DESIGN OF ROCK STRUCTURE BREAKWATER WITH LARGE OVERTOPPING ALLOWED/LOW CREST STRUCTURE

(Case Study on Madras Port, India)

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ABSTRACT

Tugas utama dari proyek ini adalah untuk mendesain "kepala" dari breakwater (pemecah gelombang) sebelah utara bersama-sama dengan bagian "belalai" breakwater. Tujuan dari perencanaan ini adalah untuk mendapatkan breakwater yang dapat bertahan pada kondisi gelombang harian dan ekstrim (badai). Sedangkan tujuan lainnya adalah untuk menyediakan alur masuk pelayaran yang tenang ke kolam pelabuhan. Tetapi untuk kondisi tertentu, kapal-kapal tidak dapat berlabuh di pelabuhan karena kriteria dari akhir perencanaan adalah struktur yang dapat/menerima limpasan (overtopping) yang cukup besar. Kondisi ini didefinisikan sebagai maksimum gelombang transmisi adalah kurang dari 1 m dalam periode ulang kejadian 50 tahunan.

Keywords *: Rock Breakwater, Low Creste Strukture*

INTRODUCTION

The emphasis of design is the structural performance The design shall account for all aspects like capital and maintenance cost, material availability operational requirements and construction methodology. Another important think is the project has to pay attention on environmental and coastal morphology, like the possibility of sediment transport due to change of coastal line.

A breakwater with rock structure in rubble mound type is chosen based on folowing reason:

1. The location of breakwater is not at deep water, so rubble mound type breakwater with rock material can be used.
2. The rubble mound breakwater is not more collapse directly than vertical breakwater caused by land settlement.
3. The rubble mound breakwater can be maintained with reposition the flattening material or replacement material that flattening out from the structure with new ones.
4. The material of rock can absorb of wave energy, so reflection of rubble mound breakwater is smaller than vertical structure breakwater.
5. The quarry is abudance of durable rock.
6. It is economically feasible to produce and deliver to the site a sufficient quantity of rock.
7. During construction, the using of land equipment is very enable and

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does not usually require large-scale construction equipment such as work barges.
8. Less enviromental impact due to smaller reflected waves and more water exchange.
9. The slope of rubnle mound bre-akwater provides and suitable place for sea life to live.

AVAILABLE DATA

Bathymetry

Depth contours are relatively parallel with shoreline. The position of north bre-akwater extends until 11 m depth. The foreshore is of a relatively shallow slope of 1:50 (0.20).

Wave Climate

The monsoon seasons are distinguished at Madras-port, a north-east monsoon in the period of mid October to mid Janua-ry and a south-west monsoon in the period of mid April to mid August. This monsoon period coincides with the oc-currence of tropical depressions and cy-clones. On average cyclones take place once in every two years.

Daily of wave climate is given in pro-bability that resultant of sea and swell (in wave height and period class). Ba-sed on this data, the crest level of wor-king road can be determined.

Others wave climate data are (1) the tracks of hindcast storms in period 1980-1993, (2) the wave conditions at deep water for each of the grid points and for various return periods, (3) the relationship between significant wave height, Hs, and the mean wave period, Trn and (4) the extreme wave climate is used to determine parameter design to calculate dimension of structure.

Tide

The sea on Madras port location has a moderate semi-diurnal tide with a tidal range of 1.1 m for spring tide and 0 4 m for neap tide.

Water Properties

Monthly mean seawater temperatures vary from 26.5°C to 29°C. Off the coast, the maximum salinity amounts to 34 ppt. For temperature 26°C and 28°C in 34 ppt salinity condition, the sigma values are 22.42 and 21.78. Thus, these give an average water density of 1022 kg/m^3.

Soil Conditions

A field investigation has been done on soil conditions. At the head position of the north breakwater the layer of soft clay between -10.5 m - 15 m. This me-ans the thick of soft claY is 4.5 m and this layer must be replaced by sand to avoid of slip failures.

BOUNDARY CONDITION

Design Water Level

The sea has a moderate semi-diurnal tide. The variation of water level based on tidal variation can be shown at below table.

Table 1. The Tidal variation at loaction

Water Level	Elevation (m CD)
MHWS (Mean High Water Spring)	+ 1.10
MHW (Mean High Water)	+ 0.80
MSL (Mean Sea Level)	+ 0.65
MLW (Mean Low Water)	+ 0.40
MLWS (Mean Low Water Spring)	+ 0.10

Note : *CD is Chart Datum*

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Table 1 shows the calculation to determine of magnitude design water level caused by the effects of tidal variations and its related parameters. To calculate the design water level, MHWS is used as water level base with considered some parameters (see Table 2)

Table 2. The calculation of contribution to design water level (for structure)

Parameters	Value
Bottom level	-11.0 m CD
Astronomical tide	+ 1.10 m CD
Seasonal variations	0.10 m
Wind set-up/storm surge	0.23 m
Wave set-up	0.03 m
Barometric pressure	0.20 m
The design water level	**+ 1.66 m CD**
Design Water Depth	**11 + 1.66 = 12.66 m**

For daily working road, the mean high water level (MHW) and seasonal variatrons are considered. These parameters are grven in Table 3.

Table 3. The parameters of the design water depth for daily working road

Parameters	Value
Mean sea level	+ 0.80 m CD
Seasonal variations	0.10 m
Design water level	**+ 0.90 m CD**
Design Water Depth	**11 + 0.90 = 11.9 m**

Wave Condition

The analysis of daily wave climate can be used to design the crest leve! of a working road on a rubble mound breakwater. In this case suppose that only 1% of the time during a year it is allowed that rough sea conditions rnay stop the construction. And that constructionh as stop as soon as 2% of the waves will reach the crest of this working during run-up.

Daily Wave Climate

Data of probability that resultant of sea and swell occur in the given height and period class at the CD-11m contour near. MADRAS port in an average year for all direction sector is available. This data is analysed to make an exceedance daily wave climate curve, as shown Frgure 1.

Table 4. Calculation of probability of wave height exeedance ffor daily wave climate

Significant Wave Height Hs (m)		**% of Occurrence**	**Accumulative % Time of Exceedance**
0 - 0.25	> 0.00	14.78	100.00
0.25 - 0.50	> 2.25	41.36	85.22
0.50 - 0.75	> 0.50	25.37	43.86
0.75 - 1.00	> 0.75	12.82	18.49
1.00 - 1.25	> 1.00	4.51	5.67
1.25 - 1.75	> 1.25	1.06	1.16
1.75 - 2.25	> 1.75	0.09	0.10

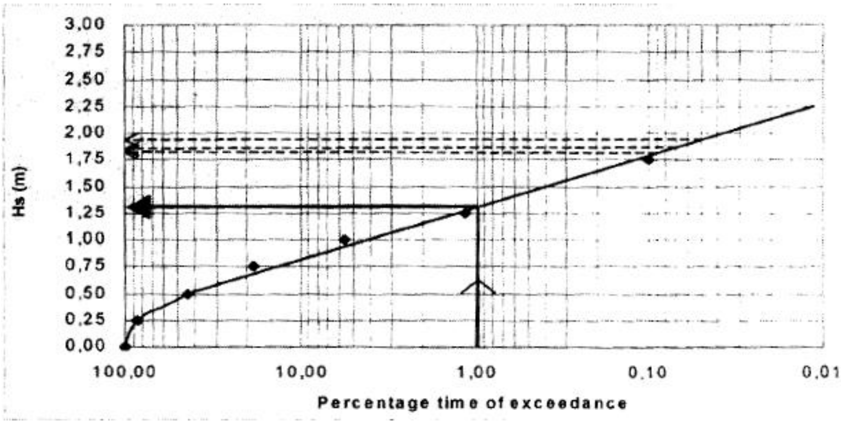

Figure 1. Exeedance curve of probability of wave height for daily wave climate

From the graph, the **1.0-%** wave height was estimited to be equal to **1.3 m**. If this value is plotted on graph of relation between significant wave height Hs and mean wave period Tm; (attachment; Figure 8), it has wave period **T=5** second.

The data about storm duration is not available. So, estimation of storm duration,comparison between some of long time of storm duration (4,6 and 8 hours) are made. Reading aloud of. Wave heights Hs each hour of storm duratlon trom tne Figure 1 (dash line) can be shown at below table.

Difference 1/1 year of Hs value between 4, 6 and 8 hours storm duratton are retativety small (insignificant). There fore, a six hours storm can be used for this project with 1/1 year **Hs = 1.87 m.**

Extreme Wave Climate

Near MADRAS-Port various wave parameters have been calculated on grid points 1-18. The exceedance curves are establlshed for average a cyclonic hindcast estimated wave height (actuatty grid points 17/18). Base on previous statisticcal analysis the 95 % boundary conditions for the average significant wave height were also defined as $Hs_{95\%} = Hs_{average} + 1.96.\sigma$, where σ is standard deviation. The confidence band for grid 17/18 was calculated for each return period using the extreme wave conditions in deep water. The results are given in Table 5 and plotted in a normal-logarithmic graph, Figure 2.

Table 5. Estimated 1/1 Year wave height for different storm duration

Strorm Duration (hrs)	No of Storm per year	% of time Exceedance	Hs (m)
4	(365*24)/4 = 2190	0.05	1.91
6	(365*24)/6 = 1460	0.07	1.87
8	(365*24)/8 = 1095	0.09	1.82

Table 5. The calculation of exeedance values of extreme wave height in deep water

Parameters	Return Period (year)				
	5	10	25	50	100
$H_{Average}$ (m)	4.13	5.30	6.58	7.48	8.33
Standard Deviation (σ)	.18	0.2	0.52	0.77	1.02
$H_{Average}$ + 1.64 σ m (95% Upper Boundary)	4.43	5.63	7.43	8.74	10.00
$H_{Average}$ - 1.64 σ m (95% Lower Boundary)	3.83	4.97	5.73	6.22	6.66
H etimated (m) Cyclone Hindcast)	4.20	5.70	7.20	8.20	9.10

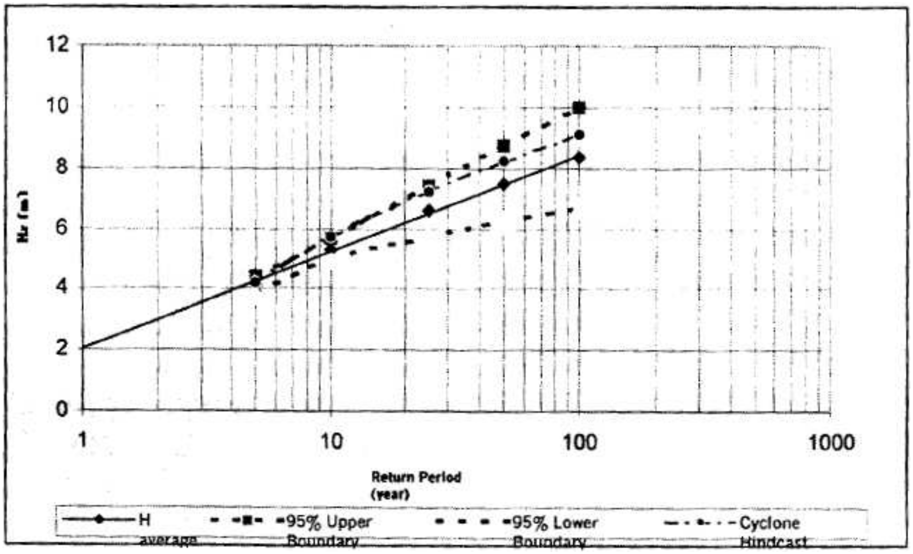

Figure 2. Extreme wave climate at deep water

Based on the exceedance curve of the deep-water wave heigh above, it can be estimated that the 1/1-year wave height is about **2.0 m**. This value is reliable if it is compared with the 1/1 year wave height of daity wave ctimate (**Hs = 1.87 m**).

Wave Breaking

The maximum significant wave height can be established at breakwater head by using the CUR method. Data which needed to calculate maximum H_s at the head breakwater in the handout *exercise breakwater design* are as following:

- The peak period is 1.1 to 1.3 times larger than mean period. We have calculated H_s/h value for variation peak period ($1.1T_{m0}$, $1.2T_{m0}$ and $1.3T_{m0}$) and then plotted on scatter graph (see figure 3.3). From this graph, the values of H_s/h for variation of peak period are relatively same, so for this design is taken $1.2T_{m0}$.

- Relationship between the significant wave height and significant wave period is shown Figure 8.
- The slope of foreshore and water depth at breakwater head can be estimted from Figure 2. It is approximately 1 : 50.
- The maximum significant wave height in front of breakwater is calculated based on cyclonic hindcast estimation.
- The value of Hs/h can be read from design graphs for uniform foreshore slopes which are given in CIRIA special publication 83/CUR report 154 (on exercise breakwater design lecture note is attached, too).

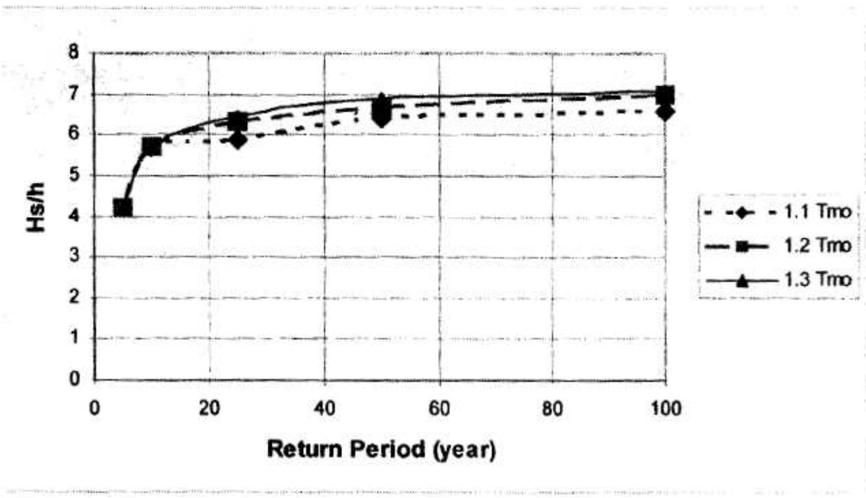

Figure 3. Value of Hs/h for peak period variation

Table 6 Calculation the maximum Significant Wave Height (Hs) at breakwater head (CUR Method)

Parameters	Return Period				
	5	10	25	50	100
H_{0s} (m)	4.20	5.70	7.20	8.20	9.10
T_{m0} (m)	7.50	8.30	8.90	9.30	9.60
T_{0p} (s) = 1.2 T_{m0}	9.00	9.96	10.68	11.16	11.52
L_{0p} (m) = 1.56 T_{0p}^2	126.36	154.75	177.94	194.29	207.03
h (m)	12.66	12.86	12.86	12.66	12.66
h/L_{0p}	0.100	0.082	0.071	0.065	0.061
S0p = (H_{0s}/L_{0p})	0.033	0.037	0.040	0.042	0.044
S (bed slope)	0.02	0.02	0.02	0.02	0.02
H_s/h	no breaking		0.500	0.528	0.550
H_s	4.20	5.7	6.33	6.68	6.96

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Calculation of maximum breaking significant can be done base on energy method, which means that the significant wave height is H_{m0}. The significant wave heights H_m and $H_{1/3}$ are almost equal in deep water. However, in shallow water they differ. Furthermore, The BREAKWATER program requires sometimes $H_{2\%}$. Based on H_{m0} calculation the significantwave $H_{1/3}$ and the 2% wave height $H_{2\%}$ can be calculated according Battjes and Groenendijk, 2000. The calculation is based on their work which can be summarised as follows:

$$\boldsymbol{H_{m0}} = \boldsymbol{4\sqrt{m_0}}$$

$$\boldsymbol{H_{rms}} = \left(2.69 + 3.24\frac{\boldsymbol{\sqrt{m_0}}}{\boldsymbol{h_m}}\right)\boldsymbol{\sqrt{m_0}}$$

$$\boldsymbol{H_{\tilde{\tilde{tr}}}} = (0.35 + 5.8\tan\boldsymbol{m})\boldsymbol{h_m}$$

$$H_{\tilde{\tilde{tr}}} = \frac{H_{tr}}{H_{rms}}$$

$$H_{1/3} = H_{rms} * H_{\tilde{\tilde{1//3}}}$$

$$H_{2\%} = H_{rms} * H_{\tilde{\tilde{2\%}}}$$

Table 7 shows calculation process and the result are plotted on graph Figure 4.

Table 7. Calculation of the maximum Significant Wave Height (Hs) at breakwater head (Energy Method)

Parameters	Return Period				
	5	10	25	50	100
$H_{0s} = H_{m0}$	4.20	5.70	6.33	6.68	6.96
m_0	1.10	2.03	2.50	2.79	3.03
H_{rms}	3.11	4.35	4.90	.21	5.46
H_{tr}	5.90	5.90	5.90	5.90	5.90
$\tilde{\tilde{H}}_{tr}$	1.90	1.36	1.20	1.13	1.08
$\tilde{\tilde{H}}_{1/3}$	1.407	1.348	1.331	1.324	1.320
$\tilde{\tilde{H}}_{2\%}$	1.795	1.633	1.612	1.671	1.599
$H_{1/3}$	4.37	5.87	6.52	6.90	7.20
$H_{2\%}$	5.58	7.11	7.90	8.71	8.73

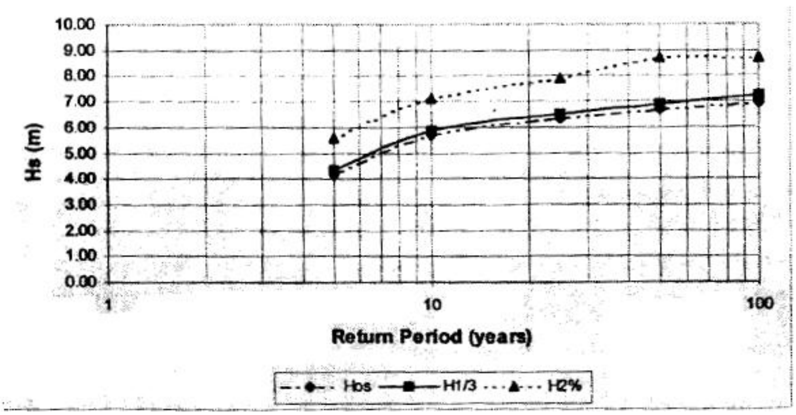

Figure 4. The Exeedance Curves of H_{0s}, $H_{1/3}$, and $H_{2\%}$

DESIGN OF STRUCTURE

Design alternative is based on slope angle variation of structure in 1:2, 1:3 and 1:3.5. The final of design activity is to choose the most effective structure and economically with limitation of boundary condition like volume of working, vavability of rocks material based on grain size which needed and ease during working time with attention in limitation of equipment.

Basic parameters, which used to design are based on previous calculation. The general condition will be applied to the structure are :

1. Structure is permeable structure, so the notional permeability factor, P = 4.
2. Breakwater structure is designed in safety and economically background, so the amage level for trunk section structure is on some flattening out in (S = 4 and Nod = 2). For head section, the level design is in start to damage (S=2 and Nod = 0.5), because a breakwater head has more progressive damage than trunk breakwater.
3. Return period of design is 1 in 100 years, because it is small difference between value of significant wave height in 1/50-year condition and 1/100-year condition.

Crest Level of Working Road

The daaily wave climate can be used to design the crest level of working road on rubble mound breakwater. Two criteria must be considered to determine the working road crest level.

1. Omly 1% of the time during a year is allowed that rough sea conditions may stop the construction. From exeedance curve or probability of wave height for daily wave climate (figure 3.1), $H_{s1\%}$ is estimated to be 1.3 m, with T - 5 sec. So, the wavelength is 1.56 x 5^2 = 39 m.
2. The construction must to stop as so on as 2% of the waves reach the working road crest level during run-up.

Van der Meer and Slam proposed the prediction formulas for run-up versus surf similarity parameter are :

$$\frac{Rux}{H_s} = a\xi_m for \xi_m < 1.5 \qquad \frac{Rux}{H_s} = b\xi_m for \xi_m < 1.5$$

with $\xi_m = \dfrac{\tan\alpha}{\sqrt{H/L_m}}$

The value a, b and c coefficients of the run-up level 2% are a=0.96; b=1.17; c=0.46. The results of run-up calculation for different slope of structure are shown in Table 8.

Table 8. Caalculation of 2% Run-up from $H_{1\%}$

Slope of Structure	1 : 2	1 : 3	1 : 3.5
ξ_m	2.74	1.83	1.56
R_{ux}/H_s	1.85	1.54	1.44
R_{ux} (m)	2.42	2.01	1.87

The type of breakwater that will be design is rock structure with large overtopping allowed (low-crested structure). It is a kind of **permeable breakwater structure**. So, the run-up for permeable structure (P>0.4) is limited to maximum:

$$\frac{Rux}{H} = d$$

From the Table 5 (page 21) of Breakwater Design Lecture Notes, for run-up level **i=2%, d** can be found **1.97**. So, **$R_{u2\%}$ = d x H_s = 1.97 x 1.3 = 2.56 m.**

We take last calculation because it is safer, thus, the working road crest level is
= $R_{2\%}$ + seasonal variations + MHW
= 2.56 + 0.10 + 0.8 m CD
= + 3.46 m CD

Crest Level of Structure

The low crested breakwaters are designed to be able to transmit wave energy into area behind the breakwater. For determination of the crest level, the criteria of allowed overtopping is essential. The large overtopping condition is defined as a maximum transmitted wave height of Hs, 1 m under the 1/50 year condition for this design. Following this condition, the transmission coefficient for 1/50 years return period wave height can be determined as follows:

$$Ct = \frac{transmitted\ wave\ height}{incident\ wave\ height} = \frac{1}{6.9} = 0.145$$

The performance of transmission is dependent on the structure geometry, crest freeboard, crest width, water depth, permeability and on the wave conditions (wave height and period).

$$C_t = a\frac{R_C}{D_{n50}} + b$$

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

$$a = 0.031\frac{H_i}{D_{n50}} - 0.24$$

where :
C_t = wave transmitted factor
R_c = crest freeboard
D_{n50} = nominal diameter of rock armour (m)
H_i = incident wave height in 1/50-year condition (m)
S_{op} = wave steepness
B = structure width

Reduction Factor

The stability of a conventional low-crested breakwater above still water level can be related to the stability of a non-or-marginally overtopped structure. The required rock diameter for an overtopping breakwater can be determined by application of a reduction factor for the mass of armour using Van der Meer (1990a) as follows:

Reduction factor $D_{n50} = \dfrac{1}{1.25\text{-}4.8.R_p^*}$

for $0 < R_p^* < 0.052$

$$R_p^* = \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}}$$

Where:
R_c = crest freeboard
D_{n50} = nominal diameter of rock armour (m)
H_s = design significant wave height (m)
S_{op} = wave steepness

Crest Width

The width of the crest can be small a required minimum width B_{min} should be provided, where (SPM, 1984):
B_{min} = (3 to 4) D_{n50}

According SPM, 1984, consider as a general guide for overtopping conditions that the minimum crest width should equal the combined widths of armour units (n=3).

Table 9. Calculation of Crest Level

Parameters	Values			Unit	Notes
slope of structure	1 : 2	1 : 3	1 : 3.5		variation slope design
S	4	4	4		damage level (design : start to damage)
$H_{S\,(1/50)}$	6.90	6.90	6.90	m	previous wave calculation
$T_{m\,(1/50)}$	9.30	9.30	9.30	Sec	previous wave calculation
ξ_m	2,21	1.47	1.26		$\xi_m = \frac{\tan\alpha}{\sqrt{2\pi H / gT_m^2}}$
breaking type	transition	plunging	plunging		
P	0.4	0.4	0.4		Design : Permeable Structure
N	2250	2250	2250		Assumption
ρ_r	2670	2670	2670	Kg/m^3	Data
ρ_w	1022	1022	1022	Kg/m^3	Data
Δ	1.61	1.61	1.61		$\Delta = \frac{\rho_r}{\rho_w} - 1$
D_{n50} **(for plunging type)**	1.99	1.62	1.50	m	$\frac{H_i}{\Delta D_{n50}} = 6.2.P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5}$
B	5.96	4.86	4.5	m	B = (3 - 4) D_{n50} take B = 3.5 D_{n50}
S_{op}	0.042	.042	.042		Previous Wave Calculation
Ct	0.145				$H_{T(=1m)}$ / $H_{s(1/50)}$
b	0.38	.41	.42		$b = -5.42\,.s_{op} + 0.0323\,\frac{H_i}{D_{n50}} - 0.0017\left(\frac{B}{D_{n50}}\right)^{1.84} + 0.51$
a	-0.13	-0.11	-0.10		$a = 0.031\,\frac{H_i}{D_{n50}} - 0.24$
R_c	3.55	3.93	4.20	m	$C_t = a\,\frac{R_c}{D_{n50}} + b$
Crest Level	5,21	5.59	5.86	+ mCD	1.66 CD + R_c

Weight of Rock Armour Unit

Weight of rock armour unit in deep water is obtained based on Van der Meer formulas depend on plunging or surging waves

$$\frac{H_i}{\Delta D_{n50}} = 6.2.P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5}$$

And for surging wave:

$$\frac{H_i}{\Delta D_{n50}} = 6.2.P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha\,\xi_m^P}$$

and $D_{n50} = \left(\frac{M_{50}}{\rho_r}\right)^{\frac{1}{3}}$

Where:

Δ = ρ_r/ρ_w-1

S = damage level

N = maximum number of wave (m)

ξ_m = irribaren number = $\frac{\tan\alpha}{\sqrt{2\pi H / gT_m^2}}$

M_{50} = weight of rock armour (m)

According the Original Hudson formula to calculate weight armour unit is written by:

$$M_{50} = \frac{\rho_r . H^3}{K_D \Delta^3 \cot \alpha}$$

In the 1973 edition of SPM the values given for K_D for rough angular stone in two layer on breakwater trunk where :
K_D = 3.5 for breaking waves
K_D = 4.5 for non breaking waves

The damage level of Hudson formula is equal to definition of start to damage (S= 2).

The value of ρ_r = 2670 kg/m^3 and ρ_w = 1022 kg/m^3. The maximum number of Hs design during storm which attack the structure (N) is predicted based on storm recording at st641221. The value of N is 6 hours x 3600 second / 9.6 second = 2250 times.

The thickness of layers is given by (SPM, 1984).
$\mathbf{t_a=t_u=t_f=n.k_t.D_{n50}}$

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

$\mathbf{N_a = n.k_t.(1-n_v)}$ D_{50}^{-2}

Where:

$\mathbf{t_a=t_u=t_f}$	=	thickness of armour, under layer or filter
n	=	number of layers
k_t	=	layer thickness coefficient
n_v	=	volume porosity

For rough rock and number of layer (n) = 2, value of k_t is 1 and n_v = 0.4 (SPM, 1984). According to Van der Meer (1988a), the thickness of armour for P=0.4 is $\mathbf{2D_{n50}}$.

Weight of Rock Under Layer and Core

Rubble mound structures in coastal and shoreline protection are normally constructed with an armour layer and one or more under layers (filter).

The SPM (1984) recmmends for the stone mass of the Under Layer :
(1/10 - 1/15) $\mathbf{M_{50\ armour}}$

According to Van der Meer (1988a), the thickness of Under Layer for P = 0.4 is $\mathbf{1.5D_{n50armour}}$. To calculate $D_{n50Under\ Layer}$ and $D_{n50core}$, the formula can be applied:
$\mathbf{D_{n50armour} / D_{n50Under\ Layer} = 2}$
$\mathbf{D_{n50Under\ Layer} / D_{n50core} = 4}$

For this design we use Van der Meer (1988a) formula.

Toe Protection

In most cases a toe protect the armour layer on the seaside near the bottom. If the rock in the toe has the same dimension as the armour, the toe will be stable.

Gerding' test (1993) were performed in order to establish the influence of wave height, wave steepness and water depth on toe stability. One of main conclusions was that the wave steepness had no influence. His analysis resulted below formula:

$$\frac{H_i}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6\right) N_{od}^{0.15}$$

where:

h_t	=	surface elevation of toe from sea water level.
N_{od}	=	damage level
	=	0.5 (start to damage)
	=	2 (some flattening out)

= 4 (complete flattening out of the toe)

D_{n50} of under layer is taken for toe protection calculation. This applies to a staandard toe sixe of about 3-5 rocks wide and 2-3 rocks high. For wider toe structures a higher damage level can be applied before flattening out occurs.

Calculations for Trunk Section

Based on above theories, calculations for trunk section are made. Calculations are made in table with some notes in perhaps reader can follow them. For comparing, a calculation based on Breakwat. Program is shown, too. The final design will compare between 3 methods calculation (Van der Meer formula; manual calculation and Breakwat program and Hudson formula; manual calculateon)

Breakwater Head

According to Jensen (1984), when a wave is forced to break over around head it leads to large velocities and wave forces. For a spesific wave direction only a limited area of the head is highly exposed. It is an area around the still water level where the wave orthogonal is tangent to surface and on the lee side of this point. It is therefore general procedure in design of heads to **increase the weight of the armour to obtain the same stability as for the trunk section. Alternatively, the slope of the round head can be made less steep, or combination of both** (in CIRIA, 1991).

The damage curve for a head is often steeper than for a trunk section (Jensen, 1984). The damage curve for head is often steeper than for a trunk section. A breakwater head may show progressive damage. This **means that the head section has more (unxpected) failure than trunk section if both structure are designed in same level**.

No spesific rules are available for the breakwater head. The required increase in weight can be a factor between 1 and 4, depending on the type o armour unit. The factor for rock is closer to 1. So, for this design does not need any factor for increase the rock weight. Nevertheless, **to give more stability for head section structure, damage level is designed in start to damage (S=2).**

The calculation of breakwatr head is shown in the above table. The damage level of Hudson formula is start to damage (S=2), so we try to compare the Hudson's with others calculation for head section.

Table 10. Calculation of Rock Armour Layer, Under Layer, Core and Toe Protection for Trunk Section

Parameters	Values			Unit	Notes
slope of structure	1 : 2	1 : 3	1 : 3.5		slope design variation
S	4	4	4		damage level (some flattening out)
$H_{S\ (1/100\ years)}$	7.20	7.20	7.20	M	previous wave calculation
$T_{m\ (1/100\ years)}$	9.60	9.60	9.60	Sec	previous wave calculation
ξ_m	2.2	1.5	1.3		$\xi_m = \frac{\tan\alpha}{\sqrt{2\pi H / gT_m^2}}$
breaking type	transition	plunging	plunging		
P	0.4	0.4	0.4		Porosity factor for permeable Structure
N	2250	2250	2250		Data
ρ_r	2670	2670	2670	Kg/m^3	Data
ρ_w	1022	1022	1022	Kg/m^3	Data
Δ	1.61	1.61	1.61		$\Delta = \frac{\rho_r}{\rho_w} - 1$
ARMOUR LAYER					
R_p^*	0.041	0.046	0.049		$R_p^* = \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}}$
Reduction Factor D_{n50}	0.951	0.970	0.985		Reduction for low created $= \frac{1}{1.25 - 4.8.R_p^*}$
$D_{n50\ armour\ layer}$ (for plunging)	1.98	1.65	1.55	m	$\frac{H_i}{\Delta D_{n50}} = 6.2.P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$
$M_{50\ armour\ layer}$	20.719	11.982	9.943	kg	$D_{n50} = \left(\frac{D_{n50}}{\rho_r}\right)^{\frac{1}{3}}$
K_t	1	1	1		Layer thickness coefficient and volumetric porosity for rough rock and n = 2 (SPM, 1984)
n_v	0.37	.042	.042		
$n_{armour\ layer}$	2	2	2	layer	number of armour layer
$t_{armour\ layer}$	4.0	3.3	3.1	m	armourthckness t_a=n k_t D_{n50}
$N_{armour\ layer}$	0.32	0.46	0.52	$unit/m^2$	number of units per m^2 of armour
UNDER LAYER					
$D_{n50\ Under\ Layer}$	0.99	0.82	0.77	m	**$D_{n50armour}$ / $D_{n50Under\ Layer}$ = 2**
$M_{50\ Under\ Layer}$	2589.8	1497.8	1242.8	kg	
$t_{Under\ Layer}$	2.97	2.47	2.32	m	**$t_{Under\ Layer}$ = 1.5*$D_{n50\ armour}$**
CORE					
$D_{n50\ Under\ Layer}$	0.25	0.21	0.19	m	$D_{n50Under\ Layer}$ / $D_{n50core}$ = 4
$M_{50\ Under\ Layer}$	40.5	23.4	19.4	kg	
TOE					
N_{od}	2.0	2.0	2.0		damage level (start to damage)
h_t	7.92	9.02	9.39	m	$\frac{H_i}{\Delta D_{n50}} = \left(0.24\frac{h_t}{D_{n50}} + 1.6\right) N_{od}^{0.15}$ D_{n50}=$D_{n50Under\ Layer}$

Table 11. Calculation of Rock Armour Layer for Head Section Based on Hudson Formula

Parameters	Values			Unit	Notes
slope of structure	1 : 2	1 : 3	1 : 3.5		slope design variation
$H_{S\ (1/100\ years)}$	7.20	7.20	7.20	M	
$T_{m\ (1/100\ years)}$	9.60	9.60	9.60	Sec	previous wave calculation
Δ	1.61	1.61	1.61		$\Delta = \frac{\rho_r}{\rho_w} - 1$
ρ_r	2670	2670	2670	Kg/m^3	Rock density
K_D	transition	plunging	plunging		Stability coefficient
Cot	2	3	3.5		slope
Reduction Factor D_{n50}	0.943	0.950	0.956		
$M_{50\ armour\ layer}$	28.453	19.387	16.942	kg	$M_{50} = \frac{\rho_r . H^3}{K_D \Delta^3 \cot\alpha}$
$D_{n50\ armour\ layer}$	2.20	1.94	1.85	m	

For comparing purpose, the calculation of Head section based on BREAKWAT program is done too

Table 12. Calculation of Rock Armour Layer, Under Layer, Core and Toe Protection for Head Section

Parameters	Values			Unit	Notes
slope of structure	1 : 2	1 : 3	1 : 3.5		slope design variation
S	2	2	2		Damage level (start to damage)
$H_{S\ (1/100\ years)}$	7.20	7.20	7.20	M	previous wave calculation
$T_{m\ (1/100\ years)}$	9.60	9.60	9.60	Sec	previous wave calculation
ξ_m	2.2	1.5	1.3		$\xi_m = \frac{\tan\alpha}{\sqrt{2\pi H / gT_m^2}}$
breaking type	transition	plunging	plunging		
P	0.4	0.4	0.4		Porosity factor for permeable Structure
N	2250	2250	2250		Data
ρ_r	2670	2670	2670	Kg/m^3	Data
ρ_w	1022	1022	1022	Kg/m^3	Data
Δ	1.61	1.61	1.61		$\Delta = \frac{\rho_r}{\rho_w} - 1$
R_p*	0.040	0.042	0.044		$R_p^* = \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}}$
Reduction Factor D_{n50}	0.946	0.955	0.962		Reduction for low created $= \frac{1}{1.25 - 4.8 . R_p^*}$
$D_{n50\ armour\ layer}$ (for plunging)	2.26	1.86	1.74	m	$\frac{H_i}{\Delta D_{n50}} = 6.2 . P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} * \mathrm{Re}\,d.Fac$
$M_{50\ armour\ layer}$	30.69	17.313	14.052	kg	$D_{n50} = \left(\frac{D_{n50}}{\rho_r}\right)^{\frac{1}{3}}$

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Parameters	Values			Unit	Notes
K_t	1	1	1		Layer thickness coefficient and volumetric porosity for rough rock and n = 2 (SPM, 1984)
n_v	0.37	0.37	0.37		
$n_{armour\ layer}$	2	2	2	layer	number of armour layer
$t_{armour\ layer}$	4.5	3.7	3.5	m	armourthckness t_a=n k_t D_{n50}
$N_{armour\ layer}$	0.25	0.36	0.42	unit/m^2	number of units per m^2 of armour
UNDER LAYER					
$D_{n50\ Under\ Layer}$	1.13	0.93	0.87	m	$D_{n50\ armour}$ / $D_{n50\ Under\ Layer}$ = 2
$M_{50\ Under\ Layer}$	3871.2	2164.2	1756. 5	kg	
$t_{Under\ Layer}$	3.40	2.80	2.61	m	$t_{Under\ Layer}$ = 1.5*$D_{n50\ armour}$
CORE					
$D_{n50\ Under\ Layer}$	0.28	0.23	0.22	m	$D_{n50\ Under\ Layer}$ / $D_{n50\ core}$ = 4
$M_{50\ Under\ Layer}$	60.5	33.8	27.4	kg	
TOE					
N_{od}	0.5	0.5	0.5		Damage level (start to damage)
h_t	10.33	11.68	12.08	m	$\frac{H_i}{\Delta D_{n50}} = \left(0.24\frac{h_t}{D_{n50}} + 1.6\right)N_{od}^{0.15}$ D_{n50}=$D_{n50Under\ Layer}$

Discussion

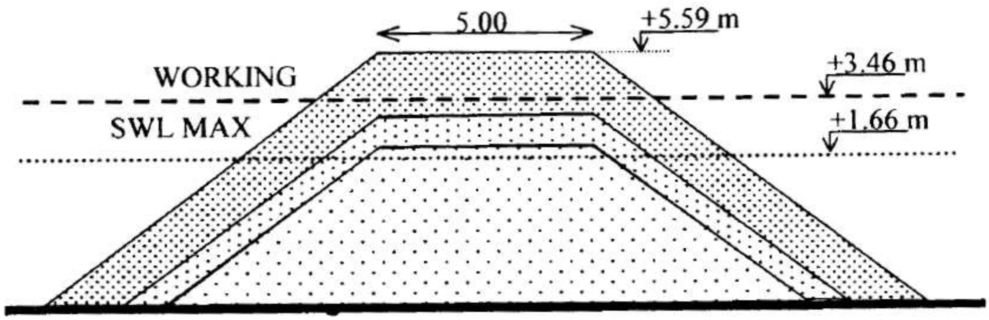
1. The results of 3 methods $M_{50\ armour\ layer}$ calculation are summarized on above table. The calculation based on Hudson formula is done only for Head section, because it is equal with Van der Meer formula where its damage level is start to damage (S=2).
   From the table, differences in calculation results are significant. Nevertheless, we take Van der Meer formula in manual calculation as design values. This matter is only clearance in calculation process and if we compare with Hudson Formula, the value is smaller but is not too much difference.
2. For slope of structure 1:2, $M_{50\ armour\ layer}$ is 20.7 ton for trunk section and 30.97 t (say 31 ton) for head section. These materiaal M_{50} varies are not available according to Curve Trial Blast 3 exc>15T.
3. For trunk section structure, we take 1: 3 slope structure. The design weight of material 11,982-kg (say 12 t) for armour layer, is available. And, slope 1: 3 is not gentler than slope 1:3.5. The damage level is 4 (some flattening out)
4. For head breakwater structure, slope 1:3.5 is taken, as design value with damage level is 2 (start to damage).
5. Elevation of working road is enough for dump truck and other land-based heavy equipment to operate during construction, because minimum differrence between MSL and working road is 1.5 m to avoid effect of splash and spray (CIRIA, 1991).
   During construction, driving directly on rock fill is not possible with rubber tires, but accessibility can be maintained by dozing fines over the surface with a bulldoer. For elevation of working road (+3.46 m CD), wide of working road is (5 + (5.59-3.46) x 3 x 2)=17.78 m. This is eno-

ugh sufficient space has to be provided for allow passing of dump trucks, with the (hidraulic) crane in operation.

6. Based on calculation of structure design and available of stones quarry, the rock classes for this design can be proposed as follow.

Table 13 Comparing for Varies Calculation Both Trunk Section and Head Seaction

TRUNK SECTION			
	1 : 2	**1 : 3**	**1 : 3.5**
Van der Meer Formula (Manual Calculation with reduction factor)	20,719 kg	11,882 kg	9,943 kg
Van der Meer (BREAKWAT Program)	15,500 kg	8,471 kg	6,722 kg
HEAD SECTION			
Van der Meer Formula (Manual Calculation Calculation with reduction factor)	30,969 kg	17,313 kg	14,052 kg
Hudson Formula (Manual Calculation with reduction factor)	28,453 kg	19,387 kg	16,942 kg
Hudson Formula (BREAKWAT Program without reduction factor)	33,950 kg	22,640 kg	19,400 kg
Van der Meer (BREAKWAT Program)	23,590 kg	12,840 kg	10,190 kg



ROCk CLASSES	WEIGHT/SIZE ANGLE	NOTES
Class I	10 ton- 15 ton 1.5 m - 1.8 m	For armour layer Trunk : 11,982 kg Head : 14,052 kg
Class II	2 ton - 10 ton 0.90 m - 1.50 m	For Under layer Trunk : 1242.8 kg Head : 1756.5 kg
Class III	20 kg - 2 ton 0.20 - 0.90 m	For core Trunk : 19.4 kg Head : 27.4 kg
Class IV	5 kg - 20 kg 0.12 mm - 0.20 m	For bed protection

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RECOMMENDATION

Breakwater is very expensive structure . Structure failure can cause loss a lot of money. There are different phenomena for each location where breakwater built. So, it is strong recommedation to make model test, which can desribe phenomena around prototype. Unpredicted phenomena from model test can be perfected in design before construction time.

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