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Probabilistic Methods in Dutch Offshore Geotechnics

Méthodes probabilistes en géotechnique offshore aux Pays-Bas

Geotechnische Anwendung der Zuverlässigkeitstheorie bei Niederländischen Meeresbauten

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SUMMARY

This paper presents a review of the main geotechnical applications of probabilistic methods with respect to the quality control of the design, construction and maintenance of the storm surge barrier in the south western part of the Netherlands. Attention is paid respectively to the overall fault-tree of the barrier, reliability analysis of the stability of the pier foundation including the economic optimization, risk analysis of flow slides due to scour at the edges of the seabottom protection.

RÉSUMÉ

L'article traite d'applications des méthodes probabilistes pour le contrôle de la qualité du projet, de la construction et de l'entretien de la fondation du barrage anti-tempête dans le sud-ouest des Pays.-Bas. Il traite de l'arbre d'erreur pour le barrage, du calcul de la sécurité de la fondation des piliers, prenant en compte l'optimisation économique et l'analyse du risque d'érosion aux bords du tapis de protection posé sur le sol marin.

ZUSAMMENFASSUNG

Der Aufsatz stellt die wichtigsten Anwendungen der Zuverlässigkeitstheorie auf die Qualitätskontrolle des Entwurfs, der Erstellung und der Unterhaltung des Fundaments eines Sturmflutwehrs in den Südwest-Niederlanden dar. Die folgenden Themen werden behandelt: der Fehlerbaum für das Sturmflutwehr als ganzes, die Stabilitätsanalyse des Pfeilerfundamentes mit Einschluss des ökonomisch optimalen Entwurfs sowie die Risikoanalyse des durch Erosion an den Rändern der Senkmatten verursachten Setzungsfliessens.

1. INTRODUCTION

Since half of the Netherlands is situated below mean sea level the need for an adequate safety system of the Dutch sea defences is obvious. The storm surge barrier in the south-western part of the Netherlands is one of the last large sea defence projects under construction. The barrier in the mouth of the Oosterschelde basin crosses three tidal flow channels (Roompot, Schaar and Hammen) over a total length of 3000 m. with a maximum water depth of 35 m. (Fig. 1).
The storm surge barrier consists of 65 large concrete piers with a submerged weight of 12.000 tons. The piers, prefabricated in a dry dock, were placed by means of a special lift-vessel as gravity offshore structure on the sand bottom, which is covered with filter mattresses. In between the piers steel gates can be lowered during severe storm floods from the North Sea. Under normal conditions the gates are lifted and allow the sea water to flow into the Oosterschelde basin with the tidal movements, in order to maintain the natural state and ecosystem of the estuary. To protect the subsoil and the filter layers against scouring a rubblemound sill was placed around the piers and up to a distance of 600 m. from the piers a bottom protection was applied (Fig. 2).
The barrier as a system has to meet very high safety requirements according to the governmental codes of acceptable flood excess frequencies. It must be emphasized that the quality of the barrier proceeds from the quality of the different structural elements, such as the concrete piers, the steel gates, the bottom protection and the granular foundation. To provide a consistent quality control of the overall system, probabilistic methods were applied with which the different structural elements can be evaluated at a comparable level of reliability.

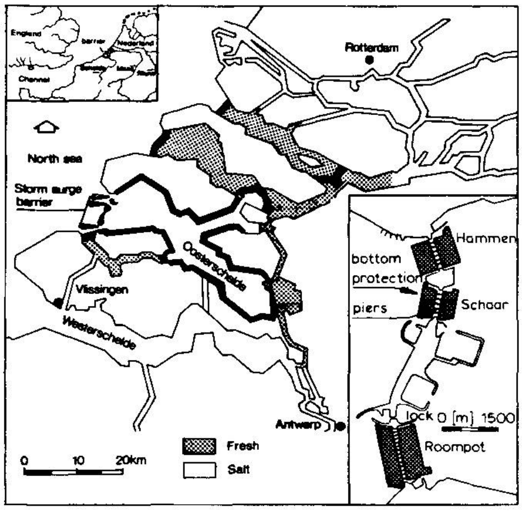


fig. 1 Location of the storm surge barrier.

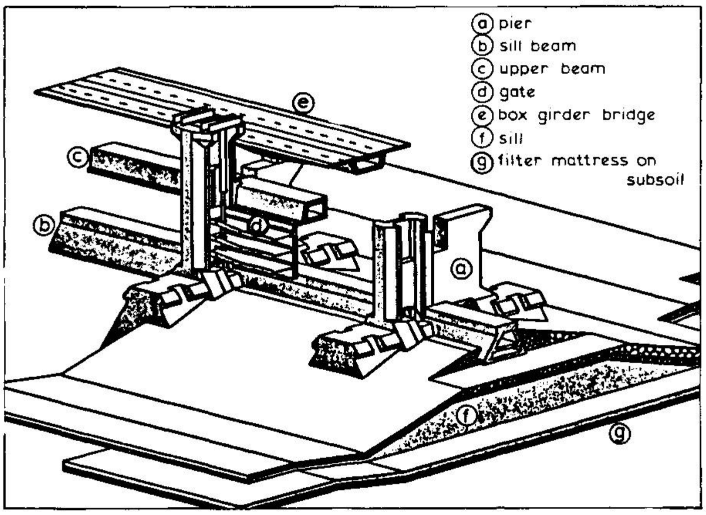
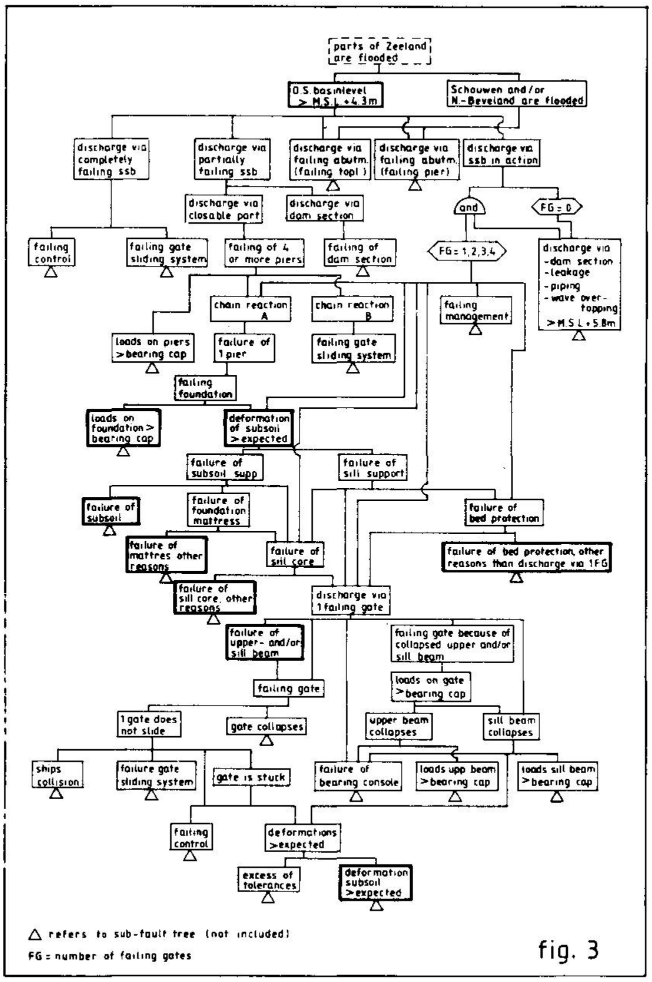


fig. 2 The storm surge barrier.

2. PROBABILISTIC APPROACH STORM SURGE BARRIER

Three steps can be distinguished in the application of probabilistic methods in the design and construction of the storm surge barrier (s.s.b.) [1].
The first step is based on the concept of the design excess frequency of storm surge water levels, according to the design codes for the overtopping height of the Dutch dikes, in which the water level appeared to be the main loading parameter. The s.s.b. however has to withstand, next to storm surge levels, other loading conditions such as water levels inside the basin (headloss in two directions), waves, currents (also in case of malfunctioning of the lowering system of the gates). Since the s.s.b. has to be designed for those loading combinations which potentially will yield most dangers to the structural reliability, the concept of design excess frequency for water levels has to be extended to each potential loading condition. This means that dependent on the structural element to be considered, the probability of all relevant loading parameters have to be taken into account.

The second step in the introduction of probabilistic methods is the performance of reliability analyses. In this type of analysis the uncertainties of both the loading parameters and the structural resistance parameters are accounted for. Since complete safety is unattainable the need for the assessment of an acceptable level of unsafety, expressed in terms of probability of failure, became apparent. For the s.s.b. as a whole a target probability of failure of 10^{-7} per annum (p.a.) is considered acceptable. This safety requirement is based upon the present probability of loss of live due to accidents in the Netherlands (order 10^{-4} p.a. per individu) and the number of fatalities after a possible storm flood catastrophe (order 1000).
To verify if the s.s.b. as a system satisfies the overall safety requirement, each of the components of the structure that contributes to the loss of stability of the s.s.b. has been evaluated. Fault tree - event tree analyses [1] proved to be a useful instrument to systematically assess the contributions of each component to the overall safety of the structure (Fig. 3).



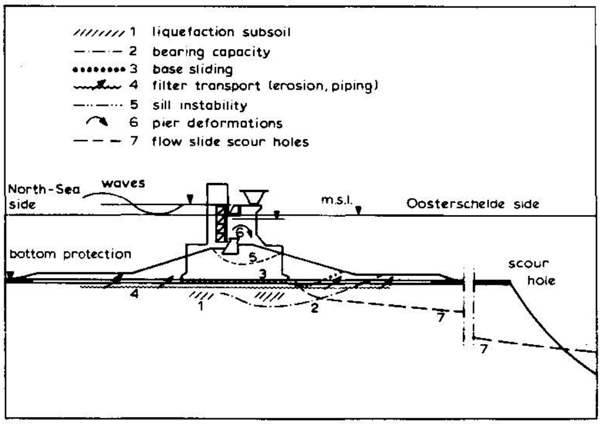


fig. 4

fig. 4 Presentation of the main geotechnical limit states.

fig. 3 Main fault tree of the s.s.b.

The conditions at which the structural components start to fail are referred to as limit states. For the assessment of the reliability of each of these limit-states different probabilistic technics are available. In general it is endeavoured to model the failure mechanisms associated with the limit states of the components by way of amathematical relationship in which the basic strenght (m_j), the loading variables (s_i) and the geometric variables (g_k) are corporated. These relationships can be based on theoretical principles, model tests or on empirical data. Any limit state criterion may be expressed in terms of the basis variables by means of the reliability function Z according to:

$$Z = f\,(m_j, s_i, g_k) = o \qquad (1)$$

Current methods of probabilistic analyses of safety of structures belong to one of the following three categories, referred to as levels I to III [2].

Level III: concerns analysis of safety based on the use of the exact probability density functions of each of the variables involved in the reliability function.

Level II: concerns a number of approximation methods in which the reliability function is linearized at a selected point (or points).

Level I: includes semi-probabilistic methods where a sufficiently large distance between the strength - and the loading-function is created by the use of partial safety factors with respect to the characteristic value of the basic variables.

As third and last step in the probabilistic approach of the quality control, risk analyses for some structural components and construction operations on the site are performed. In this type of analysis also the effect of attaining the limit states, as far as it concerns economic damage, is evaluated. For some design or construction alternatives economic optimum solutions were derived, under the restriction that the overall target safety of the s.s.b. must be maintained.

3. GEOTECHNICAL LIMIT STATES

The main geotechnical limit states, indicated in the fault tree by thick lines, are depicted in figure 4.
The majority of the limit states concerns loadings under extreme storm conditions, that is when the s.s.b. is closed. However some limit states are critical even under daily tidal conditions with lifted gates. An example of this last type is the flow slide mechanism (liquefaction) due to gradually steepening of the slopes of the scour holes at the edges of the bottom protection.
For the evaluation of the limit states the conventionally applied geotechnical stability and deformation models are not always considered adequat, due to the complexity of the s.s.b., the lack of experience with similar offshore structures and the desired high level of structural quality.
In addition a great number of large scale model tests and finit element calculations were performed, all supported by an extensive geotechnical site investigation programme. The next paragraphs present some applications of probabilistic methods with respect to the quality control of a number of ultimate and serviceability limit states for the foundation design of the barrier.

4. PIER STABILITY AND OPTIMIZATION SILL GEOMETRY

During severe storms, when the gates will be lowered, the piers have to withstand high loadings due to the headloss and the wave action on the barrier. The resistance R of the piers with respect to geotechnical instability consists of two parts: first the passive earth pressure of the sill material at the back-wall of the pier R_p and second the sliding resistance at the base of the pier R_b (Fig.5). Since the horizontal load and associating moments are most uncertain, the stability is evaluated in terms of safety with respect to the total horizontal load (H). So the reliability function can be expressed as:

$$Z = m.\ (R_b + R_p) - H \tag{2}$$

where m is a factor to account for the uncertainty of the calculation model.
The passive resistance R_p was determined using the Kötter-equations to estimate the coëfficiënt of passive earth pressure K_p.
Two modes of failure, concerning the modelling of the base sliding resistance R_b were evaluated. The first mode, referred to as the bearing capacity, regards curve-linear sliding plane passing through the subsoil. This failure mode is modelled by formulae for the bearing capacity of shallow foundations given by J. Brinch Hansen [3]. The second mode of failure, assuming a horizontal sliding plane between base and foundation, is modelled by a simple friction law:

$$R_b = V * \tan \delta_b \tag{3}$$

where V is the total pier weight and δ_b the base friction angle.

For the bearing capacity mode of failure the vertical component of the passive pressure is ignored. No generation of excess pore pressure due to the cyclic loading in the foundation is expected, since the subsoil was compacted adequatly.

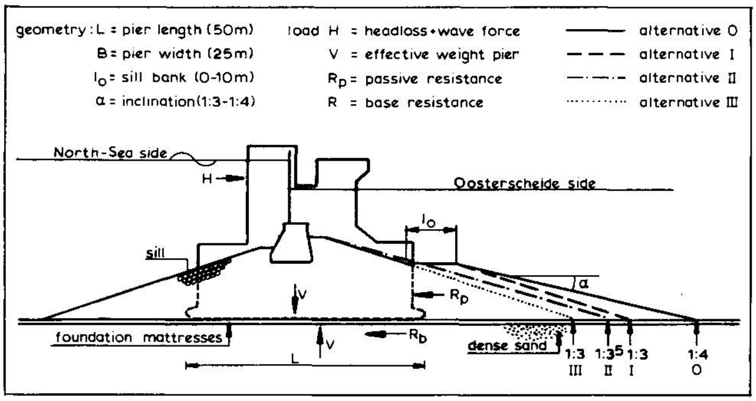


fig. 5 Design alternatives of sill geometry.

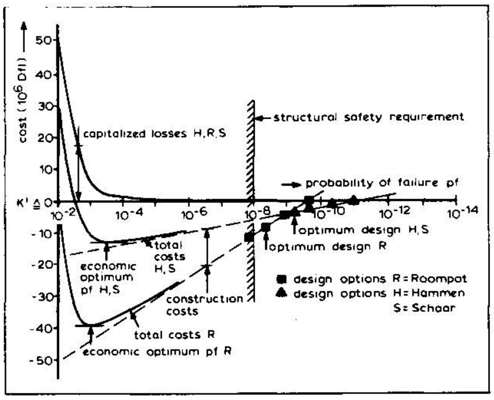


fig. 6 Determination economic optimum sill design.

First the safety requirement was assessed by a level I approach resulting in a minimum overal safety factor FS min ($FS = (R_b + R_p$ design/H design).
Next it was verified by level II analysis (advanced first order, second moment) if the level I safety requirement satisfies the target probability of failure of the foundation according to the fault-tree (p_f target = 10^{-8} p.a.).
From both level I and level II reliability analyses it turned out that the base sliding mode of failure was most critical [4]. Calculation results for different design alternatives show an almost perfect loglinear relationship between the actual safety factor FS and the probability of failure p_f:

$$FS = a * \log p_f + b \tag{4}$$

where a and b are constants. The minimum safety factor (FSmin = 1.5) appeared to correspond very well with the target probability of failure 10^{-8} p.a.

In the final phase of the design it was considered if the original size of the sill (o-alternative) could be reduced in order to save on construction costs. Reduction of the sill size however implies a reduction of the pier stability since both the pier weight V and the passive resistance R_b will decrease.
Cost calculations for three design alternatives (I, II and III, Fig.5) show the following relationship between the total initial construction costs of the sill C and the associated probability of failure p_f.

$$C = C_o + c * \log p_f \tag{5}$$

where C_o indicates the constant part of the initial costs, not influenced by the variation of the sill size, and c denotes a constant.
On the other hand also the risk associated with the increased probability of failure due to the sill reduction can be expressed in terms of costs. To this purpose the capitalized cost M of annual reservation (=fictive insurance premium) to avert the cost of repair due to unexpected failure during the service life of the structure (± 200 years), has been determined. The amount of the premium is given by the product of the probability of damage (=assumed to equal the probability of failure p.a.) and the associated economical loss if damage occurs:

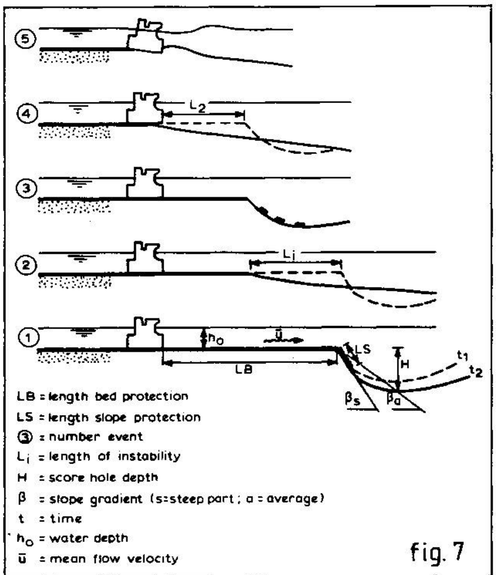
$$M = \sum_{n=1}^{200} \frac{p_f * S}{(1+i)^n} \approx \frac{p_f * S}{i} \tag{6}$$

where i is the rate of interest, corrected for inflation.
Minimization of the total costs R (=sum of the construction costs C and the capatalized damage cost M) produces the economic optimum sill design (ie. the optimum probability of failure). Fig. 6 shows the procedure for the four design options of respectively the sills in the Roompot channel and the Schaar/Hammen channels.

From the risk analysis [5] it turned out that the economic optimum probability of failure is higher than the target probability of failure (=p_f target). This means that p_f target, based on an acceptable level of probability of loss of live, is decisive. On the basis of the risk analysis it was decided to select design variant II for the Roompot and variant III for both the Schaar and the Hammen.

5. RISK ANALYSIS FLOW SLIDES DUE TO SCOUR

The s.s.b. is an obstacle in the tidal flow pattern. In consequence of this scour holes, up to a depth of 50 m below the sea bottom, will develop at the edges at both sides of the bottom protection. If the slopes of the scour holes get too steep they can become instable (Fig. 7). Depending on among other things the relative density of the sand either shearing of the scour slopes (to be modelled by a circular slip analysis) or flow slides may occur.



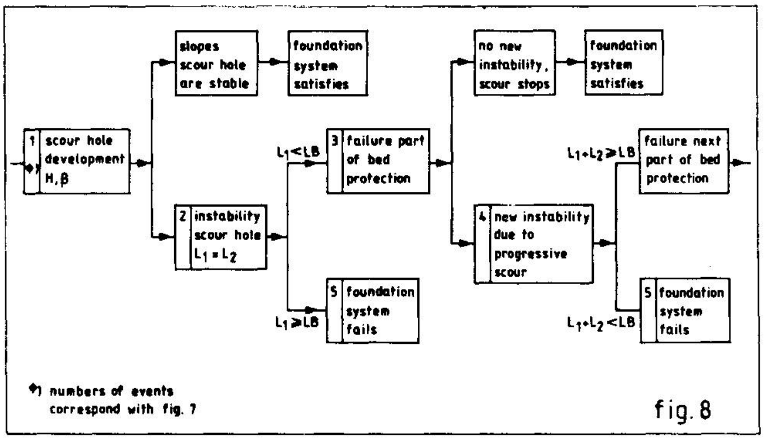


fig. 8 Simple event-tree of scour hole instabilities.

fig. 7 Progressive failure due to scouring.

A flow slide (or subsidence flow) is defined as a geotechnical instability where a saturated sand mass undergoes very large deformations ("flows") as a consequence of the development of excess pore water pressures and a simultaneous reduction of the shear strength, induced by the tendency of the sand to decrease in volume. Flow slides in the loosely packed Oosterschelde holocene sandlayers are of particular importance because they are associated with very flat slopes after failure. Risk analyses for different construction options were performed to decide which measures should be taken to improve the slope stability up to the desired level of safety. Each option provides information on the selected length of bed protection, the hydraulic conditions (water levels, currents), soil conditions (density), the construction schedule (time) and the maintenance program during the scour process (both during and after the construction period).
The risk analysis for evaluating the limit states in case of instabilities occuring at the scour hole, was performed in five steps [6]:

5.1. Prediction of scour hole development:

The scour hole is characterized by the maximum scour hole depth H and the slope gradient β (Fig. 7). From many hydraulic scale model tests, checked by several prototype tests, it was concluded that the growth of the scour depth can be described by the empirical formula [7]:

$$H_{max}(t) = (\alpha\bar{u} - u_{cr})^{1.7} * h_0 * 0.1 * (\rho_s/\rho_w - 1)^{-1.7} \qquad (7)$$

where ρ denotes the density of the sand particles s and the water w, u_{cr} the critical transport velocity of the sand and α the dimensionless hydraulic scour factor (determined from hydraulic model tests); the other parameters are explained in fig. 7.

Due to the uncertainty of the different basic variables (especially the scour factor α) a significant coefficient of variation (order 20%) with respect to the prediction of the scour hole depth has to be counted with.
The slope gradient β can be described in a similar way as H; however the uncertainties are even greater. As a conservative assumption (that is with relatively steep slopes) a standard shape of the scour hole, which is only a function of H, is established. Starting from this principle the scour hole development for each section can be described adequatly by the scour depth as a function of time.

5.2. Assessment scour hole stability criteria:

The stability criteria for both the shearing mechanism and the flow slide are based upon empirical data, obtained from extensive observations of the slopes of the coast in the south western part of Holland during the last century. More than 1100 instabilities nearshore are reported, from which over 200 have been analysed in more detail. The profile measurements show a relation between the number of observed failures and the slope gradient (both the average and the steepest part) before failure occurred. Dependent on the type of soil (loosely packed holocene sand or dense pleistocene sands) the observed failures are divided into flow slides and shearing (sliding planes). Although a justified theoretical modelling of the flow slides, that can support the observed data is not available yet, the empirical data were used as black box prediction model.
To account for the length effect of the edges of the bed protection, the total length of 6 km. has been divided into statistically independent sections of 100 m. width. This somewhat conservative assumption meets the observations, which showed that the width of the zone affected by a flow slide or a shearing generally is in the order of 50 to 200 m.

5.3. Assessment of the damage length:

Given the event that an instability (shearing or a flow slide) occurs at the scour slopes, it is important to know what the damage consequences are. These consequences are expressed in the length (L) at which the adjacent bed protection looses its sand protection function.
The damage length L of the previously mentioned observed failures show a very great variation. For shearing this length varies from 0.5 to 2 H (H = max. depth of the channel or the scour hole) and for the flow slides from 0.7 to 8 H. The observed variation however could be explained by probabilistic back-calculations (first order-second moment) assuming that the volume of eroded sand equals the volume of sedimentated sand after failure. This hypothesis leads to a simple expression of L as a function of the stochastic variables H, β and some empirical geometry-factors.

5.4. Prediction probability of failure of the foundation:

The probability of failure of the pier foundation can be calculated straight forward from combination of steps 5.1, 5.2 and 5.3 according to the scheme of the simple event tree in fig. 8. The foundation fails (=definition) if the total damage length L after failure of the scour hole slopes exceeds the actual length of the bottom protection LB. It must be emphasized that also the damage effect of several succeeding smaller slope instabilities was taken into account (Fig. 7).
In practice however the actual probability of failure will be influenced by additional measures based on observed data during the scour process. If for example damage of the bed protection is detected, repairs will be carried out as soon as possible. In fact the quality of the inspection and maintenance system (o.a. frequency of monitoring, mobilization time for repair equipment) directly influences the probability of failure. In the risk analysis assumptions are made on this type of fuzzy information, directly translated in reduction of transmission probabilities.

5.5. Verification design and control measures:

After determination of the probabilities of the relevant events it has to be checked if the predicted contributions of the scour hole instabilities to failure of the s.s.b. system correspond with the target probability of safety according to the overall fault tree of the system. To judge the effectiveness of slope protection measures the probabilities of flow slide both with and without a protection with dumping layers on the scour slope are predicted.
From the risk analyses [6] it appeared that, apart from the shores of location Roompot East, slope protection measures are not necessary from the point of view of structural safety. For the Roompot East shore extra measures, consisting of a lenghtening of the bed protection in combination with a slope protection, are necessary to obtain the required safety level.

6. CONCLUSIONS AND RECOMMENDATIONS

Probabilistic methods proved to be a useful tool in the practice of the quality control of the design, the construction and the maintenance of the foundation of the complex s.s.b. offshore structure.
Relevant information for decision making can be obtained from probabilistic calculations. This information can vary from a simple check list of points of attention (as obtained from event-fault trees) to the selection of an economic optium design or construction alternative (as obtained from risk analyses).
Probabilistic methods can be used for the evaluation of a wide range of different types of geotechnical limit states dealing with a.o. stability, deformation, soil-structure interaction, flow slides and filter transport mechanisms.
Although the present state of knowledge of probabilistic methods deserves a wider practical implementation in geotechnics, further research is still necessary. This concerns among others the combined probability of failure of a structure associated with a number of simultaneous and partly correlated limit states, including the effect of auto-correlation of the basic system variables.

7. ACKNOWLEGDEMENT

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