Optimum design of rubble mound breakwaters cross section in random sea

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This paper aims to automate the design procedures for rubble mound breakwaters. Also, in this paper we introduce a computer program that optimizes the design process. The design can be optimized for volume of the stone used (minimum volume), for quarry usage (minimum waste) or for construction cost and finally for maintenance cost along the expected life time of the breakwater. All the elements of the breakwater are considered in the analysis, the cover layer, the filter and the core. The design satisfies functional performance structural stability and minimizes the overall cost, that is the construction cost and lifecycle maintenance cost.

يهدف هذا البحث الى تقديم طريقة يتم بها ايجاد جميع التصميمات الممكنة لحواجز الأمواج الكوميه ومنها يتم أختيار التصميمات التى تحقق أقل حجم ممكن، أقل تكلفة إنشاء ، أقل تكلفة صيانة وأيضا أفضل أستخدام ممكن للمحجر مع ملاحظة أن جميع هـــذه البدائل تجعل حاجز الامواج أمن أنشائيا وأيضا تكون الأمواج خلف الحاجز بالارتفاعات التصميمية المطلوبة.

Keywords: Rubble mound breakwaters, Optimum design of rubble mound

1. Introduction

A successful design of a rubble mound breakwater must satisfy two basic criteria, the structural stability criteria and the functional performance criteria. The structural stability criteria means that the breakwater has to withstand the extreme waves that may occur in a long period, 50 or 100 years, associated with change in the water depth without sustaining significant damage. The functional performance criteria means that the wave heights in the lee side of the breakwater must not exceed some prescribed value regardless the wave heights in the seaside of the breakwater. The properties of the breakwater affecting wave transmission ought to be designed to satisfy this condition. In addition to the structural and functional criteria the design of the breakwater must be economic, in the sense that the overall cost of the structure must be minimized. Minimizing the cost means that a balance between the construction cost and maintenance cost for the lifetime of the breakwater is made, so that the total sum of them is minimized. Also, an optimum use of the fragmentation of the quarry stone must be taken into account. If possible, most researchers in the field of breakwater design recommend using natural stones in place of man-made units. This paper deals with the design of breakwaters of natural stone.

From above it can be seen that the design procedures consist of the following:-

1. Define the expected life of the breakwater.
2. Define the extreme wave conditions and maximum change in water depth.
3. Calculate the wave condition at the construction site.
4. Specify the maximum wave height admitted in the leeside of the breakwater.
5. Use the above information to fix the geometrical dimension of the structure "functional criteria".
6. Select the size, thickness and slope of the different layers to satisfy the structural criteria.
7. Calculate the level of damage due to waves higher than the design wave and find out the total cost of the construction and maintenance, also check the required material versus the fragmentation curve of the quarry.

8. Repeat step (5) through (7) to get an optimum design.

2. Extreme design conditions

The selection of design wave height depends on either the wave breaks or does not break. If the waves do not break, then the selection of the design wave depends on the type of the structure. For rigid structures such as a cantilever sheet pile wall it is recommended to use H_1, where H_i is the average of the highest 1/i waves .For semi-rigid structures such as steel sheet-pile cell using design wave height between H_1 and H_{10} is recommended. For flexible structures such as rubble mound breakwater using design wave ranging between H_5 and H_3 was accepted in the past. Now, the shore protection manual , SPM[1], recommends using H_{10} for the design purposes. However, the program introduced here is flexible and asks the user to select a design wave height H_3, H_5 or H_{10} as example. The wave height at the site of construction is related to the wave height in the deep water through a series of the transformation, to be introduced later. Generally, there are two methods to find the sea state characteristics at deep water. The first method is called hindcasting and is based on using meteorological or wind data. The second of them is called forecasting and is based on using the collection of data provided by wave-record instruments and related to water surface elevation. Both methods can be used to find H_S or the wave spectrum S(f). From those quantities., we can calculate H_5 , H_{10} or any H_i given the probability distribution function, PDF of the wave heights. In general distribution of the wave heights in deep water is assumed to obey the Rayleigh distribution. It must be kept in mind that the above mentioned method gives short-term distribution, and in the design of Coastal Structures, we need long term distributions.

Many distributions are available to represent the long-term statistics of the wave heights. Some of them are log normal distribution, Extreme Type I distribution (also called Gumbel or Fisher Tippett I distribution), Extreme Type II distribution (also called Fretchet or Fisher – Tippett II distribution) and Extreme Type III distribution (also called Weibull distribution). There are two alternatives for the last distribution, namely they are III_L and III_U distributions. The first is a lower bound distribution, while the second is an upper bound distribution. The reader is referred to Sarpkaya and Isaacson [2], for a detailed description of those distributions. Selecting one or another is a site dependent.

3. Design wave and storm survey

The longer the life of the breakwater, the higher the wave that may be sustained by it. The return period is given by,

$$T_R = \frac{r}{Q(H)} = \frac{r}{1-P(H)} \quad . \tag{1}$$

Where;

r is the time interval between successive wave records, usually 3, 6 or 12 hours

Q(H) is the probability that the measured wave heights will be greater than or equal H , and called the probability of exceeding .

P(H) is the cumulative probability function.

Thus if we select the return period to be 50 or 100 years, then we can calculate P(H) and from these we can find the design wave. The return period or recurrence interval, T_R, is the average time interval between successive events of the design wave being equaled or exceeded. A quantity related to Tr is encounter probability E, given by

$$E = 1-\left(r / T_R\right)^{L/r} . \tag{2}$$

This is the probability that the design wave will be equaled or exceeded during prescribed period L. When $T_R / r >> 1$, eq. (2) takes the form

$$E = 1 - \exp\left(-L / T_R\right) . \tag{3}$$

For example, with a return period 50 years, the probability that the design wave will be exceeded in 10,50 and 100 years are 0.181, 0.632 and 0.865 respectively.

Now, after specifying the return period T_R, and finding the probability P(H) associated with it, it remains to calculate the deep water design wave height.

Two methods are in common use. The first of them consists of plotting the available data on a height exceedence diagram, as shown in fig. 1. for a log-normal distribution. The data points lie on a straight line. This line is to be extended and the wave height corresponding to the known probability is picked. This method does not suit computer application.

The second method depends on selecting a theoretical distribution and fitting its parameters to the wave data. In this paper we select the lower bound Weibull distribution, given by

$$P_W(H_S) = P(H < H_S) = 1 - \exp\left[-\frac{(H_S - H_O)^m}{\beta}\right], \quad (4)$$

where;

m is the shape parameter, which determines the basic shape of the distribution, its value varies between 0.75 and 2.0,

H_O is the location parameter and J defines the lower limit of Hs during storm. B the scale factor which controls the degree of spreading along the vertical axis.

To find the parameters m, H_O and β the method of moments, the method of least squares or the method of maximum likelihood may be used. For a detailed discussion, the reader is referred to Tucker [3].

An important factor that must be taken into account in the design process is the variation in water level, that change the design depth h_D given by Brun [4].

$$h_D = h_O + A_S + S_O + S_P + S_S + W_S, \quad (5)$$

where;

h_O is the still water level (SWL).

A_S is the astronomical tide, changes.

S_O is the changes due to the approaching storm, that appears before the storm and is called "for runner".

S_P is the changes due to variation in atmospheric pressure and is called Seiche.

S_S is the changes due to wave breaking and is called wave setup

W_S is the changes due to wind blowing over the surface of the water and is called storm surge.

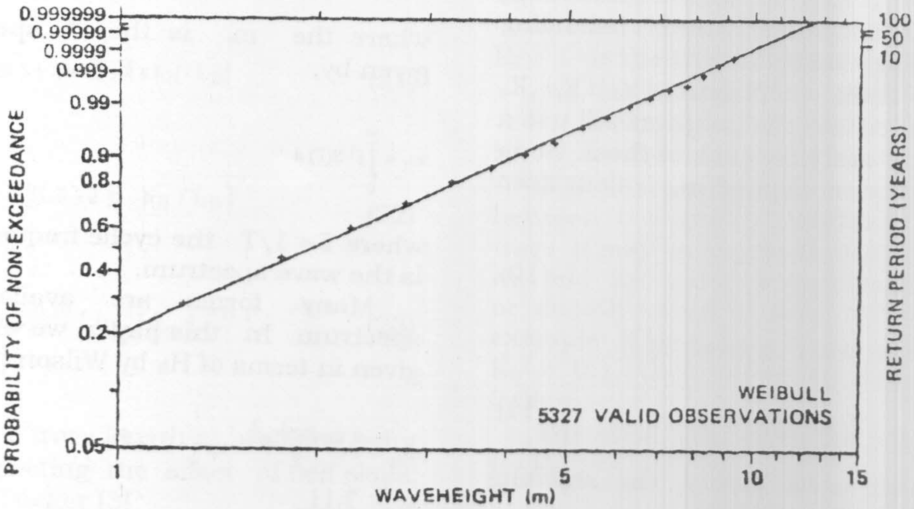

Fig. 1. Weibull distribution for H_o = 15 meter.

In general the design water depth will be taken as,

$$h_D = h_O + A_S + h_S , \quad (6)$$

where, h_S combines all the factors mentioned in eq. (5) and not shown in eq. (6). h_S has an exponential commutative distribution that can be obtained from Weibull distribution by taking m = 1, that is

$$P_E(h_S) = P(h \le h_S) = 1 - \exp\left[-\left(\frac{h - h_O}{\Theta}\right)\right], \quad (7)$$

where h_O is the average value of the deviation from SWL during storm is the shape parameter.

In the following analysis it will be assumed that

$$P_E(h_S) = P_{WL}(H_S). \quad (8)$$

4. Design wave

The design wave at the site, is related to H_S, of the deep water, equation (4), by Ippen [5],

$$H_D = K_D K_r K_a K_S H_S \quad (9)$$

where K_D, K_r, K_a and K_s are the diffraction, refraction, attenuation and shoaling coefficients, respectively.

The first three coefficients, K_D, K_r, K_a must be specified or the program will use a default value of unity for each of them. Goda [6] introduced the concept of equivalent deep water wave H_E given by

$$H_E = K_D K_r K_a H_S . \quad (10)$$

Hence, the design wave is given by

$$H_D = K_S H_E . \quad (11)$$

From the linear wave theory, the shoaling coefficient is given by, SPM [1]

$$K_S = \left[\tanh(kd)\left(1 + \frac{2kd}{\sinh(kd)}\right)\right]^{-0.5} , \quad (12)$$

where k = 2/L, d, L are the wave number, water depth and wave length, respectively. To find the wavelength L, we use the dispersion relation given by,

$$\omega^2 = g\, k \,\text{tash}\,(kd) , \quad (13)$$

where T, $\omega = 2\pi / T$, are the wave period and radian frequency, respectively. For deep water the wave length L_O, to be used is given by, (13)

$$L = 1.56\, T^2 . \quad (14)$$

A relation of the form,

$$T_P = a\, H_D^C + b , \quad (15)$$

may be used to relate the peak period T_P, to the design wave height H_D, where a, b and c are constants. In this paper, we will use the mean wave period T_m given by,

$$T_m = m_o / m_1 , \quad (16)$$

where the m_i is the i^{th} spectral moment given by,

$$m_i = \int_0^\infty f^i\, S(f)\, d , \quad (17)$$

where f = 1/T the cyclic frequency, and S(f) is the wave spectrum.

Many forms are available for the spectrum. In this paper, we will use the P-M given in terms of H_S by Wilson [7]

$$S_\eta = \frac{8.1X\,10^{-3}\, g^2}{\omega^5} \exp(-B/\omega^4) \quad (18)$$
$$B = \frac{3.11}{(H_S)^2}.$$

The mean wave period, T_m, is needed to calculate the number of waves N during a storm of duration T_S,

$$N = T_S / T_m \; . \tag{19}$$

T_m will be used to find out the weight of the armor layer blocks. The shoaling coefficients, K_S, given by eq. (12) is for regular wave Goda [6], mentioned a method to find out the shoaling coefficient taking into account the randomness of the wave field. The difference between the shoaling coefficients for the two cases is on the order of 2 to 3 percent and may be neglected in practical design procedures.

The design wave may be controlled by the depth limitation, that is it may break. Generally, there are two types of criteria to predict wave breaking. The first of them determines the breaking height in terms of local parameters such as water depth, and / or bed slope. Goda [6], introduced the following formula for the breaking height H_B

$$H_B / L_O = 0.17 \left[1 - \exp\left(-1.5\pi\, h_B / L_O (1 + 15 \{ \tan\beta \}^{4/3})\right)\right] \tag{20}$$

where L_O, and h_B , are the wave length in deep water, water depth at breaking point and bed slope, respectively.

Some other formulas that give H_B are, Horikawa [8],

$$H_B / L_B = 0.14 \tanh \{(0.8 + 5 \tan\beta) 2 \pi h_B' / L_B\} \tag{21}$$

$\tan\beta < 0.1,$

$$H_B / L_B = 0.14 \tanh \{(1.3) 2 \pi \; h_B / L_B\} \tag{22}$$

$\tan\beta > 0.1,$

$$H_B / L_B = 1.09 (\tan\beta)^{0.19} (h_b / L_o)^{-0.1} , \tag{23}$$

where, L_B is the wave length at breaking point

When neglecting the effect of bed slope, we can write, Tucker [3]

$$H_B = \lambda \; d , \tag{24}$$

where λ = 0.78 and 0.8 for conoidal wave theory, and Dean stream function theory, respectively.

The other criterion to find the breaking height neglects the effect of water depth. Some formula are, Horikawa [8],

$$H_B / H_E = 0.76 (\tan\beta)^{1/7} (H_E / L_O)^{-0.25} , \tag{25}$$

$$H_B / H_O = (\tan\beta)^{0.2} (H_O / L_O)^{-0.25} . \tag{26}$$

$C = 0.51 - 0.11 (B / d)$, and $\qquad B / d << 3.2$.

In this paper we will use eq. (20), to determine the breaking height.

5. Functional criterion

Controlling the wave height in the leeside of the breakwater, H_L, is a major concern in the design of harbors. It determines the shut down time, the safety of operations inside the harbor and the occurrence of resonance.

The relation between H_L and the design wave, which is the smaller of the two wave heights given by eqs. (11) and (20) is given by,

$$H_L = \sqrt{K_D^2 + K_T^2 + K_O^2} \;\; H_D , \tag{27}$$

where;

K_D is the diffraction coefficient due to the existence of the breakwater,

K_T is the transmission coefficient due to the passage of the waves through the body of the breakwater ,

K_O is the over toping coefficient.

For short waves K_T may be in the range between 0.1 and 0.3, SPM [1]while for long wave it may be approach unity. In this paper, K_D and K_T must be supplied to the program or default value of zero will be used. As an example if H_D = 5 m, H_L = 1 m and K_T = 0.15, K_D = 0.1, then using eq. (27) we get K_O = 0.0866.

The SPM [1], gives the following formula for K_O,

$$K_O = (1.0 - F/R \; , \tag{28}$$

$$C = 0.51 - 0.11\,(B/d) \qquad B/d << 3.2\,, \qquad (29)$$

where;

F is the freeboard, elevation of the cheats of the breakwater above the MWL

B is the width of the cheats

d is the water depth

R is the run up along the slope of the breakwater, assuming that it extend long enough.

The SPM [1], introduced a series of graphs to find the run up R as a percentage of deep water wave height, for smooth impermeable slope. These curves do not take into account the effect of permeability. Vander Meer, [9] introduced the following equation to calculate the Run up R, inters of the design wave height H_D

$$R/H_D = a\,\zeta_m \qquad \zeta_m < 1.5\,, \qquad (30\text{-a})$$

$$R/H_D = b\,\zeta_m^c \qquad \zeta_m > 1.5\,, \qquad (30\text{-b})$$

$$R/H_D = d \qquad P < 0.4\,, \qquad (30\text{-c})$$

where,

P is the permeability, and

ζ_m is the surf similarity parameter, and given by,

$$\zeta_m = \tan\alpha \Big/ \sqrt{H_D/(g/2\pi)\,T_m^2} = \tan\alpha \Big/ \sqrt{(H_D/L_m)}, \qquad (31)$$

where, Ln = 1.56 T^2_m, and α is the seaward slope of the structure.

Fig. 2 shows different values of P for different arrangements of the cover and filter layers a, b, c, d constants given in table 1.

Another formula to calculate the Run up is given by [10],

$$R/H_D = \frac{a\zeta}{(1+b\zeta)} \qquad R/H_D = A\,[1 - \exp(B\zeta)], \qquad (32)$$

Table 1
Coefficients of eq. (30) for rock armored slope, Abbott[9].

Run-up exceedence probability, p %	a	b	c	D
0.1	1.12	1.34	0.55	2.5
2	0.96	1.17	1.97	1.97
5	0.86	1.05	1.68	1.68
10	0.77	0.94	1.45	1.45
50	0.47	0.60	0.34	0.82

where a, b, A and B are empirical coefficients defined in terms of the seaward slope, the design wave height, and wave length at the toe of the breakwater.

For quarry stone breakwaters with highly permeable core a = 0.692 , b= 0.504. For multi layered quarry stone breakwaters a = 0.775, b = 0.361The values of A and B are given by A = 1.451 to 1.370, and B = - 0.523 to -0.596. The above data from Smith [10]. The surf similarity parameter ζ_m is a major parameter in defining the type of breaking. For spilling $\zeta\zeta_m < 0.5$, while for plumping $0.5 < \zeta_m < 3.3.$, and finally for surging $\zeta_m > 3.3$. The Run-down is usually 1/2 to 1/3 the Run-up. The maximum Run-down may be used to indicate the down word extension of the main armor.

Now, if we have the design wave height and the maximum allowed wave height inside the harbor, we can use eq. (28) to find the over toping coefficients. Using combinations of α, B, F that satisfy the required functional criterion. Those different sets are to be tested for hydraulic stability and economical aspects.

6. Hydraulic stability criterion

The new trend in rubble mound breakwaters design is to use natural stones, whenever possible. That is if a quarry is found near by the site of construction and the design wave height allows using natural stone, then it is recommended to use quarry stones in the construction. Many formulas are developed around the world to find the weight of the armor layer as a function in the design wave height. Many formula are tabulated in Bruun [4]. The two most famous

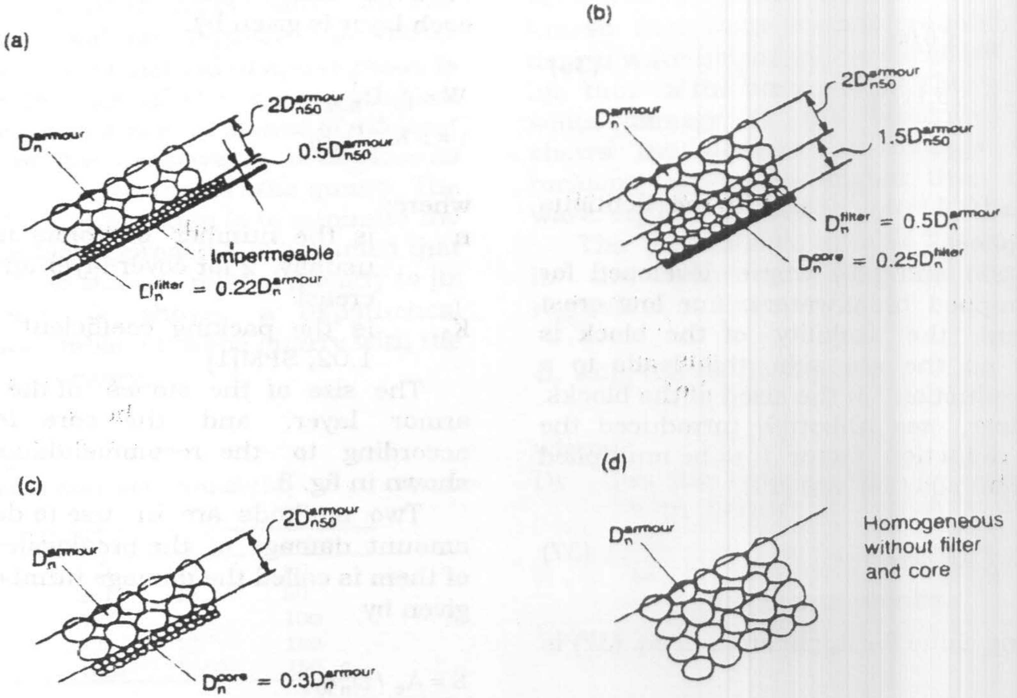

Fig. 2. Permeability values for different structures.

design formulas are those due to Iribarren and Hudson. Van der Meer [6] estimated the uncertainty in using Hudson formula for rock armor to a coefficient of variation of 18%. He also, introduced a set of design equation in which the coefficient of variation is estimated to be 6.5% and 8%, respectively for eqs. (28) and (29) that follow Van der Meer, see Abbott [9] design equation are given by ,

$$\frac{H_D}{\Delta D_{n50}} = 6.2\, S^{0.2}\, P^{0.13}\, N_Z^{-0.1}\, \zeta_m^{-0.5}, \quad (33)$$

for plunging waves $\zeta\zeta_m < \zeta\zeta_{mc}$, and ,

$$\frac{H_D}{\Delta D_{n50}} = 1.0\,(S^{0.2}\, P^{0.13}\, N_Z^{-0.1})\,(\cot\alpha)^{0.5}\, \zeta_m^{-0.5} \quad (34)$$

for surging waves $\zeta_m < \zeta\zeta_{mc}$ where,

$$\zeta_{mc} = \{6.2\, P^{0.31} (\tan\alpha)^{0.5}\}^{1/(P+0.5)}, \quad (35)$$

where;

$D_{n50} = (W_{50}/\rho g)^{1/3}$ the diameter (weight) of the 50 % size in the gradation,

W_{50} is the median weight of rocks

$\Delta = \rho/\rho_W - 1$ ρ and ρ_W are mass density of the rock and water, respectively,

S is the damage number,

ΔT is the storm duration,

$N_Z = \Delta\Delta T/T_m$ is the number of waves in the storm,

P is the porisity.

When $\cot\alpha >= 0.4$ eq. (33) only should be used. The limits of application of eqs. (33) and (34) are:

$$0.1 \le P \le 0.6,$$
$$0.005 \le S \le S_m \le 0.06, \quad (36)$$
$$2.0\ t/m^3 \le \rho \le 3.1\ t/m^3, \text{ and}$$
$$N_Z \le 7500,$$

where $f_2 = 1/1.4\ F/H_D > 1.5$ is the fictitious wave steepness.

Eqs. (28) and (29) where developed for non overtopped breakwaters. For low crest breakwaters, the stability of the block is increased on the sea side, this leads to a possible reduction in the sized of the blocks. Vande Meer, see Abbott[9] introduced the following reduction factor ,f_1 to be multiplied by D_{n50} from eqs (28) and (29),

$$f_1 = \left(1.25 - 4.8 \frac{F}{H_D} \sqrt{S_m / 2\pi}\right)^{-1}. \quad (37)$$

The limiting value for application of eq. (37) is

$$0 < \frac{F}{H_D} \sqrt{(S_m / 2\pi)} < 0.052 . \quad (38)$$

In contrary to the sea side of the breakwaters, where the stability increase with reducing the free board, the stability of the leeside increase with increasing the free board. Long waves may cause severe damage to the crest of the breakwaters and the upper part of the leeside. Therefore, when S_m <= 0.025 the nominal size of the primary layer of both the seaside and the harbor side must be equal. When S_m >= 0.025, the following reduction factor should be applied to the primary layer of the leeside. However in all cases the nominal size of the leeside should not be greater than that of the seaside.

$$f_2 = (0.5 - F/H_D + 0.65)^{-1}\ F/H_D \le 1.5, \quad (39\text{-a})$$

$$f_2 = 1/1.4\ F/H_D > 1.5 . \quad (39\text{-b})$$

The upper side of the toe layer is located at elevation (SWL–1.5 H_D)and that for the leeside is located at elevation SWL –H_L where H_L is given by eq. (28), for Kt > 0. If no overtopping is allowed then the elevation will be SWL. The weight W, and thickness t for each layer is given by,

$$W = \rho_r D_{n50}^3 \quad (40)$$
$$t = n K_\Delta D_{n50},$$

where;

n is the number of stones in the layer, usually 2 for cover layer and 3 for the creast

K_Δ is the packing coefficient, usually = 1.02, SPM[1]

The size of the stones of the secondary armor layer, and the core is selected according to the recommendation of SPM shown in fig. 3.

Two methods are in use to describe the amount damage of the breakwater. The first of them is called the damage number S and is given by

$$S = A_e / D_{n50}^2 , \quad (41)$$

where A_e is the eroded area. For 2< S < 3 the damage is classified as minor. The second method is called the relative damage D given by,

$$D = \frac{\text{number of displaced units}}{\text{number of units in test area}} . \quad (42)$$

The relationship between D and S is given by

$$D = 1.25\ S . \quad (43)$$

This comes from equating the no damage criterion of Van der Meer (S = 2) to the no damage criterion (D = 25). Smith[10],gave other formula to relate D and S . They depend on the porosity of the armor layer.

7. Construction and maintenance cost

The overall cost of breakwaters is divided between construction cost and maintenance cost. The computer program developed in this paper optimizes the construction cost in three different ways. Assuming the construction time is a controlling factor and the time of construction is proportional to the volume of the breakwater, then the program selects from the different alternatives the one with minimum volume regardless any other factors. The second method of optimization is to optimize the use of the quarry product. That is the percentage of stones of different sizes used in the breakwater is as close as possible to the outcome of the quarry. The last method of optimization is to minimize the cost. In the last approach it is assumed that the price of the stone is related linearly to its weight. Table 2 shows a hypothetical fragmentation table of some quarry with the price of each category.

Table 2
Fragmentation table, Bruun [11]

From	To	%	price
0.0	0.5	45	40
0.5	2.0	18	50
2.0	4.0	15	100
4.0	8.0	12	180
8.0	10.0	1040	250

The second item influencing the cost of the breakwaters is the maintenance cost. It is known that there is some probability that the design wave height will be exceeded during the life time of the breakwater. This will result in some damage to the breakwater. Table 3 shows the amount of damage when the incident waves are higher than the design wave. eq. (43) is used to convert D to S.

The data shown in table 3. can be casted in the following analytical form using regression analysis,

$$D_r = D_{H_d} \, EXP(S_r (H/H_D - 1 \, , \qquad (44)$$

where;

Dr is the damage due to incident waves with height h given a design wave H_D.

S is an empirical coefficient to fit the scale model test results.

Values for S_r are given in table 4.

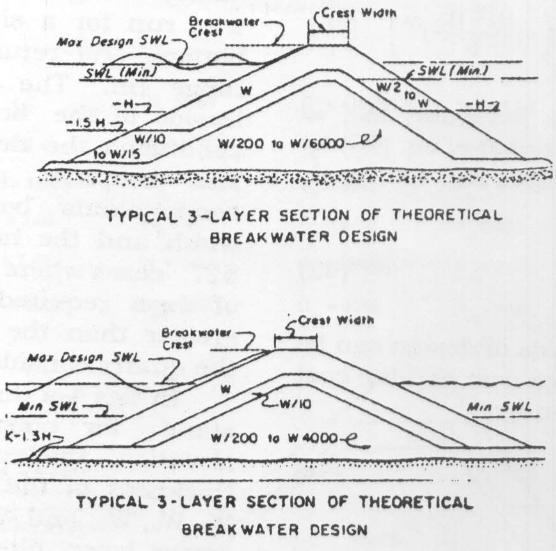

Fig. 3. Typical 3 layers design for breakwaters. a) Shallow water. b) Deepwater.

Table 3
H/H_D as a function of cover layer damage and type of armor, SPM[1]

UNITS	0-5	5-10	10-15	15-20	20-30	30-40	40-50
Quarrystone smooth	1	1.08	1.14	1.20	1.29	1.41	1.54
Quarrystone rough	1	1.08	1.19	1.27	1.37	1.47	1.56

Table 4
Coefficients for analytical prediction of breaking damage, Smith [10]

Unit	Wave Condition	% D(H_D)	S_R
Quarry stone(rough)	Non breaking	3.0	6.95
Quarry stone	Breaking	2.0	3.65

The percentage of damage per year , D_{year} , is given by,

$$D_{year} = N_S \int_{H_D}^{H_{max}} f(H)\, D_R(H)\, dH\,, \tag{45}$$

where;

N_S is the number of storms per year,

H_{max} is the upper limit of the and will be taken equal to the wave height corresponding to a return period of 1000 year,

f(H) = dF(H)/dH is the the probability density function of exceeding, where F(H) is given by (4), hence

$$f(H) = (m/\beta)\frac{(H_S - H_O)^{m-1}}{\beta}\ EXP\left(-[\frac{H_S - H_O}{\beta}]^m\right). \tag{46}$$

The maintenance cost per year, M_C, is given by multiplying the result of eq. (40) by the volume V of the cover layer and the repair cost per unit volume M_U,

$$M_C = M_U\ V\ D_{year}\,. \tag{47}$$

The different alternatives of design can be checked for the maintenance cost and the cost of the breakwater A_C is given by,

$$A_C = C_C + N_Y * M_C\ , \tag{48}$$

where ;

C_C is the construction cost, and

N_Y is the expected life of the breakwater.

If the cash needed for maintenance is deposited with annual interest i, so that the outcome per year is Mc, then the total cost is given by,

$$A_C = C_C + M_C\frac{(i+1)^{N_Y} - 1}{i\,(1+i)^{N_Y}}\,. \tag{49}$$

Where;

C_C is the construction cost, and

N_Y is the expected life of the breakwater.

If the cash needed for maintenance is deposited with annual interest i, so that the outcome per year is Mc, then the total cost is given by,

$$A_C = C_C + M_C\frac{(i+1)^{N_Y} - 1}{i\,(1+i)^{N_Y}}\,. \tag{50}$$

A more general way to measure the damage that takes into account the storm duration is given by Smith [10].

8. Case study

Figs. 4-a through 4-d show the four different designs optimized for volume, construction cost, maintenance per storm and quarry usage. For illustration, the program was run for a site with d=5.5 m, with flat bottom, and return period 25 years and tidal range 1m. The allowed wave height in the leeside of the breakwater is 0.5 m. For those conditions the design wave height is 4.84 m. The computer program gives 806 different combinations between the face slope, crest width and the height of the breakwater. Only 527 cases where considered, because the rest of them required armor layer stones with size greater than the maximum size available in the quarry considered.

In figs 4-a through 4-d, B,S,H, Ta, and T_f, stand for crest width, face slope, crest elevation, thickness of the armor layer and thickness of the filter layer, respectively. W_a, W_f, W_c, W_b, and W_t represent the weight of the armor layer, filter layer, core, berm and the total weight per meter. respectively. While C_a, C_f,C_c, C_b, and C_l represent the cost of the corresponding parts and, C_m gives the maintenance cost per a extreme single storm.

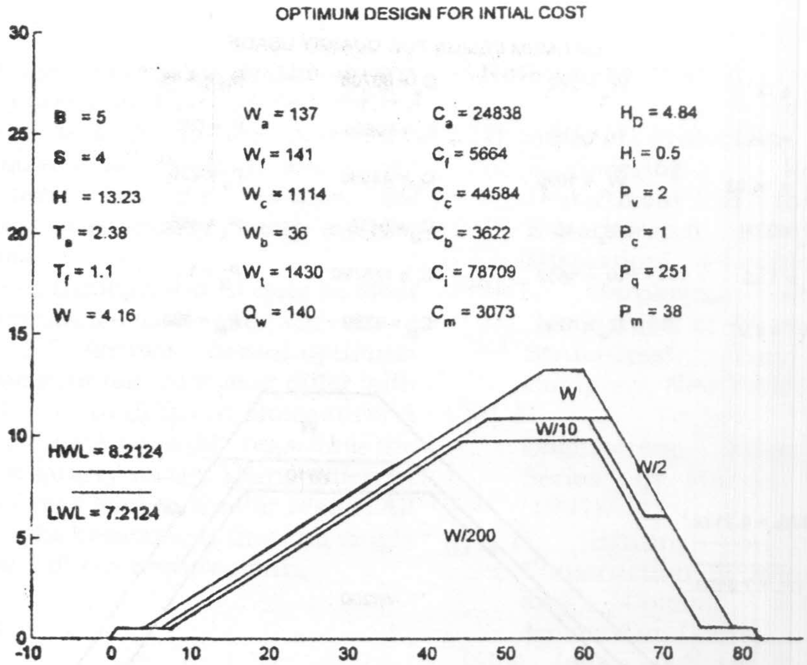

Fig. 4-a. Optimum design for initial cost.

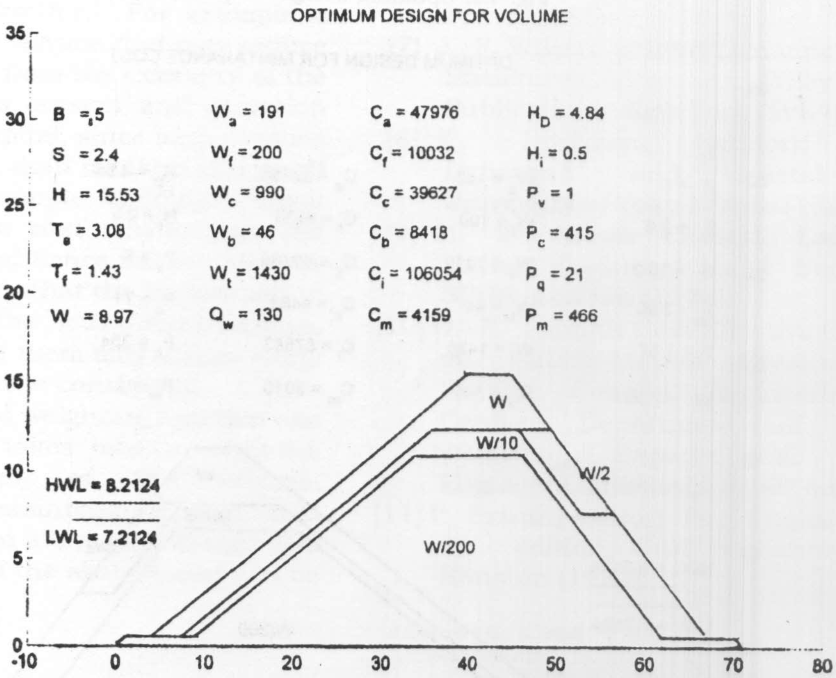

Fig. 4-b. Optimum designs for volume.

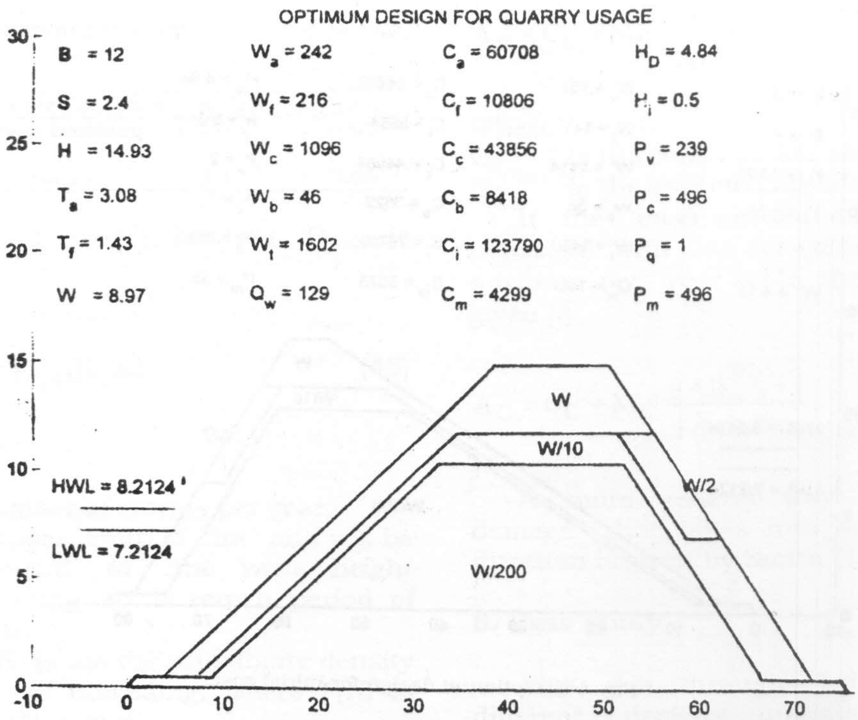

Fig. 4.c. Optimum design for quarry usage.

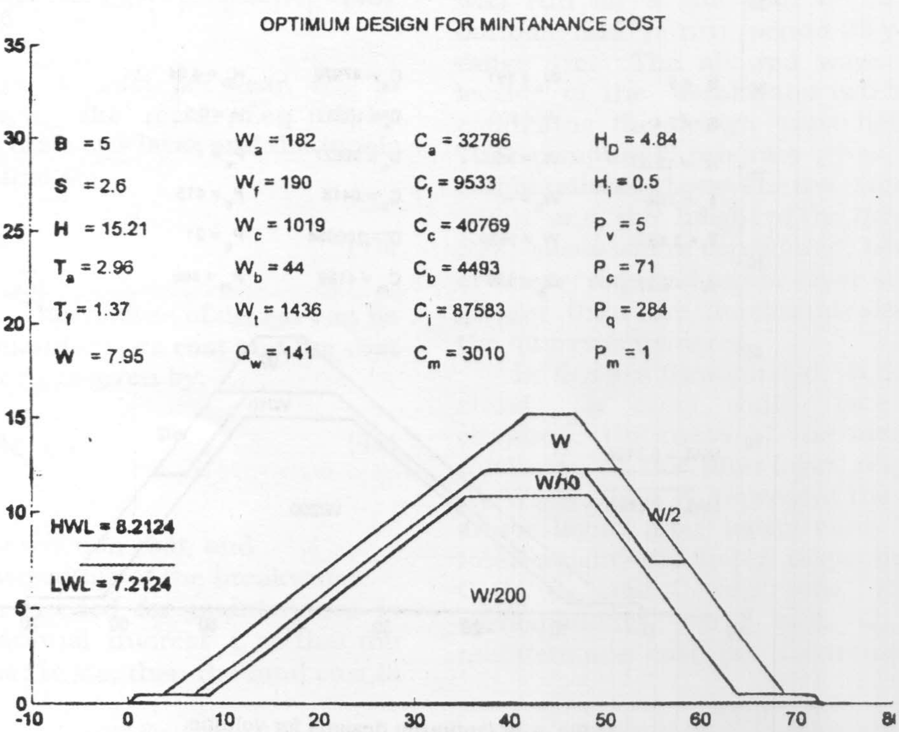

Fig. 4.d. Optimum design for maintenance cost.

Q_w represents the percentage of the quarry production in excess of that needed for the breakwater. The less the Q_w the better the usage of the quarry is. P_v ,P_c ,P_q and P_m are the priority level for the volume, the construction cost, the quarry usage and the maintenance cost.

From figs. 4-a through 4-d, it may be clear that, the maintenance cost may vary by a factor close to 1.5 for two different optimum designs The construction cost may differ with a factor up to 1.7 for to different alternative. A similar argument can be said regarding the volume and the quarry usage. Using different fragmentation curves lead to similar result. All of them lead to the conclusion that, no single design will satisfy all the requirements

9. Conclusions

Due to the huge number of alternative designs, that satisfy the breakwater functional and structural criteria, a lot of caution must be applied to select the proper design of the breakwater. For example a design for minimum volume that may reduce side effects resulting from the existence of the breakwater, such as erosion and accretion may require a lot capital, since bigger stones are more expensive than medium and small ones. As another example building a break water with minimum cost may increase the maintenance cost, and hence the overall cost. It is not by necessity that the breakwater to be build is one of the four optimized ones, since selecting one of them may violate many other factor that must be considered.

If a user specified weighting function can be introduced, that takes into account the relative importance of the volume, construction cost, maintenance cost and quarry usage, then an alternative design that take into account all the above factor can be selected

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