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Reliability of Exceptional Structures

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***Abstract*—Codes for the design of structures provide a practical instrument for ensuring that the reliability of normal purpose structures is sufficient and overall cost efficient. Modern design codes, such as the Eurocodes have been calibrated using principles of decision theory and methods of structural reliability. For the purpose of maintaining a simple design format the design codes are, however, only calibrated for the most relevant types of structures, materials and load combinations. The design codes thus have a limited domain of application and therefore whenever exceptional structures are considered it is necessary to perform reliability assessments specifically for the structure of consideration. The present paper first introduces various categories of exceptional structures and thereafter describes the basic principles for the assessment of the reliability of such structures. Finally an example considering some of the practical aspects of reliability assessment of exceptional structures are given.**

***Index Terms*—Reliability, exceptional, structures, design, assessment, deterioration, inspection, maintenance, decommissioning, acceptance criteria.**

I. Introduction

During the past century considerable effort has been devoted to the development of a rational basis for the design of structures, resulting in a number of modern design codes[1,2,3,4]. The modern codes aim to ensure the economical design, construction and operation of structures in compliance with assumed operational conditions and given requirements for the safety of personnel and the environment. The development of the modern design codes has been based on the principles of economical decision analysis and modern reliability methods (see e.g. JCSS[5] and ISO[6]). For the verification of the structural reliability in regard to the relevant failure modes the modern design codes provide a set of so-called design equations relating the design resistance of the structure for the individual failure modes and the corresponding design load-effects. Due to the fact that loads and resistances are subject to uncertainties design values for resistances and load effects are introduced in order to ensure an adequate level of reliability. Design values for resistances are introduced as a characteristic value of the resistance divided by a partial safety factor (larger than 1) and design values for load effects are introduced as characteristic values multiplied by a partial safety factor (larger than 1). Furthermore in order to take into account the effect of simultaneously occurring variable load effects so-called load combination factors (smaller than 1) are multiplied on one or more of the variable load effect.

For the purpose of ensuring a practically applicable design basis the design codes (design equations, characteristic values, partial safety factors and load combination factors) have been calibrated for normal structures, i.e. such that structures of usual dimensions and designs, build of well-known materials and constructed and maintained using established procedures achieve an adequate and homogeneous level of reliability. This, however, implies that structures, which are not normal in the above-mentioned sense fall beyond the application area of the design codes. These types of structures may be seen as being exceptional structures. For such structures the design verifications and for that matter any "fit for purpose" assessment must take basis in reliability assessments for the specific structure of consideration.

In the following first a categorization of different types of exceptional structures is introduced and discussed. Thereafter the basic principles for the reliability verification of such structures are outlined and finally an example is given considering the reliability verification of an exceptional structure.

II. DIFFERENT CATEGORIES OF EXCEPTIONAL STRUCTURES

Traditionally exceptional structures are usually associated with structures fulfilling new purposes or of extreme dimensions or innovative designs see e.g. Figures 1 - 2.

Figure 1 Examples of structures of extreme dimensions. To the left the Great Belt Link under construction and to the right a principal sketch of the Troll offshore platform.

However in accordance with the definition outlined in the foregoing exceptional structures include all structures falling beyond the application area of the design and assessment codes. When categorizing such structures it is useful to differentiate between new structures, i.e. structures to be designed and existing structures i.e. structures, which for some reason are subject to a reliability assessment.

For new structures exceptional structures include structures

- fulfilling new purposes or of exceptional dimensions and innovative designs
- build using new materials or innovative combinations of materials
- constructed and maintained according to new methods and strategies
- subjected to unusual loads and load combinations
- subjected to unusual environmental exposures
- associated with extreme consequences in case of failure
- being especially difficult to decommission

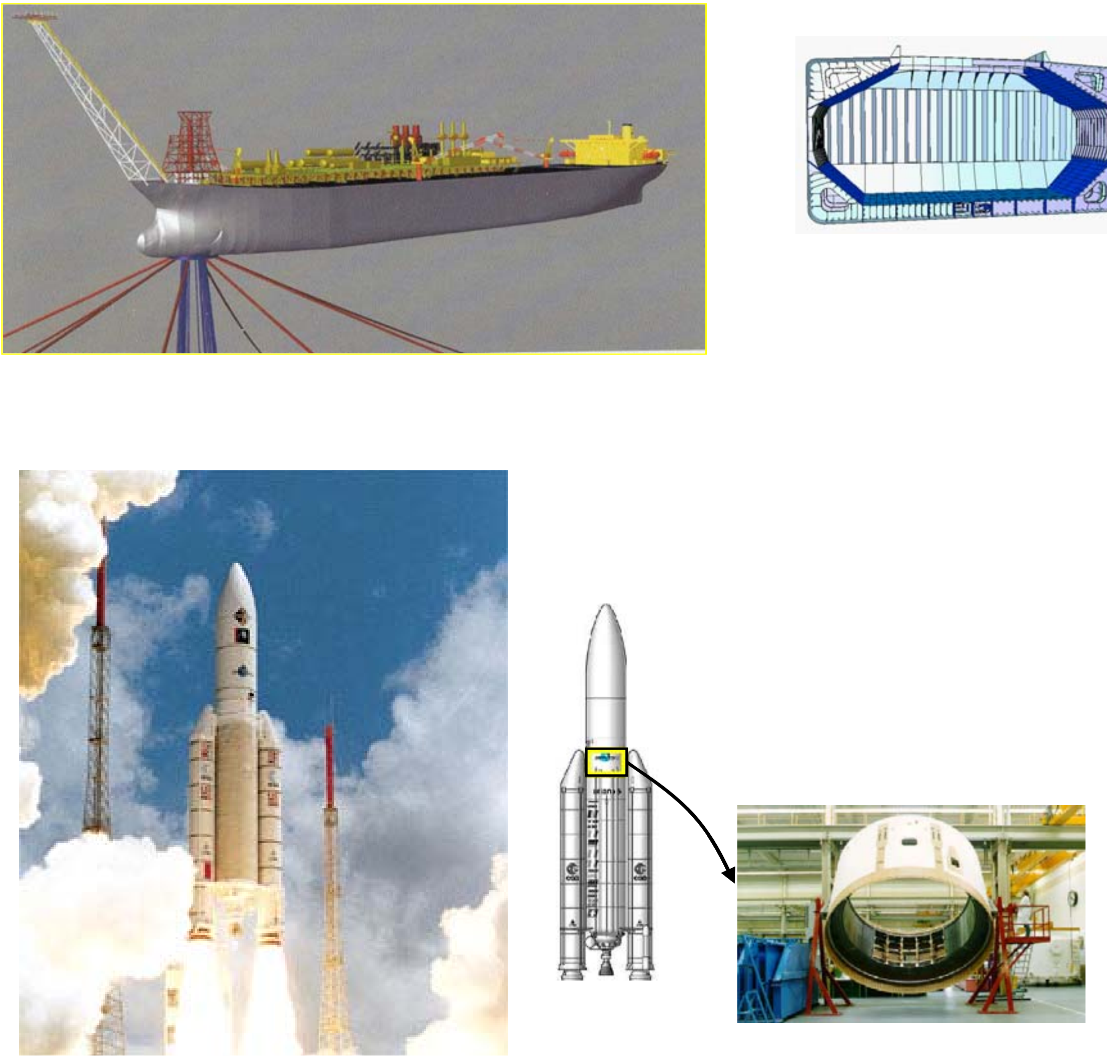

Figure 2 Examples of structures for new purposes. Upper figures illustrating a floating production and storage facility and lower figures illustrating the Ariane 5 aeronautical structure.

For existing structures exceptional structures include structures

- having been designed according to out dated standards
- exhibiting unforeseen degrees of deterioration
- having been subjected to accidental damages
- having been subject to extreme loads or environmental exposures
- subject to changed operational conditions
- unexpectedly to be decommissioned

In principle structure specific reliability assessments must be made for all the above-mentioned structures. The engineering profession has to some extent recognized this fact but only within the last decade the problem has been approached in a more systematic and consistent way using the principles of decision analysis and structural reliability theory. In the following the basic principles for such reliability assessments will be outlined.

III. Principles of Structural Reliability Analysis

The overall aim of structural reliability analysis is to quantify the reliability of structures under consideration of the uncertainties associated with the resistances and loads. The structural performance is assessed by means of models based on physical understanding and empirical data. Due to idealizations, inherent physical uncertainties and inadequate or insufficient data the models themselves and the parameters entering the models such as material parameters and load characteristics are uncertain. Structural reliability theory takes basis in the probabilistic modeling of these uncertainties and provides methods for the quantification of the probability that the structures do not fulfill the performance criteria.

A. Uncertainty modeling

The uncertainties, which must be considered, are the physical uncertainty, the statistical uncertainty and the model uncertainty. The physical uncertainties are typically uncertainties associated with the loading environment, the geometry of the structure and the material properties. The statistical uncertainties arise due to incomplete statistical information e.g. due to a small number of materials tests. Finally, the model uncertainties must be considered to take into account the uncertainty associated with the idealized mathematical descriptions used to approximate the actual physical behavior of the structure. The probabilistic modeling of uncertainties highly rests on a Bayesian statistical interpretation of uncertainties implying that the uncertainty modeling utilizes and facilitates both the incorporation of statistical evidence about uncertain parameters and subjectively assessed uncertainties. Modern methods of reliability and risk analysis allow for a very general representation of these uncertainties ranging from non-stationary stochastic processes and fields to time-invariant random variables, see e.g. Melchers [7]. In most cases it is sufficient to model the uncertain quantities by random variables with given distribution functions and distribution parameters estimated on basis of statistical and/or subjective information. In the probabilistic model code by JCSS [8] an almost complete set of probabilistic models are given covering most situations encountered in practical engineering problems.

B. Probability of failure

The performance criteria are normally expressed in terms of limit state equations $g(\boldsymbol{x})$ and so-called failure events F

$$F = \{g(\mathbf{x}) \le 0\} \qquad (1)$$

where the components of the vector $\mathbf{x}$ are realizations of the so-called basic random variables $\boldsymbol{X}$ representing all the relevant uncertainties influencing the probability of failure. The basic random variables must be able to represent all types of uncertainties that are included in the analysis.

Having established probabilistic models for the uncertain variables the problem remains to evaluate the probability of failure corresponding to a specified reference period. However, also other non-failure states of the considered component or system may be of interest, such as excessive damage, unavailability, etc. In general any state, which may be associated with consequences in terms of costs, loss of lives and impact to the environment are of interest. In the following, however, for simplicity these states are not differentiated.

Having defined the failure event the probability of failure may be determined by the following integral

$$P_f = \int_{g(\mathbf{x})\leq 0} f_{\mathbf{X}}(\mathbf{x})d\mathbf{x} \tag{2}$$

where $f_{\mathbf{X}}(\mathbf{x})$ is the joint probability density function of the random variables $\boldsymbol{X}$. This integral, illustrated in Figure 3 as a volume integral of the joint density function in the failure domain is, however, non-trivial to solve and numerical approximations are expedient. Various methods for the solution of the integral in Equation (2) have been proposed including numerical integration techniques, Monte Carlo simulation and First and Second Order Reliability Methods (FORM/SORM). Numerical integration techniques very rapidly become inefficient for increasing dimension of the vector $\boldsymbol{X}$ and are in general irrelevant.

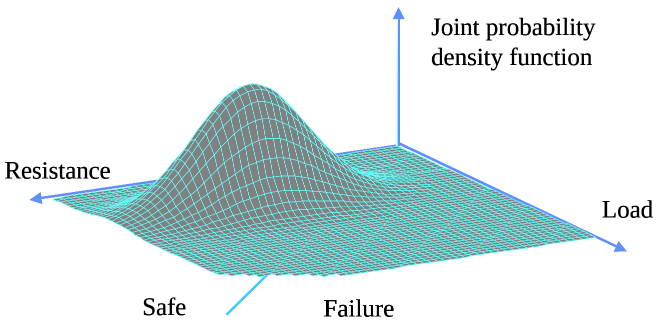

Figure 3 Illustration of the failure probability integration problem in two dimensions.

The first developments of First and Second Order Reliability Methods (FORM/SORM) took place almost 30 years ago with pioneering work performed by Basler [9], Cornell [10] and Hasofer & Lind [11]. Since then these methods together with advanced Monte Carlo simulation techniques have been refined and extended significantly and by now they form the most important methods for reliability evaluations in structural reliability theory. For the most common practical purposes the problem of estimating probabilities may be considered as solved. Several commercial computer codes have been developed for FORM/SORM and simulation analysis and the methods are widely used in practical engineering problems and not least for code calibration purposes, see e.g. STRUREL [12] and Proban [13].

C. Reliability updating

When assessing existing structures a significant difference as compared to the situation where new structures are designed should be noticed. Namely the fact that for existing structures various types of information may be available. The probabilistic assessment of existing structures is treated in detail in JCSS [14]. Examples of information, which is available or can be made available at a given cost, are

- The structure has survived

- Material characteristics from different sources
- Geometry
- Damages and deterioration
- Capacity by proof loading
- Static and dynamic response to controlled loading

In the assessment of existing structures such new information can be taken into account and combined with the *prior* probabilistic models, i.e. the probabilistic models formulated before the new information was available by reliability updating techniques. The result is so-called *posterior* probabilistic models, which may be used as an enhanced basis for the reassessment engineering decision analysis.

Given an inspection result of a quantity which is an outcome of a functional relationship between several basic variables probabilities may be updated by direct updating of the relevant failure probabilities, using the definition of conditional probability

$$P(F|I) = \frac{P\{F \cap I\}}{P\{I\}} \quad (3)$$

F = Failure
I = Inspection result

Inspection or test results relating directly to realizations of random variables may be used in the updating. This is done by assuming the distribution parameters of the distributions used in the probabilistic modeling to be uncertain them selves. New samples or observations of realizations of the random variables are then used to update the probability distribution functions of these distribution parameters.

Assume that a random variable *X* has the probability distribution function $F_X(x,q)$ and density function $f_X(x,q)$ where q are the distribution parameters. Furthermore assume that one or more of the distribution parameters, e.g. the mean value and standard deviation of *X* are uncertain themselves modeled by random variables *Q* with probability density function $f_Q(q)$. Then the probability distribution function for *Q* may be updated on the basis of observations of *X*, i.e. $\hat{x}$. The general scheme for the updating is

$$f''_Q(q \mid x) = \frac{f'_Q(q)\, L(q \mid \hat{x})}{\int f'_Q(q)\, L(q \mid \hat{x})\, dq} \quad (4)$$

where $f_Q(q)$ is the distribution function for the uncertain parameters *Q* and $L(q|\hat{x})$ is the likelihood of the observations or the test results contained in $\hat{x}$. ´´ denotes the *posterior*, ´ the p*rior* probability density functions of *Q*. The likelihood function $L(q|\hat{x})$ may be readily determined by taking the density function of *X* in $\hat{x}$ with the parameters *q*. For discrete distributions the integral is replaced by summation.

The observations $\hat{x}$ may not only be used to update the distribution of the uncertain parameters *Q* but also to update the probability distribution of *X*. The updated probability distribution function for *X* is often called the predictive distribution or the Bayes distribution. The *predictive* distribution may be assessed through

$$f_X^U(x) = \int_{-\infty}^{\infty} f_X(x|q)\, f_Q''(q \mid \hat{x})dq \qquad (5)$$

In Raiffa and Schlaifer [15] and Aitchison and Dunsmore [16] a number of closed form solutions to the *posterior* and the predictive distributions can be found for special types of probability distribution functions known as the natural conjugate distributions.

D. Reliability and partial safety factors

In code based design formats such as the Eurocodes [1], design equations are prescribed for the verification of the capacity of different types of structural components in regard to different modes of failure. The typical format for the verification of a structural component is given as design equations such as

$$G = zR_C / \gamma_m - \left(\gamma_{G_a} G_c + \gamma_Q Q_C\right) = 0 \qquad (6)$$

where

R_C	is the characteristic value for the resistance
z	is a design variable (e.g. the cross sectional area of the steel rod considered previously)
G_C	is a characteristic value for the permanent load
Q_C	is a characteristic value for the variable load
γ_m	is the partial safety factor for the resistance
γ_G	is the partial safety factor for the permanent load
γ_Q	is the partial safety factor for the variable load

In the codes different partial safety factors are specified for different materials and for different types of loads. Furthermore when more than one variable load is acting load combination factors are multiplied on one ore more of the variable load components to take into account the fact that it is unlikely that all variable loads are acting with extreme values at the same time.

The partial safety factors together with the characteristic values are introduced in order to ensure a certain minimum reliability level for the structural components designed according to the code. As different materials have different uncertainties associated with their material parameters the partial safety factors are in general different for the different materials. The principle is illustrated in Figure 4 for the simple r-s case.

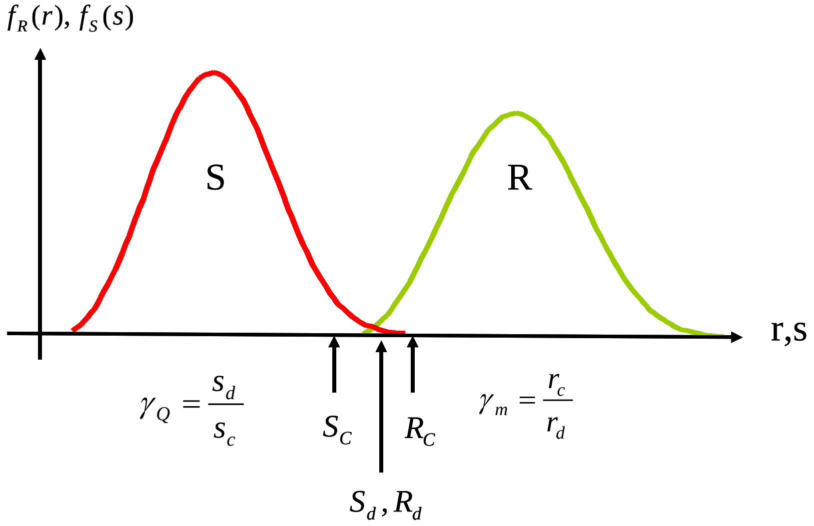

Figure 4 Illustration of the relation between design values, characteristic values and partial safety factors.

In accordance with a given design equation such as e.g. (6) a reliability analysis may be made with a limit state function of the same form as the design equation but where the characteristic values for the resistance and load variables are now replaced by basic random variables, i.e.

$$g = zR - (G + Q) = 0 \quad (7)$$

For given probabilistic models for the basis random variables *R*, *G* and *Q* and with a given requirement to the maximum allowable failure probability it is now possible to determine the value of the design variable *z* which corresponds to this failure probability. Such a design could be interpreted as being an optimal design because it exactly fulfils the given requirements to structural reliability.

Having determined the optimal design *z* we may also calculate the corresponding design point in the original space, i.e. $\mathbf{x}_d$ for the basic random variables. This point may be interpreted as the most likely failure point, i.e. the most likely combination of the outcomes of the basic random variables leading to failure. Now partial safety factors may be derived from the design point for the various resistance variables as

$$\gamma_m = \frac{x_c}{x_d} \quad (8)$$

and for load variables

$$\gamma_Q = \frac{x_d}{x_c} \quad (9)$$

where x_d is the design point for the considered design variable and x_c the corresponding characteristic value.

E. Optimality and acceptance criteria

It is well known, but not always fully appreciated, that the reliability of a structure as estimated on the basis of a given set of probabilistic models for loads and resistances may have limited bearing to the actual reliability of the structure. This is the case when the probabilistic modeling forming the basis of the reliability analysis is highly influenced by subjectivity and then the estimated reliability should be interpreted as being a measure for comparison only. In these cases it is thus not immediately possible to judge whether the estimated reliability is sufficiently high without first establishing a more formalized reference for comparison.

Such a reference may be established by the definition of an optimal or best practice structure. The idea behind the "best practice" reference is that if the structure of consideration has been designed according to the "best practice" then the reliability of the structure is "optimal" according to agreed conventions for the target reliability. Typical values for the corresponding target annual failure probability are in the range of 10^{-6} to 10^{-7} depending on the type of structure and the characteristics of the considered failure mode. Using this approach the target reliability is determined as the reliability of the "best practice" design as assessed with the given probabilistic model.

The determination of the "best practice" design can be performed in different ways. The simplest approach is to use the existing codes of practice for design as a basis for the identification of "best practice" design. Alternatively the "best practice design" may be determined by consultation of a panel of recognized experts.

In case where the probabilistic modeling does not rest on subjective assessments the most rational approach is to establish the optimal design on the basis of the economic decision theory. By considering the expected total benefit $E[B]$ associated with the considered structure

$$E[B] = I \cdot (1 - P_F(C_D)) - C_D - C_F \cdot P_F(C_D) = I - C_D - (I + C_F)P_F(C_D) \quad (10)$$

where *I* is the expected benefit from the structure, C_F is the cost consequence in case of failure, C_D is the cost of some risk reducing measure, e.g. an increase of a dimension, and where the probability of failure is a function of the costs invested in the risk reduction we have that the optimal investment in risk reducing measures may be determined from the following optimality criterion.

$$\frac{\partial E[B]}{\partial C_D} = -1 - (I + C_F) \cdot \frac{\partial P_F(C_D)}{\partial C_D} = 0 \quad (11)$$

from which the cost efficient level of risk reducing measures may be determined. Having determined these we may, by application of (10) assess the feasibility of the considered structure by recognizing that the total expected benefit of the structure shall be larger than zero.

Without going in to the details prevailing the derivations it is interesting to notice that it is possible, based on recent research work by Nathwani and Lind [17] and Rackwitz [18] to establish optimal values for risk reduction costs when also the consequences of loss of human lives are considered by means of the Life Quality Index. The Life Quality Index, *L* is a compound social indicator defined as

$$L = g^w e^{1-w} \quad (12)$$

where *g* is the gross domestic product per year per person, *e* is the life expectancy at birth and *w* is the proportion of life spent in economic activity. In developed countries it may be assumed that w=1/8. g lies in the interval of $US 2600-14000 being average numbers ranging from poor to well developed countries. The life expectancy at birth e being 56 years in poorly developed countries, 67 years in medium developed countries and 73 years in highly developed countries, see e.g. Skjong and Ronold [19]. The LQI implies that a risk reducing measure is feasible if

$$\frac{\Delta e}{e} \geq -\frac{\Delta g}{g}\frac{w}{1-w} \tag{13}$$

which may be obtained from (12) as explained in Nathwani and Lind [17]. From (13) the optimal risk reducing measure for saving the life of a person may be identified by considering the case of equality. Then we obtain

$$\left|\Delta g\right|_{\max} = \frac{g}{e}\frac{1-w}{w}\Delta e = \frac{g}{2}\frac{1-w}{w} \tag{14}$$

which may be interpreted as the optimal acceptable costs per life year saved and where it has been assumed that number of life years saved by saving one individual Δe in average equals $\Delta e = \frac{e}{2}$.

From (14) we may now readily calculate to optimal costs of saving the life of one individual, also called the optimum acceptable implied cost of averting a fatality (*ICAF*) from

$$ICAF = \frac{ge}{4}\frac{1-w}{w} \tag{15}$$

from which it may be found that optimum values of *ICAF* lies in the range of $US 2 – 3 x 10^6.

These costs may be included in (10) when the optimal investments into safety are considered and thus treated within the same framework as any other asset loss. It should be noticed that as a consequence hereof the acceptable failure probability associated with a specific project or structure depends on its specific characteristic, i.e. the monetary consequences in case of failure together with the expected benefits of the activity.

In Tables 1 - 2 target failure probabilities and corresponding target reliability indexes are given for ultimate limit states and serviceability limit states, respectively based on the recommendations of JCSS [8]. Note that the values given correspond to a year reference period and the stochastic models recommended in JCSS [8].

Table 1: Tentative target reliability indices β (and associated target failure probabilities) related to a one-year reference period and ultimate limit states

Relative Cost of Safety Measure	Minor consequences of failure	Moderate consequences of failure	Large consequences of failure
High	**β=2.3($p_F \approx 10^{-2}$)**	**β=3.1($p_F \approx 10^{-3}$)**	**β=3.7($p_F \approx 10^{-4}$)**
Normal	**β=3.1($p_F \approx 10^{-3}$)**	**β=3.7($p_F \approx 10^{-4}$)**	**β=4.3($p_F \approx 10^{-5}$)**
Low	**β=3.7($p_F \approx 10^{-4}$)**	**β=4.3($p_F \approx 10^{-5}$)**	**β=4.7($p_F \approx 10^{-6}$)**

Table 2: Tentative target reliability indices (and associated probabilities) related to a one-year reference period and irreversible serviceability limit states

Relative Cost of Safety Measure	Target Index (irreversible SLS)
High	**β=1.3(p_F≈10^{-1})**
Normal	**β=1.7(p_F≈5 10^{-2})**
Low	**β=2.3(p_F≈10^{-2})**

IV. Reliability Analysis for Decommissioning of Offshore Structures

Exploitation of gas and oil reserves offshore has been ongoing for more than 30 years in the North Sea. Many of the production facilities installed in the early phases have by now or will in the near future reach the end of their production service life. By international conventions it is required that provision is made for the decommissioning of these facilities. Risk and reliability studies are presently conducted for the feasibility assessment of different removal options in connection with the decommissioning of the concrete structures in the North Sea (Faber et al. [20]), see Figure 5.

Figure 5 Illustration of a typical concrete offshore structure to be removed.

Several of the structures were not originally designed for the removal and the reliability of the structures thus need to be verified for this special load situation with due consideration of their condition after almost 30 years in operation. No codes of practice exist for such assessments and such structures are indeed exceptional structures. The structural reliability analyses are performed using FORM/SORM analysis in consistency with the approaches and models proposed in the JCSS probabilistic model code JCSS [8]. The results of the structural reliability analysis are then combined with operational risk analysis for the overall feasibility assessment of the different removal options. For this purpose Bayesian probabilistic nets are used, providing a basis for the systematic assessment of the probability that the operation will fail and the expected total costs due to various possible adverse events during the removal activity. A Bayesian probabilistic net is illustrated in Figure 6 corresponding to the situation where a Condeep platform is being re-floated.

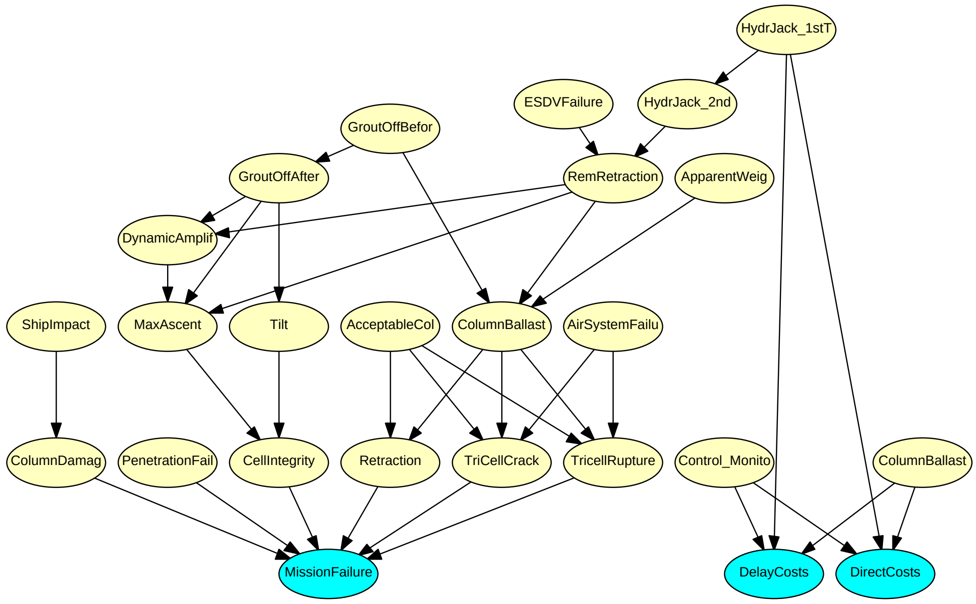

Figure 6 Bayesian probabilistic net for the risk and reliability analysis of a Condeep platform being re-floated.

The result of the risk and reliability analysis using Bayesian probabilistic nets for all phases of a removal option may be presented in diagrams as illustrated in Figure 7. Results as those provided in Figure 7 are very useful in the feasibility assessment of the individual removal options and not least for the comparison of the risks between different removal options.

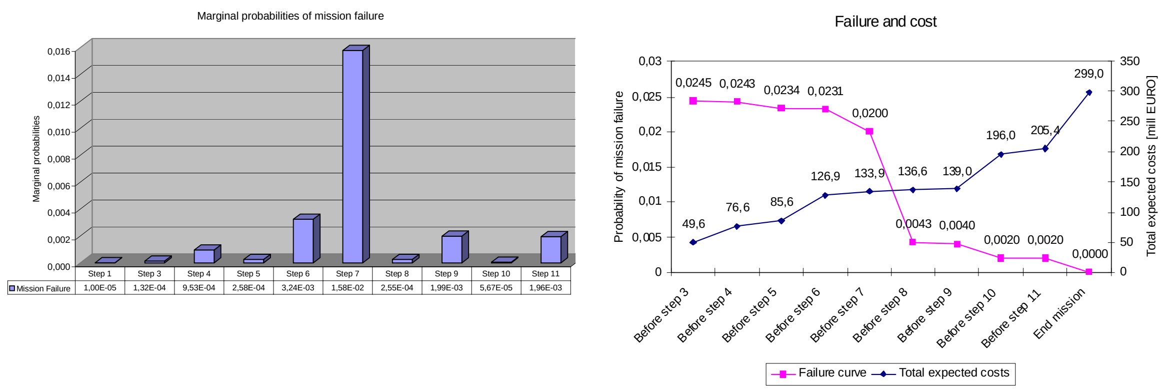

Figure 7 Principal curves showing the character of the analysis results. Computed marginal probabilities of removal failure for the individual steps during the removal operation for one option (right) and the assessed development of the removal failure probability and corresponding expected costs during the process of one removal option.

From Figure 7 the risk contributions from the individual phases of a specific removal option are clearly identified. This provides a basis for targeting further measures of risk reduction to these phases. Also the development of the probability of failure of the removal operation and the corresponding development of expected costs due to the various possible adverse events is identified. These curves indicate the costs, which may be expected to occur before the probability of success of the removal option has reached an acceptable level.

V. Conclusions and Discussions

The issue of reliability of exceptional structures has presented within the context of modern reliability analysis and code formats for the design of ordinary structures. It has been shown that acceptance criteria for the reliability of exceptional structures as well as the reliability level underlying codified design basis for ordinary structures may be established on the basis of optimality considerations taking into account the costs associated with improving the reliability and the consequences in case of failure. In this context it has furthermore been shown that the cost consequences associated with the potential loss of persons can be taken directly into account in the formulation of the optimality problem.

An appropriate level of reliability in design of ordinary structures has traditionally been ensured by prescribed safety factors, which to a certain extend may incorporate the consequences of structural failure for different categories of structures. Some modern reliability based design codes also specify so-called target levels of reliability for structures differentiated in accordance with consequences of failure and the costs associated with increasing the reliability. However, until now the categorization of the failure consequences has been qualitative and rather crude. For ordinary structures this approach may be sufficient but for extra ordinary structures such as high-rise buildings, dam structures, offshore production facilities and nuclear power facilities, where the consequences of failure and the benefits obtained from the structures are very particular a more direct quantification of sufficient and optimal levels of reliability is required.

VI. References

[1] OHBDC, "Ontario Highway Bridge Design Code", Ministry of Transportation, Ontario, Canada.
[2] NBCC, "National Building Code of Canada", National Research Council of Canada, Ontario, Canada,1977.
[3] AASHTO-LRFD, "Bridge Design Specifications", Association of State Highway and Transportation Officials, Washington DC, 1994.
[4] EUROCODES, "Basis of Design and Actions on Structures - Part 1: Basis of Design", ENV 1991-1, 1994.
[5] JCSS, Background Documentation, Eurocode 1 (ENV 1991) Part 1 : Basis of Design. Joint Committee on Structural Safety, 1996.
[6] ISO/DIS 2394 General principles on reliability for structures, Revision of the first edition (ISO 2394:1986), 27-11-1996.
[7] Melchers, Structural Reliability Analysis and Prediction. Wiley, 1999.
[8] JCSS, "Probabilistic Model Code", The Joint Committee on Structural Safety, 2001. http://www.jcss.ethz.ch/
[9] Basler, E. Untersuchungen über den Sicherheitsbegriff von Buawerken, Schweitzer Archiv für angewandte Wissenschaft und Technik, 4, 1961.
[10] Cornell, C. A. A Probability Based Structural Code, ACI-Journal, Vol. 66, 1969, pp. 974-985.
[11] Hasofer, A.M. and Lind, N.C., An Exact and Invariant First Order Reliability Format, Porc. ASCE, J. Eng. Mech. Div., 1974, pp. 111-121.
[12] STRUREL, Version 6.1, Theoretical, Technical and Users manual, RCP-GmbH, Munich, 1998.
[13] DNV, Sesam, "PROBAN Version 4, Theory Manual", DNV Research Report no. 93-2056.
[14] JCSS Guideline for Reliability Based Assessment of Existing Structures, Rilem, 2001.

[15] Raiffa, H. & Schalifer, R. "Applied Statistical Decision Theory". Harward University Press, Cambridge University Press, Cambridge, Mass., 1961.
[16] Aitchison, J. and Dunsmore, I.R., "Statistical Prediction Analysis", Cambridge University Press, Cambridge, 1975.
[17] Nathwani, J.S., N.C. Lind and M.D. Pandey: Affordable Safety by Choice: The Life Quality Methods. Institute for Risk Research, University of Waterloo, Canada, 1997.
[18] Rackwitz, R.: A new Approach for setting Target Reliabilities. Proc IABSE Conf. Safety, Risk and Reliability – Trends in Engineering, Malta, 2001, IABSE, Zürich, 2001, pp. 531-536.
[19] Skjong, R. and K. Ronold: Social Indicators and Risk Acceptance. Proc. OMAE 1998, Lisbon, Portugal.
[20] Faber, M.H. Kroon, I.B. and Kragh, E., *Risk Assessment of Decommissioning Options using Bayesian Networks*, Proceeding to OMAE2001, Rio de Janeiro, 2001.