CONCEPTUAL DESIGN FOR THE BREAKWATER SYSTEM OF THE SOUTH OF DOSON NAVAL BASE: OPTIMISATION VERSUS DETERMINISTIC DESIGN

by

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KEYWORDS

Port and Coastal Infrastructure Engineering, rubble mound breakwater; economic optimisation; reliability analysis; fault tree establishment.

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ABSTRACT

In 2006 a Vietnamese Engineering Consultancy Company carried out a design study of a Naval Base at the location of the South of Doson Peninsula in Vietnam. A deterministic approach applied to the conceptual design of the breakwater system of the Naval Base resulted in a cross-section with a big armour layer and a rather low crest level. This paper applies an economic optimisation approach to derive the optimal geometry for the breakwater system. Results show that when considering total costs over the lifetime of the breakwater system, the block size can be reduced and the crest height can be increased. Some failure to the armour layer can be accepted while the wave transmission and overtopping should be limited in accordance with their related costs.

1.   INTRODUCTION

As part of the national policy of developing the sea-related economy, a new deepwater seaport will be built in the North of Vietnam in the near future. Considering governing conditions such as natural conditions, existing infrastructures and transport networks, the area at the South of Doson peninsula has been considered of great interest. Doson peninsula situates in Haiphong province, the biggest port region in the North of Vietnam, and has a strategic role in the regional economy and security.

Doson peninsula is northwest-southeast oriented. The two sides of the peninsula have different morphology. While the shoreline on the north side of the peninsula is composed by many gently sloped sandy beaches, the south side of the peninsula has some narrow and steep beaches intermixing with rock and shingle. The beaches on the north side are now a famous tourist attraction with hundreds of thousand visitors each year. On the contrary, the waterfront on the south side seems to be a rather undeveloped region with only some mooring points and temporary berths for small fishing boats and crafts. The location and shape of Doson peninsula are presented in Figure 1.

In 2006, Construction Consultation Joint Stock Company for Maritime Building (CMB) carried out an investment study of building a complex in Doson, named the South of Doson Naval Base, including a modern naval base, ports, harbours and a breakwater system. This complex will be financed and owned by the Vietnamese Navy. Besides being a modern base for naval ships, some parts of the South of Doson Naval Base will also serve as commercial ports. The estimated throughput is 30 million tons per year. When the Naval Base is put into service, it is expected to become a major sea port of the North of the country, and share the cargo flow burden with Haiphong port which is now in difficulty due to its shallow and silted channel.

The Naval Base and its breakwater system will be constructed in phases. In the first phase, from 2010 to 2020, the breakwater system will have a length of 6km. The breakwater system is expected to be lengthen up to 16.7km as from 2020. The expected total lifetime of the breakwater system is 100 years. A master plan with the breakwater's length of 39.2 km has also been approved by the Vietnamese Navy and the Haiphong province's authorities. But the expansion will depend on the real effectiveness of the port. The proposed final layout of the Naval Base and its breakwater system in 2020 is presented in Figure 2.

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Figure 1: Location and shape of Doson peninsula

In the investment study of CMB, the master plan of the Naval Base including port areas, harbours and layout of breakwater system has been decided. The rubble mound type with an armour layer made by Tetrapod has been chosen for the breakwater. During their conceptual design for the breakwater structure, CMB applied only a deterministic approach. The strength of breakwater structure and the construction cost were used as criteria to select the design structure geometry. In the CMB's study, failure costs were not mentioned.
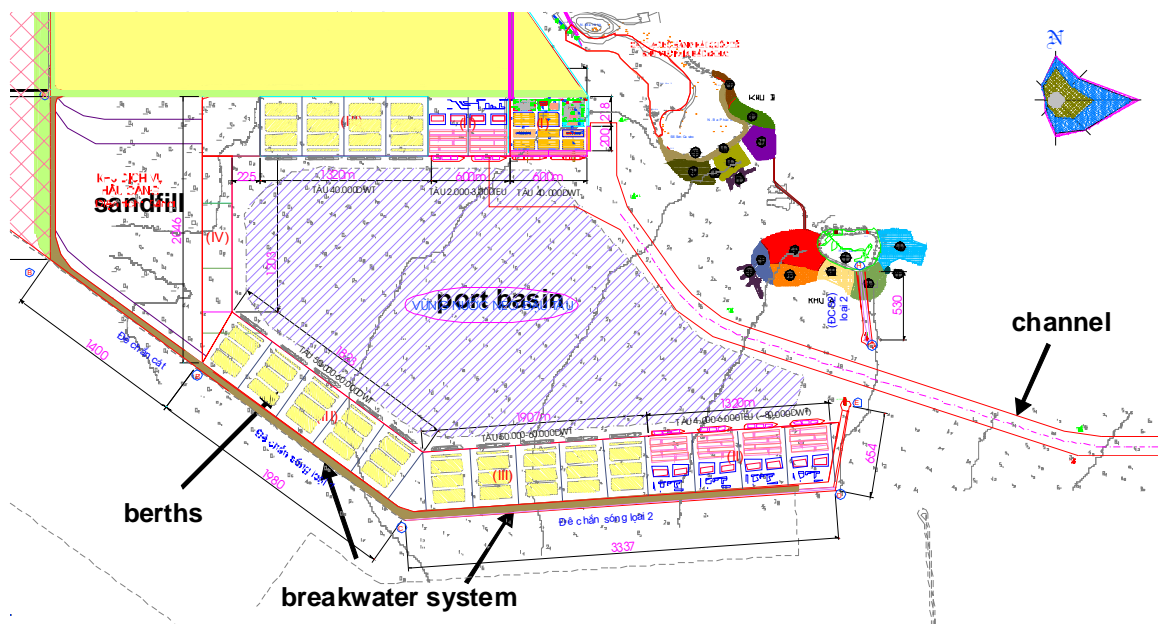

Figure 2: Master plan in 2020

This paper aims at deriving a probabilistic optimisation for the conceptual design of the breakwater cross-section, taking into account the uncertainties.

In an optimisation process, optimal criteria must be defined. Availability of the naval ships' navigation can be considered as an important criterion when optimising the design of a naval base. However, in this paper, the breakwater system of the South of Doson Naval base is optimised as for that of a commercial port. In case of a breakwater without amenities, and the probability of loss of life due to failure is very small, but the economic losses are serious, an economic optimisation is sufficient.

In the view of economic optimisation, the total lifetime cost is minimised within the constraints of design variables and operational conditions. The probability of total stoppage of port operations will be calculated by reliability analysis of governing failure mechanisms including the collapse of the breakwater, e.g. failure of the amour, failure of the toe, settlement and the failure of the hydraulic performance, e.g. the excessive wave overtopping, transmission, diffraction and reflection. These failure mechanisms contribute to the top event, i.e. the stoppage of port operation, as depicted in Figure 3 and are discussed in some next parts. The monetary damage caused by the relevant failure mechanisms and the downtime of port operations will be determined. Changes in investment costs when doing such things as altering the weight of blocks or raising the crest level to reduce the respective failure probabilities will be calculated as a function of the decision variables. Finally, optimal dimensions will be obtained by minimising the total investment cost while keeping a proper level of risk.
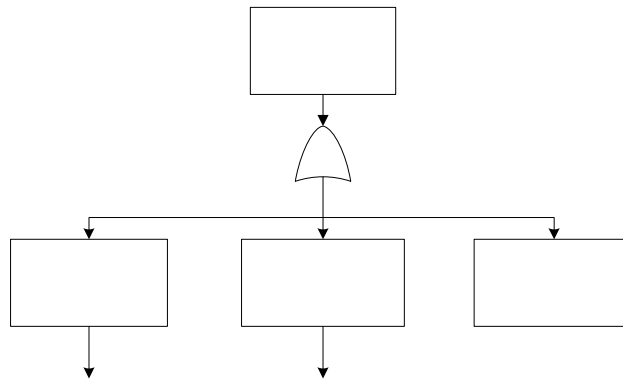

Figure 3: Fault tree with top event and intermediate events

2.   BOUNDARY CONDITIONS

The boundary conditions for breakwater design include the hydraulic boundary conditions (wave parameters, water level, tide and surge) and the geotechnical conditions (i.e. soil properties). For the three major categories of failure modes as depicted in Figure 3, different wave climates should be considered. The Ultimate Limit State (ULS) of breakwater happens under typhoon conditions, while the Serviceability Limit State (SLS) is affected by the normal cycle monsoon.

In the area of Doson, wind and tide data are available from long time observation. However, the wave data at the south side is not available. Only a short time wave survey was carried out before the design with two one-month observations. CMB also carried out some numerical modelling in order to derive the proper wave data for designing.

This paper does not focus on how CMB treated the data. Only the final result is presented here. The distribution laws of wave data are referred from literatures.

The significant wave height caused by typhoons at the construction site follows a Gumbel distribution with a mean value of 3.55m and a standard deviation of 0.4m. The wave period is 7s. During normal wind conditions, observation shows that the east (E) and southeast (SE) wind dominate the area. Despite the fact that it lasts for more than 3 months every year, the northeast (NE) monsoon hardly affects the site. The significant wave height during normal conditions can be assumed to follow a Weibull distribution with a mean value of 1,6m and a standard deviation of 0,2m. The wave period during normal conditions is 5.5s.

From the tidal observation, the high design water level is 3.75m +CD. During a typhoon, a storm surge can reach up to 1m.

Soil investigations have indicated that the geotechnical profile consists of some layers of clay before reaching the bedrock.

3.   DETERMINISTIC DESIGN

Structural alternatives

At the beginning of the design process, three structural alternatives were considered. They were the rubber-mound type, the composite type and the vertical caisson type. Considering the experience of local contractors, the ability of construction equipments and other aspects, the rubber mound type was chosen. The typical cross-section of the breakwater includes a core made of rocks and stones weighing between 10kg and 200kg, a secondary amour layer made of quarry rock with a weight of 1 ton, and a primary armour layer made of 2-layer Tetrapod. The Tetrapod block weights 12 tons. In the first phase (2010-2020) the crest height is 5.3m + CD. In the second phase (as from 2020) the crest height will reach 9.5m +CD by constructing a wave wall. Hereafter the cross-section without the wave wall will be mentioned as "low profile" and the cross section with the wave wall will be mentioned as "high profile". In the second phase, berths and quay will be constructed at the lee side of the breakwater. Under the breakwater, a soil improvement will be carried out, replacing the soft clay by sand.

Design formulae and geometry

The deterministic design was presented in the Design Report of CMB, mostly following common design guidelines. Some components are briefly presented here, including the Tetrapod primary armour, the secondary armour of rocks, the crest height and the crest width.

The weight of the Tetrapod block was calculated by Hudson's formula:

$$W = \frac{\gamma_B . H_{SD}^3}{K_D . \left(\frac{\gamma_B - \gamma}{\gamma}\right)^3 . \cot\alpha} \qquad (1)$$

where W is the median weight of blocks, H_{SD} is the characteristic wave height, K_D is the stability coefficient, γ_B is the unit weight of blocks, γ is the unit weight of water and α is the slope angle.

The thickness of the primary armour layer was calculated using the formula provided in CEM (2002):

$$r = n.k_\Delta . \left(\frac{W}{\gamma_a}\right)^{1/3} \qquad (2)$$

where r is the average layer thickness, n is the number of concrete armour units in the thickness, W is the weight of individual armour units and k_Δ is the layer coefficient.

The secondary armour layer made of rocks whose weight is about one-ten to one-fifteen that of the armour unit in the primary cover layer.

The crest elevation was decided from the allowable overtopping discharge. The crest width was calculated, as a general rule of thumb, to equal the combined width of three armour units, as in the following formula:

$$B = n.k_\Delta . \left(\frac{W}{\gamma_a}\right)^{1/3} \qquad (3)$$

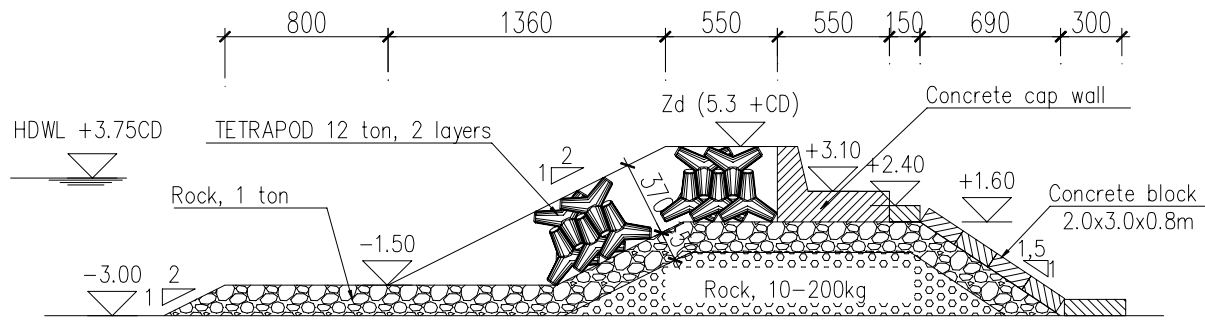The proposed cross-section is given in Figures 4 and 5 with both the high and low profile.

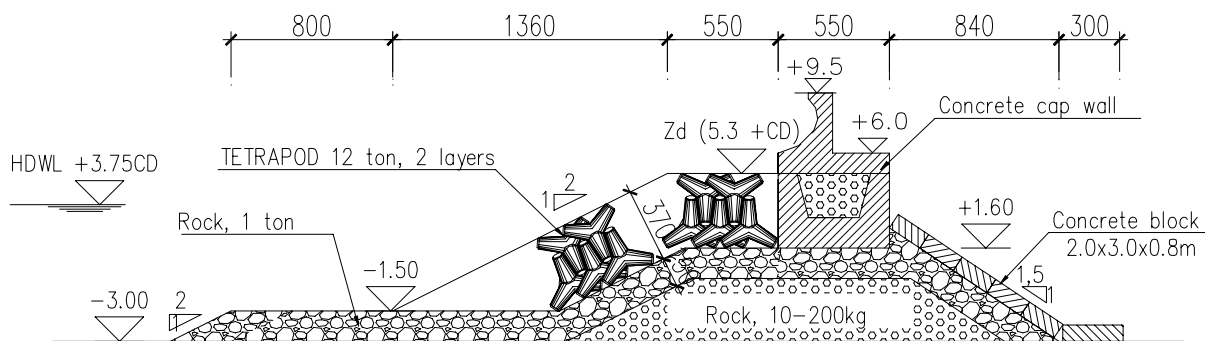**Figure 4: Typical cross-section, low profile (from 2010 to 2020)**

Figure 5: Typical cross-section, high profile (from 2020)

4.   FAILURE MECHANISMS AND RELIABILITY ANALYSIS

General in failure mechanisms

In order to make a probabilistic optimisation, the mechanisms that lead to total system failure will be defined. A failure is defined as a condition in which the structure loses its specified functionality. Each failure mode is described by a formula. The interaction (or correlation) between failure modes and the contributions of each one to the system failure can be defined in a fault tree. A fault tree describes the relationships between the failure of the system and the events leading up to this failure. The fault tree gives a logical succession of all events that lead to one unwanted "top event". An example of a fault tree with top event can be seen in Figure 3.

The objective of the breakwater in this case is "*to protect the harbour basin against unacceptable wave action*". Within this objective, the top event in the fault tree of the breakwater system is the "port downtime", i.e. the stoppage of the port operations due to malfunction of the breakwater system. Three major mechanisms causing this top event can be distinguished:

-Collapse of the breakwater.

-Excessive wave height in the basin.

-Obstruction of the entrance channel.

Collapse of the breakwater belongs to the Ultimate Limit State (ULS). As a result of collapse, the protective function of the breakwater will be lost. In this study, it is assumed that port operations are only active during normal conditions. Whenever a storm comes, the port stops its operation and the ships have to leave the berths and flee the storm. Therefore, "port downtime" is neglected during a storm, but the collapse of the breakwater leads to no longer basin protection afterward.

The second mechanism is the excessive wave height inside the harbour basin during normal weather condition. The port operations at the lee side, i.e. ship manoeuvering, berthing, loading and unloading etc., can only be carried out within a maximum allowable wave height in the harbour. Whenever the wave height exceeds the maximum allowable height, the breakwater system can be considered to fail. This failure mechanism occurs without severe collapse, and can be regarded as Serviceability Limit State (SLS).

The last mechanism is not closely related to the protective function of the breakwater. Obstruction of the entrance channel can be the consequence of a bad breakwater layout. A ship runs aground in the navigation channel will cause traffic obstruction and affect the port operations. Due to the fact that the master plan was fixed, this mechanism falls beyond the scope of this paper.

Each limit state is described by a limit state function (LSF). In the next parts, LSF of some major failure modes will be discussed.

Ultimate Limit State of the breakwater

The breakwaters are composed of many parts, e.g. armour layer, concrete cap, toe structure. Collapse of one of these parts will weaken the resistance of other parts and, eventually, the whole structure. ULS failures occur under an extreme condition, in this case a typhoon.

Failure modes in cross-section of a rubble mound breakwater can be the instability of the primary armour layer, the erosion of the toe, the excessive wave overtopping, the slip circle, the excessive settlement. In the proposed cross-section, the outer slope has a rather small steepness of 1/2 then the slip circle can be considered as of minor hazards. The soil improvement will help reduce the settlement. Therefore, in this paper, the first three mechanisms are discussed.

To calculate the hydraulic stability of the Tetrapod armour layer, the following formula is applied (Van der Meer, 1988):

$$\frac{H_s}{\Delta . D_n} = f_i (3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85) s_{om}^{-0.2} \qquad (4)$$

where H_s is the significant wave height at the location of the breakwater [m], N_{od} is the number of displaced units within a strip with width D_n, N is the number of wave, f_i is the coefficient denoting the difference between slope angle of the tested model and the real design, s_{om} is the wave steepness, D_n is the characteristic diameter of armour elements [m] and Δ is the relative density of material.

D_n and Δ can be calculated as follows:

$$D_n = \sqrt[3]{V} = \sqrt[3]{\frac{M}{\rho_c}} \qquad (5)$$

$$\Delta = \frac{\rho_c}{\rho_w} - 1 \qquad (6)$$

in which V is the volume of blocks, M is the mass of blocks [ton], ρ_c and ρ_w are the density of concrete and water, respectively [ton/m^3].

The following reliability function can then be derived:

$$Z = f_i \left(3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85 \right) s_{om}^{-0.2} \left(\frac{\gamma_c}{\gamma_w} - 1 \right) D_n - H_s \qquad (7)$$

The erosion of the toe is one of the major failure mechanisms. According to Van der Meer et al. (1995), the following formula is valid:

$$\frac{H_s}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6 \right) N_{od}^{0.15} \qquad (8)$$

The equation then is reformulated to the reliability function as following:

$$Z = \left(0.24\frac{h_t}{D_{n50}} + 1.6\right)N_{od}^{0.15}\left(\frac{\rho_r}{\rho_w} - 1\right)D_{n50} - H_s \qquad (9)$$

where h_t is the water depth at the toe, ρ_r is the density of rock.

The last ULS mechanism to be considered is the excessive wave overtopping. In the first phase when no structures are at the lee side of the breakwater, a high wave overtopping during an extreme condition is acceptable. In the second phase of investment, berths and quay will be built at the lee side of the breakwater. Therefore the wave overtopping must be less than a maximum allowable value, even during an extreme condition. This failure mode is analysed only within the "high profile". The relation between average overtopping discharge, wave height and armour characteristics is formulated as follows, according to EurOtop (2007):

$$\frac{q}{\sqrt{gH_s^3}} = 0.2C_r \exp\left(-2.6\frac{R_c}{H_s.\gamma_f}\right) \qquad (10)$$

in which R_c is the crest freeboard, γ_f is the influence factor for roughness, C_r is the reduction factor due to effect of armoured crest berm. C_r is defined as follows:

$$C_r = 3.06\exp\left(-1.5G_c / H_{m0}\right) \qquad (11)$$

where G_c is the crest berm width.

The reliability function can be derived:

$$Z = q_{allow} - q \qquad (12)$$

where q_{allow} is the maximum allowable overtopping discharge.

Serviceability Limit State of the breakwater

The SLS of a breakwater relates to the tranquillity of the port basin. During normal conditions, the breakwater system can be considered to fail if the wave height inside the basin excesses a maximally allowable height.

The wave inside the port basin is a combination of wave refraction - diffraction via the entrance, wave transmission through and overtopping the breakwater, and locally generated wave. The locally wave generated by wind or ship can be considered as minor and neglectable.

The wave height in the port basin can be calculated by the following formula:

$$H_{basin} = \left(K_{dif} + K_t\right)H_{sea} \qquad (13)$$

where H_{sea} is the wave height outside the basin, K_{dif} is the diffraction coefficient and K_t is the transmission coefficient.

According to the CIRIA, CUR, CETMEF (2007), K_t can be calculated as follows:

$$K_t = 0.46 - 0.3\frac{R_c}{H_s} \qquad (14)$$

where R_c is the crest freeboard and H_s is the incident wave height.

The reliability function for the excessive wave height is the following:

$$Z = H_{allow} - H_{ba\sin} \qquad (15)$$

where H_{allow} is the allowable wave height inside the port basin.

In principle, the wave heights inside a basin should be calculated on some locations, such as in front of the berth, at the turning basin or at the entrance. In this paper, only the wave height in front of the berth is analysed. In order to safely handle cargo, the allowable wave height at the berth is assigned to be 0.8m. According to CMB (2006), the diffraction coefficient at the berth location is 0.325 in accordance with the E wind direction, which gives the strongest diffraction phenomenon. The time duration for the E wind direction is about 3 months in total. In the second phase when the wave wall is built, the wave transmission phenomenon is omitted and the wave height inside the basin is only caused by diffraction. It seems obviously that if the wave height inside the port basin is acceptable for the "low profile", the excessive wave height will no longer happen with the "high profile". Therefore, the SLS hazards can be assumed to exist in only the first 10 years of the breakwater's lifetime.

Summary of failure probabilities of top event

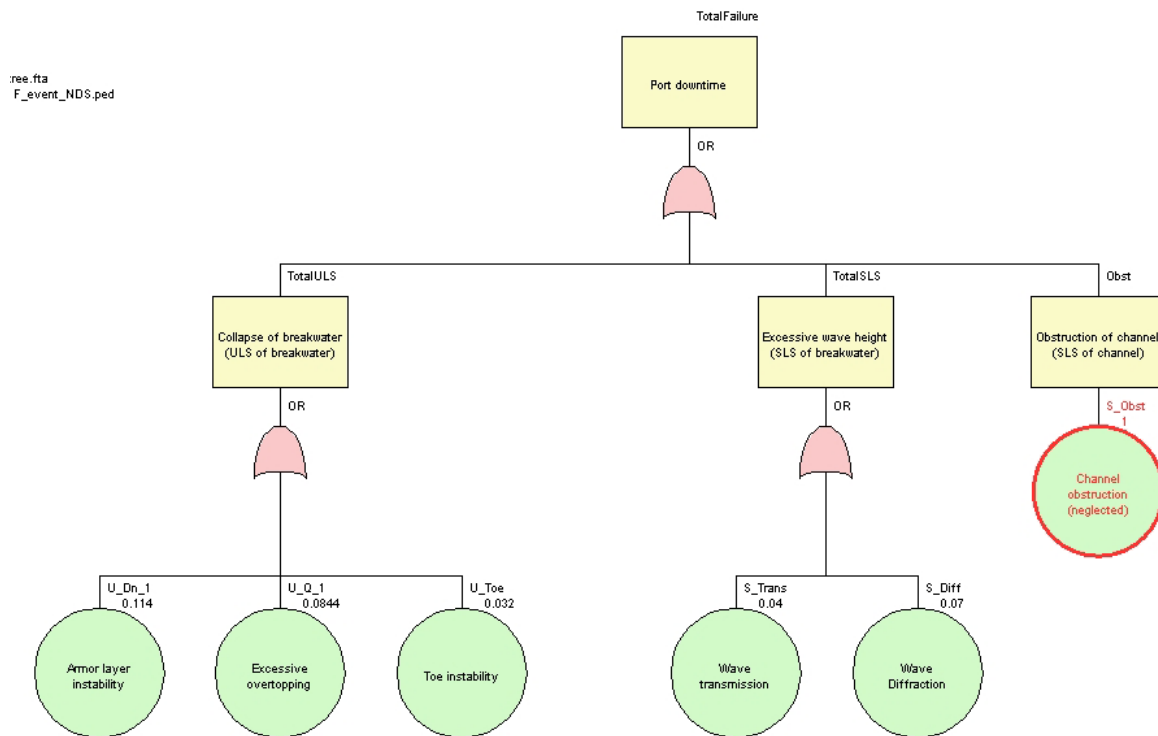Failure modes in ULS and SLS are included in the fault tree as depicted in Figure 6.

Figure 6: Fault tree analysis of breakwaters

Table 1 summaries input parameters applied for the reliability function and failure analysis. Some computing programs are used to facilitate the calculation process. VAP[5] is used to calculate the failure probability of each individual failure mode, while OpenFTA[6] is used to analyse the fault tree. An example of OpenFTA's script and its results is given in Figure 7.

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[5] VAP is a software for stochastic variables and reliability analysis provided by Institute Structural Engineering IBK, Zürich, Switzerland.
[6] OpenFTA is a free and open source software for fault trees construction and analysis provided by Formal Software Construction Ltd., Cardiff, Wales, United Kingdom.

No.	Variables and parameters	Type	Distribution parameters	
			μ	σ
1.	H_s (ULS -during extreme conditions)	Gumbel	3.55 [m]	0.4
2.	s_{om}	normal	0.045	0.004
3.	N	normal	3000	200
4.	H_s (SLS - during normal conditions)	Weibull	1.6 [m]	0.2
5.	h_t	normal	6.25 [m]	0.5
6.	G_c	normal	5.5 [m]	0.5
7.	ρ_c	normal	2.2 [ton]	0.1
8.	ρ_r	normal	2.6 [ton]	0.05
9.	N_{od} (Tetrapod layer)	deterministic	0.5	
10.	N_{od} (toe)	deterministic	2	
11.	ρ_w	deterministic	1.025 [ton]	
12.	q_{allow}	deterministic	5×10^{-3} [m^3/s per m]	
13.	H_{allow}	deterministic	0.8 [m]	
14.	γ_f	deterministic	0.38	

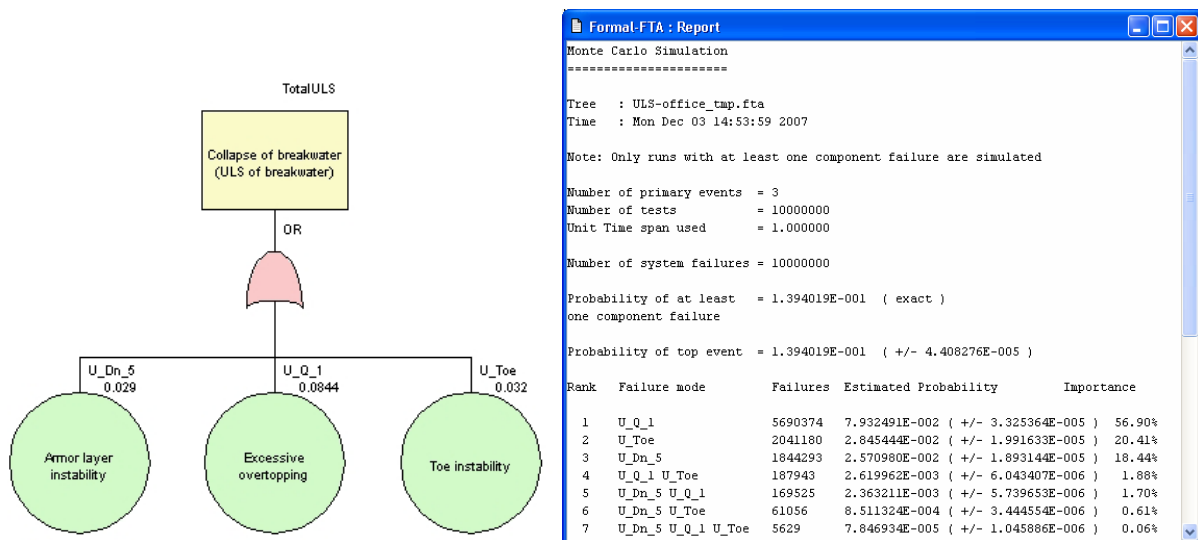**Table 1: Input values for reliability analysis**

Figure 7: Analysis of ULS failure by OpenFTA

5.   ECONOMIC OPTIMISATION

Cost quantification

An economic optimal design is defined as the design for which the total lifetime costs are minimal with some design constraint.

The generalised total cost includes the costs of construction, the expected costs of failure and the costs of maintenance.

The costs of construction can be formulated as follows:

$$I_{cons} = I_0 + I(d) \qquad (16)$$

where I_0 is a initial cost which does not depend on the design variables (e.g. mobilisation cost or cost for design), d is the vector of design variables and I(d) is the construction cost that is a function of the design variables.

The expected costs of failure consist of the economic damage and the costs of repair. The expected costs of failure are calculated for the total lifetime of the breakwater, including the costs of repair and

the economic losses caused by the suspension of harbour operations. In case of a collapse of the breakwater, the failure costs per one event are multiplied by the probability of ULS failure. In case of an intranquillity, the cost per year of interrupted operation is multiplied by the probability of SLS failure per year.

The following formula is applied to calculate the expected costs of failure:

$$I_{failure} = \sum_{i=1}^{N} \frac{C_{ULS} P_{f,ULS}(d)}{(1+r)^i} + \sum_{i=1}^{M} \frac{C_{SLS} P_{f,SLS}(d)}{(1+r)^i} \qquad (17)$$

in which C_{ULS} and C_{SLS} are the damage costs in case of ULS failure and SLS failure, $P_{f,ULS}$ is the probability of ULS failure, $P_{f,SLS}$ is the probability of SLS failure, r is the interest rate, M and N are the reference time in which ULS and SLS failures can occur.

Normally, the reference times M and N are the same as the breakwater is exposed to both ULS and SLS hazards in the same period of time. In case the reference times of ULS and SLS are different, the costs must be separated as in (17). As mentioned in the previous part, the SLS hazards of this breakwater exist only in the first ten years of its lifetime. Therefore M and N are different and should be distinguished.

The costs of maintenance can be calculated by the following equation:

$$I_{main} = \sum_{i=1}^{N} \frac{C_{main}}{(1+r)^i} \qquad (18)$$

where C_{main} is the maintenance cost per year.

C_{main} depends on the maintenance strategy, which in turn depends very much on the authorities, the port's budget distribution and other aspects. In this paper the maintenance cost is neglected.

The total cost now becomes:

$$I(D_n, Z_d) = I_0 + I(d) + \sum_{i=1}^{N} \frac{C_{ULS} P_{f,ULS}(d)}{(1+r)^i} + \sum_{i=1}^{M} \frac{C_{SLS} P_{f,SLS}(d)}{(1+r)^i} \qquad (19)$$

Optimisation procedure

During a design process, a number of alternative structural concepts can be generated. Within one structural concept, a number of alternative structural geometries may be possible. A fully optimisation of structure design is likely to be a trial-and-error process with many iteration circle and a large number of boundary conditions.

To decide whether a geometry is applicable in one structural concept, an acceptable value of the failure probability should be prescribed.

The set of acceptable geometries is defined as:

$$D = \{z \mid P_f(d) \le P_{f,\max}\} \qquad (20)$$

where d is a vector of design variables and $P_{f,max}$ is the maximally acceptable failure probability of the structures.

Eq. (20) shows an infinite number of acceptable geometries of the structural alternative. Even when applying some additional geometric requirements, the combination of all constraints will still leave a large number of acceptable geometries. For a final choice, it is necessary to analyse the cost of every alternative geometry within a solution space. The combination of the investment with a reliability requirement leads to the cheapest design that just suffices the reliability requirement.

An economic optimisation process of a breakwater structure in this way can be mathematically written as:

$$\min_{z} I_i(d)$$
$$s.t.\, P_{f,i}(d,x) \le P_{f,\max}$$

(21)

where i is the counter of the component of the breakwater structure, I is the cost of the component, P_f is the failure probability, x is the vector of random input variables, z is the vector of design parameters and $P_{f;\max}$ is the maximum failure probability.

In general, not only variables describing the geometry of the structure are design variables. Alternative choices may also be available for the materials used in construction. The constraint of P_f can be described under the form of some input parameters' constraint. For one structural concept, the costs and failure probability can be expected to be described by continuous differential functions of design variables.

In the procedure for a full probabilistic optimisation for a breakwater structure, the scheme of Voortman (1999) will be based on.
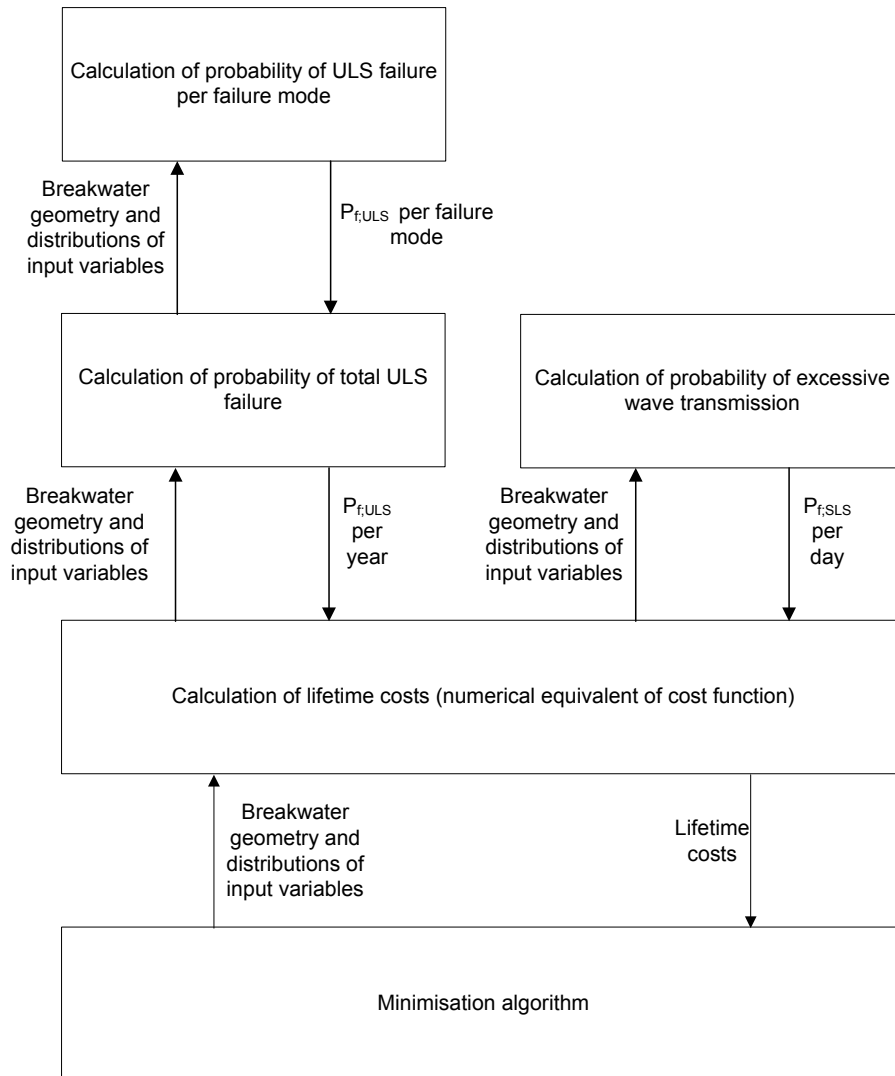

Figure 8: Structure of optimisation process (Voortman et al, 1999)

In this scheme, procedures for the calculation of ULS and SLS failure have been considered in previous parts of this paper and will be used as inputs for the cost functions and the minimisation block. The minimisation algorithm generally uses derivatives of the cost function. However, such an

algorithm requires the cost function to be continuously differentiable. In this paper, a numerical optimisation is carried out by varying the design variables with the proposed deterministic values as the starting point.

Numerical result

Optimisation is carried out for the weight of Tetrapod and the crest height of the breakwater. Assuming that the height of the wave wall is fixed, the crest height in the low profile determines the wave transmission as well as the wave overtopping during extreme conditions. The weight of Tetrapod decides the hydraulic stability of the primary armour layer. The design constraints are the allowable values of wave height in port basin, the overtopping discharge and the stability numbers of armour layer and toe.

The minimisation problem is rewritten as follows:

$$\min I\left(D_n, Z_d\right) = I_0 + I\left(D_n, Z_d\right) + \sum_{i=1}^{N} \frac{C_{ULS} P_{f,ULS}\left(D_n, Z_d\right)}{\left(1+r\right)^i} + \sum_{i=1}^{M} \frac{C_{SLS} P_{f,SLS}\left(D_n, Z_d\right)}{\left(1+r\right)^i} \qquad (22)$$

$$s.t \left(H_{basin}, q, N_{od}\right) \le \left(H_{allow}, q_{allow}, \left[N_{od}\right]\right)$$

Estimated costs of failure are summarised in Table 2. It is assumed that when a total collapse occurs, the port has to cut its capacity by a half and this situation would last for one year.

Item	Description	US $
Cost of one day suspension of port operation		
Cost of shipping operation	US $5000 vessel/day; average 1 vessel/day + 2 day waiting extra	15,000
Loss of direct income	throughput 30 mil ton/year; port dues US $5 / ton	410,000
Loss of indirect income	bad reputation, shipping lines, other parties	320,000
		745,000
Indirect economic damage	multiplier 1,5	
Total		**1.12 x 10⁶**
Cost of collapse of breakwaters due to an extreme event		
Structural damage		
Damage to breakwater	20% of construction cost	20% I_{cons}
Damage to other structures in port	quay, harbour lights, store house	500,000
Mobilisation of contractor	estimate	100,000
Subtotal		600,000 + 20% I_{cons}
Economic damage		
Alternative transportation of goods	throughput 30 mil ton/year, cut off 50% in 1 year, extra cost US $ 5 / ton:	75 x 10⁶
Cost of shipping operation	US $5000 vessel/day; average 1 vessel/day	912,500
Loss of direct income	throughput 30 mil ton/year; port dues US $ 5 / ton	75 x 10⁶
Loss of indirect income	bad reputation, shipping lines, other parties	60 x 10⁶
Indirect economic damage	multiplier 1,5	
Subtotal		320 x 10⁶
Total structural and economic damage		**320.6 x 10⁶ + 20% I_{cons}**

Table 2: Estimated cost of failure

Results of the optimisation process with regards to the block weight (represented by D_n) and the crest height Z_d are given in Figures 9, 10 and 11.
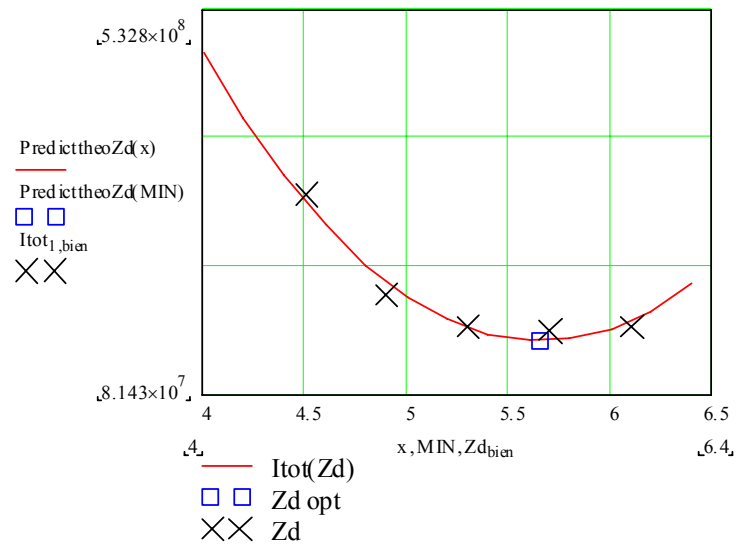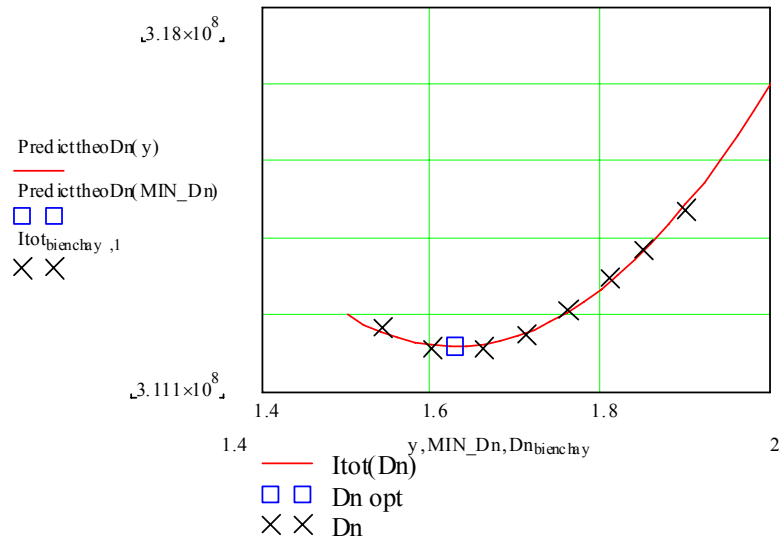

Figure 9: The total costs as a function of crest height Z_d

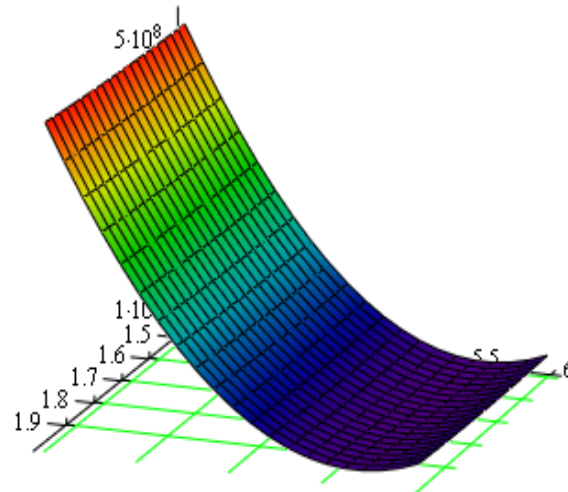**Figure 10: Total costs as a function of block nominal diameter D_n**

Figure 11: Total costs as a function of Z_d and D_n

The exact solution is found at Z_d = 5.657m and D_n = 1.65m which is equivalent to a Tetrapod weighting 9.8 ton. Considering a rounding up gives an optimal geometry with a Tetrapod of 10 ton and a crest height of 5.6m +CD.

The result shows that the optimised geometry has a smaller Tetrapod block but a higher crest than the proposed deterministic geometry. This result is quite reasonable. While a smaller Tetrapod armour layer can withstand severe typhoons, the higher crest considerably reduces the wave overtopping and transmission. The port downtime decreases and results in a better economic efficiency in a long lifetime. Deterministic design made by CMB calculated only the bearing capacity of the structure, not the consequences in the structure's lifetime. Therefore in the deterministic design an economic optimum could not be achieved.

6.   CONCLUSIONS

Application of economic optimisation with probabilistic analysis to the conceptual design of a rubble mound breakwater has been demonstrated in this paper and proves to be a rational way to achieve an optimal geometry alternative. In this study, the Tetrapod block weight and the crest level were optimised by balancing the investment costs with the reduction of the risk of failure. Other design variables can also be optimised following the same approach. The conceptual optimal values can be used as good starting points in the detail design procedure.

Regarding the implementation of the optimisation process, a minimisation algorithm providing reliability estimates in a relative short time with a suitable optimisation software should be used. Minimisation algorithms that use derivatives of the cost function are only applicable if the cost function is continuously differentiable. When a small number of design variables is optimised, a simple optimisation software can easily handle. However, in case a large number of design variables to be analysed, the optimisation software should be stable and easy-to-trace, especially when a disconvergence occurs.

Probabilities of failure in this study are all considered as constant in time. During a detail design procedure, time-dependent processes such as the sea level rise, the deterioration of materials and structure's components and the growth of the cargo throughput should be analysed and integrated into the reliability calculations.

7.   ACKNOWLEDGMENTS

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