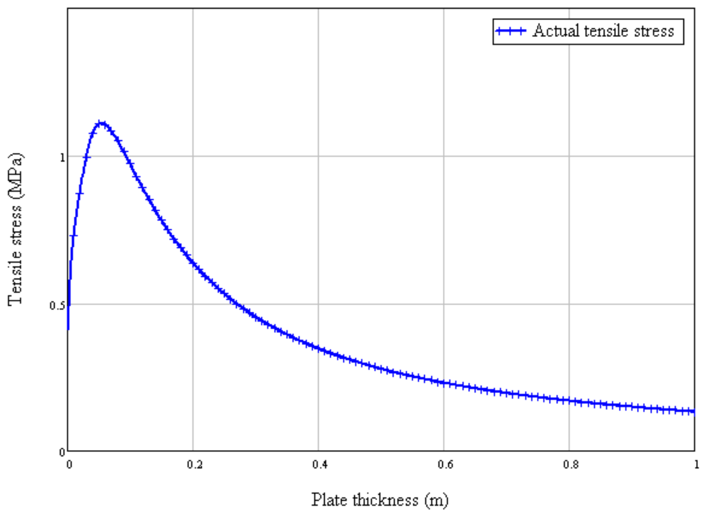
In which:

σ_{max}	= bending strength	[MPa]
P_{max}	= maximum pressure of triangular wave load	[MPa]
c	= coefficient of compression of subsoil	[MPa/m]
E	= elastic modulus of the cover layer	[MPa]
D	= thickness of the cover layer	[m]
ν	= constant of Poisson (assumed to be ν =0.35)	[-]
z	= half the triangular load width, z=0.5H	[m]
H	= design wave height H_{m0}	[m]

In figure A1.3 an example is given of the resulting relation between plate thickness and actual tensile stress. The curve in figure A1.4 shows a local maximum (in this case at a plate thickness of approximately 0.06 m and a tensile stress of 1.1 MPa). Only points at the right hand side from this local maximum (i.e. thicknesses larger than 0.06 m), may be used for design calculations.

Figure A1.5

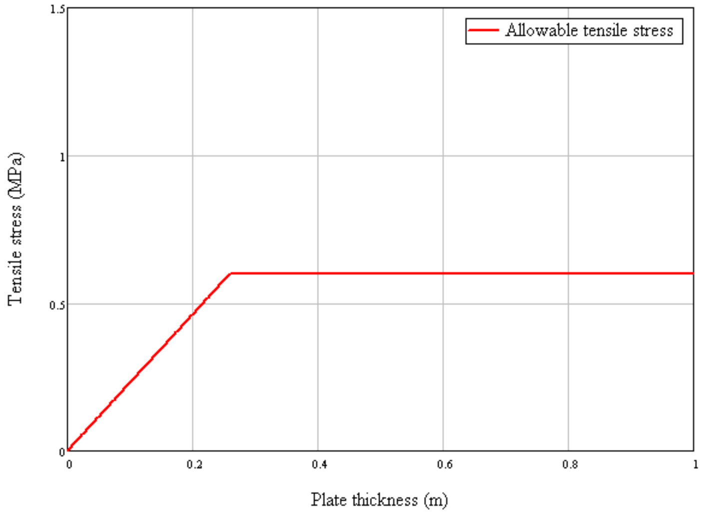
Actual tensile stress as a function of plate thickness for:
H_d = 1.3 m
q = 5.0
c = 30 MPa/m
E = 3,000 MPa



The basic assumption is made that the macroscopic properties of the plate apply if the thickness of the plate is not smaller than a predefined number of stone sizes D_{n50}. It is assumed that a thickness of at least 6.5 times D_{n50} is required for the plate to reach full design flexural strength. For smaller plate thicknesses the maximum tensile stress is found by linear interpolation. Figure A1.5 shows the resulting design strength in relation to plate thickness, for aggregate with a grading of 30/60 mm, D_{n50} = 40 mm.

Figure A1.6

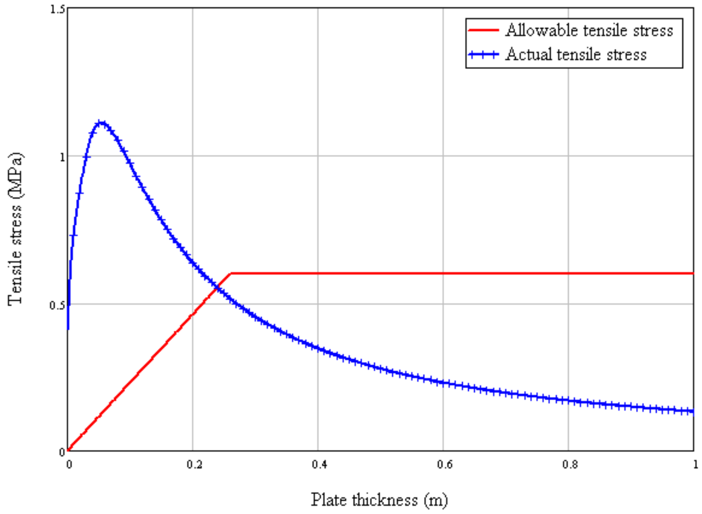
Assumed development of flexural strength as a function of plate thickness.



Finally, in order to determine the minimum required plate thickness given the input wave load and material parameters, the calculated maximum tensile stress is equalled to the design flexural strength. This can best be done visually, as shown in figure A1.7. It is important that the intersection of the two lines is at the right of the local maximum of the local maximum in the actual tensile strength curve.

Figure A1.7

Graphical determination of required layer thickness.



The calculation and graphical presentation of the actual tensile stress as a function of plate thickness cannot easily be performed by hand calculation. It is advised to program the given formulae into math or spreadsheet software and check the outcome by producing graphs.

In the following figures, several typical scenarios are pre-calculated for use in preliminary design of PBA structures.

Figure A1.8

Design layer thickness for PBA limestone 30/60 mm.

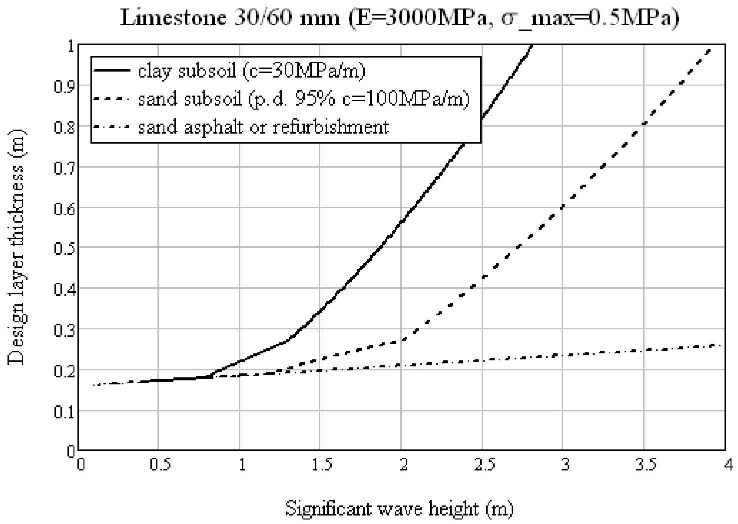


Figure A1.9

Design layer thickness for PBA limestone 20/40 mm.

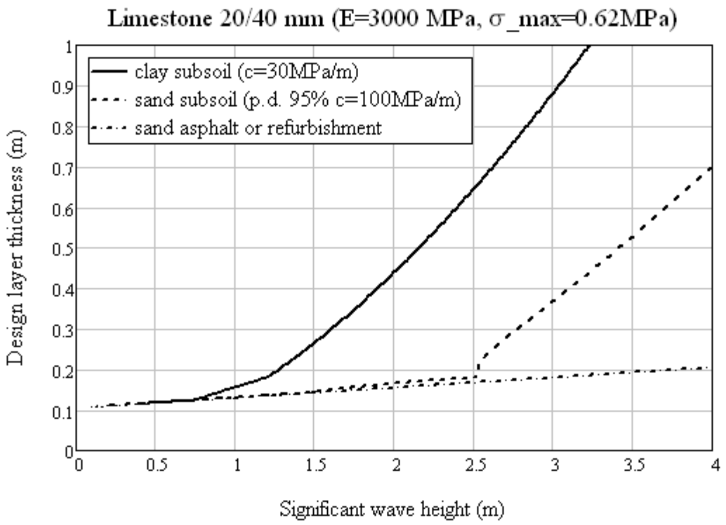
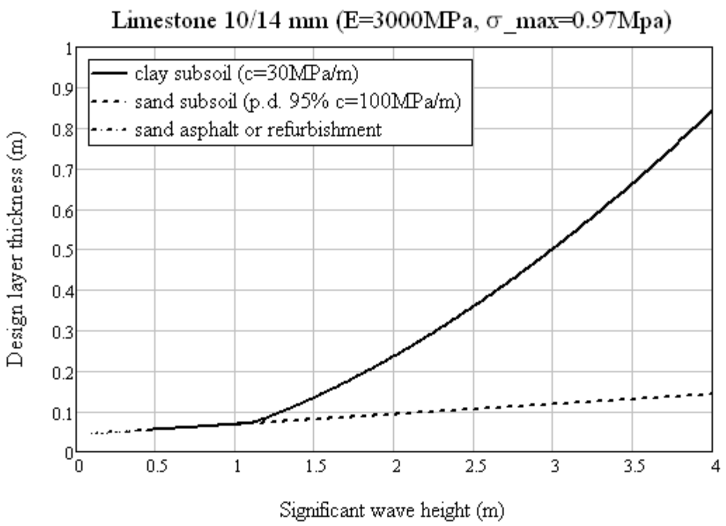


Figure A1.10

Design layer thickness for PBA limestone 10/14 mm.



ANNEX 2 Design for uneven settlements and gaps

Design for uneven settlement

Estimation of flexural stresses resulting from uneven settlements is based on the following assumptions:

- the spatial distribution of uneven settlement is such that no gaps are formed beneath the revetment;
- deformation of the PBA revetment is purely elastic – no creep or relaxation occurs.

The maximum flexural stress in the bottom of the plate revetment is obtained from the settlement curve w(x). This curve can be either estimated or derived from a graphical representation of settlement in a soil body. The flexural stress can then be calculated as follows:

$$\sigma_{\max} = K \frac{6}{D^2} \frac{d^2 w}{dx^2}$$

In which:

σ_{max}	= maximum flexural stress	[MPa]
K	= stiffness of the plate, described by: $K = ED^3 / 12(1-\nu^2)$	
E	= elastic modulus of the cover layer	[MPa]
D	= thickness of the cover layer	[m]
ν	= constant of Poisson (assumed to be ν =0.35)	[-]

In case of a relatively slow deformation process, which is usually the case with settlements, the value for the elastic modulus of PBA is lower than the value used for highly dynamic mechanisms such as wave impact. The elastic modulus for slow deformations can be derived experimentally.

Subsequently the design value for flexural strength calculations should be reduced with the calculated extra stresses from imposed deformations.

Example

Now, an *example* is given for the first estimation of flexural stresses as a result of an estimated settlement curve. It is assumed that the settlement *w(x)* has a parabolic shape over distance *L* and a maximum value of *δ* at x = 0.

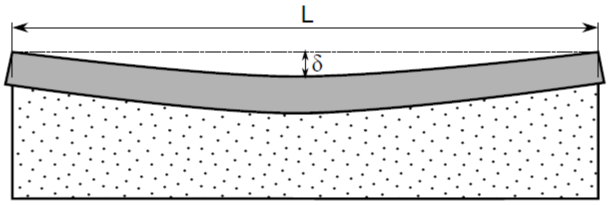


Figure A2.1
Parabolic deformation over length *L* and depth δ.

The settlement curve is then described by:

$$w(x) = ax^2 - \delta$$

With

$$a = \frac{\delta}{(\frac{1}{2}L)^2}$$

The second order derivative is:

$$\frac{d^2w}{dx^2} = 2a$$

So that the maximum flexural stress is described by:

$$\sigma_{\max} = K\frac{6}{D^2}2a = K\frac{6}{D^2}\frac{2\delta}{(\frac{1}{2}L)^2} = 48\frac{ED}{12(1-\nu^2)}\frac{\delta}{L^2}$$

Assuming a bending modulus of E = 1,000 MPa, a layer thickness D = 0.15 m and a gap with a depth of δ = 0.05 m and a span of L = 20 m, the maximum tensile stress in the plate as result of this deformation will be approximately 0.09 MPa. This extra tensile stress can be taken into account in design of the cover layer thickness by reducing the design flexural strength accordingly.

Design for gaps under the revetment

Design for (deep) gaps under the revetment is not principally different to the design for uneven settlements. However, the dimensions of the gap are such that a hollow cavity is formed which is bridged by the revetment.

The stresses in the revetment are determined by the self weight q of the cover layer, the plate stiffness K, and the dimensions of the gap. Now consider a circle shaped gap beneath the revetment. The revetment will bend under its own weight. Deformation is described by a circle shaped plate, fixed at its edges and loaded by its own weight.

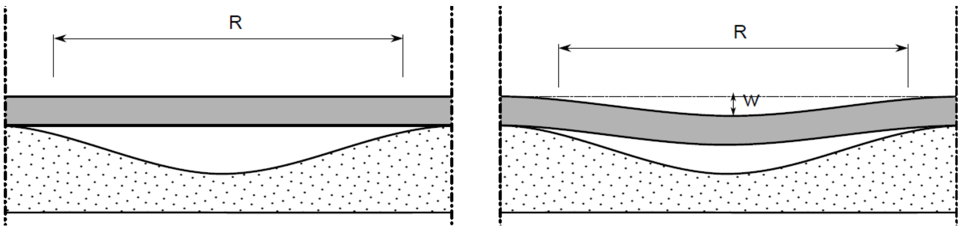


Figure A2.2

Deformation by own weight in case of circular shaped gap under the revetment.

The depth of the gap $w(x)$ is a function of the distance x from the centre and the radius of the gap R:

$$w(x) = \frac{\rho_{PBA}gD}{64K}(x^2 - R^2)^2$$

The second order derivative is:

$$\frac{d^2w}{dx^2} = -\frac{\rho_{PBA}gD}{16K}(R^2 - 3x^2)$$

So that the maximum flexural stress (at x=0) is described by:

$$\sigma_{\max} = -K\frac{6}{D^2}\frac{-\rho_{PBA}gD}{16K}R^2 = \frac{3\rho_{PBA}g}{8D}R^2$$

Assuming a layer thickness D = 0.15 m, bulk density of ρ_{PBA} = 1400 kg/m^3, and a gap radius of R = 2.5 m, the maximum tensile stress in the plate as result of this deformation will be approximately 0.22 MPa. This extra tensile stress can be taken into account in design of the cover layer thickness by reducing the design flexural strength accordingly.

The example given above shows that the effect of a gap under the revetment can be far more significant than uneven settlement. In practice these gaps are not the result of natural settlement but result from preceding damage (i.e. material loss from subsoil) or irregularities during construction. When detected, cavities under the revetment should always be fixed as soon as possible.

ANNEX 3 Design for subsoil liquefaction

The following text is an excerpt from the article:
OUMERACI, H, et. al. (2010) Hydraulic performance, wave loading and response of ELASTOCOAST revetments and their foundations. XIèmes Journées Nationales Génie Côtier – Génie Civil –

Stability analysis of sand core beneath the revetment against soil liquefaction:
Generally, wave-induced cyclic loads generate both pore water pressure u and effective stresses σ′ inside the subsoil. The total pore pressure $u_{tot}(z,t)$ in the seabed is composed of the hydrostatic component $u_0(z)$ and the wave induced component u(z,t):

$$u_{tot}(z,t) = u_0(z) + u(z,t)$$

The wave-induced component u(z,t) is also called "excess pore pressure" (in excess of the hydrostatic pressure). Because of the resetting of the pressure transducers to zero (u=0) at still water level before each individual test, the initial hydrostatic component $u_0(z)$ is not considered in the data analysis.
Liquefaction occurs, if the excess pore pressure u(z,t) inside the soil reaches the value of the initial effective stress σ'_0 which means, that the shear resistance τ_s of the soil tends to zero (τ_s=(σ′-u) tanφ with φ=internal friction angle). In general, soil liquefaction may be induced by two mechanisms (i) upward pressure gradient in the soil during the passage of the wave trough (*transient* or *instantaneous liquefaction* and (ii) Build up of mean excess pore water pressure (*residual liquefaction*)
During the passage of a wave trough the induced excess pore pressure in the soil becomes "negative" in the sense that an upward directed pressure gradient ($u_o - u_t$) is generated. This gradient results from the decrease of transient pore pressure u_t with increasing depth z´ (figure 24).

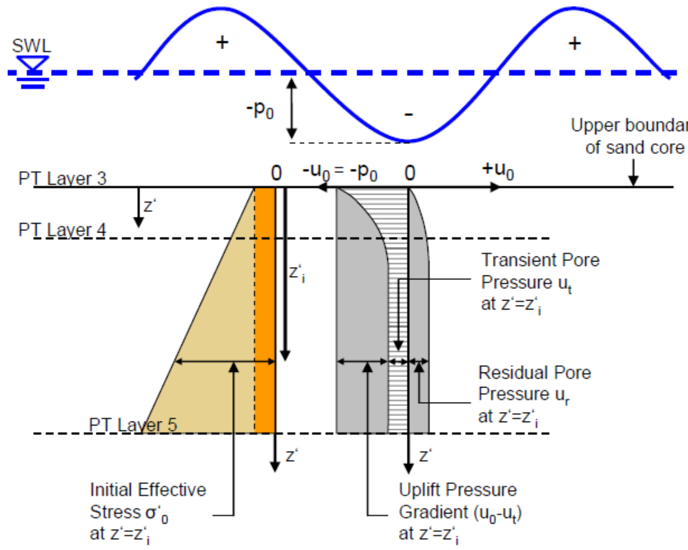


Figure 24. Distribution of initial effective stress, excess pore pressure amplitude and uplift pressure gradient in the sand core under the passage of a wave trough

If the pressure gradient ($u_o - u_t$) at a certain location z in the sand core reaches the effective stress σ'_{vo} due to the submerged weight of the soil $(\sigma'_{v0})_s$ and that of the revetment $(\sigma'_{v0})_r$ at this location,

$$\sigma'_{v0} = \underbrace{\rho'_s \cdot g \cdot z}_{(\sigma'_{v0})_S} + \underbrace{\rho_r \cdot g \cdot d_r}_{(\sigma_{v0})_r}$$

with:

σ'_{v0}	Initial effective vertical stress	[N/m^2]
ρ'_s	Bulk density of submerged soil (sand)	[kg/m^3]
ρ_s	Bulk density of soil (sand)	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]
g	Gravitational acceleration	[m/s^2]
z	Depth of sand core	[m]
ρ_r	Density of revetment including filter layer	[kg/m^3]
d_r	Thickness of filter layer and Elastocoast revetment	[m]

the soil gets into suspension and behaves like a fluid. This is called *transient soil liquefaction*, as this phenomenon is limited to the short period during the passage of the wave trough. *Transient liquefaction* occurs if the pressure difference (u_0 - u_t)=0 becomes equal to or larger than the initial effective stress σ'_{v0}:

$$\sigma'_{v0} - (u_0 - u_t) = 0$$

with:

σ_{v0}	Initial effective vertical stress	[N/m^2]
u_0	Initial (hydrostatic) pore water pressure	[kg/m^3]
u_t	Instantaneous (transient) excess pore water pressure	[Pa]

Moreover, *residual* (or *mean*) pore pressure u_r may build up gradually from cycle to cycle. If it reaches the value of the initial effective normal stress σ_{v0}, the contact between the soil grains vanishes and the soil behaves like a fluid. This is called residual soil liquefaction:

$$\sigma'_{v0} - u_r = 0$$

with:

u_r	Residual excess pore water pressure	[Pa]

Though residual pore pressure is relatively rare or not significant under wave action alone, both residual pore pressure u_r and transient pore pressure u_t should be considered in the limit state equation for soil liquefaction:

$$\sigma'_{v0} - \left[(u_0 - u_t) + u_r\right] = 0$$

Based on this equation, the stability analysis of the sand core beneath the revetment against soil liquefaction can be performed as schematically illustrated in figure 25.

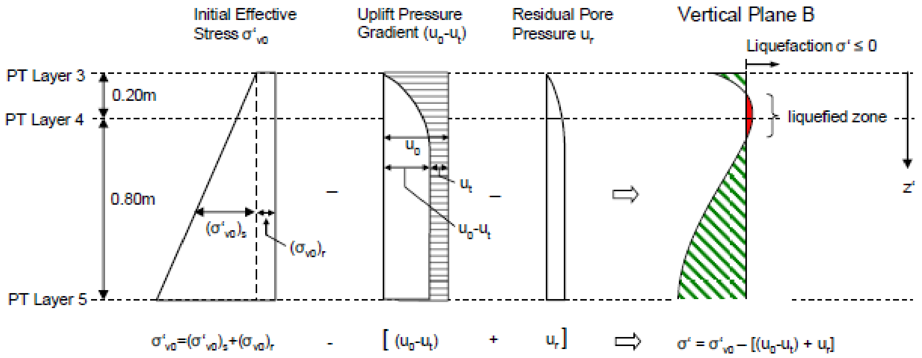


Figure 25. Stability analysis against soil liquefaction beneath the revetment (principle sketch).

ANNEX 4 Design for uplift

Introduction

The periodic variation in water level by wave action can cause a temporary difference in water levels in front of and behind the revetment. The resulting overpressures tend to lift the cover layer. The time scale of this mechanism is in the order of one wave period. Under normal conditions the permeability of a PBA-revetment is such that overpressures are quickly relieved through the open pore space of the structure. However, it is possible that this open space is filled with fine sediments. Bijlsma (2008) [lit. 2] modelled the hydraulic conductivity of PBA and found that clogging of the structure strongly affects the permeability, to such an extent that dynamic overpressures can cause heave of the cover layer. The phenomenon of clogging of the PBA-revetment has been observed under prototype conditions [lit. 4].

Figure A4.1

Extreme example of PBA revetment completely clogged with organic material (Bijlsma, 2009).

Design for overpressures

For design for overpressures, a relatively simple structural method is adopted from (Vrijling, 2000) [lit. 26]. Originally intended for use with block revetments, this model represents the mechanical behaviour of a block revetment under wave loading. Vrijling assumed that a hydrostatic water pressure, defined by the still water level, exists behind the revetment. At the outside of the revetment the water pressure reaches a minimum during wave run-down. The self weight of the revetment and the unit interaction by friction and clamping of the blocks determine the structure resistance to the overpressures.

The wave run-down z_{rd} can be related to the significant wave height H_s [m] and the surf similarity (Irribarren) parameter ξ [-]. It is described by the following expression:

$$z_{rd} = \frac{H_s \xi}{3}$$

In which:

z_{rd}	= 2% run-down on straight smooth slopes ($R_{d2\%}$)	[m]
H_s	= significant wave height	[m]

ξ = surf similarity parameter from peak wave period (ξ_p) [-]

Figures A2.2 and A2.3 show the schematization of forces acting on the revetment.

Figure A4.2
Schematization of the block revetment at wave run down (Vrijling, 2000).

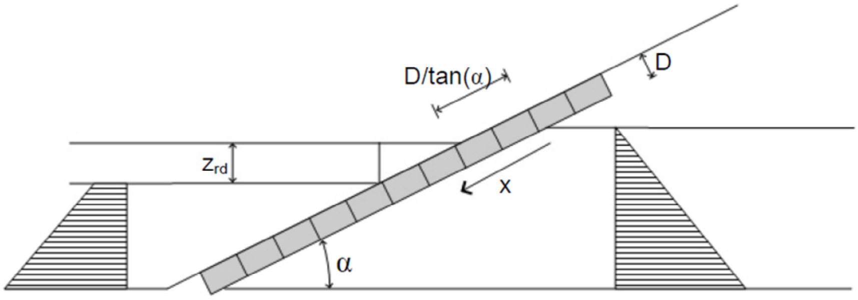
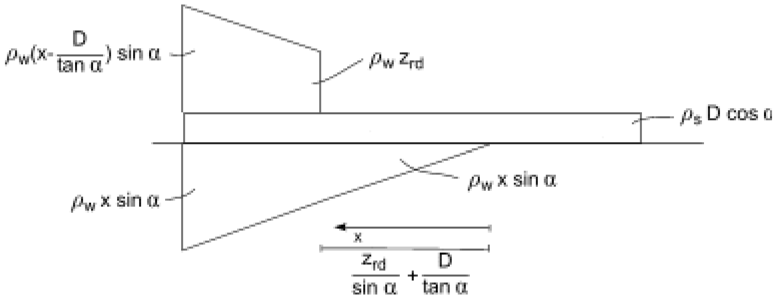
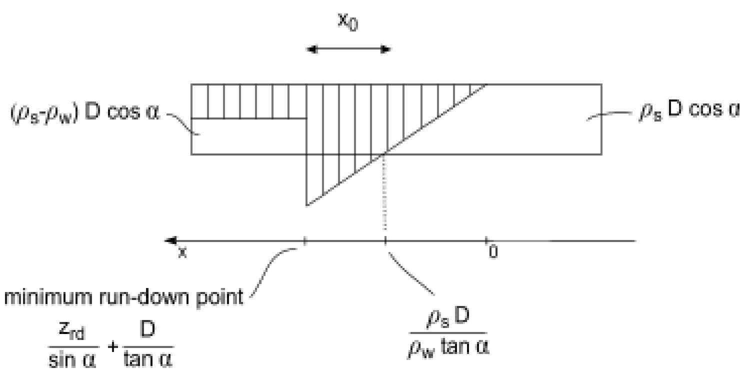


Figure A4.3
Water pressures and self weight acting on the revetment (Vrijling, 2000).



The total loading of the revetment is resultant of these hydrostatic pressures and the self weight of the structure, as shown in the figure below.

Figure A4.4
Resultant loading of the revetment (Vrijling, 2000).



The length over which the total loading becomes negative equals:

$$x_0 = \frac{z_{rd}}{\sin\alpha} - \frac{\Delta D}{\tan\alpha} = \frac{H_s \xi}{3\sin\alpha} - \frac{\Delta D}{\tan\alpha}$$

The stability of an infinitesimal part of the revetment at the minimum run-down point can be found by equalling the resultant water pressure with the weight of the revetment.

$$\rho_s g D \cos\alpha = \rho_w g x \sin\alpha \qquad \text{for} \qquad x = \frac{z_{rd}}{\sin\alpha} + \frac{D}{\tan\alpha}$$

After some algebra, this results in the following equilibrium:

$$H_s = \Delta D \frac{3\cos\alpha}{\xi}$$

Structural model for the PBA revetment

For use with plate type constructions such as the PBA-revetment, this method is adjusted. The PBA-revetment is a coherent structure with a flexural moment capacity. The flexural moment capacity counteracts the nett remaining overpressures when the self weight is insufficient to do so.

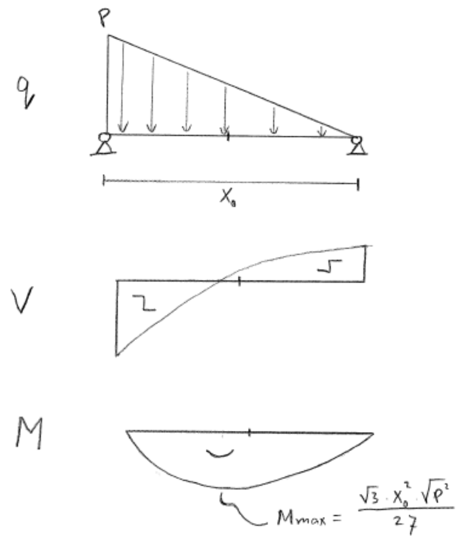
The Elastocoast revetment has the capability to deform (bend) under loading. The bending moment capacity for an Elastocoast revetment is constant and depends on the bending strength and the thickness of the cover layer:

$$M_{capacity}(x) = \sigma_{\max} \frac{1}{6} D^2$$

In which:

$M_{capacity}$ (x)	= moment capacity of PBA plate	[MN·m/m]
σ_{max}	= design bending strength	[MPa]
D	= PBA layer thickness	[m]

The flexural moment acting on the revetment as a result of the hydrostatic water pressure is estimated by schematizing the part of the revetment over which the total loading becomes negative in the form of a basic mechanical problem.



In this schematization the length x_0 is the length over which the total loading becomes negative, which was defined earlier as:

$$x_0 = \frac{H_s \xi}{3\sin\alpha} - \frac{\Delta D}{\tan\alpha}$$

In which:

x_0	= length over which total loading becomes negative	[m]
H_s	= significant wave height	[m]
ξ	= surf similarity parameter from peak wave period (ξ_p)	[-]
α	= slope angle of revetment	[°]
Δ	= relative density of (clogged) PBA layer ($= (\rho_s - \rho_w)/\rho_w \approx 1.1$)	[-]

P is defined as the maximum resultant pressure after subtraction of the self weight of the revetment:

$$P = \rho_w g \left(\frac{H_s \xi}{3 \sin \alpha} + \frac{D}{\tan \alpha} \right) \sin \alpha - \rho_s g D \cos \alpha$$

In which:

ρ_{pba}	= density of (clogged) PBA (≈2,100 kg/m^3)	[kg/m^3]
ρ_w	= density of (sea) water (=1025 kg/m^3)	[kg/m^3]

The maximum occurring flexural moment can then be defined as:

$$M_{\max} = \frac{\sqrt{3} x_0^2 \sqrt{P^2}}{27}$$

Finally, the maximum occurring moment M_{max} is equalled to the flexural moment capacity $M_{capacity}$:

$$M_{capacity} = M_{\max}$$

Solving this set of equations gives a value for the cover layer thickness that is required to achieve a stable situation. The solving can be done iteratively or with the help of mathematical software. For preliminary design the graph given below should be used.

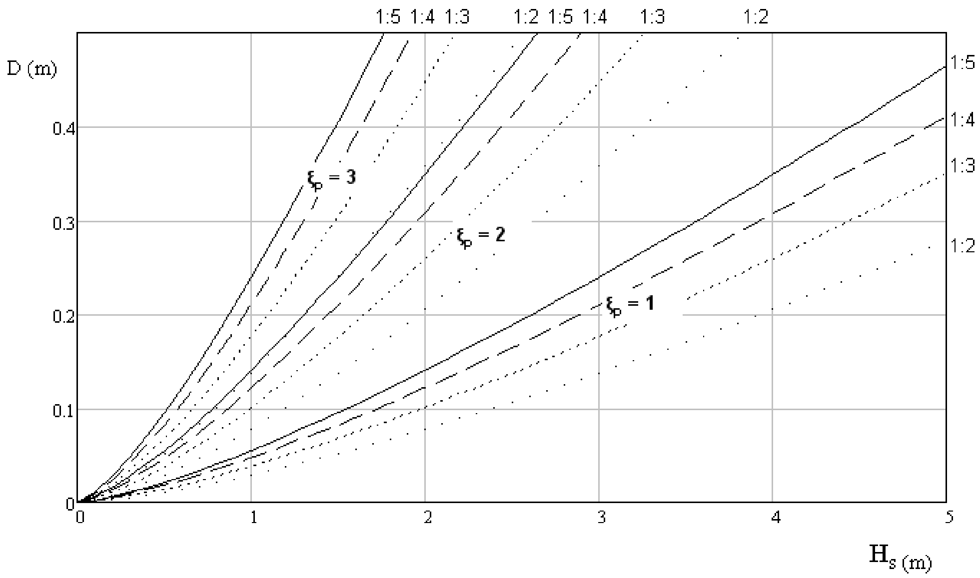
Design graph

The graph below can be used for design of a PBA-revetment for overpressures. It is assumed that the revetment is completely silted up and the flexural moment capacity is 0.62 MPa.

The graph clearly shows the influence of the wave breaker type, described by the surf similarity parameter ξ and the slope of the revetment.

Figure A4.5

Design for overpressures with flexural moment capacity of 0.62 MPa.



1:5 1:4 1:3
1:2 1:5 1:4
1:3
1:2
D (m)
0.4
0.3
0.2
0.1
0
0
1
2
3
4
5
ξp = 3
ξp = 2
ξp = 1
1:5
1:4
1:3
1:2
Hs (m)

ANNEX 5 Hydraulic conductivity of the PBA revetment

Introduction

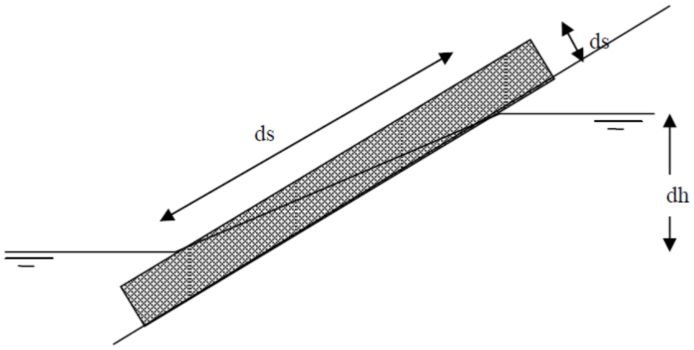
The hydraulic conductivity of a porous structure consisting of granular material depends mainly on the aggregate grading, which determines the porosity n and the size of the fine fractions of the material. Also, the conductivity is strongly dependent on the hydraulic gradient through the material. The hydraulic gradient is defined as [lit. 27]:

$$i = \frac{dh}{ds}$$

In which dh is the hydraulic head difference (m) and ds is the distance (m) over which this head difference works. The gradient i (-) can thus be seen as the slope of the groundwater level in the direction over which the distance is defined. In example, for parallel flow through the filter layer of a dike, the gradient is approximately the same as the dike slope $i \approx 0.25 - 0.50$. The perpendicular gradient through this layer is much larger; it depends on the head difference and the thickness of the cover layer and can be as much as: $i \approx 2.0 - 10$.

Figure A5.1

Definition sketch of gradients through a filter on a dike slope (Bijlsma, 2008b).



The flow rate q (m/s) through a medium can now be described with Darcy's law [lit. 27]:

$$q = k \cdot i$$

The hydraulic conductivity k (m/s) is a proportionality constant, which relates the amount of water that will flow through the medium with the hydraulic head over that medium.

Hydraulic conductivity of a PBA revetment

The hydraulic conductivity of granular material can be predicted theoretically with a model that is normally used for calculations with granular filter layers. The hydraulic conductivity is determined by the size of the fine fragments in the aggregate and the hydraulic gradient that exists over the thickness of the cover layer. The relationship between hydraulic conductivity k, gradient i and the size of the fine fraction D_{15} is given by (TAW, 2003) [lit. 21]:

$$k = \frac{-a_f + \sqrt{a_f^2 + 4b_f i}}{2b_f i}$$

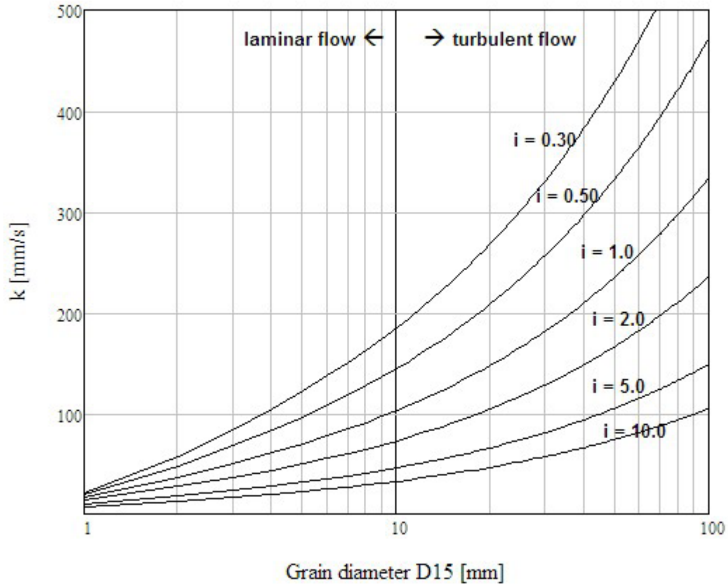
For the coefficients a_f and b_f the following values have been obtained from conductivity measurements:

$$a_f = 160 \frac{\nu}{g} \frac{(1-n)^2}{n^3 D_{15}^2} \qquad b_f = \frac{2.2}{g n^2 D_{15}}$$

In which ν=1.2×10^{6} m^2/s is the kinematical viscosity of water, n is the porosity (-) and D_{15} is a measure for the size of the fine fractions in the granular material. The relationship between hydraulic conductivity k, gradient i and the size of the fine fraction D_{15} is given in the figure below:

Figure A5.2

Hydraulic conductivity *k* related to grain size and hydraulic gradient over a filter with a porosity of *p*=0.5 (Bijlsma, 2008b).



Hydraulic conductivity of a silted-up structure

The hydraulic conductivity of the PBA revetment is strongly reduced if its open pore volume is silted-up with fine material such as sand. The hydraulic conductivity of a silted-up structure is determined by the open pore volume of the composite and the hydraulic conductivity of the fine material. It can be determined with a simple rule:

$$k_{silted\ structure} = p_{PBA} \cdot k_{filling\ material}$$

In which p_{PBA} is the porosity of the PBA revetment and $k_{filling\ material}$ is the hydraulic conductivity of the material that is contained in the open pore volume. The hydraulic conductivity for several soil types are given in the table below.

Table A5.1

Hydraulic conductivity of different soil types (Verruijt, 1999).

Soil type	*k* (m/s)
Gravel	$10^{-3} - 10^{-1}$
Sand	$10^{-6} - 10^{-3}$
Silt	$10^{-8} - 10^{-6}$
Clay	$10^{-10} - 10^{-8}$

ANNEX 6 Visual assessment

During and directly after construction

Discoloration as a result of moist rocks

Coating becomes murky and light coloured instead of transparent because tiny air bubbles are formed by reaction of one of the components with water. These bubbles are entrapped between the rock and the coating and weaken the bonding.


Figure A461

Discoloration.

Note: presence of dust can also cause discoloration, but does not weaken the strength bonding! Colour depends on the colour of the dust, which is entrapped in the coating.

Stripping as a result of moist rocks

Because of PU does not stick to a moist surface, the PU coating is separated from the rock. Bonding between the rocks can be weakened.

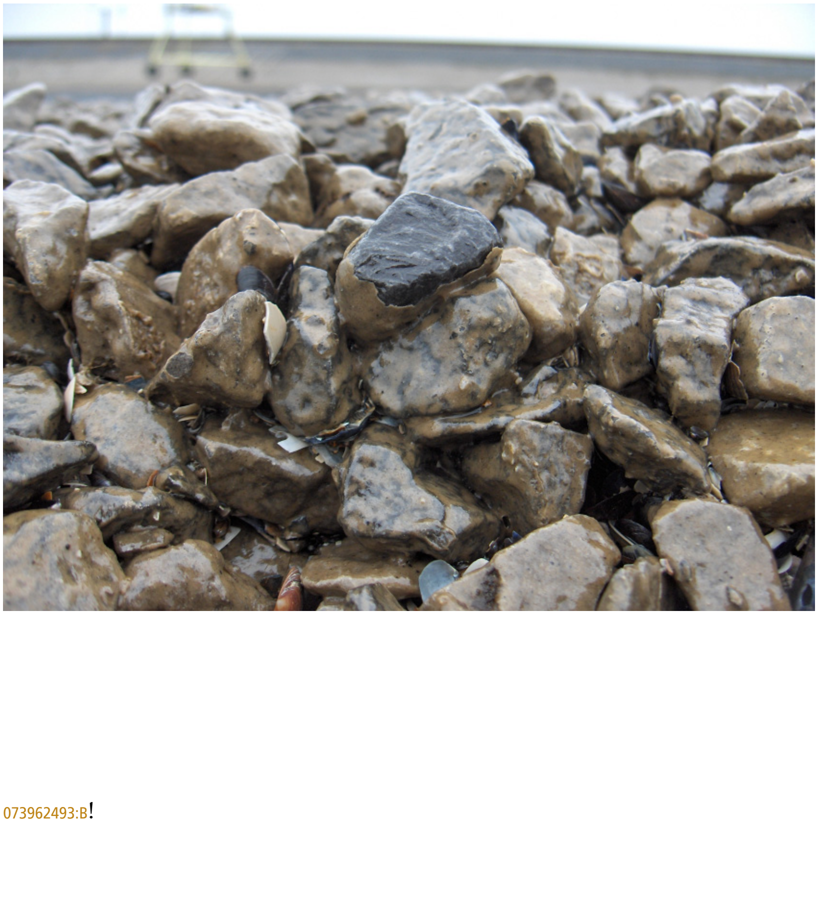


Figure A6.2

Stripping.

Rain during curing process

When it starts raining after the PU-components have been mixed with the aggregate, but the PU has not fully cured yet, small pits may be created in the coating of the top layer. This does not influence the strength of the bonding between rocks.

Figure A6.3

Rain craters.

Complete lack of bonding as a result of wet rocks

If the surface of the rock is wet before and during mixing with the PU, the hydrophobic behaviour of the unhardened PU prevents bonding with the rock.

Figure A6.4

Lack of bonding.

During the lifetime of the structure

Micro scale damage

Individual rocks may break or may be removed from the revetment by hydraulic loads or other loads, such as floating debris or vandalism. This is not a problem if the damage is not concentrated.

Figure A6.5

Micro scale damage – breach of rock

Figure A6.6

Micro scale damage – breach of PU-connection

Surface weathering

By the erosive forces of water and sand and other materials scraping over the revetment, the coating may erode from the rocks. This only occurs on the surface and does not weaken the structure.

Figure A6.7

Surface weathering.

Macro scale – clogging

Depending on the sediment concentration of the water and the position and orientation of the structure, fine materials can be trapped in the open pores of the PBA revetment. This negatively influences the hydraulic conductivity of the revetment.

Figure A6.8

Macro scale – clogging with shells, sand and organic material. Formation of puddles on horizontal parts of the revetment.

Failure during extreme conditions

Breakage

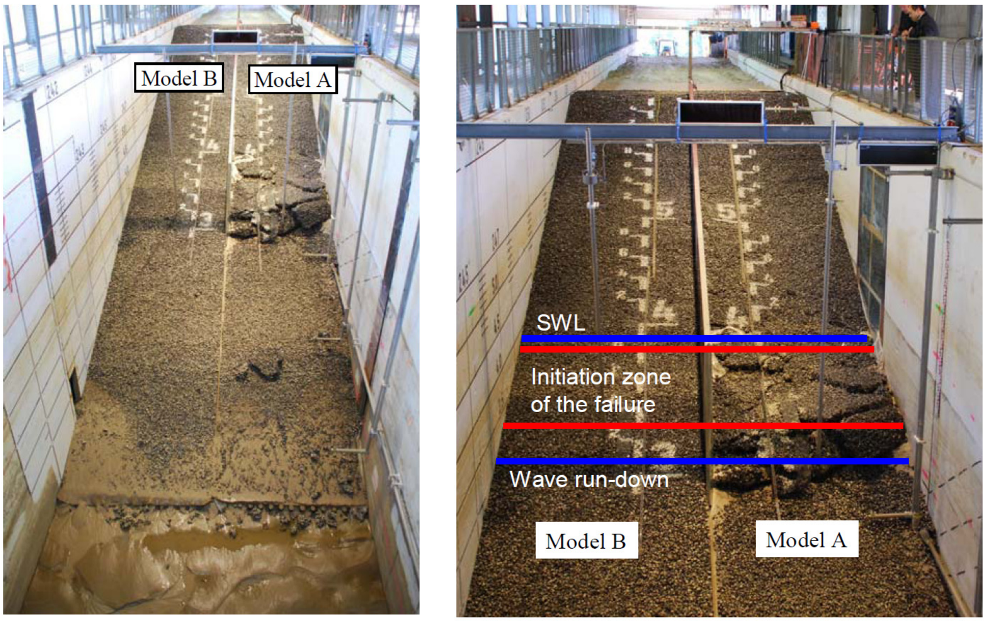
Breakage of the cover layer occurs when the flexural strength is exceeded by occurring flexural stresses as a result of (hydraulic loads). This can cause significant gaps in the revetment. Before repairing these gaps it must be checked whether the underlying filter layer is still intact.

Figure A6.9

Breakage (LWI 2010).

Subsoil liquefaction

Subsoil liquefaction leads to an upward displacement of the revetment from the dike slope. This movement can result in failure by breakage. The damage zone by soil liquefaction is concentrated just below still water level.



(a) Overall view

(b) Detailed view

Figure A6.10

Soil liquefaction (LWI 2010).

COLOPHON

POLYURETHANE BONDED AGGREGATE REVETMENTS

DESIGN MANUAL

CLIENT:

BASF POLYURETHANES GMBH

STATUS:

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