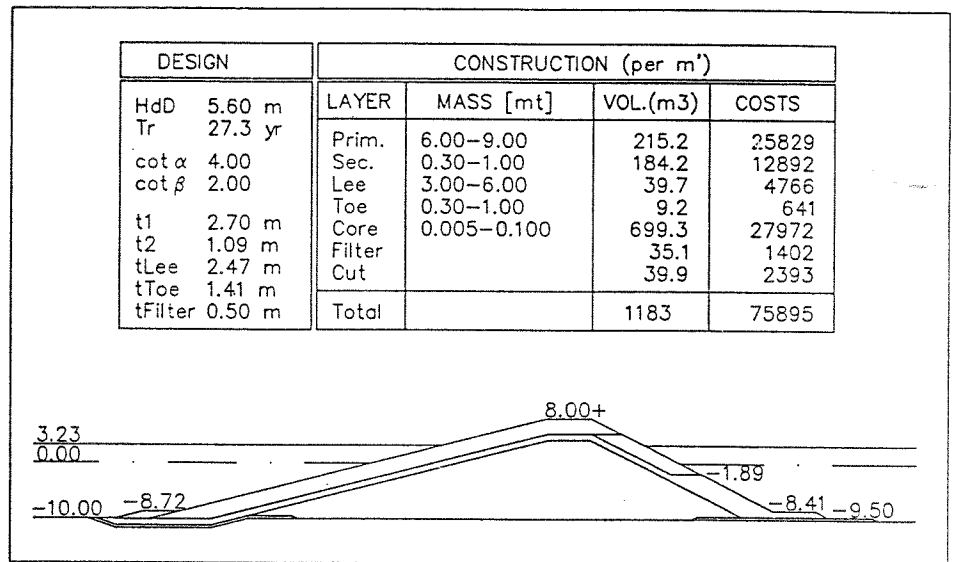


Computer Aided Optimum Design of Rubble-Mound Breakwater Cross-Sections

Manual of the Rumba Computer Package, Release 1

May 1989

Wiebe de Haan



COMPUTER AIDED OPTIMUM DESIGN OF RUBBLE-MOUND BREAKWATER

CROSS-SECTIONS

Manual of the RUMBA Computer Package, Release 1.

Wiebe de Haan

Delft University of Technology
Faculty of Civil Engineering
Hydraulic and Geotechnical Engineering Division
Hydraulic Engineering Group
P.O.Box 5048
2600 GA DELFT
The Netherlands

May 1989

ABSTRACT

The computation of the optimum rubble-mound breakwater cross-section is executed on a micro-computer. The RUMBA computer package consists of two main parts: the optimization process is executed by a Turbo Pascal programme, the second part consists of editing functions written in AutoLISP. AutoLISP is the programming language within AutoCAD.

The quarry production, divided into a number of categories, and long-term distributions of deep water wave heights and water levels, form the basis of the computation. Concrete armor units have been excluded from the computation. Deep water wave heights are converted to wave heights at site. A set of alternative cross-sections is computed based on both functional performance criteria, and Van der Meer's stability formulae for statically stable structures. Construction costs and maintenance costs are determined of each alternative. The optimum is derived by minimizing the sum of the construction costs and maintenance costs. Moreover, the programme provides means to economize the use of the quarry.

At this stage the computer programme is useful for feasibility studies of harbour protection or coastal protection in regions, where use can be made of a quarry in the neighbourhood of the project site and the use of concrete armor units is excluded in advance. Briefly a method is described to extend the computer programme to the use of concrete armor units.

TABLE OF CONTENTS

ABSTRACT	ii
TABLE OF CONTENTS	iii
ACKNOWLEDGEMENT	v
1. Introduction	1
2. Theoretical background of the computer package	2
2.1. Introduction	2
2.2. Long-term distributions of waves and water levels	3
2.3. Wave conditions at the breakwater site	4
2.4. Structural design criteria	6
2.5. General lay-out of the cross-section	6
2.6. Computation of the cross-section	7
2.6.1. Crest elevation	7
2.6.2. Primary armor layer	8
2.6.3. Computation of the other layers	10
2.7. Construction costs	11
2.8. Maintenance costs	12
2.9. Optimum design conditions	13
2.10. Adjustments to the cross-section by means of AutoCAD	15
2.11. Conclusions	16
3. General features of the RUMBA computer package	18
3.1. General scheme and contents of the programmes	18
3.2. Hardware and software requirements	19
4. Description of OPTRUMBA.PAS	20
4.1. General flow scheme	20
4.2. Main Menu of OPTRUMBA.PAS	21
4.3. Input of data	22
4.3.1. Option "P": Input of physical data	22
4.3.2. Option "Q": Input of quarry data	24
4.3.3. Option "S": Input of structural data	26
4.4. Option "R": Computation of alternative cross-sections	30
4.4.1. Development of a number of alternative cross-sections	30
4.4.2. Design storms	32

4.4.3.	Crest elevation	32
4.4.4.	Primary armor layer	34
4.4.5.	Lee armor layer	35
4.4.6.	Secondary armor layer	36
4.4.7.	Crest widths	37
4.4.8.	Toe protection	38
4.4.9.	Construction costs	38
4.4.10.	Maintenance costs and total costs	41
4.5.	Output of results	43
4.5.1.	Option "O": Output on screen	
	Option "L": Output on printing-paper	43
4.5.2.	Option "C": Cost curves	46
4.4.3.	Option "A": Structural data of optimum design to AutoCAD	48
5.	Description of the programme CADRUMBA.LSP	49
5.1.	The RUMBA.DWG prototype drawing	49
5.2.	Drawing of the optimum cross-section	50
5.3.	Effects of changing values of geometrical variables	52
5.3.1.	Useful features of the AutoLISP programming language	52
5.3.2.	Effects of changing values of geometrical variables	52
5.4.	Changes to the cross-section by means of EDIT commands	56
6.	Description of CRVRUMBA.LSP	59

LIST OF SYMBOLS

LIST OF REFERENCES

ACKNOWLEDMENT

The author gratefully acknowledges prof.dr.ir. E.W. Bijker, ir. M.H. Lindo, dr.ir. J.W. van der Meer, ir. W. Meermans, and drs.ir. J.K. Vrijling.

Their critical comments and constructive suggestions have enhanced the quality of the contents of the computer programme.

1. Introduction

Lately the use of micro-computers has increased tremendously. At the same time developments in the area of Computer Aided Design (CAD) have led to an increase of the performance/price ratio of CAD-software packages. These packages are employed more and more in the hydraulic engineering practice. However, usually CAD is used for drafting purposes only.

The present manual describes the use of the RUMBA (from Rubble Mound BreakwAter) computer package, which computes the optimum rubble-mound breakwater cross-section based on a minimum of total costs over the lifetime of the structure. This package has been developed during a study on the application of CAD within the design of breakwaters. Use has been made of the CAD-software package AutoCAD Release 9.

The theoretical background of the programmes is explained in chapter 2. Moreover the contence of chapter 2 is composed in such a way that it can be published as a paper in "Coastal Engineering" as well. Therefore some sentences written in this chapter are sometimes repeated elsewhere in the manual. Chapter 3 describes the general scheme of the RUMBA package, which mainly consists of two parts. The first part consists of the optimization written in Turbo Pascal (chapter 4). The second part, described in chapter 5, consists of functions which allow any adjustments to the cross-section. These functions are written in AutoLISP, the programming language embedded within AutoCAD. The last chapter describes an AutoLISP programme, which is able to draw the costs curves for presentation purposes.

2. Theoretical background of the computer package.

2.1. Introduction.

Hydraulic structures, and breakwaters in particular, can be characterized as unique. Due to its unique character it is difficult to automatize the design of breakwaters by means of the computer. This is the main reason why developments of Computer Aided Design (CAD) are hardly applied in the engineering practice of breakwater design.

This chapter describes the results of a study, in which the computation of the economic optimum rubble-mound breakwater cross-section is executed on a micro-computer. The secondary object of the study was to determine to what extent CAD-software can contribute to the design of rubble-mound breakwaters. Use has been made of the software package AutoCAD Release 9.

Optimization requires a balance of functional effectiveness (benefits) against structural costs, consisting of construction costs and maintenance costs. The benefits consist of the decline of the expected damages or tangible losses to property or operations which the breakwater is intended to protect. Smith (1987) described a systematic cost-effective optimization procedure in which all these costs items are represented. The economic benefits, however, are very site specific and therefore very difficult to express in a general computer code.

The RUMBA computer package computes the optimum rubble-mound breakwater cross-section by minimizing the sum of construction costs and maintenance costs over the lifetime of the structure. A similar approach was used by Le Mehaute and Wang (1984). Inherent in the construction of rubble mounds is the production of the quarry. To avoid excessive quarry blasting, the categories of rock required for the structure should match the quarry fragmentation curve as closely as possible (Bruun, 1985). At this stage of the development of the RUMBA-package, use of concrete armor units cannot be processed yet, but can be added easily.

Hydraulic stability and functional performance of the structure are based on the physical conditions at the proposed breakwater site.

2.2. Long-term distributions of waves and water levels.

In long-term statistics all data are used to predict the tail of the distribution where the extremes are located. It is assumed that a continuous record of significant wave heights and wave peak periods has been collected at a deep water site near the design location. Often these values are determined from observation periods of $\Delta t = 3, 6, \text{ or } 12$ hours. The significant wave height is defined as the average of the largest one-third of

the wave heights within the period Δt . Concerning the type of distribution, it has been shown that the best fits are usually obtained by the lower bound Weibull distribution (Houmb, 1981). Adopting the same kind of notation as used by Isaacsen and Mackenzie (1981), the lower bound Weibull distribution can be written as

$$P(H_{sD}) = \text{Prob}(H_{sD} \leq H_{sD}) = 1 - \exp\left[-\left(\frac{H_{sD}-\epsilon}{\theta}\right)^\gamma\right] \quad (2.1)$$

where:

- H_{sD} = deep water significant wave height
- \underline{H}_{sD} = randomly chosen value of H_{sD}
- ϵ = location parameter
- θ = scale parameter
- γ = shape parameter

An example of the Weibull distribution is illustrated in Fig.2.1. The location parameter, ϵ , is the lower bound value of H_{sD} . Hence, $H_{sD} \geq \epsilon$. The shape parameter, γ , usually varies from 0.9 - 1.4. In the example $\gamma = 1.0$ is assumed.

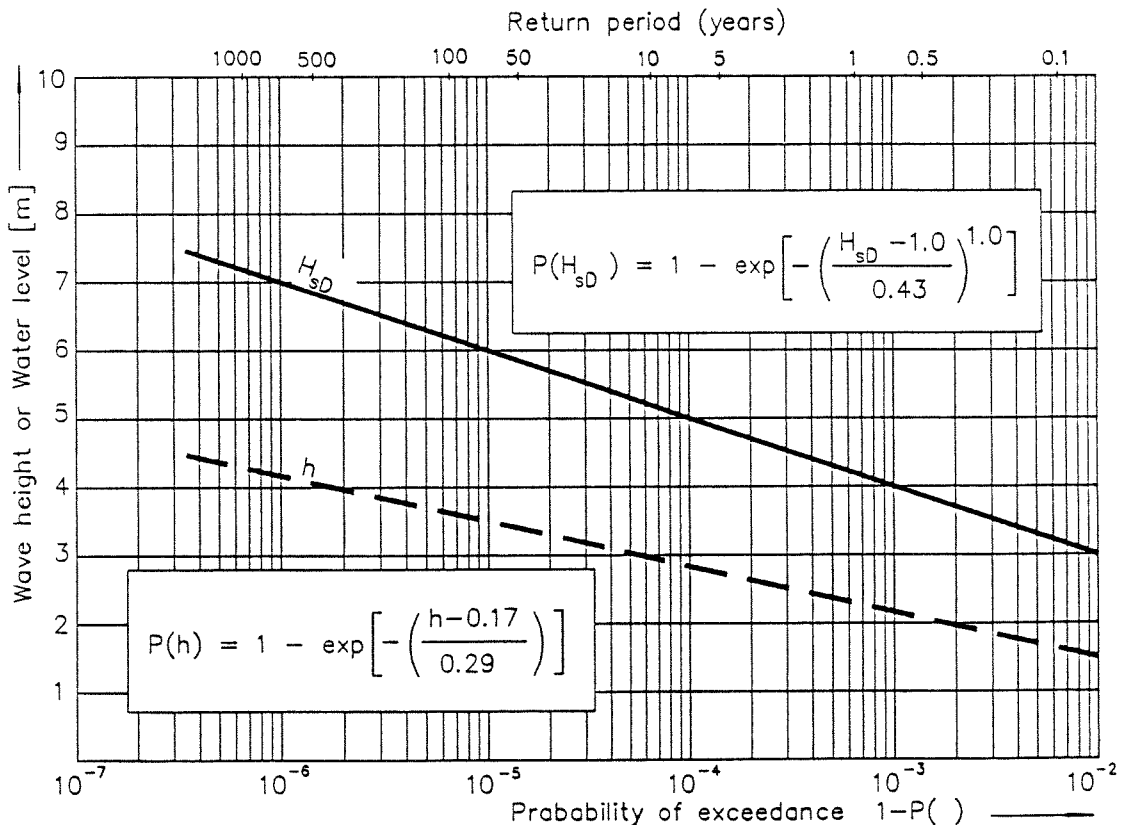


Fig.2.1. Examples of long-term distributions of wave heights and water levels.

In civil engineering practice probabilities are often expressed in terms of return periods, T_r

$$T_r = \frac{1}{\nu * [1 - P(H_{SD})]} \quad (2.2)$$

where $\nu = 24*365/\Delta t$ is the number of storm events per year. Thus, "one storm" lasts Δt hours.

A record of storm water levels, including high tide as well as wind and wave set-up, is required at the proposed breakwater site. The plotted data usually fit the Weibull distribution, where γ has a value close to 1.0. For $\gamma = 1.0$ the Weibull distribution is equal to the exponential distribution, written as

$$P(h) = \text{Prob}(h \leq h) = 1 - \exp\left[-\left(\frac{h - \epsilon}{\theta}\right)\right] \quad (2.3)$$

where h = storm water level.

An example of the exponential distribution of h is also illustrated in Fig.2.1. Moreover return periods are shown which were determined for $\Delta t = 6$ hrs. It is assumed further that both the storm water levels and storm waves occur simultaneously.

2.3. Wave conditions at the breakwater site.

From the so-called "scatter" diagram a relation between the peak period, T_p , and H_{SD} has to be derived. The peak period is defined as the period of maximum energy density of the wave spectrum. In a general form the relation can be written as

$$T_p = a * H_{SD}^c + b \quad (2.4)$$

where a , b , and c are coefficients.

An important parameter in the hydraulic stability of a particular structure is the wave climate at site. Therefore the deep water wave data have to be transformed to local conditions. Neglecting the influences of refraction and diffraction, the transformation is determined by wave breaking and shoaling.

From linear wave theory the shoaling coefficient K_{sh} is defined as (Shore Protection Manual, SPM, 1984)

$$K_{sh} = \left[\tanh(kd) * \left(1 + \frac{2kd}{\sinh(kd)} \right) \right]^{-1/2} \quad (2.5)$$

where:

$$\begin{aligned} k &= 2\pi/L_D &&= \text{wave number} \\ L_D &= (g/2\pi) * T_m^2 &&= \text{deep water wave length} \\ g &= &&= \text{gravitational acceleration} \\ d &= d_m + h &&= \text{depth at site} \end{aligned}$$

The mean sea level (MSL) is adopted as the level of reference. Therefore the depth at site is the sum of mean depth d_m and water level h . According to the SPM deep water conditions are valid for $d > L_D/2$. The use of the mean wave period, T_m , in the formula will be in line with the use of the same characteristic wave period in the run-up and stability equations used lateron. The mean wave period is a fixed fraction of the peak period depending on the shape of the wave spectrum.

Wave breaking is determined by the US-CERC criterion, which gives the most reasonable estimate of the breaker index γ_{br} over a wide range of conditions (Gaillard, 1988)

$$\gamma_{br} = B * \left[1 + \frac{A*d}{L_D} \right]^{-1} \quad (2.6)$$

where empirical coefficients:

$$\begin{aligned} A &= 6.963 * [1 - \exp(-19*i_b)] \\ B &= 1.56 * [1 + \exp(-19.5*i_b)]^{-1} \end{aligned}$$

The bottom slope is represented by i_b . The value of the local significant wave height H_{sL} is computed using either

$$H_{sL} = \gamma_{br} * d \quad (2.7)$$

or

$$H_{sL} = K_{sh} * H_{sD} \quad (2.8)$$

whichever yields a smaller value.

2.4. Structural design criteria.

A number of alternative cross-sections is developed, each of which is characterized by a deep water design significant wave height, H_{dD} , and a seaward slope angle, $\cot\alpha$. The subscript d in H_{dD} is used to distinguish the design significant wave height from the generally used significant wave height H_{SD} . Physically both wave heights are totally equal. The design of each alternative is based on two types of criteria

- a) functional performance
- b) hydraulic stability.

The functional performance criterion, which will be equal for all alternatives, imply an upper bound transmitted significant wave height H_t at the lee side of the structure during a storm of certain return period. For example, a significant wave height of 0.30 m might be a treshold value for damage to vessels moored behind a breakwater. In case this situation may happen statistically with a recurrence interval of 25 years, the functional criterion has to read $H_t = 0.30$ m and $T_r = 25$ yrs.

The "no damage" criterion will be the design objective for hydraulic stability of each alternative.

2.5. General lay-out of the cross-section.

Each alternative is constructed of several layers of quarry stone, as illustrated in Fig. 2.2. The masses indicated represent the 50% values of the mass distribution curve of the stones in the particular layer. The cross-section is basically the same as the recommended three-layer section shown in the Shore Protection Manual (SPM), 1984. The core of quarry run stones is covered with the secondary armor layer of larger stones, and the exterior primary armor layer of very large quarry stones. It is specifically stated that in the current programme concrete armor units cannot be applied yet. The toe construction consists of light armor units to support the lower portion of the primary

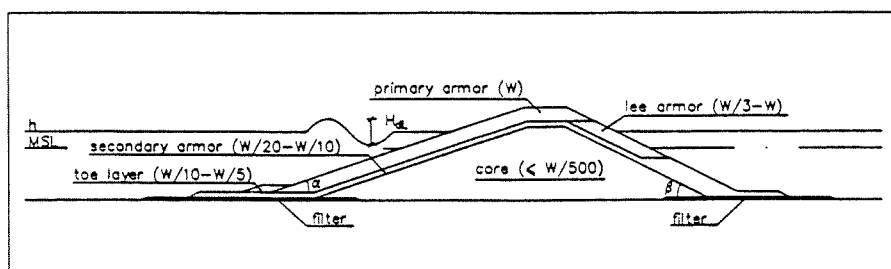


Fig.2.2. General lay-out of the cross-section.
armor layer and protect the underlying extension of the secondary

armor layer from direct wave attack. The toe protection is constructed after the lower part of primary armor stones has been placed.

The crest of the breakwater consists of an integer number of (at least three) primary armor blocks. The mass of the lee side armor stone is dependent on the expected rate of wave overtopping: the lesser the overtopping, the lesser the mass. Loss of easily erodable material from the natural bottom is prevented by the undermost filter layers.

2.6. Computation of the cross-section.

2.6.1. Crest elevation.

At first instance the value of $\cot\alpha$ is used for the computation of the breakwater crest elevation. This elevation will be based on the functional performance criterion. Neglecting wave transmission through the breakwater, the transmitted wave height is dominated by wave overtopping. This restriction was also valid for the experiments executed by Hwang and Tang (1988). By curve fitting they have developed linear and quadratic relations between the transmission coefficient K_t and the ratio F/R_s , in which F represents the freeboard or the height of the crest above still water level (SWL), and R_s the significant run-up. The linear significant relation is expressed as

$$K_t = H_t/H_{sL} = 0.478 * (1.16 - F/R_s) \quad (2.9)$$

Significant run-up relations were developed at Delft Hydraulics (Stam, 1988)

$$R_s/H_{sL} = 0.72 * \xi_m \quad \text{for} \quad \xi_m \leq 1.5 \quad (2.10.a)$$

$$R_s/H_{sL} = 0.88 * \xi_m^{0.41} \quad \text{for} \quad 1.5 < \xi_m \leq 2.8 \quad (2.10.b)$$

$$R_s/H_{sL} = 1.35 \quad \text{for} \quad \xi_m > 2.8 \quad (2.10.c)$$

where:

$$\xi_m = \tan\alpha / \sqrt{s_m} \quad = \text{surf similarity parameter}$$

$$s_m = H_{sL} / (g/2\pi) T_m^2 \quad = \text{fictitious wave steepness}$$

The wave overtopping characteristics, corresponding with the functional T_r -period, are determined by successively solving eqs. (2.2), (2.1) and (2.3), (2.7) and (2.8), and (2.10). Freeboard F remains as the only unknown parameter in eq. (2.9). Finally, the crest elevation is determined by the sum $h+F$.

2.6.2. Primary armor layer

The design of the primary armor layer is based on the criterion of hydraulic stability. Use of the Van der Meer design formulae for statically stable structures (1988-1) requires both $\cot\alpha$ and H_{dD} . Depending on the breaker type, these formulae are

for plunging waves $\xi_m \leq \xi_{m,i}$:

$$H_{dL}/\Delta D_{n50} * \sqrt{\xi_m} = 6.2 * P^{0.18} * (S/\sqrt{N})^{0.2} \quad (2.11)$$

for surging waves $\xi_m > \xi_{m,i}$:

$$H_{dL}/\Delta D_{n50} = 1.0 * P^{-0.13} * (S/\sqrt{N})^{0.2} * \sqrt{(\cot\alpha)} * \xi_m^P \quad (2.12)$$

where:

- D_{n50} = nominal diameter of the stone
- Δ = relative mass density $(\rho_r - \rho_s)/\rho_s$
- ρ_r, ρ_s = mass density of stone and sea water, respectively
- P = permeability of the structure
- S = damage level A/D_{n50}^2
- A = erosion area in a cross-section
- N = number of waves $\Delta t/T_m$

The transition from plunging to surging waves is described by the intersection of both formulae:

$$\xi_{m,i} = \left[6.2 * P^{0.31} * \sqrt{\tan\alpha} \right]^{1/P+0.5} \quad (2.13)$$

The permeability may vary from $P = 0.1$ for the impermeable core to $P = 0.6$ for the homogeneous structure. Reasonable P -values for the basic structure of Fig. 2.2. are to be found in the range 0.4 - 0.5.

The equations above were developed for non-overtopped structures, exclusively constructed of rock material. For low breakwater crests, however, an increase of stability is gained on the sea side of the structure. Increase of stability can be transformed into a reduction of the size of the primary armor nominal diameter. In formulae (Van der Meer, 1988-2):

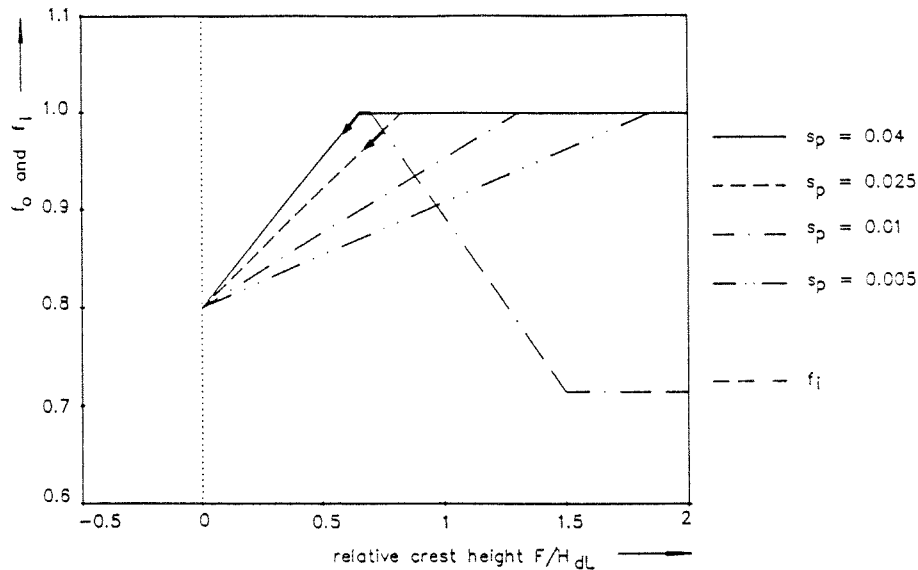


Fig.2.3. Armor reductions as function of the relative crest height.

$$f_o = 1/(1.25 - 4.8R^*) \quad \text{for } 0 < R^* < 0.052 \quad (2.14.a)$$

$$f_o = 1 \quad \text{for } R^* \geq 0.052 \quad (2.14.b)$$

where:

$$f_o = D_{n50}(\text{overtopped})/D_{n50}(\text{non-overtopped})$$

$$R^* = F/H_{dL} * \sqrt{(s_p/2\pi)}$$

$$s_p = H_{dL}/(g/2\pi)T_p^2$$

The relation is illustrated in Fig. 2.3. Note that T_p has been used in the formula of R^* . Substituting eq. (2.14) in eq. (2.11) or eq. (2.12), the coefficient f_o appears in the nominator left from the equation sign.

The criterion of "no damage" will be accomplished by $S = 2$. However, for slopes of $\cot\alpha > 3$, even $S = 3$ may be allowed. After having determined the breaker type by means of eq. (2.13), the nominal diameter D_{n50} , including reduction factor f_o , is the remaining unknown parameter in either eq. (2.11), or eq. (2.12). Next the average mass W of the primary armor stone and the thickness, t , of the armor layer are derived from, respectively,

$$W = \rho_r * D_{n50}^3 \quad (2.15)$$

and

$$t = n * k_\Delta * D_{n50} \quad (2.16)$$

where the number of armor stones in the primary layer usually is $n = 2$, and the packing coefficient $k_{\Delta} = 1.02$, which corresponds with the value of random placement of quarry stone.

Prior to the computation a maximum mass value W_{\max} was given. This value can be set in accordance with the availability of heavy stones from the quarry. Whenever $W > W_{\max}$, the alternative will be rejected.

2.6.3. Computation of the other layers.

The average quarry stone mass of the secondary and toe layer is related to the mass applied in the primary layer (see Fig. 2.2). Solving eqs. (2.15) and (2.16), the thickness of both layers can be computed.

As opposed to the stability of the sea side of the structure, an increase of stability of the lee side is gained for relatively high breakwater crests. However, some caution is required as no, or hardly any, experiments have been carried out on this matter. Especially long waves may cause havoc on the breakwater crest and the upper lee side due to overtopping. Therefore for incident waves of $s_p < 0.025$ the nominal diameter of the lee armor blocks will be equaled to the nominal size of the primary armor blocks, whatever the height of the crest. For $s_p \geq 0.025$ a reduction of the lee armor D_{n50} , as function of the relative crest height F/H_{dL} , will be allowed (see also Fig. 2.3)

$$f_i = 1/(0.5 \cdot F/H_{dL} + 0.65) \quad \text{for } F/H_{dL} \leq 1.5 \quad (2.17.a)$$

$$f_i = 1/1.4 \quad \text{for } F/H_{dL} > 1.5 \quad (2.17.b)$$

where $f_i = D_{n50}(\text{lee, overtopped})/D_{n50}(\text{primary, non-overtopped})$.

To avoid that the lee armor D_{n50} becomes greater than the D_{n50} of the primary armor, values of $f_i > f_o$ will not be accepted. In that case f_i is equaled to f_o , which has been illustrated by the arrows in Fig. 2.3. Whenever these nominal diameters, and consequently the average masses, are found to be equal, the hydraulic stability of the lee side has to be verified by other means. The leeward slope angle $\cot\beta$ is fixed and will not be changed during the entire computation.

A number of storm conditions of return periods smaller than or equal to the return period corresponding with H_{dD} , during high tide as well as low tide, are considered to determine the elevation of the upper side of the toe layer. Its level will be represented by the lowest value of $SWL - 1.5 \cdot H_{sL}$. A similar approach is used for the lower bound level of the lee armor layer, where H_t is computed using eq. (2.9) for each separate storm condition. The lowest level from either $SWL - H_t$ for $K_t > 0$, or SWL , if no overtopping occurs, will be appointed to the elevation of the underside of the lee armor layer.

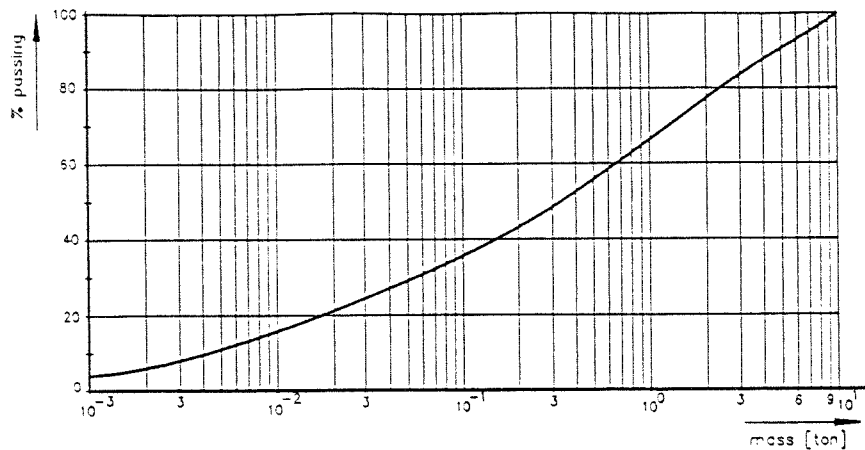


Fig.2.4. Quarry fragmentation curve.

2.7. Construction costs.

The construction costs, $\$C$, of an alternative per meter run are simply derived by multiplying the volume of each construction layer by the unit price of the quarry material applied. The quarry production is therefore divided into a number of successive mass categories. Figure 2.4 shows a fragmentation curve indicating the percentage distribution of blocks the quarry is able to supply. A number of mass categories is deduced from the curve of which the characteristics are collected in Table 2.1. Each category is labelled by a unit price comprising all costs of blasting, selection, transport, stock-piling, and placing of the material per cubic meter. A certain quarry category will be allocated to a construction layer when the required average mass value of the layer in question lies between the upper and lower limit mass values of that category. The unit price of the filter material is assigned separately.

Cat.	Mass range [ton]	Min. pass. [%]	Max. pass. [%]	Yield [%]	Unit price [-]
1	6.000 - 9.000	93.0	98.0	5.0	120.0
2	3.000 - 6.000	84.0	93.0	9.0	120.0
3	1.000 - 3.000	66.0	84.0	18.0	90.0
4	0.300 - 1.000	48.0	66.0	18.0	70.0
5	0.100 - 0.300	34.0	48.0	14.0	70.0
6	0.005 - 0.100	5.0	34.0	29.0	40.0
7	0.000 - 0.005	0.0	5.0	5.0	40.0

Table 2.1. Quarry yield and unit price per category.

2.8. Maintenance costs.

At the design damage level $S = 2 - 3$, minor settlements will have occurred near the water line. These settlements will not give rise to particular concern yet. Storms of greater intensity than the design storm, however, will lead to loss of material, which has to be replaced afterwards. The expected maintenance costs for each storm period of Δt , $E(\$_M)_{\Delta t}$, can be expressed as

$$E(\$_M)_{\Delta t} = \int_{H_{dD}}^{\infty} p(H_{SD}) * \$_{Dam}(H_{SD}) * dH_{SD} \quad (2.18)$$

where:

- $p(H_{SD})$ = $dP(H_{SD})/dH_{SD}$ = probability that H_{SD} occurs during a storm period Δt
 $\$_{Dam}(H_{SD})$ = costs of repair of damage caused by a storm of significant wave height H_{SD} .

The integral, which corresponds with the hatched area below the probability density curve in Fig. 2.5, is solved numerically. Infinity is approximated by H_{SD} caused by a storm of return period $T_r = 10,000$ yr. The derivative of the Weibull cumulative distribution reads

$$p(H_{SD}) = \frac{\alpha}{\theta} * P(H_{SD}) * \exp \left[- \left(\frac{H_{SD} - \epsilon}{\theta} \right)^{\gamma - 1} \right] \quad (2.19)$$

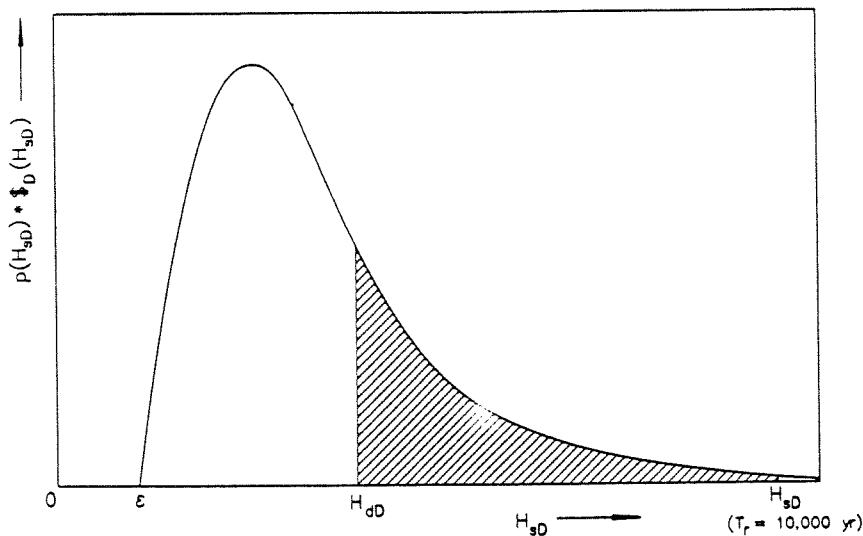


Fig.2.5. Weibull probability density curve.

The repair costs, term $\$_{\text{Dam}}(H_{\text{SD}})$, are directly related to the damage level S , which is the only unknown parameter in either eq. (2.11) or eq. (2.12). For that purpose H_{dL} is replaced by H_{sL} , and the value of D_{n50} is equal to the design nominal diameter of the primary armor stone. Next the damage level is transformed into a percentage of damage, $\text{Dam}\%$, by means of $\text{Dam}\% = 1.25 * S$. Factor 1.25 is derived from the assumption of equivalence of Van der Meer's "no-damage" criterion $S = 2$, to the SPM "no-damage" criterion of $\text{Dam}\% = 2.5\%$. Eventually the repair costs are built up of two parts:

- a) mobilization costs of equipment assumed to be 5% of the total construction costs
- b) costs of filling up the gaps of the structure, depending on the percentage of damage $\text{Dam}\%$.

Assuming no correlation between subsequent storm events and repair between storms causing significant damage, the expected yearly maintenance costs, $E(\$_{\text{M}})_{\text{yr}}$, can be expressed as

$$E(\$_{\text{M}})_{\text{yr}} = \nu * E(\$_{\text{M}})_{\Delta t} \quad (2.20)$$

In order to compare these annual maintenance costs to the initial construction costs, $\$_{\text{C}}$, it is necessary to determine what sum of money, $\$_{\text{M}}$, set aside now at compound interest i , will just pay for this maintenance over the life-time of the structure. Hence,

$$\$_{\text{M}} = (\$_{\text{M}})_{\text{yr}} * \left[\frac{(1+i)^L - 1}{i * (1+i)^L} \right] = (\$_{\text{M}})_{\text{yr}} * \text{pwf} \quad (2.21)$$

where:

- i = interest rate i per year expressed as a decimal
- L = expected life-time of the structure in years
- pwf = present worth factor.

At the end of the computation the sum of construction costs, $\$_{\text{C}}$, and the capitalized maintenance costs, $\$_{\text{M}}$, yields the total costs, $\$_{\text{T}}$, of the alternative.

2.9. Optimum design conditions.

Construction costs, capitalized maintenance costs, and the total costs, are plotted in a graph, in which the costs versus deep water design wave height is illustrated. Smoothly drawn curves represent the path of costs of one particular design outer slope. Fig. 2.6. shows the curves based on the long-term distributions of Fig. 2.1, and the quarry data of Table 2.1. The coefficients of eq. (2.4) were set at $a = 5.50$, $b = 7.54$, and $c = 1.00$, and $T_p/T_m = 1.20$. A set of curves, belonging to an outer slope, is

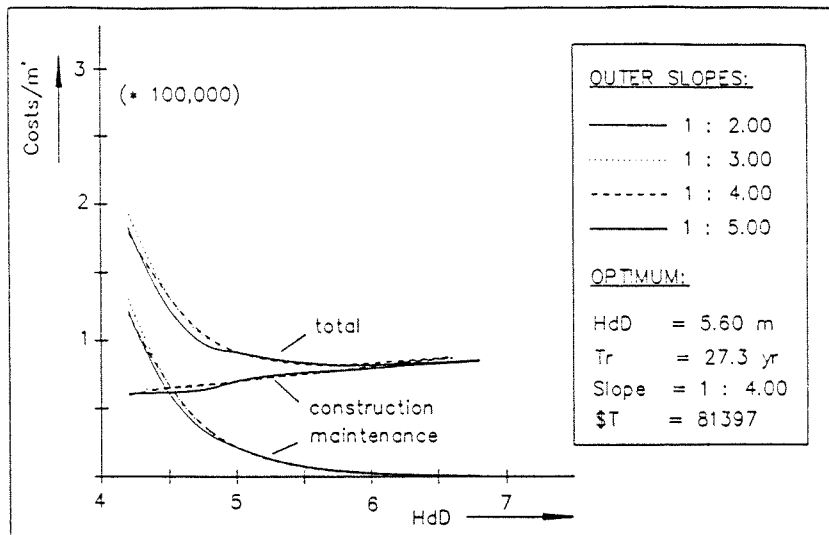


Fig.2.6. Costs curves with optimum design conditions.

cut off as soon as the required average mass of the primary armor becomes greater than the value of W_{\max} . In the example, where W_{\max} was fixed at 9.0 ton, the $\cot\alpha = 2.00$ slope is too steep to attain hydraulic stability. Therefore, the costs curves belonging to this slope do not appear at all in Fig. 2.6. The optimum design conditions are represented by the lowest value of all total costs values computed. These conditions are shown in the table of Fig. 2.6 as well.

Many examples with a wide variety of input data have already been examined. Each example showed the maintenance curves lying very close to each other, and showed a relatively small increase of construction costs versus H_{dD} . This pattern is due to the resemblance of each alternative to the basic cross-section of Fig. 2.2, and, for the greater part, due to the fact that concrete armor units have been excluded from the computation. The optimum design conditions tend towards gentle outer slopes at design storms of large recurrence interval. The preference of gentle slopes result from the relatively large increase of stability for the lower values of ξ_m (eq. 2.11).

Since the total costs curves show a rather flat pattern for the greater values of H_{dD} , an alternative based on conditions "close to" the optimum might as well be selected as the optimum. In order to make a responsible design, the final choice should also be based on the economic use of the quarry. For that purpose the computer programme calculates, for each alternative cross-section, what percentage of each quarry category is required for the structure. The required percentages are accordingly compared with the yield percentages of the quarry. The greatest value of the ratio of Required[%] to Yield[%] determines the factor with which the quarry production has to be multiplied in order to meet the requirements of the construction. The resulting excess production, which is gross production minus required production,

Cat.	Mass range [ton]	Requir. [m ³ /m']	Requir. [%]	Yield [%]	Gross [%]	Excess [%]
1	6.000 - 9.000	215.2	18.8	5.0	18.8	0.0
2	3.000 - 6.000	39.7	3.5	9.0	33.8	30.3
3	1.000 - 3.000	0.0	0.0	18.0	67.5	67.5
4	0.300 - 1.000	193.3	16.8	18.0	67.5	50.7
5	0.100 - 0.300	0.0	0.0	14.0	52.5	52.5
6	0.005 - 0.100	699.3	60.9	29.0	108.8	47.8
7	0.000 - 0.005	0.0	0.0	5.0	18.8	18.8
TOTAL		1147.6	100.0	98.0	367.6	267.6

Table 2.2. Excess quarry production expressed in percentages.

is a measure of economic quarry use: the lower the excess, the more economic the quarry use. Table 2.2 shows the computation of the excess production percentage of the optimum alternative. A total excess percentage of 267.7% may be considered uneconomic in terms of effective quarry use. An alternative of the same record, and designed for $\cot\alpha = 5.00$ and $H_{DD} = 5.80$ m ($T_r = 43.2$ yr), with practically the same amount of total costs, shows a more economic excess percentage of 137.2%. Based on these computations the latter alternative might be selected as the optimum.

2.10. Adjustments to the cross-section by means of AutoCAD.

All computations described above are executed by a Turbo Pascal programme. The data of the optimum alternative can be send to a programme within AutoCAD. This programme draws the cross-section, and a table consisting of design specifications, volumes of the quarry categories required, and construction costs (Fig.2.7).

By changing the values of geometrical parameters of the cross-section, adjustments to the cross-section can easily be made. These adjustments can be necessary, for instance, to connect one section of the breakwater to a neighbouring section. After the values have been changed the AutoCAD programme computes and draws the new cross-section, volumes, and construction costs.

Still the cross-section is built up according to a fixed sequence of computations. A CAD-system offers the possibility to edit a drawing by means of the mouse or the digitizing tablet. Therefore within AutoCAD any changes required can be made to the cross-section. Related volumes and construction costs are computed thereafter. However, some care has to be taken since reckless changes to the cross-section may heavily violate the results of the optimization process.

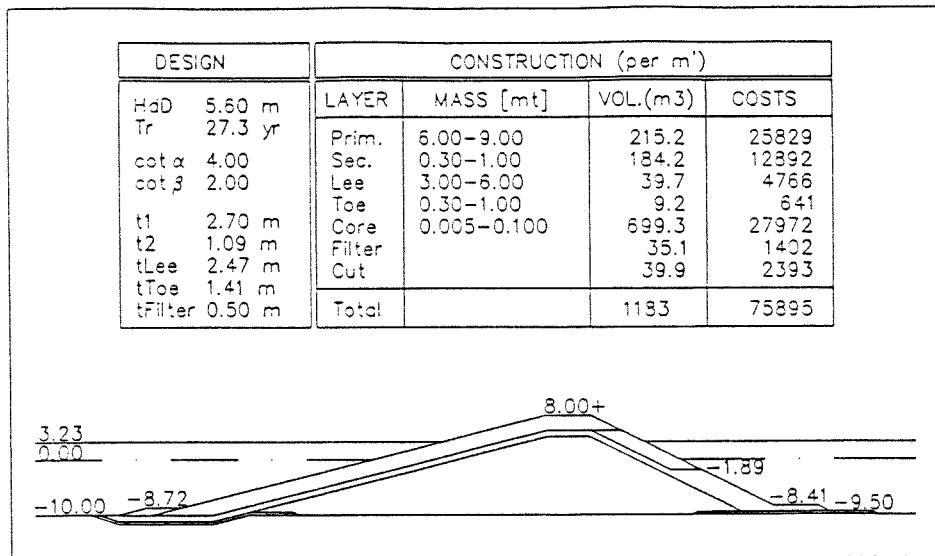


Fig.2.7. Cross-section of the optimum alternative.

2.11. Conclusions

At this stage the computer package is useful for feasibility studies of harbour protection or coastal protection in regions where a quarry can be found near the project site and the use of concrete armor units is excluded in advance. Given the physical conditions at or near the proposed site, and the (expected) quarry production distribution, the software offers a quick tool to show how the optimum rubble-mound breakwater may look like, based on a minimum of total costs during the structural lifetime. Due to the memory limitations of a micro-computer it is not possible to include very detailed computations within the entire optimization process.

A comparison of costs between the use of concrete armor units and the exclusive use of rock material cannot be made yet. In the next stage of the software development hydraulic stability formulae of concrete armor units have to be implemented. Van der Meer (1988-3) has developed formulae for cubes, tetrapods, and accropodes. These formulae were derived from experiments, which were carried out for only one cross-section for each armor unit. Therefore the slope angle, $\cot\alpha$, is fixed to the slope angle applied in the model.

The optimization can be executed in a similar way as described for the the rock material. A number of masses and related unit prices of one of the concrete armor units has to be entered. The average mass, W , of the primary armor layer is computed by the appropriate stability formula. The armor unit of mass greater than and closest to the value of W , will be allocated to the primary layer. The remaining computations in outline follow the procedures described in the previous sections.

The computation of the optimum rubble-mound breakwater cross-section is entirely executed by a Turbo Pascal programme. Due to the absence of non-changing elements within the cross-sections, the application of a CAD-system is limited to minor adjustments to the cross-section. Moreover, since the CAD-software occupies the major part of the available memory of a micro-computer, it is impossible to execute large programmes, like for instance the optimization described above, within a CAD-environment. Concerning the design of rubble-mound breakwaters the D in CAD is limited principally to the meaning of Drafting.

3. General features of the RUMBA computer package.

3.1. General scheme and contents of the programmes.

The RUMBA computer package consists of two main programmes, namely OPTRUMBA.PAS, and CADRUMBA.LSP. File extension ".PAS" designates a Turbo Pascal programme, whereas extension ".LSP" denotes a programme written in AutoLISP, the programming language embedded within AutoCAD. Figure 3.1 shows the general scheme of the programmes and the order in which they have to be invoked.

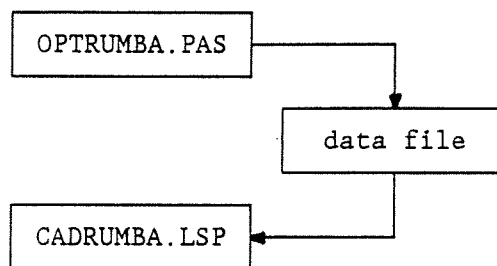


Fig.3.1. General scheme of the RUMBA computer package.

The general features of both programmes are described briefly below:

OPTRUMBA.PAS

- * Input of
 - Weibull distribution of deep water wave heights
 - exponential distribution of storm surge levels
 - physical site conditions
 - quarry fragmentation
 - structural specifications.
 - functional performance criteria
 - hydraulic stability criteria
- * Development of a number of alternative cross-sections, each characterized by design wave height and outer slope.
- * For each alternative computation of
 - crest elevation based on given functional performance criteria
 - hydraulic stability based on Van der Meer's design formulae for statically stable structures
 - cross-section and volumes of construction layers
 - construction costs and maintenance costs
- * Determination of the optimum design conditions
- * Output of geometrical data of the optimum cross-section.

CADRUMBA.LSP

- * Drawing of the optimum cross-section and a table of required quarry material and construction costs on screen.
- * Minor changes to the optimum structure by changing values of geometric parameters, or by means of mouse, or ,if available, by digitizing tablet.
- * Computation of new volumes and construction costs after adjustments have been made.

3.2. Hardware and software requirements.

The software requires an IBM-compatible computer with at least 640K memory, provided with a hard disk, and a 80X87 numeric co-processor.

Programme OPTRUMBA.PAS, and nested units, have been written in Turbo Pascal 4.0. In order to avoid running out of memory, the executable code, named OPTRUMBA.EXE, has to be saved on hard disk. Moreover the disk is used to link the programme and units during compilation. The Turbo Pascal compiler settings therefore have to read:

```
Compile/Destination/Disk
Options/Compiler/Link buffer/Disk
Options/Compiler/Numeric processing/Hardware
```

The latter setting is necessary to use the co-processor.

The AutoLISP programme CADRUMBA.LSP runs within AutoCAD Release 9, or later. Use of AutoLISP functions requires Advanced Drafting Extensions 1, 2, and 3. The number of files a programme can have open at once, has to be increased to 20. Therefore the boot-up file CONFIG.SYS file should contain the line

```
FILES=20
```

For proper operation of the AutoLISP programme the following lines have to be inserted in the AUTOEXEC.BAT file:

```
SET ACADFREERAM=15
SET LISPHEAP=35000
SET LISPSTACK=10000
```

Programmes, related units, and data files, of the RUMBA package have all been stored in directory C:\PROG.

4. Description of OPTRUMBA.PAS.

4.1. General flow scheme.

The figure below shows the general flow scheme of the programme OPTRUMBA.PAS. The inner loop of index i is executed n_D times, which is equal to the number of design wave heights, H_{dD} . The

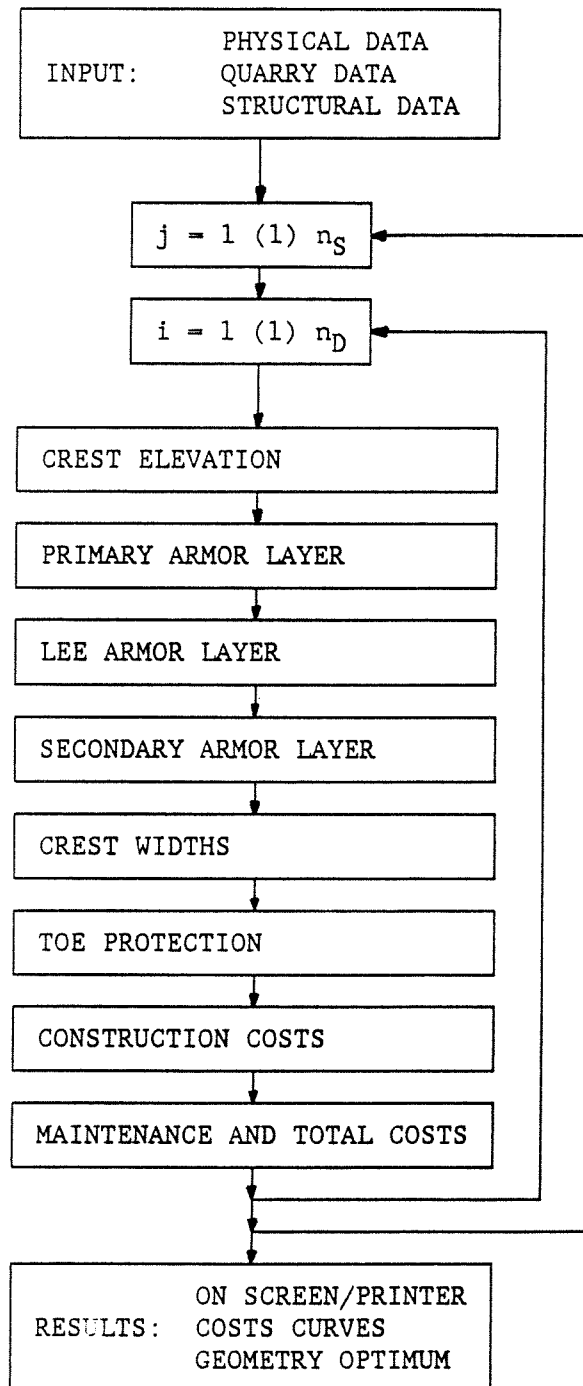


Fig.4.1 General flow scheme of OPTRUMBA.PAS

outer loop of index j corresponds with the number of outer slope angles, $\cot\alpha$.

4.2. Main Menu of OPTRUMBA.PAS.

After the executable code of OPTRUMBA.PAS has been loaded, its Main Menu will appear on screen (Fig. 4.2). In broad lines the flow scheme of Fig.4.1 can be recognized. Four sections can be distinguished within the Main Menu:

- input of data (options "P", "Q", "S", "D")
- computation (option "R")
- output of data (options "O", "L", "C", "A")
- exit programme (option "E").

Option letters "P", "Q", and "S" will invoke the corresponding data input sub-menu. By selecting option "D" all required input data will be read at once from default files:

- \PROG\PHYRUMBA.DAT (containing physical data)
- \PROG\QUARUMBA.DAT (containing quarry data)
- \PROG\STRUMBA.DAT (containing structural data).

Computation of the alternative cross-sections by selecting option "R", cannot be started before all three types of data have been read. A similar restriction is valid for the output section. Output options cannot be selected before any computation has been executed. Warnings have been built within the programme. Option "E" will leave the programme after confirmation.

M A I N M E N U O P T R U M B A	
Option	Description
	INPUT:
P	Physical Menu
Q	Quarry Menu
S	Structural Menu
D	Default data files
R	Run programme
	OUTPUT:
O	On screen
L	Listing on printer
C	Costs curves
A	AutoCAD (optimum only)
E	Exit programme

Fig.4.2. Main Menu of OPTRUMBA.PAS

In the following sections all parts of the programme will be explained in the same order as they are nested within OPTRUMBA.PAS. The input sequence is reproduced exactly and is preceded by a vertical bar. Current values of variables are given between square brackets. The given values correspond with the values of the data used for the example of Section 2.9. If other values are required these can be entered behind the arrow. Explanations will be given separately underneath.

4.3. Input of data.

4.3.1. Option "P": Input of physical data.

The Physical Menu can be reached by selecting option "P" from the Main Menu. Figure 4.3 shows the Physical Menu.

P H Y S I C A L M E N U	
Option	Description
I	Input from data file
W	Wave periods, water levels, and miscellaneous
L	Long-term distributions
S	Store physical data
M	Main Menu

Fig.4.3. Physical Menu.

Option "I": Input from data file.

- An existing file of physical data can be read by entering the name of the data file, including its path-name. Extension ".DAT" will be inserted by the programme.

Option "W": Wave periods, water levels, and miscellaneous.

Peak period T_p is written as a function deep water significant wave height H_{SD} , according to:

$$T_p = a \cdot H_{SD}^c + b \quad (\text{eq. 2.4})$$

coefficient	a =	[0.550]	⇒
coefficient	b =	[7.540]	⇒
coefficient	c =	[1.000]	⇒

Ratio peak period to average period T_p/T_m	[1.200]	==>
Storm duration	[6.000 hrs]	==>
Mean high tide level	[1.800 m]	==>
Mean low tide level	[-1.700 m]	==>
Mean depth	[10.000 m]	==>
Bottom slope	[0.010]	==>
Density of sea water	[1.030 t/m ³]	==>

- Ratio peak period to average period, T_p/T_m :
The run-up formulae, and Van der Meer's design formulae, require the average period. The ratio of wave periods is dependent on the wave spectrum:

$$\begin{aligned}
 T_p/T_m &= 1.01 \pm 0.02 && \text{for small spectra} \\
 T_p/T_m &= 1.15 \pm 0.04 && \text{for Pierson-Moskowitz spectrum} \\
 T_p/T_m &= 1.42 \pm 0.10 && \text{for broad spectra.}
 \end{aligned}$$

- Storm duration, Δt :
Storm duration is assumed to be equal to the interval period of wave height registration.
- Mean high tide level, h_{ht} :
Average flood level in meters with respect to MSL.
- Mean low tide level, h_{lt} :
Average ebb tide level in meters with respect to MSL.
- Mean depth, d_m :
Average depth at the breakwater site in meters with respect to MSL.
- Bottom slope, i_b :
Mean slope of the bottom seaward from the structure. This value will only be used for computation of the shoaling coefficient.
- Density of sea water, ρ_s , in units of ton/m³.

Option "L": Long-term distributions.

The cumulative probability distribution for long-term deep water wave heights is described by the Weibull distribution, written as:

$$P(H_{sD}) = \text{Prob}(H_{sD} \leq H_{sD}) = 1 - \exp\left[- \left(\frac{H_{sD} - \epsilon}{\theta} \right)^\gamma \right] \quad (\text{eq. 2.1})$$

where:

ϵ = location parameter	[1.0000]	==>
θ = scale parameter	[0.4343]	==>
γ = shape parameter	[1.0000]	==>

The long-term distribution of water levels is described by the exponential distribution of the form:

$$P(h) = \text{Prob}(\underline{h} \leq h) = 1 - \exp\left[-\left(\frac{h - \epsilon}{\theta}\right)\right] \quad (\text{eq. 2.3})$$

where:

ϵ = location parameter [0.167] ==>
 θ = scale parameter [0.289] ==>

Option "S": Store data.

Physical data can be stored on a data file. If data were read from an existing data file, that file name will be default. Another name can be chosen by answering "N" to question "Is the file name correct? (Y/N)". Don't forget the path name, if necessary. Extension ".DAT" will be inserted by the programme.

Option "M": Main Menu.

This option will switch from the Physical Menu to the Main Menu.

4.3.2. Option "Q": Input of quarry data.

The Quarry Menu (Fig.4.4) appears after selection of option "Q" from the Main Menu. Within this sub-menu the options "I", "S", "M" represent similar procedures as described in the previous section. In fact the Quarry Menu consists of one relevant option.

Q U A R R Y M E N U	
Option	Description
I	Input from data file
Q	Quarry production and unit prices
S	Store quarry data
M	Main Menu

Fig.4.4. Quarry menu.

Option "Q": Quarry production and unit prices.

Density of quarry material	[2.650 ton/m3]	==>
Number of non-overlapping quarry categories (min = 4, max = 10)	[7]	==>
Category 1:		
- upper limit mass	[9.000 ton]	==>
- upper limit passing percentage	[98.00 %]	==>
- unit price per m3	[120.0 -/m3]	==>
Entered values correct? (Y/N)		==> Y
Category 2:		
- upper limit mass	[6.000 ton]	==>
- upper limit passing percentage	[93.00 %]	==>
- unit price per m3	[120.0 -/m3]	==>
Entered values correct? (Y/N)		==> Y
Etc.etc.		
Category 7:		
- upper limit mass	[0.005 ton]	==>
- upper limit passing percentage	[5.00 %]	==>
- unit price per m3	[40.0 -/m3]	==>
Entered values correct? (Y/N)		==> Y
Unit price per m3 of filter gravel	[40.0 -/m3]	==>
Unit price per m3 cut	[60.0 -/m3]	==>
Entered values correct? (Y/N)		==> Y

- At the start a description is given in what order the quarry data have to be entered. A number of non-overlapping mass categories is assumed. From an economical and practical point of view a factor of 3 or more is required between minimum and maximum block mass of each category. An exception may be allowed for the greater mass categories as they are selected more accurately. The appropriate values of the quarry categories given above were taken from Table 2.1.
- Unit price per m3 of filter gravel:
The unit price of the filter gravel is assigned seperately.
- Unit price per m3 cut:
The unit price of cut comprises the costs of dredging near the toe of the structure, if necessary.
- After all data have been entered a summary of quarry data is given, which will correspond with the data of Table 2.1. Corrections can be made by answering "N" to question "Entered values correct? (Y/N)". The whole input sequence of quarry data will then be repeated.

4.3.3. Option "S": Input of structural data.

Option "S" from the Main Menu invokes the Structural Menu, of which the contents are shown in Fig. 4.5. Options "I", "S", "M" can be distinguished in the Structural Menu as well. Again they represent the same (kind of) procedures as described in Section 4.3.1.

S T R U C T U R A L M E N U	
Option	Description
I	Input from data file
N	Number of units in layers
W	Width and thickness of layers
E	Economy
A	Average mass ratios of layers
F	Functional performance criteria
H	Hydraulic stability criteria
S	Store structural data
M	Main Menu

Fig.4.5. Structural Menu.

Option "N": Number of units in layers.

Thickness of primary armor layer	[2 units]	==>
Thickness of secondary armor layer	[2 units]	==>
Thickness of lee armor layer	[2 units]	==>
Thickness of toe layer	[2 units]	==>
Width of breakwater crest	[3 units]	==>
Width of toe crest	[8 units]	==>

- In general, the thickness of layers or width of crests, expressed in number of blocks, are requested above. The given number of blocks across the breakwater crest will be the minimum number. The number will be adjusted in the programme as soon as the width does not satisfy certain minimum widths specified at option "W".

Option "W": Width and thickness of layers.

Minimum width of breakwater crest	[6.50 m]	==>
Minimum width of core crest	[6.50 m]	==>
Width of berms	[8.00 m]	==>
Thickness of filter layer	[0.50 m]	==>

- Minimum width of breakwater crest and/or core crest:
These widths may be necessary for future use or during the construction phase. If no special requirements are necessary, these values have to be set equal to zero.

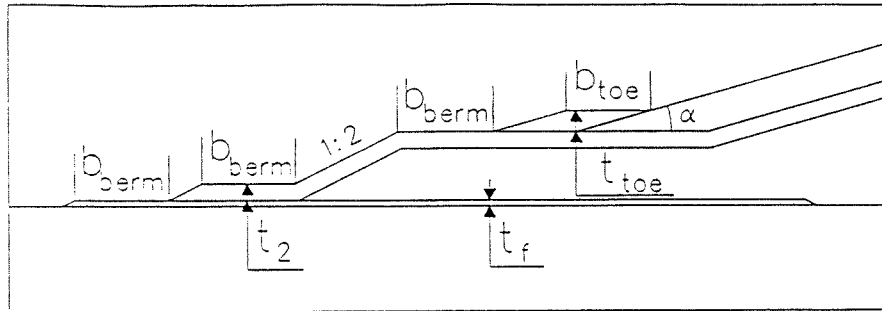


Fig.4.6. Details of the sea side of the structure near the bottom.

- Width of berms, b_{berm} :
The length of the extension of the secondary armor along the bottom, and the filter layer, is dependent on the value of the berm width. Details are shown in Figs. 4.6 and 4.7.
- Thickness of filter layer, t_f :
The composition of the filter layer is not computed by the programme. Just the thickness has to be given.

Option "E": Economy.

Expected life of breakwater	[50 yrs]	=>
Interest percentage	[7.00 %]	=>

- Expected lifetime of the structure, L , in years, and interest percentage, $i\%$, are important factors in the computation of the maintenance costs.

Option "A": Armor mass ratios.

Ratio primary to secondary armor mass	[15.00]	=>
Ratio primary to toe layer mass	[7.00]	=>
Ratio primary to core mass	[500.00]	=>

- After the average primary armor mass, W_1 , has been computed, average masses of the other layers will be related to W_1 according to a certain ratio. These ratios are more or less based on the Terzaghi filter rule. Reasonable ratios to be

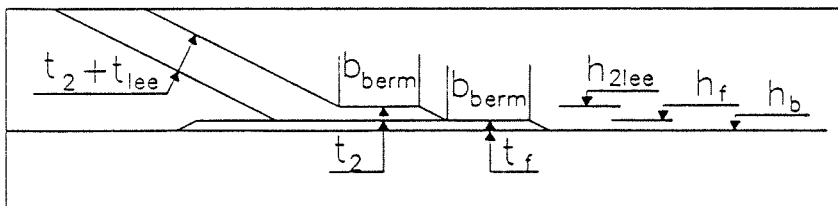


Fig.4.7. Details of the lee side of the structure near the bottom.

allocated, are in the ranges of:

$$10 \leq W_1/W_2 \leq 20$$

$$5 \leq W_1/W_{\text{toe}} \leq 10$$

$$500 \leq W_1/W_{\text{core}} \leq 1000$$

Since a quarry category is allocated to a construction layer when the average mass value of the latter lies within the mass limits of the category, the use of the quarry can be guided, to some degree, by the ratios given above.

Option "F": Functional performance criteria.

Upper bound transmitted wave height Ht	[0.30 m]	=>
Return period of Ht	[25.00 yrs]	=>

- The meaning of the functional performance criteria are described in Section 2.4.

Option "H": Hydraulic stability criteria.

Damage level	[2.00]	=>
Permeability P		
min = 0.10 (impermeable core)		
max = 0.60 (homogeneous structure)	[0.45]	=>
Upper bound average block mass	[9.000 ton]	=>
Cotangent of lee side slope	[2.00]	=>
Packing coefficient	[1.02]	=>

- Damage level, S:
To fulfill the criterion of "no damage", the value of the damage level, S, has to be set equal to the value corresponding with "start of damage". Table 4.1. shows appropriate values of S depending on the seaward slope.

DAMAGE LEVEL S		
cot α	start of damage	filter layer visible (thickness 2 blocks)
1.5	2	8
2.0	2	8
3.0	2	12
4.0	3	17
5.0	3	17

Table 4.1. Lower and upper damage levels for rock slopes (Van der Meer, 1988-1).

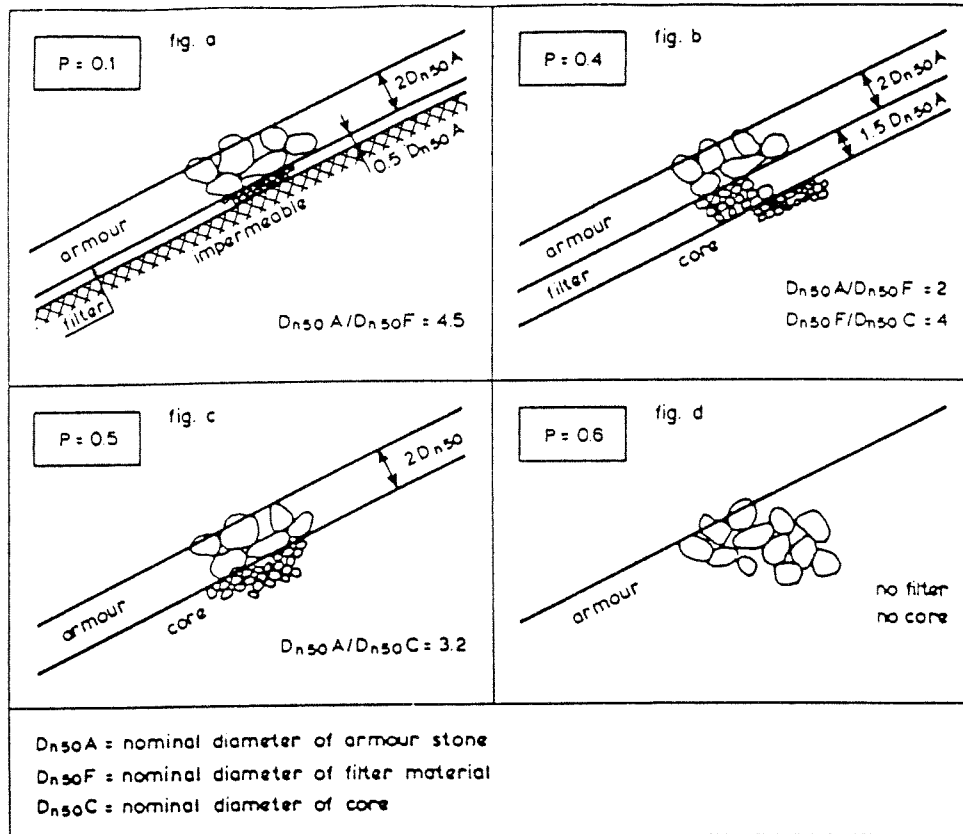


Fig.4.8. Permeability coefficient assumptions for various structures (Van der Meer, 1988-1).

- Permeability, P :
The permeability coefficient P depends on the type of structure, which is illustrated in Fig. 4.6.
- Upper bound average block mass, W_{\max} :
The value of W_{\max} determines the crucial point of rejection of an alternative. Whenever the average mass of the primary armor layer $W_1 > W_{\max}$, the alternative in question will be cancelled. The value entered cannot be greater than the upper limit mass of quarry category 1.
- Cotangent of lee side slope, $\cot\beta$:
For reasons of stability the cotangent of the lee side slope should be at least $\cot\beta \geq 1.5$. The leeward slope of the structure will not be changed during the entire computation.
- Packing coefficient, k_{Δ} :
This value is needed for computation of the armor layer thickness, t (see eq. 2.16). The SPM (1984) shows relevant values in Table 7-13 on page 7-234.

4.4. Option "R": Computation of alternative cross-sections.

4.4.1. Development of a number of alternative cross-sections.

After option "R" of "Run programme" has been selected from the Main Menu, a table containing 14 storm conditions will appear on screen. Meaning of the columns is described briefly below.

Column:

Tr : Return period of storms ranging from 10 times per year to once per thousand years.

HsD : Deep water significant wave height, H_{sD} , derived from the given Weibull distribution. Rearranging eq. (2.1) yields:

$$H_{sD} = (-\theta) * [\ln(1-P(H_{sD}))]^{1/\gamma} + \epsilon \quad (4.1)$$

where $P(H_{sD})$ is computed from eq. (2.2).

HsL : Local significant wave height, H_{sL} . The computation of H_{sL} was described in Section 2.3. Figure 4.9 shows the computational scheme of H_{sL} , starting from a given T_r .

Tp : Peak wave period, T_p , derived from eq. (2.4).

Depth : Depth, d , at the breakwater site, being the sum of the mean depth, d_m , and the water level, h . The water level is based on the exponential distribution. Rearranging eq. (2.3) yields:

$$h = (-\theta) * \ln[1 - P(h)] + \epsilon \quad (4.2)$$

Since it is assumed that h and H_{sD} occur simultaneously during a storm event of recurrence interval, T_r , the values of $P(h)$ and $P(H_{sD})$ are equal to each other.

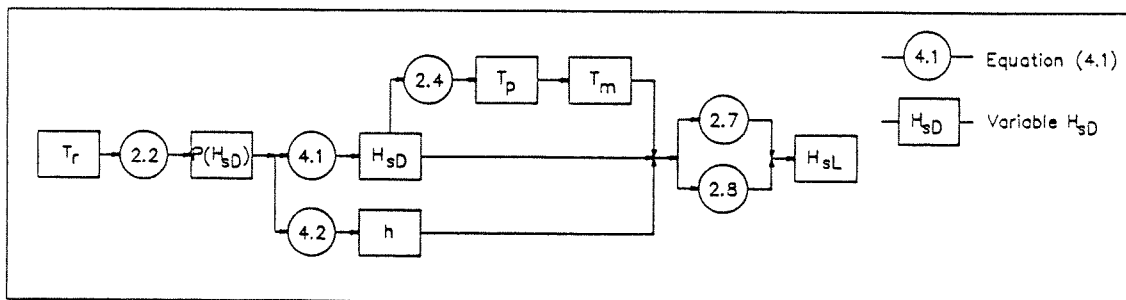


Fig.4.9. Computational scheme of H_{sL} starting from T_r .

PExc : Probability of exceedance:

$$P_{Exc} = \text{Prob}(H_{sD} \geq H_{sD}) = 1 - P(H_{sD}) = (1/\nu)/T_r \quad (4.3)$$

The latter equivalence is equal to eq. (2.2).

Broken: If "yes" H_{sL} was calculated by eq. (2.7), the wave breaking criterion. On the other hand, if "no", the shoaling criterion given by eq. (2.8), yielded the smallest value of H_{sL} .

The meaning of the storm data table is twofold:

1. Physical data can be checked on proper input.
2. Meaningfull values can be given for a range of deep water design wave heights H_{dD} .

Each alternative cross-section to be created will be characterized by a deep water design wave height, H_{dD} , and the outer slope (hor:vert = $\cot\alpha:1$). The lower and upper limits of H_{dD} correspond with $T_r = 0.10$ yr and $T_r = 500$ yrs, respectively. After a H_{dD} -range has been created satisfactory, a range of outer slopes has to be given. Due to the validity of the Van der Meer stability formulae, the range of outer slopes is limited to $\cot\alpha = 1.5 - 6$. The maximum number of H_{dD} is 20, whereas the maximum number of outer slopes is 10. Thus, a total of $20 \times 10 = 200$ alternatives can be defined. The input of the example was:

Enter a range of deep water design wave heights (column HsD):

- minimum design wave height (≥ 4.164 m) [4.20 m] \rightarrow
- stepwise increment of design wave height [0.20 m] \rightarrow
- maximum design wave height (≤ 6.863 m) [6.86 m] \rightarrow

REMARK: 14 design wave(s) will be computed.

Entered values correct? (Y/N) \rightarrow Y

Enter a range of cotangents of outer slopes (= $\cot\alpha$):

- minimum cotangent (≥ 1.50) [2.00] \rightarrow
- stepwise increment of cotangent [1.00] \rightarrow
- maximum cotangent (≤ 6.00) [5.00] \rightarrow

REMARKS: Number of design waves = 14
Number of design slopes = 4
Total of 56 breakwater(s) will be computed!

Entered values correct? (Y/N) \rightarrow Y

The number of design waves corresponds with parameter n_D of the scheme of Fig.4.1, whereas the number of design slopes

corresponds with parameter n_s . After the last confirmation the computation of the alternative cross-sections starts. The progress of computation is presented on screen. Moreover, the loop indexes are shown for each procedure. The conclusion may arise now that the loops of the scheme of Fig.4.1 should have been drawn around each procedure block separately. Correct! For the sake of simplicity, however, the loops were drawn around the entire computation.

The procedures of the computation are described in the sections below.

4.4.2. Design storms.

For each design storm, signified by H_{dD} , the same storm parameters are computed as mentioned in the table at the start of option "R". T_r is related to $P(H_{dD})$, the latter which is computed by means of eq. (2.1). Figure 4.10 shows the computational scheme of the storm parameters starting from the given H_{dD} . It is repeated that the index d is used to distinguish the design significant wave height from the generally used significant wave height H_{sD} . In the output section of the programme the resulting design storm conditions can be shown (see Section 4.5.1).

4.4.3. Crest elevation.

The functional performance criteria given in the Structural Menu, option "F" (Section 4.3.3.), are used to determine the crest elevation of each alternative. Starting from the given storm return period the same sequence of calculations are made as in Section 4.4.1 to derive the storm parameters (H_{sD} , H_{sL} , T_m). The storm represented by these parameters will cause the given upper bound transmitted wave height, H_t , at the lee side of the structure.

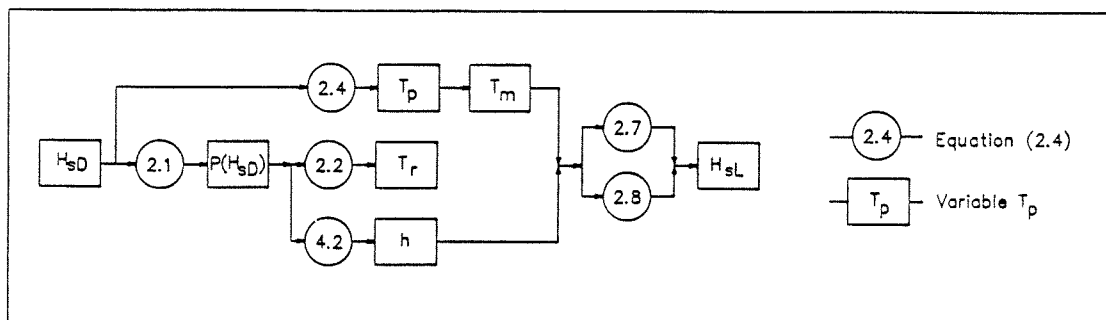


Fig.4,10. Computational scheme of H_{sL} starting from H_{sD}

Rearranging eq. (2.9), which describes the relation between the transmission coefficient, $K_t = H_t/H_{sL}$, and the ratio, F/R_s , yields:

$$F/R_s = -2.09*K_t + 1.16 \quad (4.4)$$

Note that only wave transmission by overtopping has been taken into account, and that the influence of the width of the breakwater crest has been neglected. Moreover, the experiments were conducted only for a smooth impermeable breakwater with 1 : 1.50 slope. Hence, the results have to be judged with these restrictions in mind.

The significant run-up, R_s , is computed from eq. (2.10), and finally the crest elevation, h_1 , is computed by

$$h_1 = h + F \quad (4.5)$$

The whole sequence of the computation of the crest elevation is illustrated in the scheme of Fig. 4.11. Since only one storm condition, based on the functional performance criterion, is considered for all alternative cross-sections, the rate of overtopping, and therefore the crest elevation, will exclusively depend on the steepness of the outer slope.



Fig. 4.11. Scheme of the computation of the crest elevation.

4.4.4. Primary armor layer.

The Van der Meer stability formulae for statically stable structures were shown by eq. (2.11), for plunging waves, and eq. (2.12) for surging waves. These formulae were developed for non-overtopped structures. The reduction of the size of the armor blocks, allowed for lower crested breakwaters, is given by eq. (2.14). The reduction factor, f_o , defined as $D_{n50}(\text{primary, overtopped})/D_{n50}(\text{primary, non-overtopped})$, can be computed at this stage of the programme.

After the type of breaker has been determined by equation (2.13), the nominal diameter of the primary armor layer, D_{n50} , of each alternative is computed from a combination of equations (2.11) and (2.14), (2.12) and (2.14), respectively:

For plunging waves:

$$D_{n50} = \frac{H_{dL} * \sqrt{\xi_m} * f_o}{6.2 * \Delta * p^{0.18} * (S/\sqrt{N})^{0.2}} \quad (4.6)$$

For surging waves:

$$D_{n50} = \frac{H_{dL} * f_o}{1.0 * \Delta * p^{-0.13} * (S/\sqrt{N})^{0.2} * \sqrt{(\cot\alpha) * \xi_m^p}} \quad (4.7)$$

Hence, the D_{n50} computed above corresponds with $D_{n50}(\text{primary, overtopped})$. Next the average mass, W_1 , and the thickness, t_1 , of the primary armor layer, are computed by, respectively,

$$W_1 = \rho_r * (D_{n50})^3 \quad (4.8)$$

$$t_1 = n_{t1} * k_{\Delta} * D_{n50} \quad (4.9)$$

where n_{t1} = the number of blocks in the primary layer.

Placing of primary armor stone is continued on the crest of the breakwater. Whether the same category is continued on the lee side as well, will depend on the expected rate of overtopping. This aspect will be explained in the next section.

4.4.5. Lee armor layer.

The reduction of the size of the lee armor blocks, which is allowed for relatively high breakwater crests, is described by eq. (2.17) and illustrated in Fig.2.3. The reduction factor, f_i , is defined as $D_{n50}(\text{lee, overtopped})/D_{n50}(\text{primary, non-overtopped})$. Combining f_i and f_o , the nominal diameter of the lee armor is computed by

$$D_{n50}(\text{lee, overtopped}) = \frac{f_i}{f_o} * D_{n50}(\text{prim., overtopped}) \quad (4.10)$$

For reasons given in Section 2.6.3., eq. (4.10) is valid for wave steepness $s_p > 0.025$ and $f_i \leq f_o$. Whenever these conditions are not fulfilled, the average mass of the lee armor is equaled to the applied average mass of the primary layer. Hence,

$$D_{n50}(\text{lee, overtopped}) = D_{n50}(\text{primary, overtopped}) \quad (4.11)$$

Take care whenever the nominal diameters, and therefore the average masses, of the primary and lee armor are found to be equal. Lee armor blocks may then be too light to withstand the impact of overtopping waves.

The level to which the lee armor is continued, h_{lee} (see Fig.4.12) depends on the transmitted wave height and the water level during a storm event. Therefore the wave transmission is computed for a number of storm events at different levels of still water (SWL)

- at high tide SWL = h
- at turning of tide SWL = h - h_{ht}
- at low tide SWL = h - h_{ht} - h_{lt}

where:

- h_{ht} = mean high tide level
- h_{lt} = mean low tide level.

The number of storms to be considered depends on the return period of the design storm. This storm, as well as storms of return periods smaller than T_r -design, are taken into account. Appropriate storm conditions are therefore taken from the Storm Data table at the beginning of the computation (Section 4.4.1).

Determination of the transmitted wave height, H_t , at the lee side is now carried out in reversed order compared to the computation in Section 4.4.3. H_t is computed from the transmission coefficient, $K_t = H_t/H_{dL}$, which, in turn, is derived from eq.

(2.9). Level h_{1ee} is computed using either

$$h_{1ee} = SWL - H_t \quad \text{for } K_t > 0 \quad (4.12.a)$$

or

$$h_{1ee} = SWL \quad \text{for } K_t \leq 0 \quad (4.12.b)$$

Transmission by overtopping will occur for values of $K_t > 0$. Wave transmission through the breakwater will also contribute to the transmitted wave height. Although this contribution has not been computed separately, its negative effect on the stability of the inner slope is implicitly processed in the conservative design criteria of h_{1ee} .

The lowest of all values of h_{1ee} determines the lower bound level of the lee armor layer of the alternative at consideration. Checks on the computation have concluded that storms of small recurrence intervals during ebb tide with no overtopping (eq. 4.12.b) result into the lowest level of h_{1ee} .

So far all the structural alternatives have been computed. Henceforward exclusively the structures for which $W_1 \leq W_{max}$ will be considered, while the remaining will be discarded.

4.4.6. Secondary armor layer.

The average mass of the secondary armor layer, W_2 , is related to the average mass of the primary armor layer, W_1 . The value of the mass ratio has been given in the input section of the programme (Section 4.3.3). The thickness of the secondary armor, t_2 , is found by accordingly using eqs. (4.8) and (4.9).

During the secondary armor computation a correction is made to the value of h_{1ee} , as soon as this value is smaller than the value of level h_{2lee} , defined as

$$h_{2lee} = -d_m + t_f + t_2 \quad (4.13)$$

where t_f = thickness of the filter layer (see also Fig.4.7).

If $h_{1ee} < h_{2lee}$ then $h_{1ee} = h_{2lee}$. This procedure is necessary to avoid "penetration" of the lee armor into the secondary armor and filter layer. The condition, however, will only be fulfilled in very shallow waters.

4.4.7. Crest widths.

In this procedure the widths of primary armor crest, b_1 , secondary armor crest, b_2 , and core crest, b_{core} , are computed (Fig.4.12). The sequence of computations starts from the minimum number of blocks, n_{b1} , required for the breakwater crest:

$$b_1 = n_{b1} * k_{\Delta} * \left[\frac{W_1}{\rho_r} \right]^{1/3} \quad (4.14)$$

If b_1 does not satisfy the minimum required width in meters, given in Section 4.3.3, the number of blocks will be increased accordingly. Next the widths of the secondary armor crest, b_2 , and the core crest, b_{core} , are computed from, respectively:

$$b_2 = b_1 - t_1 * \frac{1 - \cos\alpha}{\sin\alpha} - (t_1 * \cos\beta - t_{lee}) * \frac{1}{\sin\beta} \quad (4.15)$$

$$b_{core} = b_2 - t_2 * \left[\frac{1 - \cos\alpha}{\sin\alpha} - \frac{1 - \cos\beta}{\sin\beta} \right] \quad (4.16)$$

where t_{lee} = thickness of the lee armor layer.

These width values are valid in case the width of the core crest satisfies the minimum required width of b_{core} . If not, the core crest width is increased to the minimum required value, also given in Section 4.3.3, after which the breakwater crest is computed in reversed order based on the minimum width of the core crest. The value of b_1 is corrected for an integer number of primary armor blocks. A final correction is needed for b_2 and b_{core} according to eqs. (4.15) and (4.16), respectively.

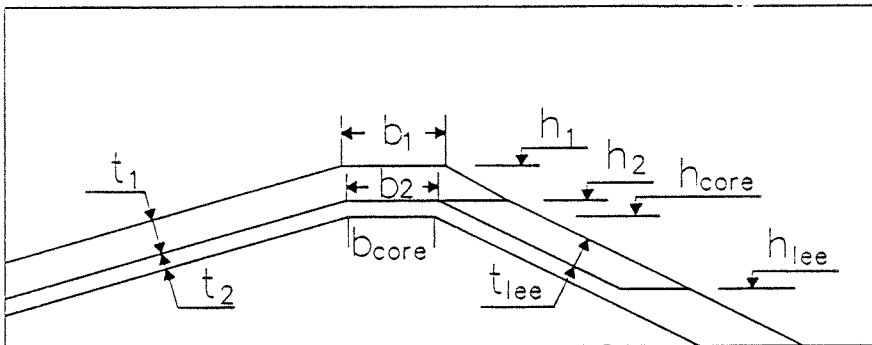


Fig.4.12. Details of the crest of the structure.

4.4.8. Toe protection.

The upper side level of the toe layer, h_{toe} , is determined following a similar procedure as executed for the computation of the lowest level of h_{lee} (Section 4.4.5.). Again appropriate storm conditions of return periods smaller than or equal to T_r -design are considered at the following SWL-conditions:

- at high tide SWL = h
- at low tide SWL = h - h_{ht} - h_{lt}

The design level of h_{toe} will be determined by the smallest value of

$$h_{toe} = SWL - 1.5 * H_{sL} \quad (4.17)$$

Checks on the results have concluded that the lowest level of h_{toe} is derived from the structural design storm during ebb tide.

As for the secondary armor blocks, the nominal mass of the toe blocks is also related to W_1 . Width and thickness of the toe berm is dependent on the respective number of toe blocks, specified in the input section (Section 4.3.3.).

4.4.9. Construction costs.

At this stage of the programme essential geometric parameters have been determined to compute the cross-section and the construction costs of each alternative. Five different situations may occur near the sea side toe of the structure. To be more precisely, in detail the cross-section near the toe is determined by the construction layer which is intersected by the bottom line. In the computation use is made of an auxiliary level, h_{base} , defined as:

$$h_{base} = h_{toe} - t_{toe} - t_2 - t_f \quad (4.18)$$

where t_{toe} = thickness of the toe layer.

From deep to shallow water the various cross-sections details are described below. For

$h_b < h_{base}$ the structure will be in fill completely (Fig. 4.13.a)

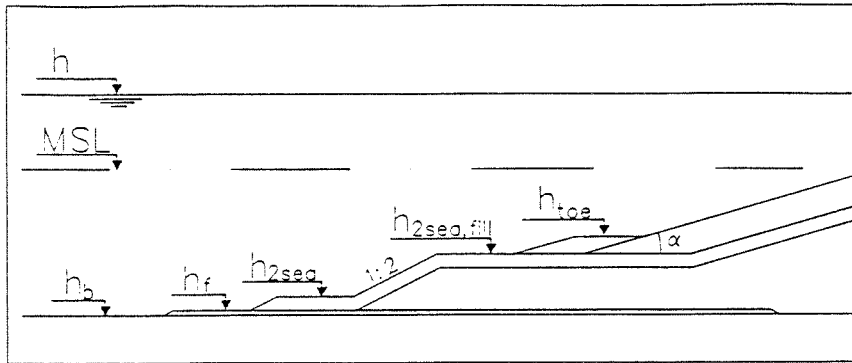


Fig.4.13.a. Details of the toe construction: berm in fill.

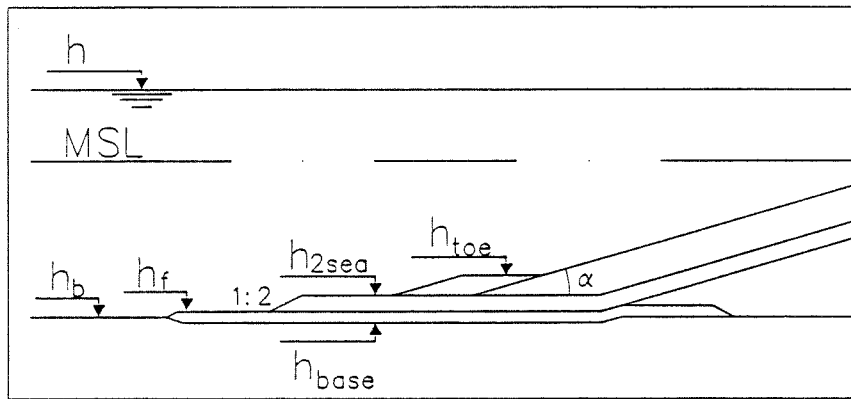


Fig.4.13.b. Details of the toe construction: filter layer intersected by the bottom line.

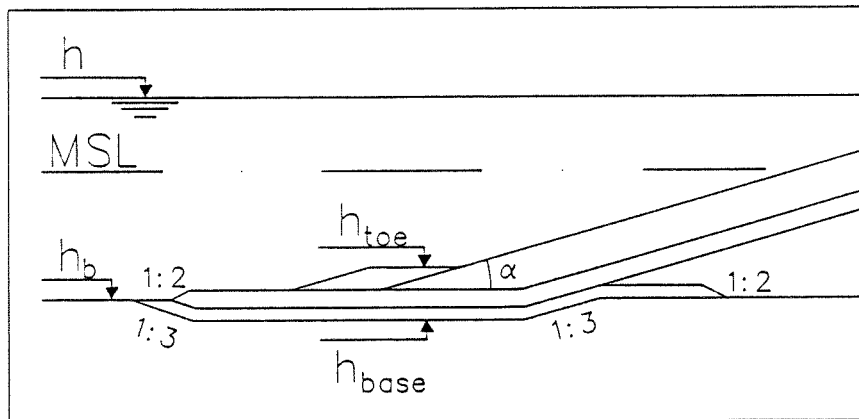


Fig.4.13.c. Details of the toe construction: secondary layer intersected by the bottom line.

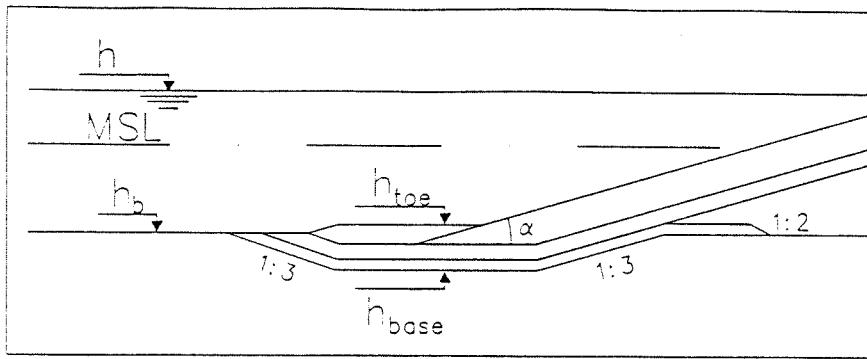


Fig.4.13.d. Details of the toe construction: toe layer intersected by the bottom line.

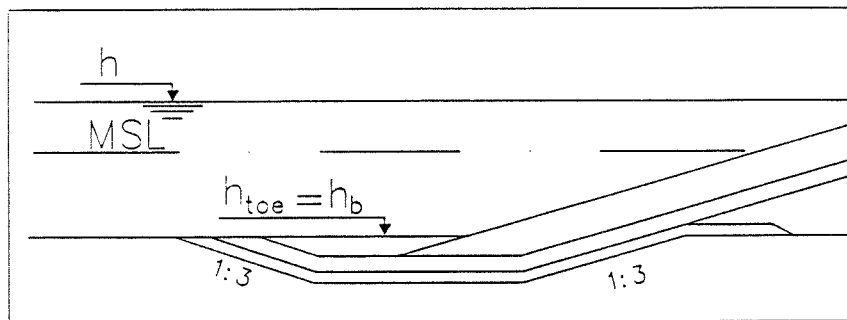


Fig.4.13.e. Details of the toe construction: in shallow waters.

$h_{base} \leq h_b < h_{base} + t_f$ the bottom line intersects the filter layer (Fig. 4.13.b)

$h_{base} + t_f \leq h_b < h_{toe} - t_{toe}$ the bottom line intersects the secondary armor layer (Fig. 4.13.c)

$h_{toe} - t_{toe} \leq h_b < h_{toe}$ the bottom line intersects the toe protection (Fig. 4.13.d)

where h_b = bottom level.

In more shallow waters for which the computed $h_{toe} < h_b$, the toe crest will be levelled to the bottom line (Fig. 4.13.e).

The slope of toe protection is constructed parallel to the seaward slope of the primary armor layer. The remaining layers in fill are constructed at slope 1:2. Dredged slopes are assumed at 1:3.

After the type of cross-section has been determined for a

particular alternative, the cross-sectional area of each construction layer is calculated by means of the Green theorem. According to the theorem the area of any polygon consisting of n vertices can be computed from:

$$A = 0.5 * (x_1y_2 - x_2y_1 + x_2y_3 - x_3y_2 + \dots \\ \dots + x_{n-1}y_n - x_ny_{n-1} + x_ny_1 - x_1y_n) \quad (4.19)$$

where x and y are the coordinates of the vertices. Multiplying the unit costs of the applied quarry category by the cross-sectional area yields the construction costs of the particular layer. The construction costs of the alternative, $\$C$, is attained by summation of all layer costs, as well as dredging costs, if necessary.

4.4.10. Maintenance costs and total costs.

In the preceding sections the $n_D * n_S$ alternative cross-sections have been designed for an acceptable damage level S, given in the input section. The expected yearly maintenance costs, $E(\$M)_{yr}$, given by eq. (2.20), is approximated by

$$E(\$M)_{yr} = \nu * 0.5 * \sum_{k=0}^{20} C_k * p(H_{SD})_k * \$_{Dam}(H_{SD})_k * \Delta H_{SD} \quad (4.20)$$

where coefficient $C_k = 1$ for $k = 0$ and $k = 20$
 $C_k = 2$ for $1 \leq k \leq 19$

$k = 0$ corresponds with H_{dD} , while $k = 20$ indicates a storm of return period $T_r = 10,000$ yr. The probability density function, $p(H_{SD})$, of the Weibull distribution is given by eq. (2.19). The repair costs required after a storm of intensity H_{SD} has attacked the structure, term $\$_{Dam}(H_{SD})$, are obtained following the next sequence of computations:

1. Knowing H_{SD} , the storm parameters H_{sL} and T_m are deduced according to the computational scheme of Fig. 4.10.
2. The damage level can be computed directly from Van der Meer's design formulae.

For plunging waves:

$$S = \sqrt{N} * \left[\frac{H_{sL} * \sqrt{\xi_m}}{6.2 * \Delta * D_{n50} * P^{0.18}} \right]^5 \quad (4.21)$$

For surging waves:

$$S = \sqrt{N} * \left[\frac{H_{sL}}{\Delta * D_{n50} * P^{-0.13} * \sqrt{(\cot\alpha) * \xi_m P}} \right] \quad (4.22)$$

where N is the number of waves during storm period Δt . In the formula above, reduction of damage, applicable for relatively low structures, has been omitted. A reduction of damage on the exposed side of the structure will be nullified by an increase of damage on the crest and lee side of the structure.

3. Damage level, S, is transformed into a percentage of damage Dam% by means of (see also Section 2.8.)

$$\text{Dam\%} = 1.25 * S \quad (4.23)$$

4. The repair costs are built up of two parts:

- a) mobilization costs of equipment, a fixed amount assumed to be 5% of the total construction costs, $\$C$;
- b) costs of filling up the "gaps" of the structure.

Dependent on the amount of damage the repair costs are computed as follows:

$$\$(h_{sd}) = 0.05*\$C + (\text{Dam\%/100})*1.5*\$1 \quad \text{for} \quad \text{Dam\%} \leq 10$$

$$\$(h_{sd}) = 0.05*\$C + (\text{Dam\%/100})*(\$1+0.5*\$2+\$1_{lee}+\$_{toe}) \quad \text{for} \quad 10 < \text{Dam\%} \leq 20$$

$$\$(h_{sd}) = \$C \quad \text{for} \quad \text{Dam\%} > 20$$

where:

$\$1, \$2, \$1_{lee}, \$_{toe}$ = construction costs of primary armor, secondary armor, lee armor, and toe protection, respectively.

Repair of minor damages ($\text{Dam\%} \leq 10$) for which primary armor blocks are required only, will consequently result in an excessive production of unused quarry material. Therefore factor 1.5 has been added in the first equation. In the second equation $0.5*\$2$ roughly denotes the sea side part of the secondary armor layer. For damages greater than 20% the functional performance is affected in such a way that the structure has to be rebuild completely.

The maintenance costs capitalized over the lifetime of the structure, $\$M$, are derived by multiplying the expected annual maintenance costs, $E(\$M)_{yr}$, by the present worth factor,

pwf (eq. 2.21).

Finally the total costs of the structure $\$T$ is the sum of construction costs and capitalized maintenance costs:

$$\$T = \$C + \$M \quad (4.24)$$

These computations are repeated for all relevant cross-sections. The cross-section belonging to the minimum value of total costs will be the optimum design at the given conditions.

4.5. Output of results.

4.5.1. Options "O" and "L": Output on screen or on printing-paper.

After computations have finished, options "O" and "L" of the Main Menu will present the numerical results on screen or on printing-paper, respectively. Both options will invoke the same Output Menu from which the required output can be selected (Fig. 4.14). Output of rejected alternatives is not available and will thus not be given. Columns of design storm conditions H_{dD} , T_p , and Depth have been added to most of the output tables.

Due to the limits of the screen the results are shown per outer slope angle, $\cot\alpha$. Each successive <ENTER> will flip to a milder slope. For data exchange to the printer, don't forget to set the

O U T P U T M E N U	
Option	Description
F	Functional performance
D	Design storms
I	Inner slope and crest levels
S	Sizes of blocks
B	Breadth of crests
L	Layer thickness
A	Average mass
W	Mass of armor stone
V	Volumes of material per m'
Q	Quarry use
P	Prices per m3
C	Construction costs
O	Overview of costs
M	Main Menu

Fig.4.14. Output Menu.

printer "on line". On paper the three names of input data files will also be shown.

The additions "1", "2", "toe", "lee", and "core" on top of a lot of columns refer to primary armor, secondary armor, toe protection, lee armor, and core, respectively. In the text of this manual these additions are denoted as subscripts.

From top to bottom the options of the Output Menu contain:

Option "F" - Functional performance.

Shows the wave data of the storm corresponding with the functional performance criteria. Moreover, the values of transmission coefficient, K_t , as well as of relative freeboard, F/R_s , are shown.

Option "D" - Design storm data.

The wave parameters of the n_D design storms are shown. These parameters were computed in Section 4.4.2. The table consists of the same elements as the table at the start of "Run programme".

Option "I" - Inner slope and crest levels.

Shows the cotangent of the lee side slope $\cot\beta$ and the crest elevations of the layers with respect to MSL. The value of $\cot\beta$ will be equal for all alternatives.

Option "S" - Sizes of blocks.

The thickness of one layer of material in the respective layer. In fact, the values are determined by

$$\tau = k_{\Delta} * \left[\frac{W}{\rho_r} \right]^{1/3} \quad (4.25)$$

Option "B" - Breadth of crests.

The width of the crests. Note that the number of primary armor blocks across the crest is simply derived by dividing b_1 by the size of block from option "S".

Option "L" - Layer thickness.

These values are determined by multiplying the size of blocks (option "S") by the number of blocks in the respective layer. Thickness of the filter layer, t_f , is allocated in the input section of the programme and will not be changed during the entire computation.

Option "A" - Average mass.

An exception is made here since average masses are shown of both feasible structures and rejected structures. The latter is meant to see what mass, which will be greater than the maximum allowable mass, W_{\max} , would actually be required for the construction layers.

Option "W" - Mass of armor stone.

This option shows the quarry categories allocated to each construction layer. Due to the limits of space across the screen or paper, no columns of T_r , H_{dD} , and Depth are shown. These columns have been replaced by column "i", which represents the index number of design storm condition.

Option "V" - Volumes per meter run.

The volume of each layer, as well as the volume of cut, in units of $[m^3/m']$.

Option "Q" - Quarry use.

The required quarry production can be shown for each alternative separately. Showing all alternatives might become time consuming or lead to excessive output on paper. Therefore a selection of structures you are interested in, can be made by choosing an index range of "i" and "j", representing design storm conditions and outer slopes, respectively.

The output table of each selected alternative shows the required volume of each quarry category (column Requir. $[m^3/m']$). The output table of the optimum alternative is given in Table 2.2. In the column Requir. [%] the required volume is given in percentage of the total volume of the alternative (excluding the volume of the filter layer), whereas column Yield [%] shows the quarry production in percentage of the total quarry yield. Usually the yield percentages of some categories will not suffice the percentages required for the structure. Therefore extra blasting has to be done. The amount of extra blasting is determined by the greatest value from

$$\text{factor} = \text{Requir. [\%]} / \text{Yield [\%]} \quad (4.26)$$

Column Gross [%] shows the gross quarry production determined from

$$\text{Gross [\%]} = \text{factor} * \text{Yield [\%]} \quad (4.27)$$

while the last column shows the excess production in percentages

$$\text{Excess}[\%] = \text{Gross}[\%] - \text{Requir.}[\%] \quad (4.28)$$

The category responsible for the greatest value of eq. 4.26 can easily be traced as its excess production will be equal to zero. The smaller the sum of excess percentages the more economic the use of the quarry.

Note that excess percentages are computed afterwards, which means that they have not been included in the optimization!

Option "P" - Prices per cubic meter.

Prices correspond with the unit prices of the applied quarry category.

Option "C" - Construction costs per meter run.

Construction costs of each layer, including dredging costs, per meter run. Values have been determined by multiplying the volumes (option "V") by the unit prices (option "P"). The last column shows the summation of all layers, representing the construction costs, $\$C$.

Option "O" - Overview of costs.

Columns of the construction costs, $\$C$, the capitalized maintenance costs, $\$M$, and the total costs, $\$T$, of each alternative.

Option "M" - Main menu.

This option will bring you back to the main menu.

4.5.2. Option "C": Cost curves.

Selecting option "C" from the Main Menu gives access to a graphical presentation of the costs versus design wave height. Answering "N" to the question "Draw TOTAL COSTS curves only?" will show the total costs curves, $\$T$, as well as the curves of construction costs, $\$C$, and maintenance costs, $\$M$. Often this may result in an accumulation of lines and points near the confluence of construction and total costs curves. Therefore the total costs curves can be selected separately by answering "Y" to the question above.

The abscis is given by deep water design wave heights H_{dD} , while the costs per meter run are given along the ordinate Fig. 4.15. Each time the <ENTER>-button is pressed, the curves belonging to the successive milder slope appear on screen. Slope angle $\cot\alpha$ in question is mentioned in the legend. The points of each graph are plotted first after which a Bezier polynomial curve is drawn along the points. The curve passes through the first and last

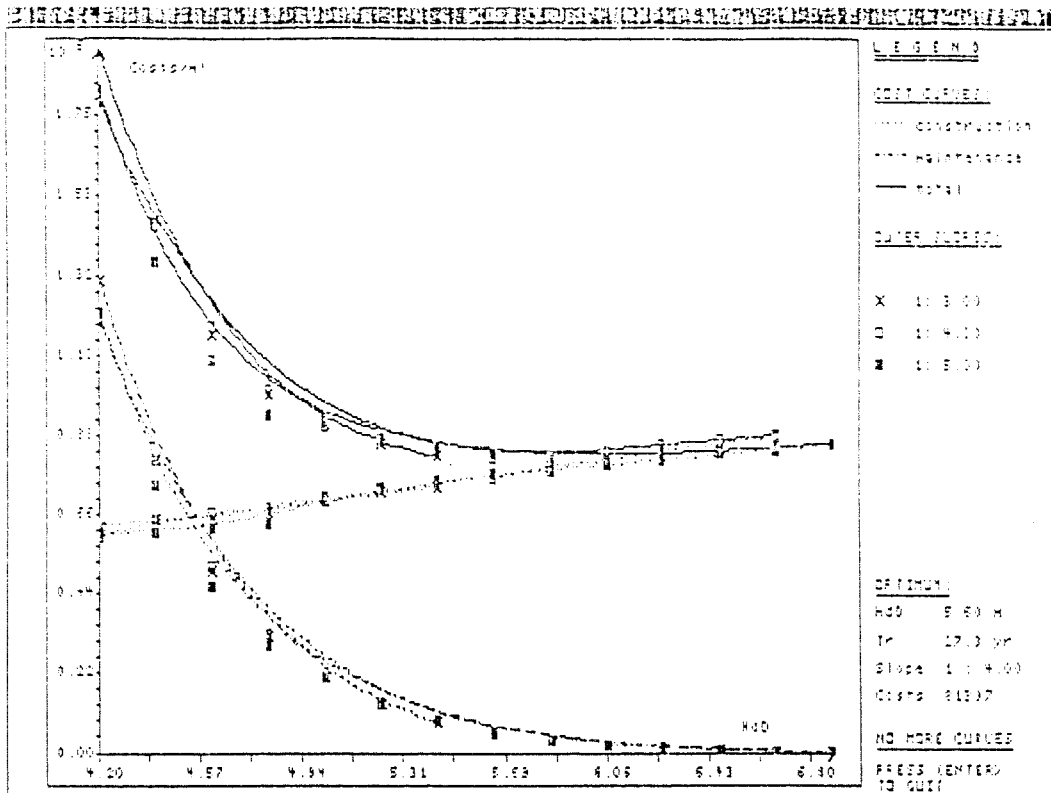


Fig.4.15. Curves of costs versus design wave height, H_{DD} .

point, and passes as close as possible to each of the other points. Curves are cut off as soon as the required primary armor mass passes the maximum W_{max} . The minimum required number of guiding points for the Bezier polynomial is three. If less, the cost curves belonging to the particular outer slope cannot and will not be drawn at all.

The last set of curves is marked by "NO MORE CURVES" in the lower right corner of the screen. At the same time the optimum design conditions together with the resulting total costs, are presented as well. A screen dump can be made by pressing the <PRINTSCREEN>-button of your keyboard. After the next strike of the <ENTER>-button the question "Send costs curves data to AutoCAD? (Y/N)" appears. If you wish to have a smoother plot of the curves (for a report, for example), you have to answer positively. The appropriate data will then be send to a data file, which can be read by programme CRVRUMBA.LSP, residing in AutoCAD (Section 5.). Any conceivable name, without extension ".DAT", may be given to the data file. Default name is "\PROG\CRVRUMBA.DAT". Finally the programme switches to the main menu.

4.5.3. Option "A": Structural data of optimum design to AutoCAD.

In order to make a drawing of the optimum cross-section by means of programme DRWRUMBA.LSP, appropriate geometrical data have to be sent to a data file (Option "A" from the main menu). Moreover, the required quarry category per construction layer accompanied with their respective unit price, are sent as well. Due to the memory limitations of a micro-computer it is not possible to keep all geometrical data of each single structural alternative in store. Therefore the computation has to be repeated for the optimum design conditions. These conditions were shown in the legend of the costs curves picture.

Option "A", thus, invokes practically the same sequence of procedures as option "R", where, in this case, only one structure (the optimum) is computed. Required minimum and maximum design wave height have already been set equal to the optimum design wave height. Similarly, the values for minimum and maximum $cot\alpha$ correspond with the optimum outer slope. After confirmation of these values the computation is carried out in accordance with Section 4.3.2. to Section 4.3.9. Then the default data file name "\PROG\DRWRUMBA.DAT" arises. If you are satisfied with the given name, answer "Y" to the question "Is the file name correct? (Y/N)". A negative answer gives access to type any other name without extension ".DAT". After all data have been sent the programme again switches to the main menu.

The procedure above allows the opportunity to send geometrical data of executable structures not based on optimum design conditions. In that case required minimum and maximum values of H_{dD} and $cot\alpha$ have to be entered, after selecting option "A". Appropriate values can be taken from the output tables of option "O" or "L". Be sure that minimum and maximum values are equal to each other or, at the least, no more than one structure is computed. An error message appears when this condition is not fulfilled. Selecting the default data file at the end might overwrite (optimum) geometrical data previously made. In that case choose another name.

5. Description of the programme CADRUMBA.LSP.

5.1. The RUMBA.DWG prototype drawing.

After loading AutoCAD its Main Menu appears on the text display screen. The file containing the prototype drawing, RUMBA.DWG, resides on directory C:\DWG. To display the drawing on the screen reply as follows:

```
Enter selection: 2
Enter NAME of drawing: \DWG\RUMBA
```

Don't type extension ".DWG" as it is automatically appended to the name you entered. Once the drawing name has been entered the drawing editor is loaded and the prototype drawing appears on the screen. The display shows a rubble-mound breakwater cross-section, which corresponds with the general lay-out of Fig. 2.2. Although not visible directly, each construction layer has been drawn in a separate drawing layer. Table 5.1. shows the drawing layer settings. The drawing layer "CUT" is intended for the polyline indicating the part of the structure in cut. Drawing layer "REF" contains the MSL-line signifying the level of reference. The remaining continuous white lines and white texts are drawn in drawing layer 0 (= zero). This is the standard layer automatically created by AutoCAD.

CONSTRUCTION LAYER	DRAWING LAYER	COLOR	LINE TYPE
Primary armor	PRIM	2. yellow	continuous
Secondary armor	SEC	1. red	continuous
Lee armor	LEE	11.	continuous
Toe protection	TOE	4. cyan	continuous
Core	CORE	3. green	continuous
Filter layer	FILTER	6. magenta	continuous
	CUT	13.	continuous
	REF	7. white	dashdot
	0	7. white	continuous

Table 5.1. Layer settings of the prototype drawing.

Three text style names have been defined:

- 1) STANDARD - used in the tables
- 2) LEVELTEXT - used for depicting the elevations
- 3) GREEKS - used for α and β in the table of design parameters.

As soon as the command prompt appears at the bottom of the screen, the drawing editor is ready for further action.

5.2. Drawing of the optimum cross-section.

Before the (optimum) cross-section can be drawn the guiding programme CADRUMBA.LSP has to be loaded and evaluated first. This file can be found in directory C:\PROG. Therefore, enter from the keyboard (without extension ".LSP!")

```
Command: (load "/PROG/CADRUMBA")
```

Note that forward slashes have been used. Back slashes "\" are interpreted as text control characters. To obtain one back slash in a string you have to type "\\". Hence, the string can also be entered as follows:

```
Command: (load "\\PROG\\CADRUMBA")
```

If the operation fails, the name of the file is returned as a string. Successful operation is rewarded with the message

```
Loading /PROG/CADRUMBA.LSP. Please wait ...
```

Depending on the hardware it takes about 15 seconds to one minute to load and evaluate the AutoLISP expressions. The operation has finished as soon as C:EDITRUMBA is returned, followed by AutoCAD's command prompt. Three command functions have now been added to AutoCAD, namely INPUT, DRAWRUMBA, and EDITRUMBA. These functions can be used as long as you work within the environment of drawing RUMBA.DWG.

To visualize the cross-section of the optimum breakwater determined by OPTRUMBA.PAS, its data have to be entered first. Type

```
Command: INPUT
```

The command function responds with

```
Name of data file </PROG/CADRUMBA.DAT>:
```

Again the default name is "/PROG/CADRUMBA.DAT", which can be confirmed by pressing either the <ENTER>-key, or the space bar

(in AutoCAD both may be used to enter a command). If you want data from any other file, type its name, including path-name, without extension ".DAT". If, by mistake, the file does not exist, all AutoLISP expressions defined within function INPUT, will scroll through the command area at the bottom of the screen. Don't panic, just try again. Successful operation is responded with "nil".

All preparations have been done now. The optimum breakwater cross-section will be drawn after you have entered

Command: DRAWRUMBA

The drawing process starts with completely erasing the prototype cross-section. Next design storm and some important structural data appear in the design table. Then the cross-section is drawn according to the geometrical data of the data file. Switching from one drawing layer to the other is shown in the information bar on top of the screen. After the cross-section has been drawn the construction table is filled in. The total of costs shown in the table corresponds with the construction costs, $\$C$. The last drawing actions consist of levels and dimensions. Command function DRAWRUMBA has finished as soon as AutoCAD's command prompt reappears. The drawing on the screen now looks like Fig.2.7. All information given on the screen is equal to the data of the optimum structure determined with OPTRUMBA.PAS.

If you wish to have the picture on paper, simply activate the PLOT command. If you save the current drawing as a ".DWG" file (command SAVE), don't forget to choose another name. The prototype drawing will be overwritten, otherwise. For quick views within the drawing editor later on, the current drawing can also be stored as a slide file of extension ".SLD". Enter the MSLIDE command followed by the file name you wish (without extension ".SLD").

When you leave the drawing editor and return to the Main Menu, use the QUIT command. The END command will overwrite the prototype drawing with the current drawing. Whenever you return to the Main Menu of AutoCAD, the command functions added as well as the variables created within the functions (explained in the next section), are lost. Thus, the whole sequence of commands explained in section 5.1 and this section, has to be repeated each time you start from the Main Menu.

So far hardly any AutoCAD experience is required from the operator. On the other hand very little of the facilities of a CAD-system have been used yet. The drawing editor offers the facility to make changes to the optimum cross-section. The effects of these adjustments on volume quantities and construction costs can be computed easily. The next two sections go deeper into the matter.

5.3. Effects of changing values of geometrical variables.

5.3.1. Useful features of the AutoLISP programming language.

The command functions added to AutoCAD have been built up of AutoLISP expressions. Single AutoLISP expressions, however, can also be entered any time during an editing session. This implies that, for instance, values of variables, once created while editing, can be changed any time during that editing session.

The basic assignment function in AutoLISP, expressed in the convention used in the AutoLISP Programmer's Reference, is defined as:

```
(setq <var1> <expr1> [<var2> <expr2>] ...)
```

This function sets the value of variable <var1> to <expr1>, <var2> to <expr2>, and so on. Note that AutoLISP expressions have to be written between brackets.

A value can be verified by entering the variable name, preceded by an exclamation point "!". The current value is returned underneath. If the variable does not exist, "nil" is responded.

In order to make effective use of AutoLISP expressions, study of the AutoLISP Programmer's Reference is highly recommended by the author.

5.3.2. Effects of changing values of geometrical variables.

The programme structure of the command function DRAWRUMBA largely corresponds with the procedure Construction Costs within the programme OPTRUMBA.PAS (Section 4.4.9). Thus, the structural computations described in Sections 4.4.3. up to and including 4.4.8, are not executed by the command function DRAWRUMBA. The variables necessary to calculate the contours and volumes of the construction layers, have therefore been taken from the data file with the help of the INPUT command. The values have been assigned to variables, whose names, for the greater part, are equal to the names used in the Pascal programme. Table 5.2. shows the variables created and allocated within command function INPUT.

The first three variables from the table, and the minimum and maximum mass of the layers will not be used for the computation of the cross-section. The values are simply included for presentation purposes. AutoCAD does not distinguish between upper and lower case characters. In the table upper case characters are used for the sake of clarity.

VARIABLE DESCRIPTION	VARIABLE NAME
deep water design wave height	HdD
storm surge level	h
return period of design storm	Tr
cota and cotβ	cotA cotB
mean depth	MeanDepth
crest elevation of breakwater	h1
lower bound level of lee armor	hLee
crest elevation of toe protection	hToe
width of breakwater crest	b1
width of berms	bBerm
thickness of filter layer	tFilter
minimum and maximum mass per layer	W1Min W1Max W2Min W2Max WLeeMin WLeeMax WToeMin WToeMax WCoreMin WCoreMax
block size per layer ($= k_{\Delta} * (W/\rho_r)^{1/3}$)	Block1 Block2 BlockLee BlockToe
unit price per layer and cut	U1 U2 ULee UToe UCore UFilter UCut
number of blocks:- thickness primary layer	nBlockt1
- thickness second layer	nBlockt2
- thickness lee layer	nBlocktLee
- thickness toe protection	nBlocktToe
- width toe protection	nBlockbToe

Table 5.2. Names of variables created within command function INPUT

Once created within INPUT, the values allocated to the variables can be checked. For example, the value of h1

```
Command: !h1
7.999868
```

During the DRAWRUMBA process more variables are created. Their values can be retrieved as well, afterwards. Names of variables of importance are mentioned in Table 5.3.

After the optimum cross-section has been drawn on the screen (Section 5.2.), it is possible to change the values defined within command function INPUT, and to see how they affect the construction. For example, you want to know the construction costs in case the lee side slope has to be built with a milder slope. The current slope is 1 : 2.0 and it has to be set to 1 : 2.5. Type

VARIABLE DESCRIPTION	VARIABLE NAME
thickness of primary armor layer	t1
thickness of secondary armor layer	t2
thickness of lee armor layer	tLee
thickness of toe protection	tToe
width of secondary armor crest	b1
width of core crest	bCore
level of secondary armor crest	h2
level of core crest	hCore
level of bottom	hBottom
level of base (Fig. 4.13)	hBase
level h_{2Sea} at fill (Fig.4.13.a)	h2SeaFill
level h_{2Sea} (Fig.4.13)	h2Sea
level h_{2Lee} (Fig.4.7)	h2Lee
level of filter sea side (Fig.4.13)	hFilterSea
level of filter lee side (Fig.4.7)	hFilterLee
all vertices	p1 ... p40

Table 5.3. Some names of variables created within command function DRAWRUMBA.

```
Command: (setq cotB 2.50)
2.500000
```

Next activate

```
Command: DRAWRUMBA
```

The same sequence of actions will be executed as described in Section 5.2. The values of the table will be erased as well. The new value of $\cot\beta$ appears in the table and the cross-section is drawn according to the new value (Fig.5.1.). If desired, the new drawing can be saved (command SAVE), plotted (command PLOT), and/or a slide file can be made of it (command MSLIDE). Slide files can be shown during editing by means of the command VSLIDE, followed by the name of the file, without extension ".SLD".

It is allowed to change more than one variable before the DRAWRUMBA is activated. The next example checks the number of blocks on the breakwater crest, increases the number to 8, and sets variable hLee 1.0 m below MSL. The sequence of commands is as follows:

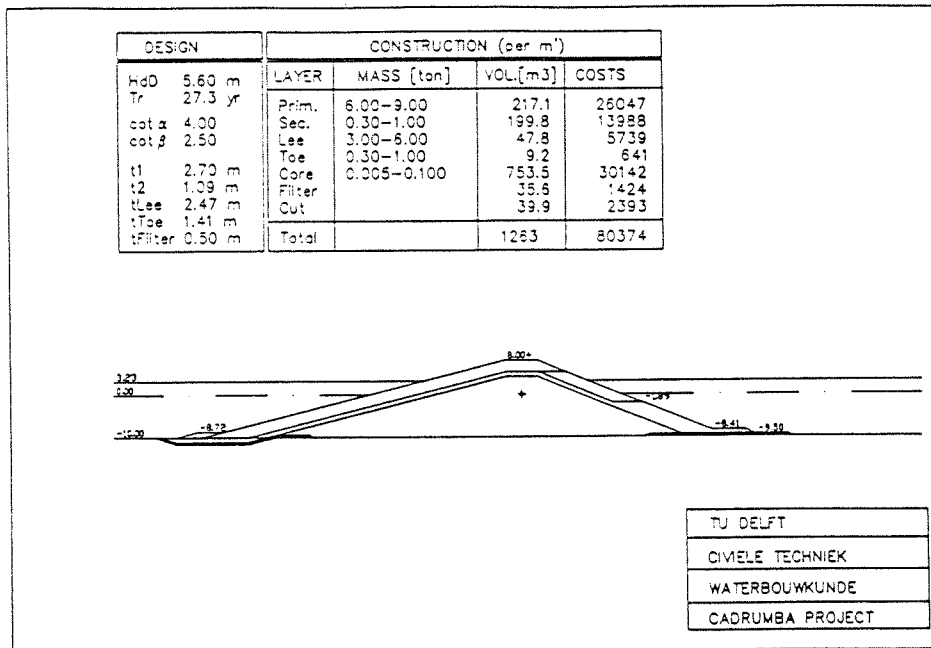


Fig.5.1. Adjustments to the cross-section: milder leeward slope.

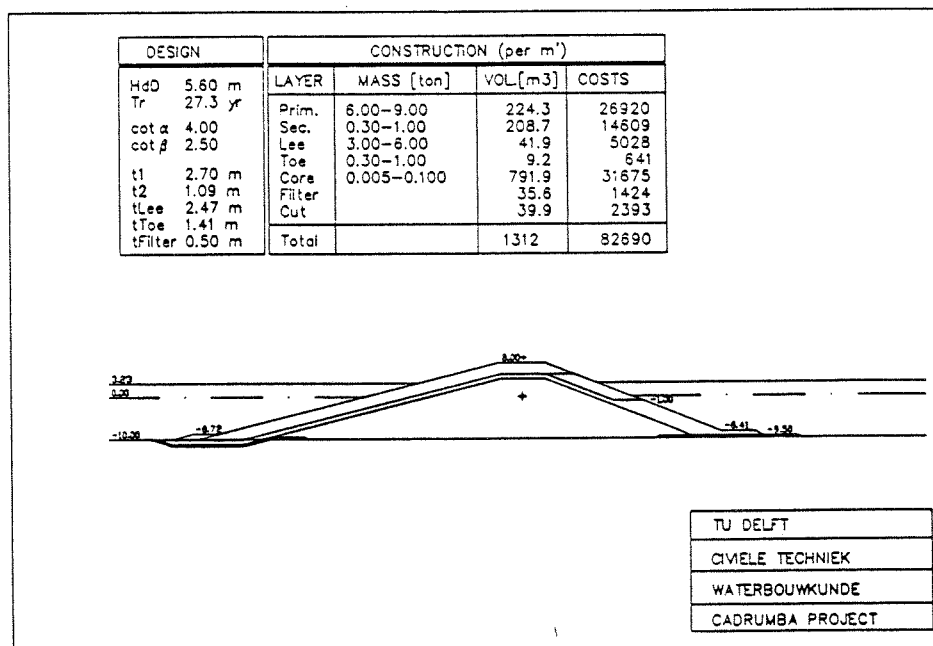


Fig.5.2. Adjustments to the cross-section: crest width increased, higher level of h_{lee} .

```
Command: (/ b1 Block1)
5.999999
Command: (setq b1 (* 8 Block1))
10.788780
Command: (setq hLee (- 0 1.0))
-1.000000
Command: DRAWRUMBA
```

The resulting cross-section is shown in Fig.5.2.

Only changes to variables defined in the command function INPUT (Table 5.2.) will have effect on the computation. The variables defined within DRAWRUMBA (Table 5.3) are related to the INPUT variables. Therefore, changing the DRAWRUMBA variables is meaningless, since they will be reset during execution of DRAWRUMBA, according to the values of the related variables.

In principle any existing variable can be changed. However, some changes will heavily violate the optimization process executed in OPTRUMBA.PAS. If, for example, variable cota is changed the resulting cross-section will most probably not fulfill the optimum design conditions. In that case you'd better leave AutoCAD, execute a run with OPTRUMBA.PAS, select an alternative of the required outer slope close to the optimum (if not the optimum), and send its data to the data file. Then return to AutoCAD and execute the sequence of commands as described before.

The conclusion is permitted to state that AutoLISP offers a handy tool to evaluate the cross-section and construction costs after minor changes have been made. However, up to now the structure has been drawn according to a fixed sequence of AutoLISP expressions within DRAWRUMBA. That's why the cross-sections still look basically the same. By using AutoCAD's editing commands it is possible to deviate from the basic structure.

5.4. Changes to the cross-section by means of EDIT commands.

Depending on the AutoCAD experience of the operator any changes can be made to the structure. AutoCAD therefor offers plenty of editing commands. Especially the PEDIT command is very usefull. After changes have been made satisfactory, enter

```
Command: EDITRUMBA
```

The EDITRUMBA command function will rewrite the construction table according to the new volumes. The cross-section itself and the values of the design table are left untouched.

Command function EDITRUMBA contains AutoLISP functions, which provide access to entities on the graphics screen. To select the appropriate construction layer on the drawing, use is made of so-called filters. The filter specifications consist of the name of

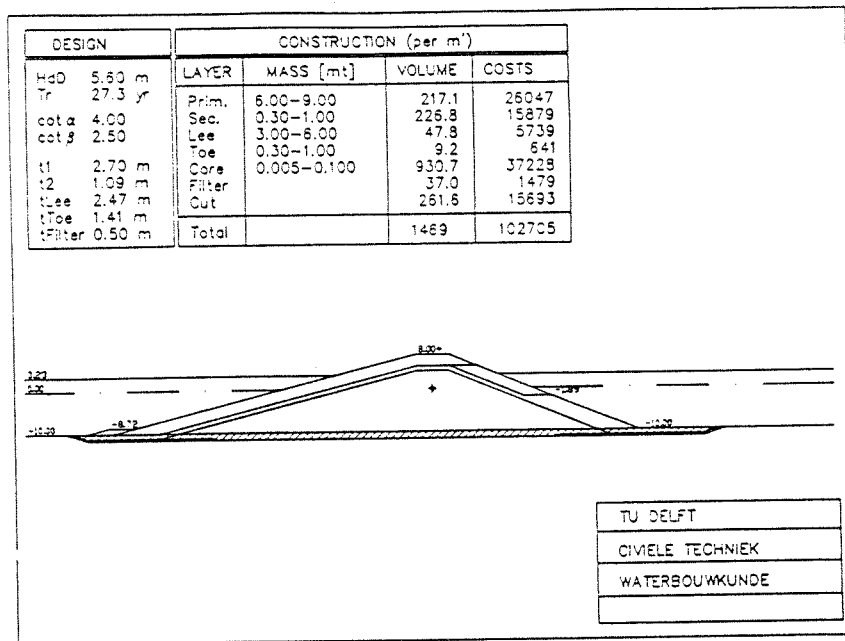


Fig.5.3. Adjustments to the cross-section: subsoil removed.

the drawing layer and the entity name "POLYLINE". The AutoLISP function scans the entire drawing and creates a selection-set containing the entities that match the specified criteria. The volume per meter run, or in other words, the area enclosed by the polyline, can be withdrawn from the entity/entities by means of the AREA command.

Actually the same entity access functions are also used within DRAWRUMBA command. DRAWRUMBA has built up the drawing such that, except for the filter layer, one entity will be found per drawing layer. For example, the filters to select the primary armor layer read: layer "PRIM", entity "POLYLINE". The filter layer consists of two parts which will yield two selected entities. The areas of both entities are added and the total amount appears in the construction table.

As stated before, any changes can be made to the drawing. To select the appropriate construction layers, the restrictions are that the construction layers remain in the corresponding drawing layers (see Table 5.1.), and the lines have to be of entity "POLYLINE". Figure 5.3. shows an example where part of the (bad) subsoil has to be dug before the structure can be built. In Fig.5.4. the contours of the bottom are taken into account as well.

If, by accident, DRAWRUMBA is called after changes have been made by mouse or digitizing tablet, these changes will be lost. The cross-section will then be redrawn according to the current values of the INPUT variables, and the fixed sequence of AutoLISP commands.

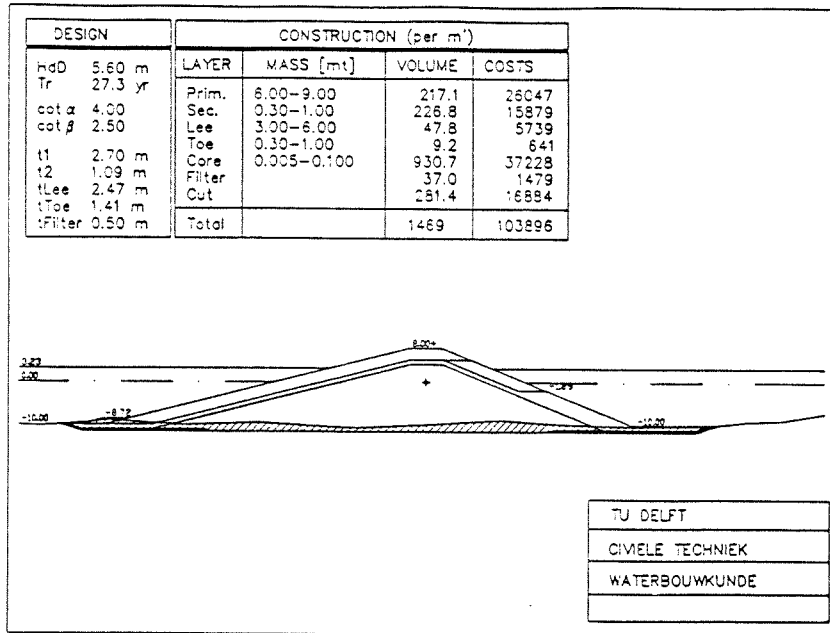


Fig.5.4. Adjustments to the cross-section: subsoil removed, taking into account the contours of the bottom.

6. Description of CRVRUMBA.LSP.

The AutoLISP programme CRVRUMBA.LSP does not form part of the structural programmes described in the previous chapters. It is written purely to make a smooth plot of the cost curves shown in OPTRUMBA.PAS, option "Cost Curves".

From the AutoCAD Main Menu the prototype drawing has to be called:

```
Enter selection: 2
Enter name of drawing: /DWG/CRVRUMBA
```

The drawing already shows a neat presentation of cost curves within a box. Parts of the coordinate axes are placed outside the box. Lateron, if necessary, they can be connected to the main axes drawn inside the box.

Besides drawing layer "0", ten layers have been added. The number of layers added corresponds with the maximum number of outer slopes to be defined. The layer settings are shown in Table 6.1.

LAYER NAME	NR.	COLOR	LINE TYPE
0	7.	white	continuous
1	1.	red	continuous
2	2.	yellow	continuous
3	3.	green	continuous
4	4.	cyan	continuous
5	5.	blue	continuous
6	6.	magenta	continuous
7	7.	white	continuous
8	10.		continuous
9	11.		continuous
10	14.		continuous

Table 6.1. Layer settings of the prototype drawing CRVRUMBA.

Load the programme by entering

```
(load "/PROG/CRVRUMBA")
```

Loading has finished as soon as the one and only function defined, CURVES, is returned. Before entering the CURVES command parts of the drawing have to be erased by hand. At the least remove the current cost curves to avoid confusion with new curves to be drawn. In principle all the entities between the main axes

and the box lines have to be erased. These actions can be postponed until the CURVES command has run.

After preparations have finished, enter

Command: CURVES

which responds with the prompt

Name of the data file (without extension .DAT)
</PROG/CRVRUMBA.DAT>

Type the name of the data file which was created at option "Cost Curves" of OPTRUMBA.PAS (Section 4.5.2). One single strike of the <ENTER> key or the space bar is needed when data have to be read from the default data file. The data file consists of slope data, optimum design conditions, and the coordinates of each guiding curve point. The latter values correspond with the output table of "Overview of Costs" from OPTRUMBA.PAS.

The CURVES command continues with drawing the cost curves. The set of curves, belonging to the first outer slope, are drawn in layer "1", the second in layer "2", and so on. The outer slope at consideration is written outside the box at screen coordinates (10,7) and below. Drawing of the curves takes much more time than in the Pascal programme. For each curve the sequence of commands is:

- draw first section of the polyline
- draw second section of the polyline
- join second section with previous section
- draw next section of the polyline
- join it with previous sections
- etc.
- draw last section of the polyline
- join last section with previous sections
- draw Bezier spline curve using the coordinates of the polyline as guiding points.

Again a minimum of 3 points is required for the Bezier function. If less, the curve cannot be drawn.

The values along the vertical axis may range from 0 to 10. The costs, however, usually are far beyond this range. To fit the values within the range of 0 to 10, the actual costs were divided by the variable Multiplier. The value of Multiplier was taken from the data file. The greatest value of the total costs, $\$T$, determines the value of Multiplier, which in general can be written as

$$\text{Multiplier} = 10^n \quad , \text{ where } n = 1, 2, 3, \dots$$

The applied value of Multiplier is written between brackets at screen point (2.0,9.5).

After command function CURVES has finished, some editing efforts have to be undertaken by the operator. The axes, or part of the axes, outside the box can be connected to the main axes (AutoCAD command MOVE). Never move the main axes inside the box. Their coordinate values correspond with the AutoCAD drawing coordinates, which, in turn, form the basis of the costs curves drawing. Move the Multiplier value to a suitable place along the ordinate axis. Similarly, the values of outer slopes and optimum design conditions have to be replaced by the new values.

Finally, when you have edited the drawing to your entire satisfaction, it can be send to the plotter. The curves illustrated in Fig.4.15 may then look like the curves of Fig. 2.6. Without doubt the latter figure will be much more suitable for presentation purposes.

LIST OF SYMBOLS

In general, index:	1	= primary armor layer
	2	= secondary armor layer
	lee	= lee armor layer
	toe	= toe layer
	core	= core
	f	= filter
	berm	= berm
	b	= bottom
	base	= fictitious base
A	= 1.	erosion area in a cross-section
	2.	empirical coefficient
B	=	empirical coefficient
C	=	coefficient
	index: k	= index number
Dam%	=	damage percentage
D_{n50}	=	nominal diameter of the stone
$E(\$_M)$	=	expected maintenance costs during a period
	index: Δt	= storm duration
	yr	= one year
F	=	freeboard
H_d	=	design significant wave height, index: D = deep water
		L = local
H_s	=	significant wave height, D = deep water
		L = local
H_t	=	transmitted significant wave height
K_{sh}	=	shoaling coefficient
K_t	=	transmission coefficient
L	=	expected life-time of the structure in years
L_D	=	deep water wave length, $(g/2\pi) * T_m^2$
MSL	=	mean sea level
N	=	number of waves, $\Delta t / T_m$
P	=	permeability coefficient of the structure
$P()$	=	cumulative probability distribution
R	=	run-up, index: s = significant
	*	= run-up parameter, $F/H_{dL} * \sqrt{(s_p/2\pi)}$
S	=	damage level, A/D_{n50}^2
SWL	=	still water level
\$	=	costs, index: C = construction
		Dam = damage
		M = maintenance
		T = total
T	=	wave period, index m = mean
		p = peak
		r = return period of storm events
W	=	average mass of the quarry stone
	index: max	= maximum

LIST OF SYMBOLS (continued)

a, b, c	= coefficients
d	= depth at site, index: m = mean depth
f_i	= factor, $D_{n50}(\text{lee, overtopped})/D_{n50}(\text{primary, non-overtopped})$
f_o	= stability factor, $D_{n50}(\text{overtopped})/D_{n50}(\text{non-overtopped})$
g	= gravitational acceleration
h	= storm water level index: ht = high tide lt = low tide
i	= 1. interest rate per year 2. index of H_{dD} -loop
i_b	= bottom slope
j	= index of $\cot\alpha$ -loop
k	= wave number, $2\pi/L_D$
k_Δ	= packing coefficient
n	= number of armor stones in the layer
n_D	= number of design waves
n_S	= number of design outer slopes
$p(\)$	= probability density function
pwf	= present worth factor
s_m	= fictitious wave steepness, $H_{sL}/(g/2\pi)T_m^2$
s_p	= fictitious wave steepness, $H_{sL}/(g/2\pi)T_p^2$
t	= thickness of the armor layer
Δt	= storm duration
α	= angle of seaward slope of structure
β	= angle of leeward slope of structure
γ	= shape parameter
γ_{br}	= breaker index
Δ	= relative mass density, $(\rho_r - \rho_s)/\rho_s$
ϵ	= location parameter
ν	= number of storm events per year
ρ	= mass density, index: r = rock s = sea water
θ	= scale parameter
ξ_m	= surf similarity parameter, $\tan\alpha/\sqrt{s_m}$ index: i = transition from plunging to surging waves

LIST OF REFERENCES

- BREAKWATERS Design and Construction, 1983. Proceedings of the Conference, ICE, London.
- Bruun, P. (Editor), 1985. Design and Construction of Mounds for Breakwaters and Coastal Protection. Elsevier, Amsterdam, 938 pp.
- Delft University of Technology. Lecture notes:
- f11A Coastal Engineering, Volume I: Introduction
 - f5N Coastal Engineering, Volume III: Breakwater Design
 - b76 Korte Golven
 - b78 Windgolven
- Gaillard, P., 1988. Numerical Modelling of Wave-induced Currents in the Presence of Coastal Structures. Coastal Engineering, No.12: 63 - 81.
- Houmb, O.G., 1981. Latest Developments in Wave Statistics. In: P. Bruun, Port Engineering. Gulf Publishing Company, Houston, Texas: Chapter3, Appendix B, Part 1.
- Hwang, C.H., Tang, F.L.W., 1988. Wave Transmission by Overtopping due to Random Waves. Ocean Engineering, Vol.15, No.3: 261 - 270.
- Isaacsen, M. de St. Q., Mackenzie, N.G., 1981. Long-term Distributions of Ocean Waves: a review. Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol.107. No.WW2: 93 - 109.
- Le Mehaute, M., Wang, S., 1985. Wave Statistical Uncertainties and Design of Breakwater. Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol.111, No.5: 921 - 938.
- Shore Protection Manual, 1984. U.S. Army Corps of Engineers. Government Printing Office, Washington, D.C.
- Smith, O. P., 1987. Cost-effectiveness of Breakwater Cross-sections. Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol.113, No.5: 447 - 460.
- Soerjadi, R., 1966. Over de Berekening van de Profielgrootheden van Veelhoekige Doorsneden. Heron, Jaarg.14, No.3
- Stam, C.J.M., 1988. Wave Run-Up on Statically Stable Rock Slopes under Wave Attack (in Dutch). Delft Hydraulics, M1983, PartIII.
- Van der Meer, J.W., 1988-1. Rock Slopes and Gravel Beaches under Wave Attack. Doctoral thesis, Delft University of Technology. Also: Delft Hydraulics Communication, No.396: 152 pp.

Van der Meer, J.W., 1988-2. Low-crested and Reef Breakwaters.
Delft Hydraulics, Report H198/Q638.

Van der Meer, J.W., 1988-3. Stability of Cubes, Tetrapods and
Accropode. In: ICE, Design of Breakwaters, Proceedings of the
Conference on Breakwaters, Eastbourne, 1988. Thomas Telford,
London, Chapter 6: 59 - 68.