Computation of a coastal protection, using classical method, the PIANC-method or a full probabilistic approach?

Henk Jan Verhagen
Hydraulic Engineering Section, Delft University of Technology, The Netherlands

Abstract

In a classical design approach to breakwaters a design wave height is determined, and filled in into a design formula. Some undefined safety is added. In the method using partial safety coefficients (as developed by PIANC [1992] and recently also adopted by the Coastal Engineering Manual of the US Army Corps of Engineers [CEM [2003]]) the degree of safety is formalised, and safety factors are given. However, because this method is still rather complicated, a Monte Carlo probabilistic approach allows the designer more flexibility and is also able to use predefined safety values.

Design principles

A fundamental starting point for designers is that the strength of a construction should be such that the probability of failure becomes negligibly small. In mathematical terms this means that the probability that \( Z < 0 \) (\( Z \) is the difference between strength and load, or \( Z = R - S \)) should be small.

In a very simple design this problem can be solved easily. For example if one needs to design the cable in a crane, the design force in the cable \( F \) is equal to the design mass to be lifted, multiplied with acceleration of gravity. The strength of the cable depends on the intrinsic strength (\( \sigma \)) of the cable material, multiplied with the cross sectional area \( A \) of the cable:

\[
\text{strength:} \quad R = A \cdot \sigma \\
\text{Load:} \quad S = M \cdot g \\
Z = R - S = A \sigma - Mg
\]

For critical conditions (brink of failure) \( Z = 0 \). The critical cross sectional area (which is in fact the design parameter) is

\[
A_{\text{crit}} = \frac{Mg}{\sigma}
\]

\( M \) is the mass of the heaviest load to be lifted (design load). This is a clear input parameter, it is defined by the client; \( \sigma \) is provided by the manufacturer of the cable and \( g \) is always known. Because there is always an uncertainty, some safety factor \( \gamma \) is added:
The magnitude of $\gamma$ is usually given in professional codes and standards; if not, it is usually based on experience (in case of breakwater design PIANC has issued values of $\gamma$ to be used in the design, as will be mentioned later).

However, in the design of coastal structures there is a complicating factor. Take for example the old-fashioned Hudson formula for breakwater armour.

$$M = \frac{\rho_s H_s^3}{K_D \Delta^3 \cot \alpha}$$

In this equation there is one load parameter ($H_s$), and four strength parameters ($A$, relative density; $M$, mass of armour unit; $K_D$, strength coefficient; $\alpha$, slope). The strength parameters can be determined easily and relatively accurate. Each of these parameters is Gaussian distributed with a relatively small standard deviation. So, at the strength side of the equation, the problem is very comparable to the cable example mentioned before.

But for the load parameter ($H_s$) one cannot determine an “average” value. It has to be a significant wave that does not occur too often. And related to the waveheight is the wave period (which is usually also present in the more advanced design equations). It means that the definition of our “design wave” or “design storm” is a key problem in our design. Because in our design we use the significant wave during the design storm, we will use the symbol $H_{ss}$ for this purpose.

**The classic approach**

The classic way of computing the required block size is using the design formula, using a design wave height with an exceedance of $P_f$ during the lifetime of the structure. For example a lifetime of 50 years and a probability of failure of 20% during lifetime makes gives the following exceedance:

$$f = -\frac{1}{t_L} \ln(1-p) = -\frac{1}{50} \ln(1-0.2) = 4.5 \cdot 10^{-3} = \frac{1}{225}$$

This implies that a storm with a probability of exceedance of $1/225 = (4.4 \cdot 10^{-3})$ has to be used. The magnitude of such a storm can be derived from a statistical analysis of storm data (see for example KAMPHUIJS [2000]). For the coast of the Netherlands this design storm is characterised by $H_{ss} = 8.64$ m [VERHAGEN, 2003].

Suppose we design a breakwater at Scheveningen, a port along the Dutch coast, using cubes. The design formula for cubes, as given by Van der Meer [D’ANGREMOND AND VAN ROODE, 2001], is:

$$\frac{H_{ss\text{-design}}}{\Delta d_n} = \left(6.7 \frac{N_{\text{od}}^{0.4}}{N^{0.3}} + 1.0\right) s_m^{-0.1}$$
For $N_{od}$ a value of 0.5 is recommended by Van der Meer. The wave analysis of the waves in front of the Dutch coast has shown a wave steepness of 5.6%. There are approximately 4000 waves in a storm. For cubes with heavy aggregate (basalt split) one may use a concrete density of 2800 kg/m$^3$, which results in a value of $\Delta = 1.75$.

Filling in these values in the above design formula results in a $d_n$ of 2.61 m, or a block weight of 50 ton. In case one applies the Hudson formula with a $K_D$ value of 5 (head) and a slope of 1:1.5, one gets a $d_n$ of 2.5 m, and a weight of 45 ton.

Because the depth is limited near the mouth of the harbour of Scheveningen 6 m below MSL (i.e. 9.5 m below design level), one may assume that waves never can become larger than $9.5/2 = 4.75$ m. In that case the required block weight is only 8.3 ton according to Van der Meer, and 7.5 ton according to Hudson.

In order to be at a safe side, one may use a higher breakerindex ($\gamma_b = 0.7$). Then the design waveheight becomes 6.8 m, which results with Van der Meer to a block of 24 ton, and with Hudson to a block of 22 ton.

**The method of Partial Coefficients**

The method of partial coefficients is worked out in PIANC [1992]. In this method safety coefficients are added to the design formula. There are safety coefficients for load and for strength.

**The partial safety coefficients for load**

The first step is also in this case the determination of the return period of the design wave. As in the classic approach, for a design life of 50 years and a probability of failure during the design life, this leads to a return interval of 225 years.

For determination of the partial safety coefficient for the load, one has to start with an extreme value distribution for the storms, for example the Weibull distribution. For this purpose the distribution is given as:

$$Q_{tl} = \left\{1 - \exp\left[-\left(\frac{H_{ss} - \gamma}{\beta}\right)^\alpha\right]\right\}^{N_{s}t_{L}}$$

In this equation $\alpha$ and $\beta$ are the parameters of the Weibull distribution, and $\gamma$ is the threshold value. $N_s$ is the number of observations per year. This equation can be reworked to

$$H_{ss} = \gamma + \beta \left[-\ln\left(1 - \exp\left(\frac{\ln Q_{tl}}{N_s t_{L}}\right)^\gamma\right)\right]^{\gamma/\alpha}$$

For $Q_{tl}$ one should enter the non exceedance probability for $N_s t_{L}$ events during lifetime. This means that $Q_{tl} = P^{t_{L}}_{f\text{-lifetime}}$. For a design life of 50 years $Q_{tl}$ is 0.364, for 100 years $Q_{tl}$ is 0.366.

During the design of the breakwater of Scheveningen (around 1970), only the Hudson formula was available; the design of this breakwater is therefore based on this formula and verified with model tests. The applied blocks in Scheveningen have a weight of 25 ton.
The values of the parameters follow from a statistical analysis of wave data. For the Dutch coast [Verhagen 2003], these data are: $\alpha = 1.24; \beta = 1.17; \gamma = 1.22; N_s = 87.3$. For a practical case this leads to the following values of $H_{ss}$ for $t = t_L (50, 100)$, $H_{ss}$ for $t = 3 t_L (150, 300)$, $H_{ss}$ for $t = t_{20\%} (225, 450)$.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Method of determination</th>
<th>Typical value for $\alpha'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height</td>
<td>Accelerometer buoy, pressure cell, vertical radar</td>
<td>0.05 – 0.1</td>
</tr>
<tr>
<td>Significant wave height offshore</td>
<td>Horizontal radar</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>Hindcast, numerical model</td>
<td>0.1 – 0.2</td>
</tr>
<tr>
<td></td>
<td>Hindcast, SMB method</td>
<td>0.15-0.2</td>
</tr>
<tr>
<td></td>
<td>Visual observation (Global wave statistics)</td>
<td>0.2</td>
</tr>
<tr>
<td>$H_{ss}$ nearshore determined from offshore $H_{ss}$ taking into account typical nearshore effects (refraction, shoaling, breaking)</td>
<td>Numerical models</td>
<td>0.1-0.2</td>
</tr>
<tr>
<td></td>
<td>Manual calculation</td>
<td>0.15-0.35</td>
</tr>
</tbody>
</table>

In this equation $P_f$ is the allowable failure during lifetime (not to be mistaken with $P_f$ _lifetime_, which has been defined that the probability of exceedance of the “once in $t_L$-years storm” during the life time $t_L$. The standard deviation $\sigma'_{Qil}$ is given in the mentioned PIANC report (copied in table 1) as a function of the type of observations available, see table. The data for the Dutch coast are based on accurate observations, so a value of $\sigma'_{Qil} = 0.05$ is realistic.

The safety coefficient is given as:

$$\gamma_{H_{ss}} = H_{ss}' \left(1 - \frac{H_{ss}}{H_{ss}^{ul}}\right)^{1.5} + \frac{0.05}{\sqrt{P_f N}}$$

Table 1: Typical variation coefficients for sea state parameters [from PIANC 1992]
$P_f$ was 20%, $N$ is the number of “storms” in the PoT-analysis, for this example it is 1746. $k_\alpha = 0.027$ and $k_\beta = 38$ (see table 2). This leads to:

$$\gamma_{H_{ss}} = \frac{8.64}{7.71} + 0.2 \left(1 + \frac{1.389}{7.71}\right) + \frac{0.05}{\sqrt{0.2 \cdot 1746}} = 1.13$$

The safety coefficient consists of three parts. The first part gives the correct partial safety coefficient, provided no statistical uncertainty and measurement errors related to $H_{ss}$ are present. The middle term signifies the measurement errors and the short-term variability related to the wave data. The last term signifies the statistical uncertainty of the estimated extreme distribution of $H_{ss}$. The statistical uncertainty treated in this way depends on the total number of wave data $N$, but not on the length of the observation period.

If extreme wave statistics are not based on $N$ wave data, but e.g. on estimates of $H_{ss}$ from information about water level variations in shallow water, then the last term disappears and instead the value chosen for $\sigma'$ must account for the inherent uncertainty.

In the table below the value of $\sigma'$ and $N$ is changed to the values for simple manual calculations and a shorter dataset. It is clear from this table that the effect of the length of a dataset is less important than accurate observations.

<table>
<thead>
<tr>
<th></th>
<th>base example</th>
<th>use $\sigma' = 0.35$</th>
<th>use $N = 10$ storms</th>
<th>Use $\sigma' = 0.35$ and $N = 10$</th>
</tr>
</thead>
<tbody>
<tr>
<td>basic safety coefficient</td>
<td>100%</td>
<td>87%</td>
<td>99%</td>
<td>84%</td>
</tr>
<tr>
<td>measurement errors</td>
<td>0%</td>
<td>13%</td>
<td>0%</td>
<td>13%</td>
</tr>
<tr>
<td>statistical uncertainty</td>
<td>0%</td>
<td>0%</td>
<td>1%</td>
<td>3%</td>
</tr>
</tbody>
</table>

The partial safety coefficient for strength

The safety coefficient for the strength can be calculated using

$$\gamma_z = 1 - (k_\alpha \ln k_\beta P_f)$$

in which $k_\alpha$ and $k_\beta$ are coefficients determined by optimisation and given in the PIANC manual [1992]. These values are copied in table 2. The value of $P_f$ is the allowable probability of failure during lifetime.

<table>
<thead>
<tr>
<th>Formula, type of construction</th>
<th>$k_\alpha$</th>
<th>$k_\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hudson, rock</td>
<td>0.036</td>
<td>151</td>
</tr>
<tr>
<td>Van der Meer, rock, plunging waves</td>
<td>0.027</td>
<td>38</td>
</tr>
<tr>
<td>Van der Meer, rock, surging waves</td>
<td>0.031</td>
<td>38</td>
</tr>
<tr>
<td>Van der Meer, Tetrapods</td>
<td>0.026</td>
<td>38</td>
</tr>
<tr>
<td>Van der Meer, Cubes</td>
<td>0.026</td>
<td>38</td>
</tr>
<tr>
<td>Van der Meer, Accropodes</td>
<td>0.015</td>
<td>33</td>
</tr>
<tr>
<td>Van der Meer, rock, low crested</td>
<td>0.035</td>
<td>42</td>
</tr>
<tr>
<td>Van der meer, rock, berm</td>
<td>0.087</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 2: coefficients used to determine the partial safety factor $\gamma_z$

In the Coastal Engineering Manual the same approach is followed, however in that manual the values of $\gamma_{H}$ and $\gamma_z$ are directly given in tables as a function of $P_f$ and $\sigma'$. For cubes, one can apply the Van der Meer equation:

$$\frac{1}{\gamma_z} \left(6.7 \frac{N_{od}^{0.4}}{N^{0.30}} + 1.0\right) s_m^{-0.3} \Delta d_n \geq \gamma_{H_{od}} H_{od}$$

Using the data from the wave climate as given before, for the harbour of Scheveningen one may fill in the values of $N_{od} = 0.51$, $N = 4000$, $\Delta = 1.75$, $s_m = 5.6\%$. 

In this equation, $s_m$ is the significant wave height, $N_{od}$ is the number of “storms” in the PoT-analysis, $N$ is the number of wave data, $\Delta$ is the wave height variation, and $s_m$ is the significant wave height.

The safety coefficient consists of three parts. The first part gives the correct partial safety coefficient, provided no statistical uncertainty and measurement errors related to $H_{ss}$ are present. The middle term signifies the measurement errors and the short-term variability related to the wave data. The last term signifies the statistical uncertainty of the estimated extreme distribution of $H_{ss}$. The statistical uncertainty treated in this way depends on the total number of wave data $N$, but not on the length of the observation period.

If extreme wave statistics are not based on $N$ wave data, but e.g. on estimates of $H_{ss}$ from information about water level variations in shallow water, then the last term disappears and instead the value chosen for $\sigma'$ must account for the inherent uncertainty.

In the table below the value of $\sigma'$ and $N$ is changed to the values for simple manual calculations and a shorter dataset. It is clear from this table that the effect of the length of a dataset is less important than accurate observations.
This gives a \( d_n = 2.75 \), or 58 tons.

Realise that in the above equation for the wave height the \( H_{ss} \) is used which has a probability of exceedance of once in the lifetime of the structure, i.e. the “once in 50 years storm” (7.71 m). This wave height is multiplied with \( \gamma_H \) (1.13), resulting in a total wave height of 8.71 m.

Traditionally one should use a wave height with a probability of 20% during the lifetime of 50 years. This wave has a yearly exceedance of the “once in 225 years wave”, and is 8.64 m (in the PIANC guidelines it is called \( H_{ss \text{ npf}} \)). It is very comparable to the found value of 8.71 m.

PIANC does not give any guidance for shallow water conditions. This implies that the approach in fact cannot be applied for the port of Scheveningen.

**Probabilistic approach**

Instead of the method with partial safety coefficients, one may also apply a full probabilistic computation, either on level 2 or level 3. For level 2 one may apply the FORM method, for level 3 one may apply the Monte-Carlo method. In the examples below the computer program VaP from ETH-Zürich is applied.

The first step is to rewrite the design equation as a Z-function. For cubes the Z-function is:

\[
Z = \left( A \frac{N^{0.4}}{N^{0.30}} + 1.0 \right) s_m^{-0.1} \Delta d_n - H_{ss}^{\text{npf}}
\]

(note: in the VaP-programme the variable Z is called G)

In this equation 7 variables are used. In a probabilistic approach one has to determine for each parameter the type of distribution.

The constant 6.7 from the Van der Meer equation is here replaced by a coefficient \( A \) with a normal distribution. This coefficient has a mean of 6.7 and a standard deviation describing the accuracy of the equation itself. According to Van der Meer the standard deviation of the coefficient \( A \) is approximately 10% of its value.

Wave steepness is assumed to be Normal distributed. The average steepness as well as the standard deviation of the steepness can be calculated from the dataset with wave observations [VERHAGEN, 2003]. If one considers only the heavier storms (i.e. storms with a threshold of e.g. 4.5 m), the average steepness is 0.058, with a standard deviation of 0.0025.

We have then 56 storms in 20 years (i.e. 2.8 storms per year) with an average duration of 7.2 hours. This means that the average period is 6.9 seconds, and that there are consequently 3700 waves in a storm. However, the duration of the storms varies quite a lot (it may go up to 20), which means that we have 10000 waves. This means that \( N \) will have a large standard deviation. One can fit the calculated waves in each storm and fit this to a distribution. However, the effect of the number of waves is not that high so one may
assume a Lognormal distribution (on cannot use a Normal distribution, because that the number of waves may become negative when using high standard deviations).

Because the standard deviation in the block size is so small, this parameter can be considered as a deterministic value. The same is true for the acceptable damage level. Because this is a target value, one should also use here a deterministic value.

$H_{ss}$ has a Weibull or Gumbel distribution. For the wave-height one may enter for example a Weibull distribution, using the values of $\alpha$, $\beta$ and $\gamma$ as determined before. In VaP one can enter the values either as moments or as parameters. It is more convenient to use parameters in this example. This results into the following input table:

<table>
<thead>
<tr>
<th>parameter</th>
<th>type</th>
<th>mean</th>
<th>St. deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Normal</td>
<td>6.7</td>
<td>0.67</td>
</tr>
<tr>
<td>$N_{nod}$</td>
<td>Deterministic</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>$N$</td>
<td>Lognormal</td>
<td>3000</td>
<td>3000</td>
</tr>
<tr>
<td>$s_m$</td>
<td>Normal</td>
<td>0.058</td>
<td>0.025</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>Normal</td>
<td>1.75</td>
<td>0.05</td>
</tr>
<tr>
<td>$d_n$</td>
<td>Deterministic</td>
<td>2.4</td>
<td>-</td>
</tr>
</tbody>
</table>

Using VaP, one can compute the probability of $(Z<0)$, which is the probability of failure. The target probability is $1/225 = 0.0044$ per year. However, VaP gives the probability per event. Because we have 87.3 storms per year, the target probability per event becomes $0.0044 / 87.3 = 50*10^{-6}$. The weight of a cube of 2.4 m is 43 ton. For this example VaP gives, using the FORM method a probability of failure of $45*10^{-6}$, which is quite near to the target value. A Monte-Carlo computation gives a probability of failure of $70*10^{-6}$, so quite comparable. This computation is based on the fact that we have defined $N_{nod} = 1$ start of failure. However, this is quite some damage. Lowering the value of $N_{nod}$ to 0.5 means that we have to increase the block size to 2.65 m (51 ton) in order to obtain the same probability of failure. Using a $N_{nod}$ of 0 means an increase to 3.80 m (151 ton).

In the above calculations the uncertainty in the determination of the parameters of the Weibull distribution has not taken into consideration. In fact this is not correct, but it is seldom used. The mathematical correct way is to consider $\alpha$, $\beta$ and $\gamma$ as stochastic para-

---

2 Because VaP uses a somewhat different notation, one should use in the programme the following conversion: parm1 = $u = \beta + \gamma$; parm2 = $k = \alpha$; parm3 = $\varepsilon = \gamma$
meters with a means and a standard deviation. One can determine these values directly from the dataset, but it is not possible to apply these values directly in a probabilistic computation. Therefore in practice this problem is solved by introducing an extra variable \( M \). This variable has a mean value of 1 and a standard deviation which expresses the variation in the prediction of \( H_{ss} \) using a Weibull or Gumbel distribution. The Van der Meer formula then becomes:

\[
Z = \left( A - \frac{N^{0.4}}{N^{0.30} + 1.0} \right) s_n^{-0.1} \Delta d_n - MH_{ss}^M
\]

A problem is that the standard deviation of \( M \) depends on the value of \( H_{ss} \). On can determine this value using the design value for \( H_{ss} \).

The standard deviation is given as:

\[
\sigma'_{M} = \frac{\sigma_M}{H_{ss-design}}
\]

For \( \sigma_M \) an expression has been derived by [GODA, 2000]. He has derived that \( \sigma_M = \sigma_z \cdot \sigma_x \) in which \( \sigma_x \) is the standard deviation of all \( H_{ss} \) values in the basic dataset and \( \sigma_z \) is defined by:

\[
\sigma_z = \left[ 1.0 + a(y - c)^2 \right]^{1/2} \sqrt{N}
\]

\[
a = a_1 \exp[a_2 N^{-1.3}]
\]

in which the coefficients are given by:

<table>
<thead>
<tr>
<th>distribution</th>
<th>( a_1 )</th>
<th>( a_2 )</th>
<th>( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gumbel</td>
<td>0.64</td>
<td>9.0</td>
<td>0</td>
</tr>
<tr>
<td>Weibull, ( \alpha = 0.75 )</td>
<td>1.65</td>
<td>11.4</td>
<td>0</td>
</tr>
<tr>
<td>Weibull, ( \alpha = 1.0 )</td>
<td>1.92</td>
<td>11.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Weibull, ( \alpha = 1.4 )</td>
<td>2.05</td>
<td>11.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Weibull, ( \alpha = 2.0 )</td>
<td>2.24</td>
<td>11.4</td>
<td>0.5</td>
</tr>
</tbody>
</table>

\( y \) is the reduced variate for the design value, our design value is a 1/225 wave, so \( y = [\ln(87.3*22)]^{1/1.24} = 6.34 \).

In the example used, we have a Weibull with \( \alpha = 1.24 \); because this is not in the table, an interpolation is needed. For the example \( a_1 = 2.0, a_2 = 11.4 \) and \( c = 0.35 \) is used.

This results in a value of \( \sigma_z = 0.024 \). Given a \( \sigma_x \) of 6.85 (follows from dataset), it means:

\[
\sigma'_{M} = \frac{\sigma_M}{H_{ss-design}} = \frac{\sigma_z \cdot \sigma_x}{H_{ss-design}} = \frac{6.85 \cdot 0.024}{8.64} = 0.02
\]

Introducing this in VaP gives a probability of failure of 157*10^-6. In order to bring this back to the required 50*10^-6 we have to increase the cube size to 2.42. So, this can be neglected. However, if our dataset would have been considerably smaller (for example only 100 storms), this would change the value of \( \sigma_z \) to 0.175 and consequently \( \sigma'_{M} \) to 0.14. In order to get a probability of failure in the order of the required 50*10^-6 we have to increase the cube size to 2.70 (from 43 to 54 ton) for the case with \( N_{od} = 1 \). This shows that the size of the dataset has a considerable impact on the required block size.
Probabilistic calculation in case of a shallow foreshore

All statistical applications do not consider physical limitations. Amongst others this means that in deep water, statistics will give very high waves with very low probability. But in shallow water these high waves cannot exist at all. In general the depth limits the significant wave height in shallow water. So: \( H_s = \gamma_b h \) in which the breaker index \( \gamma_b \) has a value in the order of 0.6. For individual waves \( \gamma_b \) may have values up to 0.78, but for the significant wave height this value is much lower, sometimes even up to 0.5.

For foreshores with a gentle slope one may assume that \( \gamma_b \) may have an average value of 0.55, with a standard deviation of 0.05. The waterdepth in this equation is the total depth, i.e. the depth below mean sea level + the rise of waterlevel due to tide and storm surge. With this information, one can rewrite the Van der Meer equation for cubes to:

\[
Z = \left( A \frac{N_0^{0.4}}{N_0^{0.30}} + 1.0 \right) s_n^{0.1} \Delta d_n - \gamma_b (h_{\text{surge}} + h_{\text{depth}})
\]

The waterdepth below mean sea level has a Normal distribution, but the standard deviation in this parameter is usually so low, that it can be considered as a deterministic value. Of course in case one expects large bed fluctuations, one may also enter this value as a stochastic parameter with a Normal distribution. The surge has an extreme value distribution. For this value one can use a Gumbel distribution.

As an example the harbour of Scheveningen is used again. The waterdepth in front of the breakwater in Scheveningen is 6 m below mean sea level. Long term waterlevels are available from Hook of Holland, which are very identical to Scheveningen. The exceedance can be given by:

\[
Q = 1 - \exp\left\{ -\exp\left( \frac{h_{\text{surge}} - \gamma}{\beta} \right) \right\}
\]

In this equation is \( \gamma \) the intercept at the 10\(^{0}\)-line, and \( \beta \) is the slope parameter. From the diagram one can derive that \( \gamma \) equals 2.3 and that the slope \( \beta \) is 0.30. However VaP uses the parameters \( u (= \gamma) \) and \( \alpha (=1/\beta = 3.289) \).

The equation is based on the maximum surges per year, so the exceedance is also per year, and not per storm. This implies that in this case the target probability of failure is 1/225 = 0.0044.
These values can be entered in an adapted equation in VaP, see table. This leads to $d_n$ of 1.55 m, or a block weight of 11 ton (using $N_{od} = 1$). In principle one can add also in this case the statistical uncertainty by adding a factor $M$ in front of the surge height in the equation. However, because of the long dataset and the limited extrapolation, this uncertainty is very small and may be neglected.

Changing the standard deviation in $\gamma$ from 0.05 to 0.1 has a considerable effect. To obtain the same target probability of failure, one has to increase the blocks to $d_n = 1.8$ m (16 ton).

An error made in this computation is that is assumed that the deep water wave steepness remains the same after breaking. This is probably not the case. The higher waves will break (usually as spilling breakers) which decreases their height considerably, but usually not the period. As a consequence the wave steepness in broken waves is much less than the wave steepness at sea. Because in the Van der Meer formula for cubes a low steepness gives smaller cubes than a high steepness, neglecting the change in steepness is a conservative approach. Realise that in case of rip-rap the opposite is true.

**Conclusions**

Because in most cases the classical approach cannot guarantee a pre-defined probability of failure, it usually results in overdimensioning the structure. A method as defined by PIANC and also adopted by the CEM uses standardised probabilities. However, the computations are not really simple, and because the additional work needed for a full probabilistic computation using a Monte Carlo method is rather limited, one should consider to use the last method, especially because this method is much more flexible than the method of using partial safety coefficients.

**References**


PIANC [1992] Analysis of rubble mound breakwaters; report of working group 12 (PTC II)

VERHAGEN, H.J. [2003] Analysis of 60000 wave observations between 1979 and 1999 from Meetpost Noordwijk for the determination of a design storm, Delft University teaching paper