University of Stavanger

Assessment of existing offshore structures for life extension

Doctorial Thesis by

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Abstract

The subject of this thesis is evaluation of possible life extension of existing offshore jacket structures. This thesis is a contribution to the further development of the theoretical background for the procedures and standards in life extension of offshore installations. The relevant standards for life extension of existing offshore jacket structures are reviewed, with focus on ultimate limit state analysis and fatigue analysis. In general, it can be said that the existing standards and procedures recommend assessment of existing structures for life extension based on: 1) Linear analysis and component checks, 2) Non-linear system strength analysis and component checks, and 3) Structural reliability analysis for the ultimate limit state check and: 1) SN fatigue analysis, 2) Fracture mechanics crack growth analysis, and 3) Structural reliability analysis for the fatigue limit state.

This thesis proposes that a risk evaluation of an ageing structure is needed as a part of the assessment. Such a risk evaluation should include identification of hazards and failure modes, and possibly include an identification of the preventive measures (barriers) for reducing the likelihood of these hazards and failure modes. A review of hazards and failure modes for an ageing offshore jacket structure are presented, and preventive actions to limit the hazards have been investigated using a barrier analysis approach. It is concluded that a review of hazards and failure modes should be included as a part of an assessment for life extension, as installation specific hazards and failure modes may be present.

Further work on system strength parameters for an offshore jacket structure is also included. It is chosen to represent the system strength with the reserve strength ratio indicator, and a reasonable criterion for this parameter is established. Similarly, it is chosen to represent the damage strength with the damaged strength ratio and a recommendation with respect to this parameter is also given. The robustness towards wave-in-deck loading is represented by introducing a reserve freeboard ratio. A criterion for this reserve freeboard ratio has also been suggested. The combined hazard of wave-in-deck loading and the system strength is found to be very important and combined criteria is found necessary.

The use of probabilistic methods in assessment for life extension is also evaluated. In this thesis it is focused on a predictive Bayesian approach for probabilistic assessment. Within this approach the assigned probabilities are a measure of the uncertainty about the structure, and not an element of the structure itself. Decision methods based on the probabilistic analysis should take into account this understanding of the probability. A coherent methodology for decision making based on the predictive Bayesian approach is found, and presented in this thesis. The focus is by this shifted towards minimising the risks with the model used for the probabilistic analysis, instead of meeting a criterion for failure probability.

Finally, the degradation of an offshore jacket structure has been simulated through the life of the structure. Development in the failure probability has been studied with respect to different inspection and repair scenarios. Effects like subsidence and load redistribution after a component has failed are accounted for. From a purely structural point of view, life extension seems to be possible for an offshore jacket structure providing that sufficient inspection and repair is performed, the structure has sufficient strength and the freeboard is sufficient. Possible hazards like corrosion and pile related failures have not been included in this study, and the conclusion is based on these limitations.

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The opinions expressed in this document are those of the authors, and they should not be construed as reflecting the views of my employer, the Petroleum Safety Authority Norway.

Notation

Symbols

All symbols are defined as they first appear in the text. The most common symbols are listed below.

Roman letters:

a	crack depth
a_0	initial crack depth
a_C	critical crack depth
A	crack growth parameter
c	crack length parameter, crack length is $2 \cdot c$
C_1, C_2, C_3	wave load coefficients
C_4, C_5	wave-in-deck load coefficients
C_{1F}, C_{3F}	wave load coefficients used for fatigue load calculations
D	wave direction
E[ ]	expectation operator
F(a)	geometry function for crack growth
$f_X(x)$	probability density function of X
$F_X(x)$	probability distribution function of X
g( )	limit state function
H	wave height
H_{max}	annual maximum wave height
H_s	significant wave height
H_0, H_c	parameters in Weibull distribution of (significant) wave height
L^{ult}_{100}	ultimate capacity of the jacket for the wave load profile with an annual probability of 10^{-2} of exceedance (100 years return period).
L_{100}	the wave load profile for the wave height with an annual probability of 10^{-2} of exceedance (100 years return period).
L_D	design life of a structure
m_a	material parameter in SN curve
m	crack growth exponent
N	number of occurrences
N_w	number of cycles in a sea-state
P_f	failure probability
t,T	time or timeperiod
T_z	mean zero up-crossing period

Greek letters:

α, ξ	uncertainty functions for wave load and resistance
α_H, β_H	parameters in Gumbel distribution of annual maximum wave height
γ	parameter in Weibull distribution of long term (significant) wave height
η	crest height / crest height elevation
η_c	critical crest height / crest height elevation
σ	stress
$\Delta\sigma$	stress range
$\Delta\sigma_{eq}$	equivalent stress range

Abbreviations

ALS	accidental limit state
BS	base shear
CBA	cost benefit analysis
COV	coefficient of variation
DFF	design fatigue factor
DSR	damaged strength ratio
EC	eddy current inspection
FB	freeboard
FLS	fatigue limit state
FM	fracture mechanics
FMD	flooded member detection inspection
MPI	magnetic powder inspection
NFM	new failure modes
NPV	net present value
OTM	overturning moment
PoD	probability of detection
RFR	reserve freeboard ratio
RIF	residual strength factor
RSR	reserve strength ratio defined as L^{ult}_{100} / L_{100}
LR	load redistribution
SF	safety factor
SLS	serviceability limit state
SN	stress versus number of cycles curves used in fatigue design
SR	structural redundancy
ULS	ultimate limit state
WID	wave in deck

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Part 1

1 Introduction

1.1 Background

Offshore jacket structures have been used in petroleum activity for decades. They are the most commonly adopted structure for shallow and intermediate water depths (say $d<150m$). The existing jackets in e.g. the North Sea area have typically been designed for a life of around 20 years. Improvements in the possible oil recovery from several fields have increased the interest for using these structures well beyond their initial design life. Even if rather large reconstructions, repairs and inspections have to be performed, using existing installations beyond their design life will in many cases be economically preferable. A major concern in this regard is that requirements regarding safety should not be compromised.

The age distribution for installations in the UK continental shelf (UKCS) and the Norwegian continental shelf (NCS) shows that a relatively large number of installations have passed 20 years (data from 2003). See Figure 1-1.

Safety of structures is generally assumed to be obtained by design according to established standards and methods, for an expected design life. If a structure is intended to be used beyond its design life, a thorough control of the structural safety must be executed. In particular this will be important with respect to fatigue and other continuous degradation mechanisms. Rules and regulations may have been slightly altered since the original design. The loading pattern (e.g. due to subsidence) and the environmental load may have changed, and the structure may have deteriorated to an unknown extent during decades in harsh weather.

In principle, proper safety of an existing structure can be ensured by requiring compliance with the latest rules and regulations. However, it is not obvious how to perform such safety compliance with regards to life extension of existing structures. In particular, documenting additional fatigue life for a structure that has reached its original fatigue design life is not possible using design regulations, even if no cracks have been detected. It is therefore of importance to develop a scheme which presents a minimum of work to be done in order to ensure proper future safety of a structure well beyond its original design life.

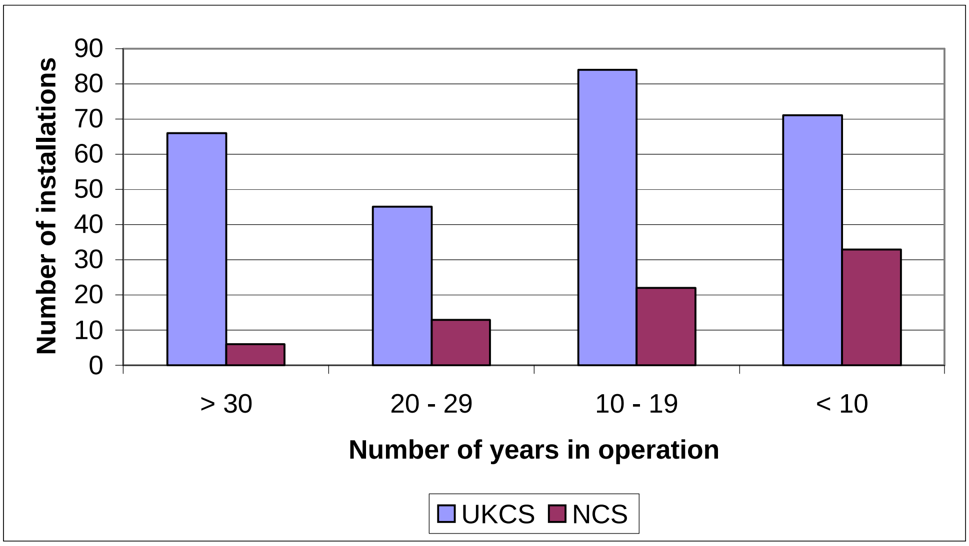

Figure 1-1: Age distribution of existing installations on the British continental shelf (UKCS) and the Norwegian continental shelf (NCS), ref DTI (2003) and NPD (2003). Only main offshore structures with platforms above water, excluding bridge supports and flare towers, are included in the figure.

1.2 Problem description

Offshore structures used today are relatively safe with regards to overload from wave and current loading, provided that the load pattern assumed in design is not significantly altered. For most installations, the major hazards and risks to personnel are caused by the activities on the topside, e.g. drilling and crane operations. The contribution of structural failure to the total number of historic accidents is less than 10% based on worldwide data (Kvitrud et al 2001), and even lower if the fabrication and transportation phase are disregarded. This is a favourable situation, as a collapse of the sub-structure would lead to a large-scale accident with the possibility of many fatalities. However, it should be noted that these statistics are based on a population where very few structures have experienced wave and current loading close to the design assumptions. Hence, historic data of failures due to wave and current loading may be somewhat underrepresented in these statistics.

The safety of a structure against environmental loads, which initially was acceptable, may not continue to be such in an extended lifetime. Subsidence, degradation and other changes to the original design basis may have occurred, and may occur at a higher rate in the extended life. *Degradation* of the structure due to e.g. fatigue and corrosion may decrease the ability of structures to withstand overload due to wave and current loading. Furthermore, several offshore fields are experiencing *subsidence* as a result of petroleum production. Subsidence results in a decreased safety margin towards wave in deck loading, being the worst hazard for many of the offshore structures of jacket type. Also, *improvements in knowledge* about the wave conditions can result in a similar decrease of safety margin towards wave in deck loading.

Finally, lack of knowledge of the structure (e.g. drawings, steel type, welding procedures, inspection results and details about earlier repairs etc.) may result in a high uncertainty about the structures ability to withstand wave and current loading, compared to the uncertainty at the design stage of the structure. In contrast, a well managed structure may have significantly lower uncertainty compared to the uncertainty at the design stage of the structure. A well managed structure in this context would mean that the operator has detailed knowledge about the structure, inspection and repair results. Measurements and experience from structural behaviour in extreme wave and current loading will also reduce this uncertainty.

The major task in this thesis is to use a system approach for evaluating whether a structure is safe for use beyond its initial design life, taking the above mentioned issues into account.

1.3 Previous work on this subject

This thesis is a synthesis of work from a broad area. Previous works in these areas are reviewed below.

Analyses of the robustness of structures using barrier analysis has previously been used as a general accident prevention method in the safety assessment of engineering systems. Barriers to prevent accidents were first described by Haddon in the mid 1960's (Haddon 1973 and1980), and have since been applied to many areas of engineering safety. Important contributions in this area are attributed to Reason (1997), Kjellèn (2000), Sklet and Hauge (2002) and Kragh (2000). However, to the author's knowledge, barrier analysis has not been used to evaluate structural safety prior to the first publications associated with this thesis (Ersdal 2002, Ersdal and Friis-Hansen 2004).

An important part of this thesis is to identify a set of safety indicators that provide a reasonable measure of structural safety against collapse. Structural reliability analysis is used to assess the safety of structures based on the selected safety indicators. It is also used to evaluate the acceptance criteria for these indicators. An introduction to structural reliability analysis can be found in Ang and Tang (1975, 1984), Toft-Christensen and Baker (1982), Madsen et al (1986), and Melchers (1987). The applications of structural reliability analysis to structural system failure taking into account overload, fatigue, and inspections have been studied in Dalane (1993) and Moan (1983, 1997 and 2005).

The indicators of the system strength for offshore jacket structure is based on non-linear collapse analysis (push-over analysis) as established for jacket structures by among others Søreide and Amdahl (1986). This approach was further developed by Hellan (1995) and Skallerud and Amdahl (2002). Similar work on the assessment of structures in operation has been evaluated in the work of Bea and Craig (1993), Dalane (1993), and by DNV, SINTEF and BOMEL in the Ultiguide project (DNV 1999). Based on work from these authors among others, the API (API 2000) and ISO (ISO 2004) recommendations for assessment of structures has been developed.

Degradation of metallic structures due to fatigue has been studied by a large number of researchers. Fatigue occurs in structures exposed to cyclic loading. The awareness of fatigue started in the mid 19th century with the occurrence of fatigue failures in the railway industry (Gordon 1978). Later, fatigue problems also occurred in the Comet aeroplanes and the Liberty ships (Gordon 1978). An increased focus on fatigue problems in the offshore industry was caused by the collapse of the semi-submersible platform Alexander L. Kielland in 1980, due to a fatigue failure (Moan 1981). Among others, the Fatigue Handbook (Almar-

Næss 1985) was developed in the wake of the Alexander L. Kielland accident. More recently, fatigue crack growth has been studied by among others Lassen (1997). An overview of more recent work is given in Barltrop and Adams (1991) and Etube (2001).

In this work, inspection and subsequent repair is viewed as a way to prevent fatigue and corrosion failure in members and joints. Whether the inspection and repair can function as a sufficient barrier is evaluated. The amount of inspection will be critical, and will be based on the inspection planning by the owners of the installations. Inspection planning has for the last decade or so been based mainly on probabilistic analysis, often called risk based inspection (RBI). A workable version of structural reliability based inspection was first introduced around the mid 1980's with the development of structural reliability analysis (SRA), see among others Madsen et al (1987), Madsen et al (1989) and Madsen and Sørensen (1990). This development makes it possible to update, in principle, any possible stochastic model that describes the events. A software system for structural reliability based inspection was developed by Aker Engineering in 1990 (Aker Engineering 1990), and has since been used for inspection planning for several jacket structures. Further developments of this methodology are published in DNV (1996), Sørensen et al (1991 and 1992), Faber and Sørensen (2000), Dalane (1993) and in Straub (2004).

Assessment of structures for use beyond their initial life has not been given as much attention as design of new structures. Repair methods for welds for service life extension have been evaluated in, among others, Haagensen and Skallerud (2004). However, behaviour and safety of the structure as a system has not been evaluated in their work. Due to a number of accidents with ageing aircrafts, the problem of ageing has been considered by aviation engineers. This has led to updated requirements for life extension and updates to the regulation. An overview of how the aviation society has treated the ageing problem is given in Bristow and Minter (2000). Within the offshore industry, the safety of ageing structures has been evaluated in DnV's work for HSE (HSE 1999b), Statoil (2002), Akpan et al (2002) and BOMEL's work for HSE (BOMEL 2003). Assessment of the integrity of offshore structures for life time extension after damages like dents and fatigue cracks etc. has been described in Diamantidis (2001) and PPCoN (1998). SAFERELNET (2004) is an ongoing EU funded project, with a clear subtask for evaluating ageing structures and life extension. The results from this project are not complete at present, but the preliminary results in SAFERELNET (2004) have been included in this project.

The primary source for this work on decision making based on risk and reliability analysis is Aven (2003), but also Benjamin and Cornell (1970), Schrader-Frechette (1991) and MacLean (1986) has been considered. Decision making based on cost optimalization is based on Rackwitz (2000, 2001), Kübler and Faber (2002). The use of Life Quality Index (LQI) and the Implied Cost of Avoiding a Fatality (ICAF) is based on the work of Nathwani et al (1997), Skjong and Ronold (1998) and Skjong (2001).

1.4 Aim and scope

The aim and scope of this thesis is to identify methods for evaluating the safety of a structure beyond its design life. A proper initial structural safety can be harmed by e.g. degradation, component failures, subsidence and new knowledge about the structure or wave conditions.

The basic assumption in this work is that *for a robust and damage tolerant structure the proper structural safety is not restricted by the occurrence of single component failures.* In this context, robust and damage tolerant means that the structure has an acceptable probability of failure due to extreme loading in intact condition or with a single member or joint failure. The major task is then:

- Establish indicators for robustness and damage tolerance. These indicators should ensure that failure due to wave overload accounting for possible wave-in-deck impacts is acceptable in intact condition and with one member failed. The damage tolerance and robustness of a jacket structure is evaluated by a barrier analysis, and indicators for these barriers are established. Acceptance criteria for these indicators are developed based on common practice and structural risk and reliability analysis.
- Evaluate the necessary inspection intervals needed to prevent a single failure from developing into a critical failure and multiple joint and member failures from occurring. The inspection is viewed as the safety barrier to prevent failure in members and joints, and to find these failures and repair them if they have occurred.

Not all offshore jacket structures are damage tolerant, or it may not be feasible to inspect all critical components of the structure. In these cases it is not possible to fully rely on this above mentioned damage tolerant design and maintenance philosophy. Such problems are not evaluated in this thesis.

In this thesis, the assessment of a structure for possible life extension is evaluated based on non-linear structural analysis and structural risk and reliability analysis. The analyses need to take into account degradation, change in physical environment and condition, and knowledge about the structure. The limitations with respect to the proper safety of the structure will be evaluated based on established models of degradation, established methods of evaluating structural system strength and established methods of structural risk and reliability analysis. The possibilities of major repair and improvements of structure and welds are not considered in this work. However, repair of fatigue cracks, if found, are modelled in the numerical simulation presented in this thesis. Decision methods with regards to acceptable lifetime extension will be evaluated and used together with the structural risk and reliability analysis for decision support.

Ideally, assessment of an offshore structure for lifetime extension should be standardised. The method used for assessment should be consistent, and at the same time give good opportunities for lifetime extensions if the structure is sufficiently safe. The method as described here will be investigated for use as a possible foundation for such a standard.

The focus and examples used in this thesis are related to steel jackets in offshore petroleum activity. However, the methods should be transferable to life extension of other structural types. Furthermore, they should be applicable for evaluation of structures after change of the design basis, e.g. as a result of damages and / or changes to the use of the structure.

1.5 Thesis overview

A graphical overview of the main focus areas of the thesis is given in Figure 1-2. Chapter 2 and 3 include a review state-of-art in life extension assessment and methods. Chapter 4 is a discussion about safety of the structure and the possible hazards and failure modes for a structure evaluated for life extension. Based on the existing standards and procedures, an assessment of an existing structures for life extension can be based on linear analysis (design analysis check), non-linear system strength analysis and structural reliability analysis (probabilistic assessment). The probabilistic assessment and system strength assessment is evaluated in chapter 5 and 6 respectively. The degradation of a structure and its effect on the safety of the structure is included in Chapter 7, taking into account inspection and repair.

Chapter 1: Introduction

Chapter 2: Assessment of existing structures
Chapter 3: Methods applied in thesis

Literature review and discussion of existing methods

Chapter 4: Failure modes of Jacket structure.	Chapter 5 and Part II papers: Assessment by probabilistic methods.	Chapter 6: Assessment of system strength.	Chapter 7: Assessment by fatigue by simulations.
Safety of structures. Hazards, failure modes, barriers and barrier indicators. Quantitative / qualitative risk modelling of jacket structures exposed to wave and current loading.	Probabilistic assessment and decision making. The use of probabilistic analysis and decision methods based on probabilistic methods.	System strength analysis of a jacket and the influence of freeboard.. Evaluation of indicators for system strength of the structure (barrier indicators) and acceptance criteria for these barriers.	Simulation of ageing structures. Evaluation of the safety of a structure with increasing age including degradation by simulation of wave loading, crack growth and overload failure.

Chapter 8: Concluding remarks

Figure 1-2: Graphical overview of thesis.

1.6 Limitations

This thesis evaluates the jacket structures exposed to wave loading. Other structural parts such as piles and foundation are not evaluated. Also, other major hazards such as earthquake loading, boat impact and corrosion are outside of the scope of this thesis. Piles will clearly be an important element of a full assessment of an existing structure for life extension. Piles will degrade due to fatigue and corrosion. It is also difficult to inspect the piles of an offshore jacket structure. Earthquake loading and boat impact loading may be governing for some structures. However, earthquake loading and boat impact are not studied and definite conclusions on such hazards cannot be made based upon this thesis. Corrosion will definitely be an important hazard for the structure in cases where the corrosion protection is not sufficient for the extended life, or where corrosion allowance from design is not sufficient for the extended life. Hence, also corrosion effects on an ageing structure in a life extension would need a specific investigation.

2 Assessment of existing structures

2.1 Introduction

This chapter gives a state of the art description of procedures for assessment of existing structures. The structural engineering discipline has mainly focussed on the design phase of structures. Hence, literature, guidelines and recommended practises are mainly developed for design of new structures. The guidelines and recommended practices that exist for assessment of existing structures do not have the same level of practical experience and may have shortcomings compared to regulations and recommended practises for design.

Assessment of existing structures is performed in order to extend service life of the facility, as new methods of production and new discoveries may result in a request for life extension. From an economic point of view the continued use of an existing installation will in many cases be preferable, compared to a new installation. This will be preferable for several installations even with major modifications to the structure. Assessment will also be needed after addition of more equipment, more personnel and deterioration and damage to the structure. The purpose of an assessment of existing structure for possible life extension should be to ensure that the probability of structural failure is acceptable. In order to achieve this, several assessment procedures are proposed. A few of these will be presented in the following text.

The most general accepted standard for offshore structures is the ISO 19900 "Petroleum and natural gas industries – Offshore Structures – Part 1: General Requirements" ISO (2002). This standard gives general design rules and general rules for assessment of existing structures. The Norwegian regulations (PSA 2004) refer to NORSOK N-001 (NORSOK 2004) for structural design, which again refer to ISO 19900 (ISO 2002) for assessment of existing structures. However, ISO 19900 is a rather general standard, not very specific on how to perform assessment. The standard gives some indications that a Design Code format, a Reserve Strength Ratio format and a probabilistic format are acceptable. The ISO 19900 refers to ISO 19902 (ISO 2004) for design and assessment of offshore steel structures. A detailed assessment procedure for existing structures is found in ISO 19902, and this procedure is presented in this chapter. Other standards, like API RP2A-WSD (API 2000) and ISO/DIS 13822 (ISO 2000), also includes detailed procedures for assessment of existing structures. These are also presented in this chapter. Furthermore, it is worth mentioning that the Joint Committee for Structural Safety (JCSS) has developed a procedure for probabilistic assessment of existing structures (Diamantidis 2001). Operators with experience with assessment of existing structures have developed their own procedures for such assessment (PPCON 1998). Finally, there is at present an ongoing EU founded research project that includes a work package on life extension of structures (SAFERELNET 2004).

In general terms, based on the reviewed standards and recommendations, the existing assessment procedures consist of the following steps:

- Consideration of whether assessment is needed.
- Information review (Design, fabrication, installation and operation history)
- Screening of structural state (major damages, major changes, deviations from design)
- Analysis of the structure (design analysis, ultimate strength analysis or probabilistic analysis)
- Decision making (acceptable as is, modification needed or abandonment).

The general form of these procedures is also presented as a flowchart, as shown in Figure 2-1.

The purpose of the assessment of an existing structure is to ensure that the structure has an acceptable safety. A reasonable goal could be that the existing structure should have a comparable level of safety as a newly designed structure. In order to ensure this, a method of evaluating the safety of the structure has to be established. Possible ways of evaluating the structural safety is by proof-loading or by an analysis that estimates this safety. However, proof-loading is not easily applicable for offshore structures, leaving analysis as the main option.

2.2 Safety of structures

The purpose of a structural analysis is to achieve sufficiently safe and effective structures. Methods of designing structures have developed from “trial and error” to linear elastic design by load and strength calculations and code check. The development of linear elastic design has to some extent been a process of “trial and error”. The design against new failure modes have been introduced into the code checks as more experience on incidents and accidents has been collected. Important aspects of this development are the understanding of structural behaviour aspects like brittle cracking and fatigue. Developments on structural loading have also been important, as vortex induced vibrations and wave slamming loads. Linear elastic design and the codes used today for offshore structures are relatively mature. Failures are relatively rare for structures in operation on a field. However, it should be noted that very few offshore structures have been exposed to any loading comparable with the design loading. Hence, very few failures should be expected and the failures that have been seen are rather due to errors in design, fabrication or installation.

For a jacket structure it is important to design both against global loads (e.g. wave loads, earthquake loads and boat impact loads) and local loads (e.g. local wave loads, wave slamming and vortex induced vibrations). In some cases failure at a local level may result in unacceptable situations. A failure may start at a local level (e.g. by overload of a single member or fatigue cracking in a joint) and cause a critical situation by itself. An example can be if the failed member can fall and damage other important installations or humans or is carrying important equipment like risers, conductors or caissons. The collapse of modules on topside, e.g. living quarter, would also be a major concern in itself. However, the main concern is if the local failure escalates into a total collapse of the structure.

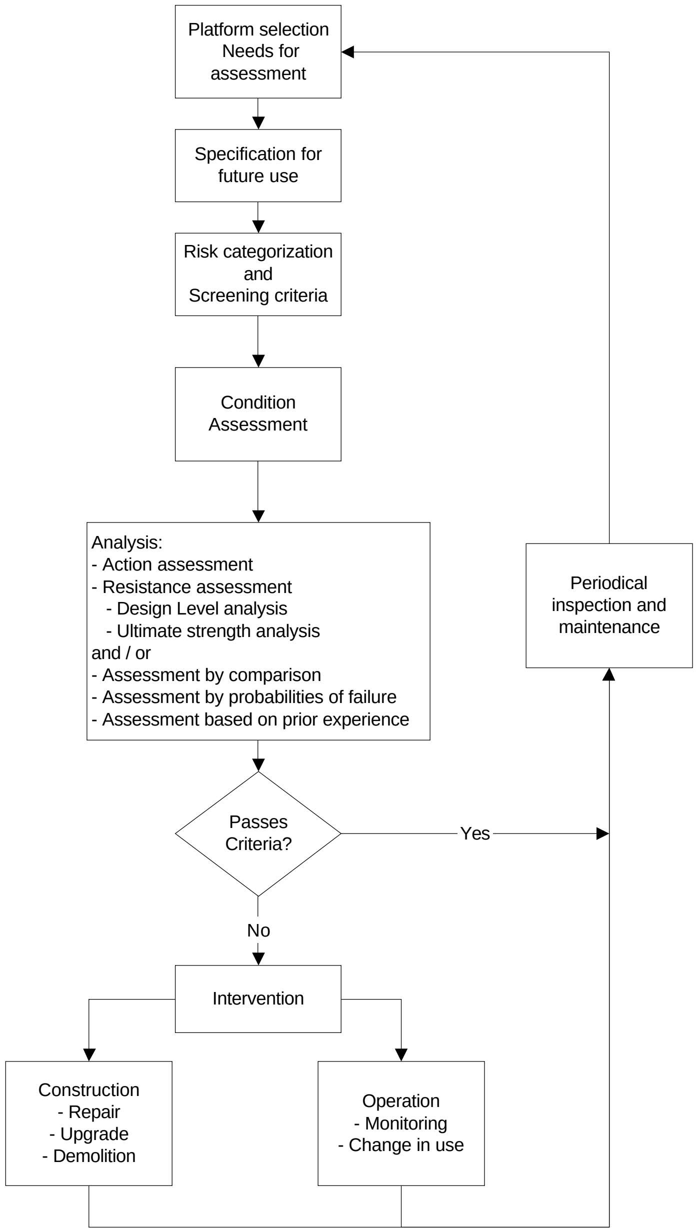

Figure 2-1: General assessment procedure

When designing new structures, the most used method for ensuring safe structures is the use of linear elastic analysis and component checks by standardized formulas. The members and joints are checked for overload (ultimate limit state), fatigue (fatigue limit state) and for accidental load situations such as explosion, fire and boat collisions (accidental limit state). Most recognised standards and recommended practices will include the relevant failure modes[1]. Structures designed according to such standards are tacitly assumed to be sufficiently safe.

As the safety of the structure in most cases depends on the structural system strength to withstand global load, an alternative approach is to evaluate this system strength directly. This may be done by linear elastic or non-linear elasto-plastic analysis. However, non-linear analysis is the most commonly used method for this type of analysis. The analysis is typically performed using a non-linear finite element program taking into account non-linearities in both geometry and material behaviour. According to the lower bound theorem in the theory of plasticity[2] (see e.g. Chakrabarty 1987), the result of such non linear analysis is closer to, but lower than, the collapse load of a structure compared to a linear elastic analysis. The collapse analysis check can only be used for checking margins of safety against design loads (ULS) and accidental loads (ALS). The fatigue limit state (FLS) also needs to be taken into account, as it is not a part of the non-linear system analysis in itself. The fatigue limit state may be accounted for by the same methods as used for fatigue limit state analysis in linear analysis and component checks. The effect of fatigue failures can, however, be evaluated in the system collapse check. Hence, a fatigue limit state evaluation combined with a non-linear system analysis will yield more information about the robustness of the structure than a linear analysis with component check. It is at present not common to design structures according to such a non-linear collapse analysis, but the system capacity check of a structural design is increasingly noticed as an important check of the structure.

The third main alternative is the use of probabilistic methods as a framework for the analysis. Probabilistic methods have been introduced rather recently (in a historic perspective compared to linear elastic design) as a possible approach for designing and assessing structures. The probabilistic analysis may be in the form of structural reliability analysis (SRA) or quantitative structural risk analysis (QSRA). In SRA the parameters used in a linear analysis and component check or the non-linear collapse analysis are modelled as random variables the in accordance with the expected probability distribution for the parameter rather than just the characteristic value. The probability of the limit state being exceeded is then calculated, using the same strength formulas as in a deterministic analysis. The resulting probability of failure is hence a probability of exceedance of the limit state of a code, and not the overall probability of failure for the structure. The analysis can be

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[1] When designing new and untested concepts, a more thorough analysis of possible failure modes may be necessary and standards and recommended practices may not be fully sufficient.

[2] A fundamental requirement in structural analysis is that the calculations should be on the conservative side. According to the lower bound theorem of plasticity, an external load in equilibrium with the distribution of internal stresses, which nowhere exceeds the acceptable plastic stresses in material, is less or equal to the collapse load, if ductility is acceptable (Chakrabarty, 1987). Normally this is checked by using a linear elastic analysis, giving statically admissible forces, followed by a component check of stresses according to accepted standards, and the use of ductile material. A method typically used for collapse analysis (also called pushover analysis), including geometric stiffness and plastic hinges, will also give a solution according to the lower bound theorem. This solution is theoretically closer to the lower bound solution than the linear elastic solution. The established safety factors are developed based on linear analysis methods, and these safety factors cannot necessarily be directly adapted to non-linear solutions without some adjustment, if the goal is the same level of safety.

performed on component basis, checking the individual component reliability against a target minimum reliability. The analysis could also be performed on a system basis, checking the total loads and the system strength against target reliability.

The difference between a SRA and QSRA has traditionally been that the SRA has only included the uncertainties related to physical parameters and lack of knowledge about the physical parameters and the models used in the analysis, while QSRA also include failure modes resulting from errors by humans and accidental events. QRSA also includes consequences of the various failure modes. Both SRA and QSRA give a direct measure of the safety, and as such they are the most obvious choice for evaluating the safety of a structure. Also the decision making on what is sufficient safety is an important task in this type of analysis. Methods for making decisions based on SRA and QSRA are proposed in literature and standards. These include comparing the calculated failure probability with target failure probabilities, the use of Cost Benefit Analysis, implementation of the ALARP, and the use of Multi Attribute Analysis.

Existing structures versus new designs

When focusing at existing structures at the end of their calculated design life rather than the design of new structures, the main question will be if the safety established in design is still valid. Using similar analysis methods as mentioned above, but taking into account experience and inspection results, may be a possible method for evaluating the continuous safety of the structure. Alternatively, the objective of inspection of the structure and the maintenance of the structure may be to ensure an as-constructed state at all time. Also, any part of the structure that is not meeting the assessment requirements may be strengthened (e.g. grouted) or replaced with new structural parts.

Other ways of reducing the risk to personnel related from structural failure are to introduce risk prevention and mitigation procedures, e.g. by evacuation procedures. Evacuation may be an option if the main hazard is caused by a predictable event as excessive wave loading or wave in deck loading. Evacuation may not be a sufficient preventive action if degradation is the hazard primarily of the structure, where the structure can fail at low sea-states where evacuation is not effectuated. Evacuation is not evaluated in detail in this thesis. It should be regarded as a possible alternative if detailed calculations and maintenance is not sufficient, or subsidence is significant.

In most cases there is a significant difference in the knowledge about an existing structure and a structure in design phase. It is important to account for this fact when evaluating the safety of existing structures. If the structure is well managed and maintained, the knowledge about the structure may be significantly increased. This increased knowledge based on construction and operation data may be used to increase the accuracy of the structural analysis, which will reduce the uncertainty about the structural performance. Structural design codes have to be slightly on the safe side in order to take care of uncertainties (material selection, fabrication methods, etc) at the design stage. This conservatism may be excluded to some extent in an assessment of an existing structure in form of reduced safety margins, if such uncertainties are reduced. However, it is also likely that information about an existing structure is difficult or impossible to find, e.g. due to a poor data management systems. This will increase the uncertainty about the structure as compared to the design case. Hence, considerable penalties in form of larger safety margins should be required (Stacey et al 2002).

For an existing structure it can of course be concluded that it has not failed yet. It has withstood the loads that the structure has been exposed to. The nature of the loads that it has been exposed to is however not necessarily known in great detail. Data from inspection can show the fatigue cracking, subsidence and marine growth and other parameters that are important for the loading on the structure. Information from the actual material certificates and welding certificates may give information about the steel quality and welding quality. Further, there may have been performed simultaneous measurements of waves and current and the corresponding stresses and motions in the structure. Finally, for existing structures it is often economically favourable to perform more detailed analysis[3], new inspections, tests and measurements rather than redesigning and increasing the steel weight, reconfiguring the structural parts or replace the existing structure with a new structure. In contrast, in the design of a new structure, detailed analysis often costs more than a structural change early in the design phase. These differences are discussed in Kallaby et al (1994) and Moan and Vårdal (2001), and a summary of these discussions is shown in Table 2-1.

Table 2-1: Assessment versus design.

	Design criteria	Assessment criteria
Environment	Forecast from existing data collection	As criteria for "new " platform, with inclusion of recent data collection and use of: - current state of the art review - experience from adjacent fields - additional data from actual field sea-states
Loading	Possibly conservative evaluation from proposed use of structure	Conservative evaluation from as-built records and use of recent survey info on: - marine growth - appurtenances - removals / additions / modifications - topsides weight control - wind areas
Foundation	Forecast from site investigation and laboratory testing of soils	As criteria for "new" platform with inclusion of: - subsidence information - current state-of-the-art review - experience form adjacent fields - post-drive foundation analyses - scour survey and maintenance
Structural model	Topology and dimensions may be changed. No service inspection available. Conservative modelling using global percentages to cover not-finalized details and simple geometric assumptions.	The structural dimensions are fixed and known. In-service inspection may be applied. Actual characteristic strength of steel based on actual material certificates may be used. Structural performance may have been measured and used to update structural analysis.
Stress analysis	The time for analysis is critical. Strict compliance with code of practice and regulatory documents.	The quality of the analysis is critical. Sufficient time for model tests, removing of conservatism where possible, redundancy studies to determine ultimate strength of structure and foundation, sensitivity studies on various parameters to improve confidence levels.
Results	Structure has members and joints with acceptable utilization.	Structure has some stresses up to yield stress, but some assessment standards allow for some yielding if the structure has proven strength and redundancy.

[3] Design codes tend to use very simplified formulations for utility, capacity etc under the assumption that the formulas should be easy to use, rather than to use the most exact formulation which may be cumbersome to use but will be more exact and will eliminate some of the conservativism that is needed in general and simplified formulations.

Most of these items could be up-dated information that should be included in an assessment of an existing structure. If the up-dated information gives a positive result with regards to the safety of the structure, this should be accounted for in a life extension procedure. In other situations, the information may indicate a reduced safety of the structure, which should also be accounted for.

Some procedures allow for a lower acceptance criteria to be used for assessment of existing structure compared to design, e.g. lower load factors or Reserve Strength Ratio criteria. Moan and Vårdal (2001) conclude based on Table 2-1 that in design "High uncertainty requires a large safety margin", but in an assessment situation "Reduced uncertainty and a reduced safety margin may be applied". Such a reduced acceptance criteria must be based on the fact that uncertainty about loading or capacity of the structure being reduced. It can not be an automatic part of a reassessment procedure for any existing structure. The uncertainty about loads is relatively difficult to reduce, and would require simultaneous measurements of environment and structural response. Furthermore, the major uncertainty about the environmental loads is generally related to the future environment (e.g. future waves). Reducing this uncertainty would require long series of measurement. However, some possibilities are mentioned in Table 2-1. If lower acceptance criteria should be allowed, they must be based on reduced epistemic uncertainties after some years of experience. For a structure where the probability of failure is dominated by the fatigue failure modes, the "no detection of a propagating crack" after say 20 years of service may be an indicator of improved reliability. However, for a structure that is dominated by ultimate limit state or accidental limit state failure modes this would not be the case. As an example; a structure that is dominated by failure due to wave in-deck will not benefit from the same information unless the structure has experienced severe weather with a wave in deck incident.

A probabilistic method will take this reduction or increase of uncertainty into account and express it in an updated safety of the structure in a rational way. This updated safety should be accounted for in an assessment of an existing structure, both when it benefits the assessment of the structure and when it does not.

The possibility of new failure modes for ageing structures

When moving from designing new structures to extending the life of existing structures, it will be of great importance to evaluate whether there is a possibility for failure modes of the existing structure that was not considered in the design phase.

> *"With structures there are often several alternative possible modes of failure. Naturally the structure breaks in whichever of these ways turns out to be the weakest – which is too often the one which nobody had happened to think of, let alone do sums about".*
> Gordon (1978)

The checking of possible additional failure modes for ageing structures is an important task independent of the method used for structural analysis. Even the use of linear elastic design code methods and deterministic fracture mechanics for evaluating crack growth may not ensure a safe structure beyond its intended design life. As an example: A structure that is evaluated based on linear analysis and component checks. The component check is found to be satisfying. The structure is also checked for fatigue. Some components have low fatigue life, but adequate inspection intervals are found by deterministic fracture mechanics. The inspection interval is set equal to the time it takes a crack of a given size to grow to a through thickness crack. The given size is chosen as the maximum crack that was likely to be missed in the inspection. It is under these conditions possible that the weakest failure mode is the failure modes of two or more simultaneous cracks occurring within the same inspection interval. This possible failure mode is not accounted for in the linear elastic design.

Load redistribution after failure of a component will be an effect that boosts such a failure mode. This will be studied further in Chapter 7 of this thesis.

2.3 Assessment procedures

The emphasis of this review of assessment procedures is given to the ISO/DIS 19902 as this standard is referred to in the Norwegian regulations (PSA 2004). Regarding the review of API RP 2A WSD (API 2000) and ISO/DIS 13822 (ISO 2000) focus is more on differences between these standards and ISO/DIS 19902.

2.3.1 Assessment process proposed in ISO/DIS 19902

ISO 19902 (ISO 2004) is so far only a draft standard, and changes in this standard might occur in the final version. Earlier versions of this ISO standard have been based on API. This paper reviews the DIS (Draft International Standard), issued in September 2004.

The standard states that it is the owners' responsibility to maintain and demonstrate fitness for purpose of the platform for the given site and operating conditions. The goal is to demonstrate that the annual probability of a severe structural failure is sufficiently low. The acceptable annual probability depends on regulatory requirements supplemented by regional or industry standards and practices. The ISO 19902 clearly states that the design philosophy for existing structures allows for accepting limited damage to individual component, provided that both the reserve against overall system failure and deformations remain acceptable. The standard is intended for application to existing jacket substructures, but could also be used for topside structures.

The ISO 19902 procedure includes both a check of the ultimate limit state and the fatigue limit state. Generally, if one of the platform assessment initiators exists, the structure shall undergo an assessment with possibly 5 analysis levels and 2 empirical methods. The first empirical method is to compare the structure with similar structures. The second empirical method is assessment by prior experience. Level 1, is a linear analysis and component check. Level 2 is also a linear analysis and component checks, but now with refined actions and resistances. Level 3 is a linear elastic redundancy analysis. Level 4 is a non-linear analysis on system level including component checks as an integrated part of the non-linear system analysis. Finally, level 5 is a check by using structural reliability analysis. If the structure is found acceptable at a level, no higher levels of checking are necessary. Prevention and mitigation measures to reduce the occurrence rate and the consequences of structural failure should be considered during all stages of the assessment process. A flowchart of the procedure proposed in ISO 19902 is shown in Figure 2-2.

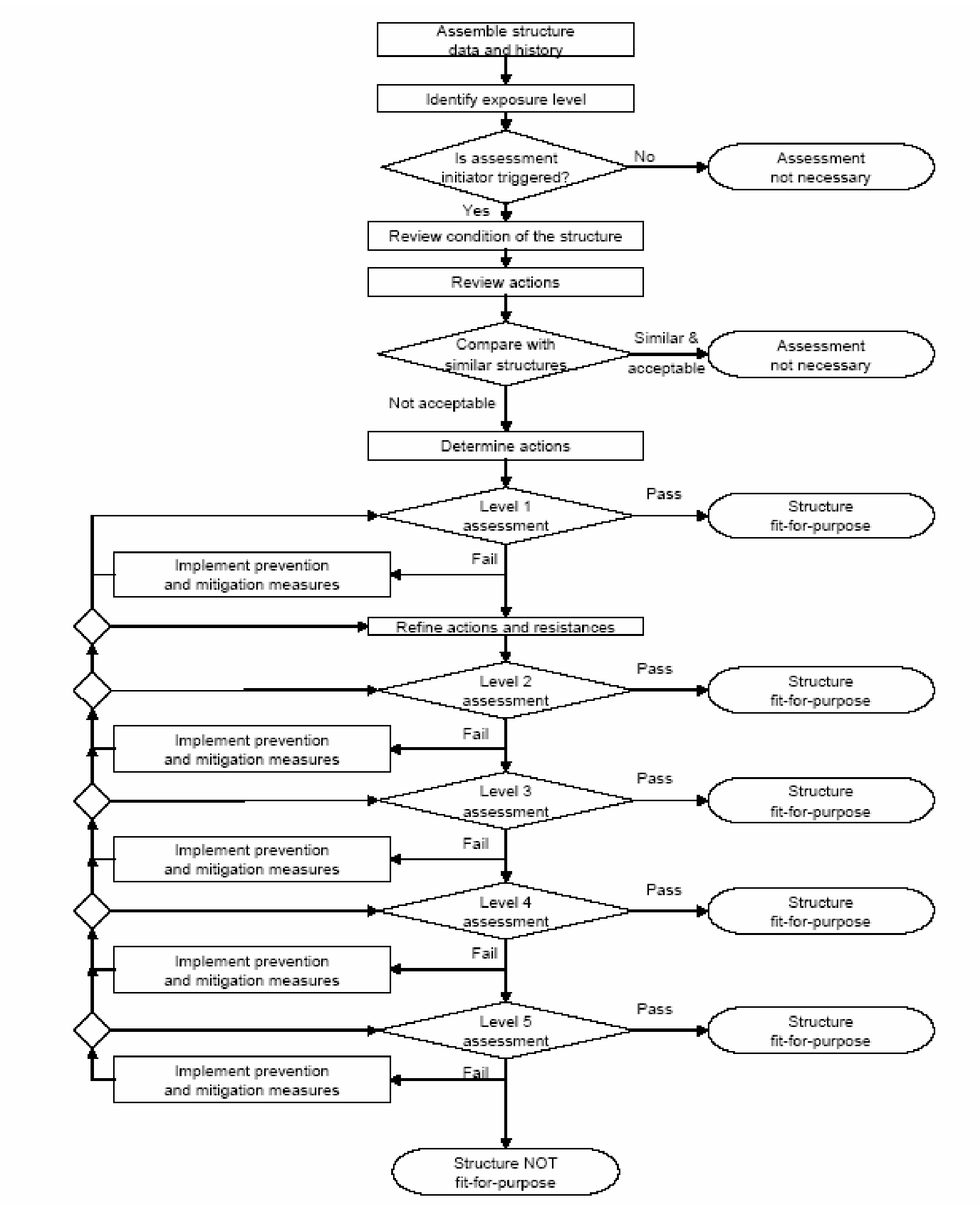

Figure 2-2: Assessment procedure proposed in ISO DIS 19902 (ISO 2004)

An existing platform should undergo an assessment to demonstrate fitness for purpose if one or more of the following conditions exist:

- Changes from the original design or from previous assessment basis:
    - Addition of personnel or facilities;
    - Modification of facilities;
    - More onerous environmental criteria;
    - More onerous component or foundation resistance criteria;

    - Physical changes to the platform's design basis, e.g. scour or subsidence;
    - Inadequate air gap.
    - Exceedance of intended design life.
- Damage or detoriation of a primary structural component. Minor damage may be accepted, but the cumulative effects of damage should be documented and accounted for in a global resistance assessment.

Sufficient information must be collected to allow for an engineering assessment of the platform's overall structural integrity. Information of the platform's structural condition and facilities, with particular attention to data that cannot be explicitly verified (e.g. pile penetration), should be collected. General inspection of topside, underwater, splash zone and foundation should be performed, and decision on whether more detailed inspections and possible soil borings is necessary, should be performed based on engineering judgement.

Changes from the design basis or previously assessment basis, as indication of more onerous environmental conditions, are one of the triggers for a new assessment. "The metocean (meteorological and oceanographic) data required for an assessment are the same as for design, as are environmental design situations and actions" (ISO 2004 p 319). Further "The existing deck height, with an allowance for any future subsidence within the design service life, shall be determined. The deck height shall be checked for potential inundation as this can limit the overall structural reliability. The asymmetry of the wave crest versus wave height should be considered, preferably supported by measured data" (ISO 2004 p 319). Other loadings should be considered as for design.

A structure may, according to ISO 19902, be assessed by comparison with a similar nearby structure, where sufficient similarity can be demonstrated and the structure used for comparison is found fit for purpose. There is listed a series of requirements to the structure under assessment and to the similar structure. Comparison with similar structures is not the focus of the work in this thesis, and these requirements are not further discussed.

In general, the assessment of structural components is to be performed in accordance with the design analysis clauses of the ISO 19902 standard, taking into account the current condition or future intended condition, accounting for any damage, repair, scour, modifications, or other factors that may affect the structural performance or integrity.

An ultimate strength analysis (Level 4 – Non linear analysis and component check) is intended to demonstrate that a structure has adequate strength and stability to withstand a significant overload, with respect to the applied loads. Local overstress and potential local damages are accepted, but total collapse or excessive / damaging deformations are to be avoided. The ratio between the design loading (usually 100 year loading) and the collapse / ultimate capacity is then established, and usually termed reserve strength ratio (RSR). The RSR shall be determined for all wave directions and the lowest value obtained shall be the structure's RSR. The criteria for acceptance of the RSR may differ depending on local requirements, but the general acceptance criterion in ISO 19902 is set to 1.85. If the RSR check is found acceptable, no further analysis is necessary.

The highest level of assessment is the structural reliability analysis (SRA). In ISO 19902 it is noted that the use of SRA requires extreme care and there is insufficient knowledge of the statistics to enable requirements or recommendations to be included in a standard. ISO 19902 states (ISO 2004 p 323) "As the reliability determined by an SRA is highly dependent on the knowledge and skill of the analyst and the data upon which the analysis is based, it is not possible to provide acceptance criteria in this document. It is recommended that thorough validation of the techniques and application of those techniques is undertaken, that acceptance criteria are agreed between the regulator, where one exists, and the owner, recognizing the scope for over-optimism in determining the RSR, and that the results of the SRA are combined with consideration of the costs and benefits of strengthening or other remedial measures".

If it can be demonstrated that a structure has already withstood events that are no less onerous than those for which it is to be assessed the structure may be assumed to be acceptable. However, the prior event must be representative for all components and storm directions. There are given some limitations to the use of assessment by prior exposure, e.g. it is mentioned that if the environmental loading has exceeded the design environmental loading, it is likely that the environmental database should be revisited and this event should be included in the database, and that new analysis will show that this event no longer represent a extreme event.

Three different methods for assessing the structure with respect to fatigue are proposed (ISO 2004 p 324). The assessment shows adequate fatigue durability if one of the following criteria is met:

- "The results of a fatigue assessment in accordance with Clause 16 (the design fatigue clause of ISO 19902) show that the fatigue lives of all members and joints are at least equal to the total design service life, and the inspection history shows no fatigue cracks of unexplainable damage".
- "A fatigue assessment in accordance with Clause 16 has identified the joints with the lowest fatigue lives and periodic inspection of these joints finds no fatigue cracks or unexplained damage".
- "Where fatigue lives of any members and joints are calculated to be less than the total design service life of the structure and fatigue damage has been identified, the structure may be assumed to be fit-for-purpose providing conservative fracture mechanics predictions of fatigue crack growth demonstrate adequate future life and periodic inspection monitors crack growth of the members or joints concerned".

As this section of the ISO 19902 is addressing life extension of installations, it is reasonable to assume that the design service life mentioned above relates to the total expected life of the structure including life extension. Hence, total design life would imply original and extended life of the structure.

According to the ISO 19902 (ISO 2004 p 318) an "extension of the design service life may be accepted without a full assessment if inspection of the structure shows that time-dependent degradation (i.e. fatigue and corrosion) have not become significant and there have been no changes to the criteria for design".

2.3.2 Assessment process proposed in API RP 2A WSD

The API RP 2A WSD (API 2000) is the most commonly used standard for design of fixed offshore structures in the United States. The standard is also used in countries outside US, but is not commonly used in Norway. However, the standard is of interest as it is the only existing offshore standard that takes assessment of existing structures to a detailed level.

The standard is stated to be applicable only for the assessment of platforms, which were designed in accordance with 20th or earlier editions of the same API standard. Structures designed after the 21st edition, should be assessed in accordance with the criteria originally used for the design. By this clause API is limiting the possibility for using assessment of existing structure to minimize the structural cost by building a platform intended to have extra modules, but not fully designed for these additional modules, and then adding them later in an assessment program under reduced criteria.

The elements of selection of platforms for assessment, categorisation of safety level for the installation and condition assessment does not differ significantly from the ISO 19902 procedure.

There are two potential sequential analysis checks mentioned in API RP 2A WSD, a design level analysis and an ultimate strength analysis. The analysis in itself seems to be the same as mentioned in ISO 19902, but the acceptance criteria are different. Design level analysis procedures for assessment are similar to those used for new platform design, including the application of all safety factors, the use of characteristic rather than mean yield stress, etc. However, lateral environmental load may be reduced to 85% of the 100-year condition for high consequence platforms, and to 50% for low consequence platforms. In the ultimate strength analysis, the Reserve strength Ratio (RSR) is defined as the ratio of platforms ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading. A RSR of 1.6 is required for high consequence platforms, and 0.8 for low consequence platforms.

In addition, assessment of similar platforms by comparison, assessment through the use of explicit probabilities of failure and assessment based on prior exposure are regarded as acceptable alternative assessment procedures subjected to some limitations. In general these assessment methods seem to be similar to these of ISO 19902.

2.3.3 Assessment process proposed in ISO/DIS 13822

ISO/DIS 13822 (ISO 2000) is a draft international standard, and changes in this standard might occur in the final version. The two standards mentioned previously are both based on reserve strength analysis and design analysis, while ISO/DIS 13822 is mainly a reliability-based assessment standard.

Again, the elements of selection of structures for assessment do not differ significantly from the ISO 19902 procedure. The objective of the assessment shall be specified in terms of future performance required for the structure in an agreement with owner, authorities and the assessing engineer. The required future performance shall be specified in the utilisation plan and safety plan. Scenarios related to a change in structural conditions or actions should be specified in the safety plan in order to identify possible critical situations for the structure. The wording in this standard may differ from ISO 19902, but the elements are to a large extent similar.

This standard includes an option of a preliminary assessment and, if found necessary, a detailed assessment. The *preliminary assessment* includes a verification of documents, a check of occurrence of large actions, change in soil conditions and misuse of the structure and preliminary inspection of the structure for possible damage of the structure. ISO/CD 13822 clause 4.4.5 states that "The preliminary inspection may clearly show the specific deficiencies of the structure, or that the structure is reliable for its intended use over the remaining working life, in which case a detailed assessment is not required. Where there is uncertainty in the actions, action effects or properties of the structure, a detailed assessment should be recommended in accordance with Section 4.5". Section 4.5 of the ISO/DIS 13822 is the section for detailed assessment of the structure.

The *detailed assessment* as described in ISO/DIS 13822 includes similar elements as the ISO/DIS 19902, including detailed documentary search and review, detailed inspection and material testing, determination of actions, determination of properties of the structure and structural analysis. The degradation of the existing structure should be taken into consideration.

Section 4.5.6 states that "The verification of an existing structure should normally be carried out to ensure a target reliability level that represents the required level of structural performance". In the informative annex F it further stated that "The target reliability level used for verification of an existing structure can be determined based on calibration with the current code, the concept of the minimum total expected cost, and/or the comparison with other social risks. The requirements should also reflect the type and importance of the structure, possible future consequences and socio-economical criteria". Proposals for target failure probabilities for various limit states are also given.

An alternative approach to the structure intervention, which may be appropriate in some circumstances, is to control or modify the risk by imposing load restrictions, altering aspects of the use of the structure, and implementing monitoring and control regime.

ISO/DIS 13822 also opens for an assessment based on satisfactory past performance with a set of limitations not reviewed herein.

The client in collaboration with the relevant authority should make the final decision on interventions, based on engineering judgement and the recommendations in the report and considering all the information available.

2.3.4 Assessment process proposed by Joint Committee on Structural Safety

The assessment procedure for existing structures by the Joint Committee on Structural Safety (JCSS) is published in Diamantidis (2001). It is based on probabilistic methods for assessing the structures, and gives recommendations and advice for the structural reliability analysis and assessment in general. The JCSS has also published a "Probabilistic Model Code" (JCSS 2001), giving advice on the use of structural reliability analysis. The details of recommendations for structural reliability analysis will not be presented here, but some of the general recommendations for assessment of existing structures are interesting and will be presented.

In general the procedure does not significantly differ from the previously described, but two elements are focussing directly on important aspects.

The first is the definition of hazard scenarios. On the basis of the utilisation plan and in view of the planned future use of the examined structure, a list of hazards likely to act on the structure must be defined. The term *hazard scenario* is a rather broad concept. It calls for imagining a situation, transient in time, which a structure might happen to undergo which would endanger its integrity and thereby put human lives at risk. The *hazard scenario* concept is especially applicable to existing structures since a direct application of existing codes is not possible.

The second is the *safety plan*. The *safety plan* assigns the appropriate counteracting safety measures to the defined hazard scenarios. Measures can be drawn from a number of categories:
- Eliminating hazard scenarios at the source of its leading hazard.
- Avoiding hazards scenarios by changing intentions or structural concepts.
- Controlling hazards scenarios by safety devices, warning systems as well as by checking, supervision, inspection followed by adequate corrective measures.
- Overpowering hazards scenarios by dimensioning using adequate safety margins.
- Accepting hazard scenarios because they either cannot – without prohibitive cost – be counteracted by one or more of the above mentioned measures.

The list of accepted risks clarifies who profits from accepting risks and who bears the consequences. Conceptionally, both should be the same person or body.

The hazard scenarios and safety plan to prevent these hazard scenarios is in line with the check for possible new failure modes for ageing structures mentioned earlier in this chapter. The use of hazard-, failure mode- and barrier-identification described later in this thesis will try to cover these aspects.

2.4 Discussion and comments to the standards

The ISO 19902 is the most relevant standard for fixed offshore steel structures (jackets), which is the focus for this thesis. Hence the following discussion will concentrate on this standard. The ISO 19902 standard seems to be a relatively good procedure for assessment of existing structures for life extensions. However there are some questions that need to be addressed, and to some extent these are discussed further in this thesis.

2.4.1 Hazard and failure mode identification and counteracting measures

The ISO/DIS 13822 and JCSS give explicit recommendations for performing a hazard and failure mode identification, and subsequently define and implement counteracting safety measures. This is, as described earlier, an important task that should be a natural task in all standard procedures for assessment of existing structures.

The similar recommendation in ISO/DIS 19902 and API RP 2A WSD is more implicit. Both these two standards are directly focussed on offshore jacket standards, and it is likely that the intention in these standards is to include the additional hazards and failure modes, rather than to describe a procedure for evaluating such additional hazards and failure modes. However, examples of not included hazards and failure modes may be:
- Multiple fatigue cracks reducing the structures capacity within an inspection period, leading to unacceptable high probability of failure.
- Corrosion of structure leading to damages not experienced within the calculated design of the structure.
- Subsidence or worsening wave climate leading to increased wave-in-deck loading probability.

The ISO/DIS 19902, ISO/DIS 13822 and JCSS procedure further give recommendation for prevention of risk, risk control and avoidance of hazards. For an offshore jacket structure, the principles of risk control and hazard avoiding may be illustrated by the mitigation examples of jacking of subsiding structures and evacuation of personnel prior to a forecasted storm.

Identification of additional hazards and failure modes for ageing structures may be necessary. This is recommended in ISO/DIS 13822 and the JCSS procedure. Examples of identification of hazards and failure modes, and a method of analysing counteracting measures are presented in Chapter 4 of this thesis.

2.4.2 Ultimate limit state check

In terms of structural analysis with respect to the ultimate limit state, the standards presented here require some common types of analysis. Hence the discussion is focussed on the analysis type, rather then the individual standards.

Linear analysis and component checks (Design level check)
The safety of a structure is usually regarded to be implicitly acceptable if a structure designed after a recommended design code. This is based on the fact that most design codes are developed over years, and factors of safety have been set to a reasonable level based on the society's acceptance of risk. Later these load and resistance factors have been calibrated through reliability analyses.

In combination with hazard and failure mode identification followed by an implementation of counteracting measures, linear analysis and component checks according to accepted standards appear to be a sound method of assessing an existing structure. However, it may prove to be un-necessarily rigid and may reject structures that have an acceptable safety against loss of integrity. However, if there is significant uncertainty about the structure and its status, the use of design code standards may be insufficient as they do not take into account the increased uncertainty about the structure by recommending higher safety factors for such situations.

A possible remedy for taking into account changed uncertainty or experienced loading when using linear analysis and component checks may be by adjusting the safety factors when uncertainty is significantly changed. The safety factors are established based on a certain assumed epistemic uncertainty in the design formulas and load calculations. If e.g. the epistemic uncertainty is reduced, the safety factors may be reduced accordingly.

This possible reduction can not be an automatic feature for any existing structure, but must be based on a real documented change in the uncertainty about the structure. Taking full account of this relation between uncertainty and safety factors should also imply increased safety factors in the presence of high uncertainty due to lacking knowledge about structure and loads. A procedure where the scaling of the safety factors represents the actual knowledge about structure and loading needs to be established before this can be used as a general method. Care should be taken where the aleatory uncertainty is dominating, as is the case with wave loading.

Linear analysis and component checks with reduced load level
Standards accepting a reduced load level either accept lowering the historic safety level or believe that an existing structure is more reliable than a structure at design stage. The increased belief in an existing structure must be due to an increased knowledge of the structure. A reduction of the load level is then seen as an alternative remedy for taking into

account the reduced uncertainty when performing detailed analysis of a structure or the structure has been exposed to a load level equal to or more severe than the design load level (see the above discussion). It may seem awkward to reduce the load level, when in fact experience would rather say more about the strength and performance of the structure. As an example, if the structure has experienced a excessive wave loading without any damage, the uncertainty about the structures strength (ability to withstand wave loads) may be updated and reduced (Ersdal et al 2003). Hence, it is in this thesis not recommended to use reduced load level in assessment of structures, but rather evaluate a possible reduced uncertainty in a probabilistic sense and reduce the safety factors on the structural strength accordingly.

The load level may be reduced also due to change of use of the structure, as indicated in ISO/DIS 19902 for unmanned installations. If a different design load level is accepted for unmanned installations compared to a manned installation, a reduction of load level in accordance with a change in manning must be regarded as justified. However, such evaluations are not focussed in this thesis.

Non-linear analysis and component checks

Non-linear analysis and component checks includes what is sometimes called collapse analysis or push-over analysis. In both cases a reserve strength ratio of the structure can be established, see Chapter 3 for a description of the reserve strength ratio. A Reserve Strength Ratio (RSR) analysis as presented in the proposed procedures alone will not cover possible local failure modes (e.g. global loads as earthquake, ice loads and local wave slam loadings and vortex induced vibrations). The RSR in itself does also not give good indications about the safety of the structure if topside loading (variable and permanent) is dominating the design of the structure.

If the main failure mode of the structure is caused by wave-in-deck loading, the RSR still may be an informative description of the strength of the structure, but the RSR is in these situations not a sufficient measure.

There is some variation in the proposed requirements for the acceptance criteria of a structure based on the RSR value. As an example, the API requires RSR>1.6 for high consequence platform, and RSR>0.8 for low consequence platforms, while the requirement of a RSR>1.85 is given in ISO/DIS 19902. There is not a clear reference for the basis of these acceptance criteria. An evaluation of necessary RSR should be developed, and evaluated with respect to the present live- and dead-load on topside, damage and deck clearance.

Identification of good indicators of structural safety on a high level is needed. ISO/DIS 19902 identifies RSR as a sufficient indicator in the assessment using non-linear analysis and component checks. This should be evaluated and if RSR is found sufficient the acceptance criteria for RSR should be evaluated. The influence of wave-in-deck loading, topside loading and damaged strength should be evaluated in addition and in cooperation with the RSR. If needed combined criteria may have to be developed. An assessment based on linear or non-linear redundancy analysis (RSR analysis), if necessary in combination with other indicators of safety, is regarded as a possible sound method provided that a hazard and failure mode identification as mentioned above is performed. Additional local analysis needs to be performed on components for local phenomenon as wave slam and vortex shedding.

Further discussion of the use of non-linear system analysis and other system indicators are found in Chapter 6 of this thesis.

Probabilistic approach.
The main objections towards using structural reliability analysis (SRA) in assessment of structures are that SRA does not calculate a failure probability of the structure that is representing the "true" failure probability and that generally accepted target reliabilities does not exist. An additional objection is that very few engineering offices have the knowledge to do a proper reliability analysis of a structure. Hence, the use of SRA for assessment of existing structures should be well documented by the analyst and exposed to extensive evaluation by other parties (e.g. verification party, governmental organisations).

No acceptance criteria for SRA are given in ISO/DIS 19902, but rather a recommendation to find acceptance criteria in agreement with local regulators. In ISO/DIS 13822 target reliability level used for verification of an existing structure are recommended to be developed based on one or more of the following methods:

- calibration with the current code
- minimum total expected cost,
- comparison with other social risks.

It should here be noted that other social risks would be a measure of historical risk, and as such a "true" risk. If the assigned failure probability and risk of structural failure is supposed to be compared with such a true risk, it seems reasonable that the assigned failure probability of structure is also seen as a "true" risk. A discussion of the existing of such a "true" risk for an uncertain future event is discussed further in Chapter 5.

The problem of gross errors in design, fabrication, installation or operation is not accounted for in any of the procedures discussed in this chapter. This failure mode is difficult to handle in such procedures, but inspection may give some evidence that the gross errors likely to lead to fatigue damage are less likely after some years of operation. Gross errors can be implemented in SRA, and the effect of inspections and experience can be used to update the uncertainty about the presence of gross errors.

The most promising method, in the author's view, is a probabilistic method based on structural reliability analysis. These analyses take into account the actual uncertainty about the structure, and attempt to evaluate the safety of the structural system directly. However, the standards for assessing existing structures based on structural reliability methods are not sufficiently mature at present time. Chapter 5 of this thesis is devoted to further discussion and recommendations for the use of probabilistic analysis in assessment of existing structures.

Assessment by prior exposure
Very few offshore structures have experienced the environmental conditions that the current design codes require, so this method is not relevant in most cases. As an example, if a structure is designed to withstand a loading with an annual probability of exceedance of 10^{-4} (which is the case on the Norwegian Continental Shelf), it is off-course very unlikely that the structure will ever experience this loading in its design life. However, if a probabilistic SRA method is used, an updating of the failure probability or the capacity probability distribution on the basis of the experienced load may be used to take into account the experience. This is further evaluated in Chapter 6 of this thesis and in Ersdal et al (2003).

2.4.3 Fatigue limit state assessment

The fatigue limit state is mentioned as a part of the assessment of structures in ISO/DIS 19902, ISO/DIS 13822 and in the JCSS procedure. The API RP 2A does not explicitly mention the fatigue limit state in the assessment procedure. The ISO/DIS 13822 and the JCSS procedure is focussed on the use of SRA, and the same discussion as mentioned on the ultimate limit state about SRA also applies for the fatigue limit state.

According to the ISO/DIS 19902 an extension of the design service life may be accepted without a full assessment if inspection of the structure shows that time-dependent degradation (i.e. fatigue and corrosion) have not become significant and there have been no changes to the criteria for design. After some years of experience, fatigue cracking may be visible in a component, but taking into account that the structural components in general are designed in order to have 2.5% (approximately) probability of fatigue cracking, it is not likely that one has experienced much cracking in a structure at the end of the calculated design life. If signs of such degradation are evident, it is rather a sign of some type of error in the design or fabrication. And, when entering an extended life, the probability for experiencing degradation will increase. This assumption is evaluated in Chapter 7 of this thesis.

If the calculated fatigue design life is reached and cracks have been identified in inspections and repaired, ISO/DIS 19902 recommends that fracture mechanics is used to identify the time for a crack to develop to a critical and use this time as an inspection interval. It is stated that a conservative method should be used, implying possibly that the initial crack is assumed relatively large, e.g. 1 mm. If a weld is designed for 20 years fatigue life, a crack of 1 mm will develop to a through thickness crack in approximately 4 years. If one would like to identify a possible crack in the formation, rather than after it has developed to a through thickness crack, an inspection interval of approximately 2 years has to be expected. This does not, however, take explicitly into account the possibility of multiple fatigue cracks, but seems like a sufficiently short inspection interval to ensure that the development of multiple fatigue cracks will possibly lead to substantial repair work, but not a failure of the structure. This method can tested by the simulations such as those presented in Chapter 7 of this thesis.

If a member should fail due to e.g. fatigue, corrosion or accidental damage, it is important that the structure does not fail as a system. This can be checked in various ways. As an example a certain damaged strength ratio (DSR) may be required after failure of single components. There is no requirement of the damage capacity of the structure after fatigue damage has occurred in the ISO/DIS 19902 procedure.

2.5 Conclusions

Further investigation into several elements of the existing standards seems to be needed, as discussed in Chapter 1.4. In this thesis the focus is on the following elements:

- The hazards and failure modes that are relevant for an ageing structure subjected to possible life extension should be evaluated. A general evaluation of such hazards and failure modes is performed in Chapter 4.
- The use of probabilistic methods in assessment for life extension. The methodology given in the existing standards seems immature, and decision making based on the probabilistic analysis is not sufficiently clear. ISO 19902 does not offer any advice on decision making. ISO 13822 offers three different options for decision making based on a probabilistic assessment, but these seems to have an underlying understanding of the assigned probabilities as "true" failure probabilities. A further investigation into

the understanding of probabilities and decision making based on probabilistic assessment is given in Chapter 5 and paper A and B of part II of this thesis.

- The use of system strength assessment in assessment for life extension. The recommendations given in the existing standards for system strength assessment is further investigated in Chapter 6. Additional indicators needed for a system strength assessment is evaluated in order to include wave-in-deck problems, damaged strength evaluations and the possibility of introducing new failure modes for larger waves. Also, the acceptance criteria for these indicators are evaluated.
- The degradation of an existing structure due to fatigue is evaluated in Chapter 7 for a damage tolerant structure. An evaluation of the possibility of simultaneous fatigue cracks and its effect on the safety of the structure is evaluated. Also, the effect of inspection and repair on reducing any increase of failure probability is evaluated.

In addition it should be noted that the non-linear analysis and component checks in the reviewed standards does not include local analyses for phenomenon as wave slamming and vortex shedding, nor does it include an evaluation of possible escalating hazards resulting from large deformations due to the excessive loading.

3 Methods for structural reliability and risk analysis

3.1 Structural reliability analysis

Structural reliability analysis (SRA) is used to analyse failure and associated probabilities of load-strength systems. An overview of structural reliability analysis can be found in Ang and Tang (1975, 1984), Toft-Christensen and Baker (1982), Madsen et al (1986), and Melchers (1999). The performance of a component is described by a limit state function *g*. The limit state function is a function of a set of random variables $\mathbf{X} = (X_1, X_2, \ldots, X_n)$ describing the load and capacity of the structural component. Properly formulated the event $g(\mathbf{X}) \leq 0$ defines component failure. The probability of component failure is then given by the probability $P_f = P(g(\mathbf{X} \leq 0)$. The reference period corresponding to the calculated failure probability is defined by the load quantity of the limit state function for structures with time independent strength and where the load is taken as the maximum load in a given reference period. This reference period would typically be one year or the design life of the structure giving an annual failure probability or a life failure probability. In the simplest form two variables would be applied, representing the strength, X_1, of the component and the load, X_2, on the component. The limit state function would then take the form $g(\mathbf{X}) = X_1 - X_2$. The difference $Y = g(\mathbf{X}) = X_1 - X_2$ is called the safety margin of the component.

If $\mathbf{X}$ is described by a joint probability density function f_X, the failure probability of a structural component with respect to a single failure mode can formally be written as shown in Equation 3.1:

$$P_f = \int_{g(\mathbf{x}) \leq 0} f_X(\mathbf{x}) \cdot d\mathbf{x} \tag{3.1}$$

In the two-variable case, the joint probability function and the limit state can be illustrated by Figure 3-1. In essence, the failure probability is the integral of the joint probability function in the failure domain.

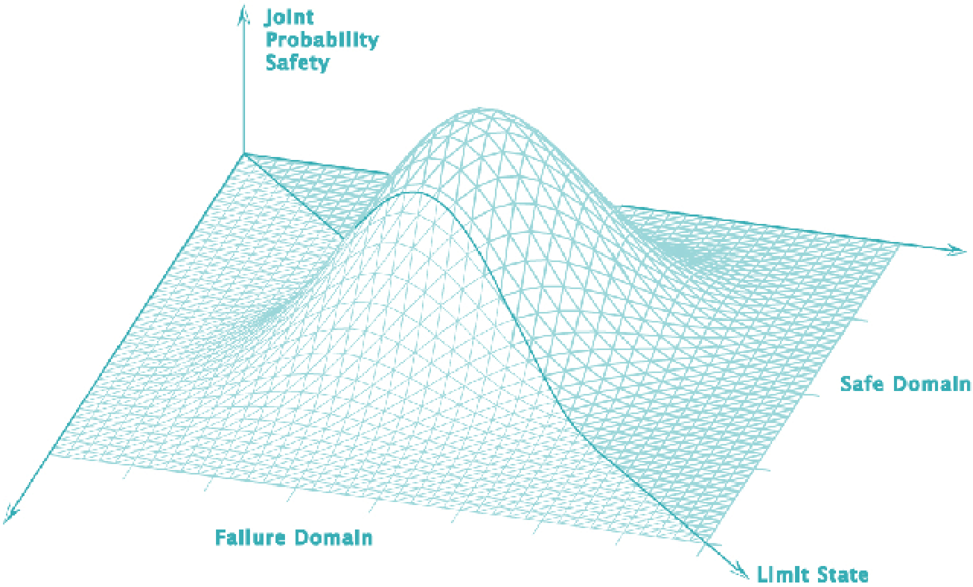

Figure 3-1: Joint probability function and limit state (SAFERELNET 2004)

The integral in Equation 3.1 can generally not be solved analytically. Numerical methods as e.g. Monte Carlo simulations or semi-analytical approximate methods as FORM (First Order Reliability Method) or SORM (Second Order Reliability Method) are normally used. In this thesis, failure probabilities are mostly calculated using Monte Carlo simulations. The basic idea herein is that a large number of simultaneous realizations of the basic variables (**X**), are generated from their probability distributions. The number of samples falling into the failure domain, N_f , is identified. Probability of failure is then estimated based on the total number of realizations and the number of samples falling into the failure domain. For further description of structural reliability methods, reference is given to Ang and Tang (1975, 1984), Toft-Christensen and Baker (1982), Madsen et al (1986), Melchers (1987) and (Bury 1975).

Determination of distributions and parameters from observed data
Establishing adequate probabilistic models for the variables are in most cases the major challenge of the reliability analysis. The probability models and their parameters should be based on a good representation of real data. Determination of the probability distribution is further discussed in e.g. Haldar and Mahadevan (2000).

System analysis
A structural system will normally fail as a result of a chain of failures of different components. The system may furthermore fail due to several different failure modes, e.g. different chains of component failures. A component failure may be defined by a single limit state function, but a structural system will normally need to be described by several limit state functions. The failure modes of a structural system may be described by a fault tree or a

reliability block diagram, but can also be described with the use of structural reliability analysis by the use of series and parallel system formulation (Melchers 1999).

The description of a structural system by use of series and parallel systems is a rather complex approach to system analysis, especially when non-linear behaviour is to be taken into account. An approximate approach is to model the structure directly as a system in a non-linear analysis and evaluate the failure modes directly (see e.g. Skallerud and Amdahl 2002). The system behaviour is then modelled with one resistance (or strength) parameter.

Updating
When new information becomes available the calculations of probability of failure of the structure can be updated. New information that is relevant for the failure probability of a structure may be:

- Inspection events, e.g. measurements of crack lengths or the lack of detected cracks.
- Experienced loading on the structure with survival of the structure.
- Repair or modification events.

Detailed description of updating techniques may be found in e.g. Madsen et al (1986). The general updating procedure for an inequality event is described by:

$$P_f^U = P(g \leq 0 \mid f > 0) = \frac{P(f > 0 | g \leq 0)}{P(f > 0)} \cdot P(g \leq 0) = \frac{P(g \leq 0 \;\mathbf{|}\; f > 0)}{P(f > 0)} \tag{3.2}$$

where g is the failure function for the structure as described earlier, and f is a limit state function describing the occurred event. Similar updating for an equality event may also be described, see e.g. Madsen (1986).

Failure probability may be estimated by Monte Carlo simulations, and the updated failure probability is given by the number of simulations satisfying *g£0* and *f>0*, divided by the number of simulations satisfying *f>0.* The probability *P(f>0)* illustrates the probability that the event should occur (e.g. surviving a wave of a given height or having no cracks after a given number of years).

Alternatively, the probability distribution functions for the parameters in the probabilistic description may be updated as a result of the event, see e.g. Ang and Tang (1975, 1984).

3.2 Risk analysis

A risk analysis is a collection of several activities performed to provide support for decision making. Typically a quantitative risk analysis would include:
- System identification and description
- Hazard identification
- Consequence identification
- Probability analysis (probability of hazards, probability of consequences based on model)
- Description of the risk

A qualitative risk analysis would include the same elements, except that it would not include the quantitative probability analysis.

The structural reliability analysis could serve as the method for assigning probabilities. However, within risk analysis the most commonly used methods are *fault trees* and *event tree* analysis. To some extent Bayesian Probabilistic Network (Jensen 1996) has also been used for the probabilistic analysis. When structural reliability analysis have been used, it has not been common to include accidental hazards as accidental loads and gross errors in the analysis. The accidental hazards for a structure are traditionally only included in the risk analysis, and not in the structural reliability analysis. Structural reliability analysis is often used to model the distribution of physical parameters, e.g. strength and load parameters, and determine the probability of load exceeding strength. In this thesis, accidental hazards have also been included in the structural reliability analysis.

In this work, structural reliability analysis and Bayesian probabilistic networks have been evaluated as means to assigning the probabilities for the risk analysis. In addition, a qualitative approach using Barrier analysis has been applied.

3.2.1 Bayesian Probabilistic Networks

Bayesian probabilistic networks (BPNs) are a method for calculating the probability of events that can be described by a causal network. A causal network consists of a set of variables (nodes) and a set of directed links between the variables (conditionals). In a causal network, if there is a link from the event A to the event B, it means that A has an influence on B to occur (or causing B to occur). The wording of family relations is used, and in this case B is called a child of the parent A. Bayesian probabilistic networks have a graphical interface, which make them intuitive and relatively easy to communicate. Since the model building focuses on causal relationships between the variables, it thus reveals the analyst's intuitive and analytical understanding of the problem. In addition to the causal network, the Bayesian Probabilistic Network includes a probabilistic model for analyzing the probabilities of any of the events in the network. The calculation of probabilities is in principle based on Bayes' rule $P(B|A) = \dfrac{P(A|B) \cdot P(B)}{P(A)}$. A thorough description of Bayesian probabilistic networks is given in Jensen (1996).

A Bayesian Probabilistic Network consists of (Jensen 1996):

- A set of variables and a set of directed edges between variables.
- Each variable has a finite set of mutually exclusive states.
- The variables together with the directed edges form a directed acyclic graph (no feedback cycling so that A becomes dependent on A).
- To each variable A with parents $B_1,\ldots, B_n$ there is attached a conditional probability table $P(A|B_1,\ldots,B_n)$.

For each of the nodes (uncertain events) in the BPN a matrix has to be defined giving the possible (discrete) states together with their corresponding probabilities. If a node is dependent on the state of other nodes i.e. if arrows are pointing into it, a matrix of conditional probabilities has to be defined.

Figure 3-2 shows an example of a Bayesian Probabilistic Network, fulfilling the requirement of directed acyclic graph. The probabilities to specify are: P(A), P(B), P(C|A,B), P(E|C), P(D|C), P(F|E) and P(G|D,E,F). With these probabilities (probability tables) defined, the system can be solved for all nodes of the system. The resulting probability would in this example be the probability of the event G, which may be compared to the top event in a fault tree.

The product of a BPN is the probabilities of the modelled events of occurring, similar to the product of a fault tree. A clear difference from a fault tree analysis is that a link between any starting events can easily be established. As an example, the start event A in Figure 3-2 may, however, be influenced by the start event B. This can easily be modelled by adding a link from B to A and establishing the probability table for P(A|B). The events may also have several states, not limited to two states (working or not working).

A node in a Bayesian Probabilistic Network may act as an “AND” gate or an “OR” gate as used in a fault tree. Hence, the BPN can be used instead of an event tree or fault tree if desired.

The use of Bayesian Probabilistic Networks in marine and offshore engineering has been studied and exemplified by Friis-Hansen (2000). In Friis-Hansen (2000) it is pointed out that the use of Bayesian Probabilistic Networks has a number of advantages over fault trees and event trees. Among these are the difficulty of accounting for dependencies between causes of a given event when using fault trees and the possibility of using noisy gates. A noisy gate is possible in a BPN, in contrast to fault trees where the logical gates are limited to AND gates and OR gates with ones and zeros. A noisy gate is a logical gate where probabilities are used to describe the possibility that the output is different from the logical output given the inputs. The disadvantage of BPN is that all variables are discrete. Hence, it will in some cases be difficult to calculate the failure probability and to include new information in the calculations.

A full risk analysis of a jacket structure with the use of Bayesian Probabilistic Network has been attempted in this work. However, the approach was not very adequate for the structural life extension problem. Due to this, BPN’s have only been used to a limited degree herein.

3.3 Barriers and barrier analysis

The concept of barriers has its basis in Haddon’s energy model, see Figure 3-3, and is originally two of Haddon’s ten strategies to prevent hazards (Haddon 1980). The two barrier strategies are to separate the hazardous energy and the vulnerable target by a physical barrier or by an administrative or physical barrier separating the hazardous energy and the vulnerable target in time or space. The barrier concept is discussed in Sklet and Hauge (2002), and the following discussion is to some extent based on their report. In addition, a review of the use of the barrier concept in Haddon (1980), Reason (1997), and International Atomic Energy Agency (1996) is included.

The energy model, see Figure 3-3, represents an effort to systemise the analysis of accident causes in a way similar to that of analysing the cause of diseases (Kjellén, 2000). The hazard is energy exchange, which is mechanical, chemical, thermal, electrical, etc. The energy flow can be wanted (the flow of hydrocarbons in a pipe), or of an unwanted type. Of concern here are the phenomena that involve the transfer of energy of such amounts, and at such rates of change that people could be injured or objects could be damaged.

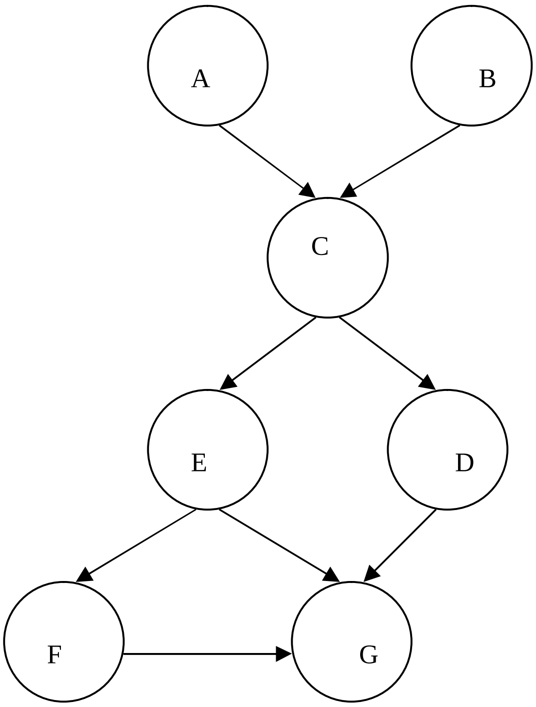

Figure 3-2: Bayesian Probabilistic Network example (Jensen 1996)

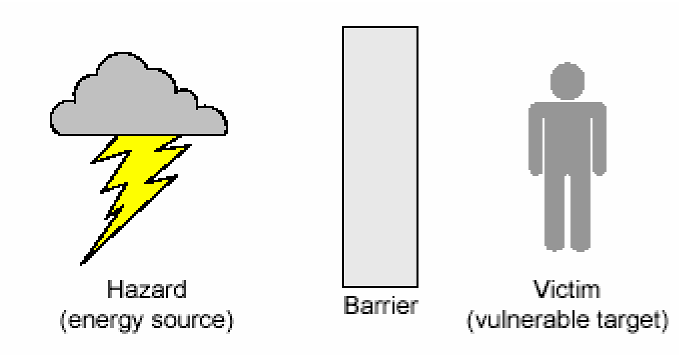

Figure 3-3: The energy model (Kjellén, 2000)

External energy sources are the energies acting on the vulnerable target from the surroundings, e.g. wave loading and human actions leading to e.g. gross errors. Internal sources are the energies in the vulnerable target that is acting on itself. The weight of the structure is such an internal energy. Based on the hazardous energies, the important strategies to secure the installation can be identified. A formal method of identifying these strategies is

Haddon's ten preventive strategies as listed in Table 3-1 (Haddon, 1980). The strategies are directed towards reducing the hazard (e.g. fatigue failure in structures), towards physical barriers (e.g. sufficient strength in structure with one brace missing) and towards protecting the vulnerable target (e.g. evacuation equipment).

Table 3-1: Haddon's ten preventive strategies (Kjellén, 2000)

Hazard (energy source)	Barriers	Vulnerable target
Strategies related towards the hazard:	Strategies related to barriers between hazard and target:	Strategies related to the vulnerable target:
1. Prevent build-up of energy 2. Modify the qualities of the energy 3. Limit the amount of energy 4. Prevent uncontrolled release of energy 5. Modify rate and distribution of energy	6. Separate in time or space the source and the vulnerable target 7. Separate energy source and the vulnerable target by physical barriers	8. Make the vulnerable target more resistant to damage from the energy flow 9. Limit the development of loss (injury or damage) 10. Stabilise, repair and rehabilitate the object of the damage.

Barriers are also mentioned as one of the functions for the defences in Reason's "defence in depth" (Reason 1997). Reason's defences are designed to serve one or more of the following functions:

- to create understanding and awareness of the local hazard
- to give clear guidance on how to operate safely
- to provide alarms and warnings when danger is imminent
- to restore the system to a safe state in an off-normal situation
- to interpose safety barriers between the hazards and the potential losses
- to contain and eliminate the hazards should they escape this barrier
- to provide the means of escape and rescue should hazards containment fail.

The term barriers are often used for the slices of Swiss cheese when presenting Reason's Swiss cheese model as shown in Figure 3-4. However, Reason (1997) is using the term "Defences" or "Defences, barriers and safeguards" for these cheese slices, indicating that the barriers are one of the defences as shown above.

Limiting the concept of barriers to the definition by Haddon has the benefit that it limits the use to the defences that do prevent the vulnerable target from being exposed to the hazardous energy in the accident event. An approach to barrier analysis and thinking is presented in Vinnem et al (2003) and by the Norwegian initiative "Working together for Safety" (Samarbeid for Sikkerhet 2004). This includes using the definition barrier function, barrier system (possibly consisting of several elements) and performance influencing factors. The barrier function is the function the barrier is intended to perform to prevent the realisation of a hazard or limit the damage of the hazard by stopping the chain of events. The barrier system is the technological, human or organisational system that is ensuring that the barrier function is fulfilled. A performance influencing factor is any condition that is influencing the performance of the barrier system.

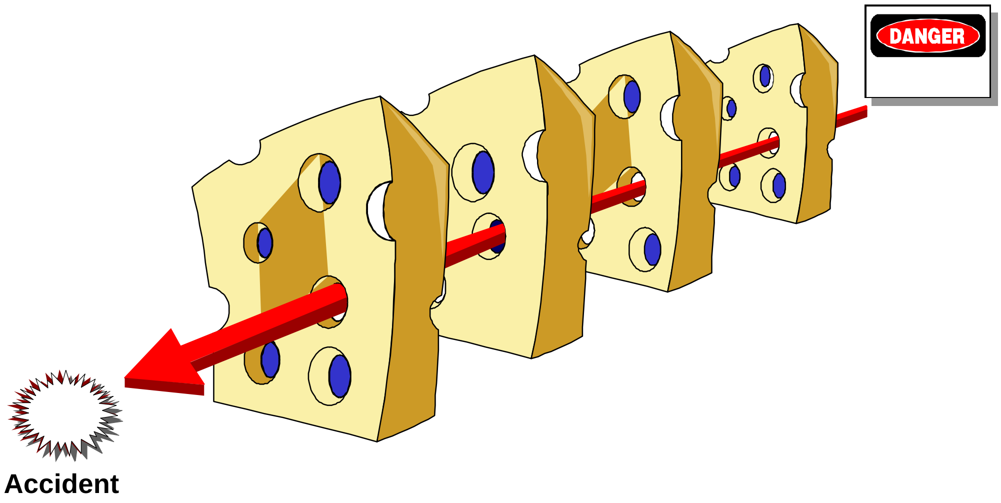

Figure 3-4: Swiss cheese model (Reason 1997)

For an offshore jacket structure, the barrier function can be exemplified with “prevention of overload of the structure”. The barrier system would be the actual strength of the structure. The performance influencing factors may be the standards used in design, the quality assurance during design and fabrication, the degradation during operation and the maintenance of the structure during operation.

The approach presented by Vinnem et al. (2003) give emphasis on the preventive actions that really do influence the chain-of-events in an action, but also take into account that influencing factors like regulations, standards, quality assurance and safety culture does play an important role in the barriers performance. The barrier analysis in this thesis will be based on this approach.

Barrier analysis
The first step in the process is to identify a possible set of hazardous events and failure modes for a structure during operation. These events and failure modes should typically cover the known incidents and accidents. Reasonable imaginary scenarios should also be included. A possible way forward in this regard is to perform a simplified accident investigation for the known incidents and accidents. It should be ensured that all known accident scenarios are illustrated as hazard scenarios. These are usually the basis for the standardised requirements. The root-causes for potential structural and marine failures should be identified.

In order to identify the defences / barriers (hazard reduction strategies, physical barriers and vulnerable target protection strategies), the different identified failure paths are analysed using Haddon’s ten preventive strategies for reducing damage from hazards. For a general case, the barriers / defences should be barriers identified in the standards and regulations, but for individual installations specific barriers may be identified. As an example, a wave breaking walls around an installation or jacking of the topside of a jacket may be regarded as a defence / barrier, but is not required in any regulation.

The barrier performances are measured with respect to their functionality and effectiveness, their reliability and availability and their robustness (see e.g. PSA 2004). As the barriers related to structures are generally passive and physical, their robustness is generally the important measure. If maintenance and repair is viewed as a barrier, the reliability of e.g. inspection methods may be an important part of the barrier performance.

In some instances, the effectiveness of barriers can be directly measured by rather simple parameters. As an example the structural strength against wave overload may be measured by one or a few simple parameters describing the structural robustness. In other cases, a breakdown of these barriers into measurable parameters is needed. As an example the structural integrity management system including maintenance and repair may be a potential barrier, but it includes a lot of elements that have to be evaluated. Important elements in the structural integrity management system may be the maintenance philosophy, the quality of inspection planning, the execution of inspections, the handling of inspection results, the implementation of repair work etc. An analysis of the performance influencing factors, as introduced in Vinnem et al (2003), is an approach to identify measurable parameters that may be indicators of the barrier effectiveness.

With the use of numerical indicators for the barrier performance, the risk analysis by barrier analysis may be regarded as semi-quantitative. Acceptance criteria for these indicators can be developed, and the indicators can be measured according to the acceptance criteria. There are a set of formal requirements that indicators should satisfy, in addition to the requirements relating to offshore petroleum HES management as outlined above. The indicators should satisfy the following formal requirements (Kjellén, 2000):

- observable and quantifiable
- sensitive to change
- transparent and easily understood
- robust against manipulation
- valid

3.4 Bayesian Decision Analysis

Bayesian decision analysis is described in Benjamin and Cornell (1970), Føllesdal et al. (1984) and Faber (2003). This presentation of Bayesian Decision Analysis is based on these references.

The development of decision analysis was initiated by Thomas Bayes (1706-1763) and Daniel Bernoulli (1700-1782). At the decision point, it is assumed that we can chose between actions A_1, A_2, …, A_n. Each action may have a set of possible consequences C_1, C_2, …, C_n. The value of each consequence is measured by some sort of value v_1, v_2, …, v_n. Given the choice of action A_i, each consequence has a probability of p_{i1}, p_{i2}, …, p_{in} of occurring. This may be illustrated in a decision tree as shown in Figure 3-5.

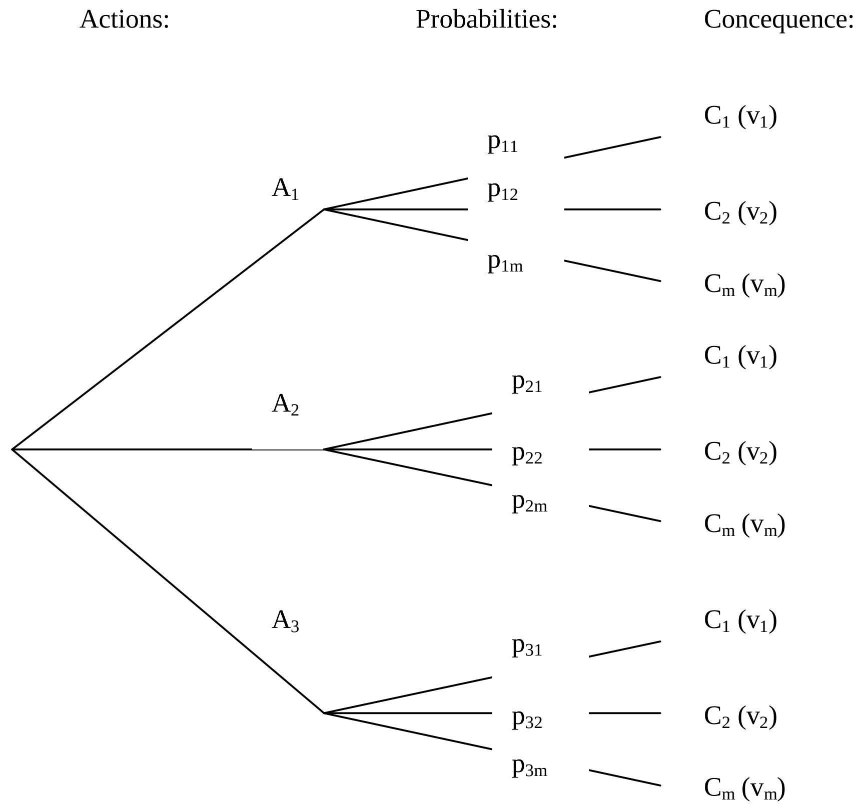

Figure 3-5: Decision tree (Føllesdal et al 1984)

The decision analysis is now to evaluate which of the actions A_1, A_2, …, A_n that should be taken. The utility of action A_i can be calculated as:

$$U(A_i) = \sum_{j=1}^{m} p_{ij} \cdot v_j \tag{3.3}$$

The action that results in the highest utility should then be chosen. This fundament of decision analysis is used in cost benefit analysis and in multi attribute analysis, as described in the following section.

3.5 Cost benefit analysis

Cost benefit analysis (CBA) is used to establish an evaluation of the total costs and the total expected benefits of an action. Costs and benefits related to an action are hereafter called consequence of the action, in line with the presentation of Bayesian Decision Analysis. The purpose of the evaluation is to establish the economically optimal action (decision). Normally, the consequences are expressed in monetary values, and the net present values of the future consequences are used. Net present value is a form of calculating the discounted cash flow, which means that the future consequences are discounted by an interest rate.

The net present value (NPV) of a series of future consequences, C_t, with value v_t, can be written as:

$$NPV = \sum_{t=0}^{N} \frac{v_t}{(1+r)^t} \tag{3.4}$$

where t is the year the consequence C_t is to be realised, and r is the interest rate.

A CBA evaluates the net present value of all consequences of an action. In an economical analysis, the action would be preferable if the net present value of the consequences is more than zero. If the choice is between different actions, the action giving the highest net present value would be preferable.

In cost benefit analysis under uncertainty, one or more consequences are subjected to a probability of being realised. This may be dealt with by assigning probabilities to alternative outcomes and then using these probabilities to calculate the expected outcome in line with Bayesian Decision Analysis. The value of each outcome is multiplied by its probability and the expected value is calculated.

$$E[U] = \sum_{i=1}^{n} v_i \cdot P_i \tag{3.5}$$

where v_i is cost or income and P_i is the probability of realisation of cost or income v_i.

For a jacket structure the expected life cycle benefit $E[B]$ may be written as:

$$E[B] = E[I] - E[C_{FD}] - E[C_F] \tag{3.6}$$

where $E[I]$ is the net present value of expected income[4], $E[C_{FD}]$ is the net present value of expected field development costs and $E[C_F]$ is the net present value of expected failure costs. The expected value operations are performed in regard to the uncertainties associated with the loading and the capacity of the structure.

The expected net present values may also be written on an integral form. The incomes and costs are then discounted by a discounting function $\delta(t)$.

$$\delta(t) = e^{-\gamma \cdot t} \tag{3.7}$$

Here, $\gamma = \ln(1+r)$, r denotes the annual interest rate and t is the time at which the consequence (income or cost) occurs.

The expected income is discounted by the discounting function, and taking into account the income reliability:

[4] Income in this context means the income taking into account the operational costs (including maintenance of the structure).

$$E[I] = \int_0^T i(t) \cdot \delta(t) \cdot R_i(t) dt \tag{3.8}$$

where T is the design lifetime, $i(t)$ the income function, $\delta(t)$ the discounting function, $R_i(t)$ the income reliability function. The income reliability function expresses the probability that at time t the income is obtained.

Hence, $E[B]$ can be written as:

$$E[B] = \int_0^T i(t) \cdot \delta(t) \cdot R_i(t) dt - C_{FD} - \int_0^T C_F \cdot \delta(t) \cdot g_n(t) \cdot dt \tag{3.9}$$

where C_{FD} are the field development cost, C_F the failure costs and $g_n(t)$ is the probability density function of the time to failure. The field development costs are expected to be realised immediately, and is not discounted.

The assumption that failures occur as realisations of a stationary Poisson process and that time intervals between failure events are independent and exponentially distributed allows for an analytical evaluation of Equation 3.9. Based on this assumption, only one failure event may occur in a sufficiently small time interval. Therefore, the probability of the union is simply the addition of the probabilities of the individual events. The income reliability function may be derived as shown in Rackwitz (2001):

$$R_i(t) = e^{-\lambda \cdot t} \tag{3.10}$$

By means of the annual probability of failure, the annual failure rate, λ, may be written as:

$$\lambda = \ln\left(\frac{1}{1 - P_f}\right) \tag{3.11}$$

where P_f is the annual probability of failure.

The main principle of a traditional CBA is to transform all values into monetary values. The transformation of environmental values and safety values into monetary values are however not straight forward, and the attempts to define a method of monetary value for the potential loss of a statistical life is not generally accepted. A possible method to account for possible fatalities was developed by Nathwani et al. (1997) as the Life Quality Index (LQI). From this index, Skjong and Ronold (1998) derived the amount of money, which should be invested to avert a fatality (*ICAF*).

In Abrahamsen et al. (2005) a pragmatic CBA is developed. In the pragmatic CBA non-marked goods as personnel safety and environmental issues are excluded from the analysis. There is also no search for the correct objective values. Instead the sensitivity of the conclusions of the analysis is demonstrated by presenting the results of the analysis as a function of the consequences. Thus a pragmatic CBA provides decision support rather than hard recommendations.

The CBA method will be used in the thesis in decision making based on probabilistic analysis. The use of CBA as a decision method or as an input to the decision making process will also be discussed in this thesis.

3.6 Multi attribute analysis

A multi attribute analysis is a decision support method evaluating the consequences with respect to different attributes (safety of personnel, safety of environment, economy). In contrast to cost benefit analysis, there is no attempt to transform all the different consequences into monetary or other comparable units. The utility of each action is evaluated on different attributes, and the decision is made on the evaluation of the utility of all the attributes.

Multi attribute value function is shown in Equation 3.12 (Bedford and Cooke 2001 p 271):

$$v(x_1, x_2, x_3 \ldots x_r) = \sum_{i=1}^{r} w_i \cdot v_i(x_i) \tag{3.12}$$

where w_i is a weighting factor of the *i*th attribute, x_i is the *i*th attribute and $v_i(x_i)$ are the marginal value functions (Bedford and Cooke 2001 p 271). Following multi attribute theory, all factors (objectives and attributes) should be included in the analysis to achieve a complete set of objectives.

Aven’s approach to decision making based on multi attribute analysis, is however more pragmatic. Decision analysis is by Aven viewed only as an aid for decision making (Aven 2003 p 125). The purpose of a risk analysis is to support decision-making, and one should distinguish between the decision support (risk analysis, cost benefit analysis, multi attribute analysis) and the decision making itself as shown in Figure 3-6.

A more detailed description and discussion of the decision methods (Cost Benefit Analysis and Multi Attribute Analysis) is found in Section 5.

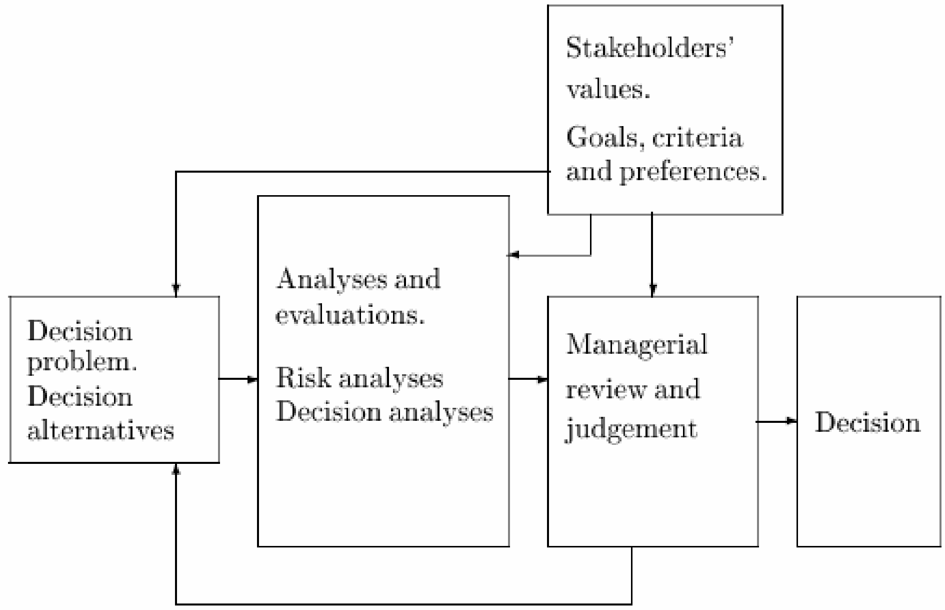

Figure 3-6: Basic structure of the decision making process (Aven 2003)

3.7 Non-linear structural analysis

Theories for non-linear collapse analysis (push-over analysis) have been established for jacket structures by Søreide and Amdahl (1986). The approach was further developed by Hellan (1995) and Skallerud and Amdahl (2002). A guideline for performing non-linear collapse analysis is given in the Ultiguide project (DNV 1999).

The theory behind the non-linear analysis of structures will not be covered here, the reader is referred to e.g. Skallerud and Amdahl (2002). However, some background information about the definitions often used in the non-linear collapse analysis is covered.

In a conventional elastic linear analysis of structures, the structure is normally checked against *first yield* in any member of the structure for loading according to the Ultimate Limit State (ULS). However, the structure will undergo local yielding for loading less than the ULS condition due to the presence of residual stresses (Søreide 1981). The ductility of the steel makes redistribution of stresses possible and ensures that the structure can safely experience some yielding. The idea behind the non-linear collapse analysis is to make the criteria for maximum load more realistic in the sense that it better simulates the real behaviour of the structure during collapse (Søreide 1981). Instead of operating with allowable stresses as in the linear elastic design, the safety requirement is incorporated as ratio between the design load and the collapse capacity of the structure with the given loading distribution. Both the linear elastic analysis and design and the non-linear analysis and design follow the lower bound theorem in theory of plasticity as the normal design principles for structures. Hence, both should be able to estimate a lower estimate of the structural strength.

The fundamental requirement in structural analysis is that the calculations should be on the conservative side. According to the lower bound theorem of plasticity, an external load in equilibrium with internal stresses, which do not exceed the acceptable plastic stresses, is less or equal to the collapse load, if ductility is acceptable (Chakrabarty 1987). Normally this is checked by using a linear elastic analysis, giving statically admissible forces, followed by a code check of stresses according to accepted standards, and the use of ductile material. As this is according to the lower bound theorem, it will in itself include a certain degree of safety towards collapse. Methods typically used for collapse analysis includes geometric stiffness and material non-linearities. Hence, the methods will also give a solution according to the lower bound theorem. This solution will in most cases be less conservative than the linear elastic solution, and hence closer to the theoretical collapse capacity.

The ratio between the design loads and the collapse capacity of the structure is established in the collapse analysis and is normally called the reserve strength ratio (RSR). An illustrative Q-δ curve (load versus deflection curve) is shown in Figure 3-7 for both an intact structure and a damaged structure. In addition to the RSR, a similar parameter giving the ratio between the collapse capacity of the damaged structure and the design load is used and often called damaged strength ratio (DSR). With reference to the load levels in Figure 3-7, the RSR and DSR can be defined as:

$$RSR = \frac{Q_u}{Q_d}, \; DSR = \frac{Q_r}{Q_d} \tag{3.13}$$

Where Q_u is the ultimate collapse capacity of the structure, Q_d is the design load for the structure and Q_r is the ultimate collapse capacity of the structure in damaged condition.

The reference level for the design load used in the RSR and DSR calculation would typically be the wave, current and wind loading with an annual probability of exceedance of 10^{-2}. In the RSR and DSR calculation the design load does not include any load factor.

An additional factor for illustrating the loss in strength when a component is damaged is given by the residual strength factor (RIF). The residual strength factor (RIF) is defined as:

$$RIF = \frac{Q_r}{Q_u} \tag{3.14}$$

Hence, the RIF is equal to $\frac{DSR}{RSR}$, and is a measure of the effect on the RSR when a member is damaged or lost.

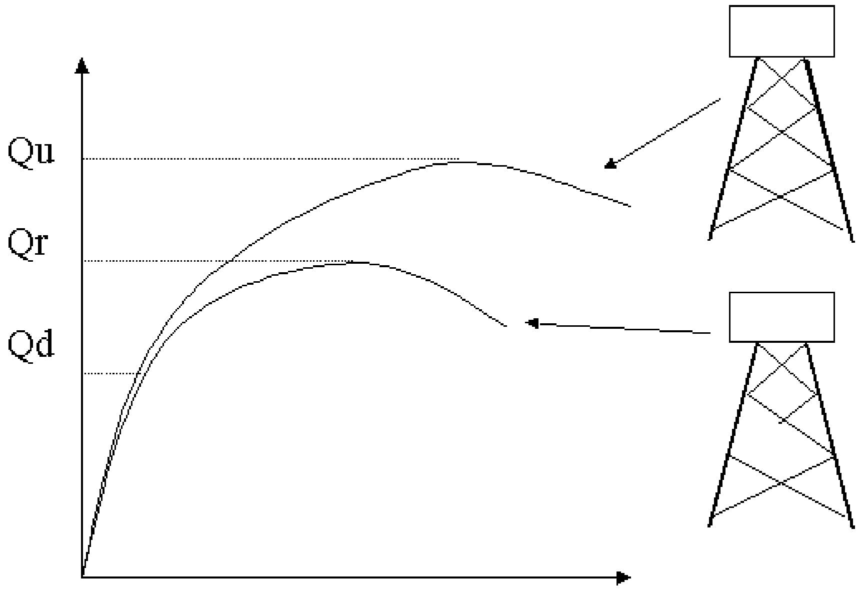

Figure 3-7: Illustrative Q-d curve for a jacket structure with indication of the design load level and collapse load level in intact and damaged situation, where Q along the vertical axis is load and d along the horizontal axis is deformation.

4 Failure modes of jacket structures

4.1 Hazard identification

The first step of a risk evaluation of a technical system is to identify the possible hazards that can threaten the integrity of the technical system. Important sources for information on hazards for offshore jacket installations are previous work on the subject (e.g. HSE 1999a and HSE 2002), experience from historic accident (e.g. from the WOAD database) and the design standards used for designing this type of structure (e.g. ISO 2004 and NORSOK 1998).

Hazards that pose a threat to the structural integrity of an offshore jacket structure, or other types of structures, can be broadly grouped into three types of sources (HSE 2002). The first hazard group is inadequate safety margin, meaning that the load exceeds the capacity of the structure. This may occur by excessive loading or insufficient strength, possibly due to degradation of the structure. The second hazard group is accidental events, which includes ship collisions, dropped objects, fire and blast. The third hazard group is often called gross errors. Gross errors are human and organisational errors that occur in design, fabrication, installation and operation. This classification of the hazards will be the basis for the following discussion.

Hazards recognised in design standards

Offshore jacket structures used in the petroleum activity are normally designed for the various phases of the life. These phases include construction, float out from construction site to transport barge, transport to field, lifting from barge to site, operation on site and, finally, removal. The design codes normally include three limit states. These limit states are ultimate limit state (ULS), fatigue limit state (FLS) and accidental limit state (ALS). In an assessment of an existing structure as discussed in this thesis, the jacket would be situated on site. Hence, the focus is on the operational phase.

Other possible hazards for a jacket structure mentioned in design codes and standards (e.g. ISO, 2004) are subsidence of sea bottom, worsening of wave climate and unexpected large marine growth.

Hazards found as root causes in historic accidents

Most of the known offshore jacket failures have occurred in the Gulf of Mexico (GOM) during hurricanes. The most severe hurricane, with respect to number of damages, is Andrew that moved through the GOM on August 24-26 in 1992 (Botelho et al. 1994). According to the WOAD (DNV Technica 1995) a total of twelve jacket type structures suffered "severe damage". Further, according to Botelho et al. (1994) ten major platforms were completely toppled and twenty-six were leaning significantly. The collapsed configuration of one of the toppled jackets is described by MMS (1994), and this structure is also inspected and analysed to find the cause of the collapse. It is reported that the inspection revealed clear evidence of bending failure of the legs in the second bay above seafloor. The leg-pile annulus was supposed to be fully grouted. However, there appeared to be no grout in the vicinity of the failures. Another platform experienced a loss of the topside, with no major damages to the jacket (DNV Technica 1995). A substantial contributor was that the deck was never fully welded to the jacket at the connection point and the deck appears to have been pushed off the top of the jacket. These examples appear to be typical for the main learning from historic accidents: failures are mainly caused by gross errors. It should be noted that very few installations have experienced loads near the ultimate limit state design loads. Hence, it is also reasonable that the failures are due to a gross error rather than the structures being exposed to design loads.

The typical gross errors found in these evaluations are introduced in the various phases of the jackets life (Moan, 1983):

- Design:
    - Important loads or load components neglected (e.g. wave in deck)
    - Failure modes neglected
    - Design of component thickness is insufficient or neglected
    - Lacking or incomplete specification of materials, fabrication procedures and operational procedures.
- Fabrication:
    - Fabrication tolerances superseded.
    - Deviations from material specifications
    - Insufficient fabrication inspection
- Operation
    - Operational errors (e.g. collisions, dropped objects, fires and explosions)
    - Insufficient maintenance (e.g. corrosion, fatigue)
    - Insufficient inspections

Gross errors would typically result in insufficient capacity of a member, or an unexpected fatigue growth.

Hazard identification in previous work

Extensive work on hazard identification is performed by the Health and Safety Executive in UK (e.g. HSE 1999a and HSE 2002). In HSE (1999a) a useful scheme for looking at hazards is presented, relating structural failure to the underlying cause insufficient strength or excessive load.

In addition to these hazards, HSE (2002) includes geological / geotechnical hazards, widespread fatigue and hazards due to changes in the use of the installation. Geological / geotechnical hazards include overloading of pile in tension or compression, degradation of pile due to cyclic loading, differential settlement, seabed scour, subsidence and slope instability. Widespread fatigue is occurrence of multiple cracking in the structure (HSE 2002), which may occur towards the end of a life of a jacket structure. This may occur as the probability of cracking in each weld of the structure increases, thereby increasing probability of simultaneous cracks. Widespread fatigue may also be influenced by load redistribution after the failure of one component. Hazards with respect to change of use are related to weight on topside and the increased probability of fire and explosion when new process equipment is installed. A summary of the hazards is given in Table 4-1.

Table 4-1: Hazards for an offshore jacket structure.

Underlying cause	Source of hazard	Specific hazard
Insufficient strength	Gross error in design, fabrication, installation or operation	Insufficient design capacity Fabrication error Operational damage Modifications
	Degradation	Subsidence Corrosion Fatigue due to: - global cyclic loading - local cyclic loading - vortex induced vibrations - wave slam Widespread fatigue Scour Differential settlement
Excessive load	Environment	Global overload due to: - wave and current load - wave in deck load - wind load - unexpected marine growth - ice and snow loads - earthquake loads Local component overload due to: - wave and current load - wave in deck load - wave slam - vortex induced vibrations - wind load - unexpected marine growth - ice and snow loads - earthquake loads Worsening of wave climate
	Operation	Deck load – weight increase Unsecured objects – centre of gravity shift
	Accidental loads	Dropped objects Ship impact Explosion Fire & heat Aircraft impact Iceberg impact Submarine slide / Seabed slope instability

4.2 Special considerations with respect to life extensions

The relevant hazards with respect to life extension of existing structures should account for hazards generally identified for offshore jacket structures. The possibility for additional hazards for ageing structures should be evaluated. The hazards have to be evaluated with respect to their relevance in a life extension study. In **Table 4-2**, a review of identified hazards and their relevance for a life extension study is presented.

Based on the evaluation shown in **Table 4-2**, the focus of life extension assessments should be on degradation of the structure due to subsidence (wave in deck problems), fatigue, corrosion, geotechnical / geological degradations and possible worsening of wave climate and the accidental load probability. The structure will normally not fail as a result of degradation alone, but a degraded structure exposed to a large wave and current loading may be a critical scenario, as discussed in next section.

Possible new hazards relevant for life extension

The hazards mentioned so far have mainly been applicable for an offshore jacket structure in its design life. In addition to these, an evaluation of possible new hazards that could occur for ageing structures is important to assess. In this section a generic evaluation of hazards will be presented. For a specific jacket structure, more specific hazards may be developed.

The experience of ageing structures in areas like the North Sea and other areas of the Norwegian Continental Shelf is sparse. Surveys performed by HSE (HSE 2002, HSE 2003), suggest that experience from other industries points out collapse of corroded structure and collapse of structure exposed to wide spread fatigue also important for offshore jacket structures.

Possible hazards may alternatively be found by methods like HAZID / HAZOP (Aven 1992). Based on such methods the following possible hazards have been identified:

- Fatigue cracking continues to develop at same spot and has been repaired several times. This will give insufficient material quality in the area if welding is used for repair.
- Degradation that occurs in many places simultaneously (wide spread fatigue).
- Accelerated fatigue in surrounding joints after a fatigue failure of a component.
- Micro-cracks in material that develop into fatigue cracks. There is a high number of micro-cracks in the material. Especially in old structures.
- Insufficient inspection and maintenance.
- Corrosion protection stops working, e.g. anodes are spent and not replaced.
- Hydrogen penetration in steel due to corrosion protection leads to hardening of material.
- A plastic deformation leads to hardening of material.
- Marine growth increases.
- Structure is designed according to old outdated standards for strength, or to outdated environmental criteria.
- Insufficient damaged strength after component failure. A component failure will be more likely in a life extension. Damage tolerance for a single failure is an important counteracting measure to ensure the safety of the installation if such a failure should occur.
- Subsidence leading to increased probability of wave in deck loading.
- Worsening of wave climate.

Table 4-2: Relevance of hazards in life extension.

Source of hazard	Specific hazard	Relevance for life extension
Gross error in design, fabrication, installation or operation	Insufficient design capacity Underestimating the wave climate Fabrication error	Fatigue related errors: Less likely than in design life Underestimation of loads: Same as in design life. Strength related error: Same as in design life.[5]
	Operational damage	Same as in design life of the structure.
	Modifications	Same as in design life of the structure, but depending on the number of modifications.
Degradation	Subsidence – increased wave in deck probability. Corrosion Fatigue due to: - global cyclic loading - local cyclic loading - vortex induced vibrations - wave slam Widespread fatigue Scour Differential settlement	Degradations develop with time and will in general be worse in life extension compared to design life. Widespread fatigue is a hazard that is relevant for life extensions, but normally not evaluated for structures in their design life.
Environment	Worsening of wave climate Extensive marine growth	Worsening of wave climate and / or extensive marine growth may increase the loading on the structure in life extension compared to design life. If wave climate and marine growth is stable, the loads will be same as in design.
Operation	Deck load – weight increase Unsecured objects – centre of gravity shift	Same as in design life.
Accidental loads	Dropped objects Ship impact Explosion Fire & heat Aircraft impact Iceberg impact Submarine slide / Seabed slope instability	Subjected to changes both in design life and during extended life. Ship traffic may increase, topside activity may increase the probability of dropped objects, fire, and explosion. Finally, climatic changes may alter the probability of iceberg impact and submarine slides.

[5] A structure would by this stage have experienced some cyclic loading, and a gross error that could result in fatigue crack development should by this stage have been exposed. If a crack due to a gross error has not developed during design life, the probability of the existence of such error is reduced. However, the crack may have remained undetected, but if a thorough inspection is performed prior to the life extension evaluation the possibility of gross errors leading to fatigue cracks should be significantly reduced.
Gross errors limiting the global resistance of the structure would not be similarly reduced, as very few installations have experienced loads in the vicinity of the design loads.

It is in this thesis chosen to focus on the wave and current loading on the structure on the excessive load side. The insufficient strength side of the problem is covered by subsidence, gross errors, degradation due to cyclic wave loading, widespread fatigue and corrosion. This limits the problem, and the safety towards wave and current loading is also viewed as representative for the safety against other loadings that may occur (e.g. earthquake).

The focus of the remaining evaluation is on the jacket structure itself, excluding piles, topside structure and appurtenances. However, the same principles should also be applicable for other structural parts, e.g. the piles. Hence, the focus will be on the following hazards:

- excessive wave and current loading
- excessive topside loading
- subsidence, wave climate and possible wave in deck
- degradation of structure due to cyclic wave loading leading to fatigue cracking and possible widespread fatigue
- gross errors leading to fatigue or reduced global capacity of structure.

4.3 Failure modes

Each hazard may lead to various unfortunate consequences following a sequence that may be called a failure mode. As an example, a failure mode may start with an unexpected large wave acting on a structure, resulting in an overload of the structural capacity. This may lead to a collapse of the structure. Another important failure mode for an offshore jacket structure is the "progressive damage due to fatigue" failure mode. The failure mode may start with an initial defect in a weld that develops into a through-thickness crack. Consequently the structure may loose parts of its resistance against collapse. If the structure in this state is exposed to a relative large wave and current load, the structure may collapse even if the load is significantly smaller than the design load.

The possible consequences should, as for hazards, be evaluated based on expected consequences in standards, experience from historic events and analytical reasoning. HSE (1999a) mention the following consequences as a result of a structural integrity failure:

- Total collapse of structure
- Loss of stability
- Hydrocarbon escapes
- Electrical fire (fires in electrical systems)
- Toxic / asphyxiating gas
- Dropped object
- Diving accident

The consequences hydrocarbon escapes, electrical fire, toxic / asphyxiating gas, dropped object, and diving accident may be a result of individual member failures in the riser, conductor and caisson support frames. They may also be a result of large deformations, total collapse of structure or large accelerations at the topside as a result of the loading (e.g. after boat impact). The fire protection and corrosion protection may also be severely damaged by large deformations.

The consequence related to loss of stability, as mentioned in HSE (1999a), is generally regarded to be applicable for floating units. For a jacket structure the loss of global stability may be an event in the failure mode leading to total collapse of the structure, but is not an end consequence in itself. Hence, the consequence is not included in the further discussions.

Experience from known accidents on jacket structures show that a leaning installation, with large deformations, is the most frequent consequence (Botelho et al 1994). An example of this is shown in Figure 4-1 (DeFranco et al, 2004). There is also an example of a toppled topside, and total collapses (Botelho et al 1994, DNV Technica 1995). As a result of this experience and the above reasoning, the consequences included in the following discussion is limited to total collapse of structure, large deformations and toppled topside.

Figure 4-1: EI-322 'A' complex after Hurricane Lilli (DeFranco et al 2004).

An illustration of the failure mode with an intact structure exposed to a large wave, as described above, is shown in Figure 4-2. The chain of event diagrams for the hazards subsidence and worsening of wave climate is not illustrated separately, as they will both only add to the probability of the possibility of a "large wave" as illustrated in Figure 4-2.

Focussing on the hazards mentioned earlier, an evaluation of contributing factors, root causes and possible escalation scenarios is performed. The result of this evaluation is presented in Table 4-3. Note that the hazards subsidence and worsening of wave climate is evaluated as contributing factors for the wave in deck and excessive wave and current hazard.

Accidents often occur as a result of several possible minor failures and errors that occur and create an unexpected weak system. This should also be considered possible for offshore jacket structures. A structure with a degraded member exposed to a wave loading may be the most important failure mode to evaluate. A structure with a degraded member will normally not be a problem in itself, and would need a certain wave and current loading to cause a collapse of the structure. The exception from this is if the degraded member is a critical member with no redundancy in the structure after failure in this member. This would imply that the member would be a leg in a 3- or 4-legged jacket. An illustration of the failure mode with a degraded member is shown in Figure 4-3. A summary of the hazards, contributing factors, root causes and possible escalation based on chain of event evaluation is presented in Table 4-3. These hazards and resulting failure modes, in form of chain-of-event diagrams will be the basis for the barrier analysis in the next section.

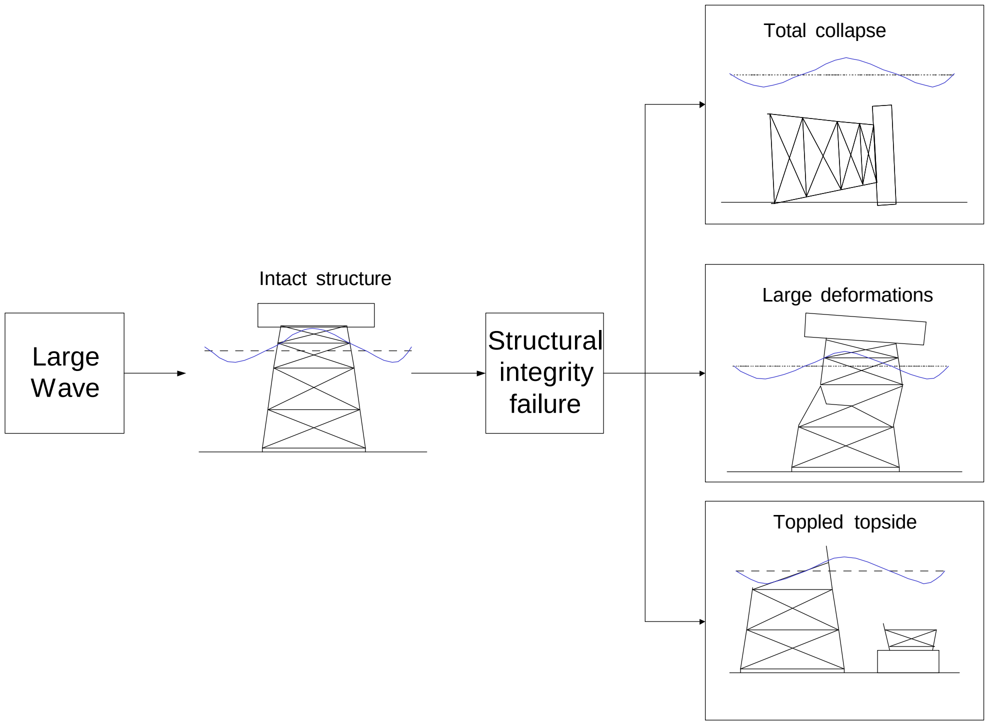

Figure 4-2: Excessive environmental load failure mode

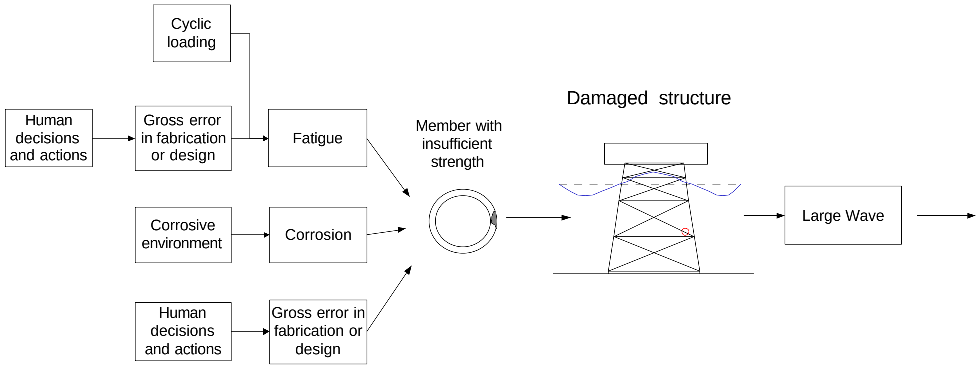

Figure 4-3: Failure or weakened member failure mode – the events for structural integrity failure following the events shown in this figure will be as for the intact structure

Table 4-3: Hazard, contributing factors, root causes and possible escalations

Hazard	Contributing factors and root causes	Further escalation	Further escalation
Excessive wave and current loading on jacket	Unfortunate combination of waves, wind and current. Amount of marine growth influence the loading. Dynamic effects. Worsening of wave climate	Overloading of jacket structure	Total collapse of structure Large deformation Toppled Topside
		Deck main structure overload	Loss of drilling rig, heli-deck, flare tower, cranes, living quarter or whole topside.
		Failure of riser, conductor and caisson guides.	Hydro carbon leak and lack of fire water, and possible explosion / fire scenario.
Wave in deck	Subsidence Worsening of wave climate	Overloading of jacket structure	Total collapse of structure Large deformation Toppled Topside
		Deck main structure overload	Loss of parts of or whole topside
Excessive topside load	Adding of modules and loads Centre of gravity shift Extreme ice loading	Overload of jacket structure Loss of global stability of jacket	Total collapse of structure Large deformation Toppled Topside
		Deck main structure overload Module failure	Loss of parts of topside, or topside modules
Insufficient structural strength. Due to corrosive environment, cyclic loading, human decisions and actions that may have led to gross errors.	Insufficient structural strength due to: -Crack growth due to fatigue -Corrosion -Gross errors in design, fabrication, transport, installation or operation	-Overload of degraded jacket structure -Accelerated fatigue of the degraded structure, and overload of structure with more than one member failure.	Total collapse of structure Large deformation Toppled Topside
		-Deck main structure overload -Drilling rig failure, Helideck failure, Flare tower failure, Crane failure, Living quarter failure	Loss of parts of topside, or topside modules
	Support of conductors and risers degraded due to: - Crack growth due to fatigue - Corrosion - Gross errors in design, fabrication, transport, installation or operation	Support of conductors and / or risers fails Failure in conductor / riser	Fire / explosion / HC spill

4.4 Barrier analysis of a jacket structure

The barrier principle can be used to evaluate jacket structure for hazards related to environmental loading in combination with hazards related to degradation of the structure.

The barrier identification is performed under the assumption that barriers should be a realisation of one of Haddon's defences (Haddon 1980). Haddon defines a hazardous energy as the starting point of the event leading to an accident. Even though the identified hazards in most cases can be defined as hazardous energies, it would require some level of abstraction. Hence, the starting point for the events leading to an accidental event is here started with the identified hazards.

As a part of the qualitative risk analysis of a jacket structure performed in this thesis, a barrier analysis is performed based on the hazard identification described earlier in this chapter.

First we will consider failure modes related to large wave and current loads on the substructure and the case wave-deck impact loading. The corresponding barriers are identified, see Figure 4-4 and Figure 4-5. Barrier system and performance influencing factors are also indicated. In each of these figures several barriers are indicated. If the first barrier should fail, the second barrier may still prevent the accident. If the second barrier should fail, the third barrier may still function etc. It should be noted that the focus in these barrier analyses is on human safety. Some slight changes would be necessary if focus is set to environmental safety or economical safety.

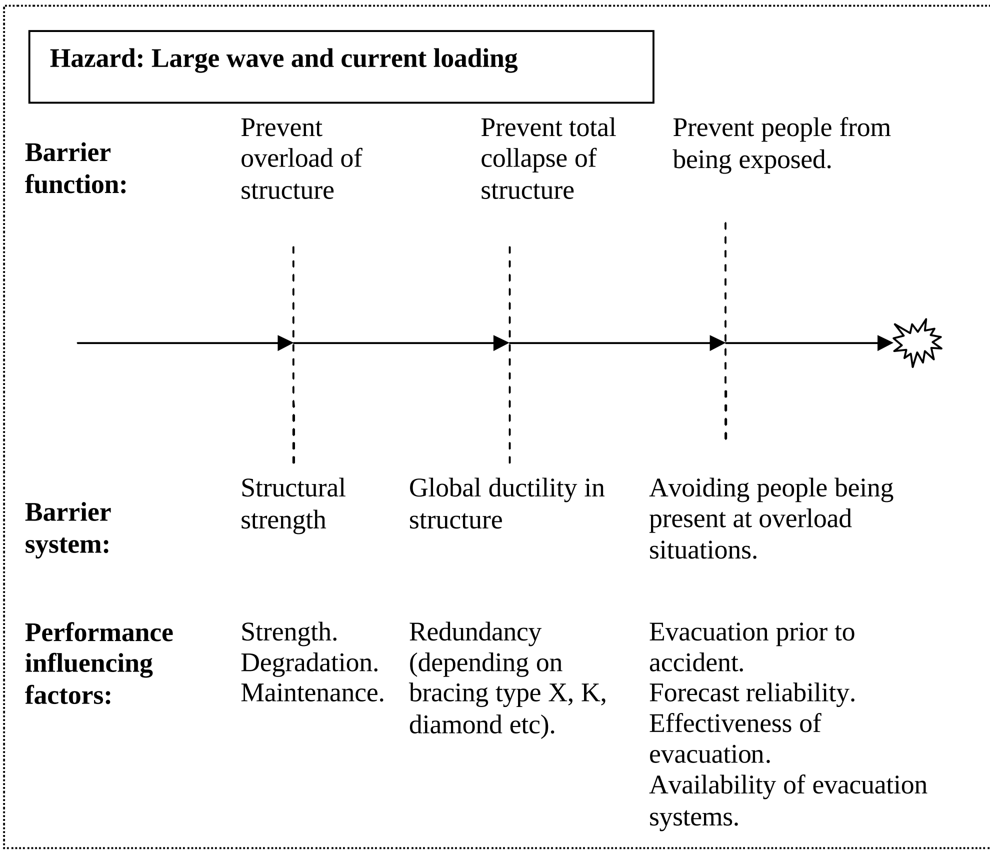

Figure 4-4: Barrier analysis diagram: Overload failure mode and barriers

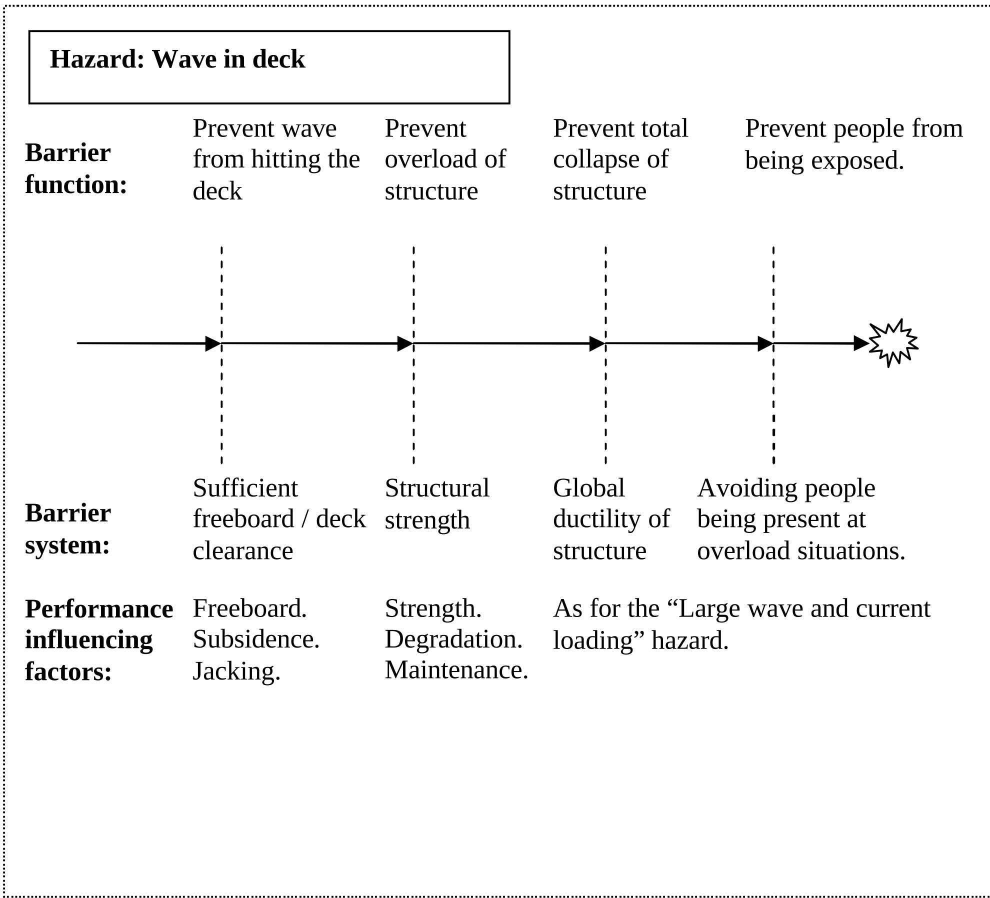

Figure 4-5: Barrier analysis diagram: Wave in deck failure mode and barriers

The hazards, failure modes and barriers presented in Figure 4-4 and Figure 4-5 can be presented in a barrier diagram (chain of event diagram with indications of barriers) as shown in Figure 4-6.

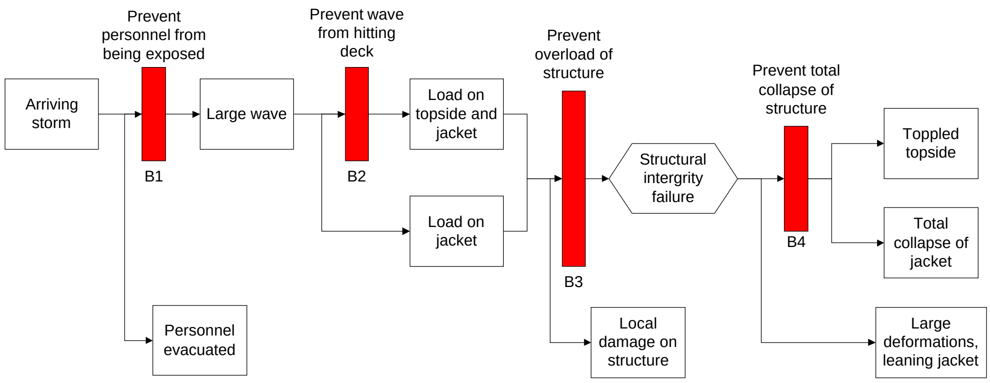

Figure 4-6: Barrier diagram for large wave hazard on a jacket structure. If a barrier is functioning, the chain of event would have to follow the alternative path as indicated.

With regards to the barrier "Prevent personnel from being exposed" indicated in Figure 4-6, this barrier will only affect the personnel. The structure will still be exposed to the remaining chain of events, even if the barrier is functioning. The chain of events and barriers relevant for the structure is as presented in Figure 4-7.

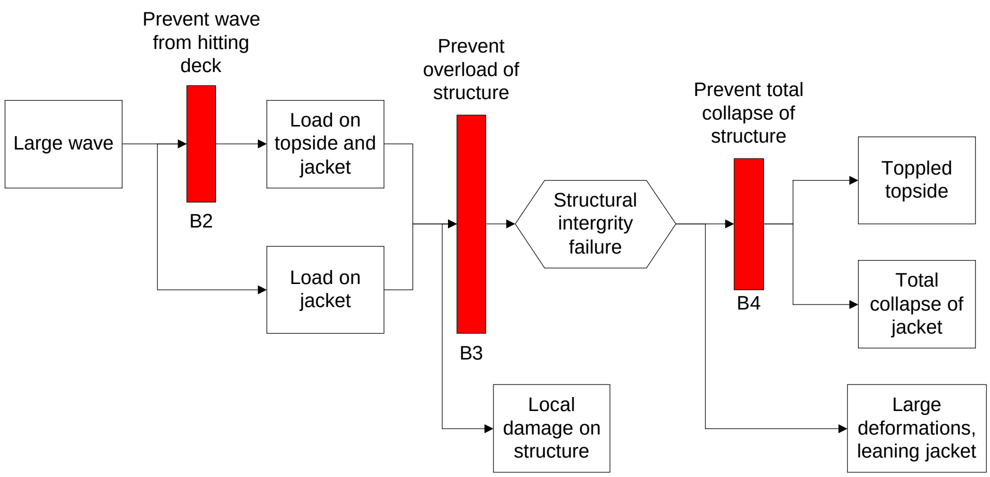

Figure 4-7: Barrier diagram for large wave hazard for the structure on a jacket structure.

The hazard, failure modes and barriers as presented in Figure 4-7 are the focus of the problems evaluated in this thesis. The degradation of structural integrity and subsidence hazards may primarily be regarded as threats to the barrier presented in Figure 4-7. Degradation would limit the performance of the barrier "Prevent overload of structure" and subsidence would limit the performance of the barrier "Prevent wave from hitting deck". The hazard "Worsening of wave climate" would increase the probability of a large wave. The hazard "worsening of wave climate" will not be discussed further in this thesis, as this is outside the scope of the work.

Similarly, fatigue degradation, gross error resulting in fatigue degradation, insufficient strength due to gross error, wide-spread fatigue degradation, fatigue spreading due to load redistribution and corrosion degradation is evaluated. As an example the barrier analysis diagram for the hazard fatigue degradation is shown in Figure 4-8.

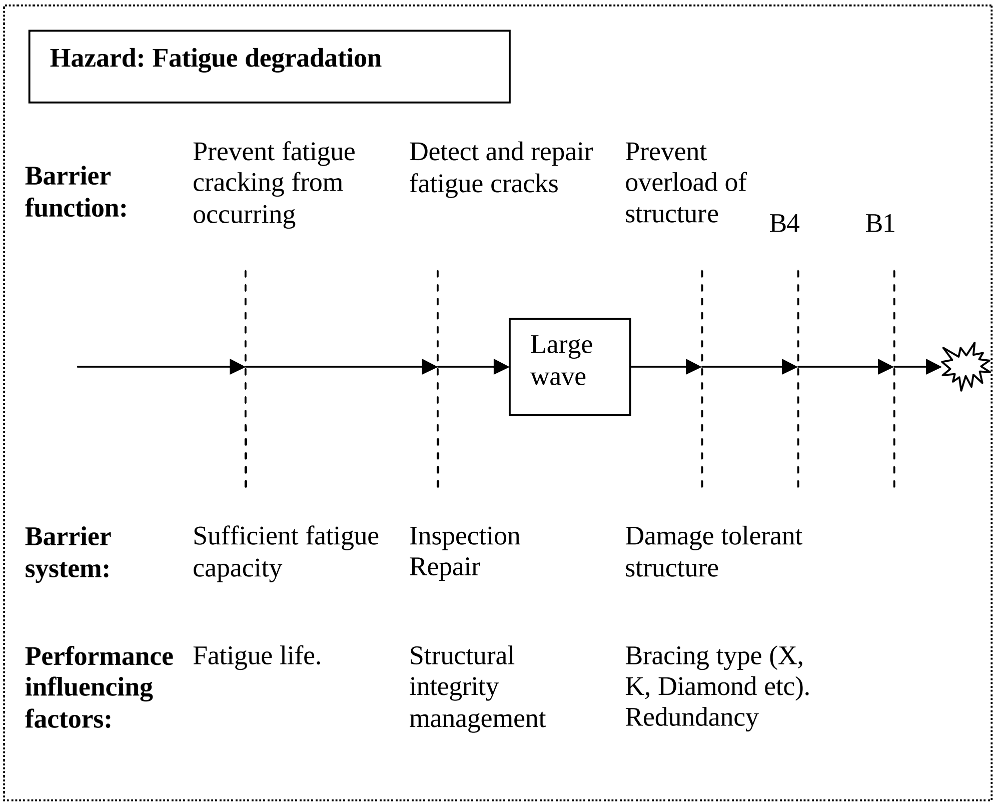

Figure 4-8: Fatigue degradation failure mode (The barrier "Prevent total collapse of structure" is denoted B4 and the barrier "Preventing personnel from being exposed" is denoted B1)

The barriers identified in the above barrier analysis and from the barrier analysis presented in Ersdal (2002) are listed in Table 4-4.

Table 4-4: Barriers for the hazardous events.

Hazard	Strategies related to the energy source (Strategy 1-5 in Haddons regime)	Strategies related to barriers (Strategy 6-7 in Haddons regime)	Strategies related to the vulnerable target (Strategy 8-10 in Haddons regime)
1) Excessive wave and current loading	Reduce wave loading and loading effect: -reduced number of conductors etc. - remove marine growth - limit dynamic effects.	Structural strength. Global ductility. *Evacuate personnel ahead of storm.* *Restrict people from exposed installations.*	Evacuation equipment that can be used under large tilt.
2) Wave in deck		Freeboard (air gap). Restrict people from exposed areas of deck. As for hazard 1.	As for hazard 1.
3) Excessive load from topside in combination with hazard 1	Weight control Modification control	As for hazard 1.	As for hazard 1.
4) Fatigue failure in combination with hazard 1	Fatigue capacity Inspection of structure, maintenance and repair.	Damaged tolerant structure. As for hazard 1.	As for hazard 1.
5) Gross error resulting in fatigue degradation of component in combination with hazard 1	Fatigue capacity. Probability of gross error crack occurring at stress hot spot. Inspection of structure, maintenance and repair.	As for hazard 4.	As for hazard 1.
6) Gross error resulting in insufficient strength of component in combination with hazard 1	Inspection of structure, maintenance and repair	As for hazard 4..	As for hazard 1.
7) Widespread fatigue failure in combination with hazard 1.	Probabilistic redundancy[6]. As for hazard 4.	As for hazard 4.	As for hazard 1.
8) Fatigue spreading due to load redistribution in combination with hazard 1.	Damaged fatigue capacity[7]. As for hazard 4.	As for hazard 4.	As for hazard 1.
9) Corrosive degradation in combination with hazard 1.	Corrosion protection (anodes, painting, coating etc.) Inspection of structure, maintenance and repair. Repair of anodes and paint.	As for hazard 4.	As for hazard 1.

[6] Probabilistic redundancy is a term used to express the significant reduction in probability for fatigue cracks occurring in multiple joints in the same structural failure mode, compared to the probability of the occurrence of one single fatigue crack in this failure mode. The term is defined by Dalane (1993) and illustrated with examples. The probabilistic redundancy may be limited for ageing structures.

[7] Damaged fatigue capacity is supposed to be a measure of the remaining structures ability to withstand the cyclic loading with one member damaged.

4.5 Indicators for structural barriers

4.5.1 Structural strength

The structural strength barrier can be measured by several methods. Traditionally design of structures and the strength of structures have been expressed in utility checks (UC). This has been done for each member in the structure, and the criteria for the structure have been that all UC's should be less than 1.0. As such, an indicator for this barrier may be the highest UC in the structure. If a member is damaged by denting, cracks or corrosion during the life of the structure, the UC's would need to be updated in accordance with the present situation. The major drawback for using this quantity as an indicator for structural strength is that the quantity is just giving information about a single member in the structure, and not the system strength. One can easily think of a structural system that has a single member that fails under a relatively low loading, but after load redistribution can take significantly higher loads without failure.

Structures ability for load redistribution can be measured by its degree of static indeterminate. A drawback of this method is that it does not give any information about the load versus strength (as the UC factor). A combination of UC and degree of static indeterminate would be necessary.

Another way of expressing the effect of redistribution after first failure can be illustrated by using a non-linear collapse analysis (also called push over analysis). The structural redundancy (SR), defined as the ratio between the maximum loading that the structure can take according to the non-linear analysis and the loading when the first member fails, can then be taken as a measure of redistribution ability. A combination of the UC and the SR would be necessary to describe the structural strength.

Alternatively, the structural strength can be measured by the reserve strength ratio (RSR). This factor includes both the structural strength and the ability to redistribute. This quantity is defined as the ratio between the maximum loading that the structure can tolerate according to the non-linear analysis and the characteristic design loading.

The indicator expressing the robustness in the strength should preferably include robustness towards small changes in capacity, e.g. damage to a structural component, and robustness for changes in load pattern (e.g. wave attacking at a higher level). The robustness towards small changes in capacity is covered by the damaged strength ratio (DSR) parameter discussed later. Robustness for small changes in load pattern may also be of importance, as failure modes may be introduced from waves exposing higher elevations of the jacket structure. In this thesis a new-failure-mode (NFM) parameter introduced, with the intension of including this effect. This parameter is discussed further in Chapter 6.

With respect to functionality of the strength barrier, deformations after the load should be limited, allowing for a structure that is fit for evacuation (no hydrocarbon leakage, no electrical fires, no dropped objects, and no toxic gas leakage). This is not fully included in the reserve strength ratio (RSR) parameter.

4.5.2 Freeboard / deck clearance

The freeboard is the distance from LAT to bottom of steel (BOS) of topside, refer to Figure 4-9. The freeboard needs to be measured relative to the relevant wave crest elevation at the location of the installation. The total crest elevation (wave crest in addition to water level variation due to astronomical variation and storm surge) should be compared to the distance between water level and bottom of steel on topside.

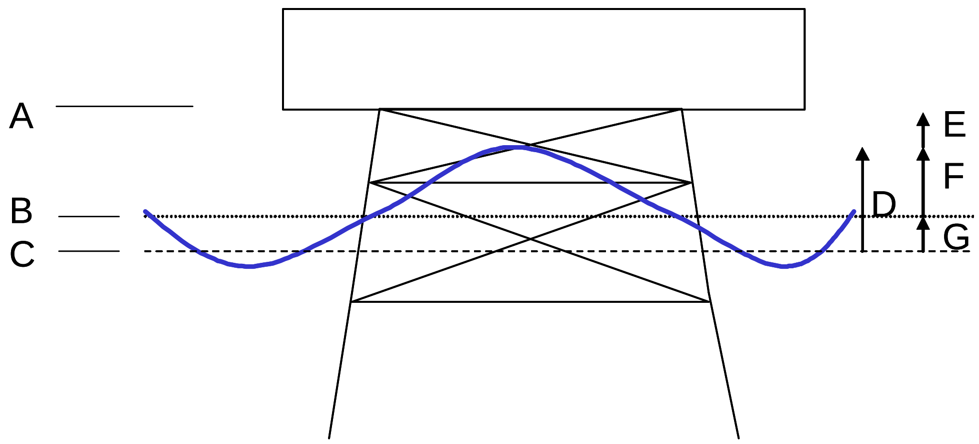

Figure 4-9: Crest elevation, storm surge and tides.

Legend for Figure 4-9:
A : Bottom of steel (BOS)
B : Highest still water level including astronomical tide and storm surge
C : Lowest astronomical tide (LAT)
D : Crest elevation including wave crest height, astronomical tide and storm surge.
E : Air gap
F : Wave crest height
G : Still water level variation due to astronomical tide and storm surge

The freeboard equals the combined distances E, F, and G.

Air gap

Air gap is a measure of the distance between the maximum wave crest elevation and the bottom of steel of topside (Air Gap = Freeboard – Wave crest elevation). At present, the 10 000 year wave-crest elevation (the wave crest elevation with an annual probability of exceedance of 10^{-4}) is normally used for air gap calculations. However, for installations designed according to older standards a 100 year wave-crest elevation (the wave crest elevation with an annual probability of exceedance of 10^{-2}) including a safety margin has also been used in determination of air gap at design.

The air gap measure is a reasonable indicator for the safety towards wave in deck, but the drawback is that it does not very well differentiate between installations in harsh environment and installations in benign environment. A two meters air gap for an installation in benign environment may be sufficient to ensure that wave in deck incidents do not occur with an acceptable probability, but for an installation in harsh environment like the Norwegian Continental Shelf this will in general be insufficient. The problem is that the air gap measure does not take into account the information about the wave crest elevation.

Reserve Freeboard Ratio
A possible way of taking the difference of the wave crest elevation at different sites into account is by using a ratio between the freeboard and the wave crest elevation.

$$RFR^{*} = \frac{Freeboard}{Wave\_crest\_elevation_{100}} \tag{4.1}$$

This reserve freeboard ratio will include some additional information about the environment at the site, as it gives a percentage extra freeboard compared to the wave crest elevation. However, this does not take into account the slope of the Weibull distribution of wave crest heights. The same limitation is also applicable for the calculation of Reserve Strength Ratio (RSR) mentioned earlier. To achieve a RFR that is comparable with the RSR, the 100 year wave-crest elevation (the wave crest elevation with an annual probability of exceedance of 10^{-2}) should be used in the calculations as this is the standard reference for the RSR calculation.

The remaining drawback is that the consequence of a wave in deck incident is not necessarily as critical as an overload of the structure. Hence, the RSR is an indicator for a severe incident. RFR, however, is an indicator of a high loading scenario with some potential for becoming a severe incident. An alternative definition of the reserve freeboard ratio is then the ratio between the critical crest height and the design crest height. The critical crest height is then the crest height that causes the jacket to collapse.

$$RFR = \frac{h_{critical}}{h_{design}} \tag{4.2}$$

The critical crest height can be conservatively estimated as equal to the freeboard.

4.5.3 Global Ductility of structure

The global ductility of the structure needs to give some information about the structures ability to absorb the energy of the large wave load without resulting in a total collapse of the structure. An installation with large deformation has some potential for surviving the storm, and for being evacuated after the storm has passed the site.

The possible difference between the load-deflection curves for a ductile and a brittle structure is shown in Figure 4-10. The ductile structure will have a larger possibility of surviving a large wave load than a brittle structure.

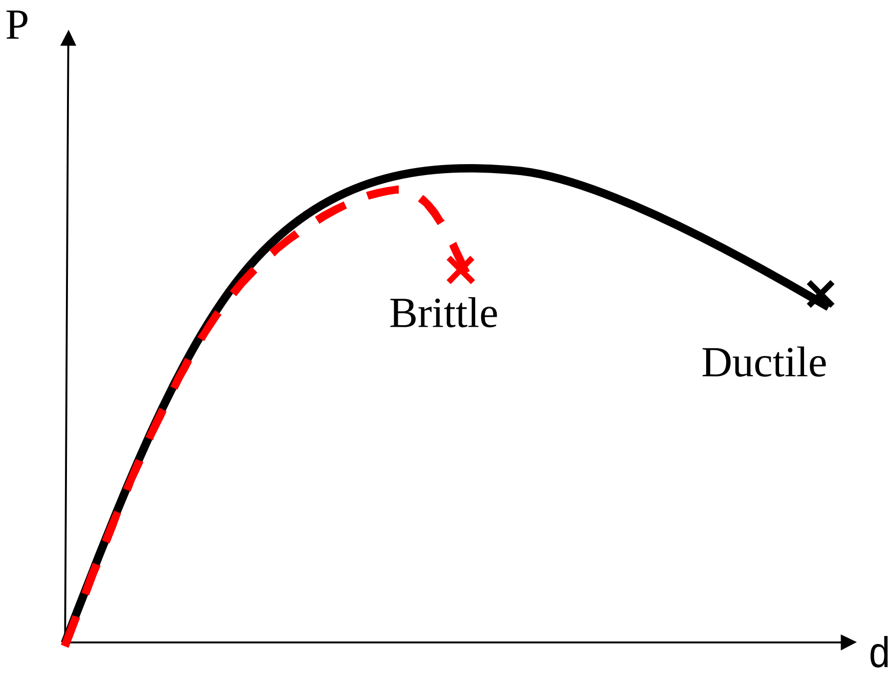

Figure 4-10: Load - deflection curves for ductile and brittle structure

The global ductility will depend on the redundancy of the structure in post collapse situation, but may also depend on the post-collapse performance and the ductility of the steel in the critical members of the structure. The bracing type of the jacket structure may be an important indicator for this global ductility. A possible measure for the global ductility may be the bracing type. A more advanced method may be to calculate the energy absorption in the structure in a collapse analysis into total collapse. However, a jacket structure with large ductility will, in damaged state, have significant deformations and this may be a severe problem for other possible failure modes of the structure as mentioned earlier (e.g. hydrocarbon contained risers and conductors in the jacket may be damaged and leak, dropped objects may occur).

The importance of the global ductility of the structure is acknowledged, but it is not further investigated in this thesis. The focus will be on the barriers that are intended to stop the accident at an earlier stage in the event-chain.

4.5.4 Structural strength of damaged structure

The structural strength of a damaged structure is most easily measured by similar methodology as used for the structural strength of intact structure. Non-linear collapse analysis with removal of braces has been used to calculate the importance of each member, and to evaluate the collapse capacity of a damaged structure (Hellan et al 1988).

The DSR is defined as:

$$DSR = \frac{Q_r}{Q_d} \tag{3.15}$$

where Q_d is the design load for the structure and Q_r is the ultimate collapse capacity of the structure in damaged condition.

4.5.5 Fatigue capacity

The simplest method of measuring the remaining fatigue capacity is probably taking the difference between the calculated fatigue design life of each component and the age of the structure. However, this method will not take into account several important aspects of the fatigue life of a structure. First, the structure will not fail with one through thickness crack. The component experiencing the through thickness crack may have significant additional life before the component fails. When the component finally fails, most jacket structures are redundant and will still be able to carry the necessary loading. Also, inspection results where updated information of existence of a fatigue cracks is not included in such a simple approach. If a component is inspected, and no cracks are determined, this can be an indication of a lower crack growth rate for this component than expected from the analysis. If a fatigue crack has been found, the most realistic remaining capacity of the component, prior to repairs, is found by fracture mechanic crack growth calculations.

The fatigue utilisation index is developed by DNV (HSE 1999b) for ageing drilling rigs as an indicator for fatigue usage. The factor takes into account the different environments that the drilling rig has been exposed to during its life. If the drilling rig has been in benign environment a significant part of its life, the “real” fatigue usage may be less than expected. However, jacket structures are in general at site continuously and exposed to the environment the jacket was intended for in all the years the jacket has been in operation. A possible correction for the actual time history of waves that the jacket has been exposed to is a possible modification. However, the deviation between statistical estimates of “normal sea-states”, which is dominating the fatigue calculations, and the actual “normal sea-states” are not believed to be significant.

The acceptable life of the structure may be significantly extended by careful use of inspections and repair of damaged components. No sufficient indicators have been found at present. A further discussion of fatigue is found in Chapter 7 of this thesis. Conclusions and possible indicators will be discussed in that section.

4.5.6 Fatigue capacity of damaged structure

If a component in a structure is damaged, this may lead to load redistribution to other components in the structure. This may further lead to an increased crack growth in these components. If fatigue capacity of such a damaged structure is to be documented, it would mean that fatigue analysis would have to be performed for a high number of hot spots and damage scenarios. At present this may be unrealistic, and simplified methods are required. However, a possible simplification is possible if damage strength calculations are performed. The damaged strength calculation will check the collapse capacity of a structure with a component removed to simulate damage. If the stress increase (increase from analysis of intact structure to analysis of damaged structure) for surrounding members and joints are checked at the design load level, an estimate of the reduction in fatigue capacity may be calculated. It is then assumed that the ratio between stress-level in intact state versus damaged state at design loads is representative for ratio between intact state and damage state at all load levels. In this case the number of fatigue cycles in a damaged state can be calculated from the SN curve formulations

$$\log(N_{damaged}) = \log(N_{intact}) + m \cdot \log(\sigma_{intact}) - m \cdot \log(\sigma_{damaged}) \tag{4.3}$$

This equation can be written as:

$$\frac{N_{damaged}}{N_{intact}} = \exp\left[m \cdot \log(\sigma_{intact}) - m \cdot \log(\sigma_{damaged})\right] = \left(\frac{\sigma_{intact}}{\sigma_{damaged}}\right)^m \tag{4.4}$$

Hence, a simplified indicator of the structures damaged fatigue capacity may be the minimum N-damaged / N-intact factor.

4.5.7 Inspection, maintenance and repair

The performance of the barrier for inspection, maintenance and repair is not easily measured on a single parameter. The performance of the barrier consists of elements such as:

- Frequency of inspections and percentage coverage of primary components
- Method of inspection
- Quality of inspection and inspection reports
- Repair methods
- Quality of repair methods
- Inspection strategy e.g. how to calculate inspection intervals and which components to inspect.

A system for measuring an organisations capability for structural integrity management (SIMCMM) is developed by Cranfield University for the Petroleum Safety Authority - Norway (Sharp et al 2004). This tool is a possible method to evaluate the organisational part of the barrier. In addition hard data of frequency of inspections and methods of inspections would be necessary to give a full picture of the performance of the barrier.

5 Assessment by probabilistic methods

5.1 Introduction

Assessment by probabilistic methods is a possible approach for life time extension. The purpose of this section is to evaluate the possibility of using probabilistic methods, and discuss limitations in this method. A promising aspect of probabilistic methods is that it makes it possible to reduce the uncertainty about a structure if a good structural integrity management and maintenance are performed and if the structure is performing well. Hence, this will have a positive effect on the failure probability of the structure. In contrast, if the structural integrity management system and maintenance is performed unsatisfactory, the uncertainty about the structure will be large. Hence, the failure probability will be relatively larger.

5.2 Understanding probabilities

As discussed in Section 2, structural reliability analysis (SRA) is seen in this thesis as the most promising method to directly evaluate the safety of the structure in an assessment for life extension. Structural reliability analysis takes into account the actual uncertainty about the structure, and provides a tool for evaluating possible changes in the uncertainty about a possible future failure of the structure. However, SRA methods are still not widely accepted. The main objection towards using SRA in assessment of structures is that they do not calculate a failure probability of the structure that is representing the "true" failure probability (Kvitrud et al 2001). The estimated failure probability, using SRA, is according to most standards and guidelines to be compared with an acceptance criterion. However, generally accepted target reliabilities do not exist. Arguments supporting this view are, among others, that probability distributions and their parameters are based on subjective choices. Analysis by different analysts may then result in differences in the resulting probabilities. With this lack of consistency in the analysis[8] it is then argued to be impossible

[8] Lack of consistency in the analysis in this context means that if several persons were to do the same SRA of the same structure, there is no guarantee in the SRA method as it is today that these persons would come up with the same resulting failure probabilities (within a small error margin)

to compare with an acceptance criterion. Most of all the critique is based on the observation that accidental loads and gross errors[9] are not included in the analysis. Based on historic data, accidental loads and gross errors represent approximately 90% of observed accidents (Kvitrud et al 2001).

The lack of general acceptance of SRA and decision methods based on SRA may be due to the fact that the SRA is a relatively new method, but also due to a difference in opinion on what the calculated failure probabilities represent. The first major distinction that should be made is between objective probabilities and subjective probabilities.

Objective probabilities

The classical (objective) view on probabilities is the belief that probabilities are real and are properties of the object (structure) in itself, independent of anyone is thinking about it or not. For an objectivist there exists a true failure probability for the structure. The probability of occurrence of an event is considered to be obtained by a relative frequency, e.g. the ratio of structures failing of an infinite (or large) population of similar jackets exposed to the similar population of loads. The problem occurs in those cases where we can not perform such an experiment. It is not feasible to build an infinite population of similar structures that is exposed to environmental loads for several years to find the probability of failure. We have one specific structure, and are looking for the failure probability of this specific structure. In these cases, frequentism cannot help us and we have to rely on an estimate of this underlying true failure probability (an estimate that is subjected to significant uncertainty).

With the classical objective understanding of probability, the basic test of validity of the calculated probability as an input to decision-making may be a comparison between predictions of the theory and observations[10]. If such a comparison is absent, there is strictly no rational basis on which to employ the probability in question.

Subjective probabilities

The alternative is to understand probabilities as representing a degree of belief about the occurrence of the event or as stated in Aven (2003 p xii) "*as a measure of uncertainty*" about the occurrence of the event where it is clear who the assessor of the uncertainty is. The quantification of subjective probabilities and the use of compatible data to improve these quantified probabilities may be achieved through Bayes' theorem, which has led to the use of the term Bayesian probability for subjective probabilities.

For a subjectivist there exists no objective true failure probability for the structure (see e.g Aven 2003). However, some approaches to risk and reliability analysis may be seen as a

[9] Niels Lind (Lind 2005) describes this lack of including gross errors, the cause of most structural failures, in the structural reliability analysis with an entertaining comparison: "One is reminded of the old joke about the drunk who looks for his lost keys under the lamppost, not because he lost them there, but because he would have a chance of finding them if they were there".

[10] A traditional scientific requirement, Poppers falsification theory, for evaluating the objectivity of a theory is that the predictions from this theory should be compared to results from tests and observations. If the predictions do not meet the observed results, the theory is falsified and should be rejected (Chalmers 1999). Other views on scientific objectivity also exists (Chalmers 1999).

mixture of an objective and a subjective understanding of probability, as subjective measures of uncertainties are used to estimate what is believed to be the true underlying risk (see e.g. Ditlevsen and Madsen 2003, Diamantidis 2001).

Concerns are often raised when using probabilities as a subjective measure of uncertainty. Science is about searching for objective truths, and a subjective opinion is not a proper way of doing science. Using this as an argument against the use of subjective probabilities, one has to believe that objective true probabilities about a future event do exist. This thesis is based on an underlying belief that such true probabilities do only exist in deterministic systems, where there is no uncertainty about the outcome, and hence probability analysis is irrelevant. This belief is along the line of Aven (2003) stating "Our view is that complete knowledge about the world does not exist in most cases, and we provide a tool for dealing with these uncertainties based on coherence. If sufficient data become available, consensus may be achieved, but not necessarily as there are always subjective elements involved in the assessment process. The objective truth when facing future performance does not exist."

Approaches to probabilistic analysis

There exist several approaches for probabilistic analysis based on a pure objective or a pure subjective understanding of probabilities. Approaches representing a mixture of objective and subjective understanding, where typically subjective probabilities are used to estimate what is believed to be the true probability, are also available. Some of these approaches are discussed in Aven (2003): The classical approach represents a pure objective understanding of probability, the best fit approach applies a mixture of objective and subjective understanding of probabilities, and the predictive Bayesian approach utilizes a pure subjective understanding of probability. It is here focussed on the best fit approach and the predictive approach. Reference is given to Aven (2003) for detailed discussion on these approaches for understanding probabilities.

The <u>classical best fit approach</u> (Aven 2003) falls into the objective understanding of probabilities. Probability is assumed to be an underlying true value of the object (structure), but the failure probability is unknown and subjective probability distributions are used to estimate this true value. There is an uncertainty about how well the estimated failure probability represents the true underlying failure probability. The estimated failure probability is based on a model and subjective probabilities rather than real data, and is sometimes called *notional* in order to differentiate this from the real failure probability of the structure. This gives limitations to the understanding and communications of the results, and their use in decision making, as acceptance criteria are based on acceptance of real frequency of failures. Examples of acceptance criteria based on acceptance of real frequency of failure may be found in e.g. DNV (1996). To cope with the problem of uncertainty about the value of the true underlying probability, standardised models and input data are sought. Examples of standardised models and input data for application in SRA can be found in e.g. DNV (1996). Further, the acceptance criteria need to be considered to be a function of the models and input data. Hence, acceptance criteria need to be calibrated with the applied model and input data. This is illustrated for SRA by Moan (1997).

This classical best fit approach to probabilities is not recommended by Aven (2003), first of all because such probabilities exists only as a mental construction, and do not exist in the real world, and secondly because such a thinking would have been rejected in the comparisons between calculated and real data (Melchers 2001a, Kvitrud et al 2001).

The traditional structural reliability analysis as found in literature and standards seems to be within the classical best fit approach[11]. However, the belief that the estimated probability represents a true failure probability is not necessarily as strong as indicated by Aven (2003). In a SRA the parameters describing the probability distributions applied in the analysis are based on information about similar structures, e.g. strength of steel distribution and wave load parameter distributions. The data from similar structures are treated by a classical frequency based approach in order to obtain these parameters and probability distributions. The obtained parameters and distributions are used in combination with physical models to predict the failure probability of the structure. As the predictions do not generally coincide with the observed data, uncertainties are introduced to the models (e.g. SN fatigue model) and these uncertainties are calibrated to represent the observed reality. When a given structure is to be assessed, the general parameters and probability distributions are used as prior parameters and distributions. If new information is obtained about the structure, the parameters and probability distributions can be updated based on the new information.

The predictive Bayesian approach, where probabilities are seen merely as a measure of uncertainty about the system, is the approach recommended by Aven (2003). What is uncertain, in this approach, is the occurrence of an event A, and the probability p(A) expresses this uncertainty. In the predictive approach there is no true probability, and it is meaningless to evaluate the uncertainty of the assigned probability (Aven 2003). The starting point for this thinking is to establish a framework based on subjective probabilities that works in practice and has a clear interpretation and focus, i.e. an approach which is easy to use in communication and decision making. The focus of the analysis in this understanding of probabilities must be on the unknown but observable quantities of the world that can be measured, and that have a true value (e.g. the strength of the structure, the number of fatalities and the occurrence of an accident). The quantities may not actually be observed; the point is that a true number exist and if sufficient recourses were made available the number could be found. The introduction of fictional model uncertainties must be avoided. Hypothetical populations and related parameters should not be introduced. The results of such a calculation of failure probability of a structure cannot be verified, as it expresses the analyst's uncertainty prior to observation and hence traditional scientific falsification methodology cannot be applied for evaluating the calculated failure probability.

The author has found the predictive Bayesian approach reasonable. It seems to eliminate some of the problems with traditional SRA, and is combined with decision methods that take, in a coherent manner, into account that the assigned failure probabilities are not "real" failure probabilities. However, the implementation to SRA is not necessarily straight forward. This thesis will provide some examples of how to implement the predictive Bayesian approach to SRA. Examples of utilising the predictive Bayesian approach in structural reliability analysis can also be found in e.g. Rettedal (1997), Rettedal et al (1998) and Aven and Rettedal (1998).

In practise for structural reliability analysis, using the predictive Bayesian approach would mean that the focus of the analysis should be on the unknown but observable quantities of the world like the structures strength, the wave load on the structure, the number of cycles a

[11] This is in accordance with the understanding of structural reliability analysis as described in Aven (2003) "The underlying thinking [in structural reliability analysis] is … along the lines of the classical approach, with best estimates,…, and uncertainty analyses of unknown parameters and calculation of the predictive distribution of the failure event."

structural detail tolerate before failure etc. These are true quantities that could be found if the necessary resources were set in place. However, the true values of these quantities are uncertain, as finding these values would in most cases result in a collapse of the structure. The unknown and observable quantities are chosen to be described by uncertainty distribution functions $f_{x_i}(x_i)$, representing the analysts uncertainty about this quantity *i*. Information from calculation models as non-linear structural analysis, analysis of wave kinematics and wave loading, fatigue analysis etc. and from tests that give information about possible differences between calculation models and real performance of structures or models of structures are used to describe these uncertainty functions $f_{x_i}(x_i)$.

The probability of failure of a structure for excessive loading due to wave loading may be modelled as $P_f = P(g(\mathbf{x}) \le 0)$ where $g(\mathbf{x}) = X_1 - X_2$. Here X_1 is the strength of the structure and X_2 is the wave loading on the structure. The uncertainty in the strength of the structure is as an example modelled with a normal distribution with mean value taken as the calculated value from non-linear structural analysis and a coefficient of variation (COV) chosen to an appropriate value so that the probability distribution represents the analyst's uncertainty.

Prior experience and stochastic variation in previous experiments and model tests may be used as background knowledge to assign an uncertainty distribution for the strength of the structure. However, in the end the final probability that has to be evaluated by the analyst, and whether this represents the analyst's uncertainty.

To illustrate the difference between a classic best fit approach and the Predictive Bayesian approach, an evaluation of assessing the probability of a fatigue failure is performed. The fatigue failure of a component is modelled in structural reliability analysis as $P_f = P(g(\mathbf{x}) \le 0)$ where $g(\mathbf{x}) = X_1 - X_2$. Here X_1 is the number of cycles that the component can tolerate before failure and X_2 is the number of cycles of a given stress range that the component has been exposed to at a given time T. Both X_1 and X_2 are uncertain observable objective quantities, and should be the focus in the predictive Bayesian approach. Based on the stochastic variation in previous model tests from similar components, the uncertainty in the value X_1 is expressed by a subjective probability distribution F_1. If detailed measurements of the stress cycles in the component and the number of cycles the component has experienced up to time T are available, there would be limited uncertainty in the X_2 parameter. However, in most cases such measurements are not available. Hence, the number of cycles and stress ranges of each cycle may be found by a structural analysis model. The stochastic variation observed in other similar components compared with similar analysis models may be used to express the uncertainty in the X_2 parameter by a subjective probability distribution F_2.

To deal with the fact that the component experiences stress cycles of different magnitude, and that the component would tolerate more stress cycles of a low magnitude than the component would tolerate of stress cycles of a high magnitude, the limit state function is rewritten as the Miner-Palmgren sum, see e.g. NORSOK N-004 (NORSOK 1998):

$$g(x) = 1 - \sum_i \frac{X_1(S_i)}{X_2(S_i)} \tag{5.1}$$

where S_i is stress range that the components is experiencing. According to DNV(1996) experimental data suggests that the Miner-Palmgren rule predicts fatigue reasonably well for random loading on loaded components. However, for welded joints it appears that the Miner-Palmgren rule is slightly non-conservative. Biases in the ratio between the predicted damage and the measured damage down to 0.7 to 0.8 have been observed. Another effect that is influencing these calculations is the fact that most fatigue analyses are applying a narrow banded Gaussian approach when determining the stress ranges and the number of such stress ranges. A more correct representation would be a process with a certain bandwith. This may be modelled by a rain-flow counting model (see e.g. Barltrop and Adams 1991), and possibly by introducing a rain-flow correction factor in order to take into account the wide banded process. According to DNV (1996), the rain-flow correction factor for wide banded processes indicates a compensating bias compared to the bias introduced by the Miner-Palmgren rule. However, experiments show some uncertainty in a prediction of fatigue failure following the Miner-Palmgren rule and the narrow banded approach (DNV 1996).

In a classical best fit approach these uncertainties are accounted for by modelling fatigue failure to occur when the total damage D exceeds Δ, where D is defined as a stochastic variable with a mean value of 1.0 and a coefficient of variation (COV) based on experiments.

$$g(x) = \Delta - \sum_i \frac{X_1(S_i)}{X_2(S_i)} \tag{5.2}$$

The focus is now moved away from the observable quantities X_1 and X_2, and the introduction of the Δ parameter may be seen as an introduction of a model uncertainty. Model uncertainties do not exist in a predictive Bayesian approach. The probability distribution describing the uncertainties in the observable quantities describes this uncertainty. Hence, in a predictive Bayesian approach this observed stochastic variation should rather be included in our evaluation of e.g. X_1, by applying a wider probability distribution than results based on one stress cycle only. In the predictive approach the limit state function given in Equation 5.1 should be used, as this gives focus on the observable quantities.

Finally, in the classical best fit approach, as it is in some cases applied in SRA, one would possibly calibrate the model developed to match probabilities found from experience from other components, as this approach focuses on estimating a “true” probability. In contrast, in the predictive model one would focus on representing the observable quantities by an uncertainty representation based on our knowledge and experience.

Discussion and conclusions

An important aspect of the predictive Bayesian approach is that in this approach there is no true failure probability of the structure. The true values are e.g. the structures strength and the number of stress cycles in a component before failure. These may be found if sufficient resources where set in place. The failure probability of the specific structure under evaluation can not be found.

In the predictive Bayesian approach, the objection towards the use of structural reliability analysis (SRA) that it does not assign the true failure probability of the structure, is then meaningless. A true failure probability does not exist, and the assigned probability is a measure of the uncertainty about the structure, not an estimate of a “true” failure probability. This objection towards the use of structural reliability analysis is a problem related to believing in a true underlying failure probability of the structure that is estimated by an imperfect method.

In the predictive Bayesian approach, the use of acceptance criteria is not recommended (see e.g. Aven et al 2005). According to this paper, this applies both for a classical approach and for a predictive Bayesian approach. In a classical approach, the uncertainty in the estimated failure probability is of such a magnitude that the "true" risk number could be significantly different from the estimate. Hence, the "true" risk does not necessarily meet the acceptance criteria even if the estimated risk does meet the criteria. In a predictive Bayesian approach, different analysts could produce different numbers depending on the assumptions made. In both cases a less mechanical tool seems more appropriate. However, the main problem with the use of acceptance criteria is that it takes the focus from obtaining a good solution with respect to safety and cost, and rather focuses on meeting these criteria. For further discussion on the use of acceptance criteria, reference is made to Aven et al (2005).

A cost benefit analysis is accordingly used in assessment of structures in combination with structural reliability analysis. The cost benefit analysis in essence provides a cost optimal reliability level for the structure. This reliability level is suggested as an acceptance criterion. Hence, the arguments against the use of acceptance criteria also apply for a mechanical use of cost benefit analysis.

It may be argued that the difference between a "best fit" SRA and an SRA based on a predictive Bayesian approach is marginal, if the assigned probabilities resulting from these evaluations are evaluated. As an example, if an analyst applying the predictive Bayesian approach is assigning similar probability distribution functions as used in "best fit" SRA, the predictive SRA and the "best fit" SRA may not differ significantly. It is correct that the quantitative result may not differ much, but the interpretation of the result is totally different. In the predictive Bayesian approach the quantitative result is a measure of uncertainty about the structure, whereas in the "best fit" SRA it is an estimate of the true underlying failure probability. The focus in the "best fit" SRA would be on calibrating the final probability for e.g. fatigue failure to fit with what is believed to be the true failure probability, introducing fictional parameters to cope with model uncertainties. In contrast, the focus in the predictive Bayesian approach is on the objective and observable quantities X_1 and X_2.

A difference in the classical approach and the predictive Bayesian approach is also found in the inclusion of gross errors and other accidental events. In the structural reliability analysis literature the inclusion of gross errors and accidental events is generally avoided in the analysis (see e.g. Ditlevsen and Madsen 2003), as the focus on the analysis is to model the uncertainty in the physical parameters included in the model. The resulting assigned probabilities from a classical approach would then represent the failure probability given no occurrence of gross errors or accidental events. The failure probability of a structure given the occurrence of gross errors and / or accidental events is often left in the open, or to more general risk analysis that may follow the structural reliability analysis. In contrast, following a predictive Bayesian approach it would be appropriate to include gross errors and accidental events in the evaluation of the uncertainty of the observable quantities, if this is found appropriate by the analyst.

Two alternative approaches to the predictive Bayesian approach may be mentioned. The first alternative would be an assessment approach based on traditional structural reliability analysis within the classical approach to probabilities, would require norms of how to model all probabilistic parameters. Further, the acceptance criteria would need to be developed based on the recommended probabilistic formulations. This would imply a large number of recommendations for probabilistic formulations. Attempts in defining such recommendations have been performed (see e.g. DNV 1996), without succeeding in becoming an accepted norm. The second alternative would be to analyse a large number of similar structures, acceptable by the regulations. The acceptance criteria could then be derived from these analyses (e.g. mean value or minimum value). However, the extent of rather cumbersome analyses required by this approach would make this approach unrealistic in practical situations.

5.3 Risk informed decisions

The risk and reliability analysis is a tool for supporting decisions. In this case the decision to be made is whether to use an existing structure for an extended life, to replace the structure with a new structure or to abandon the field and stop production. In decision making under uncertainty there will be two main questions that need to be evaluated:

- How do we deal with uncertainty about the future? What is the consequence if we act in a certain way?
- What is a good decision? Given our knowledge, what is the right thing to do?

Philosophically these issues can be categorised as epistemic questions (questions of knowledge), and ethical questions (questions of moral and norms) (Körte 2003).

Decision making is most conveniently described by an example including a limited number of outcomes and with simple preferences for the decision. As an example a card game would meet these criteria (if the purpose of playing is to win the card game), and the rules for the best decisions for most card games can be set up by a Bayesian decision analysis. In such an example, the decision has only one goal; obtaining the optimum utility expressed by the largest chance of winning the game. The right decision is then a pure utility optimisation. The outcomes are limited to a small number, and can easily be treated as by a frequency approach.

In risk informed decisions relating to structural safety there are two distinct differences to this example. It is not possible to define an underlying infinite population of equal jackets that can be tested in order to define a frequency of failure that can be used in the decision making. The assigned probabilities represent an uncertainty about the occurrence of a future event, rather than an objective probability. In addition the right decision have to consider several issues as personnel safety, environmental safety, long term economy of the society and company economy. The right decision is not necessarily the decision with the highest monetary utility. The problem of what is a good decision is a rather difficult task, with several opinions on how a good decision is reached. The two most used approaches to decision making as presented in literature on structural reliability analyses are the use of target reliabilities and the use of Bayesian decision methods including cost benefit analysis (see e.g. DnV 1996 and Faber 2003). A review of risk informed decision methods and their ethical background is performed as a part of this thesis and is included as Paper A in Part II of this thesis.

As mentioned earlier, the use of mechanical decision methods as acceptance criteria and cost benefit analysis is in this thesis found as unfit for both a predictive Bayesian approach and for a classical approach to probabilities. In the predictive Bayesian approach, a multi attribute analysis with managerial decision is proposed as the preferred decision method. A short review of the multi attribute analysis with managerial decision method is included here.

The basis for the material presented here is taken from Aven (2003) and Aven and Vinnem (2004).

5.3.1 Multi attribute analysis with managerial decision

A multi attribute analysis is a decision support tool analysing the consequences of the various measures separately for the various attributes (technical feasibility, economy, safety, etc.). Thus there is no attempt made to transform all the different attributes in a comparable unit. In general, the decision-maker has to weight non-market goods such as safety and environmental issues with an expected net present value, E[NPV], calculated for the other attributes (market goods) in the project. An alternative way to weight the different attributes is to use different ratios, based on a cost-effectiveness analysis.

A simple model of the decision process is shown in Figure 5-1 and covers the following items (Aven 2003):

1. Stakeholders. The stakeholders are here defined as people, groups, owners, authorities that have interest related to the decisions to be taken. Internal stakeholders could be the owner of the installation, other shareholders, the safety manager, labour organisations, the maintenance manager, whereas external stakeholders could be the safety authorities (the Petroleum Safety Authority Norway, the State Pollution Control Agency), environmental groups (Greenpeace etc), research institutions. Only the internal stakeholders will take part in the formal discussions - external stakeholders will play a role in e.g. the public domain (press etc).

2. Decision problem and decision alternatives. The starting point for the decision process is a choice between various concepts, design configurations, sequence of safety critical activities, risk reducing measures etc.

3. Analysis and evaluation. To evaluate the performance of the alternatives, different types of analyses are conducted, including risk analyses and cost-benefit (cost-effectiveness) analyses. These analyses may, given a set of assumptions and limitations, result in recommendations on which alternative to choose.

4. Managerial review and judgement. The decision support analyses need to be evaluated in the light of the premises, assumptions and limitations of these analyses. The analyses are based on a background information that must be reviewed together with the results of the analyses. Considerations should be given to factors such as

   - The decision alternatives being analysed
   - The performance measures analysed (to what extent do the performance measures used describe the performance of the alternatives?)
   - The fact that the results of the analyses represent judgments and not only facts
   - The difficulty of assessing values for burdens and benefits
   - The fact that the analysis results apply to models, i.e., simplifications of the real world, and not the real world itself. The modelling implies that a number of limitations are introduced, such as replacing continuous quantities with discrete quantities, extensive simplification of time sequences, etc.

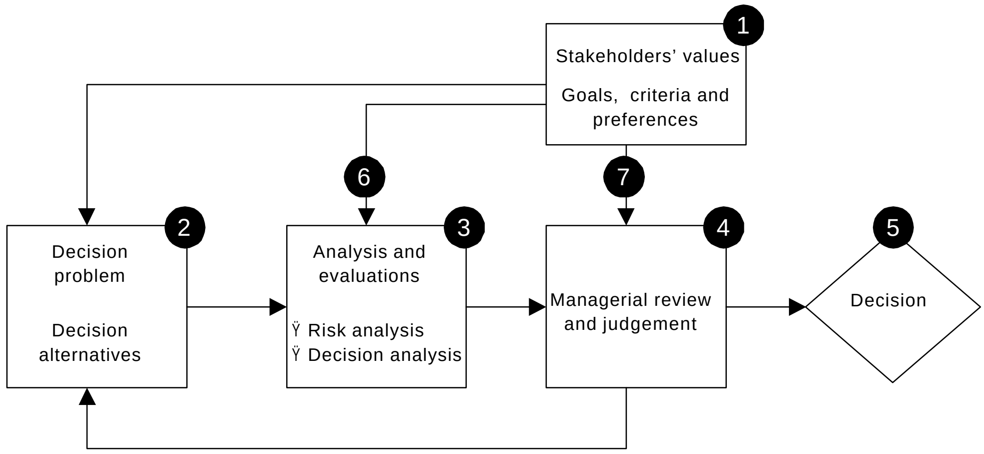

Figure 5-1: Model of the decision making process (Aven 2003)

In Figure 5-1 it is indicated that the stakeholders may influence the final decision process 7 in addition to their stated criteria, preferences and value tradeoffs 6.

For an assessment of an existing structure, the evaluated cases may include:

- D0: stop – demobilise and stop production on field.
- D1: continue to use old structure with or without modifications. Risk level due to structural failures must be established. Cost optimal modifications must be evaluated. ALARP must be applied to ensure that small investments in order to increase the safety level are performed. Various cases (D1a, D1b, etc) will be the outcome.
- D2: Build new structure: Also here more than one option is obviously possible.

Clear indication of cost (optimal solutions), human and environmental risk levels and possibilities for simple upgrades to improve safety are all needed as basis for the decision makers. An indication of the actual consequences of an uncertain event, with high criticality but low probability, should also be indicated. This can be illustrated by including a table of the necessary information and the outcome both if the structure performs according to probabilistic analysis (Dn), and information about the possible outcome if the structure fails (~~Dn~~).

The purpose of the structural reliability analysis (or risk analysis) is in this setting not on giving the decisions. The focus should rather be on establishing good solutions with respect to safety, cost and other aspects that is seen as important for the decision. If the analyst is meeting a criterion for safety, this is not sufficient to make a decision to use this solution. If improvements can be made to the solution with relatively small additional costs, the alternative with these improvements have to be included in the evaluation. The proposed solution may be compared with good practise with respect to safety, based on similar assignments of failure probability for similar structures. However, this should not be used as an acceptance criterion, but as an additional input to the decision making. However, it is reasonable to focus on alternatives that are at least reasonable within good practice and providing decision support information for these alternatives.

5.4 Reliability analysis model for a jacket structure

The problem we intent to analyse are the probability of loads exceeding the strength of the structure. Another possible failure of the structure may be severe deformations of the structure that may introduce other failure modes as mentioned in the previous section. In general, the structure may fail as a result of load exceeding the strength of the jacket structure, the piles or the topside structure and its connections to the jacket structure. The topside structure is here defined as the load carrying elements of the topside, above the top of the jacket legs. This system may be treated as a series system (the structural system fails if the piles fail OR the jacket fails OR the topside fails), as shown in Figure 5-2.

Based on traditional reliability calculations (see e.g. Aven 1992), a reasonable approximation of the probability of failure of the series system as shown in Figure 5-2 may be written as:

$$P_f\left(F_{pile} \bigcup F_{jacket} \bigcup F_{topside}\right) \approx P(L \geq S_{pile}) + P(L \geq S_{jacket}) + P(L \geq S_{topside}) \qquad (5.3)$$

However, the pile and jacket failure may be relative highly correlated. If the wave loading is the governing cause for both pile failure and jacket failure, the pile and jacket failure will be more or less fully correlated, resulting in the probability of

$$P\left(F_{Pile} \bigcup F_{jakcet}\right) \approx P(L \geq S_{pile}) \approx P(L \geq S_{jacket}) \qquad (5.4)$$

Some of these correlation effects may be modelled by using a Bayesian Probabilistic Network, as shown in Figure 5-3. The probability tables for the nodes pile failure and jacket failure would need to be defined with respect to different states of the wave loading and degradation nodes. The probability tables for the degradation nodes would need to be defined for various possible ages of the structure.

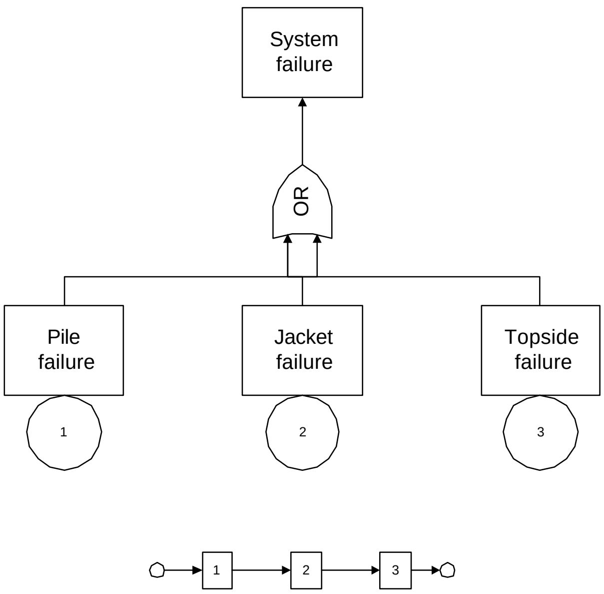

Figure 5-2: Fault tree for platform structure and associated block diagram

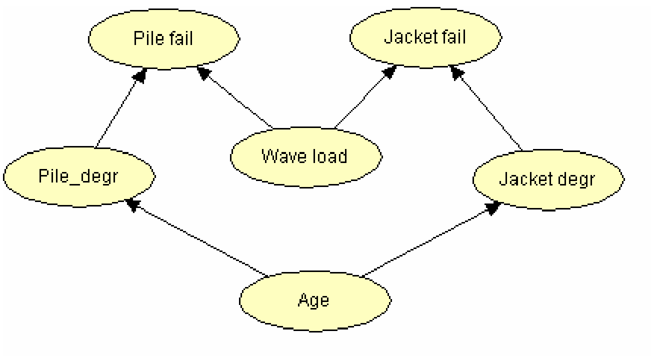

Figure 5-3: Bayesian Probabilistic Network of pile failure and jacket failure indicating some correlation effects

Piles may also be degraded due to other effects, e.g. the hammering of the piles during installation. This would not be correlated with any effect on the jacket. Hence the model in Figure 5-3 is not complete, and several effects on both the jacket and the piles are uncorrelated. Generally, Equation 5.3 will give a conservative failure probability of the system.

Each of these structural parts (piles, jacket and topside structure) may fail due to a given combination of component failures. The jacket may fail as a result of a leg failure, a leg joint failure, or a combination of brace failures. This may be described by a mixture of series and parallel systems. Figure 5-4 shows a model of an offshore steel jacket structure, indicating the block diagram for an overall series systems and local series and parallel systems.

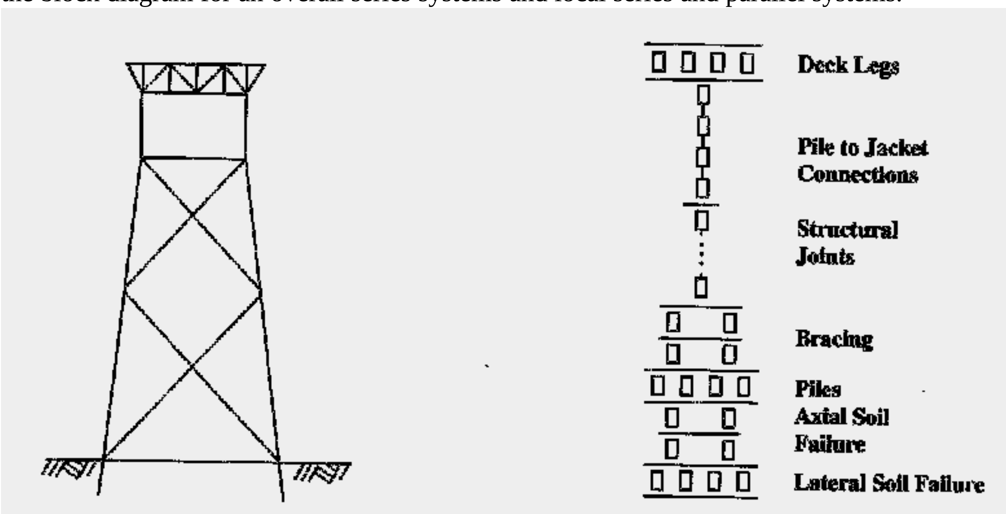

Figure 5-4: Parallel series model for offshore platform jacket structures (Moses 1995)

The structure and structural parts will further be exposed to several types of different loads (e.g. waves, wind, current, earthquake, boat impact), which has to be taken into account to get a representation of the failure probability.

This thesis focuses on the failure of the jacket structure itself in excessive wave loading. Hence, the pile and topside related failures are not considered further, nor are the jacket failure modes due to earthquake loading, vessel impact, fire and explosion, aircraft impact and possible other failure causes for the jacket structure..

A full analysis of a jacket structure following a fault tree approach would require thorough analysis of the failure probability of each component and the status of the structure after failure of this component. The failure of a critical component (e.g. leg joints) may lead directly to a system failure. However, failure of a component would in most cases not lead to system failure and the sequence of additional failures after the first failure have to be found. The first component failure may be due to loading exceeding the strength of the component (overload), fatigue degradation, gross error or other accidental loads. The second failure will be influenced by the occurrence of the first failure, e.g. the load would have to be redistributed to other components increasing the load and fatigue degradation of the other members. The failure sequence may due to this be different for the various failure causes (overload or fatigue). The resulting fault tree would be rather cumbersome, and the possible influence of one component failure on the failure probability of the other components is not easily included. Modelling the problem in a Bayesian Probabilistic Networks may allow for including such information as each node in a Bayesian probabilistic network may be dependent on other nodes. Attempts of developing a Bayesian probabilistic network has been done as a part of this project. However, no sufficiently effective and simple model for modelling structural failure has been established. The model will be rather large and cumbersome and would need input from structural reliability analysis for most nodes.

5.5 Structural reliability analysis model for a jacket structure

To avoid this rather cumbersome modelling of the structural system as a system of components, an approximate system reliability approach may be used based directly on structural reliability analysis. The benefit with the use of Bayesian probabilistic network is that gross errors and other accidental events may be modelled easily, which is commonly not included in the structural reliability analysis. However, in the following work gross errors are included in the structural reliability analysis.

In the general structural reliability formulation, the structural system is modelled to fail due to a number of possible event sequences involving failure of each component. Further, the formulation may include the event that component *j* fails given those j-1 components already have failed. An approximation of the failure probability, taking into account the possibility of failure to individual components, may be obtained by (HSE 2002 Eq 3). In this thesis a simplified version is used, where only one simultaneous component failure is assessed.

$$P_{f\_system} \approx P(sys|\overline{F}) \cdot P(\overline{F}) + \sum_j P(sys|F_j) \cdot P(F_j) \tag{5.5}$$

where $P(sys|\overline{F})$ is the probability of system failure given no component failures, $P(\overline{F})$ is the probability of no component failures12, $P(sys|F_j)$ is the probability of system failure given a failure in component j, and $P(F_j)$ is the probability of failure of component j.

The contribution from the last sum in this equation may be significant. Within the design life of a structure, a few components will be dominant and the contributions from the remaining components will be very low. Hence, often only a few components are included in the evaluation. Consideration of only a few of many alternative component failure sequences is obviously non-conservative, and even more non-conservative for ageing structures where more components will contribute significantly to the sum. On the other hand neglecting the correlation between various sequences is conservative (HSE 2002).

Correlation in strength variables is provided if joints belong to the same batch, since the between-batch variability is predominant. Correlation in stress due to common hydrodynamic factors depends upon location in the same vertical truss plane, and closeness in space. Correlations in stress concentration factors depend upon geometric similarity.

5.6 Examples of using decision methods

Cost benefit analysis
In Ersdal et al (2004), an example of cost optimal *design* of jacket structures based on their reserve strength ratio is described. The reserve strength is in this case a representation of the structural safety. The optimal reserve strength is found to depend on the cost of strengthening the structure in order to achieve higher reserve strength, and the cost of a possible failure. In general it can be concluded that a decision on the reserve strength ratio (RSR) of the jacket structure based on cost benefit analysis in the design phase result in somewhat lower RSR's compared to the RSR resulting from design based on normal standards. However, the cost of material losses and loss of income are influencing the safety requirement more than loss of lives. Hence, a paradox is obtained if the evaluation is carried out for a field consisting of two jacket structures, one carrying the process plant and production related facilities and the other carrying the living quarter. In this case the optimal safety expressed in the RSR becomes rather different for the two jacket structures. The loss of the production jacket will have significant material costs related to a potential loss, and the living quarter jacket structure will have most of its losses related to loss of lives. The cost benefit analysis with optimisation of utility will result in a high reserve strength built into the production platform, but it is cost optimal to reduce the reserve strength of the jacket under the living quarter to a much lower level. The optimal value is far below the implicit reserve strength criteria in present regulation. The moral question is then: Is it a good decision to protect the investments at a much higher level than the people?

A more relevant example for this thesis is the study of a structure that has reached its design life. The structure is assumed to have a somewhat higher failure probability than what the present technology is assumed to represent, and due to ageing processes it is also assumed that this failure probability may be increasing. The decision is to evaluate whether one

[12] $P(\overline{F})$ would in most cases be close to 1.0, and is often conservatively set to 1.0.

should continue to use the existing (and less safe) structure, or whether to build a new structure.

The additional cost for building a new installation is included with an assumed cost of 1000MNOK. Material costs of failure and implied cost of avoiding a fatality (measured using the ICAF value for Norway) is assumed independent of whether it is the new structure or the old structure that fails. The ICAF is specified as 20MNOK, and the material cost of failure is assumed to be a factor (ρ_{FM}) times the building cost of 1000MNOK. A realistic range of this factor could be from 1 - 5 times the development cost[13]. The net present value of income is specified as a factor (ρ_I) of the development costs. The factor ρ_I varied from 1 - 5 times the development costs. The expected number of fatalities given a failure is varied from 50 to 300. A possible reconstruction of a new installation after failure is not considered in this study. The new structure is assumed to have an annual probability of failure of 10^{-5}. However, changing of this failure probability to 10^{-4} does not have a significant impact on the results presented below.

The expected benefit is calculated in accordance with the method given in Ersdal et al (2004):

$$\begin{aligned} E[B] &= E[I] - E[C_D] - E[C_F] \\ &= \int_0^T i(t) \cdot \delta(t) \cdot R(t) \cdot dt - C_D - \int_0^T C_F \cdot \delta(t) \cdot g(t) \cdot dt \end{aligned} \qquad (5.6)$$

where i(t) is the income function (assumed constant in this example), δ(t) is the interest rate or discount rate (assumed to be 7%), R(t)=1-G(t) is the income reliability function (the reliability of obtaining the income taking into account the possibility of structural failure resulting in loss of the income), C_D is the cost of development (0MNOK for reuse of the old installation, 1000MNOK for a new installation), C_F is the cost of failure, and g(t) is the probability density function for time to first failure. It is assumed that the net present value of income is 5000MNOK, and that the potential material failure cost is 5000MNOK

If the probability of failure of the old structure is low, building a new structure will not be cost-effective. However, if the probability of failure for the old structure is high, the expected failure costs will at a certain point exceed the investment costs for the new installation. Using the cost benefit approach, failure probability of the old structure needs to be in the order of $10^{-1.4}$ (50 fatalities) to $10^{-2.2}$ (300 fatalities) to make these two choices balanced in a cost benefit sense (see Figure 5-5). This is 2-3 orders of magnitude higher than what is presently expected from structures in the offshore industry.

It may be argued that the implied cost of avoiding a fatality should be chosen higher, and with ICAF e.g. in the order of 200MNOK choices will be more in line with normal practise and present regulation. If an ICAF value of 200MNOK is used in a case with the conditional expected number of fatalities of 300, the balance point is changed towards a lower failure probability ($10^{-2.9}$). However, still the failure probabilities are much higher than what is expected from codes and standards. Based on this evaluation it is difficult to accept that cost benefit analysis alone can be used in a decision-making process for life extension.

[13] Development cost includes all aspects of developing a new oil and gas producing field. This would e.g. include design and construction costs for the structure, installation cost at the field, drilling cost and cost of construction and installation of pipelines.

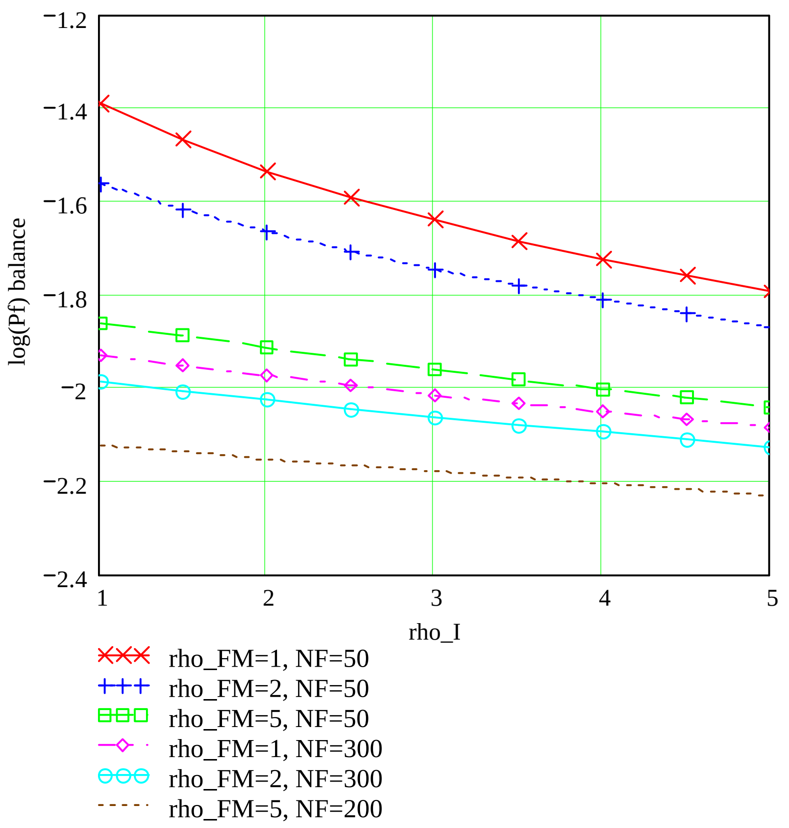

Figure 5-5: The annual failure probability of the old jacket structure that balances the benefit of continuing to use the old jacket structure and investing in a new structure for different values of ρ_{FM} (denoted rho_FM) and expected number of fatalities (NF) and increasing income ρ_I (denoted rho_I).

Multi attribute analysis with managerial decision

The example of life extension or building of a new installation can also be evaluated based on the multi attribute analysis with managerial decision. The expected number of fatalities with structural failure is set to 100, and the annual failure probability of the new installation is 10^{-5}, and for the old installation 10^{-3}. The choice is between building a new structure (D0), and continuing to use the old structure (D1). The indications of consequences, if the structure should fail, are indicated for the two choices as ~~D0~~ and ~~D1~~. This indication of consequences is included in order to ensure awareness of consequences when probabilities are small. In this setting, the risk analysis provides decision support, not the decision itself. Managerial review and judgement in accordance with stakeholders interests are needed to provide the decision. The overview based on the risk analysis as shown in Table 5-1 is the background provided to the decision makers.

Table 5-1: Risk analysis output

	Technical feasibility	Conformance with good practice	Strategic considerations	Economy (MNOK)	Safety	Delta cost for safety improvement
D0	OK	OK	-	E[B]=4000	Pf=10^{-5}	-
~~D0~~				Failure cost -5000	100 fatalities	
D1	OK	Not in agreement with good practice	-	E[B]=4874	Pf=10^{-3}	-
~~D1~~				Failure cost -5000	100 fatalities	

The managerial review and judgement will be presented for the moral dilemma for this decision. The dilemma is not hidden in the decision method, as found when cost benefit analysis is used as the decision method. The dilemmas need to be addressed by the decision makers and the assessments available for evaluation by auditors.

Comments
In this example little emphasis is put on the possible remedies to re-assess and upgrade the old structure. The focus of an assessment of an existing structure for life extension would have a focus on finding alternatives that are better with respect to safety and cost that the two extreme cases presented here. As an example, if thorough inspection and updating of the failure probability from no findings of any damage on the structure is possible, the cost of updating the failure probability would be relatively low. This could decrease our uncertainty about the structure, and alter the assigned probability of failure.

For the new structure it is here assumed that all alternatives like increasing the freeboard, strengthening the structure and the fatigue life of the structure is evaluated in order to improve the safety at small costs. The alternative D0 is the result of this process.

A well maintained structure where owners have managed their installations integrity by a good structural integrity management system would have significantly less uncertainty about the important parameters of the state of the structure than an owner with lacking structural integrity management system and little maintenance. This effect should be accounted for in the assessment in the assignment of uncertainties and in the assigned failure probabilities.

Concluding remark
The predictive Bayesian approach in combination with multi attribute decision making is found to be the most useful approach for probabilistic assessment of existing structures. A general approach for assessment of existing structures for life extension, in line with the predictive Bayesian approach with managerial decision making, would include the following items:

- Model parameters that are a representation of real world data.
- Assign probabilities as measures of uncertainty about these real world parameters, taking into full account the uncertainty that is present with respect to the management of the structures integrity.
- Update the uncertainties when new updated evidence of the real world parameters is found.
- Provide solutions for decision makers. Focus of finding solutions that are good with respect to safety, cost and other aspects that are seen as important for the specific

structure. “Good solutions” are at least reasonable in line with current practice, but achieving the level of current practise is not sufficient and should not be used as acceptance criteria if improvements can be made at reasonable cost.

- Provide the alternatives and the decision support to decision makers.
- The decision is made based on the decision support data by a managerial decision taking into account the requirements from stakeholders.

6 Assessment of system strength

6.1 Collapse analysis of jacket

The standard USFOS program (USFOS 2005) is used for the collapse analysis of the example jackets. In general non-linearity in geometry and material deformation are included. Waves are modelled using the Stokes 5th order theory. The analyses in this thesis do not take dynamic and cyclic loading effects into account. For slender deep water jackets dynamic effects may be significant. However, as this study is primarily on evaluating the RSR parameter for increasing wave height and water depth, this simplification is found acceptable.

6.1.1 Presentation of jackets

In total four jackets have been evaluated for the collapse capacity. The example jackets are all situated on the Norwegian continental shelf and are designed according to Norwegian standards and regulations. Loads and capacities are evaluated according to the NORSOK standards (e.g. NORSOK 1998, 1999 and 2004). The computer models for the jackets have been provided by the respective operators. No changes have been made to the geometry files, but the load files have been changed in order to study the effect of increasing wave height. The geometries of the jackets are presented in Figure 6-1.

The length of the jacket, number of legs, water depth, design wave height and freeboard as used in the analyses are presented in Table 6-1. These environmental data may deviate slightly from what is used in the design analysis.

Table 6-1: Key numbers for the jackets as used in the analyses.

Jacket	Jacket length (m)	Number of legs	Water depth (m)	Design wave (m)	Freeboard (m)
Jacket 1	214.5	4	190	28.5	24.5
Jacket 2	156.4	8	135.5	27.6	21.1
Jacket 3	149.9	4	127.2	27.6	22.7
Jacket 4	92.75	4	70	27	22.75

The water depths given in Table 6-1 are as modelled in the structural analyses. They correspond to the highest astronomical tide and include effects of storm surge. The freeboard given in Table 6-1 is measured from the water level as defined above to the lowest nodes modelled in the topside structure. Hence, the distance from the water level to bottom of steel of deck for these jackets will be slightly less, accounting for the dimensions of the topside components. However, the wave loading on the topside members will not occur before the centreline of the members is wetted by the wave.

Although the jacket models are obtained from real jackets and the analyses are close to realistic, no conclusions should be drawn from these analyses on the specific jackets that are the basis for these models. These analyses are performed with the maximum design wave from all directions. In the design of these jackets directional specific maximum waves are used. Furthermore, as mentioned earlier, dynamic effects are not included in these analyses. Other minor simplification may also apply.

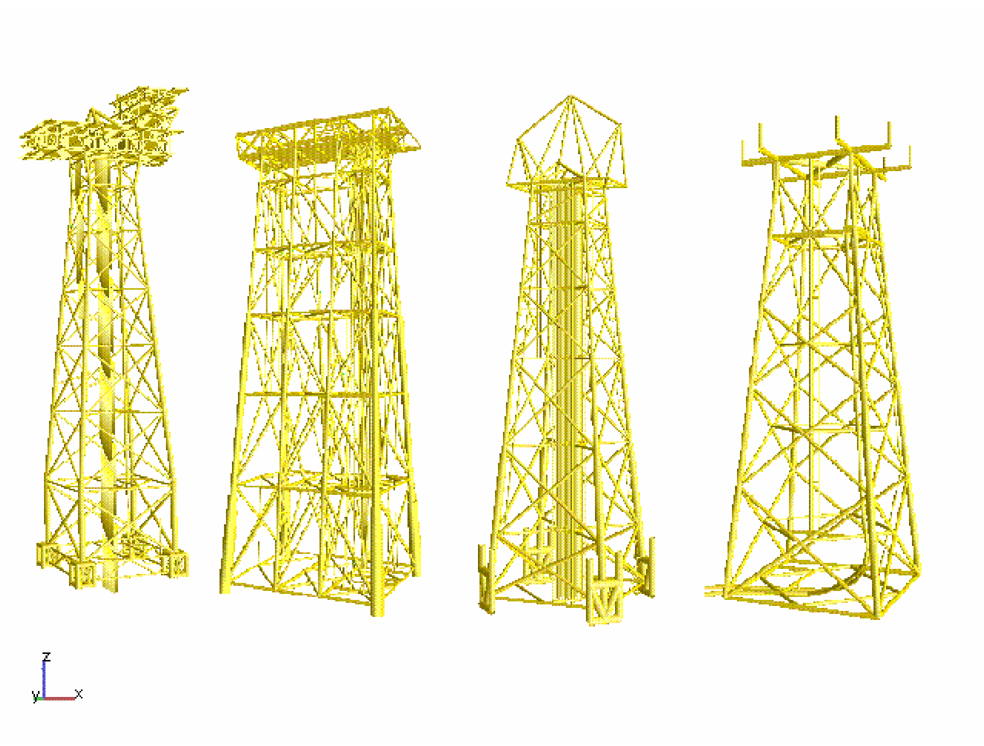

Figure 6-1: Illustration of Jacket 1, 2, 3 and 4 used in the analysis.

6.1.2 Wave loading on the jackets

The wave loading on the jackets are modelled in the USFOS program with the Stoke 5th order wave theory. Analysis with increasing wave height is performed both to evaluate the strength of the structure for higher wave loads and to establish a simplified relationship between the total load on the structure and the corresponding wave height and current speed.

Jacket 1 is analysed for wave heights of 28.5m, 33m, 37m, 40m, 42m, and 44m, current speeds of 0 m/s, 0.6 m/s and 1.2 m/s, and for directions 0 deg, 45 deg, 90 deg, 135 deg, 180 deg, 225 deg, 270 deg and 315 deg. Jacket 2, 3 and 4 is analysed for wave heights of 23m, 27m, 33m, 37m, 40m, 42m, and 44m for the following directions; 0 deg, 45 deg, 90 deg, 135 deg, 180 deg, 225 deg, 270 deg and 315 deg. Jacket 2 is analysed with current speeds of 0 m/s and 0.5 m/s. Jacket 3 and 4 are analysed with current speeds of 0 m/s, 0.7 m/s and 1.4 m/s. The resulting loading as calculated in USFOS for Jacket 1, 2, 3 and 4 are shown in Figure 6-2, to Figure 6-5 respectively. Calculated base shear for waves of the same wave height and current arriving from eight different directions is shown in the figures, resulting in some scatter in the base shear from each wave height. The increased load as waves hit the deck is clearly shown in these figures at 44m for Jacket 1, 40m, 42m and 44m for Jacket 2, Jacket 3 and Jacket 4.

It should be noted that the structures are experiencing wave in deck loading at different wave height, as shown in Table 6-2. Note that Jacket 4 is modelled with significantly larger wave skewness (wave crest to wave height ratio) compared to Jacket 1, 2 and 3. As an example, the wave skewness for a 37 m wave is approximately 0.59 for jacket 1, 2 and 3, while the wave skewness in the analyses of Jacket 4 is 0.65. This increased skewness in the analysis of Jacket 4 is a result of a different modelling of water elevation in order to increase the skewness.

Table 6-2: Wetted height of topside for different wave heights for the four jackets

Jacket	Freeboard (m)	Wetted height of topside for 37 m wave	Wetted height of topside for 40 m wave	Wetted height of topside for 42 m wave	Wetted height of topside for 44 m wave
Jacket 1	24.5	-	-	0.3 m	1.73 m
Jacket 2	21.1	0.8 m	2.93 m	4.22 m	5.65 m
Jacket 3	22.7	-	1.16 m	2.58 m	4.04 m
Jacket 4	22.75	1.35 m	3.84 m	5.36 m	6.94 m

Modelling of wave in deck is not carefully performed for any of these analyses. However, as the purpose is not to carefully evaluate the load on the topside at wave-in-deck, but rather to evaluate the jacket performance for (any) load in different positions, a simplified modelling of wave in deck is acceptable. Hence, the topside structure is exposed to the waves with the available computer model, and the resulting wave in deck loading is checked against reasonable numbers.

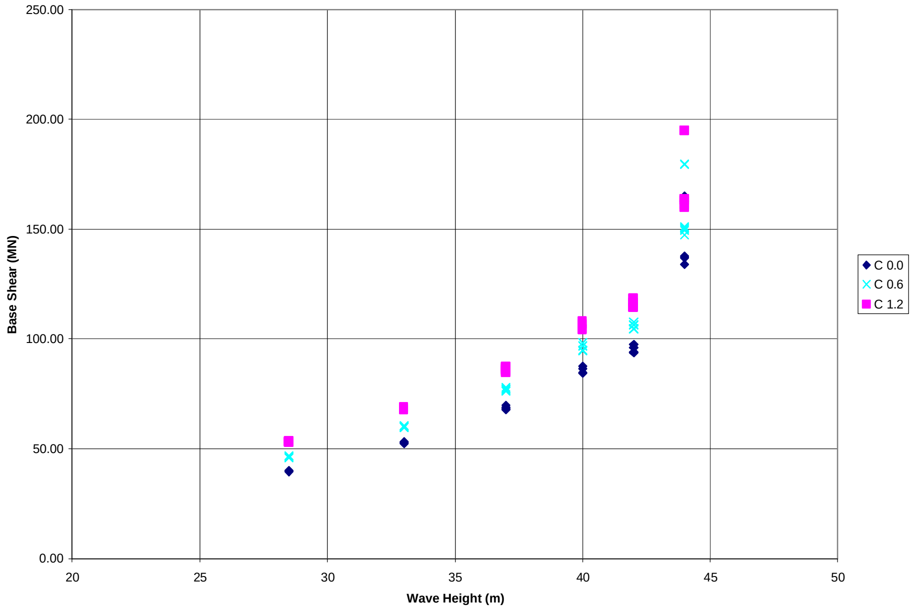

Figure 6-2: Wave and current load as a function of wave height for Jacket 1. The legend C 0.0, C0.6 and C 1.2 denotes current speed of 0.0m/s, 0.6m/s and 1.2m/s respectively.

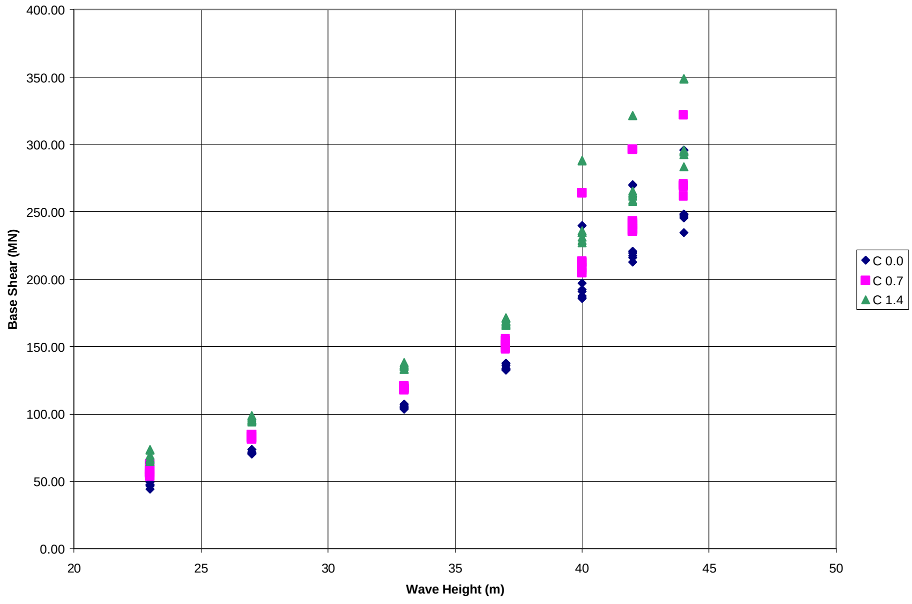

Figure 6-3: Wave and current load as a function of wave height for Jacket 2. The legend C 0.0, C 0.7 and C 1.4 denotes current speed of 0.0m/s, 0.7m/s and 1.4m/s respectively.

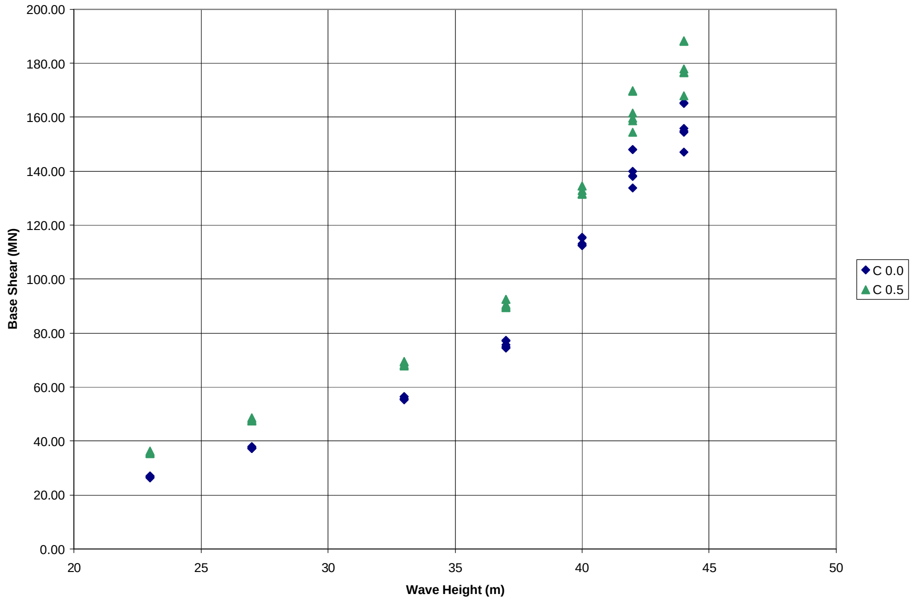

Figure 6-4: Wave and current load as a Function of Wave Height for Jacket 3. The legend C 0.0 and C 0.5 denotes current speed of 0.0m/s and 0.5m/s respectively.

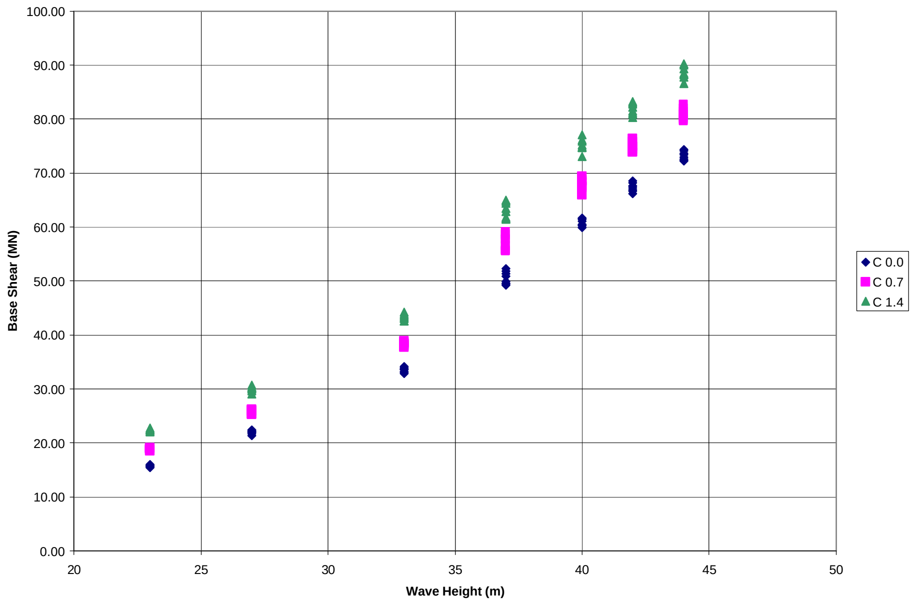

Figure 6-5: Wave and current load as a Function of Wave Height for Jacket 4. The legend C 0.0, C 0.7 and C 1.4 denotes current speed of 0.0m/s, 0.7m/s and 1.4m/s respectively.

Wave in deck loading
With the modelling performed in these analyses, USFOS calculates the wave forces on topside members when the wave hits the deck according to Morrison formula (see e.g. NORSOK 1999). A more correct estimate of the slamming load on a plane area may be written as (ref Vinje 2002):

$$F_s = \rho \cdot u \cdot c \cdot A \tag{6.1}$$

where ρ is the density of water, *u* the particle velocity in the wave, *c* is the wave velocity and *A* is the exposed area.

Based on the USFOS analysis of Jacket 1, it can be seen that an extrapolation of the wave load data without wave in deck loading would result in a wave loading of approximately 105MN from a 44m wave and zero current. The actual wave load calculated by USFOS for this wave is approximately 140MN. The wave in deck contribution is then around 35MN for a wave hitting 1.76m into the deck. The deck has a width of around 100m.

The more correct formula for wave-in-deck slamming force can then be applied to evaluate the force calculated by USFOS. The density of water is 1025 kg/m^3, the wave crest height 26.23m and the wave period 15.3s resulting in a particle velocity of 10.77m/s and a wave group velocity of 23.87m/s (calculated based on linear wave theory). The exposed area is 1.76m high and 100m in breath. The resulting slamming load should then be 46MN. Hence, the loading in the USFOS analysis is a low estimate, but within a reasonable approximation for the purpose of this analysis. The wave in deck forces on Jacket 2, 3 and 4 do not deviate significantly from the wave in deck loading on Jacket 1, and they are also found acceptable for the purpose of this investigation.

Parameterisation of wave and current loading
A simplified expression of the horizontal wave load can be formulated by (see e.g. Heideman 1980):

$$F = C_1 \cdot (H + C_2 \cdot u)^{C_3} \tag{6.2}$$

where *H* is the wave height, *u* is the current speed and C_1, C_2 and C_3 are parameters that have to be adjusted to fit the calculated loading.

If wave in deck loading is taken into account, the load can be taken as (Haver et al 2002):

$$\begin{aligned} F &= C_1 \cdot (H + C_2 \cdot u)^{C_3} \\ &\quad + C_4 \cdot L \cdot (C_5 \cdot H - FB + C_6) \quad \text{if } C_5 \cdot H + C_6 > FB \end{aligned} \tag{6.3}$$

where *L* is the width of the exposed deck frame, C_4 is the impact pressure, C_5 is the asymmetry of extreme waves, C_6 represents the possible sea structure interaction build up of the wave crest and *FB* is the freeboard. For jacket structures the C_6 parameter is assumed ~ 0.

As the wave in deck modelling in the USFOS analyses is rather coarse, only the C_1, C_2 and C_3 parameters are extracted from the analysis. The C_4 and C_5 factors are taken in accordance with Haver et al (2002), where C_4 = 0.25 MN/m^2 and C_5 = 0.62. With the proposed C_4 factor, the wave-in-deck wave load on Jacket 1 would be 0.25·100·1.76 = 44MN, which is a very close approximation of the estimate of 46MN.

Eye-fitting of the C_1, C_2 and C_3 parameters are performed and shown for Jacket 1 in Figure 6-6 with C_4 and C_5 factors according to Haver et al (2002). Similar curve fitting are made for Jacket 2, 3 and 4, and the parameters from the curve fitting are shown in Table 6-3. For all four jackets evaluated in this thesis, the best fit of the C_3 factor is close to 2.2. Hence, to minimise the number of variable factors, the C_3 factor is kept constant for all the jackets.

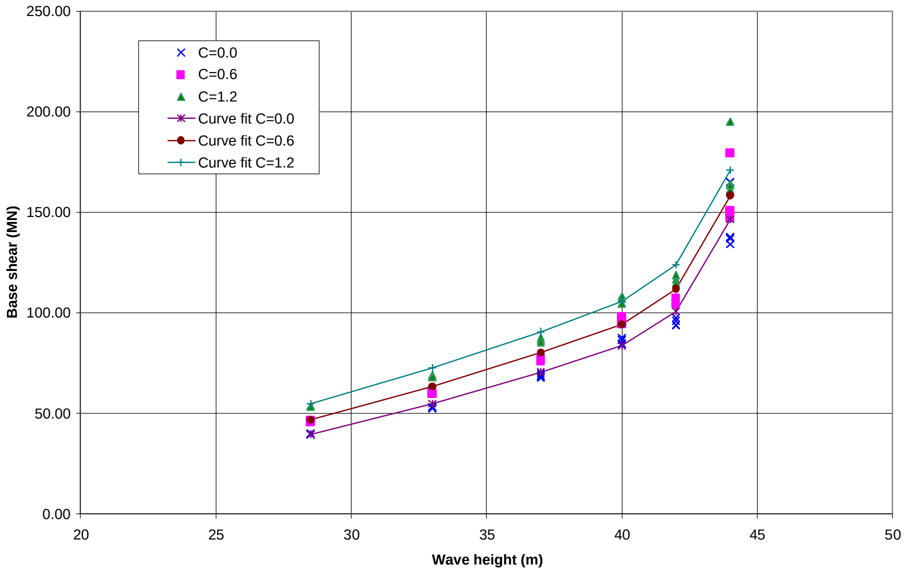

Figure 6-6: Calculated base shear and curve fitted base shear for Jacket 1. The legend C 0.0, C 0.6 and C 1.2 denotes current speed of 0.0m/s, 0.6m/s and 1.2m/s, respectively.

Table 6-3: Parameters for wave loading based on curve fitting to Equation 6.2

Jacket	C_1	C_2	C_3
1	0.025	3.4	2.2
2	0.048	3.5	2.2
3	0.027	6.5[14]	2.2
4	0.016	2.95	2.2

The C1 factor in Table 6-3 is dependent on the size of the load area of the jacket. Jacket 2 with its eight legs has the largest load area, and Jacket 4 has the lowest. An indication of the load area may be obtained by the weight of the jacket substructure divided by the length of the jacket. This would be incorrect for a jacket with larger steel thickness then usual, and would not take into account the load area of the conductors and risers. However, it may work as an indication. The weights of the jackets are 11500 ton, 18500 ton, 6100 ton and 2890 ton, respectively. The C_1 factor is plotted against Weight / Length of jacket in Figure 6-7. No

[14] The rather large C2 factor for jacket 3 may be due to that jacket 2 is modelled with a slab current profile (constant current from sea-surface to sea-bottom), whereas the other jackets are modelled with a profile decreasing from sea-surface to sea-bottom.

conclusive relationship can be drawn from Figure 6-7 between the C_1 factor and the weight / length of the jacket, but the development of C_1 seems reasonable.

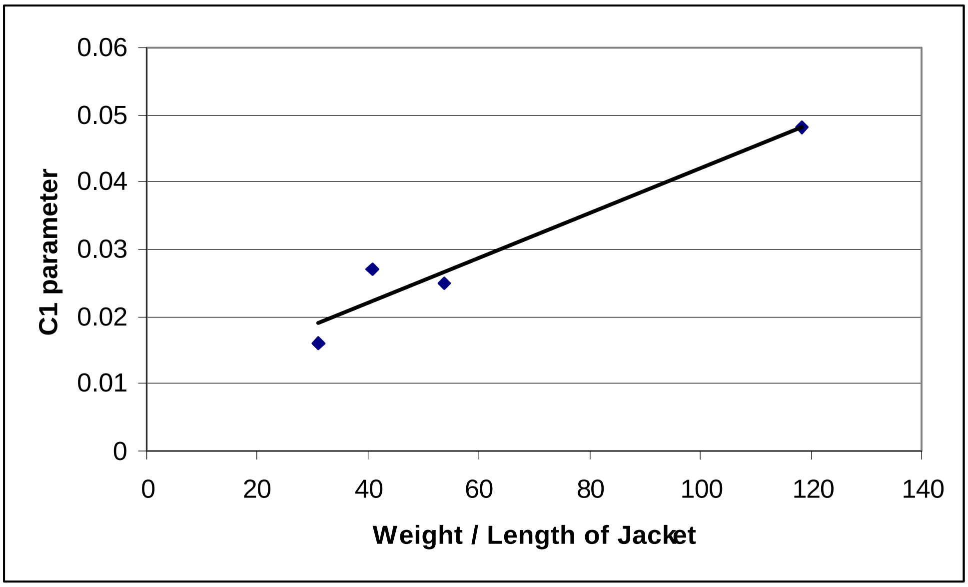

Figure 6-7: Trend-line for the C_1 factor (Equation 6.2) versus weight / length of jacket.

6.1.3 Collapse analysis of jackets

Collapse analysis (push-over analysis) is performed in the USFOS program to estimate the collapse capacity of the jackets. In USFOS, the loading is defined typically by a permanent load case (e.g. weight of topside and jacket, equipment weight on topside and buoyancy of the jacket) and by an environmental (wave, wind and current) load case. The permanent load case is first applied to the structure (in steps if necessary), and then the environmental load case is added to the structure in steps[15]. The environmental load is stepped onto the structure until the structure collapses. The collapse of the structure is defined by lack of ability to withstand the load. In this context the collapse load is defined as the maximum load the structure can withstand, before the load-deflection curve starts a negative trend.

The factor on the defined environmental load case at collapse is one of the outputs from the analysis. This factor is called the Reserve Strength Ratio (RSR), and is relative to the input environmental load case (see Equation 3.13). Generally, the reference load is the load obtained when using the wave corresponding to an annual exceedance probability of 10^{-2} together with an appropriate current profile. The choice of current profile varies considerably from the 10^{-2} annual probability current profile to the conditional mean current profile given the wave event.

[15] The stepping of the load is necessary in most cases due to numerical updating of the stiffness matrix in a non-linear analysis (see e.g. Crisfield 1996).

The reserve strength ratio (RSR) is a general and useful term for describing the strength of the structure. RSR, however, has some limitations. It is defined as the ratio between the collapse base shear and the design base shear. However, the collapse base shear is calculated by scaling the design load pattern on the structure until collapse. In practise, this is an unrealistic loading on the structure. The load on the structure will rather increase due to a larger wave, especially as a wave reaches the deck, than due to what's modelled in the typical RSR analysis. The purpose of the investigations in this section is to evaluate how sensitive the reserve strength ratio (RSR) of the example jackets is for increased wave height and increased water depth (due to subsidence or general increase of water level). If the RSR shows limited sensitivity to the wave height and water depth, the RSR alone could be a proper measure of the safety of the structure with respect to excessive wave loading. As the reserve freeboard ratio is chosen as a factor for giving information about the wave in deck problem, it is in the RSR evaluations focussed on how sensitive the structure is for waves not reaching the deck level.

If the reference wave height deviates from the design wave height, the reference wave for the RSR is denoted in this thesis as:

$$RSR(h) = \frac{UC}{BS(h)} \tag{6.4}$$

where *UC* is the ultimate capacity of the structure in an non-linear system analysis (the collapse base shear) and *BS(h)* is the base shear loading on the jacket with wave height *h.*

The $RSR(h)$ will then of course be decreasing with increasing environmental loading, as shown in Figure 6-8 for Jacket 1.

The collapse base shear load of the jacket structure is found by multiplying the $RSR(h)$ with the base shear from the reference environmental loading. As shown earlier, the base shear is increasing when wave height increases, and the $RSR(h)$ is decreasing as shown in Figure 6-8. Hence the interesting part is to evaluate the collapse base shear for increasing wave heights. The collapse base shear is here defined as $CBS = BS(h) \cdot RSR(h)$.

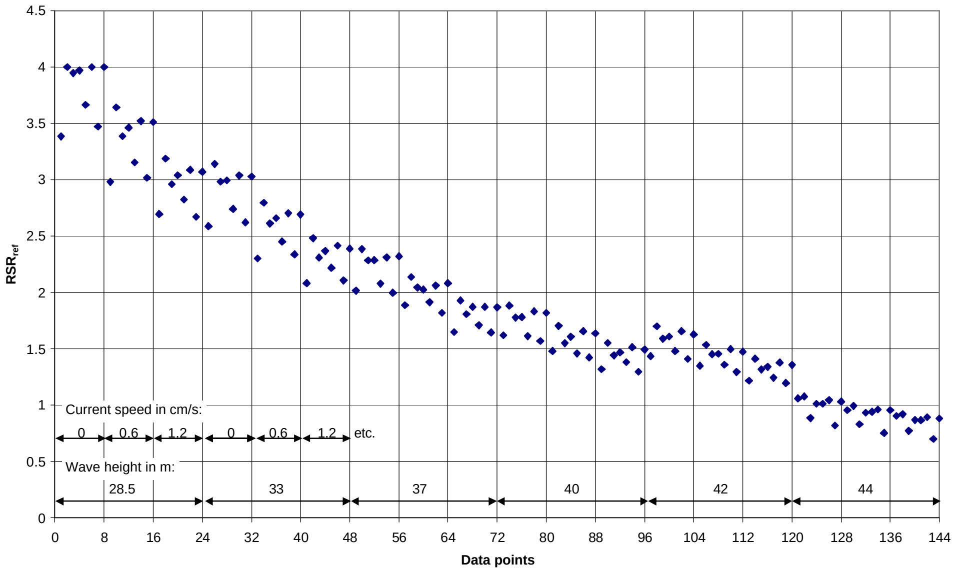

Figure 6-8: Reserve strength ratio with reference in the base shear loading from the actual wave height ($RSR(h)$) for increasing wave heights for Jacket 1. Result from analysis with eight different wave and current directions are plotted for each current speed. The current speed with associated current profile used in the design analyses are 1.2 m/s.

An illustration of the overall trends of the collapse base shear (CBS) for the four jackets are given in Figure 6-9, Figure 6-10, Figure 6-11 and Figure 6-12 for Jacket 1, 2, 3 and 4 respectively. For each wave height in Figure 6-9, Figure 6-10, Figure 6-11 and Figure 6-12, the collapse base shear for eight different directions and 2 or 3 different current speeds are marked, causing the observed scatter in the results.

The rather large scatter in the collapse base shear is to some extent due to directional differences in the strength of the structure. The largest scatter can be seen for Jacket 2 in Figure 6-10. If the data are plotted against the wave direction rather than the wave height – see Figure 6-13, the directional differences in strength are clearly observed. For each direction in Figure 6-13 all wave heights and current speeds are included, causing a scatter also in this plot. For Jacket 1, 3 and 4 a certain higher collapse base shear is seen for the diagonal cases (waves arriving from 45 deg, 135 deg, 225 deg and 215 deg). For Jacket 2, which is an eight legged jacket, the strong axis is clearly seen as for waves arriving from 90 deg and 270 deg.

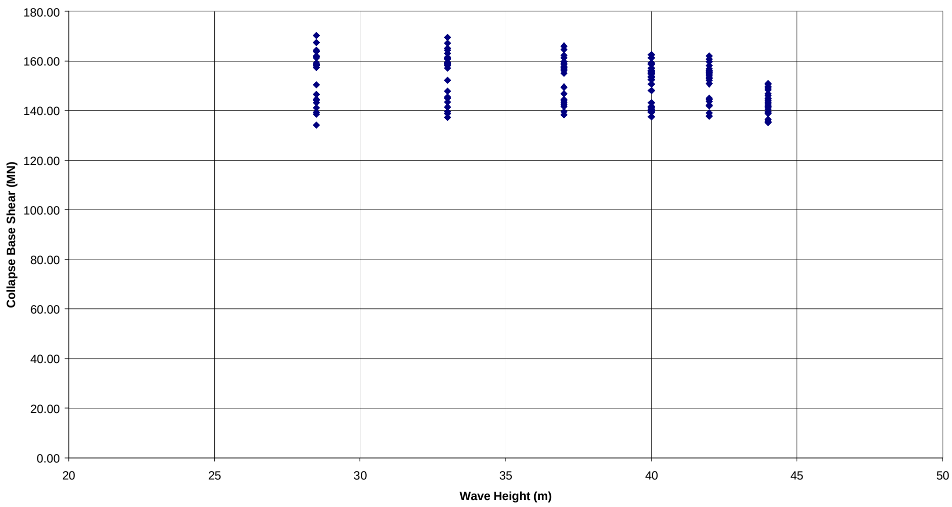

Figure 6-9: Collapse base shear for increasing wave height for Jacket 1.

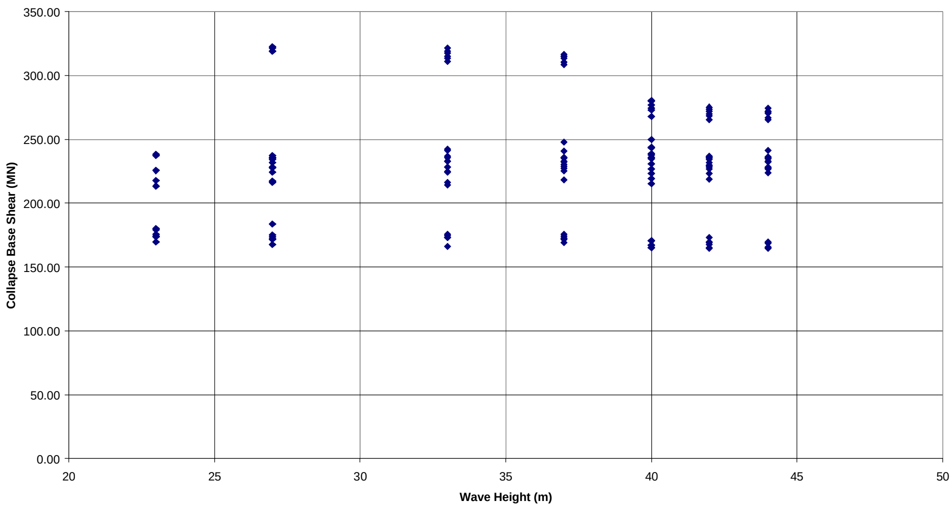

Figure 6-10: Collapse base shear for increasing wave height for Jacket 2.

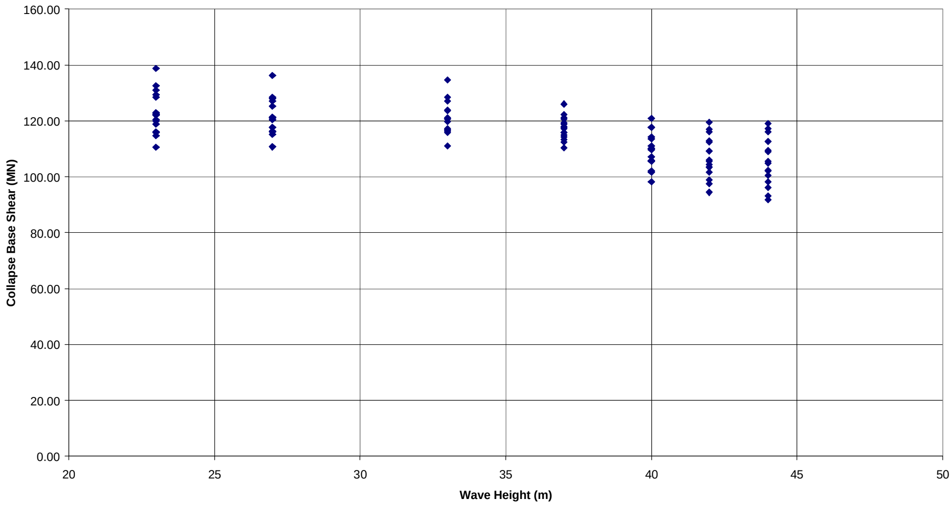

Figure 6-11: Collapse base shear for increasing wave height for Jacket 3.

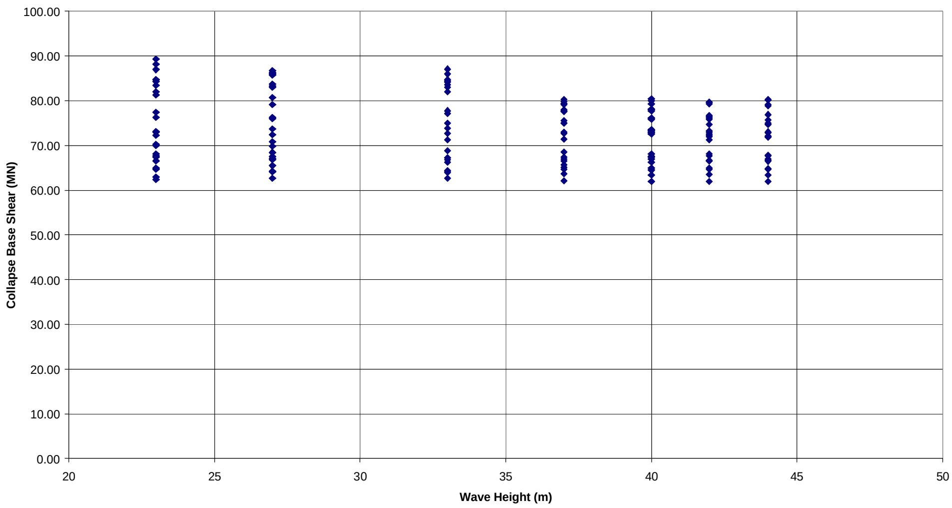

Figure 6-12: Collapse base shear for increasing wave height for Jacket 4.

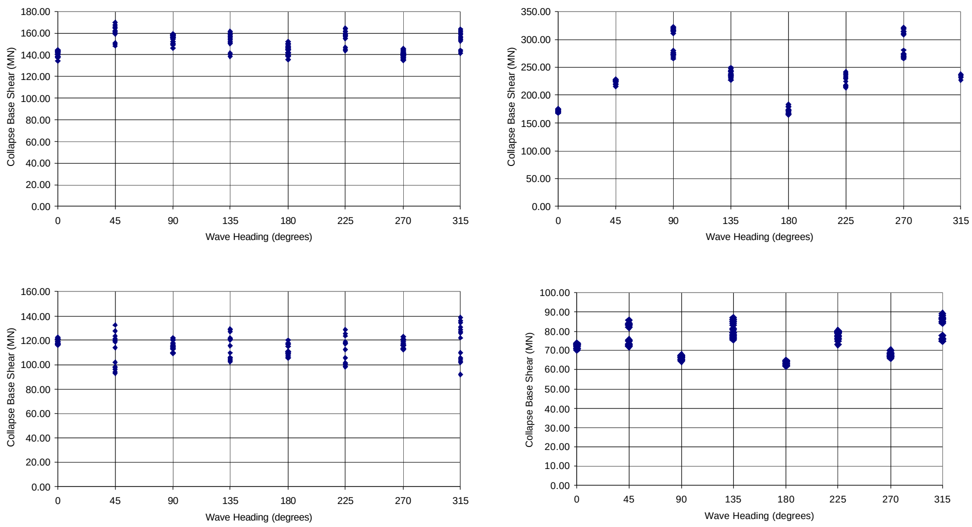

Figure 6-13: Collapse base shear for different wave directions for Jacket 1 (top left), Jacket 2 (top right), Jacket 3 (bottom left) and Jacket 4 (bottom right).

For a detailed insight into the performance of the collapse base shear for the jackets, the collapse base shear is studied for increasing wave height for each wave direction and for different current speeds. The development of the collapse base shear for each of the Jackets are plotted against wave height for each direction in Figure 6-14 - Figure 6-17.

Based on Figure 6-14 - Figure 6-17, it can be observed a tendency of a decreasing collapse load for the jackets when the wave increases. In some cases (e.g. Jacket 1 for diagonal directions) this decrease is rather slow, while in other cases the reduction is more sudden. The latter observation is linked to the wave hitting the deck (e.g. Jacket 2 for directions 45 degrees and 225 degrees). The explanation for this is that when the wave height increases, the moment arm of the load is increasing. Hence the overturning moment is influencing the capacity. This is in line with the observation that the decrease is most pronounced for the diagonal directions, where moment is taken to a large extent by axial forces in the leg members. For the head on directions the wave load will be to a larger extent be taken by a shear load in the braces, and in this case the moment is of less importance.

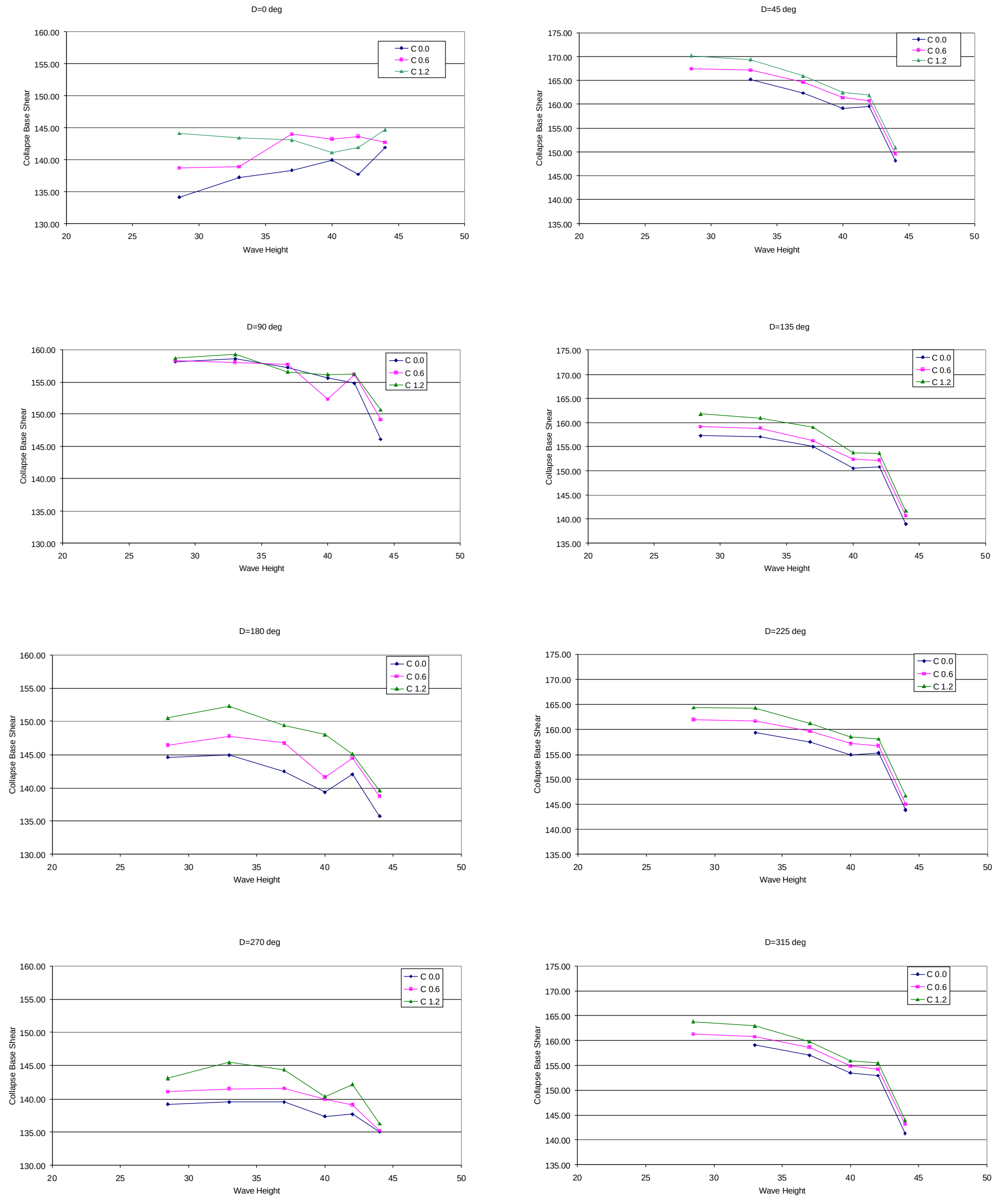

Figure 6-14: Collapse base shear for different wave directions for Jacket 1, plotted for each direction. Base shear is given in mega Newton (MN) and wave height is given in meters. The legend C 0.0, C0.6 and C 1.2 denotes current speed of 0.0m/s, 0.6m/s and 1.2m/s, respectively. D denotes the direction of the incoming wave and current.

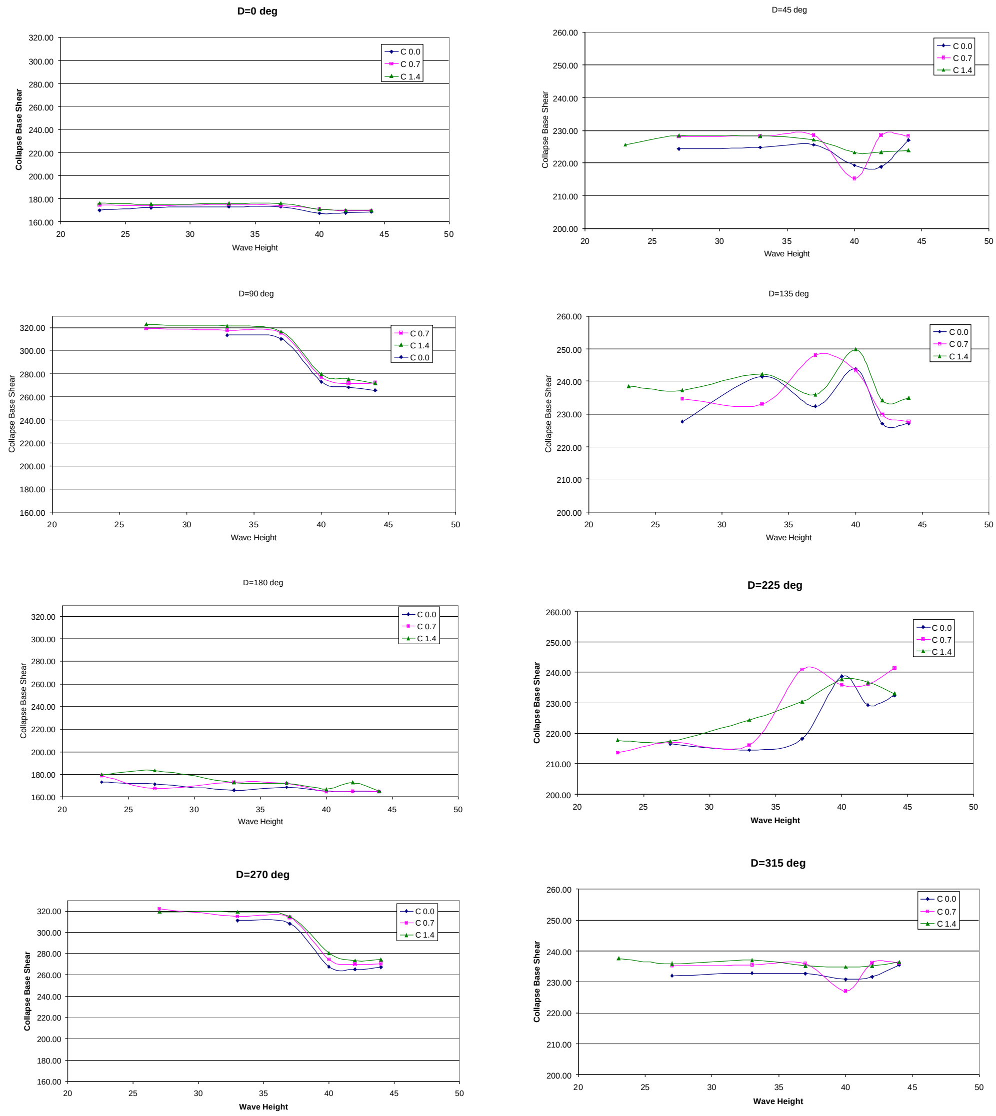

Figure 6-15: Collapse base shear for different wave directions for Jacket 2, plotted for each direction. Base shear is given in mega Newton (MN) and wave height is given in meters. The legend C 0.0, C0.7 and C 1.4 denotes current speed of 0.0m/s, 0.7m/s and 1.4m/s, respectively. D denotes the direction of the incoming wave and current.

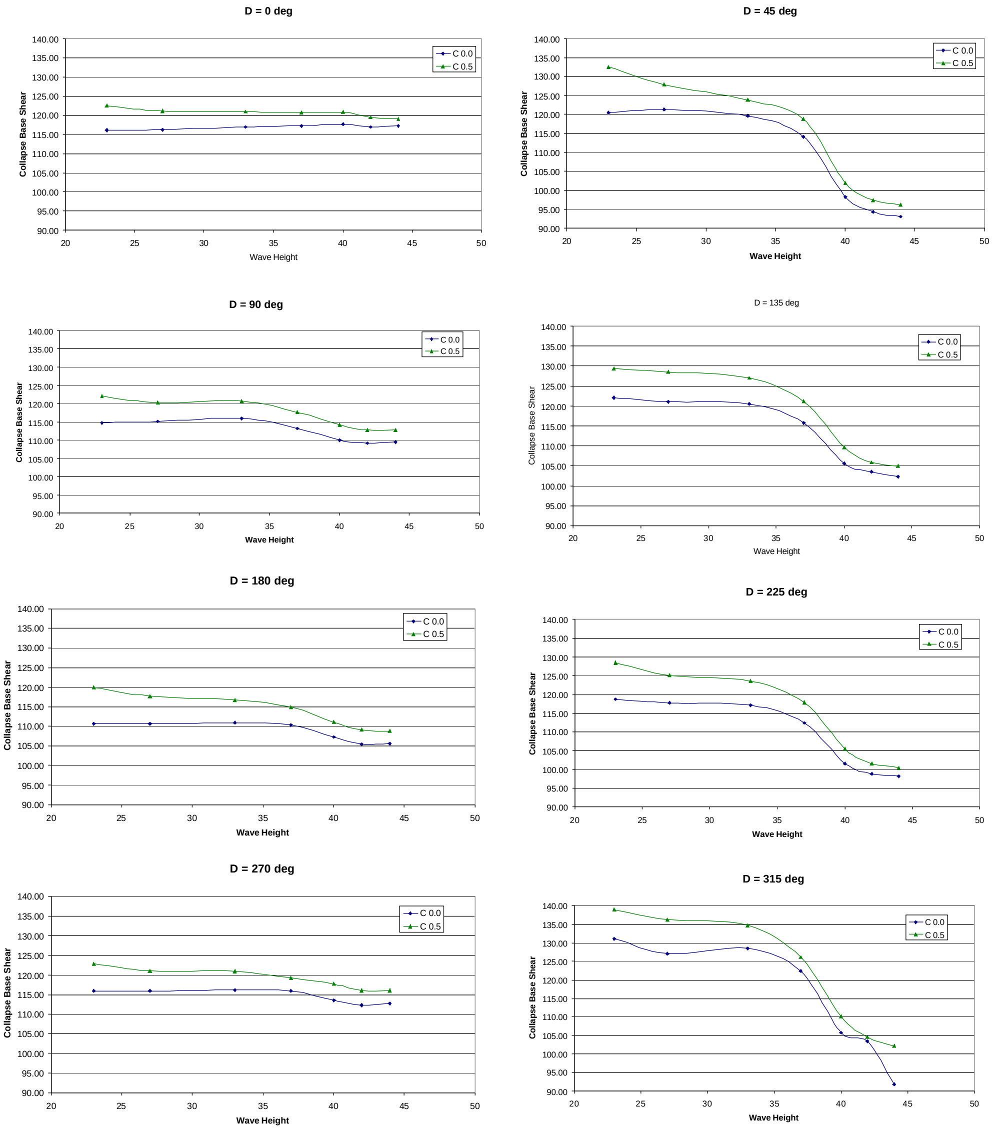

Figure 6-16: Collapse base shear for different wave directions for Jacket 3, plotted for each direction. Base shear is given in mega Newton (MN) and wave height is given in meters. The legend C 0.0 and C0.5 denotes current speed of 0.0m/s and 0.5m/s, respectively. D denotes the direction of the incoming wave and current.

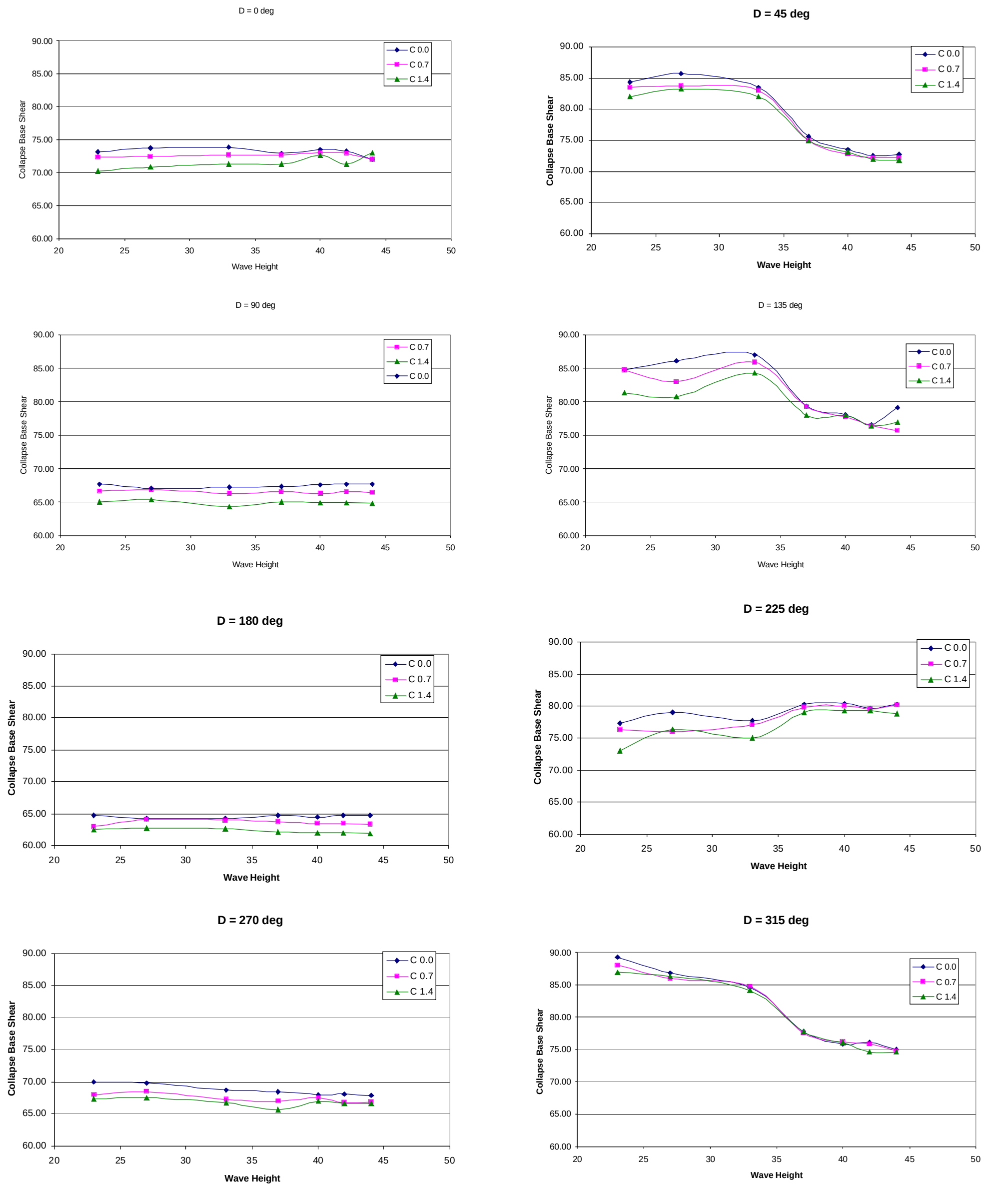

Figure 6-17: Collapse base shear for different wave directions for Jacket 4, plotted for each direction. Base shear is given in mega Newton (MN) and wave height is given in meters. The legend C 0.0, C0.7 and C 1.4 denotes current speed of 0.0m/s, 0.7m/s and 1.4m/s, respectively. D denotes the direction of the incoming wave and current.

If the collapse loads of the jackets are determined by the overturning moment, the collapse overturning moment should be constant with increasing wave height. The collapse overturning moment is shown for Jacket 1 in Figure 6-18 and Figure 6-19. It can be seen from these figures that there is no sign of decreasing collapse overturning moment in diagonal sea or head on sea. Hence, the slightly decreasing collapse base shear can be concluded to be a result of the increased moment arm of the wave force.

For the more sudden changes of capacity when the wave hits the deck, it is likely that less strong areas in the upper part of the jacket are exposed to loading. These areas may then dominate the capacity of the structure. The most dominant drop in collapse base shear is found for Jacket 2 for waves from 90 degrees and 270 degrees. The collapse overturning moment for this case is shown in Figure 6-20. There is also a clear decrease in the collapse overturning moment, indicating a possible weaker area of the jacket when wave in deck loading occurs.

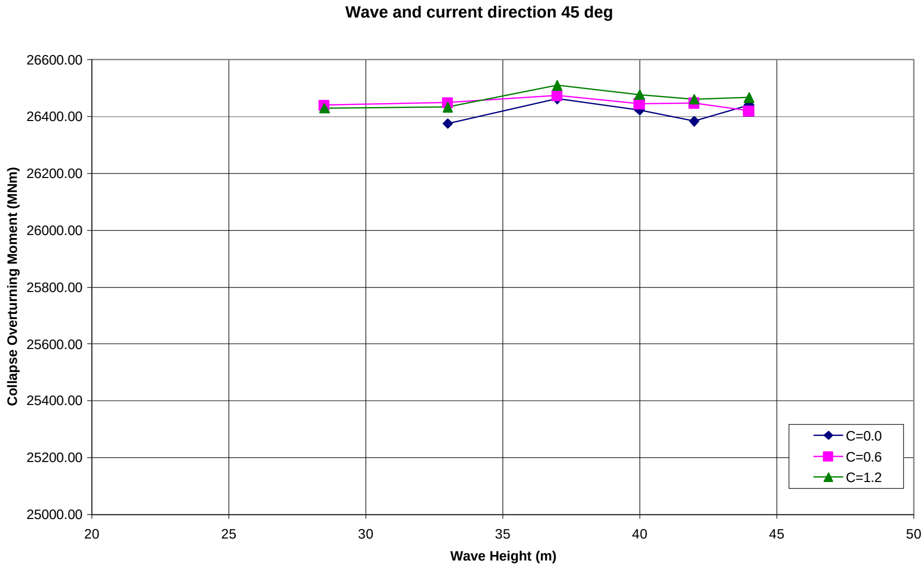

Figure 6-18: Collapse overturning moment for waves from 45 degrees for Jacket 1. Overturning moment is given in mega Newton meters (MNm) and wave height is given in meters (m). The legend C 0.0, C0.6 and C 1.2 denotes current speed of 0.0m/s, 0.6m/s and 1.2m/s, respectively.

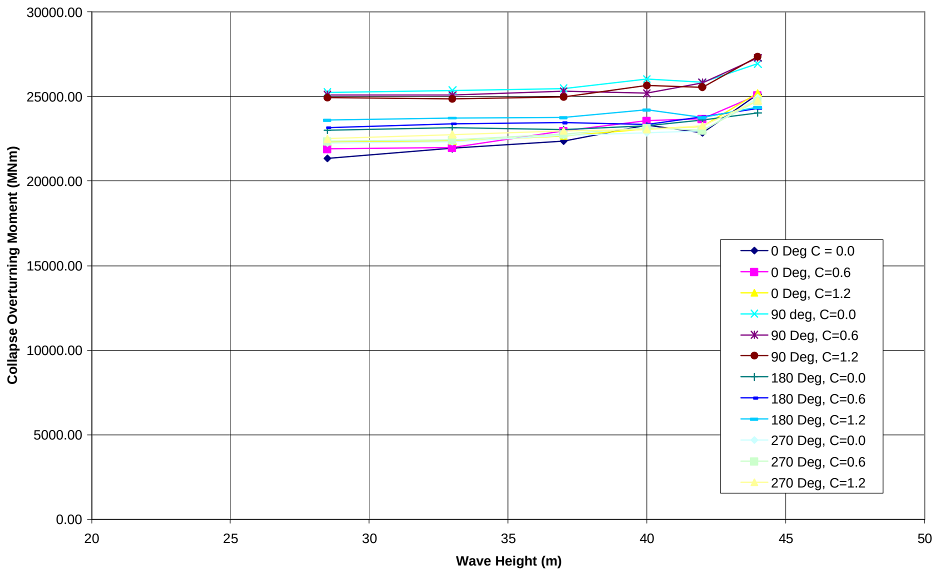

Figure 6-19: Collapse overturning moment for waves from head-on directions for Jacket 1. Overturning moment is given in mega Newton meters (MNm) and wave height is given in meters (m). The legend C 0.0, C0.6 and C 1.2 denotes current speed of 0.0m/s, 0.6m/s and 1.2m/s, respectively.

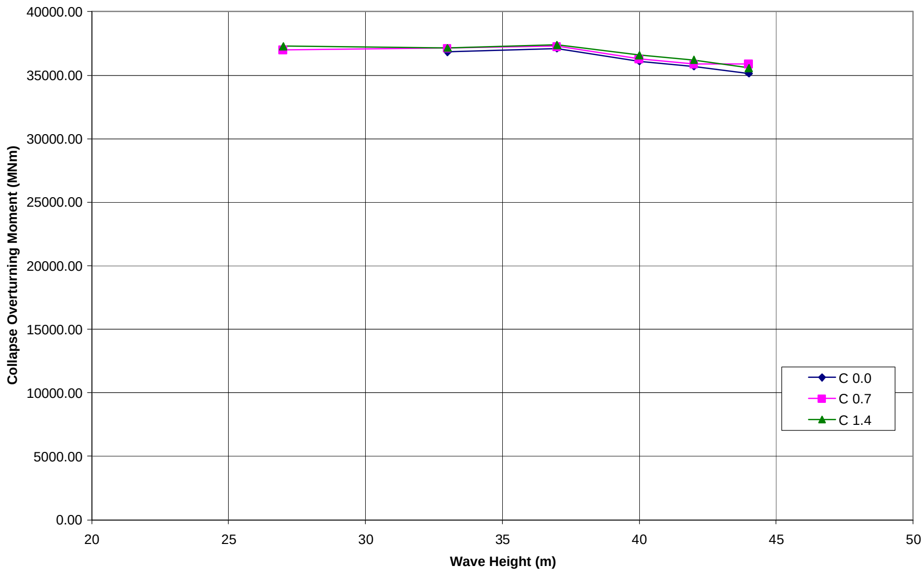

Figure 6-20: Collapse overturning moment for waves from 90 degrees for Jacket 2. Overturning moment is given in mega Newton meters (MNm) and wave height is given in meters (m). The legend C 0.0, C0.7 and C 1.2 denotes current speed of 0.0m/s, 0.7m/s and 1.2m/s, respectively.

Figure 6-21: Jacket 2 exposed to a 37m wave

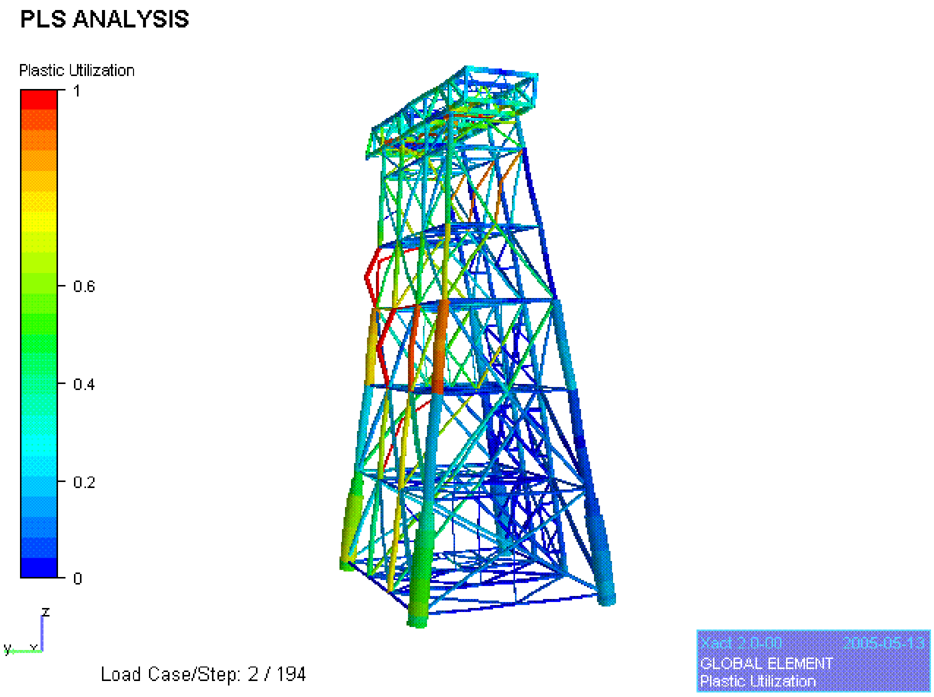

Figure 6-22: Jacket 2 exposed to a 40m wave

The effect of decreasing collapse base shear of the structure with increasing wave may be modelled as a reduction factor of the collapse load. The factor is in this thesis called NFM (New Failure Modes), and is a function of the wave height accounting for the new failure modes when the wave hit at higher levels and introduces loads in weaker areas of the jacket. Based on the analyses of these jackets the NFM factor is in all cases less than 1.1 prior to wave hitting the deck, and 1.16 in the most severe case after wave in deck has occurred.

The indication of a new failure mode is not easily seen from the deformation and stress plots from the USFOS analyses as shown in Figure 6-21 and Figure 6-22 for Jacket 2 exposed to a 37m and a 40m wave respectively. The braces in bay 2 and bay 4 seems to be the critical elements for the jacket exposed to the 37m wave, while the leg elements seems to be the critical elements for the 40m wave loading.

Some smaller variation is also observed, that is not evidently explainable. This variation may be due to numerical noise.

6.1.4 The effect of subsidence or increasing water level

Subsidence or increased water level would result in similar effect as the increased wave height, by increasing the moment arm of the wave resultant force and possibly introducing loads in weaker areas of the structure. As a result there is not found any reason for a separate study of the collapse strength of the jackets with increasing depth. However, the effect of possible new failure modes (illustrated by the NFM factor) should both be regarded as depending on the wave height increase and the actual water depth increase.

6.1.5 Effect of increased topside loading

Increasing the gravity load and live load on the topside will affect the collapse base shear. This effect is studied for Jacket 1 by factorizing the topside loading with a factor varying from 0.6 - 2 times the original topside loading. The resulting collapse base shear is shown in Figure 6-23 for 3 wave headings.

The decrease in the collapse base shear is most distinct for the diagonal case. This is reasonable as both the topside loading and the wave loading is carried by the jacket legs for this wave heading. For the head on direction (0•, 90•, 180• and 270•) the wave load is to some extent carried by the braces, and a weaker influence is seen.

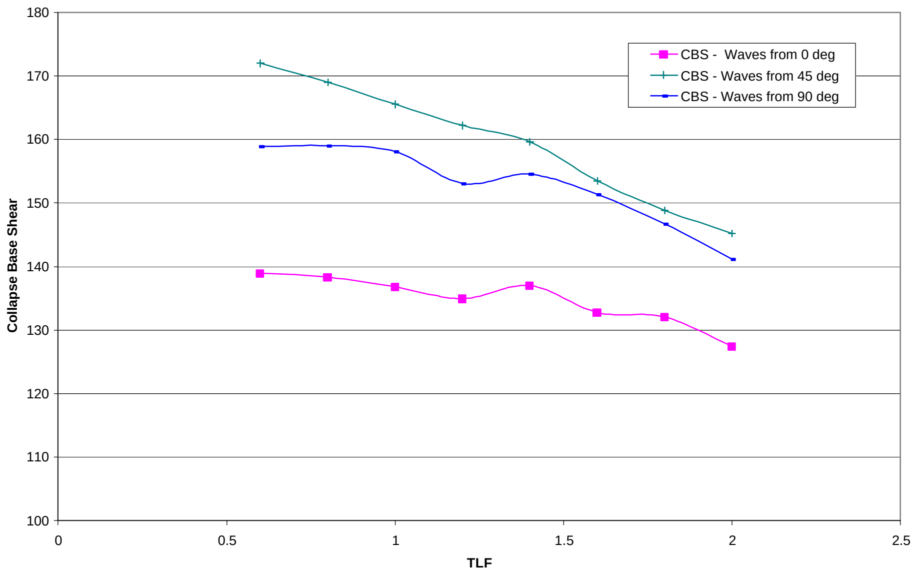

Figure 6-23: The effect on the collapse base shear with a change in topside loading for Jacket 1 (Topside gravity Load Factor - TLF). Base shear is given in mega Newton (MN) and wave height is given in meters.

6.2 Structural redundancy (SR)

The structural redundancy (SR) is a measure of the load level at first member failure to the collapse load of the structure. It is used later in a deterministic development of a RSR acceptance criterion. The SR is calculated in the USFOS analysis. The load level at the time of first member failure is found and compared with the final collapse load level.

$$SR = \frac{Q_{collapse}}{Q_{first\_member\_failure}} \tag{6.5}$$

Similar studies are reported in Skallerud and Amdahl (2002), where the ratio between first member failure and collapse load is found to be in the range between 1.0 - 1.38 with an average of 1.1. A SR=1.0 indicates that the first member failure occurs at structural system collapse. A SR=1.38 indicates a significant redundancy in the structure, as the structural collapse does not occur until the loading is increased with 38% from the load level at first member failure.

Similar results are found in the present analyses. The calculated structural redundancy (SR) is shown in Figure 6-24 to Figure 6-26. Note that Jacket 3 is modelled with piles and soil around the piles. The first "member" to obtain plastic deformations is the soil around the piles. This occurs at very low levels of loading, and hence the results for Structural Redundancy are very high, but irrelevant for this study. Jacket 3 is due to this not included in this exercise. Note also that the USFOS analysis is a geometry and material non-linear analysis. Some of the results obtained from such analysis will due to limited number of numerical iterations not necessarily find equilibrium within the yield surface defined for each member. In the following figures, some analysis where the analysis has found an equilibrium exceeding the yield surface significantly has been removed from the plot. Additional analysis with increased number of iterations would normally find an acceptable equilibrium also for these analyses.

The structural redundancy found from the analysis of Jacket 2, Figure 6-25, indicates significantly larger scatter then the results for Jacket 1 and Jacket 4. It should be noted that Jacket 2 is an eight legged jacket, where waves from 90 degrees and 270 degrees will be along the strong axis of the jacket. The scatter observed can to some extent be explained by the additional redundancy that can be expected along the strong axis of the jacket. However, there may also be some components that fail due to local wave loadings for the highest waves in the analyses. If this component failure does not affect the global capacity of the structure substantially, this may cause a high SR value as shown in Figure 6-25.

In order to give a more complete explanation of the scatter in these SR plots, the first member to fail in each analysis would need to be identified. Also, the load level at first member failure would be important. Such a thorough investigation into the scatter of these results is not performed. Hence, a general conclusion on the typical SR value for these jackets is not possible to achieve.

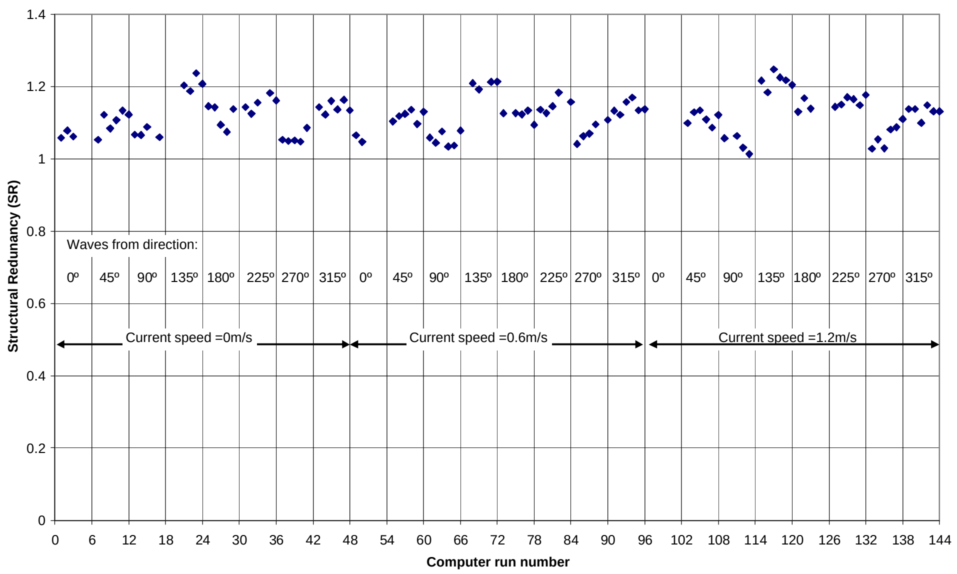

Figure 6-24: Structural Redundancy (SR) factor for Jacket 1. Result from analysis with six different wave heights are plotted for each wave and current directions.

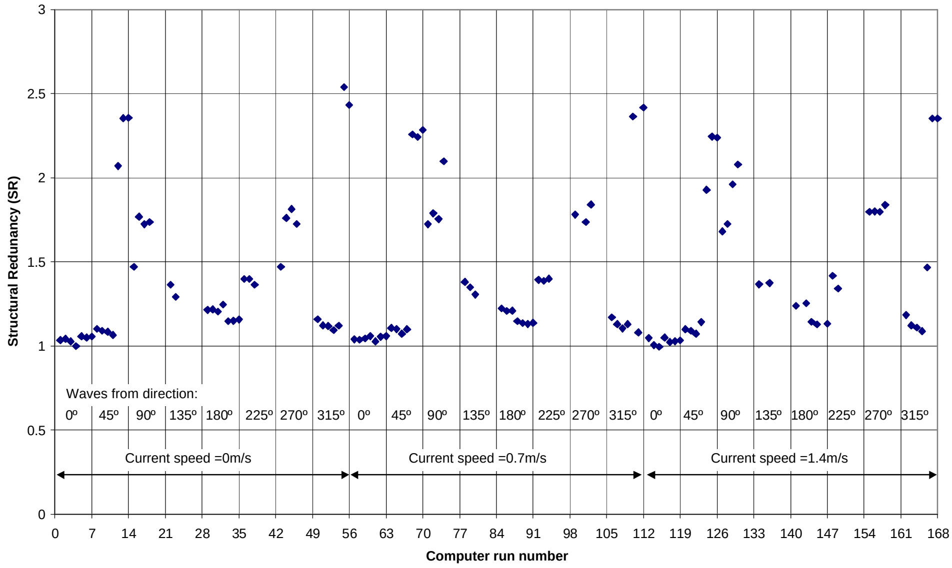

Figure 6-25: Structural Redundancy (SR) factor versus wave direction for Jacket 2

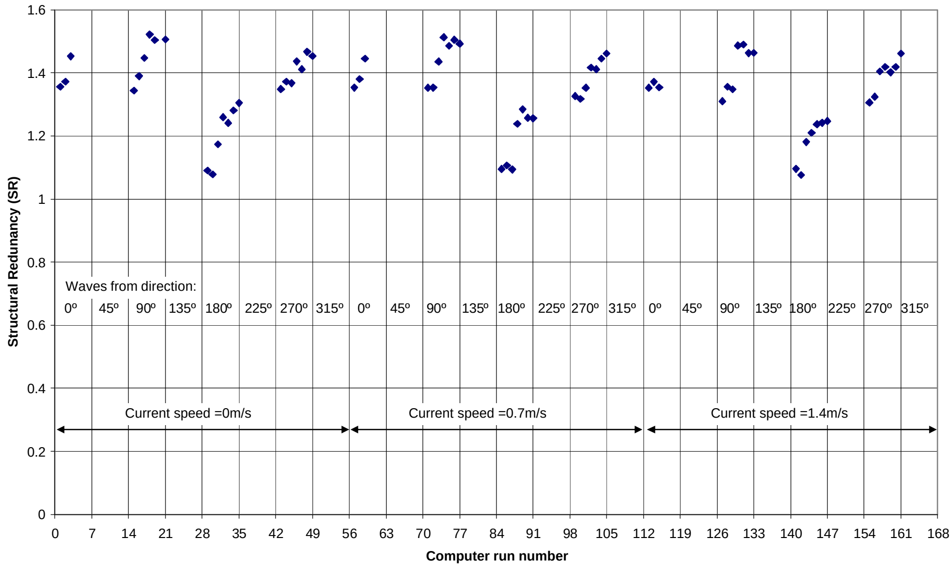

Figure 6-26: Structural Redundancy (SR) factor versus wave direction for Jacket 4

6.3 Damaged Strength Ratio (DSR)

Damaged strength ratio (DSR) is defined as the ratio between the collapse environmental load on the structure in damaged condition (one member has failed or been severely damaged) and the design environmental load (with a reference probability level for the design environmental load of 10^{-2} of annual exceedance probability – as for the RSR). The DSR may be defined as (see Equation 3.13):

$$DSR = \frac{Q_{damaged}}{Q_{design}} \tag{6.6}$$

The relative reduction of capacity from the intact state to the damaged state may be expressed by the Residual Strength Factor (RIF) defined as (see Equation 3.14):

$$RIF = \frac{Q_{\text{damaged}}}{Q_{\text{intact}}} \tag{6.7}$$

Jacket 1 and Jacket 4 are evaluated by the removal of the most critical members in the intact state collapse analysis. Normally the jackets are evaluated for the removal on one member. However, herein the removal of several members is evaluated. The RIF value for *i* damaged members is then referred to the collapse capacity of the jacket with ***i-1*** members damaged. Implying that the RIF is defined as:

$$RIF_i = \frac{Q_{damaged\,i}}{Q_{damaged\,i-1}} \tag{6.8}$$

where *i* is the number of damaged members. With one damaged member, *i* is equal to 1 and $Q_{damaged\,i-1}$ would be the same as Q_{intact} . Hence Equation 6.8 would equal Equation 6.7.

The resulting DSRs and RIFs from the analyses of Jacket 1 is shown in Table 6-4.

Table 6-4: Examples of calculated damaged strength ratio (DSR) values for 1 and 2 members removed on Jacket 1

Removed member(s)	Calculated RSR / DSR	RIF	Total reduction in capacity
Intact	3.05	1.0	1.0
201216	2.54	0.83	0.83
203216	2.57	0.84	0.84
201306	2.7	0.88	0.88
201216 & 203416	2.05	0.81	0.83·0.81 = 0.67
203216 & 201416	2.4	0.93	0.84·0.93 = 0.79

These limited analyses indicate that for X-braced jackets a RIF_1 of 0.8 for the removal of one brace seems reasonable. The reduction factor of an additional removed brace (RIF_2) may also be approximated to 0.8 based on these calculations.

A similar study is performed for Jacket 4. The members in the second bay from mud line are removed one by one and finally all bracing members are removed.

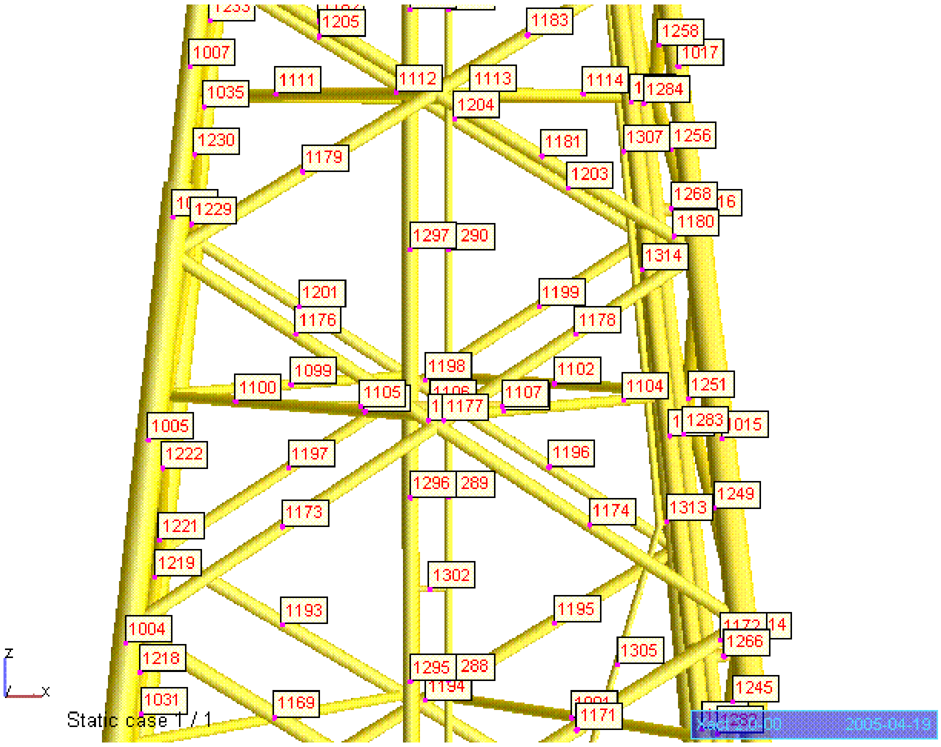

Figure 6-27: Member numbers of Bay 2, Jacket 4

The active members of Bay 2 for a wave from 0 deg is 1173, 1174, 1176, 1178, 1197, 1196, 1201 and 1199. The resulting DSRs and RIFs from the analyses of Jacket 4 is shown in Table 6-5.

Table 6-5: Calculated damage strength ratio (DSR) and residual strength factor (RIF) for Jacket 4

Removed member	Calculated RSR / DSR	RIF
Intact	2.5	1.0
1173	1.65	0.66
1173&1174	1.42	0.86
1173&1174&1196	1.05	0.74
1173&1174&1196&1197	0.8	0.76
All Bay2 members	0.45	0.56

The RIF_1 (for the first member failure) is rather small compared to Jacket 1. However, this should be expected as Jacket 4 is a diamond braced jacket and Jacket 1 is an X braced jacket. The further reduction factors are more in line with the reduction factors for Jacket 1.

6.4 Reserve Freeboard Ratio (RFR)

The purpose of this section is to develop a parameter for describing the safety (barrier indicator) against a failure mode caused by wave in deck. The parameter should preferably be developed in the same line as the reserve strength ratio (RSR). In this case it would be preferable to obtain a measure between the wave crest elevation that result in failure of the structure and a reference (design) crest elevation. The reference (design) crest elevation is chosen as the crest elevation with the annual probability of 10^{-2} of exceedance, as this is the reference probability also for the RSR. The RFR is then given as:

$$RFR = \frac{\eta_{critical}}{\eta_{design}} \tag{6.9}$$

where η_{design} is conservatively taken as the sum of the design wave crest height, the design astronomical tide (AT_{100}) and the design storm surge (StS_{100}), e.g. $\eta_{design} = \eta_{c\_100} + AT_{100} + StS_{100}$. $\eta_{critical}$ is the critical wave crest elevation that results in collapse of the structure due to wave loading. The critical crest height elevation can be taken as the point in the non-linear collapse analysis where the $RSR(h)$ becomes less than 1.0. For Jacket 1 a freeboard of approximately 25m is the critical crest elevation, see Figure 6-8. The crest elevation is then approximately 0.5m into the deck.

Simplified and conservatively, the critical wave crest may be set equal to the available freeboard, taking into account the astronomical tide and the storm surge (24.5m for Jacket 1).

$$RFR = \left(\frac{FB}{\eta_{100}} \right) \tag{6.10}$$

Most jackets tolerate a certain wave-in-deck loading. In these cases the RFR given in Equation 6.10 would be slightly conservative. However, this simplified formulation is easier available and a good first approximation.

6.5 Acceptance criteria for RSR, DSR, RIF and RFR

6.5.1 Deterministic criteria for RSR

The deterministic criteria for RSR is developed by requiring that there should be no plastic deformations in large areas (a full member or node) in an ultimate limit state check (ULS). This is performed by comparing the load at first member failure found by a linear elastic design analysis with the collapse analysis. This method is based on the work found in BOMEL (2001), and some of the definitions for code safety factors (explicit and implicit) are further described in this reference.

The second approach is directed towards the accidental limit state check (ALS), and is requiring that the RSR for the 10 000 year wave (the wave with an annual probability of exceedance of 10^{-4}) should be above 1.0 with a reasonable safety margin.

It is, in development of this criterion, assumed that the air gap is sufficient so that the 10 000 year wave does not hit the deck.

The member forces in a jacket

Design of a jacket is usually performed on member and joint basis. Herein, only the member capacity will be evaluated. The member loads in a jacket are usually calculated from gravity loads and environmental loads by a finite element method program, giving the member forces in the jacket as an influence function times the total load.

$$F = h_P \cdot P \tag{6.11}$$

As a very simple example, the action "P" could be a point load in the middle of a simply supported beam. If the member force that we are interested in is the moment in the middle of the beam, the influence function will be:

$$h_P = \frac{L}{4} \tag{6.12}$$

Giving that the moment in the middle of the beam is:

$$M = h_P \cdot P = \frac{P \cdot L}{4} \tag{6.13}$$

This influence function will be constant as long as the structural behaviour is linear. It will be approximately correct as long as the structural behaviour is reasonably linear.

The force in a jacket structure member can be written as (see Figure 6-28):

$$F = D \cdot h_d + E \cdot h_e = P_d + P_e \tag{6.14}$$

where D is the total gravity load component (i.e. dead and live load) on the structure, h_d is the influence function for gravity loads for a member (different for each member), E is the total environmental load on the structure and h_e is the influence function for environmental load for a member (different for all members).

This formulation of the forces in the member is assumed to be correct also for the non-linear analysis until failure (plastic hinge of buckling) of the first member to fail.

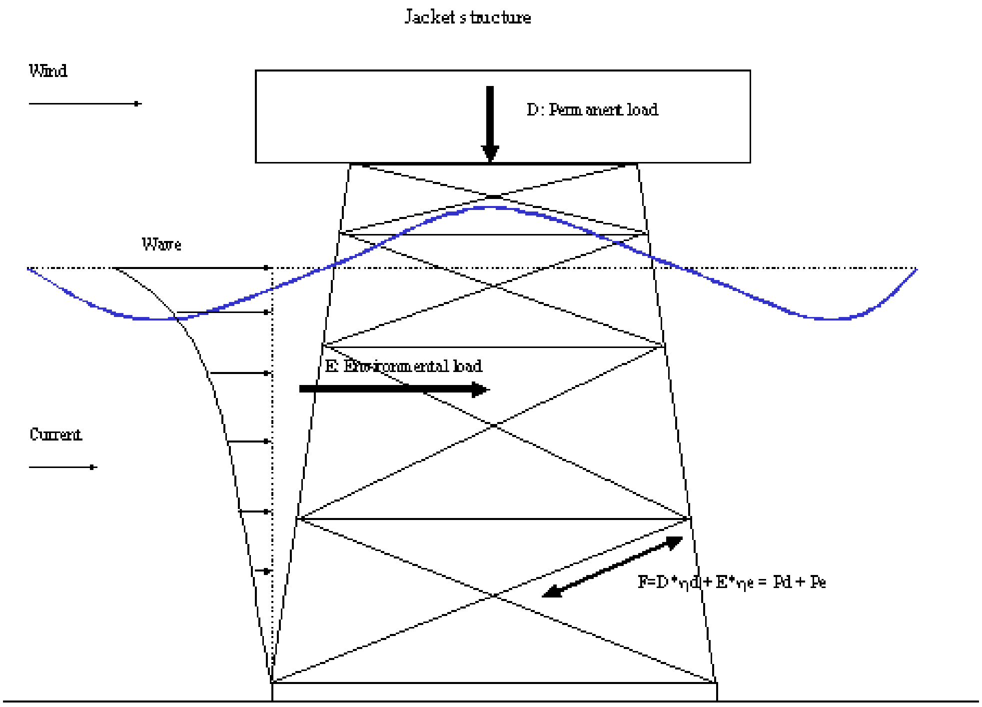

Figure 6-28: Actions (permanent gravity loads and environmental loads) on a jacket platform and resulting forces in a member

RSR requirement based on the ULS criteria
The calculations are based on the following assumptions:

- Structural failure will be initiated by failure of one single member.
- Failure modes leading to the system failure involve other similar structural members
- Effects of foundation and weak joints do not significantly influence the failure modes.

Load resistance factor design (LRFD) codes (see e.g. NORSOK 2004 and ISO 2004) typically use the following format for checking the capacity of a member:

$$\frac{R}{\gamma_{Rc} \cdot \gamma_m} \leq \gamma_d \cdot P_d + \gamma_e \cdot P_e \tag{6.15}$$

where R is resistance, γ_{Rc} is resistance safety factor for element, γ_m is the material safety factor, P_d is the gravity load contribution to the load in a member, γ_d is gravity load factor, P_e is the environmental load contribution to the load in a member and γ_e is environmental load factor.

The explicit code margin of safety (MOS) is by BOMEL (2001) defined as the ratio between the un-factored structural resistance (R_{unf}), and the necessary design resistance (R):

$$MOS = \frac{R}{R_{unf}} = \frac{(\gamma_d \cdot P_d + \gamma_e \cdot P_e) \cdot \gamma_{Rc}}{P_d + P_e} \tag{6.16}$$

Due to possible other implicit code safety factors (ICSF), the actual expected ultimate load level at the failure of the member initiating the system failure in a linear elastic design analysis is:

$$\begin{aligned} P_{f\_m\_f} &= (\gamma_d \cdot P_d + \gamma_e \cdot P_e) \cdot \gamma_{Rc} \cdot \gamma_m \cdot ICSF \\ &= MOS \cdot (P_d + P_e) \cdot \gamma_m \cdot ICSF \end{aligned} \qquad (6.17)$$

A typical calculation of a RSR value is performed by applying the gravity load, and then by adding the environmental loading in increasing steps until collapse. The member initiating the collapse fails at a lower load than the ultimate load. If the structure is behaving reasonably linear until the load level in a member initiating the system failure, this load level in a collapse or push-over analysis can be expressed as:

$$P_{f\_m\_f} = D \cdot \eta_d + \frac{RSR}{SR} \cdot E \cdot \eta_e = P_d + \frac{RSR}{SR} \cdot P_e \qquad (6.18)$$

where SR is the structural redundancy.

If the two expressions for load in the member at first failure found in Equations 6.17 and 6.18 are set equal, the resulting expression for the minimum allowable RSR may be found after some reorganisation:

$$RSR = MOS \cdot \gamma_m \cdot ICSF \cdot SR \cdot \left[1 + \left(1 - \frac{1}{MOS \cdot \gamma_m \cdot ICSF} \right) \cdot \frac{P_d}{P_e} \right] \qquad (6.19)$$

The term in brackets in Equation 6.19 is in the following denoted ρ_{PdPe}, resulting in the following mathematical formulation:

$$RSR = MOS \cdot \gamma_m \cdot ICSF \cdot SR \cdot \rho_{PdPe} \qquad (6.20)$$

where MOS is explicit code margin of safety in a linear elastic design methodology (defined above), ICSF is implicit code safety factor (i.e. any conservatism in the design formulas), γ_m is material factor, SR is system redundancy factor and the ρ_{PdPe} factor takes into account the ratio between environmental load and gravity loads.

Example NORSOK N-001(NORSOK 2004)

To exemplify the requirement of RSR found using the expression developed in Equation 6.20, the parameters found in NORSOK N-001 (2004) are applied. In NORSOK N-001 there are defined two ULS criterias, ULS-A and ULS-B. In ULS-A $\gamma_d = 0.7$ and $\gamma_e = 1.3$ and in ULS-B $\gamma_d = 1.3$ and $\gamma_e = 1.0$. In both cases $\gamma_m = 1.15$ and $\gamma_{Rc} = 1.0$.

The Structural Redundancy factor SR is set to 1.2. As mentioned in Section 6.2, it is not possible to achieve a typical value for the SR factor for these jackets. The SR factor of 1.2 seems to be in the area of the mean value of the calculated SR values for Jacket 1 and Jacket 4. The scatter of calculated SR values is relatively large for Jacket 2, indicating that it may be doubtful to use one value to represent a typical SR value for these jackets. However, the choice of SR value will not affect the resulting RSR criteria, as the criteria will later be normalized towards the actual calculated RSR values for jackets designed according to NORSOK N-001 (NORSOK 2004).

The resulting RSR criteria for the ULS-A case for different ICSF values are shown in Figure 6-29.

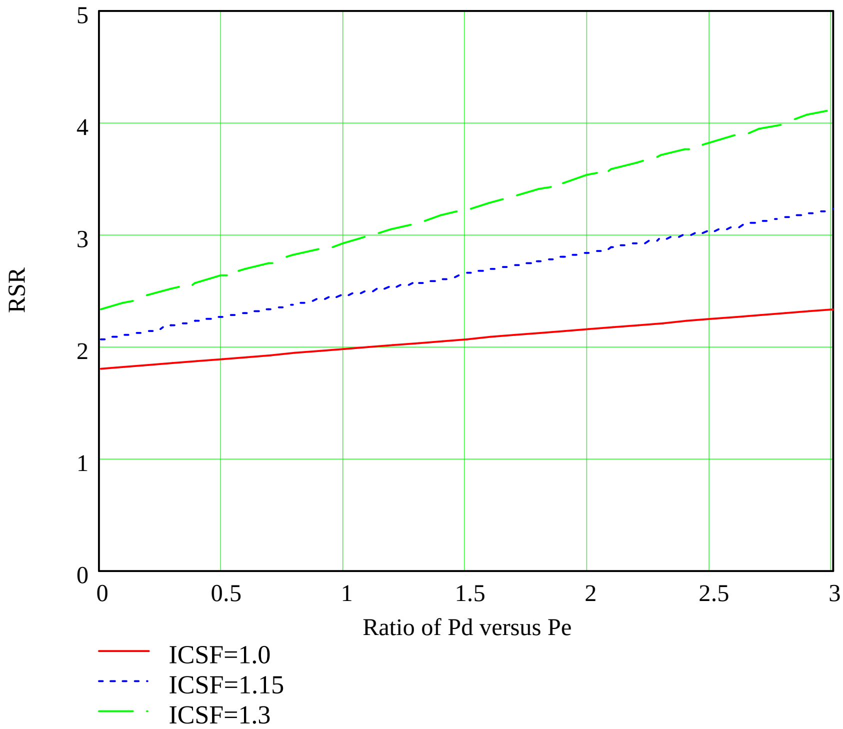

Figure 6-29: Minimum reserve strength ratio (RSR) for different ICSF's – ULS-A case with load factors according to the NORSOK N-001 standard (NORSOK 2004).

Similarly, the resulting RSR criteria for the ULS-B case can be developed. If the gravity load contribution is high, the ULS-B case is dominating the RSR requirement.

The level of implicit code safety factor (ICSF) is not directly available. It may be obtained by evaluating the RSR's of jacket structures designed according to this design code. However, this will be just an indication of the ICSF. In BOMEL (2001) ICSF ranging from 1.0 to 1.14 are indicated.

In order to evaluate the implicit code safety factor (ICSF) in the NORSOK code, RSR values from three different jacket structures designed according to this code has been evaluated. The jackets used in this evaluation are the Kvitebjørn Jacket, the Grane Jacket and the Ringhorne Jacket. On the Kvitebjørn jacket the gravity load is very small compared to the environmental load. The minimum RSR identified in the "Kvitebjørn Redundancy Analysis report" (Aker Maritime 2001) is 2.09 for wave in South West direction. As this is a diagonal direction, and the failure mode is failure starting with a leg failure, the permanent load contribution is significant. The P_d / P_e ratio is calculated assuming that the topside weight of 23 000 tonnes is distributed evenly between the four legs (56 MN in each), and that the wave and current overturning moment (7192·1.4=10069 MNm where 1.4 is the dynamic amplification factor) is taken only by the legs on the diagonal. As the distance between these legs is approximately 70m, the resulting environmental force in the leg is approximately 144 MN. Based on these assumptions, the P_d/P_e ratio for diagonal sea is assumed to be in the order of 0.4. For the head sea, the failure mode for the Kvitebjørn jacket is starting by a failure in a brace member, and hence it is assumed that the Pd contribution is negligible. The calculated RSR from USFOS analysis is 1.91 for this direction using 28.5m wave height and current of 1.2 m/s at sea surface. The RSR values for the Grane and the Ringhorne jackets are from the CODAM database (NPD 2003), and the background data for these RSR's are less known. The Grane jacket is an 8 legged jacket, and it is assumed that the P_d/P_e is approximately 0.3. The Ringhorne jacket is a 4 legged jacket, with relatively heavy topside (19000 ton). For a jacket of this type (length and weight) the base shear is estimated to 40MN and the diagonal leg spacing estimated to 50m. This would result in a P_e of 100MN, resulting in a P_d/P_e ration of 0.5. However, this ratio would vary between various jacket structures.

The data for the 3 jackets are indicated with the RSR criteria curves for ULS-A and ULS-B in Figure 6-30. The Kvitebjørn jacket is believed to be a rather optimised jacket, possibly defining the minimum system strength possible with the use of the NORSOK standard. Eye-fitting the RSR criteria curves to the results of the Kvitebjørn jacket, results in an ICSF=1.07 as shown in Figure 6-30. This may imply that the implicit code safety factor for the NORSOK code is approximately 7% (ICSF=1.07). With the estimated P_d/P_e relation for the Grane jacket and the Ringhorne jacket, these are slightly more on the safe side.

The resulting criteria for the RSR parameter in the ULS-A case can be written as:

$$RSR\left(\frac{P_d}{P_e}\right) = 1.92 + 0.277 \cdot \left(\frac{P_d}{P_e}\right) \tag{6.21}$$

An important aspect of this way of defining the RSR criteria, is that the RSR is dependent on the ratio of the load contribution from gravity loads versus the load contribution from the environmental loads *in the critical member* (P_d versus P_e). If the load level in the critical member of a jacket is equally influenced by gravity load and environmental load, the RSR criteria should be higher than if gravity load in the critical member is negligible.

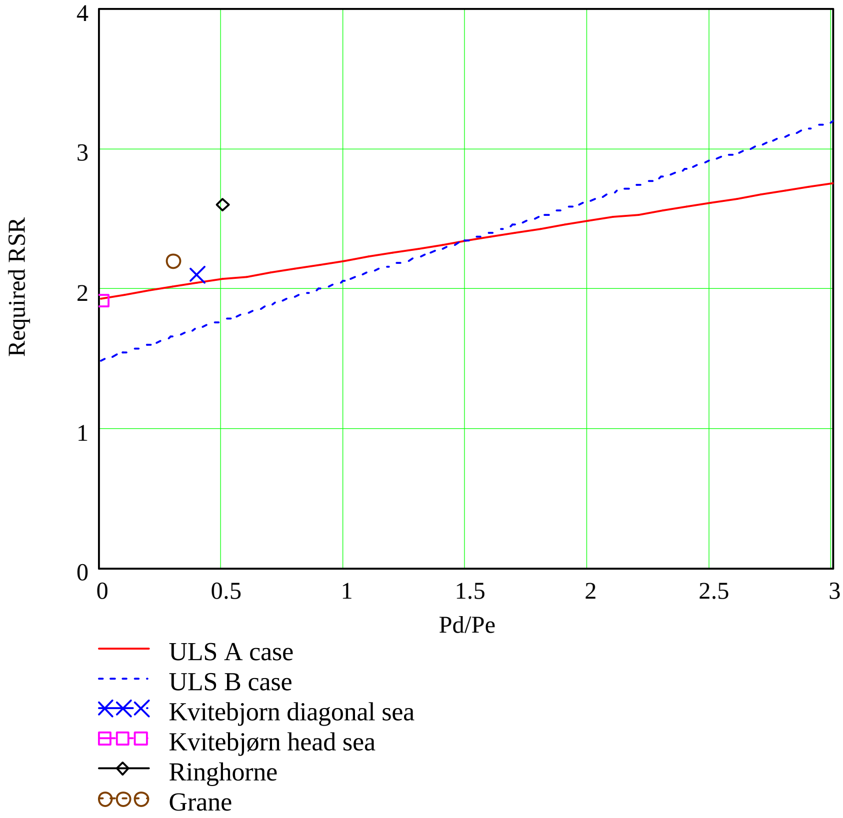

Figure 6-30: Develped RSR criteria in accordance with the NORSOK N-001 standard (NORSOK 2004) with RSR values for jackets designed according to this standard. The proposed requirement is suggested with an implicit code safety factor ICSF=1.07 in accordance with the most optimised jackets designed according to this standard. Load factors according to the NORSOK N-001 standard (NORSOK 2004.

RSR requirement based on the ALS criteria

An alternative philosophy is to say that the 10 000 year load condition should have a RSR above 1.0 with a certain safety margin. In the ALS criteria plastic deformations are normally allowed. The requirement for the RSR is then $RSR_{10000} \geq SF$ where the safety factor SF is 1.0 or somewhat higher if a safety margin is included.

A typical calculation of a RSR value is performed by applying the gravity load, and then by adding the environmental loading in increasing steps until collapse. The ultimate load is due to this ratio between the ultimate load capacity and the dead-load and environmental load as:

$$R_{als} = P_d + RSR_{10000} \cdot P_{e_{10000}} \tag{6.22}$$

The 10 000 years value for RSR can then be expressed as:

$$RSR_{10000} = \frac{R_{als} - P_d}{P_{e_{10000}}} \tag{6.23}$$

Equally, the 100 year RSR is defined as:

$$R_{ult_{100}} = P_d + RSR_{100} \cdot P_{e_{100}} \tag{6.24}$$

For North European waters the ratio between 100 years wave and 10 000 years wave is assumed to be 1.25 (NORSOK 2004). If the loading based on waves can be expressed as a function of wave height times 2.2, the following expression between 10 000 years loading and 100 years loading can be established.

$$P_{e_{10000}} = 1.25^{2.2} \cdot P_{e_{100}} \tag{6.25}$$

If the ultimate capacity of a structure exposed to a 100 year wave and a 10 000 year wave was the same, the relations could be easily established. However, the 10 000 year wave will typically increase the overturning moment relatively more than the base-shear. Also the 10 000 year wave may introduce new failure modes, e.g. in soft areas in the upper structure. The 10 000 year wave may also reach the deck of the structure, and hence introduce both new failure modes and large increase in loading.

A factor, NFM, for taking into account these new failure modes is previously introduced (see Chapter 6.1.3), and is used here to account for the difference between the collapse base shear for the 100 year wave and 10 000 year wave. The capacity for the 10 000 year wave is then expressed as:

$$R_{als} = \frac{R_{ult_{100}}}{NFM} \rightarrow R_{ult_{100}} = NFM \cdot R_{als} \tag{6.26}$$

Inserting this result into the above equations:

$$RSR_{10000} = \frac{R_{als} - P_d}{P_{e_{10000}}} = \frac{\frac{1}{NFM} \cdot \left(P_d + RSR_{100} \cdot P_{e_{100}}\right) - P_d}{P_{e_{10000}}} \geq SF \tag{6.27}$$

This equation can be used to establish a requirement for the RSR with a 100 year loading reference.

$$RSR_{100} \geq SF \cdot 1.25^{2.2} \cdot NFM + (NFM - 1) \cdot \frac{P_d}{P_{e_{100}}} \tag{6.28}$$

Indications based on Jacket 1 to 4, see Chapter 6.1.3, the value of NFM should be in the order of 1.1. As the evaluated jackets are relatively large jackets, the increase in overturning moment will be at a low percentage. For a jacket on less water depth, the NFM may be higher. For the analysed jackets, a drop of base shear at overload is not observed until wave in deck is experienced. This may not be the case for all other jackets.

With the assumptions described above, the requirement for RSR for different values of the NFM factor is shown in Figure 6-31. The safety factor (SF) is in this figure taken as 1.07, similar to the implicit code safety factor (ICSF) found previously. For values of NFM in the order of 1.1, this criterion for the RSR will not be governing as the previously developed criteria will be dominating, see Figure 6-30. With higher NFM values this criterion may be governing. However, the NFM values found from the USFOS analysis previously indicates NFM values above 1.1 only when wave hits the deck, and even in this case only with values up to 1.16, which would imply a similar criterion as shown in Figure 6-30.

It should be noted that these derivations do not necessarily apply for environmental conditions outside in North Western Atlantic Ocean. As an example, in other areas the ratio between the height of the wave with the annual probability of exceedance of 10^{-4} (10 000 year wave) and the height of the wave with an annual probability of exceedance of 10^{-2} (100 year wave) may be as large as 1.8. If the 10^{-4} criteria were introduced in such areas, the second criterion would totally dominate the RSR criterion, as shown in Figure 6-32.

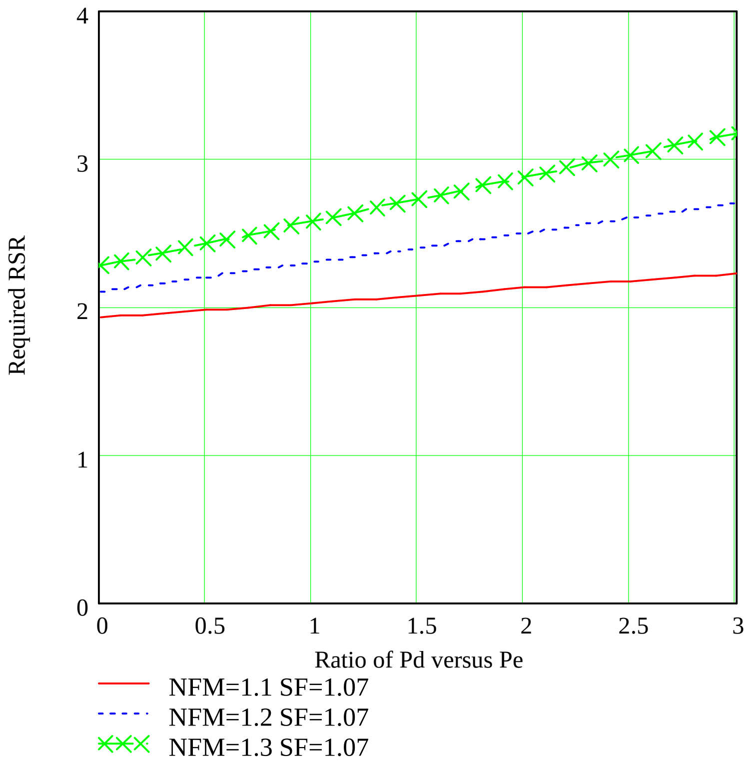

Figure 6-31: Reserve strength ratio (RSR) requirement for different values of the new failure mode (NFM) factor with the structural safety factor SF=1.07.

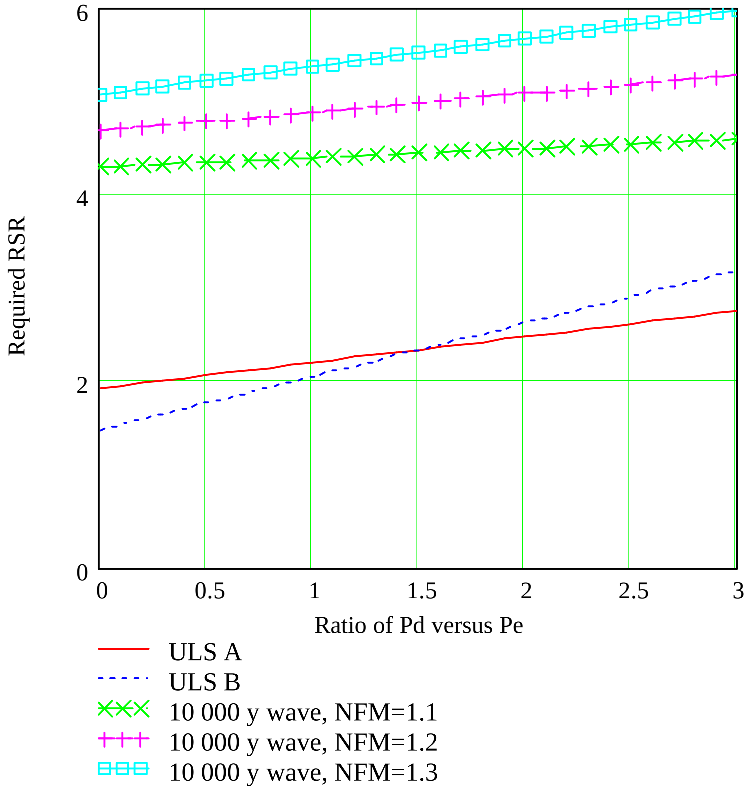

Figure 6-32: Reserve strength ratio (RSR) criteria for geographical areas with a 1.8 factor between the 10^{-4} wave and the 10^{-2} wave.

6.5.2 Probabilistic analysis of a jacket based on the RSR factor

To determine the relation between the RSR and the annual failure rate, a probabilistic model following Ersdal et al. (2003) has been applied. The failure function is written as:

$$g = Resistance - Load = Load_{100} \cdot RSR - Load \tag{6.29}$$

where $Load_{100}$ is the design environmental loading. Here the load with the annual probability of exceedance of 10^{-2} is used.

Wave Height

The annual maximum wave height, H, is assumed to follow a Gumbel distribution, i.e. the distribution function reads

$$F_H(h) = \exp\left\{-\exp\left[-\frac{h-\alpha_H}{\beta_h}\right]\right\} \tag{6.30}$$

where α_H and β_H are parameters of the distribution.

Wave load
The wave loading W is described by the following equation (Equation 6.2 without current):

$$W = \alpha_1 \cdot C_1 \cdot H^{C_3} \qquad (6.31)$$

where α_1 describes the uncertainty in the wave loading on the structure, H is the wave height and C_1 and C_3 are load coefficients that must be curve-fitted to calculated load data for the specific jacket.

In the cases when wave in deck loads are included, the following load model is used (Equation 6.3 without current)

$$W = \alpha_1 \cdot C_1 \cdot H^{C_3} + \alpha_2 \cdot C_4 \cdot L \cdot (C_5 \cdot H - FB) \qquad (6.32)$$

where α_2 is describing the uncertainty in the wave-in-deck loading on the structure, C_5 is the wave crest to wave height factor, C_4 is a load coefficient that must be curve-fitted to the calculated load data for the specific jacket, and FB is the freeboard, here defined as the distance between sea level and bottom of steel on topside.

Resistance
The resistance (*R*) is modelled as an ultimate capacity of the structure, described on a system basis. The ultimate capacity is assumed to be equal to the design loading ($C_1 \cdot H_{100}^{C3}$) multiplied by the Reserve Strength Ratio (RSR). The design loading is the loading with the annual probability of exceedance of 10^{-2}, and the RSR is the ratio between ultimate collapse load of the structure and the design loading. ξ•describes the uncertainty about the collapse resistance of the structure, i.e. R reads:

$$R = \xi \cdot RSR \cdot C_1 \cdot H_{100}{}^{C_3} \qquad (6.33)$$

Limit state equation
The failure function in Equation 6.29 can then be modelled by the following equation:

$$g = R - W \qquad (6.34)$$

Probability of failure is given by $P_f = P(g \le 0)$.

Stochastic model
The parameters of the stochastic model are given in Table 6-6.

Table 6-6: Parameters used in the simulations

Parameter	Description	Values	Comments
RSR	Reserve strength ratio	Fixed at 2.0	The RSR is assumed to be 2.0 for the structure evaluated in this paper, but the failure probability is also evaluated for RSR values from 1.5 to 2.5 to evaluate the sensitivity to the RSR.
X	Uncertainty about the resistance	Normal distributed Mean value = 1.0 Standard deviation = 0.1	X is normally distributed with a mean value of 1.0 and a COV of 0.1 as recommended by Efthymiou et al (1996). The "Guideline for Offshore Structural Reliability Analysis" issued by DNV (1996) recommends a COV for the base shear capacity of a jacket structure to be 0.05 - 0.10.
H_{100}	100 year wave	Calculated from the Gumbel distribution of annual maximum wave height	Wave height with an annual probability of exceedance of 10^{-2}
a_1	Uncertainty about the wave loading on the jacket structure.	Normal distributed: Mean value = 1.0 Standard deviation = 0.15	a_1 is normally distributed with a mean value of 1.0 and a COV of 0.15 as recommended by Haver (1995).
a_2	Uncertainty about the wave-in-deck loading on the structure.	Normal distributed: Mean value = 1.0 Standard deviation = 0.15	Assumed equal to a_1.
H Northern North Sea (NNS) case	Annual maximum wave height	According to Equation 6.30 with: a_H = 21m b_H = 1.63m	The parameters for wave height distribution are obtained by fitting a Gumbel distribution to the data for the Kvitebjørn field in Northern North Sea (Statoil 2000).
H Central North Sea (CNS) case	Annual maximum wave height	According to Equation 6.30 with: a_H = 19.22m b_H = 1.583m	The parameters for wave height distribution are obtained by fitting a Gumbel distribution to wave data for the CNS (Krogstad 2004).
C_1	Load coefficient	Fixed at 1.0	The value used for C_1 is not important for the present study, as C_1 appears in both terms in the equation (resistance and load). 1.0 is chosen merely as a reference value.
C_3	Load coefficient	Fixed at 2.2	*C_3 is found to be approximately 2.2, see Table 6-3.*
C_4	Wave in deck load coefficient	Fixed at 1000	*The ratio between C_4 and C_1 is used in the calculations. This ratio is roughly estimated by the momentum of the incoming wave for a 100m wide deck structure (solid) and related to the C_1 factor.*
C_5	Wave crest factor	Fixed at 0.62	*This value is slightly lower than the maximum for a Stoke 5^{th} order wave (0.66). However, the value may need further investigation.*

For different values of RSR, the probability of overload failure is shown in Figure 6-33 for the CNS (Central North Sea) and NNS (Northern North Sea) environment. The probability of failure is rather similar for the CNS and the NNS case, indicating that the RSR is a relatively good measure of the safety of the structure for overload from wave loading also for the two different wave climates compared in this analysis. It should be noted that no wave-in-deck is modelled and that gravity loads are not included.

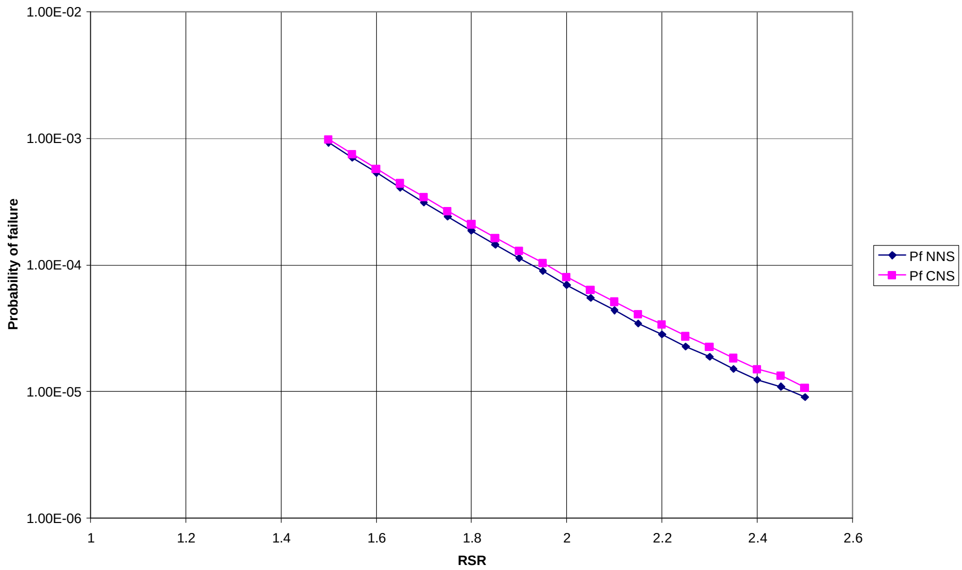

Figure 6-33: Probability of failure for different reserve strength ratio (RSR) values for two different locations – central North Sea (CNS) and northern North Sea (NNS)

According to the deterministic criterion for RSR developed previously, the RSR should be larger than 1.92 for a jacket with insignificant gravity load influence. A RSR of 1.92 would with this model imply an annual probability of failure in the order of 10^{-4}. A stronger acceptance criterion of e.g. an annual failure probability of 10^{-5} would result in a RSR criterion of 2.43.

The relationship between the RSR versus annual probability of failure is established by curve fitting of the results shown in Figure 6-33. The resulting relationship based on eye-fitting is given as:

$$P_f = 10^{-0.0886-1.9976 \cdot RSR} \tag{6.35}$$

The probabilistic analysis leading to the above relationship between RSR and annual probability of failure does not take into account the possibility of new failure modes as a result of the increased wave induced overturning moment. A possible improved failure function may be written as:

$$g = R - W = Load_{100} \cdot RSR \cdot NFM(h) - Load(h) \tag{6.36}$$

As the effect of the NFM is rather small in cases without wave-in-deck loading based on the non-linear collapse analysis presented previously, the NFM factor is not included in the following analyses.

6.5.3 Updated RSR criteria after experienced wave loading

When a structure has experienced a very large load level and succeeded in carrying this load, the degree of confidence in this structure is increased. This fact is utilized for many structures by preloading the structures to a certain load level, in order to prove sufficient structural safety. In probabilistic terms, this would imply that information is obtained about the wave load and, respectively, structural strength. The uncertainty related to these values may be reduced. Updating based on experienced waves is studied in Ersdal et al (2003), and the results presented here are taken from that reference.

The failure function for the structure is denoted g as described earlier. The evidence that the structure has survived a certain load level, described by the experienced wave height, is given by the event function, *f*:

$$f = \mathrm{b} \cdot RSR \cdot C_1 \cdot H_{100}{}^{2.2} - \mathrm{a} \cdot C_1 \cdot H_{\exp erienced}{}^{2.2} > 0 \tag{6.37}$$

The updated probability of failure for the system, after including this information is calculated by the use of Bayes formula, and given by:

$$P_f^U = P(g \le 0 \mid f > 0) = \frac{P(g \le 0 \;\mathbf{|}\; f > 0)}{P(f > 0)} \tag{6.38}$$

The failure probability is calculated by Monte Carlo simulations. The updated failure probability is given by the number of simulations satisfying *g£0* and *f>0*, divided by the number of simulations satisfying *f>0*. The probability *P(f>0)* illustrates the probability that the structure survived the load level given by $H_{experienced}$.

The updated probability of failure as a function of the experienced wave is presented in Figure 6-34. In the case without modelling gross errors, the updated probability of failure is declining after the event that the structure has survived a wave loading with an annual probability of 10^{-3} (32.3m) to 10^{-4} (36 m). The experience of lower waves does not have any effect on the updated probability of failure.

The effect of experienced loading is more significant in the cases with gross error. However, the structure must still have experienced a loading in order of the 100-year to 1 000-year loading to change the updated probability to a probability level close to the probability level of the model with no gross error. If the experienced loading is larger than the 1 000 year loading, the updated formulations (with different levels of probability of gross errors) follow rather closely.

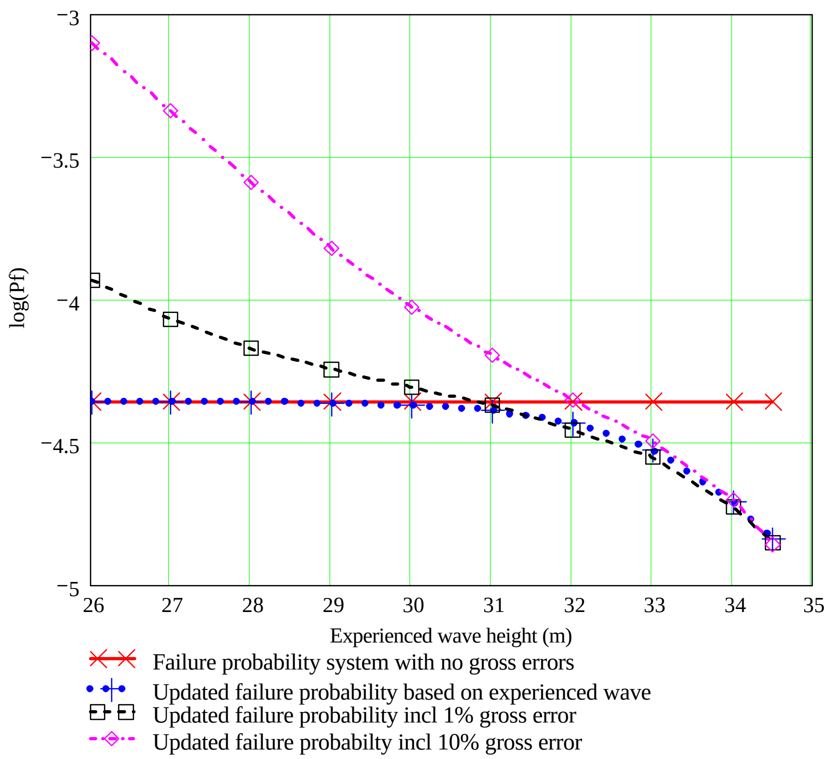

Figure 6-34: Updated annual failure probability for the jacket structure with and without gross error using Bayesian updating. Updated based on the experienced wave loading preconditioned that no damage occurred as a result of the wave loading.

The experienced loading needs to be rather high in order to have any effect on the updated failure probability. In practice, very few jacket structures have experienced waves of this magnitude. The updating of the overload failure probability does, due to this, not have a large effect.

However, it should be noted that the evidence that the structure has survived waves of lower magnitude is giving valuable information about the structure in the sense that the probability of the structure having a gross error after surviving a wave loading with an annual probability in the order of 10^{-2} (28.5m) is significantly reduced.

6.5.4 Criteria for RSR based on cost benefit analysis

A study of RSR criteria based on cost benefit analysis with the probability of member failure and wave in deck loading is also studied. This study is reported in Paper 2 in Part 2 of the thesis. In general, a RSR criterion in line with the deterministic criteria presented previously is also found based on the cost benefit approach for design of new jackets.

The effect of a member failure is modelled by the following probability model:

$$P_{f\_system} = P(sys|\overline{F}) \cdot P(\overline{F}) + P(sys|F) \cdot P(F) \tag{6.39}$$

where P(sys) represents the probability of a system failure, F represents failure of an arbitrary member or node, and $\overline{F}$ represents no failures of any member or nodes (the complimentary set). The $P(sys|F)$ is calculated based on a rather large reduction in system capacity as a result of a member failure (varying between a reduction from 0.3 to 0.7). More details about the calculation methods is found in Paper 2 in Part 2 of the thesis.

The resulting RSR criterion for increasing probability of member failure is found, see Figure 6-35. A clear indication of an increasing RSR as a result of increasing probability of member failure is seen.

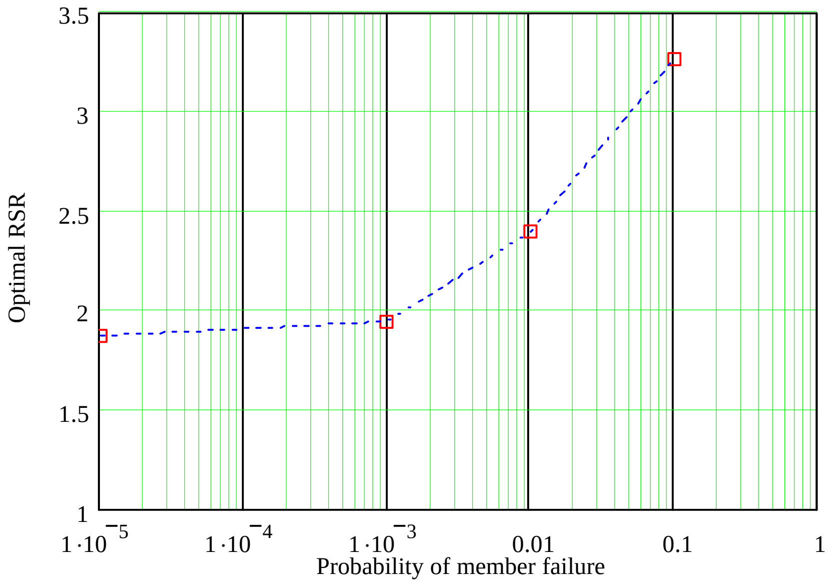

Figure 6-35: Economic optimal reserve strength ratio (RSR) with increasing probability of a member failure. Calculated values and linear interpolation line are shown in the figure.

The effect of wave in deck loading on the cost optimal value is studied for a jacket. In this case the wave loading is modelled according to Equation 6.3. The resulting RSR criterion is shown in Figure 6-36. A clear indication of an increasing RSR with a decreasing freeboard (denoted air gap in this analysis) is observed.

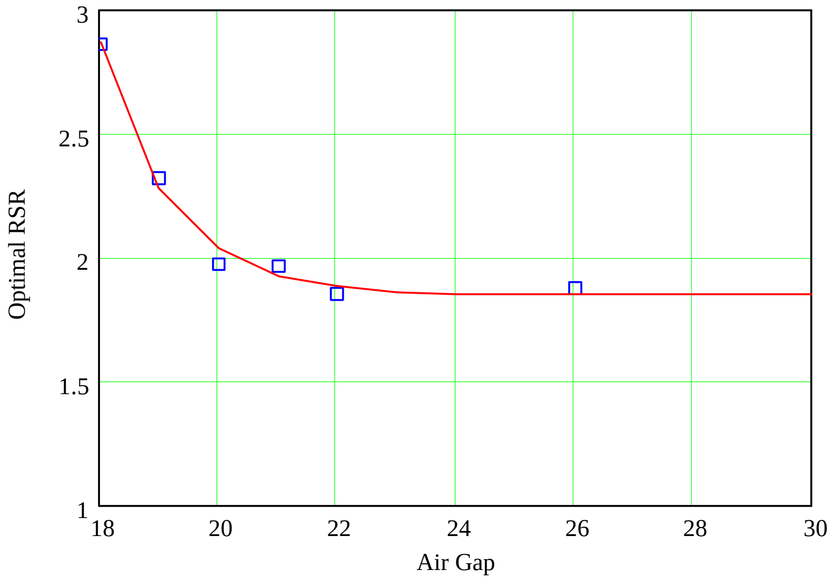

Figure 6-36: Economic optimal reserve strength ratio (RSR) with increasing air gap (freeboard) measured in meters - calculated points and curve fit. Environmental data for this evaluation is representative for northern North Sea

6.5.5 *Probabilistic criterion for RSR including topside loadings*

The limit state function for the jacket taking into account both the wave loading and the variation in topside loading may be taken as:

$$g = R - W - D - L \tag{6.40}$$

where R is the resistance of the structure, D is the gravity load, L is the live load and W is the wave and current load. The resistance of the structure depends now on the ratio of gravity loads to the environmental loads.

The gravity loads, D and L, affects the strength of the structure in a different way than the wave load. In this model the gravity loads have to be transferred to an equivalent environmental loading, that is representing the gravity loads with the effect they have on the collapse base shear capacity. A possible limit state function relating the topside loading to the wave loading:

$$g = R - W(1+\Lambda) \tag{6.41}$$

where Λ is a factor for taking into account the effect of an increase in gravity loads on the system failure.

Using the influence functions for the effect of the environmental loads and gravity loads, see Equation 6.14, the limit state function may be written as:

$$g = R - W = \xi \cdot RSR\left(\frac{P_d}{P_e}\right) \cdot C_1 \cdot {H_{100}}^{C_3} - \alpha_1 \cdot C_1 \cdot H^{C_3}\left(1 + \alpha_3 \cdot \frac{P_d}{P_e} \cdot \frac{\eta_d}{\eta_e}\right) \qquad (6.42)$$

where α_3 represents the uncertainty related to the gravity load on the structure. In BOMEL (2001) the uncertainty in the gravity load is modeled by a normal distribution with a mean value of 1.0 and a COV of 0.06, whereas the uncertainty on live loads is modeled by a normal distribution with a COV of 0.1. Herein the uncertainty on the gravity loads (including both gravity and live loads) are modeled by a normal distribution with a COV of 0.1.

From Figure 6-23 it can be seen that the decrease in collapse base shear with increasing topside gravity loads has a slope of approximately 0.1 for waves arriving from the 45 degrees direction. This should indicate that the η_d / η_e should be taken as approximately 0.1 for this direction. For this diagonal wave case the critical member is the leg. The loading in the leg can be roughly estimated by a forth of the gravity load (four legged jacket) and the wave overturning moment divided by the diagonal leg spacing.

$$F_{leg} = \frac{D}{4} + \frac{E \cdot a}{S_{leg}} = \eta_d \cdot D + \eta_e \cdot E \qquad (6.43)$$

where D is the gravity load, E is the horizontal wave load, a is the distance from the seabed to the resultant of the wave load and S_{leg} is the diagonal spacing between the legs. For Kvitebjørn the a~184m and the S_{leg} ~70m, giving an η_d=1/4 and an η_e=2.63. The ratio is then η_d / η_e ~ 0.1 as observed in the non-linear collapse analysis.

This relationship will be applicable for jackets with four legs and similar ratio between water depth and diagonal leg spacing, but it is not a general relationship. However, it is used in this study.

The deterministic criteria for RSR developed previously in Equation 6.21, defined the RSR for the ULS-A case in NORSOK N-001 (NORSOK 2004) as a function of gravity loads and environmental load contribution in the critical member. This RSR relationship, a constant RSR=1.92 and a RSR relationship calibrated based on the probabilistic analysis to obtain a constant annual probability of failure with increasing gravity load contributions is shown in Figure 6-37. The calibrated RSR relationship is then found to be:

$$RSR\left(\frac{P_d}{P_e}\right) = 1.92 + 0.2 \cdot \left(\frac{P_d}{P_e}\right) \qquad (6.44)$$

With a constant RSR=1.92, Figure 6-37 shows a clear increase in the failure probability with an increasing gravity load contribution in the critical member. A constant RSR criterion would be incorrect to apply with large gravity load contributions, if a constant annual probability of failure is sought. The deterministic RSR criterion developed earlier has a slight decreasing failure probability as gravity load contribution in critical member increases. A small correction to this relationship provides a more or less constant annual probability of failure.

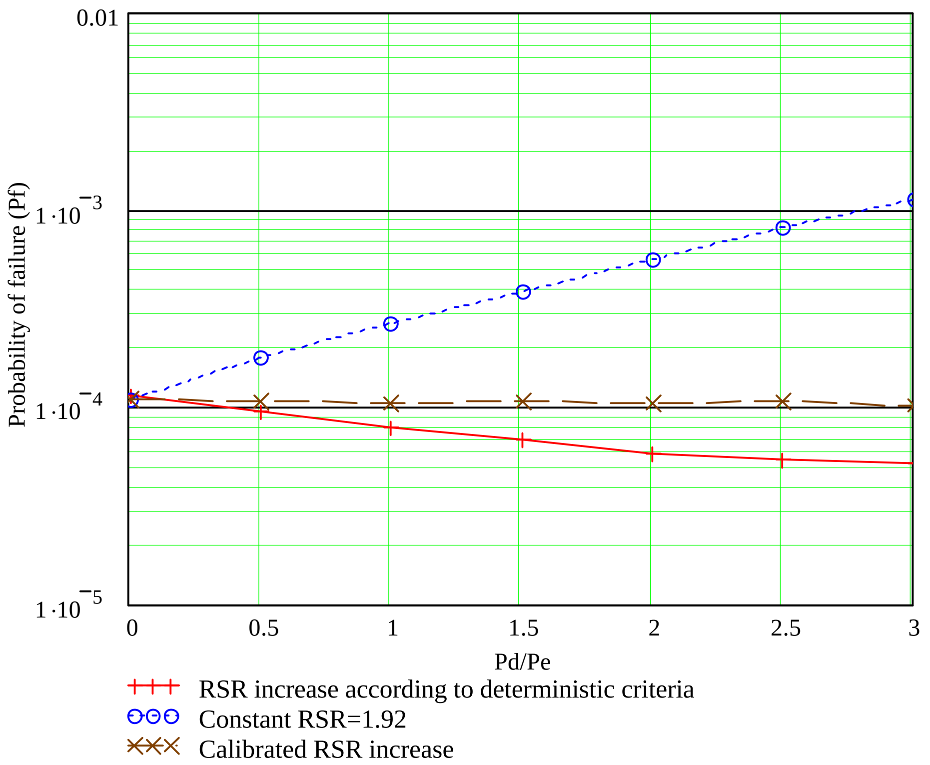

Figure 6-37: Probability of failure with increasing gravity load contribution in the critical member for a constant RSR=1.92, RSR according to Equation 6.21 and RSR according to a calibrated relationship. Note that only NORSOK N-001 (NORSOK 2004) ULS-A case is analysed.

6.5.6 Deterministic criteria for damaged strength ratio

A deterministic criterion for the damaged strength ratio (DSR) can be developed from the deterministic criterion for RSR shown previously. The structure should according to NORSOK N-003 (NORSOK 1999) be able to withstand environmental loads with the annual probability of exceedance of 10^{-2}, but without any load factors used in the analysis. If the criterion for RSR as shown in Figure 6-30 is plotted without load factors, this would be an indication of a reasonable DSR criterion. The DSR criterion is shown in Figure 6-38.

In special cases when the jacket structure is fully optimised and just meet the RSR criterion, the DSR criterion will only be met if the ratio between DSR and RSR are determined by these two curves. Then the criterion may be a residual strength factor (RIF) criterion rather than a DSR criterion, as shown in Figure 6-39.

In general a minimum criterion of a RIF of 0.8 seems to be reasonable. If a specific jacket does not meet this RIF criterion, the RSR needs to be higher than the minimum criterion mentioned earlier to compensate for this higher RIF. The damaged strength ratio criteria are a more general criterion, and should be preferred.

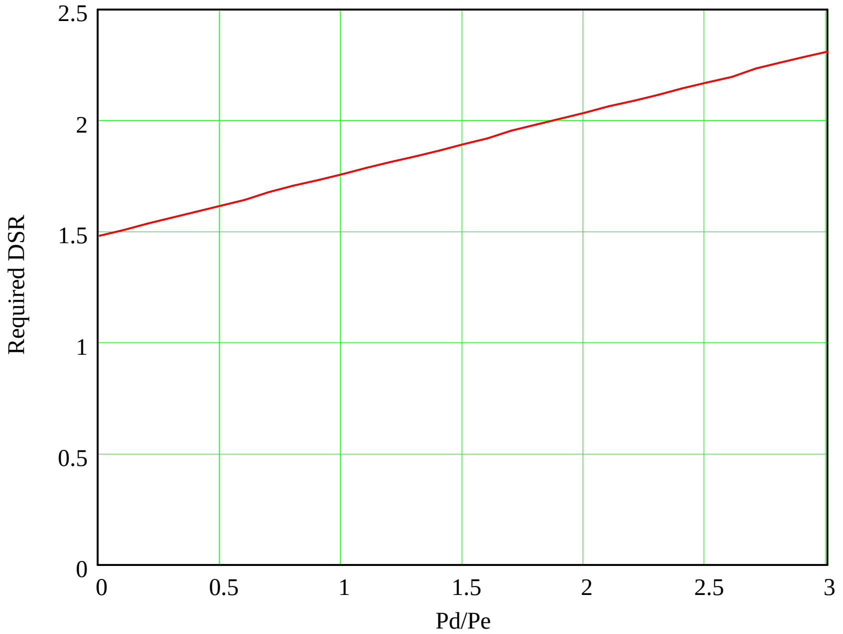

Figure 6-38: Damaged strength ratio (DSR) criterion

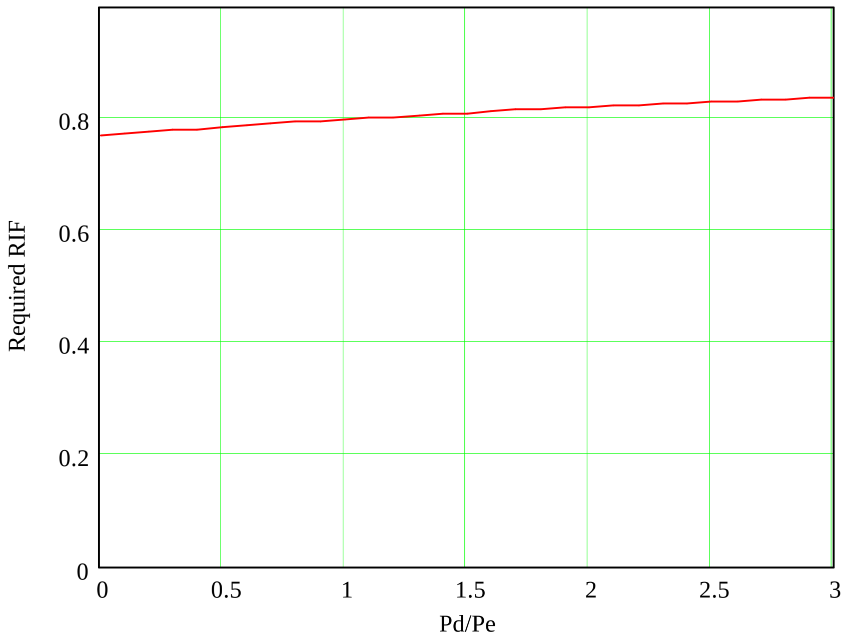

Figure 6-39: Residual strength factor (RIF) criterion

6.5.7 Deterministic criteria for the reserve freeboard ratio (RFR)

Following the same line of reasoning as for the deterministic criteria for RSR, a deterministic criterion for RFR may be established. As a minimum, the wave with the annual probability of exceedance of 10^{-4} (10 000 years wave) should not hit the deck. This would imply that:

$$RFR \geq \frac{\eta_{10000}}{\eta_{design}} \tag{6.45}$$

Using the wave with the annual probability of 10^{-2} (100 years wave) as the design wave crest reference for RFR, and a factor of 1.25 as the difference between the 10^{-4} and the 10^{-2} wave (NORSOK 2000), the minimum RFR from a deterministic view should be:

$$RFR \geq \frac{\eta_{10000}}{\eta_{100}} = \frac{C_4 \cdot H_{10000}}{C_4 \cdot H_{100}} = \frac{1.25 \cdot H_{100}}{H_{100}} = 1.25 \tag{6.46}$$

The 10^{-4} condition is often checked for a mean water level including possible effects of storm surge, whereas the 10^{-2} condition takes into account both storm surge and astronomical tide with an annual probability of 10^{-2} of exceedance (NORSOK 1999 p 42). If this difference in accounting for astronomical tide is included in the above evaluation, the minimum RFR criterion would be slightly lower. However, the non-linearity in the wave (crest to thought) is normally increasing with increasing wave heights, possibly indication an even higher RFR criterion than 1.25.

6.5.8 Probabilistic evaluation of the reserve freeboard ratio (RFR)

To determine the relation between the RFR and the annual probability of occurrence, the following limit state function is applied:

$$g = FB_0 - at - sts - \eta_c \leq 0 \tag{6.47}$$

where FB_0 is the distance between lowest astronomical tide (LAT) and bottom of steel (BOS) topside, *at* is astronomical tide, *sts* is storm surge and η_c is wave crest height.

Taking into account the reserve freeboard ratio (RFR) defined in Equation 6.10, the limit state function can be described by:

$$g = RFR \cdot \eta_{100\_elevation} - \eta_{c\_elevation} \leq 0 \tag{6.48}$$

where η_{100} is the 100 year wave crest elevation (the wave crest with an annual probability of exceedance of 10^{-2}).

The wave crest will be a certain portion of the total wave height, and can for the extreme cases in this discussion be estimated to 0.6 – 0.65 of the total wave height. This value is previously defined in Equation 6.3, and is denoted C_5. The limit state function can now be described by:

$$g = RFR \cdot \left(C_5 \cdot H_{100} + AT + StS\right) - C_5 \cdot h - at - sts \leq 0 \tag{6.49}$$

The probability of wave in deck can based on this limit state function be calculated for the data given for a Northern North Sea environment and Central North Sea environment as given in Table 6-6. The resulting annual probability of wave in deck is shown in Figure 6-40. It can be observed that the normalisation of the parameter RFR gives reasonably similar results on the two different locations.

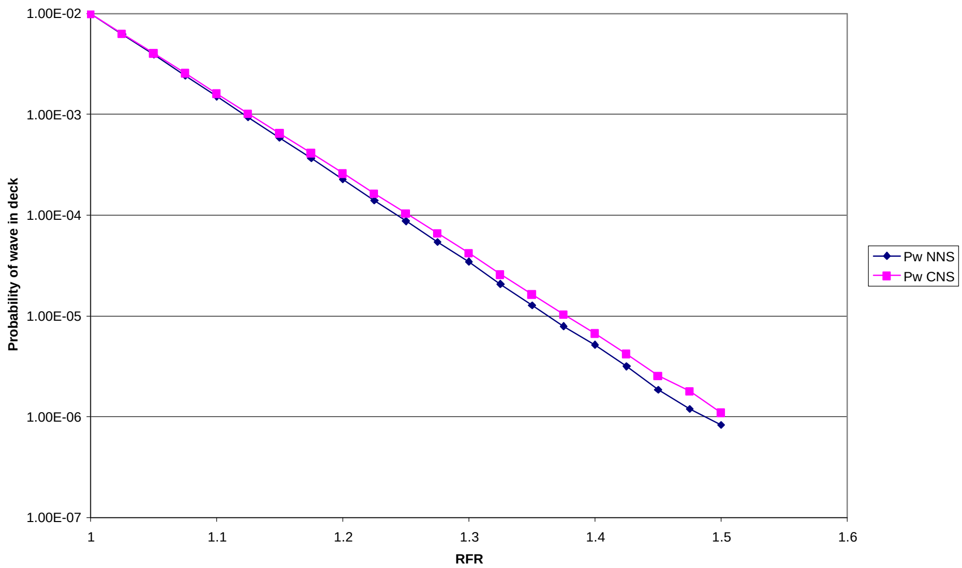

Figure 6-40: Probability of wave-in-deck for two different locations and environmental conditions for different RFR values. NNS is Northern North Sea and CNS is Central North Sea. Astronomical tide and storm surge is in both cases set to 1.5 meters.

Similarly the effect of different values of astronomical tide and storm surge may be evaluated. The astronomical tide and storm surge is varied in a range from 0m to 1.5m. The resulting probability of wave in deck from these simulations is shown in Figure 6-41.

It can be seen in Figure 6-41 that the probability of wave in deck is highest with no tide and storm surge. This may at first sight occur to be peculiar. It should however be noted that the real freeboard is larger when a tidal variation is included, as the freeboard is calculated as $FB = RFR \cdot (\eta_{desin g} + TIDE_{design})$.

The limit state function is $g = RFR \cdot (\eta_{design} + TIDE_{design}) - \eta_{elevation}$ where $\eta_{elevation}$ is the crest elevation (the sum of the wave crest, astronomical tide and storm surge). The purpose of this formulation is to normalise the probability of wave-in-deck and the RFR factor. Figure 6-41 indicates a small difference between the different conditions. Hence the normalisation is not perfect by this formulation, but a reasonable approximation.

With the given limit state functions, the RSR and RFR resulting in a given probability of occurrence can be determined. These values are shown in Table 6-7 for annual probability of occurrence of overload of a jacket with a given RSR (and infinite freeboard) and wave-in-deck for a jacket with a given RFR.

Table 6-7: Corresponding values for RSR and RFR at different values of probability of occurrence

Annual probability of occurrence	RSR	RFR
10^{-3}	1.5	1.13
10^{-4}	1.95	1.27
10^{-5}	2.5	1.4

However, the same level of annual probability of wave-in-deck as the annual probability of overload of a jacket would imply that the overload failure probability is influenced by wave-in-deck loading, resulting in a higher annual failure probability of the structure than indicated in Figure 6-41. An analysis of wave in deck forces is due to this needed in order to obtain a combined criterion for RFR and RSR.

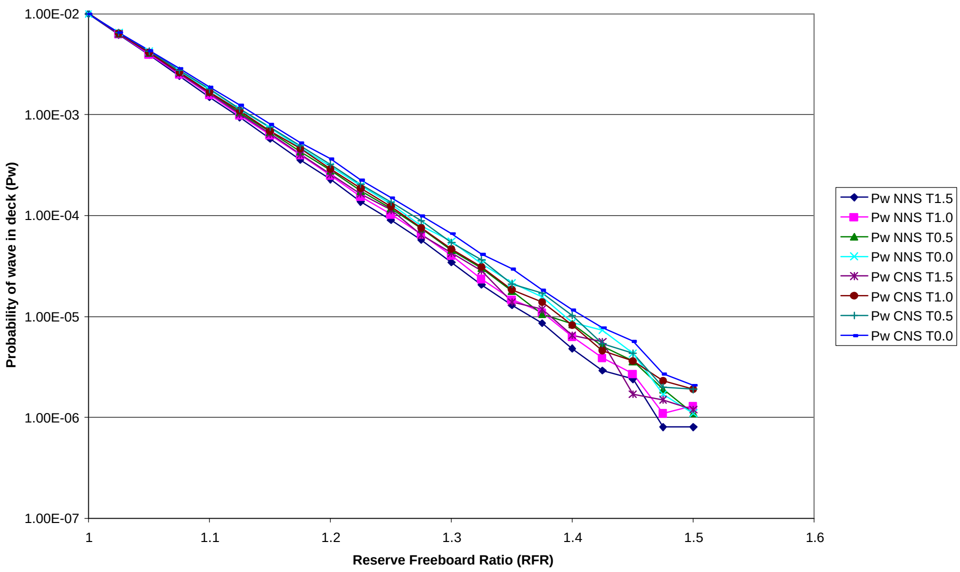

Figure 6-41: Probability of wave-in-deck for two different locations and environmental conditions and different tidal variations. NNS is Northern North Sea and CNS is Central North Sea. T1.5 indicates tidal variation of 1.5m modelled both in the design freeboard and in the probability of wave in deck calculations. Similarly, T1.0, T0.5 and T0.0 indicate tidal variation of 1.0m, 0.5m and 0.0m respectively.

6.5.9 Combined criteria for RFR and RSR

To study the effect of wave in deck loading on the probability of failure given a certain RSR, a failure function including wave-in-deck loading is used.

$$g = R - C_1 \cdot (H + C_2 \cdot u)^{C_3} - C_4 \cdot L \cdot (C_5 \cdot H - FB) \cdot s(C_5 \cdot H - FB) \quad (6.50)$$

where the function $s(C_5 \cdot H - FB)$ is a step function and equals 0 if the argument is negative and 1 if the argument is positive.

In Figure 6-42 the annual probability of failure of the jacket for wave loading overload is presented. For large RFR values the annual failure probability stabilises at the level of failure probability calculated for infinite freeboard. For small values of RFR the failure probability is significantly higher. At the RFR corresponding to the same level of annual probability of wave-in-deck occurrence and overload of the jacket for infinite freeboard, the annual probability is close to stabilising at the target annual probability level. However, the RFR needs to be slightly higher for the annual failure probability to stabilise.

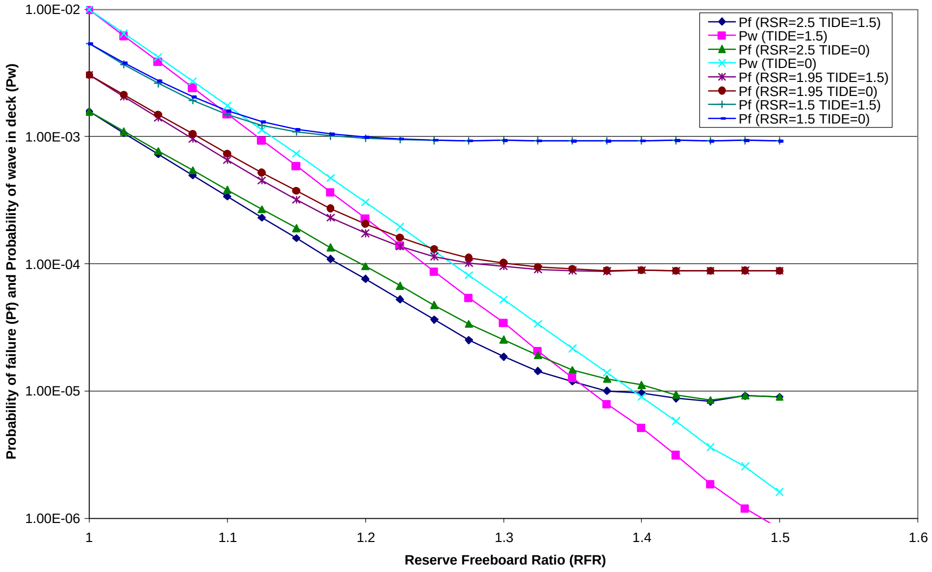

Figure 6-42: Annual failure probability for overload failure as a function of reserve freeboard ratio (RFR) for jackets with a reserve freeboard ratio (RSR) of 1.5, 1.95 and 2.5, and for astronomical tide and storm surge of 0m and 1.5m.

A simplified check of the safety of the structure can be to check the RSR and RFR according to the numbers given in Table 6-8.

As an example, if an annual probability of 10^{-4} is regarded as a reasonable assessment criterion, the RSR should be 1.95 or larger and the RFR should be 1.35 or larger. If the RFR is less than 1.35, more detailed studies including detailed wave-in-deck loading and response should be performed.

Table 6-8: Possible design criteria for reserve strength ratio (RSR) and reserve freeboard ratio (RFR) at different values of probability of occurrence

Probability of occurrence	RSR	RFR
10^{-3}	1.5	1.25
10^{-4}	1.95	1.35
10^{-5}	2.5	1.45

7 Assessment of fatigue by simulations

7.1 Introduction

The intension of this section is to illustrate the fatigue degradation of a structure by simulating the life of the structure unto its definitive failure. In the simulations, fatigue crack growth is calculated and components are removed when the component fails. The structural capacity of the jacket is reduced when a component fails. Hence the possibility of overload failure due to wave loading is increased. The structure may fail in intact state or a degraded state when exposed to a wave loading.

The purpose of the simulations is to study the expected performance of a structure designed to a design life L_D and beyond the design life. An increase in failure probability is expected when a structure is used in an extended life, as the failure probability of the individual members is increasing. However, by inspecting and repairing cracked members this increase in failure probability may be significantly reduced.

Within the design life, there should be a relative small probability of failure of the structure. When the structure is used in an extended life, the failure probability is expected to increase as a result of degradation. Information about this increase and the rate of the increase is of great importance when evaluating possible life extensions. The level of inspections and repair may have significant impacts on the time of the increase in failure probability and the rate of that increase. Intuitively, it should be possible with sufficient frequency of inspection and repair to ensure a safe structure well beyond design life. However, the number and frequency of inspections and repairs would be limited by several factors. The cost of very frequent inspections and repairs will be one of these limiting factors, but also time constraints for inspection and repair work in the summer season will limit the access to a component to once every year for most jacket components. The number of repairs of one component may also be a limiting factor e.g. if grinding or repair welding is used. However, a component may be repaired by use of clamping, in which the number of repairs are not limited except possibly from an economically standpoint.

If failure rates of components follow the bath-tub curve (see e.g. O'Connor 1991 p 8), as seen for many other types of components, the failure rate may be expected to be relatively high in the burn-in-phase followed by a relatively low rate and, finally, an increase again in the wear-out-phase. The explanation for how this could be applicable also for structural components would be that gross errors from design and fabrication may result in extensive fatigue in a burn-in-phase. When these cracks have been found and repaired, the structure performs well until the degradation due to fatigue and corrosion occurs according to theories of fracture mechanics and corrosion. Hence, the effect of gross errors should be considered in these simulations especially for the burn-in-phase.

The simulations in this section will be performed in order to study these effects of crack growth, possible member failure, inspection, repair and possible structural system failure. The simulations will further be used to evaluate which failure-modes that are critical at various stages of the installations life.

7.2 Methodology

Simulations are used for studying the expected performance of a structure beyond its original design life, L_D. The failure modes included in the simulation are; i) overload failure in intact state, ii) overload failure with one or several members damaged due to fatigue. Crack growth is modelled according to simple methods and models found in the literature. The focus herein is to include inspections at a realistic level into the simulations. The inspections will have a certain quality, and the probability of finding a crack will be represented by probability of detection (PoD) curves from literature. If a crack is found, it can be assumed to automatically be repaired, or it can be assumed that there is a likelihood of repair depending on the crack size.

7.2.1 Structural description

The structure is a generic jacket where all members have the same thickness, configuration and stress level for a given wave height. This is not fully realistic for a real jacket, but is a conservative approximation.

The stress history in the evaluated components is determined by a long term distribution ensuring a fatigue life of a certain value, e.g. the long term stress distribution is determined so that all components have a 20 year design life with a design fatigue factor (DFF) of 3.0. The non-modelled components are assumed to be unimportant.

The strength of the structure to withstand overload due to wave loading is modelled by the system strength of the structure. This system strength is represented by the reserve strength ratio (RSR). If a structural member fails due to fatigue cracking, the system strength (RSR) is reduced with a factor representing the damaged condition (ref. calculations of RIF values in Section 6 Assessment of system strength).

7.2.2 Overview of simulation procedure

1. Define the design life of all components of the structure (totally optimised with all joints designed to the same design life). All members modelled are assumed designed with a relevant design fatigue factor.
2. The strength of the structure is defined.
3. Stress levels at each joint are defined by a simplified function, depending on the significant wave height. The factors representing the long term stress level distribution is determined to give the above design life including design fatigue factors.
4. The joints have initial cracks and material constants according to assumed distributions.
5. Simulate one year of sea-states, and calculate the crack growth for each sea-state.

6. Check 1: If crack growth has reached a critical level at the end of the year, reduce the strength of the structure accordingly. If a second crack is found, reduce the strength of the structure further.
7. Check 2 (only when load redistribution is studied): If crack growth has reached a critical level, assume that stresses in surrounding joints are increased. If a second crack is found, the stresses in surrounding members will be increased again.
8. Check 3: Check for overload every year for the expected maximum wave height from a distribution of annual maximum wave heights, with the given condition of the jacket (intact or damaged state).
9. Perform inspections according to a given inspection program. Probability of detection according to regular PoD curves.
10. Continue with simulation of next year and so on until failure. Check the state of the jacket at failure (failure with no cracks, failure with one crack, etc).
11. Continue for N simulations

Reducing the strength of the structure when through thickness cracks occur is rather conservative. Studies of growth of through-wall-thickness fatigue cracks in brace members (Pereira 2004) indicate that the remaining number of cycles at through thickness cracks is significant. Pereira (2004) considers the total life of a brace in an offshore jacket structure to be in four parts. The first three parts (N_1, N_2 and N_3) include the growth up to a through thickness crack, and the fourth part (N_4) is from a through thickness crack to the final end of test or final separation of the member. Based on Pereira's findings (Pereira 2004) the number of cycles in the fourth part is approximately similar to the sum for the three first parts. This indicates that the remaining life of the component at through thickness crack is of the same magnitude as the life up to through thickness crack. Hence, the model used in these simulations reduces the strength of the structure at a significantly earlier point than what is realistic.

7.2.3 The purpose and description of cases studied

The following cases are studied in this section:
1. With and without gross errors.
2. With and without inspections and repair.
3. With different inspection programs and inspection methods.
4. With and without load redistribution.

It should be noted that sufficient data about the effect of load redistribution is not obtained. The redistribution will also be structure dependent. The example including load shedding should, as a result of this, only be viewed as an indication of the possible effect of these conditions on the safe life of the structure.

The hypothesis is that if there are no subsidence and wave in deck problems to the structure and the structure is damage tolerant, the failure probability for overload failure will not increase significantly into extended life, if a sufficient inspection and repair program is utilised. The fatigue failure probability of single members is likely to increase, but due to the damage tolerance in most jacket structures a single member failure will not result in a total failure of the structure. Structural system failure is most likely to occur if at least two member's fails due to fatigue and these failures are not found prior to the overload situation.

If these assumptions hold, the life of the structure will be relatively safe until two or more fatigue failures occur between inspections. An important evaluation is whether the inspection performed in these simulations is sufficient to limit the number of simultaneous failures in the structure during the extended life.

7.3 Probabilistic model

Sea-state description
The individual long-term sea-state, described by the significant wave height H_s, is assumed to follow a 3-parameter Weibull distribution, i.e. the distribution function reads:

$$F_{H_s}(h) = 1 - \exp\left[-\left(\frac{h - H_0}{H_C - H_0}\right)^{\gamma}\right] \tag{7.1}$$

where H_0, H_C and γ are parameters of the distribution. The parameters for significant wave height distribution are obtained by fitting a Weibull distribution to the data for the Kvitebjørn field in Northern North Sea (Statoil 2000). The following parameters are used; H_c = 2.883 m, H_0 = 0.2 m and γ = 1.46.

The mean zero crossing period for a given sea-state is, according to Krogstad (2004), set to:

$$T_z = 3.3 \cdot \sqrt{H_s} \tag{7.2}$$

In the simulations a large number of low sea-states are simulated, due to the high probability of theses sea-states. The mean zero crossing period for these sea-states becomes very low. Hence, the number of waves in these sea-states becomes very large. As these sea-states have marginal influence on the fatigue cracking of the structures, a minimum value of 5.72 s is set for the mean zero crossing period, the mean zero crossing period for a sea-state with significant wave height of 3 m.

Maximum wave height
The maximum wave height in one year, H_{max}, is assumed to follow a Gumbel distribution, i.e. the distribution function reads

$$F_{H_{max}}(h) = \exp\left\{-\exp\left[-\frac{h - \alpha_H}{\beta_h}\right]\right\} \tag{7.3}$$

where α_H and β_H are parameters of the distribution. The parameters for wave height distribution are obtained by fitting a Gumbel distribution to the data for the Kvitebjørn field in Northern North Sea (Statoil 2000). The following parameters are used; α_H = 21m and β_H = 1.63m.

Wave load
The wave loading due to an individual wave is approximated by the following equation:

$$W = \alpha \cdot C_1 \cdot H^{C_3} \tag{7.4}$$

where α describes the uncertainty in the wave loading on the structure, H is the wave height, C_1 and C_3 are load coefficients that must be curve-fitted to calculated load data for the specific jacket.

Based on the analysis presented in section 6, C_3 is found to be approximately 2.2 for extreme waves and in the order of 1.1 to 1.5 for the wave heights dominating fatigue[16].

Cyclic fatigue stresses

The cyclic fatigue stresses can be simulated directly from the individual waves. However, in a simulation over several years such a method of calculating the cyclic fatigue stress and crack growth would be very slow. In order to speed up the simulation, an equivalent stress for each sea-state may be used. The equivalent stress is determined such that the actual N stress cycles in a sea-state should result in the same crack growth (or fatigue damage) as the equivalent stress monotonely repeated N times. The equivalent stress can be found by simulating the stress cycles in a sea-state, and calculating the equivalent stress by:

$$\Delta\sigma_{eq} = \left(\frac{\sum_{i}^{N} (\Delta\sigma_i)^{m_a}}{N} \right)^{\frac{1}{m_a}} \tag{7.5}$$

where N is the number of cycles in the sea-state, $\Delta\sigma_i$ is each stress cycle, and m_a is the slope of the SN curve or FM curve.

The individual wave heights in the sea-state are simulated as a narrow banded process, following the Næss distribution for wave heights (Provesto et al 2000):

$$P\left(\frac{H}{m_0^{1/2}} \le x \right) = \left[1 - \exp\left(-\frac{x^2}{4 \cdot (1 - \rho(T/2))} \right) \right] = \left[1 - \exp\left(-\frac{x^2}{\beta} \right) \right] \tag{7.6}$$

where m_0 is the variance, ρ is the autocorrelation function of the wave record and T is a typical wave period. The value of $\rho(T/2)$ is typically found to be between -0.6 and -0.75 for ocean wave spectra (Provesto et al 2000). Næss (1985) has found the value of $\rho(T/2)$ for a Pierson-Moskowitz wave spectra and for a JONSWAP spectra with γ=3.3 and γ=7. In these calculations a JONSWAP spectra is assumed to be a good representation of the wave spectrum, indicating a $\rho(T/2)$ equal to -0.73 (Næss 1985). The β is then set to $\beta = 4 \cdot (1 + 0.73) = 6.92$.

The stress cycle caused by each individual wave is assumed to be according to the following relationship:

$$\Delta\sigma = C_{1F} \cdot H^{C_3} \tag{7.7}$$

[16] For a linear wave loading on a single pile this coefficient would be 1.0 for a fully mass dominant loading, and 2.0 for a fully drag dominant loading. A large wave (e.g. 100 year design wave or higher) the loading will be drag dominated and the coefficient should be close to 2.0. However, due to non-linearities in loading and geometry in a jacket the C_3 coefficient is closer to 2.2 for the extreme wave loading situations, as shown in Section 6. The wave heights dominating the fatigue degradation of a jacket structure is however smaller and is generally more mass dominated. As a result, the C_3 coefficient is typically in the area of 1.1 to 1.5.

where the coefficient C_3 is assumed to be in the order of 1.1 to 1.5, as described previously.

The number of cycles in a sea-state is set to:

$$N_w = \frac{D}{T_z} \tag{7.8}$$

where D is the duration of the sea-state (6 hours in the simulations), and T_Z is the zero crossing period of the sea-state.

Based on the series of Δs found for the sea-state, Δs_{eq} is calculated according to Equation 7.5. Based on several such simulations for different sea-states, a relationship between the equivalent stress and the sea-state parameters (H_s) is studied. In order to obtain a reasonable curve fit of the data, the coefficients C_{1F} and C_3 and the slope parameter for the crack growth or SN curve m_a is studied. The expression for the equivalent stress is developed with these three coefficients as input by manual curve fitting. The resulting curve fit is found to be:

$$\Delta s_{eq} = \frac{C_{1F}}{1.702 - 0.138 \cdot m_a} \cdot (H_s)^{C_3 - 0.03} \tag{7.9}$$

The simulated results and the curve fitted results are plotted in Figure 7-1, and the curve fit shows a reasonably good agreement with the simulated results.

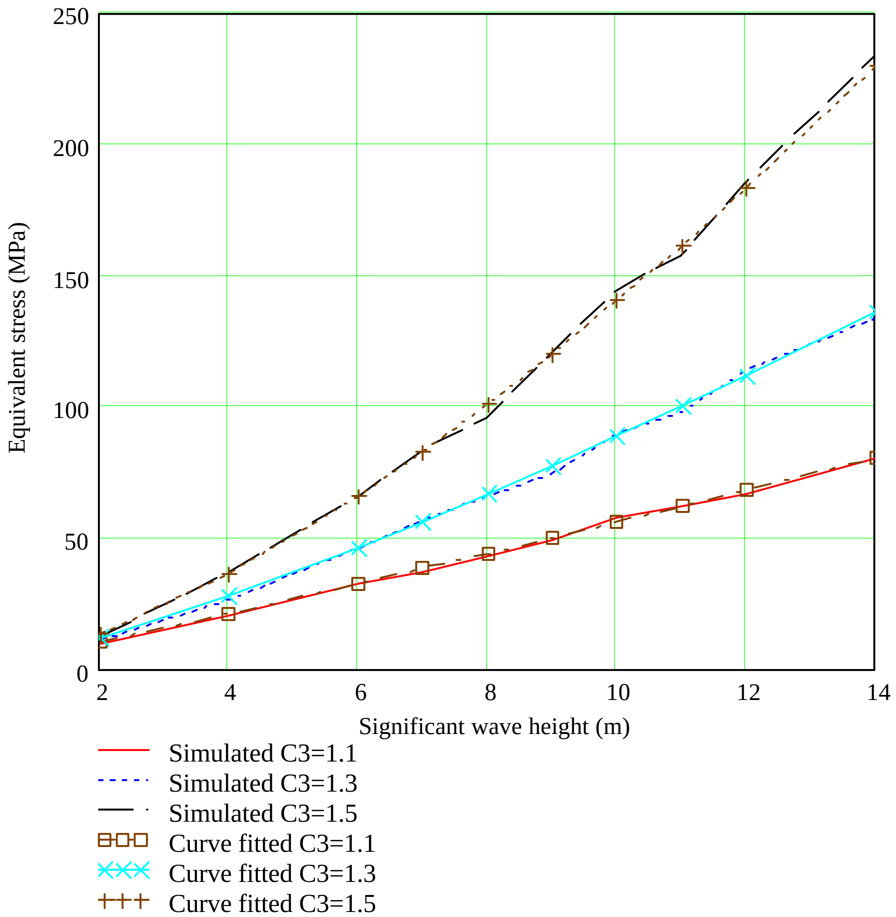

Figure 7-1: Equivalent stress as a function of significant wave height in a sea state including curve fitting. C_1 = 6.125 MPa. SN slope factor m_a=3.0.

The coefficient C_3 is related to the distribution of wave heights and the distribution of stress cycles. Generally, both the wave height distribution and the stress cycle distribution are assumed to be Weibull distributed. The wave height distribution of waves during a year would have a Weibull shape parameter of approximately 1.0, and the stress cycle distribution for a jacket would have a Weibull shape parameter (γ) in the order of 0.7 – 0.9. The C_3 coefficient can be estimated directly from these two relationships.

The distribution of stress cycles (denoted as x here) is given as:

$$F_X(x) = P(X \le x) = P(C_1 \cdot H^{C_3} \le x) = P\left[H \le \left(\frac{x}{C_1}\right)^{\frac{1}{C_3}}\right] = F_H\left\{\left(\frac{x}{C_1}\right)^{\frac{1}{C_3}}\right\} \qquad (7.10)$$

The wave height distribution is given as:

$$F_H(h) = 1 - \exp\left\{-\left(\frac{h}{\alpha}\right)^{\beta}\right\} \tag{7.11}$$

Inserting Equation 7.10 into Equation 7.11 gives that the equivalent stress cycle distribution becomes:

$$F_X(x) = 1 - \exp\left\{\left(\frac{x}{\alpha \cdot C_1}\right)^{\frac{\beta}{C_3}}\right\} \tag{7.12}$$

This results in a shape parameter for the stress cycle distribution of β/C_3. With β in the order of 1.0, C_3 will be a simple relationship with the shape factor for the stress cycle distribution (γ).

$$C_3 = \frac{\beta}{\gamma} \tag{7.13}$$

With γ in the order of 0.7 to 0.9, the C_3 will be in the order of 1.42 to 1.11.

The normal fatigue design procedure for a jacket is to determine the necessary joint thickness and configuration for the given wave loading in order to obtain the appropriate fatigue life of the jacket, including design fatigue factors (DFF). As an example, if the jacket is intended for use in 20 years, the braces in the jacket will be designed for a SN-fatigue life of e.g. 60 years as the design fatigue factor is normally 3 for braces. In these simulations, the C_{1F} factor is used as a scaling-factor to obtain various fatigue life for the jacket. For a given sea-state distribution, the deterministic SN-fatigue life is calculated for a few cases (to take into account the variation in simulated sea-states in the individual simulations). The C_{1F} factor is then chosen to give the appropriate fatigue life of the structure, see Figure 7-2. With the given sea-state parameters and the model used for stress calculations a C_{1F} factor of 6.125 MPa would result in 20 years life (design fatigue factor of 1.0 for a jacket intended for use in 20 years), a C_{1F} factor of 4.88 MPa would result in 40 years life (design fatigue factor of 2.0) and a C_{1F} factor of 4.25 would result in 60 years life (design fatigue factor of 3.0).

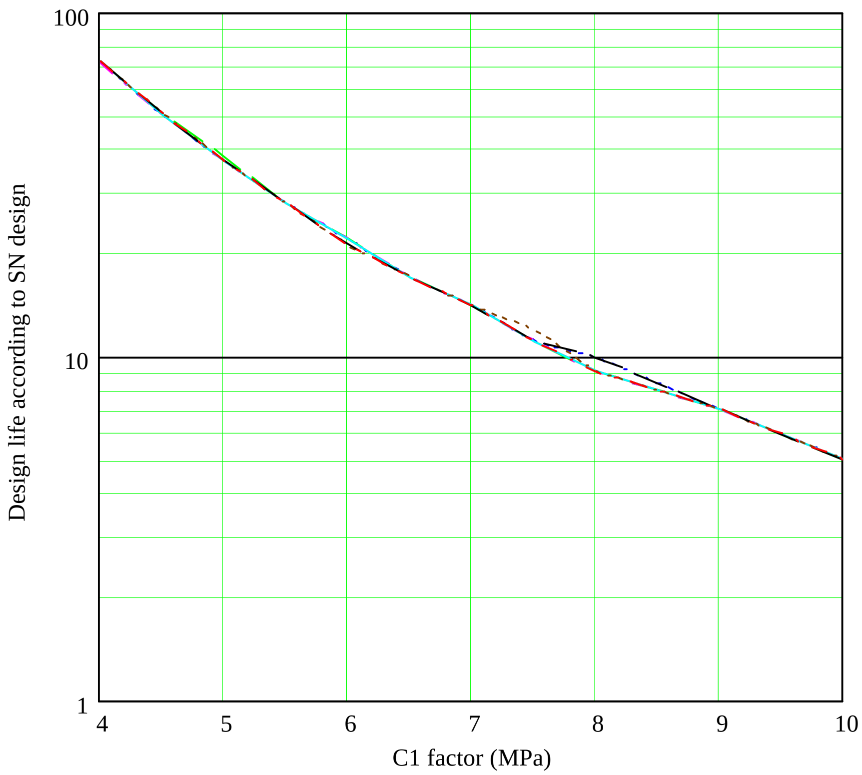

Figure 7-2: C_{1F} coefficient versus design SN-fatigue life (in years)

The purpose of using individual waves to calculate crack growth is that the effect of having a large stress cycle will open the crack, and the later stress cycles will have larger effect on the crack growth than without the early large stress cycle. This is not fully included in a method with the use of equivalent stress crack growth calculation. However, by using 6 hours sea-states, the effect of a large sea-state at an early stage in the life of the jacket will have some of the same effect.

Fatigue damage
Based on the equivalent stress level in a sea-state, the fatigue SN curve will give a number of stress cycles at this stress level that the structure can tolerate before the crack is assumed to be through thickness.

The number of cycles that the structure can tolerate of a given stress level is given by the SN curve, and can be calculated according to:

$$\log(N_{SN}) = \log(a) - m_a \cdot \log(\Delta s)$$
$$N_{SN} = \frac{a}{(\Delta s_{eq})^{m_a}} \tag{7.14}$$

where Δs_{eq} is the equivalent stress level, m_a and a are the SN curve parameters.

The damage can be calculated by:

$$d = \frac{N}{N_{SN}} \tag{7.15}$$

where d is the damage, N is the actual number of cycles and N_{SN} is the number of cycles the detail is assumed to tolerate (indicated by the SN curve).

Based on the Miner sum the damage from several sea-states can be calculated as the sum of the damage from each sea-state.

$$d_{Tot} = \sum_i \frac{N_i}{N_{SN}\left(\Delta\sigma_{eq_i}\right)} \tag{7.16}$$

for all sea-states "i".

The SN-based damage criterion given here is used to evaluate the life of the component according to traditional design methods.

Fracture mechanics crack growth

Crack growth is by fracture mechanics given by the Paris equation as:

$$\frac{da}{dN} = A \cdot \Delta K^m \tag{7.17}$$

where "A" and "m" is parameters of the crack growth curve, and:

$$\Delta K = \Delta\sigma \cdot F \cdot \sqrt{\pi \cdot a} \tag{7.18}$$

where $\Delta\sigma$ is the stress cycle, F is a geometry function and "a" is the instant crack depth.

The geometry function is dependent on the geometry of the detail under evaluation. For jacket tubular connections, the geometry function is often given by (see e.g. Dalane 1993):

$$F(a,t) = \left(1.08 - 0.7 \cdot \frac{a}{t}\right) \cdot \left(1.0 + 1.24 \cdot e^{-22.1 \cdot \frac{a}{t}} + 3.17 \cdot e^{-357 \cdot \frac{a}{t}}\right) \tag{7.19}$$

where "a" is the instant crack depth and "t" is the thickness of the material.

For each sea-state the crack growth is calculated by (see e.g. Dalane 1993):

$$da_i = A \cdot \left[\Delta\sigma_{eq} \cdot F(a_{i-1}, t) \cdot \sqrt{\pi \cdot a_{i-1}}\right]^m \cdot N_w \tag{7.20}$$

where N_w is the number of cycles in the sea-state.

Initial crack depth

The initial crack depth of the joint welds is assumed to be exponentially distributed with a mean value of 0.11mm, as recommended by DnV (1996), with density and distribution functions as:

$$f(x) = \lambda \cdot e^{-\lambda \cdot x} \tag{7.21}$$

$$F(x) = 1 - e^{-\lambda \cdot x} \tag{7.22}$$

where λ is $\lambda = \frac{1}{\mu_{a_0}}$, and μ_{a_0} is the mean initial crack depth.

In addition a probability of large initial crack as a result of a gross error is included. In case of a gross error, the initial crack depth is simulated from a uniform distribution between 0.2 and 5mm.

Aker Maritime (1998) has evaluated a large number of cracks found in inspection on several jacket installations. A total of 3366 inspections have been evaluated. In these inspections a total of 511 cracks have been found. Out of these 511 cracks, 80 – 283 cracks have been evaluated to be fatigue cracks, with 90% and 30% certainty, respectively. The remaining cracks have to be due to a type of gross error. If the highest value of number of fatigue cracks is used, the number of cracks found that is due to gross errors is 511-282 = 229. Taking into account that 3366 joints are inspected, this gives a probability of 0.068 for a gross error. In the simulations, a slightly lower value of probability of gross error equal to 5% is used, as all cracks due to a gross error are submitted to crack growth in these analyses.

Calibration of the fracture mechanics model

In these analyses the fatigue SN calculation based on parameters from NORSOK N-004 (NORSOK 1999) is taken as the basis, and the fracture mechanics parameters are calibrated to give similar result as the SN calculation. The C_{1F} factor is determined so that the SN fatigue calculation results in the required fatigue life of the component. The initial crack size is taken as described earlier. The remaining parameter to calibrate is then the A and m factors describing the fracture mechanic crack growth curves.The C_{1F} factor is decided in order to give a design SN fatigue life of 20 years.

The limit state function for SN fatigue calculation is defined as:

$$g = 1 - d_{Tot} = 1 - \sum_i \frac{N_i}{N_{SN}\left(\Delta\sigma_{eq_i}\right)} \tag{7.23}$$

According to NORSOK N-004 (NORSOK 1999) the one-slope SN-curve for tubular joint connections (T-curve) is described by log(a) = 11.764 and slope of the SN-curve described by m=3.0. The mean value of log(a) is assumed to be 2 standard deviations larger. The standard deviation of log(a) in the SN-curve is assumed to be 0.25, resulting in a mean value of log(a)=12.264.

The limit state function for fracture mechanic crack growth is defined as:

$$g = a_c - \sum_i da_i = a_c - \sum_i A \cdot \left[\Delta\sigma_{eq} \cdot F\left(a_{i-1}, t\right) \cdot \sqrt{\pi \cdot a_{i-1}}\right]^m \cdot N_w \tag{7.24}$$

where a_c is the critical crack size = 25 mm and N_w is the number of cycles in a sea-state. The slope of the fracture mechanic curve is fixed to 3.0, as for the SN-curve.

In the calibration, the same failure probability, P(t>T), of the two limit state functions is sought. The probability P(t>T) in the simulations are defined as:

$$P(t > T) = \frac{N_c(L > T)}{N_c} \tag{7.25}$$

where $N_c(L > T)$ indicates the number of simulated components with life, L, larger than T years, and N_c indicates the total number of simulated components.

The selected calibration is shown in Figure 7-3, with a ln(A) coefficient in the crack growth curve of -28.8. It can be seen from Figure 7-3 that the failure probability of the two limit state functions do not coincide at the full length of the curves. The fracture mechanic crack growth model with the above mentioned ln(A) gives a reasonable fit up to approximately the design life is reached (20 years). For $t > L_D$ the chosen ln(A) value is slightly conservative. From Figure 7-3 it can be seen that the probability of failure at the design life of the component is approximately 2.8%, which is in agreement with the anticipated failure probability of a component designed according to codified design SN curves.

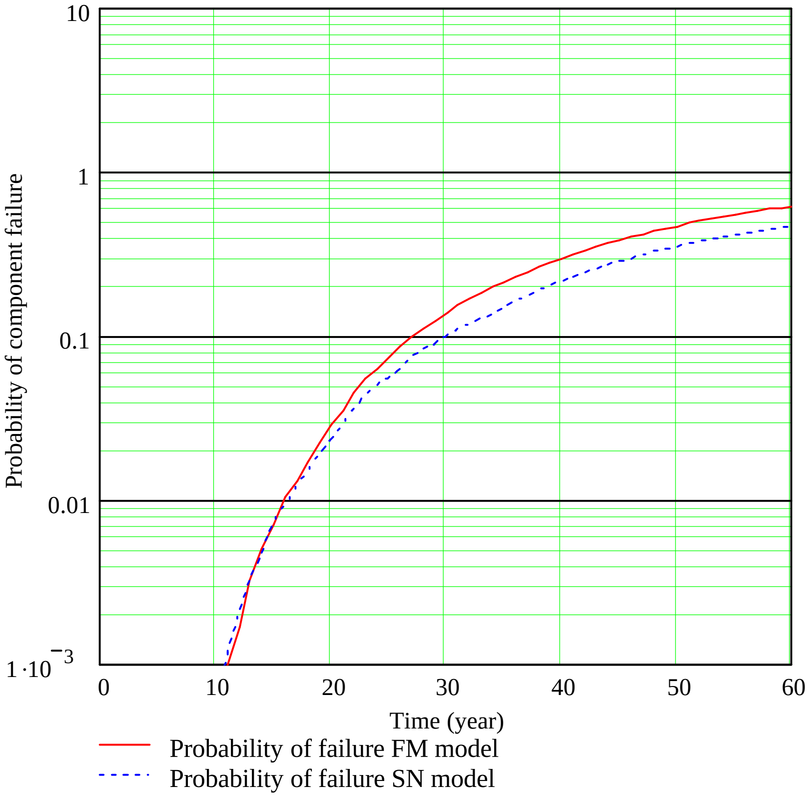

Figure 7-3: Resulting probabilities of failure using FM model and SN model after calibration

The model parameters used in the analyses, based on this calibration is shown in Table 7-1.

Table 7-1: Model parameters from calibration

Parameter	Distribution type	Mean	Standard deviation
FM material parameter ln(A)	Normal	-28.85	0.5
FM material parameter m	Fixed	3	
Initial crack size	Exponential	0.11 mm	
SN material parameter log(a)	Normal	12.264	0.25
SN material parameter m	Fixed	3	
Thichness	Fixed	25 mm	

The design life of the components according to the calibrated values of ln(A) is checked using design values for both SN curve and crack growth curve. The design crack growth analysis is performed with a fixed value of ln(A) equal to 2 standard deviations. The resulting design life for the two analyses is shown in Figure 7-4. The uncertainty in the sea-state results in a small uncertainty in the SN design life. The C_{1F} factor is chosen in order to have 20 years life as a minimum life for the simulated cases. The slightly larger uncertainty in the fracture mechanics crack growth curves is due to the uncertainty in the initial crack size that is still modelled as a stochastic variable.

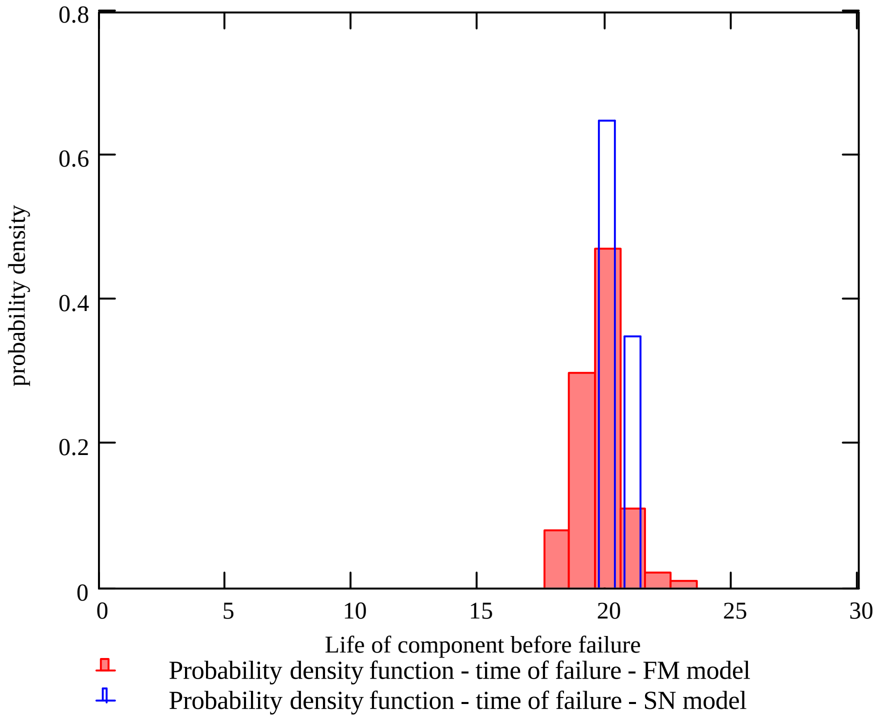

Figure 7-4: Calculated life of component using deterministic design values for FM crack growth curves and SN fatigue curves.

Resistance
At the end of each simulated year, the structure is exposed to the expected annual maximum wave taken from the Gumbel distribution presented earlier.

The initial resistance is modelled as an ultimate capacity of the structure, described on a system basis. The expected value of the ultimate capacity is assumed to be equal to the 100 year loading ($C_1 \cdot H_{100}^{2.2}$) multiplied by the reserve strength ratio and a factor counting for model uncertainty.

$$R = x \cdot RSR \cdot C_1 \cdot H_{100}^{C_3} \qquad (7.26)$$

where H_{100} is the 100 year wave height (a wave height with the annual probability of exceedance of 10^{-2}), RSR is the reserve strength ratio of the structure, i.e. the ratio between ultimate collapse load of the structure and the design load (the load with the annual probability of exceedance of 10^{-2}), and x describes the uncertainty in the resistance of the structure.

If a member is damaged due to crack growth (crack growth through thickness), the resistance is reduced with a factor RIF (ResIdual strength Factor – defined as the load capacity of the structure after one or several components have failed versus the ultimate collapse load of the structure). If "k" members are damaged, the resistance is reduced by RIF^k.

A failure function for ultimate collapse of the structure can be modelled by the following equation:

$$g = R - W = x \cdot RSR \cdot RIF^k \cdot C_1 \cdot H_{100}^{2.2} - a \cdot C_1 \cdot H^{2.2} \qquad (7.27)$$

Table 7-2: Parameters for overload calculation

Parameter	Description	Values
x	Uncertainty about the resistance of the structure.	Normal distributed - mean value = 1.0 - Standard deviation = 0.1
RSR	Reserve strength ratio	Fixed at 2.0
RIF[17]	ResIdual strength Factor	Fixed at 0.8
H_{100}	100 year wave (Wave height with a annual probability of exceedance of 10^{-2})	Fixed at 28.6 m
a	Uncertainty about the wave loading on the structure.	Normal distributed: - mean value = 1.0 - Standard deviation = 0.15
C1	Load coefficient	Fixed at 1.0

[17] The RIF values used here is based on the calculations of system strength found in Section 6.

Inspections
If the structure survived the simulated annual maximum wave height, a simulated inspection is performed.

Probability of detection (POD) curves used here is according to DNV's Guideline for Offshore Structural Reliability (DNV 1996). The POD curve is given as (DNV 1996 – Examples for Jacket platforms p 16):

$$POD(2\cdot c) = 1 - \frac{1}{1+\left(\frac{2\cdot c}{x_0}\right)^b} \tag{7.28}$$

where $2\cdot c$ is the length of the crack.

Typical values for x_0 and b for different scenarios are given in Table 7-3 (DNV 1996 – Examples for Jacket platforms p 16).

Table 7-3: POD distribution parameters (DNV 1996)

Inspection scenario	x_0	b
MPI under water	2.95	0.905
Eddy Current	12.28	1.790

The fatigue crack aspect ratio a/c is assumed to be 0.15 (Dalane 1993). In the simulations the crack depth is modelled, and the detected crack depth is then set to $a = 1/2\cdot 0.15\cdot(2\cdot c)$ where $2\cdot c$ is drawn from the POD distribution given in Equation 7.28.

With respect to flooded member detection (FMD), it is not possible to use POD curves. The flooded member detection does not work for anything but through thickness cracks. If a through thickness crack has occurred, the member may be partially flooded or fully flooded. Limited data has been found on probability of detection of a crack with flooded member technique. Visser (2002) reports a limited number of test results. When the member was fully flooded or 50% flooded, the FMD identified the flooding in all cases evaluated (20 cases). When the member was 10% flooded, 3 out of 10 cases were unidentified. A thorough model for POD with FMD would, based on these data, also include a model for the probability of degree of flooding. However, as a simplified approach it is here evaluated that 3 out of 30 cases where unidentified, resulting in a 90% probability of detection with the use of flooded member detection as a conservative approach.

Load redistribution
When a structural member fails due to a through thickness crack, the load this member was carrying will be redistributed to other members in a redundant structure. To study the possible effect of load redistribution, the stresses in the remaining joints are increased with a certain factor when a member fails. In a simple two dimensional X-frame, a shear load may be equally distributed in the two braces. If one of these braces fails, the remaining brace will take the full load, two times the load the member initially experienced. In a jacket structure, the additional X-frames in the same bay will also take some of this additional load. Sufficient studies on the increased stress level when removing a member are not performed, and would be dependent on the specific jacket structures. As an illustration of the effect a stress increase of 20 % in the remaining members when a member fails is assumed in these analyses.

<u>Simulation procedure</u>
For each simulated case:

- Draw the initial strength for the structure according to $R = \xi \cdot RSR \cdot C_1 \cdot H_{100}{}^{C_3}$
- Draw the wave load parameters according to $W = \alpha \cdot C_1 \cdot H^{C_3}$
- Draw a initial crack size for the joint from $f(x) = \lambda \cdot e^{-\lambda \cdot x}$
- Draw material parameter for crack growth curve from $\ln A = N(\ln A_{\mu}, COV_{\ln A})$
- Simulate for each year of the structures life:
  - For each sea-state in a year:
    - Draw a Hs from $F_{H_s}(h) = 1 - \exp\left[-\left(\dfrac{h - H_0}{H_C - H_0}\right)^{\gamma}\right]$
    - Calculate Tz from $T_z = 3.3 \cdot \sqrt{H_s}$ with a minimum value of 5.72 s.
    - Calculate equivalent stress cycle for sea-state from
      $\Delta\sigma_{eq} = \dfrac{C_1}{1.702 - 0.138 \cdot m_a} \cdot (H_s)^{C_3 - 0.03}$
    - For each joint crack:
      - Calculate the geometry parameter with the actual crack size from:
        $$F(a,t) = \left(1.08 - 0.7 \cdot \frac{a}{t}\right) \cdot \left(1.0 + 1.24 \cdot e^{-22.1 \cdot \frac{a}{t}} + 3.17 \cdot e^{-357 \cdot \frac{a}{t}}\right)$$
      - Calculate the crack increment from
        $da_i = A \cdot \left[\Delta\sigma_{eq} \cdot F(a_{i-1}, t) \cdot \sqrt{\pi \cdot a_{i-1}}\right]^m \cdot N_w$
      - New crack size $a_i = a_{i-1} + da_i$
      - Check if crack is larger than thickness
  - Draw a yearly maximum wave height from $F_{H_{\max}}(h) = \exp\left\{-\exp\left[-\dfrac{h - \alpha_H}{\beta_h}\right]\right\}$
  - Strength of structure is corrected for the number of member failures (k), and the failure in this year is checked according to
    $g = R - W = \beta \cdot RSR \cdot RIF^k \cdot C_1 \cdot H_{100}{}^{2.2} - \alpha \cdot C_1 \cdot H^{2.2}$
  - If structure fails, exit loop.
  - Perform inspection according to inspection program. If node is inspected, draw the smallest detectable crack length from $POD = 1 - \dfrac{1}{1 + \left(\dfrac{2 \cdot c}{x_0}\right)^b}$
  - Detected crack depth is set to 0.15·c.
  - If actual crack depth is larger than the detected crack depth, the crack is assumed to be repaired. New crack size as for a new joint.
- Check how many components that has failed at failure of system
- Check at which year the structure did fail

Comments
In order to obtain a transparent and simple model with fatigue cracking similar to historical observed crack rate, some uncertainties are not included in these analyses. As an example, uncertainty about the stress from stress concentration factors is not included. Further, the use of Miner-Palmgren summation when calculating fatigue of a component exposed to stress cycles of different magnitude gives, as mentioned previously, a larger stochastic variation in the fatigue life of the component than a constant amplitude stress cycle. Additional uncertainty in the fatigue life of the components, accounting for this observed stochastic variation, is not included. However, the same uncertainties are used in all analyses, SN-based fatigue analysis and fracture mechanic crack growth analysis. The effect of including these uncertainties about the fatigue life of a component is likely to be that a limited number of cracks would occur at an earlier stage. Also a limited number of cracks would occur at a later stage than computed in these analyses. This would imply that the point where the no-inspection curve exceeds the accumulated Pf(L>R) curve may be shifted to an earlier year. As structures are generally inspected, this would not impact the life extension in general. It may also increase the number of occurring cracks in the earlier years of the inspected simulations. However, this does not necessarily alter the conclusions as inspections seem to hold the probability of failure to a reasonable level for a period longer than expected for life extensions. Similar effect would be the result of including a larger uncertainty about the stresses in each member.

A lot of the assumptions made in the simulations are very conservative, especially the assumption that the component fails at through thickness crack.

7.4 Results

All simulations are performed with 20 years design life of the members in the structure and design fatigue factor of 3. Hence the design SN life of the components should be 60 years. The simulated jacket consists of 20 components (nodes), and the jacket can fail as an intact system or as a damaged system after a fatigue failure of one or more components. A total of seven cases are evaluated, as presented in Table 7-4.

Table 7-4: Description of cases in the simulation. FMD denotes flooded member detection, EC denotes eddy current inspection, FB denotes freeboard, WID denotes wave-in-deck, LR denotes load redistribution, C denotes corrosion and S denotes subsidence in meters per year.

Case	Gross error	Inspection and repair	Inspection interval	Load redistribution	Freeboard (subsidence)	Corrosion
1.0	No	No	N.A.	No	•	No
1.1	Yes	No	N.A.	No	•	No
2.0	Yes	FMD	3 years	No	•	No
3.0	Yes	EC	4 years	No	•	No
3.1	Yes	EC	5,6,9,4,4,…y	No	•	No
4 - WID	Yes	EC	5,6,9,4,4,…y	No	FB=23.5 (S=0.1m/y)	No
4 - LR	Yes	FMD	3 years	20% to all components	•	No

The failure probability for an intact jacket structure with a RSR= 2.0 exposed to an annual maximum wave load are used as a reference case for annual failure probability, based on the limit state function:

$$g = \xi \cdot RSR \cdot C_1 \cdot H_{100}{}^{C_3} - \alpha \cdot C_1 \cdot H^{C_3} \tag{7.29}$$

The parameters in this limit state are described in Section 6 Probabilistic analysis of a jacket based on the RSR factor. It is in Section 6 found that the annual failure probability of a jacket with a RSR=2.0 is, based on 10^8 simulations, $7.25 \cdot 10^{-5}$ for the environmental parameters used in this simulations.

7.4.1 *Case 1.0 and 1.1 – no inspections*

The purpose of case 1.0 is to evaluate the occurrence of cracks in the structure as they would be expected according to SN design curves. In design of a new structure with the use of SN fatigue curves, inspections are not an implicit part of the fatigue calculations.

In Figure 7-5 the probability P(T<t) that the life of the component T is less than the variable t on the x-axis is illustrated based on this simulation. It can be seen that the fracture mechanics calculation is slightly conservative, and that the probability of the component exceeding life of 60 years (design life 20 years and design fatigue factor of 3) is approximately 2.5% as expected.

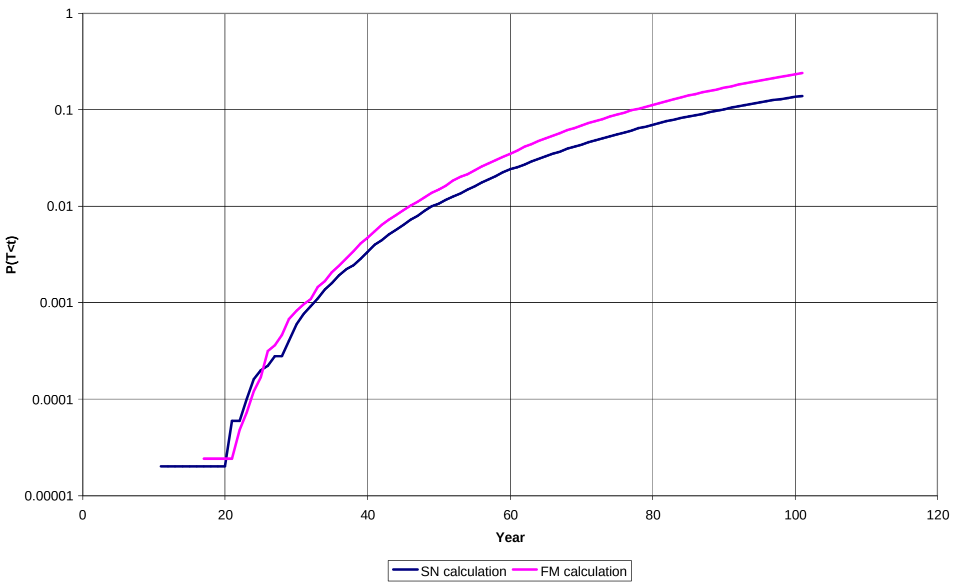

Figure 7-5: Distribution function of component life for SN calculation and fracture mechanic (FM) crack growth calculation.

Rate of occurrence of cracks larger than 1 mm

The purpose of studying the occurrence of cracks is to evaluate the rate of cracks in the jacket as a function of the age of the jackets and compare this with historic data. Several jackets have been in operation in the Norwegian Continental Shelf (NCS) and United Kingdom Continental Shelf (UKCS), and historic data of detected cracks are available

(Stacey et al 2002). Various inspection methods are used, and a reasonable crack size is difficult to assess. It is here chosen to use a depth of the cracks of 1 mm as the criteria for including the cracks in the rate of occurrence. The crack lengths at 90% of detection with the given POD curves are 42mm for magnetic powder inspection and 33mm with eddy current inspection (DNV 1996). The corresponding mean values for detected cracks are obtained by (Aker Engineering 1990):

$$l = -\frac{(2 \cdot c)_{d,p}}{\ln(1-p)} \tag{7.30}$$

where $(2 \cdot c)_{d,p}$ is the detectable crack length with probability limit p.

The mean length ($\overline{2 \cdot c}$) for detected cracks are then 14 mm for magnetic powder inspection and 18 mm for eddy current inspection. The expected crack depth $a = 0.15 \cdot c$, is then 1 mm for magnetic powder inspection and 1.3 mm for eddy current inspection.

To study the rate of the occurrence of cracks, the hazard function is used. This function can be interpreted as the instantaneous rate of specimen failure at the time *t* (Bury 1975). The hazard function, also called age specific failure rate or conditional failure rate (Dalane 1993), expresses the likelihood of failure in a time interval t to $t + \Delta t$ as $\Delta t \to 0$, given that failure has not occurred prior to time t. The hazard function can be written as (see e.g. Dalane 1993, Bury 1975 or Benjamin and Cornell 1970):

$$h(t) = \frac{P(t \le T \le t + \Delta t)}{P(t \le T)} \quad \text{as } \Delta t \to 0 \tag{7.31}$$

where T is the time to failure for the component.

The fatigue damage process is typically a time dependent process and the hazard function typically increases with time. However, possible effects of human errors may alter the hazard function of fatigue cracking. Human errors may result in abnormal initial crack size, abnormal stress concentrations and abnormal wave load processes.

The rate of cracks larger than 1 mm for case 1.0 is shown in Figure 7-6. As expected the hazard function is an increasing function increasing from zero to a rate of approximately 0.05 at the end of the design life (20 years). In a possible extended life the hazard rate continue to grow, but at a lower slope, up to a rate of approximately 0.06 where it appears to be approximately constant. The oscillations seen are assumed to be due to the limited number of simulations.

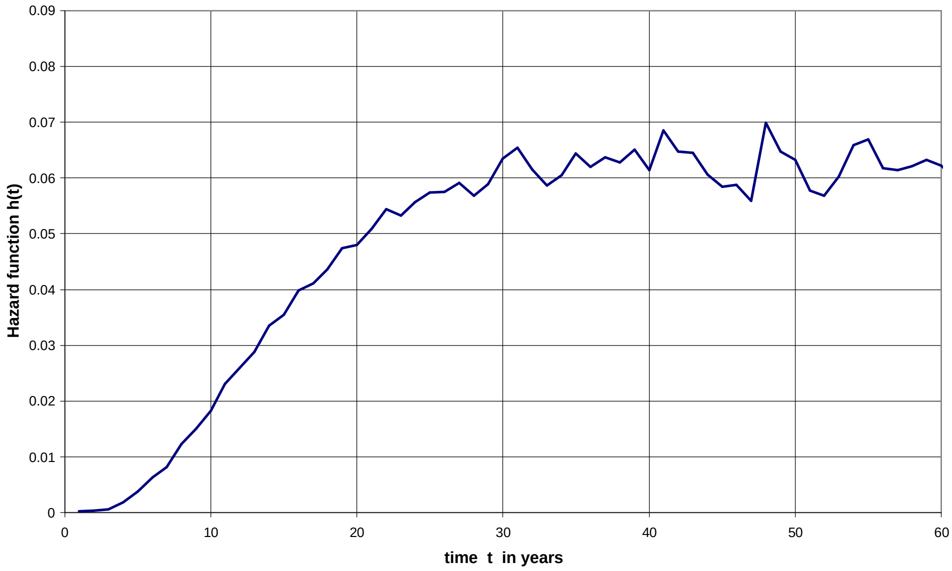

Figure 7-6: Hazard function for the occurrence of cracks larger than 1 mm in case 1.0. It can be seen that a mean value of the rate of occurrence of cracks in the design life for each component is approximately 0.025 p.a.

The purpose of case 1.1 is to evaluate the cracking rate in a more realistic situation where gross errors in form of abnormal initial crack size are included. When including gross errors it is expected that the hazard function would form a curve more like the bath-tub-curve. The hazard function based on the simulations in case 1.1 is shown in Figure 7-7. A clear indication of an increased number of cracks in the first years can be seen as a result of the modelled gross errors. It can be seen that a mean value of the rate of occurrence of cracks in the design life for each component is around 0.03 p.a. The hazard function when including gross errors is not significantly different from the case without gross errors in the time after the design life. The expected indication of a bath-tub curve is seen, where the burn in phase is represented by the large initial cracks due to gross errors and the wear out phase is defined by fatigue crack growth.

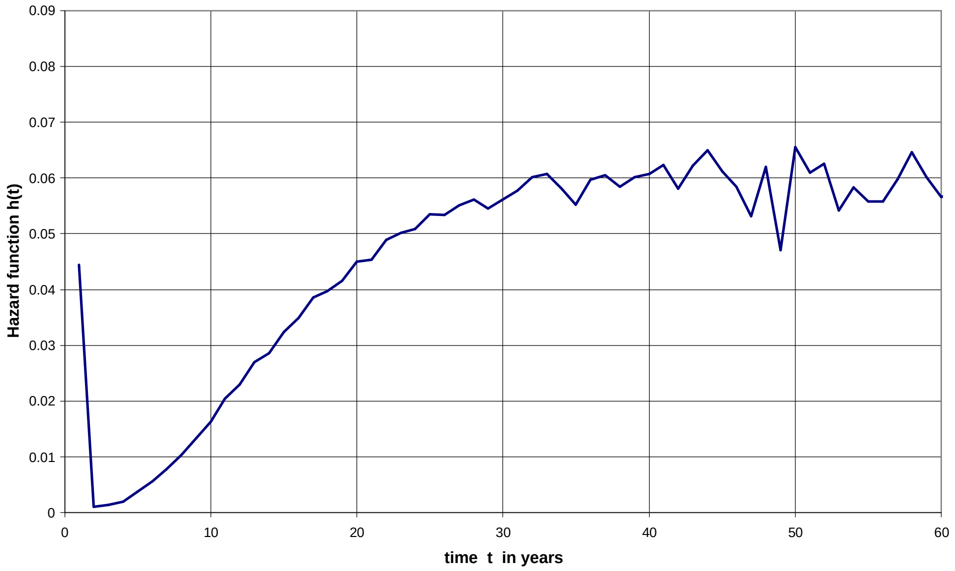

Figure 7-7: Hazard function for the occurrence of cracks larger than 1 mm in case 1.1.

In Stacey et al (2002) it is reported that based on historical data the probability of fatigue cracking in each node is approximately $2 \cdot 10^{-2}$ per annum for UK data and $5 \cdot 10^{-2}$ per annum for Norwegian data. As seen from Figure 7-6, these simulations indicate a rate of cracking of approximately $2.5 \cdot 10^{-2}$ as a mean value in the design life. When including abnormal initial cracks sizes to illustrate the effect of gross errors, the hazard rate function is as illustrated in Figure 7-7 with a rate of cracking of approximately $3 \cdot 10^{-2}$ as a mean value in the design life. The resulting hazard function for both cases is in the same region as the historical data indicate.

Using the same methodology as previously presented for rate of occurrence of small cracks, also the rate of occurrence of through thickness cracks are illustrated. As seen from Figure 7-8, the rate of through thickness cracks starts to increase at approximately the design life of the structure.

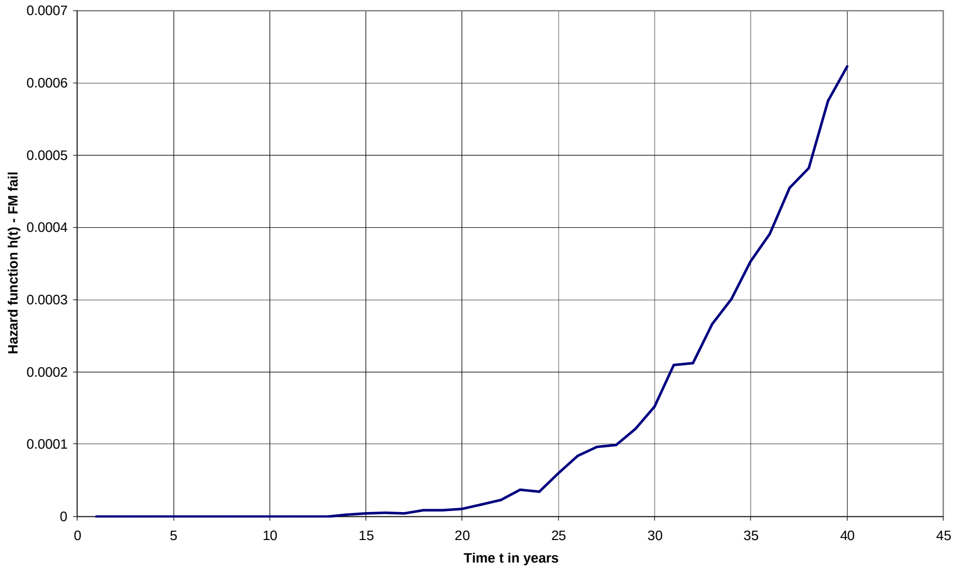

Figure 7-8: Hazard function for through thickness cracks case 1.1 based on fracture mechanics crack growth analyses.

The accumulated probability of failure, $P(T < t)$, and the annual probability of failure $P(T = t)$ are presented in Figure 7-9 and Figure 7-10 respectively. As a reference the accumulated and annual failure probability (probability of failure of $7.25 \cdot 10^{-5}$) of the intact system exposed to annual maximum wave loads is presented. It can be seen from these two figures that the probability of failure is similar to the failure probability of the intact structure up to 30 years. This would imply that, according to this simplified model, failure probability is not influenced significantly by degradation in the first 30 years. After 30 years of life, the failure probability increases gradually into an unacceptable level of safety.

One of the goals with inspections should be to keep the failure probability to a level not exceeding the reference value significantly during the extended life.

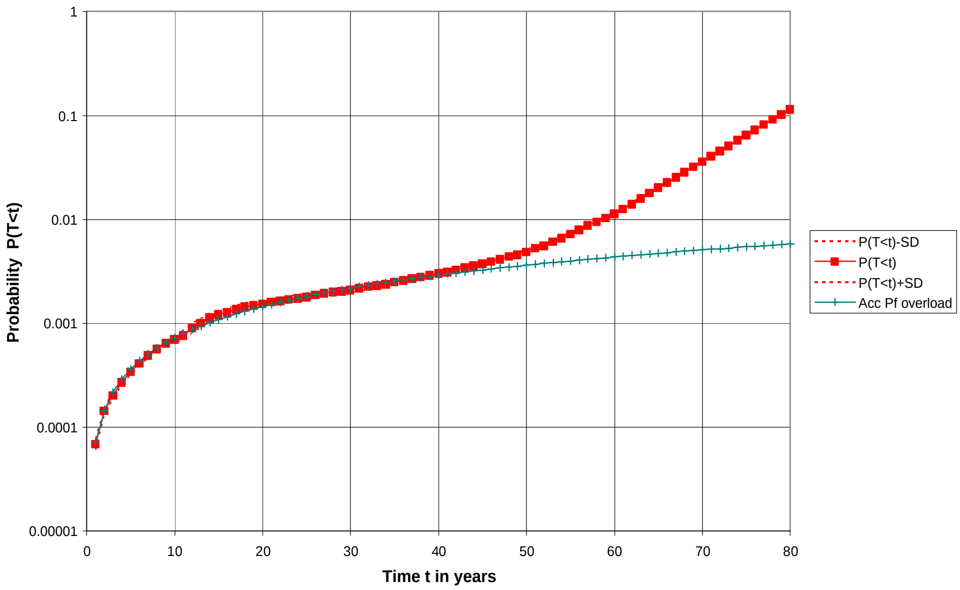

Figure 7-9: Cumulative jacket life distribution for case 1.1, including confidence bands of +/- one standard deviation (SD) and accumulated probability of failure for an intact structure (denoted Acc Pf overload). Data are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

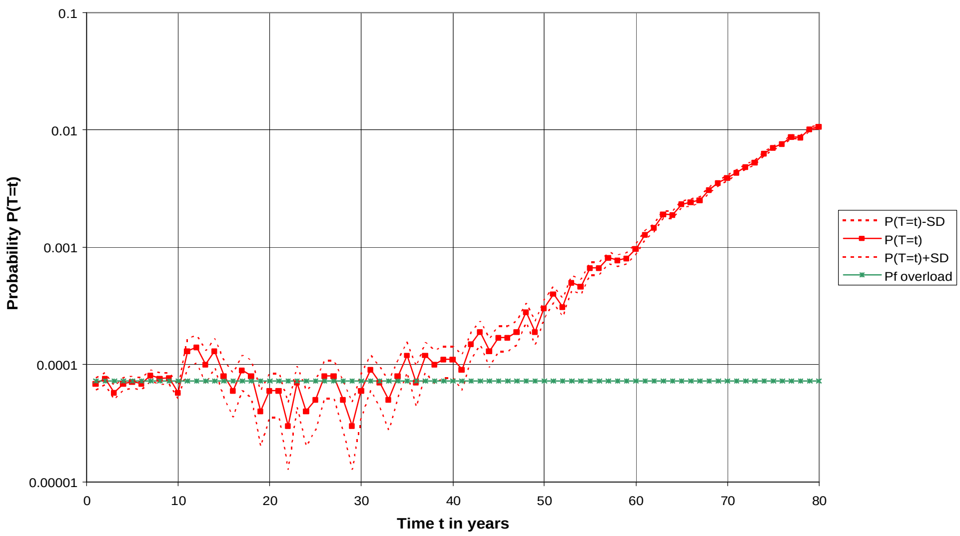

Figure 7-10: Annual probability distribution of jacket life for case 1.1, including confidence bands of +/- one standard deviation and probability of failure for an intact structure (denoted Pf overload). Oscillation in the predicted failure probability is due to limited number of Monte Carlo simulations, as seen in the more moderate oscillations in the ten first years. Data are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

Figure 7-11 illustrates the contribution to the total failure probability given the number of component fatigue failures prior to system failure. The largest contribution to the failure probability P(T=t) in the last years of the simulations are from system failure with 2 – 6 component failures.

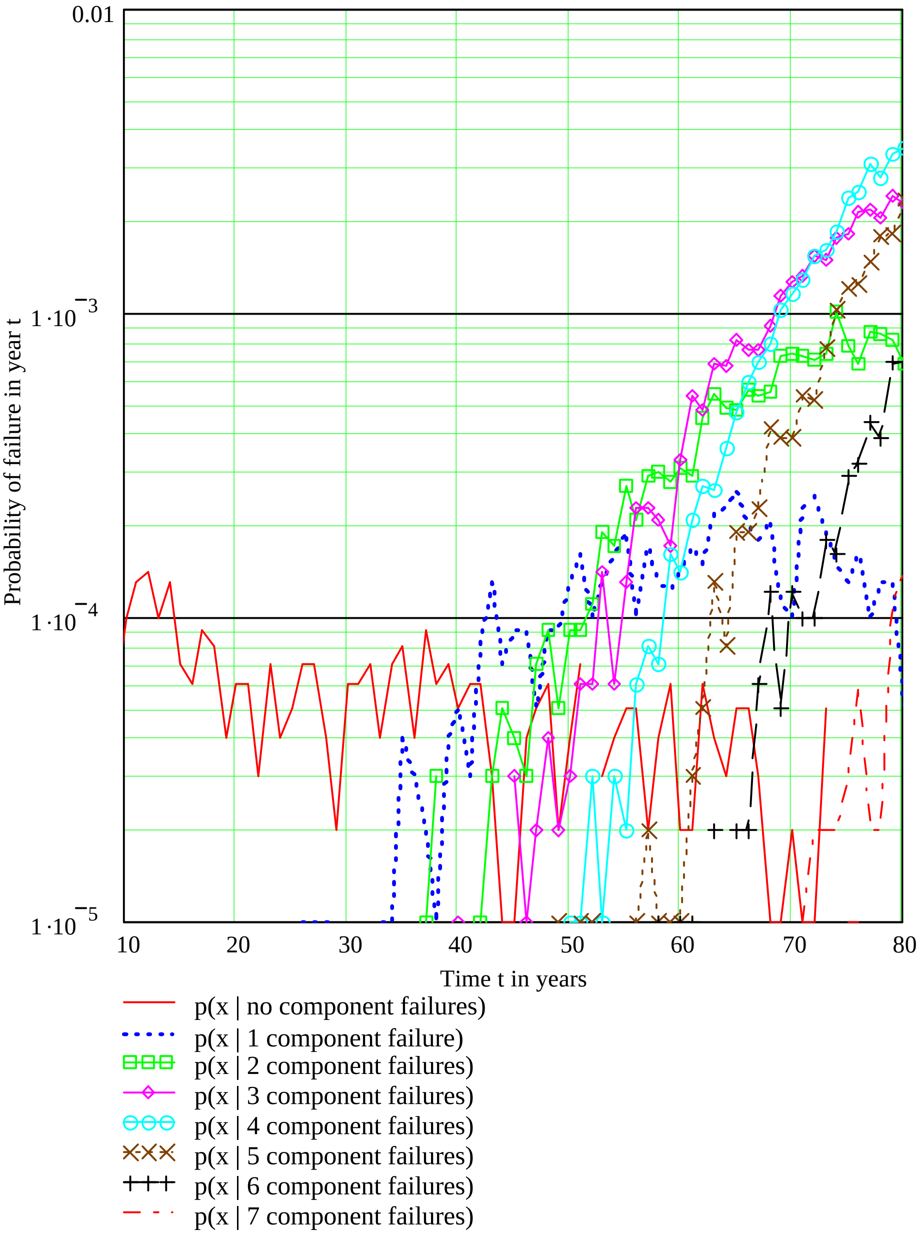

Figure 7-11: Annual probability of failure in year t given the number of component failures due to fatigue when system failure occurs for case 1.1. Oscillation in the predicted failure probability is due to limited number of Monte Carlo simulations. This figure is based on 100 000 simulations.

Based on these results, the most important hazard for a jacket structure is when 2 or more components fail. Under such a scenario, the total failure probability is increasing significantly. It should be noted that this is likely to take place well beyond the platform design life (20 years). The most important aspect of the inspections in an extended life is to keep the number of component failures to no more than one. This would result in only marginal increase in the failure probability.

7.4.2 Case 2.0 Inspection with FMD - 3 years interval

Flooded member detection (FMD) is increasingly used as an inspection method. The method is only capable to identify through thickness cracks. The method can not be used for components that are water filled or components that are grouted. Possible repair methods for a through thickness crack are underwater welding and clamping. Data on initial crack sizes after underwater repair are not available. As the purpose of the simulations is to evaluate the effect of inspections, rather than calculating the possible life of the jacket structures, a repaired crack is modelled with an initial crack size after repair according to the exponential distribution with a mean crack size of 0.11 mm. This is the same as for the initial crack distribution at design. No possibility for a gross error during repair is modelled.

The accumulated probability of failure $P(T < t)$ and the annual probability of failure $P(T = t)$ are presented in Figure 7-12 and Figure 7-13 respectively. As for the previous case, the accumulated and annual failure probability of the intact system exposed to annual maximum wave loads, as presented in Section 6, is included as a reference. It is seen from the figures that the probability of failure is similar to the reference values up to around 40 years. After 40 years the failure probability increases slightly above the reference level, but the increase is slower that for case 1.1.

An important goal of inspections is to keep the failure probability at a level not exceeding the reference value also for the extended life. This goal is not fully obtained by flooded member detection inspection for the full period. However, a significant improvement from case 1.1 is seen.

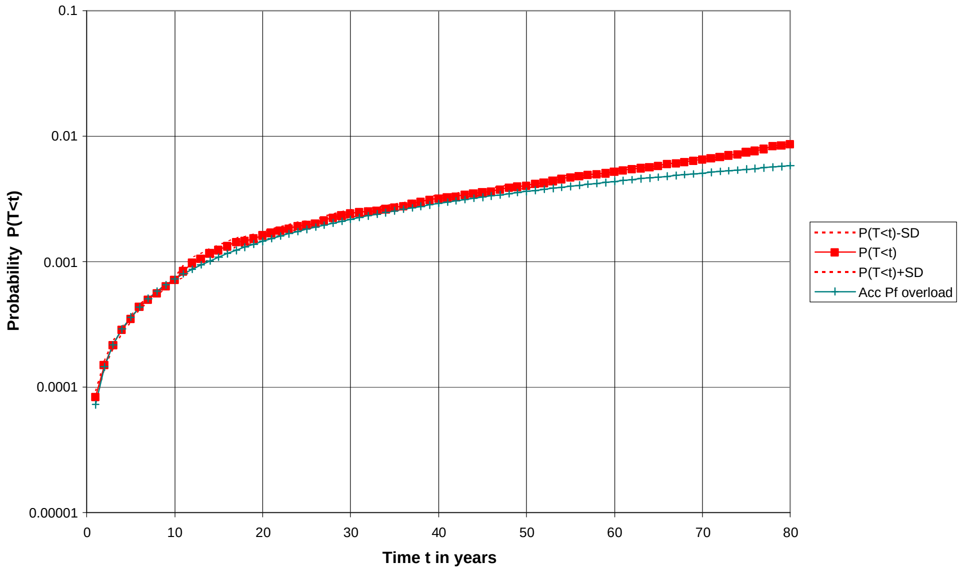

Figure 7-12: Cumulative jacket life distribution for case 2.0, including confidence bands of +/- one standard deviation and accumulated probability of failure for an intact structure (denoted Acc Pf overload). Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

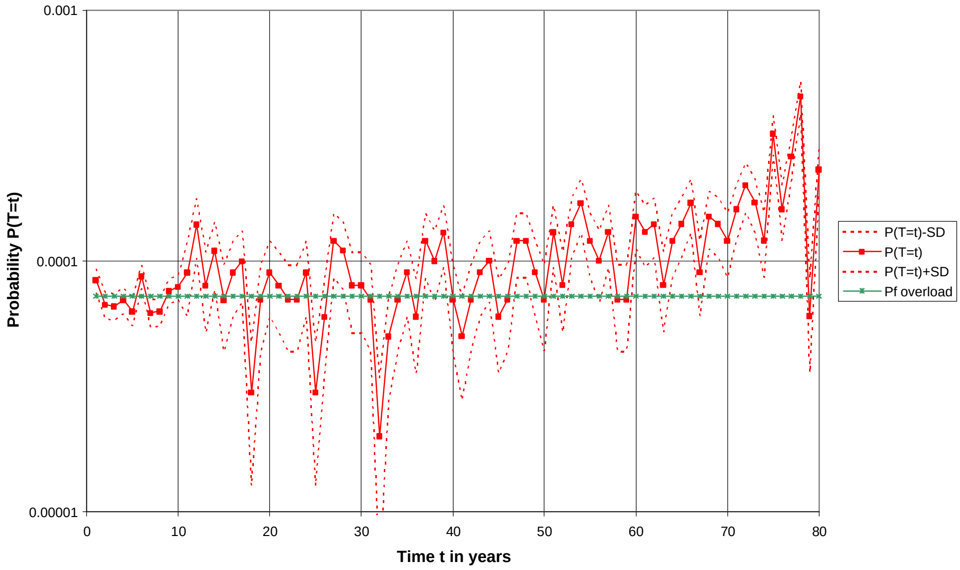

Figure 7-13: Annual probability of jacket failing in year t for case 2.0, including confidence bands of +/- one standard deviation and probability of failure for an intact structure (denoted Pf overload). Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

The rate of through thickness cracks should not be different in this case compared to case 1.1, as the flooded member detection method does not identify crack before they are through thickness. Hence, the comparison between the hazard functions for through thickness cracks for case 1.1 and 2.0 shown in Figure 7-14 is primarily a verification of this obvious similarity.

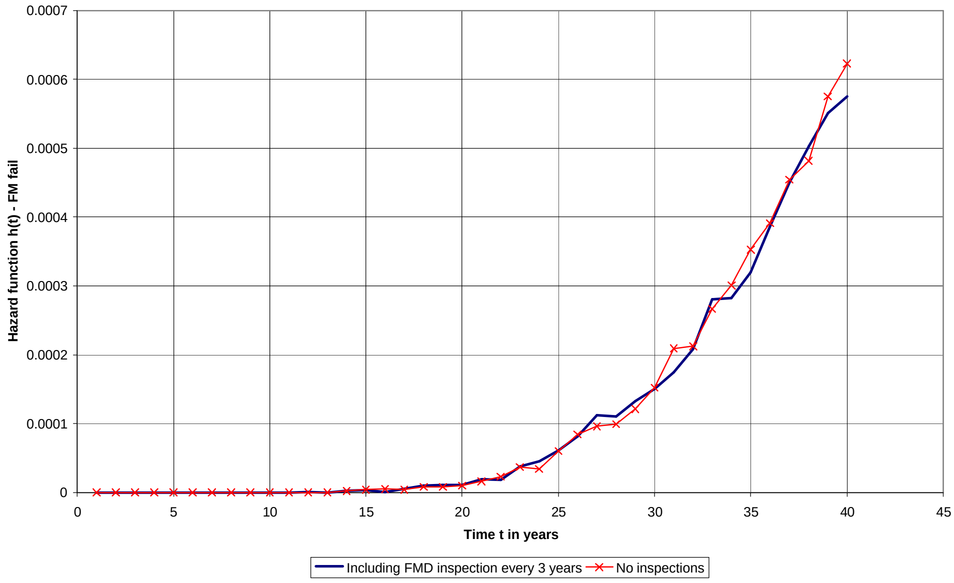

Figure 7-14: Hazard function for the occurrence of through thickness cracks when including FMD inspections compared to case 1.1 without inspections.

The wave height experienced by the structure at failure is shown in Figure 7-15. The wave height at system failure without prior component failure due to fatigue is above approximately 30 m. When one component has failed due to fatigue prior to overload failure, slightly smaller wave heights have caused the structure to fail. At four and five component failures prior to the collapse, wave heights from approximately 23-24 m and upwards have caused the structure to fail. This clearly demonstrates the weakening of the structure as the number of component failures increases.

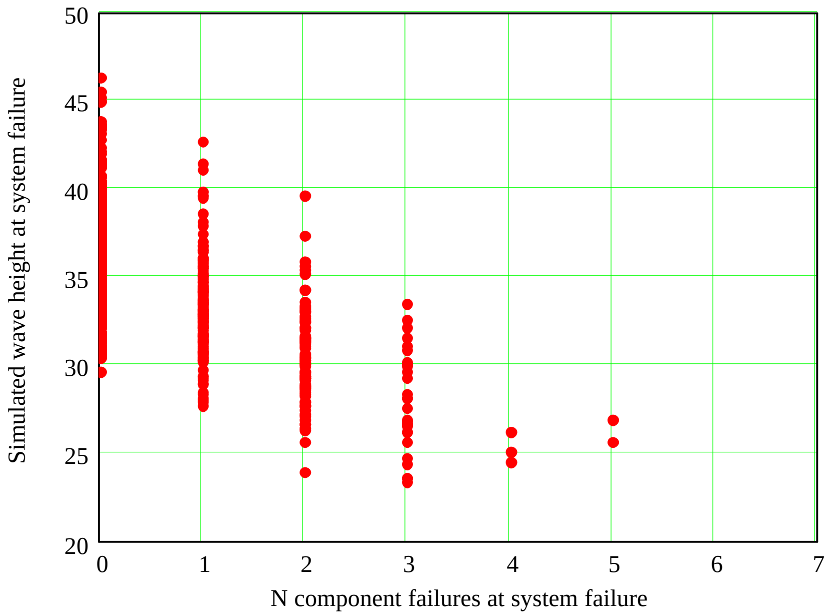

Figure 7-15: Wave height at system failure and number of component failure at system failure for case 2.0.

7.4.3 Case 3.0 Inspection with Eddy Current with 4 years interval

In this case, eddy current inspection is modelled every 4 year. If a crack is found, the crack is assumed to be repaired, and modelled with an initial crack size after repair according to the exponential distribution with a mean crack size of 0.11 mm. The repair method would probably depend on the crack size when the crack is found. For a reasonable small crack, grinding the crack is likely to be the preferred method. With larger crack sizes, welding of the crack may be a better solution. Data on initial crack sizes after underwater repair has not been obtained, but it seems reasonable to assume that crack sizes after grinding would be smaller than the initial crack sizes. Insufficient grinding is possible, and would mean that an initial crack is present. Repair using underwater welding is likely to be worse than fabrication welding. However, methods for improving an underwater weld, hammering and grinding, are likely to be used. Hence, the assumption of using the exponential distribution for initial cracks after repair with a mean value of initial crack of 0.11 mm may be conservative or non-conservative depending on the repair method. However, it seems reasonable to use this distribution in these simulations as the purpose is to illustrate the effect of inspection, and not to pinpoint the exact life of the structure.

In this case, none of the simulated cases fail with prior component failures. Hence the failure probability should be identical to the reference case for the simulated life of the jacket. The accumulated probability of life of the jacket (T) being less than time t for case 3.0 is illustrated in Figure 7-16. The annual probability of the life of the jacket (T) being t is illustrated in Figure 7-17. Also the confidence bands with +/- one standard deviation are indicated. The deviations from the reference case are due to limited simulations.

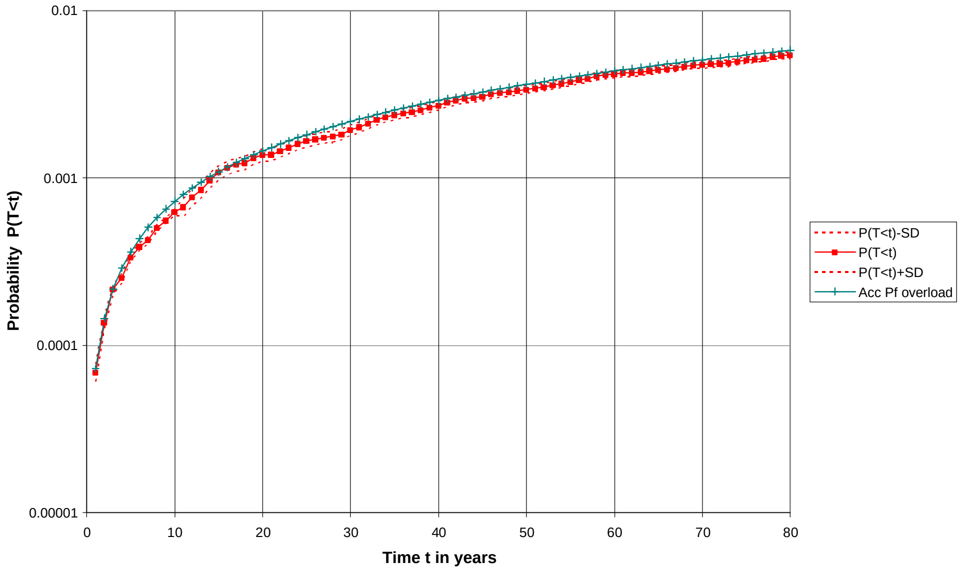

Figure 7-16: Cumulative jacket life distribution for case 3.0, including confidence bands of +/- one standard deviation. Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

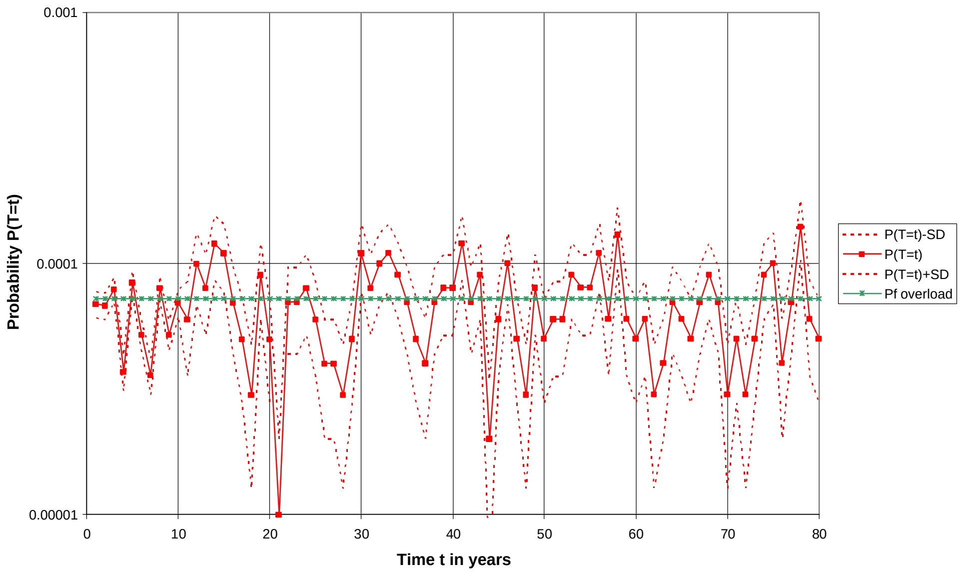

Figure 7-17: Annual jacket life distribution for case 3.0, including confidence bands of +/- one standard deviation. Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 100 000 simulations.

The failure probabilities do not exceed the reference level during the simulated life when applying eddy current inspection with a four year interval, i.e. the goal of the inspections is obtained. This indicates that it is possible to keep the failure probability at the same level as the failure probability of an intact structure with sufficient inspections.

The cost of inspections may be limiting the life of the existing structure, as the cost of inspecting at this rate may exceed the cost of building a new structure. No such evaluations are performed here, but this inspection program is not optimised in order to maintain the safety level with a minimum of safety. A slightly more optimised inspection program is applied in case 3.1 for the 20 first years.

As none of the simulated cases fails with prior component failure in this case, the wave height that causes failure to the system would be similar to the analyses of intact structures as shown in Section 6. Figure 7-18 indicates the wave height at system failure for the 509 cases experiencing failure out of 100 000 cases simulated (indicating an average annual failure probability of $\frac{509}{100000 \cdot 80} = 6.36 \cdot 10^{-5}$ which should be compared with the failure probability $7.25 \cdot 10^{-5}$ found in Section 6 for a structure with RSR=2.0).

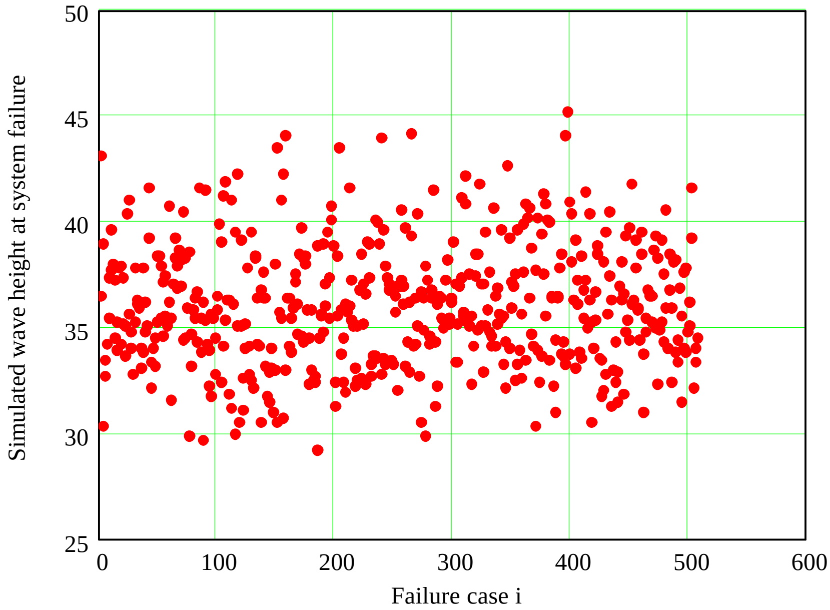

Figure 7-18: The simulated wave height leading to failure for 509 out of 50 000 simulation up to 40 years with Eddy Current inspections with 4 years intervals. Only the simulations that failed during the 80 years of simulation is included.

7.4.4 Case 3.1 Inspection with Eddy Current alternative inspection plan

The inspection plan used in this case is intended to be closer to a realistic inspection plan. It is based on information from inspection plans used in practice. The inspection plan includes an eddy current inspection of all components at year 5, 11 and 20 for the period inside the design life. If a crack is found, an additional inspection is performed after 2 years. If no cracks are found at this additional inspection, the inspection continues from start of the original inspection plan. In the extended life inspections are performed every 4 years if no cracks are found, and with 2 years interval if a crack is found. Hence, a component where no cracks are found would be inspected in years 5, 11, 20, 24, 28, 32, 36 and 40 for a 40 years inspection plan. If a crack is found after 5 years, but no cracks are found after the first crack, the component would be inspected in years 5, 7, 12, 18, 20, 24 etc.

The accumulated probability of failure of the structure before year t is shown for this case in Figure 7-19 and the annual probability of the life of the jacket (T) being t is illustrated in Figure 7-20. Confidence bands of +/- one standard deviation are also indicated. The failure probability does not exceed the accumulated failure probability of an intact structure for the first 40 years.

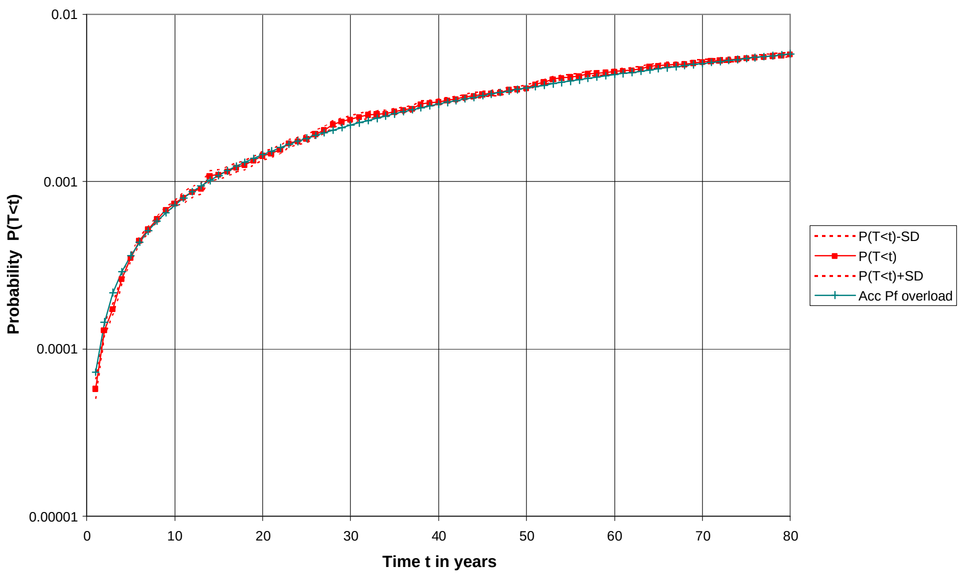

Figure 7-19: Failure probability P(T<t) for case 3.1, eddy current inspection according to a realistic inspection plan. Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 200 000 simulations.

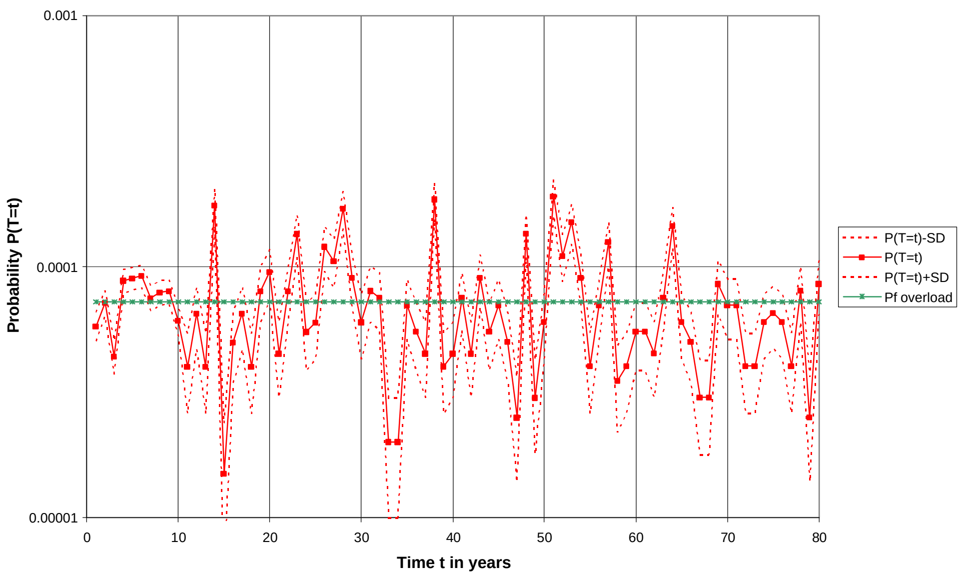

Figure 7-20: Probability of jacket failing in year t for case 3.1. Data up to 10 years are based on 1 000 000 simulations and data from 10 years to 80 years are based on 200 000 simulations.

7.4.5 Case 4 – WID including wave-in-deck loading and subsidence

Case 4-WID is included to evaluate the effect of a subsiding structure. Since freeboard is decreasing with time, this increases the probability of experiencing wave-in-deck impacts. The initial freeboard is set to 23.5 m and the structure is assumed to subside 0.1 m per year. The deck width is set to 100 m, and wave loading on deck is modelled as described in Section 6. Wave in deck loading is not included in the fatigue calculations, but is included in the overload check for the annual maximum wave load. The purpose of this case is to evaluate the importance of subsidence (and climate change) compared to component failure.

The basis for case 4 – WID is case 3.1, including eddy current inspections according to an inspection plan as given in Table 7-4.

The accumulated probability of failure of the structure before year t is shown for this case in Figure 7-21 and the annual probability of the life of the jacket (T) being t is illustrated in Figure 7-22. Confidence bands with +/- one standard deviation are shown. The simulated failure probability for this case is similar to the probability of overload due to annual maximum wave load exceeding the intact capacity of the structure for the first approximately 15 years. After 15 years the annual probability of failure is clearly exceeding the reference case.

It is interesting to note that the effect of subsidence and possible wave-in-deck loading, as simulated in this case, has a significant larger impact on the safety of the structure than the degradation of the structure due to fatigue. This clearly indicates that importance of the deck height and wave-in-deck loading should not be underestimated.

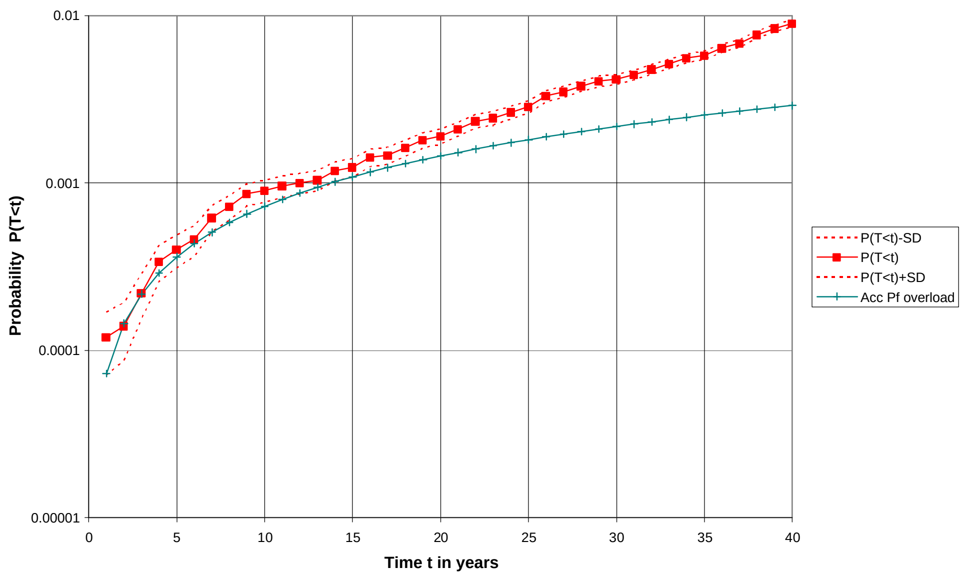

Figure 7-21: Failure probability P(T<t) for case 4-WID. Data are based on 50 000 simulations.

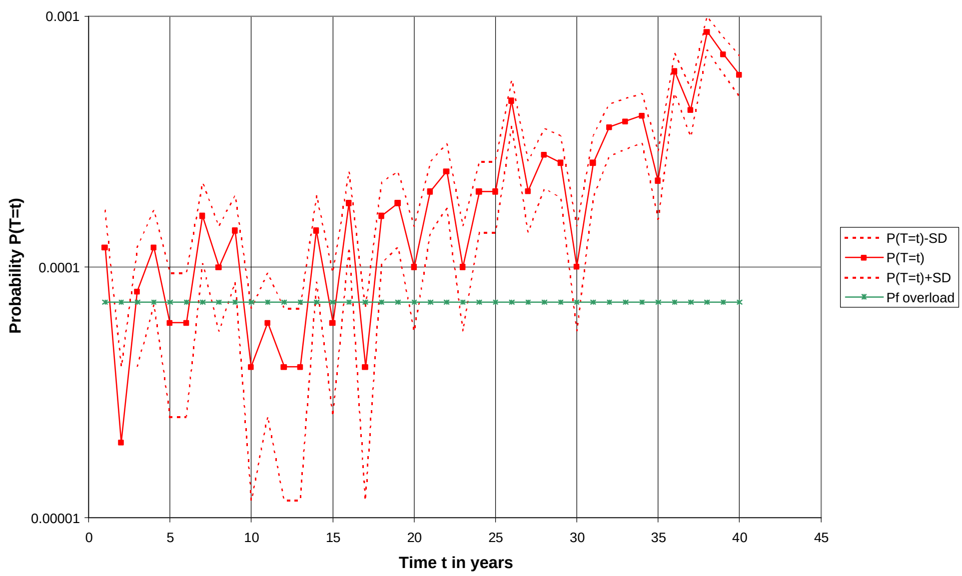

Figure 7-22: Probability of jacket failing in year t for case 4-WID. Data are based on 50 000 simulations.

7.4.6 Case 4 – LR including load redistribution at component failure

Case 4-LR is included to evaluate the effect of load redistribution when a component failure occurs. The effect of load redistribution is relevant in cases where component failures can be expected. As a result, it is not relevant to study the effect of load redistribution in a case where eddy current inspections are used, as very few component failures are expected using this inspection method. This leaves the case without inspection and the case with flooded member detection. The case with flooded member detection is most realistic and is selected as basis for this load redistribution study.

A marginal larger increase in the annual failure probability is seen for this case (see Figure 7-23 and Figure 7-24) compared to case 2.0. The failure probabilities are similar to that of accumulated failure probability of the intact structure up to approximately 50 years. After 50 years the probability of failure is clearly exceeding the reference case. However, based on this analysis the failure probability does not exceed the failure probability of the intact system for a significant number of years in addition to the 20 years of design life.

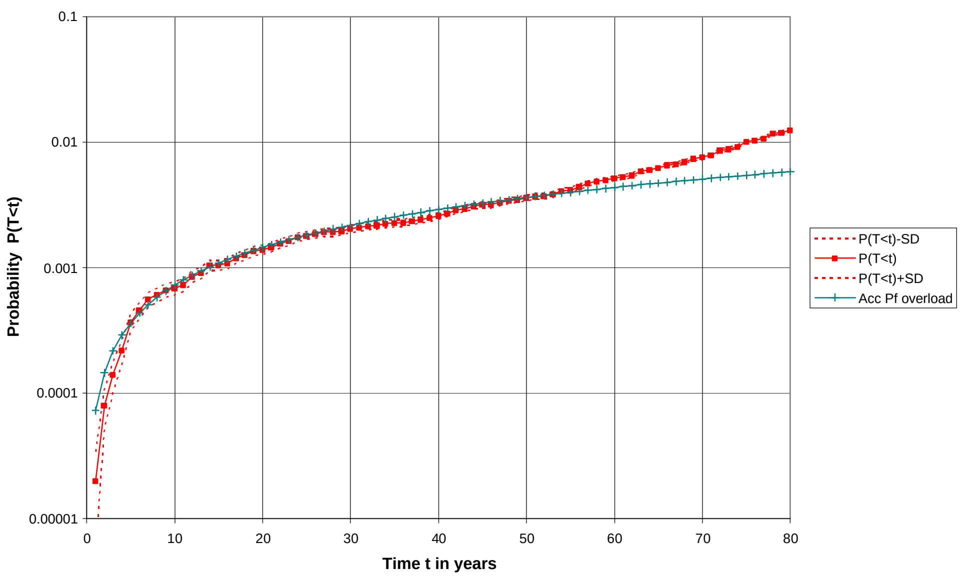

Figure 7-23: Failure probability P(T<t) for case 4-LR.

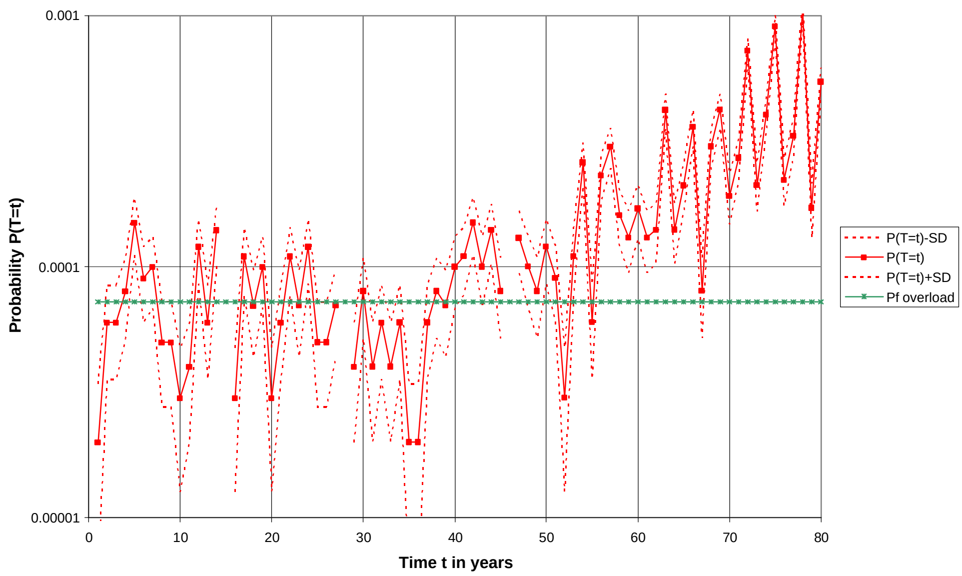

Figure 7-24: Probability of jacket failing in year t for case 4-LR.

7.4.7 Comparison of the cases

For comparison of the simulated cases, Figure 7-25 illustrates the accumulated failure probability of all the simulated cases.

From Figure 7-25 it can be seen that inspections do not have an influence on the safety of the structure in the first 30 years. The inspections in this period must be regarded as mainly a verification of the fabrication and the design analysis. If the design and fabrication has introduced an error of a greater extent than what's modelled in this analysis, or the design analysis includes errors in the stress calculation by a significant factor, cracks would occur at an earlier stage than expected. Such cases can be determined by the inspections. It seems reasonable that this should be the focus of the inspections. However, it should be noted that these simulations have been performed with an RIF=0.8, and inspection influence the safety of the structure at an earlier stage for lower values of RIF.

After 40 years, for a jacket that is designed for 20 years life, the failure probability of the non-inspected jacket simulations starts to exceed the accumulated failure probability of the intact structure. In the cases where inspections and repair has been implemented throughout the life of the jacket, such an increase is not seen. The inspections and repairs are in these cases also a part of providing the safety of the structure.

From the analysis performed in this thesis with a relatively damage tolerant structure, it seems reasonable to say that the life of a structure is not limited by degradation of the type modelled here. Inspections and repair may be used to ensure the safety to a level similar to the accumulated failure probability of overload by annual maximum wave alone. However, quite extensive inspection is modelled in these analyses, and the cost of inspections may limit the life of the structure. An optimisation of inspection intervals has not been performed as a part of these analyses, and fewer inspections may be sufficient.

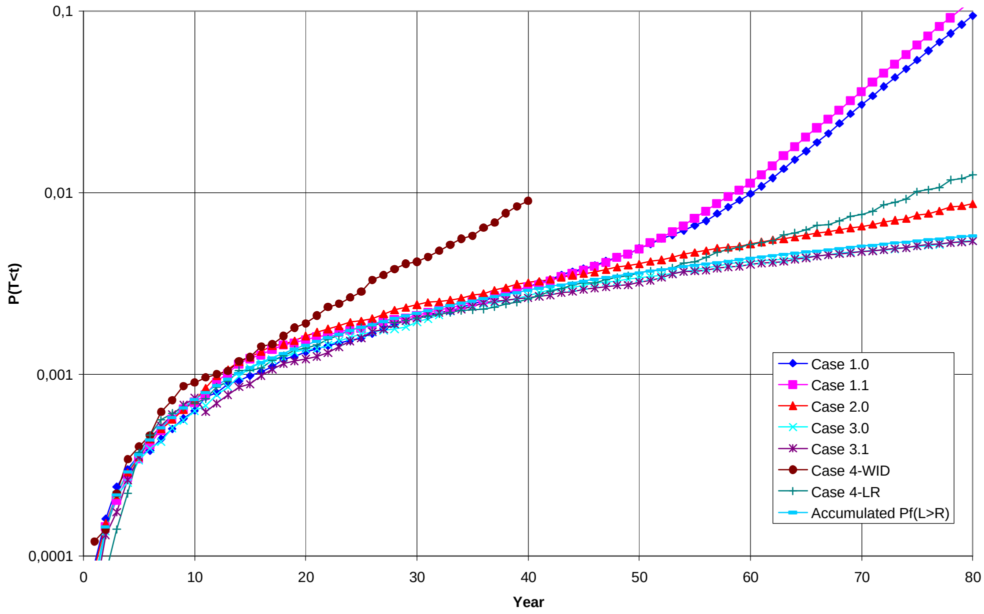

Figure 7-25: Accumulated failure probability P(T<t) for all evaluated cases. The case including a limited freeboard and subsidence is clearly the case with the largest growth in accumulated failure probability. The effect of increasing failure probability in case 4-WID is seen occurring at an early stage, for case 1.0 and 1.1 the increase is clearly seen from year 40, and for case 2.0 and 4-LR an increase above the accumulated failure probability of the intact structure is seen from a year 50.

The major influence on the structural safety, that is dominating the safety of the structure, is a possible subsidence or worsening of wave climate. If subsidence is present in a rate similar to the simulated case, the total failure probability will be dominated by the wave-in-deck hazard.

Load redistribution results in an increase in the failure probability at high ages of the structure. The effect is probably limited due to members being repaired after inspections and stress increase is limited to short periods.

Including a more realistic model of component failure, including the number of cycles from through thickness crack to member failure, would decrease the effect of degradation significantly. Hence, the overall conclusion that the safety of the structure seems to be maintained by inspection and repair should be valid also with smaller modifications to the probabilistic model.

8 Concluding remarks

8.1 Conclusions

In this work, issues related to a possible life extension of existing jackets are studied. These include a review of existing standards applicable for life extensions. The framework as described in the ISO 19902 (ISO 2004) has been found to be the most relevant framework for assessment of offshore jacket structures, which is the focus of this thesis. However, elements of this standard have been further investigated in this thesis. This includes an evaluation of the hazards and failure modes of ageing structures. Furthermore, probabilistic assessments of ageing structures and system strength evaluations have been studied with respect to their applicability to assessment of existing structures. Finally, fatigue degradation of ageing structures and its effect on the failure probability in combination with inspection and repair has been evaluated.

A structure specific evaluation of hazards and failure modes are recommended as a part of the assessment procedure. Some general elements in such an evaluation have been established in Chapter 4 of this thesis. However, a general evaluation would not be sufficient for all individual structures, and the general elements identified in this thesis should only be viewed as supporting information to the specific assessment.

With respect to the fatigue limit state, the evaluations performed in Chapter 7 indicate that a life extension of a damage tolerant structure is feasible, without compromising the safety of the structure. This requires that sufficient inspection, maintenance and repair are performed and structural system indicators are at an acceptable level. It is shown that inspections and repair may be used to ensure a safety level similar to the accumulated failure probability of an intact system. Quite extensive inspection is modelled in these analyses, and the cost of inspections may limit the life of the structure. An optimisation of inspection intervals has not been performed as a part of these analyses, and an optimization of the inspection program may be needed. The focus of this inspection program for damage tolerant structures should be to ensure that no more than one component fails before this is identified by inspections and subsequently repaired.

The simulations in Chapter 7 further indicates that subsidence and worsening of the environmental climate leading to possible wave in deck impact loading is a critical hazard for the jacket structures. Hence, this seems to be the limiting factor if applicable to the structure under assessment. If the structure is subsiding at the rate simulated in this thesis, inspections at any interval would not make it possible to ensure a safety in line with present regulation in the simulated period.

Load redistribution after a component failure, resulting in an increase in the failure probability in nearby components, has also been studied in the simulations in Chapter 7. The effect is shown to have a certain impact for structures inspected by flooded member detection.

A probabilistic analysis of the structure is included as a possible assessment method in the reviewed standards. Structural reliability analysis (SRA) is seen in this thesis as the most promising method to directly evaluate the safety of the structure in an assessment for life extension. Structural reliability analysis takes into account the actual uncertainty about the structure, and provides a tool for evaluating changes in the uncertainty about a possible future failure of the structure. However, the existing codes and regulations are unclear on how to make decisions based on these analyses. The existing codes and standards seem to deal with probabilities in a semi objective approach, and acceptable safety used for decisions is recommended to be established on this basis. A sufficient stringent method of establishing these acceptable safety levels is not included in the evaluated standards.

In this thesis, the predictive Bayesian approach is found more useful in approaching structural reliability analysis. The guidelines on probabilistic assessment of existing structures will need to be updated to take into account the predictive Bayesian approach before full use of probabilistic methods can be used for assessment. The purpose of the structural reliability analysis (or risk analysis) should be to support the decision making, not making the decision in itself. The focus should be on establishing good solutions with respect to safety, cost and other aspects that are seen as important for the decision. If the analyst is meeting a criterion for safety, this is not sufficient to make a decision to use this solution. If improvements can be made to the solution with relatively small additional costs, the alternative with such improvements have to be included in the evaluation. The proposed solution may be compared with good practise with respect to safety, based on similar assignments of failure probability for similar structures. This should not be used as an acceptance criterion, but as an additional input to the decision making. However, it is reasonable to focus on alternatives that are at least within good practice and providing decision support information for these alternatives. The final decision should be made according to the described multi attribute analysis with managerial decision based on the alternatives proposed.

An alternative to probabilistic assessment is the use of system strength analysis with deterministic criteria that can be used for evaluating the safety of the structure. The safety of the jacket structure can be described reasonably well with the parameters reserve strength ratio (RSR), damaged strength ratio (DSR) and reserve freeboard ratio (RFR). Acceptance criteria for the RSR, DSR and RFR parameters are in this thesis developed to be in consistence with normal practice.

Some concern has been raised in this thesis about the RSR parameter with respect to its sensitivity for increased wave height and water depth. This sensitivity is studied in this thesis by increasing the wave height and evaluating the collapse base shear capacity. The analysis indicates that the collapse base shear is reduced for waves at higher elevations of the jacket. For most of the analyses performed in this work, this is a rather small effect and should not be of concern when using the RSR as an indicator of structural safety. However, the collapse base shear capacity may experience a rather sudden drop around when the wave hits the deck. However, as this occurs as an effect after a wave hits the deck, the safety of the structure should be reasonably well covered if the reserve freeboard ratio (RFR) parameter is within the described acceptance criterion.

If an assessment based on system strength is performed, care should be given to the possible effects of large deformations in the structure. Large deformations may lead to non-structural incidents like hydrocarbon leakage and limitations to the use of escape equipment. Such possible escalating effects should be evaluated in addition to the structural analysis. Further, an evaluation of local load effects as wave slam and vortex shedding on components should be evaluated, as this is not included in most system strength analyses.

8.2 Recommendations for further work

The work on hazards, failure modes and barriers needs to be extended to include other major hazards as earthquake and boat impact and to include other parts of the load carrying structure such as the piles. The general procedure for barrier identification as proposed in this thesis should be applicable also for these situations.

Further work is needed on applying the predictive Bayesian approach in structural reliability analysis. This would include how to assign distribution functions for the uncertainty about the e.g. structural strength and fatigue performance as a result of the structural integrity management system including inspections and maintenance. The stochastic variations observed in the use of the Miner-Palmgren sum should be included in the assigned uncertainty about the number of cycles a component may experience before failure.

The effect of increasing wave heights on the reserve strength ratio is studied in this thesis. Similarly, the effect of increasing wave height on the damaged strength ration should be investigated. Also the effect on reserve strength ratio and damaged strength ratio of decreasing wall thickness on the components in the structure due to corrosion needs to be evaluated.

The effect of corrosion on degradation and on system strength needs to be included in the simulations. Also, improvements with respect to the uncertainty about the number of cycles a component can withstand before failure needs further investigation in order to include the stochastic variations observed in the use of the Miner-Palmgren sum. However, the most important aspect that needs to be included is the additional fatigue life of a component after a through-thickness crack.

The simulations included in this thesis do not attempt to establish an optimal inspection and repair regime for an ageing structure. Further work on the effect of reducing the inspection intervals may be important from an economical point of view.

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Part II

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Risk management and its ethical basis

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Abstract
In decision making under uncertainty there are two main questions that need to be evaluated: i) What are the future consequences and associated uncertainties of an action, and ii) what is a good (or right) decision or action. Philosophically these issues are categorised as epistemic questions (i.e. questions of knowledge) and ethical questions (i.e. questions of moral and norms). This paper discusses the second issue, and evaluates different bases for a good decision, using different ethical theories as a starting point. This includes the utilitarian ethics of Bentley and Mills, and deontological ethics of Kant, Rawls and Habermas. The paper addresses various principles in risk management and risk related decision making, including cost benefit analysis, minimum safety criterion, the ALARP principle and the precautionary principle.

1 Introduction

In today's society, risk based or risk informed decisions are made in design and operation of most technical systems. The idea of using such an approach is to ensure that the "right" decisions are made, by addressing the overall performance of the system using the proper concept, namely risk. However, a risk approach does not provide answers on what is a good or right decision – risk just describes the possible consequences and associated uncertainties. Clearly, there are dimensions of the decision making that goes beyond risk, for example ethical and political issues. This paper addresses the ethical dimension. To what extent can we justify the various risk approaches, such as the use of risk acceptance criteria and ALARP, by reference to ethical theories? Is there a link between a specific decision method and ethical principles?

The paper starts out with a brief description and discussion of ethical theories that are found relevant for risk-informed decision-making. Ethical theories ranging from John Bentley's and John Stuart Mill's Utilitarianism, to Kant's, Rawl's and Habermas' ethics are described. From this review we have a basis for discussing the ethical fundament for some common principles and methods used in risk informed decision-making, such as cost benefit analysis, the precautionary principle, the ALARP principle and the use of minimum safety standards.

The discussion of risk and ethics have been discussed earlier by many researchers, see e.g Körte (2003), Shrader-Frechette (1991), Gibbard (1986), Leonard (1986), Harasanyi (1988). Our work extends the work by Körte (2003). Shrader-Frechette (1991) is comparing a rather extreme version of cost benefit analysis, links this to Utilitarianism, and compares this with Rawls' ethics. Harasanyi (1988) discusses the cost benefit analyses and how they are linked to Utilitarianism.

2 Ethical theories

This section gives a short review of deontological ethics and consequence ethics (consequentialism). Then the utilitarianism – a version of concequentialism – and Kant's ethical theories, Hans Jonas additional imperative, Rawls' theory of justice, and Habermas discourse ethics – all versions of deontological ethics – are briefly described. The review is based on Johansen and Vetlesen (2000), Skorupski (1998), Guyer (1992), Freeman (2003), Rassmussen (1990), Encyclopaedia Britannica (2005) and Wikipedia Encyclopaedia (2005).

2.1 Deontology versus consequentialism

Two of the main ethical directions are deontology and consequentialist theories. Deontology is the view that morality either forbids or permits actions, whereas consequentialist theories express that the rightness of an action depends on the consequences of the act. The most famous deontological theory is that introduced by the German philosopher Immanuel Kant. Also Rawls' and Habermas' ethics mainly fall into deontological ethics. Within the consequentialist ethical theories, utilitarianism is the most important ethical theory in this context.

In risk informed decisions, a deontologist would claim that there is right and wrong decisions up front, whatever consequences. A deontologist may claim that it is never right to expose a person to a certain risk, even if the consequence is an increased welfare of the society as a whole.

2.2 Utilitarianism

Utilitarianism (Skorupsle 1998) is both a theory of the good and a theory of the right. Utilitarianism would regard an action as good if the action yields value in form of pleasure to humans, and right if the action yields the greatest net value for the society. Jeremy Bentham originally proposed utilitarianism in 18th century England, and the theory was further developed by John Stuart Mill.

Risk informed decisions following a utilitarian perspective would require that the outcome of the various choices are measured in some form of utility, and that these outcomes can be compared so that the choice with the best performance can be selected. This can be done in practice by using the maximum expected utility or the use of cost benefit analysis (Benjamin and Cornell 1970 and Lindley 1985). The most common measure of utility is monetary values.

Making the utilitarianism operational is difficult, and using the theory of expected utilities and/or cost benefit analysis the limitations and constraints of the tools need to be reflected in the way the analyses are used, see e.g. Aven (2003), chapter 5. These tools do not provide hard decisions, but decision support.

2.3 Kant’s deontology

Kant's theory (Guyer 1992) includes the idea of a categorical imperative. A categorical imperative, generally speaking, is an unconditional obligation, or an obligation that we have regardless of our will or desires. Our moral duties can be derived from the categorical imperative. The categorical imperative can be formulated in three ways:

- The first formulation (the Formula of Universal Law) says: "act only in accordance with that maxim through which you can at the same time will that it become a universal law.", where a maxim is a subjective or internal rule for what action to take given a set of circumstances.
- The second formulation (the Formula of Humanity) says: "Act that you use humanity, whether in your own person or in the person of any other, always at the same time as an end, never merely as a means."
- The third formulation (the Formula of Autonomy) is a synthesis of the previous two. It says that we should so act that we may think of ourselves as legislating universal laws through our maxims. We may think of ourselves as such autonomous legislators only insofar as we follow our own laws.

As mentioned earlier, the deontological ethics, as Kant’s ethics, are often seen as being in conflict with the consequence ethics as utilitarianism. However, a utilitarianist would agree with the first formulation of Kant’s ethics in that the utilitarian ethics should be universal. The conflict between Kantian ethics and Utilitarianism is rather in the Formulae of Humanity, as one could regard exposing other persons for higher risk in order to achieve personal or societal benefits or measuring the value of a person’s life in monetary value in a risk informed decision-making, to be treating the person merely as a mean. If the risk exposure is voluntary, the second formulation of the categorical imperative may not apply. If a person chooses to expose himself to risk, no other person is using this person as a mean.

A frequently used formulation for this ethics can be stated as: *Those actions are right that equally respect each human as a moral agent.* A moral agent is an individual capable of both formulating and pursuing purposes of his of her own and of being responsible for the actions taken to fulfil those purposes.

2.4 Hans Jonas Imperative

Hans Jonas (Martens and Nyblin, 1992) insists that human survival depends on our efforts to care for our planet and its future. Following Kant, he formulated a new imperative of responsibility, "Act so that the effects of your action are compatible with the permanence of genuine human life"; or as stated in Martens and Nyblin (1992): “In your present choices, include the future wholeness of Man among the objects of your will”.

If the effects of our actions today on the future generations of Man should be assessed, the uncertainties with respect to the consequences of a decision would be very large.

2.5 Rawls' principle of justice

Rawls' (Freeman 2003) primary objective in A Theory of Justice is to develop an alternative systematic account of justice that is superior to the utilitarianism. The main problem with utilitarianism, as Rawls sees it, is that it allows the rights of some people to be sacrificed for the greater benefit of others, as long as the total happiness is increased.

Rawls defines what he calls *The Original Position.* In the original position, each person would not know his or her financial situation, race, creed, religion, or state of health. In this position we were to establish the just social contract that we would agree upon. Rawls deduces that a just society would be based on two principles.

The First Principle of Justice
First of all, each person would have the most extensive system of rights and freedoms which can be accorded equally to everyone. These include freedoms of speech, conscience, peaceful assembly, and so forth, as well as democratic rights. The first principle is absolute, and may never be violated, even for the sake of the second principle. However, various basic rights may be traded off against each other for the sake of obtaining the largest possible system of rights.

The Second Principle of Justice
Secondly, economic and social inequalities are only justified if they benefit all of society, especially its most disadvantaged members. Furthermore, all economically and socially privileged positions must be open to all people equally. Unlike the utilitarians, Rawls does not allow some people to suffer for the greater benefit of others.

Shrader-Frechette (1991) links Rawls' ethical principles with the maximin principle, saying that policies having the worst possible consequences should be avoided. She argues that Rawls is taking the maximin principle as equivalent to the difference principle, saying one society is better than another if the worst-off members of the former do better than the worst-off in the latter.

Shrader-Frechette (1991), p 117, also gives a clear argument for the difference between Rawls' ethics and the utilitarian ethics: "If all members of a society have an equal, prima facie right to life, and therefore bodily security, as the most basic of human rights, then allowing one group of persons to be put at greater risk, without compensation and for no good reason, amount to violating their rights to life and to bodily security."

2.6 Habermas discourse ethics.

Habermas (Rassmussen 1990) starting point for his discourse ethics is the Kantian imperatives. However, the problem of how to claim universality for a rule of conduct is of major concern in the Kantian ethics. According to Habermas, to be able to establish whether a rule is universally acceptable involves that, ideally, all affected must have participated in establishing the rule or norm and consent to it. A requirement for this approach is that all involved are committed to achieving consensus. Habermas propose to solve this by stating that *only the norms that can find (or could find) agreement between all involved parts in a discourse can claim to be universal*.

In risk informed decisions this would mean that there is a need for a common ground between the involved parties in the activity. The involved parts in a petroleum activity, or stakeholders, would be the society, the workers, the government and the owners / investors. In order for all these stakeholders to be involved in the decision, thorough and balanced background documentation is needed from the risk analyst. The risk analyst's responsibility in this context will be to provide the necessary documentation in order to make it possible for the stakeholders to make such a decision.

3 Common decision principles in risk management

Risk informed or risk based decision methods may be seen as utility based or rights based. Among the utility based decision methods are cost benefit analysis and multi attribute analysis. Among the rights based decision methods are zero risk, bounded or constrained risk using risk acceptance criteria. These principles are briefly reviewed in the following.

3.1 Cost benefit analysis (CBA)

As an example, consider the search for an optimal design of a technical facility. Following economic theory we may formulate the design as a decision problem within the framework of Bayesian decision analysis as presented in Benjamin and Cornell (1970). In short, the decision problem may be formulated as an optimisation problem, where the expected life cycle benefit is maximized. Due to the fact that income and costs occur at different times, the expected benefit is capitalized (by means of its net present value) to the point in time when the decision is made.

The benefit can be calculated taking into account the income including the possible loss of income in case of failure / breakdown, the cost of developing the facility including the additional cost of increasing the safety of the facility, and the cost of a failure of the facility. The formula used is:

$$E[B]=E[I] – E[C_O] – E[C_D] – E[C_F]$$

where $E[B]$ is the total expected benefit, $E[I]$ is the expected income, $E[C_O]$ is the expected operational cost, $E[C_D]$ is the expected development cost and $E[C_F]$ is the expected failure cost.

According to traditionally cost benefit analysis, the optimal decision is the alternative that has the best expected benefit. The analysis is based on the transformation of loss of lives, injuries and environmental damage to monetary values. This is done by introducing for example an expected cost per expected saved lives.

Monetary numbers for the costs of avoiding a statistical fatality have been established and used in various ways. Ranges from 1 MNOK to 200 MNOK have been used in CBA (Aven and Vinnem 2005). A possible method to take into account possible fatalities was developed by Nathwani et al. (1997) as the Life Quality Index (LQI). From this index, Skjong and Ronold (1998) derived the amount of money, which should be invested to avert a fatality

ICAF (Implied Cost of Avoiding a Fatality), see e.g. Rackwitz (2000) and Rackwitz (2001). According to Rackwitz (2001), the societal loss due to losses of lives, can and should be taken into account in the design decision problem by including its cost equivalent, i.e. the expected number of fatalities NF multiplied with ICAF.

3.2 Risk acceptance criteria

Decisions based on risk acceptance criteria imply that an acceptable risk level is defined in some form, and the exposure level for personnel and environment is compared to this level. The development of these acceptance criteria can differ from predefined acceptance criteria from regulatory bodies, acceptance criteria developed from cost benefit analysis, or acceptance criteria defined by evaluating the safety level in the industry practice. The Norwegian regulation of the offshore industry is an example of using such risk acceptance criteria (PSA 2002), where major safety systems should tolerate a load level with an annual probability of 10^{-4}. The use of risk acceptance criteria is a mechanical decision tool, the risk is either less than the acceptance criterion or it is not.

This method of decision making will in principle be deontological, in so far as they have the form of clear rules and obligations. Both Kant and Rawls could be used to argue for such a decision method. Based on Kantian ethics one could argue that a person should not be exposed to a higher risk in order to benefit other persons, a company or the society. A zero risk exposure could be seen as the fulfilment of the deontological ethics. In practice a low risk level is usually used.

3.3 The cautionary and precautionary principle

The cautionary principle states that in face of uncertainties, caution should be the guiding principle. In safety applications, there exist uncertainties about the possible occurrence of hazardous situations and accidental events. Following the cautionary principle, one should seek solutions that are robust in the sense that such events are avoided and the consequences reduced in the case that such events should occur.

The precautionary principle states that in the case of lack of scientific certainty about the consequences, the activity should be avoided or measures should be implemented. Hence, it is e.g. lack of scientific certainty about the causal links between an action or incident and the possible consequence of this action or incident that is the focus of the precautionary principle. In such cases the precautionary principle should be followed, and focus should be on whether the possible consequences could be avoided, if there is an alternative better and safer solution.

3.4 The ALARP principle

The ALARP principle (***A***s ***L***ow ***A***s ***R***easonably ***P***racticable) requires an identification and consideration of a range of potential measures for further risk reduction (HSE 2003). According to the principle, risk shall be reduced to a level that is as low as reasonably practicable. The principle should provide a motivation for seeking continuous improvement. Cost benefit analysis is often used to determine what is practicable.

A version of the ALARP principle is the three region ALARP principle, were low risks (probability of occurrence lower than say 10^{-6}) are called acceptable, and high risks (probability of occurrence higher than say 10^{-4}) are intolerable and must be reduced. In between these two, we find the ALARP region where risks should be reduced according to the ALARP principle.

3.5 Multi Attribute Analysis

The basis for decision making as presented here, is taken from Aven (2003) and Aven and Vinnem (2005). The method may be described as a multi attribute analysis with managerial review and judgment.

A multi attribute analysis is a decision support tool analysing the consequences of the various measures separately for the various attributes (technical feasibility, economy, safety, etc.). Thus there is no attempt made to transform all the different attributes in a comparable unit. In general the decision-maker have to weight non-market goods such as safety and environmental issues with an expected net present value, E[NPV], calculated for the other attributes (market goods) in the project. An alternative way to weight the different attributes is to use different ratios, based on a cost-effectiveness analysis.

A simple model of the decision process is shown in Figure 1 and covers the following items:

1 Stakeholders. The stakeholders are here defined as people, groups, owners, authorities that have interest related to the decisions to be taken. Internal stakeholders could be the owner of the installation, other shareholders, the safety manager, labour organisations, the maintenance manager, whereas external stakeholders could be the safety authorities (the Norwegian Petroleum Directorate, the State Pollution Control Agency), environmental groups (Greenpeace etc), research institutions.

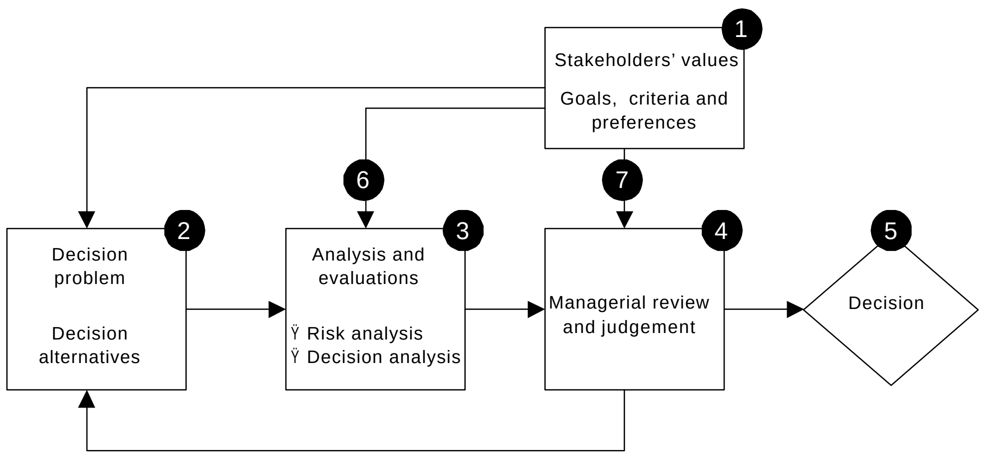

Figure 3-1 Model of the decision making process (Aven 2003)

2 Decision problem and decision alternatives. The starting point for the decision process is a choice between various concepts, design configurations, sequence of safety critical activities, risk reducing measures etc.

3 Analysis and evaluation. To evaluate the performance of the alternatives, different types of analyses are conducted, including risk analyses and cost-benefit (cost-effectiveness) analyses. These analyses may, given a set of assumptions and limitations, result in recommendations on which alternative to choose.

4 Managerial review and judgement. The decision support analyses need to be evaluated in the light of the premises, assumptions and limitations of these analyses. The analyses are based on a background information that must be reviewed together with the results of the analyses. Considerations should be given to factors such as

- The decision alternatives being analysed
- The performance measures analysed (to what extent do the performance measures used describe the performance of the alternatives?)
- The fact that the results of the analyses represent judgments and not only facts
- The difficulty of assessing values for burdens and benefits
- The fact that the analysis results apply to models, i.e., simplifications of the real world, and not the real world itself. The modelling implies that a number of limitations are introduced, such as replacing continuous quantities with discrete quantities, extensive simplification of time sequences, etc.

In Figure 1 we have indicated that the stakeholders may also influence the final decision process 7 in addition to their stated criteria, preferences and value tradeoffs 6.

4 Discussion and conclusions

Cost benefit analysis and utilitarian ethics

The calculation of optimal safety based on cost benefit analysis (CBA) can be justified with reference to the utilitarian moral philosophy (Schrader-Frechette, 1991). However, the use of cost benefit analysis does not necessarily require a utilitarian basis. If the CBA is used mechanically to produce decisions, it could be seen as a way of making the utilitarian theory operational. On the other hand, a CBA used only to inform the decision maker on the economic aspects of the decision, but acknowledging that the decision is made from a broader information basis, does not need to be utilitarian at all.

The CBA can be taken to be a rather extreme tool used for societal decisions. In this case, all societal investments in safety should be evaluated by a CBA, and only accepted if the CBA shows a positive benefit by implementing the safety investment. All attributes should be included in the CBA, including the societal costs associated with loss of lives and environmental damages. The CBA will then be a political tool for decisions, and could to large extent replace political processes. Such an implementation of CBA could be seen as a way of making the utilitarian theory operational as a political process. Schrader-Frechette (1991) argues against such an implementation of CBA, as it is more designed for risk taking than risk control. She further argues that in such a decision scheme, highly improbable accidents would not be a significant concern in decision making. The authors find this extreme version of CBA totally unacceptable, as it means that complicated political processes are replaced by more or less arbitrary mathematical one-dimensional exercises, which cannot be justified.

A more common use of CBA, is the micro-economic implementation of the CBA, where a single company and the benefits for this company is the focus of the CBA. In pure economic evaluations, where human and environmental safety is not an issue, such a use of the CBA is not problematic. In this case the decision is rather simple, as the purpose is to optimise the economy for the company. However, companies also need to include safety of personnel and environment issues in their studies, in e.g. investments projects. Then the decision making also need to incorporate ethical issues, for example related to exposure of personnel to risk for the sake of additional income. The CBA may then provide useful information for the company as a decision support. However, a mechanical use as described above cannot be justified.

The values that should be optimised according to Utilitarian theory, does not necessarily have to be cost related. The value in traditional Utilitarian theory is linked with happiness, and happiness may be measured on a totally different scale than monetary values, for example perceived safety.

One argument supporting the Cost Benefit Analysis as a basis for decision making is that in the long run it pays off (Rackwitz 2003). The control of risks (in the sense of mastering them effectively and efficiently) must obey the rules of modern decision theory and uncertainty. However, according to Høybråten (2004), studies indicate that countries that choose a balanced approach to growth (balance between health and life of individuals, protection of environment, and economic values for industry and society) are the very same countries that perform best economically. This does not, in our opinion, mean that that you can prove that a balanced approach is the right approach in risk based decision making, but rather indicate that the CBA's have strong limitations and weaknesses.

The deontological ethics and risk acceptance criteria

The deontological theories presented here (Kant, Jonas, Rawls and Habermas) all clearly focuses on the individuals right with respect to risk exposure. Hans Jonas is also clearly including the environmental concerns into the ethical basis for decisions. A zero accident philosophy would in many aspects be the right philosophy based on deontological thinking. However, in practice it is impossible to obtain a zero risk as anything more than a vision when facing uncertain future events and consequences. This would lead to more pragmatic implementations of the deontological ethics in practical decision making.

Elliott and Taig (2003) develop the practical pragmatic implementation of the deontological ethics starting by stating that "No person or organisation has a moral right to expose another to risk." This is the starting deontological principle, as mentioned earlier. They go on and state some reasonable exceptions to this statement. First it is argued that "It is morally acceptable to expose another person to risk if the purpose of the action is to reduce the net risk for this person". This is exemplified by a patient undergoing surgery for a medical condition. It is recognised that the risk creator in such situations has a moral obligation to explain the risk to the person who will face it and secure informed consent before starting out. The next extension of this exception would be "It is morally acceptable to take moderate risk in order to reduce risk for others". This can be exemplified by the work of a rescue crew or the fire department. The final extension that is formulated by Elliot and Taig (2003) is that "It is morally acceptable to ask someone to take a modest health or safety risk in order to accrue other, non-health or safety benefits for others". This is exemplified by people working in hazardous jobs. This will however, require that those at risk should where possible give their informed consent and that those creating or managing risks should be competent in minimising them within reason. The general principles for responsibility of organisations that should apply in such situations are as follows (Elliot and Taig 2003):

- "Ensure openness and transparency about risks and how they are controlled.
- Continually search for ways to reduce risk, unless they compromise other desirable outcomes.
- Use resources competently and effectively to control risk.
- Involve people in decisions that affect them as far as practicable.
- Work across the spectrum of areas for which the organisation is responsible, those for which it shares responsibility with others, and those it can influence but not control.

The use of e.g. risk acceptance criteria and the cautionary principle (in face of uncertainty, cautionary should be the ruling principle) would be such pragmatic ways of deontological thinking.

As discussed in Vinnem and Aven (2005) and Aven et al (2005) the use of risk acceptance criteria easily gives focus on reaching these criteria, rather than focussing on obtaining alternatives that are good with respect to safety, cost and other strategic criteria used in the decision making process. The use of risk acceptance criteria are also based on critical assumptions that risk can be accurate determined and compared with the criteria. However, as shown in Aven (2003) such assumptions are not in general valid.

Cautionary and precautionary principle, Rawls' principles and Hans Jonas imperative.
In all risk based or informed decisions, we are facing uncertainty. Hence the cautionary principle would make itself applicable in all risk informed decisions. Following this way of thinking, we should act with caution and not necessarily chose the economic "optimal" solution.

According to Shrader-Frechette (1991 p 127) "If technological rulemaking created a climate of maximin, a climate in which decision makers aimed at avoiding worst cases, both they and society would likely be more aware of potential accident consequences, …, and more aware of human errors in risk assessment." We see here a link between the cautionary principle and the ethics of Rawls. Caution may be seen as the implementation of the principle of Rawls' maximin principle, to choose the risk distribution where the least well off are least disadvantaged. Rawls' gives the following argument to support the maximin strategy (Shrader-Frechette 1991 p 116): 1) It would lead to giving the interest of the least advantaged the highest priority. 2) It would avoid using a utility function, designed for risk taking, in the areas of morals, where it does not belong. 3) It would avoid the utilitarian use of interpersonal comparisons of utility in defining justice. 4) It would avoid making supererogatory actions a matter of duty, as do utilitarian theories. 5) It would avoid the utilitarian dependence on uncertain prediction about the consequences of alternative policies.

The precautionary principle is being used in the case of lack of scientific certainty about the consequences, for example related to future environmental consequences (for following generations) of an action. If we were to follow Hans Jonas' imperative, we should "including the future wholeness of Man” as an object in our decision. With the limited scientific certainty we can claim for the future wholeness of man, it is difficult to see any other means of meeting Hans Jonas’ imperative then to follow the precautionary principle.

A practical way of fulfilling both these principles would be to require robustness for the unknown and uncontrollable, e.g. using barriers, and to focus on some worst case scenarios. Barriers can be used in risk reduction and accident prevention in various ways, but will in general mean that one identify the possible hazards and the failure modes following these hazards, and implement barriers to prevent these failure modes to occur. The number of barriers and the robustness, reliability and availability of these barriers may be implemented according to the probability of the hazard and the consequence of the failure.

Both these principles are in general based on deontological ethics, and are as mentioned above a pragmatic implementation of the deontological ethics.

ALARP principle and deontology

The ALARP approach can be interpreted to have a deontological element as its purpose is to achieve as low as possible risk exposure. However, the focus on “reasonably practicable” means a reference to the consequences, and would thus also include an element of consequentialism. An ALARP evaluation may be performed with focus on these consequence elements within a cost benefit approach, resulting in a process similar to a cost benefit analysis. However, the ALARP principle is in general seen as a principle focusing more on reducing risk than a cost benefit analysis. Hence, the ALARP principle seems primarily linked to deontological theories.

The three region version of ALARP includes two acceptance criteria into the evaluation, where low risks are called acceptable, and high risks are intolerable and must be reduced. The ALARP principle is, in this implementation, only applicable in between these two risk levels. The focus for a risk assessment following the three region ALARP principle will often be on reaching these targets. The primary goal would be to avoid the intolerable region, and if possible enter the acceptable region. The focus of the general ALARP principle for obtaining a good solution with respect to safety and cost can easily be lost by the focus on the acceptance criteria (tolerability limits).

Multi attribute analysis with managerial decision and ethical theories

The multi attribute analysis with managerial decision may be seen as a method for balancing the deontology and consequentialist theories. The focus of the risk analysis in this approach is to produce good alternatives with respect to safety, economy and other attributes. It includes risk reduction without acceptance criteria, and includes an element of deontology. It also includes cost benefit analysis. The decision process is, however, not mechanical on the basis of the risk analyses and these decision analyses. The decision process can be seen as a process of evaluating and weighing the different stakeholder interests. The stakeholders may to some extent have different means, and hence different agendas for the decision-making. However, by including these views and presenting them clearly, the decision making process would be open and finally end in a discourse that can be audited by the stakeholders. A managerial decision based on such a process would to large extent be in line with Habermas’ discourse ethics.

A critic to this way of thinking may ask whether anything is acceptable within the multi attribute analysis with managerial decision, as long as the decision makers find discourse.

The answer is obviously “No”. The decision makers should take into account the relevant goals, criteria and preferences by the stakeholders, but the acceptance should not be based on direct use of risk acceptance criteria. The interests of the stakeholders would in most cases be that the safety is at least in accordance with normal practice, which means that the decision process should be able to account for some indications of a reasonable risk. However, if an improved alternative, with marginal additional cost, is feasible, this should be evaluated even if the first alternative meets the safety level of normal practice. Hence, the lower value of acceptable risk should not be viewed as an absolute level, but used as an indication of a minimum representing the normal practice.

The main reason for advocating this type of risk informed decision is that the moral dilemmas are not hidden in the decision method. The dilemmas need to be addressed by the decision makers and the assessments available for evaluation by auditors. By using a pure cost-benefit analysis, the moral dilemma is hidden inside the method, and the best solution according to the computed expected NPV is chosen. Similarly with fixed risk acceptance criteria, the solution that just meets the criteria is chosen. The moral dilemma is not highlighted and not addressed. It may not even be visible for an auditor. With the type of multi attribute decision making as proposed here, the moral dilemma should be clearly defined. The dilemma need to be discussed by the decision makers, and result in a documented argument supporting the final decision. The documentation should be available for a possible audit by stakeholders, making a discussion about the decision possible, in a better way than if the moral dilemma is hidden in the decision method. The decision may not be altered, but the dilemma has been focused and discussed, and a defence for the decision is available.

Conclusions

All the evaluated decision methods seem to have a reasonable basis in sound ethical theories. There is no clear guidance on which ethical theory that is preferable to others. Hence it is difficult to use the ethical theory as a guiding principle to choose between the decision methods. However, it can be concluded that there seems to be a rational logical link between the decision methods available and the major ethical theories available. The multi attribute analysis, as presented here and in Aven (2003), is preferred as the moral dilemmas of the decision making is brought to attention for the decision makers and not hidden in the decision making method.

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Economic Optimal Reserve Strength for a Jacket Structure

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Abstract

This paper evaluates the optimal Reserve Strength Ratio (RSR) of a jacket structure based on a model taking into account the cost of field development, income from production and possible costs of failure. The failure probability related to this RSR is calculated, and compared with acceptance criteria given in existing regulations relevant for offshore facilities located on the Norwegian continental shelf.

The cost of the jacket structure includes an estimate of construction cost and the cost of increasing the strength of the jacket to higher reserve strength. The cost of failure takes into account consequences of material losses and loss of lives.

The sensitivity of the economic optimal reserve strength to the different types of uncertainty is studied. This includes a study with only aleatory uncertainties, a study with aleatory and epistemic uncertainties included. Evaluations of the optimal reserve strength ratio (RSR) of jacket structures are typically performed without looking at the possibility of member failures and the possibility of wave in deck loading. This paper will address these two possible additional failure modes to make the risk model more accurate, and to evaluate the effect on the conclusion.

The present study shows that, with the legislative requirements imposed on offshore operators on the Norwegian continental shelf, the optimal design of fixed jacket structures is governed by the minimum safety level implicit in the regulation and standards. The optimal RSR will only in a few cases indicate higher values of RSR. The economical (life cycle benefit) optimal failure probabilities can result in a less conservative design, but also an increased fatality risk for the individual worker.

Keywords

Structural Reliability, Offshore structures, Reserve Strength Ratio, Cost Benefit Analysis, Optimal Design.

[1] Petroleum Safety Authority Norway is the former Safety Division of the Norwegian Petroleum Directorate

Introduction

Economical margins for field developments have been reduced as a result of smaller fields, improved safety requirements from regulators. This in combination with higher revenue requirements by the owners, have led to extended theoretical work towards optimality and acceptance criteria for the design and operation of offshore facilities have been performed, ref Stahl (1998), Skjong and Ronold (1998), Vinnem (1996), Pinna et al. (2001), Kübler and Faber (2002). Decisions on structural strength parameters (load and resistance factors, reserve strength ratio, etc.) may be optimised on the basis of past practice and cost benefit considerations together with the legislative requirements to the safety of personnel.

To the knowledge of the authors, the full use of the possibilities with optimised design of offshore installations has not been implemented in any practical design. However, structural reliability analysis and evaluations of optimal safety levels have been used in code development (Sørensen et al 1994). Optimisation based on cost benefit analysis has, in Norwegian field developments, only been allowed for unmanned installations with insignificant environmental consequences in case of failure.

In optimising the safety of a risk exposed system, all failure modes and uncertainties associated to these failure modes should in principle be addressed. As this in practical design is an impossible task, the following fundamental principle should be applied: The risk model is sufficiently accurate when any improvement to the risk model to make it more accurate should not lead to a change in the conclusions made (Aven, 2003). Evaluations of the optimal reserve strength ratio (RSR) of jacket structures are typically performed without looking at the possibility of member failures and the possibility of wave in deck loading. This paper will address these two possible additional failure modes to make the risk model more accurate, and to evaluate the effect on the conclusion.

Also, the effect of varying the epistemic uncertainty (statistical and model uncertainties related to e.g. shape parameter, wave load, capacity) is evaluated. This will exemplify the sensitivity to the subjective expert judgement for the probability distributions used to describe the uncertainties of factors.

The following cases have been evaluated in this paper: 1) The optimal RSR without wave in deck loads and without possible member failure. 2) The effect of epistemic uncertainty is evaluated by removing the assumed epistemic uncertainties in the calculations (spread is removed, and the distribution is fitted by introducing a bias). 3) Introduction of new failure modes in order to evaluate the change in optimal RSR. The following changes are applied: a) Possible member failure reducing the structural capacity is included. The failure may be a result of fatigue, overload, corrosion, or gross errors[2]. b) A “new failure mode” is introduced by including the possibility of wave in deck loading. c) A combination of a) and b) accounting for both wave in deck and the possibility of a member failure.

The economic model follows the model described in Kübler and Faber (2002), where a further development of the methods for implementing consequences of fatalities in decision analysis, ref. Skjong and Ronold (1998) and Rackwitz (2000), for offshore structures is described.

[2] Gross error may be built in to the structure as a result of design error, a fabrication error or operational error.

Nomenclature

The following list shows the cost items for a field development and their abbreviation as used in this paper. This is mainly based on Almlund (1991).

Total cost of field development: C_{FD} or CAPEX

- Construction cost for installation (including structure, topside equipment, piles, transport and installation, management and engineering, etc) C_C.
- Well drilling and down-hole activities: C_W
- Hydrocarbon transportation system: C_{HCT}
- Sub-sea systems: C_{SUB}

Total construction cost for installation: C_C

- Topside (process, drilling area, living quarter, utilities and outfitting, wellhead area, structure): C_{TS}
- Substructure: C_{SS}
- Cost of a substructure with RSR = RSR_0 : C_{SS_Fix}
- Piles: C_{PL}
- Transport and installation: C_{MI}
- Management, Project team, engineering, insurance: C_{TE}

Cost of failure: C_F

- Loss of lives (fatalities): C_{FF} = NF ICAF
  - NF: Number of Fatalities
  - ICAF: Implied cost of avoiding a fatality
- Material losses: C_{FM}

Reconstruction cost: C_{RC}

- Assumed equal to construction cost for installation C_C. The transport system and wells are assumed intact.

Cost of operation: OPEX

- Cost of operating the facility including maintenance, well services, process, man-hours, catering, etc.

Income / Revenue: I

- Income is modelled according to a standard production profile taking into account operational costs.
- Net present value of income (I_{NPV}) is taking into account the discounting of future income.

Generalising the different cost elements as a dimensionless ratio:

Construction cost versus Field development cost:	ρ_{CC}	$= C_C / C_{FD}$
Income versus Field development cost:	ρ_I	$= I_{NPV} / C_{FD}$
Cost of failure (material costs) versus Field development cost:	ρ_{FM}	$= C_{FM} / C_{FD}$
Implicit cost of avoiding a fatality versus Field development cost:	ρ_{ICAF}	$= ICAF / C_{FD}$
Cost of fixed part of substructure (RSR = RSR_0) versus total installation:	ρ_{SS}	$= C_{SS\_Fix} / C_C$
Percentage cost increase of strengthening (increasing the RSR) of the structure:	ρ_{St}	ref Eq. 16

Economical optimal decision model

A decision theory framework for establishing economical optimal RSR for an offshore jacket structure is established in Kübler and Faber (2002).

Decision theoretical formulation

Following Sørensen et al. (1994), optimal design in structural engineering may be seen and formulated as a decision problem within the framework of Bayesian decision analysis, see also Raiffa and Schlaifer (1961). In short, the decision problem may be formulated as an optimization problem, where the expected life cycle benefit of the structure is maximized. Due to the fact that income and costs occur at different times, the expected benefit is capitalized (by means of its net present value) to the point in time when the decision is made.
In the present study the construction costs C_C, the income obtained through the production profile I, the failure costs C_F and the reconstruction costs C_{RC} are included in the life cycle benefit analysis. Each of these consequences depend on a set of design parameters, which for the present case are represented by the reserve strength ratio (RSR).

Simplified Decision Analysis

When searching for the optimal design of a structure, it should be taken into account that the structure may fail in the future and that it may be reconstructed if feasible. This again depends on the future potential income / revenue.
The decision/event tree shown in Figure 2 illustrates the pursued approach. The first node in Figure 2 represents the design and construction of the structure. At this time the reliability of the structure is decided. After the structure has been realized, in principle two events may follow. The structure may survive (event E_1) or it may fail. If the structure fails, there are again two possibilities. Either it is economically feasible to reconstruct the structure or it is not (event E_2). If economically feasible, the structure is reconstructed and thereafter again two events may follow. Either the structure fails (event E_4) or the structure survives (event E_3). Hence, in the present study, only the expected costs and incomes up until the time of the second failure are taken into account.

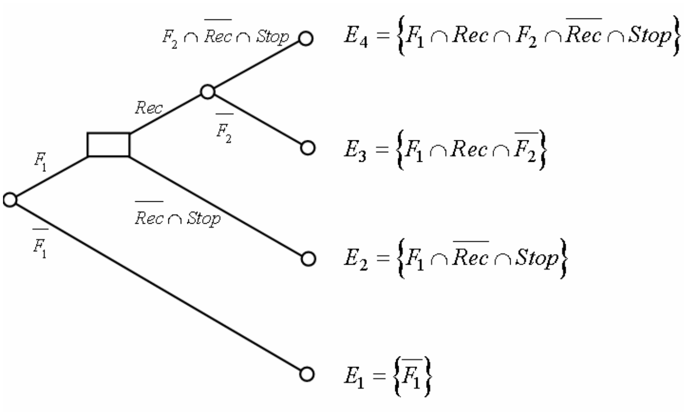

Figure 2: Simplified decision analysis

The expected life cycle benefit $E[B]$ may be written as

$$E[B] = E[I] - E[C_{FD}] - E[C_{RC}] - E[C_F], \tag{1}$$

where the expected value operations are performed in regard to the uncertainties associated with the loading and the load capacity of the structure. The incomes and costs are discounted by a discounting function $\delta(t)$.

$$\delta(t) = e^{-\gamma \cdot t} \tag{2}$$

Here, $\gamma = \ln(1+r)$, r denotes the annual interest rate and t is the time at which the consequence (income or cost) occurs.
In this paper the yearly income, $i_{net}(t)$, and the yearly cost of operating the facility, $OPEX(t)$, is integrated into the income function ($i(t) = i_{net}(t) - OPEX(t)$), as the available data for income for a field is often given with reduction for the operating cost. The expected income is discounted by the discounting function, and taking into account the "income reliability":

$$I = \int_0^T i(t) \cdot \delta(t) \cdot R_{i,2}(t) dt = \rho_I \cdot C_{FD} \cdot \int_0^T \tilde{i}(t) \cdot \delta(t) \cdot R_{i,2}(t) dt \tag{3}$$

where T is the design lifetime, $i(t)$ the income function, $\delta(t)$ the discounting function, $R_{i,2}(t)$ the "income reliability function" considering two possible failures and $\tilde{i}(t)$ is a normalized income function $\tilde{i}(t) = \dfrac{i(t)}{I_{NPV}} = \dfrac{i(t)}{\rho_I \cdot C_{FD}}$. The income reliability function expresses the probability that at time t the income is obtained.
Hence, $E[B]$ can be written as:

$$E[B] = \rho_I \cdot C_{FD} \cdot \int_0^T \tilde{i}(t) \cdot \delta(t) \cdot R_{i,2}(t) dt - C_{FD} - \int_0^T C_C \cdot \delta(t) \cdot g_1(t) \cdot dt$$
$$- \sum_{n=1}^{2} \int_0^T C_F \cdot \delta(t) \cdot g_n(t) \cdot dt \tag{4}$$

C_{FD} are the field development cost, C_C are the construction costs (also representing the reconstruction costs), C_F the failure costs and $g_n(t)$ is the probability density function of the time to the n-th failure.
Inserting the relative description of the cost and income terms, including the effect of increased cost with strengthening, Equation 4 becomes:

$$
\begin{aligned}
E[B] = C_{FD} \cdot (\rho_I \cdot \int_0^T \tilde{i}(t) \cdot \delta(t) \cdot R_{i,2}(t)dt - 1 - \rho_{CC} \cdot \rho_{SS} \cdot \rho_{St} \cdot (RSR - RSR_0) \ldots \\
- \rho_{CC} \cdot \int_0^T \delta(t) \cdot g_1(t) \cdot dt - \sum_{n=1}^{2} \int_0^T (\rho_F + NF \cdot \rho_{ICAF}) \cdot \delta(t) \cdot g_n(t) \cdot dt)
\end{aligned}
\tag{5}
$$

where RSR_0 is the RSR for the reference installations.
The assumption that failures occur as realisations of a stationary Poisson process allows for an analytical evaluation of Equation 5. Based on this assumption, only one failure event may occur in a sufficient small time interval. Therefore, the probability of the union is simply the addition of the probabilities of the individual events. The “income reliability function,” which considers two possible failures including the reconstruction decision, may be derived as shown in Kübler and Faber (2002):

$$
R_{i,2}(t) = \begin{cases} e^{-\lambda t}\left(1+\lambda t\right) & ; t \le t_0 \\ e^{-\lambda t}\left(1+\lambda t_0\right) & ; t > t_0 \end{cases} . \tag{6}
$$

where t_0 is the latest point in time for an economically feasible reconstruction.
If the structure is given up after the first failure, the “income reliability function” $R_{i,1}(t)$ is identical to the classical reliability function of the structure.
By means of the annual probability of failure, the annual failure rate becomes:

$$
\lambda = \ln\left(\frac{1}{1-P_f}\right) \tag{7}
$$

where P_f is the annual probability of failure as a relation of the RSR, as shown in the following section.

Modelling the probability of failure of a structure

To determine the relation between the RSR and the annual failure rate, a probabilistic model according to Ersdal et al. (2003) has been applied. Typical failure modes for a jacket structure are:

- Overload due to excessive wave (including wave in deck), wind and current loading.
- Overload due to excessive wave (including wave in deck), wind and current loading after damage of a member or joint. A failure of an individual member may be due to fatigue, corrosion, gross error (e.g. insufficient design, fabrication error, damage during transport or installation), accidental loading (e.g. dropped object, boat collision).

Typical jacket structure failure will also include other environmental loads (e.g. earthquake loading) and accidental loads (e.g. boat collisions, fires, explosions). However, in this study only wave loading is evaluated, with or without prior damage to a member or joint.
The probability of failure for the structural system can then be written as:

$$P_{f\_system} = P(sys|\overline{F}) \cdot P(\overline{F}) + P(sys|F) \cdot P(F) \tag{8}$$

where P(sys) represents the probability of a system failure, F represents failure of an arbitrary member or node, and $\overline{F}$ represents no failures of any member or nodes (the complimentary set).

Detailing the last part of Equation 8 becomes:

$$P_{f\_system} = P(sys|\overline{F}) \cdot P(\overline{F}) + \sum_i \sum_j P(sys|F_{i,j}) \cdot P(F_{i,j}) + \sum_i \sum_j \sum_k P(sys|F_{i,j} \cap F_{k,j}) \cdot P(F_{i,j} \cap F_{k,j}) + \ldots \tag{9}$$

where $F_{i,j}$ denotes failure of member or node "i" due to failure cause "j" and $F_{k,j}$ denotes failure of member or node "k" due to failure cause "j".
Here it is assumed that the probability of a system failure, given a failure of a member or node, is the same for all members and nodes. In this case the probability of a system failure can be simplified to:

$$P_{f\_system} = P(sys|\overline{F}) \cdot P(\overline{F}) + P(sys|F_1) \cdot P(F_1) + P(sys|F_1 \cap F_2) \cdot P(F_1 \cap F_2) + \ldots \tag{10}$$

where F_1 represent the first occurring failure in an arbitrary member or node and F_2 represent the second occurring failure in an arbitrary member or node.
The probability of failure due to fatigue and corrosion can be modelled with an established model for degradation in probabilistic analysis. The probability of a gross error can to some extent be modelled according to historic occurrences of gross errors found from inspections. In this study the reason for the failure is not further evaluated, and for simplicity the probability of a failure in any member or joint is increased from 0.001 to 0.1 to evaluate the effect.
Also the effect of a member or joint failure on the system strength (modelled by the RSR) is modelled simply by reducing the system strength by a stochastic factor uniformly distributed between 0.3 and 0.7. The reduction in system strength when a brace in a X-jacket is damaged, will often be in the order of 0.8. For K and other jacket types the reduction will be larger. If the damaged member is a leg, the capacity will be reduced significantly, in many cases down to 0.0. The range between 0.3 to 0.7 is selected to represent these possible failures.
Only the first two terms in Equation 10 is included in the calculations (two or more simultaneous failures is not evaluated).

Wave Height

The maximum wave height in one year, H, is assumed to follow a Gumbel distribution, i.e. the distribution function reads

$$F_H(h) = \exp\left\{ -\exp\left[-\frac{h - \alpha_H}{\beta_h} \right] \right\} \tag{11}$$

where α_H and β_H are parameters of the distribution.

Wave load

The wave loading W is decribed by the following equation

$$W = \alpha_1 \cdot C_1 \cdot H^{C_3} \tag{12}$$

where α_1 is a factor introduced to account for the model uncertainty in the load model, H is the wave height, C_1 and C_3 are load coefficients that must be curve-fitted to calculated load data for the specific jacket.

In the cases when wave in deck loads are included, the following load model is used

$$W = \alpha_1 \cdot C_1 \cdot H^{C_3} + \alpha_2 \cdot C_4 \cdot (\rho \cdot H - AG) \tag{13}$$

where α_2 is a factor counting for the model uncertainty, ρ is the wave crest to wave height factor, C_4 is a load coefficient that must be curve-fitted to the calculated load data for the specific jacket, and AG is the air gap, here defined as the distance between the LAT (lowest astronomical tide) and bottom of steel on topside.

Resistance

The resistance is modelled as an ultimate capacity of the structure, described on a system basis. The ultimate capacity is assumed to be equal to the design loading ($C_1 \cdot H_{100}^{C3}$) multiplied by the Reserve Strength Ratio (RSR). The design loading is the loading with the annual probability of exceedance of 10^{-2}, and the RSR is the ratio between ultimate collapse load of the structure and the design loading. ξ• is a factor counting for model uncertainty in the resistance model.

$$R = \xi \cdot RSR \cdot C_1 \cdot H_{100}^{C_3} \tag{14}$$

Limit state equation

A failure function for ultimate collapse of the structure can be modelled by the following equation:

$$g = R - W \tag{15}$$

Probability of failure is given by $P_f = P(g \le 0)$.

Stochastic model

The parameters of the stochastic model are given in Table 1.

Table 1: Parameters used in the simulations – base case (choices commented below)

Parameter	Description	Values	Comments
RSR	Reserve strength ratio	Fixed at 2.0	The RSR is assumed to be 2.0 for the structure evaluated in this paper, but the failure probability is also evaluated for RSR values from 1.5 to 2.5 to evaluate the sensitivity to the RSR.
ξ	Resistance model uncertainty	Normal distributed mean value = 1.0 Standard deviation = 0.1	ξ is in the base case normal distributed with a mean value of 1.0 and a COV of 0.1 as recommended by Efthymiou et al (1996). The "Guideline for Offshore Structural Reliability Analysis" issued by DNV (1996) recommends a COV for the base shear capacity of a jacket structure to be 0.05 - 0.10. Based on this recommendation, a COV=0.05 is also evaluated.
H_{100}	100 year wave	Fixed at 28.6 m	Wave height with an annual probability of exceedance of 10^{-2}
α_1	Load model uncertainty	Normal distributed: mean value = 1.0 Standard deviation = 0.15	α_1 is in the base case normal distributed with a mean value of 1.0 and a COV of 0.15 as recommended by Haver (1995). The base case with COV=0.15 on load model uncertainty, and the Weibull distribution on wave height, gives a total COV on the wave loading on 0.26. The "Guideline for Offshore Structural Reliability Analysis" issued by DNV (1996) recommends a COV on the wave loading of 0.4, which indicates a much higher COV of α_1 than recommended by Haver (1995). Efthymiou et al (1996) recommends a COV not greater than 8%. In order to evaluate a certain range of the different recommendations in the COV of wave loading, the COV of α_1 is varied in the range of 0.05 to 0.25 (giving a variation in COV on wave loading of 0.21 to 0.33). The sensitivity to the variation in COV in the load model is further evaluated.
α_2	Load model uncertainty – wave in deck loading	Normal distributed: mean value = 1.0 Standard deviation = 0.15	Assumed equal to α_1 in this paper. A reasonable assumption is that the uncertainty related to wave in deck loading is larger than the jacket loading, but due to a lack of data the same distribution is used.
H	Annual maximum wave height	According to Eq. 2. with: α_H = 21m β_H = 1.63m	The parameters for wave height distribution are obtained by fitting a Gumbel distribution to the data for the Kvitebjørn field in Northern North Sea (Statoil 2000).
C1	Load coefficient	Fixed at 1.0	The value used for C_1 is not important for the present study, as C_1 appears in both terms in the equation (resistance and load). A more realistic value for C_1 is in the order of 0.01 to 0.05 depending on the size of the jacket and the water depth.
C3	Load coefficient	Fixed at 2.2	For many Northern North-Sea jackets C_3 is found to be approximately 2.2, which is used in this paper.
C4	Wave in deck load coefficient	Fixed at 720	The ratio between C4 and C1 is used in the calculations. This ratio is roughly estimated by the momentum of the incoming wave for a 100m wide deck structure (solid) and related to the C1 factor.
ρ	Wave crest factor	Fixed at 0.62	This value is slightly lower than the maximum for a Stoke 5^{th} order wave (0.66). However, the value may need further investigation.

Description of the cases

The base case is as described in Table 1 where wave in deck loading is not included and the probability of member failure is equal to zero. This case should be consistent with the case described in Kübler and Faber (2002). However, the limit state function is defined differently, leading to minor differences. The remaining cases evaluated in this study are presented in Table 2.

Table 2: Studied cases

Case	Epistemic uncertainty	Probability of member failure	Air Gap
1	Included as described in Table 1	0.0	Wave in deck not included
2	$\alpha_1 = 1.13$, $\xi = 1.0$	0.0	Wave in deck not included
3	Included as described in Table 1	0.001	Wave in deck not included
4	Included as described in Table 1	0.01	Wave in deck not included
5	Included as described in Table 1	0.0	22m
6	Included as described in Table 1	0.0	20m
7	Included as described in Table 1	0.0	18m
8	Included as described in Table 1	0.001	22m
9	Included as described in Table 1	0.001	20m

Epistemic uncertainties are removed in case 2. It is assumed that the wave load distribution is well known. In order to fit the load distribution to the same distribution without the uncertainty parameter, a bias of 1.13 is included resulting in a close fit between the assumed measured wave load distribution and the wave load distribution obtained by the stochastic description. Hence, the α_1 is fixed to 1.13 and ξ is fixed to 1.0.

Figure 3 indicates that when excluding the epistemic uncertainty, the probability of failure is slightly reduced, as one would expect. The difference is increasing with increasing RSR values, but in the normal area for RSR values (up to 2.5) the difference seems to be small. Further, an increasing system failure probability is observed as the probability of a possible member failure is increased. In estimation of the probability of system failure, this indicates that including member failure will be of importance, even for small probabilities of a possible member failure. Also, Figure 3 clearly indicates that the probability of failure increases as the air gap is reduced. A rather extreme underestimation of the probability of failure could be the result of not taking into account wave in deck forces for large values of RSR, based on the present model.

Economic data

Cost of field development, installation and substructure

Data from 4 recent jacket projects in Norway is collected from Esso (1999), Hydro (1999), Statoil (1997) and Statoil (1999). The collected data consists of the cost of field development, the construction cost for the installations and the substructures. The data are presented in Table 3. All economic data are extracted from the “Plan for Development and Operation”. These data may not be the final numbers for these projects, but represents the best guess prior to construction and installation.

Table 3: Economic data extracted for recent jacket projects (MNOK)

	Topside	Jacket	Drilling	Subsea	HC Transport	Field Development Cost: C_{FD} or CAPEX
Kvitebjørn	4168	995	997	0	1424	7584
Grane	7745	1575	3045	0	2685	15050
Ringhorne	3039	586	3280	437.5	812.5	8155
Huldra	1086	488	1097	9	1367	4047
Mean value						8709

Air gap is discussed in this paper, and increasing air gap may be constrained by risers and lifting vessels. An air gap increase beyond a certain limit, not defined in this paper, may result in a sudden jump in costs. These types of additional costs are not considered in the paper.

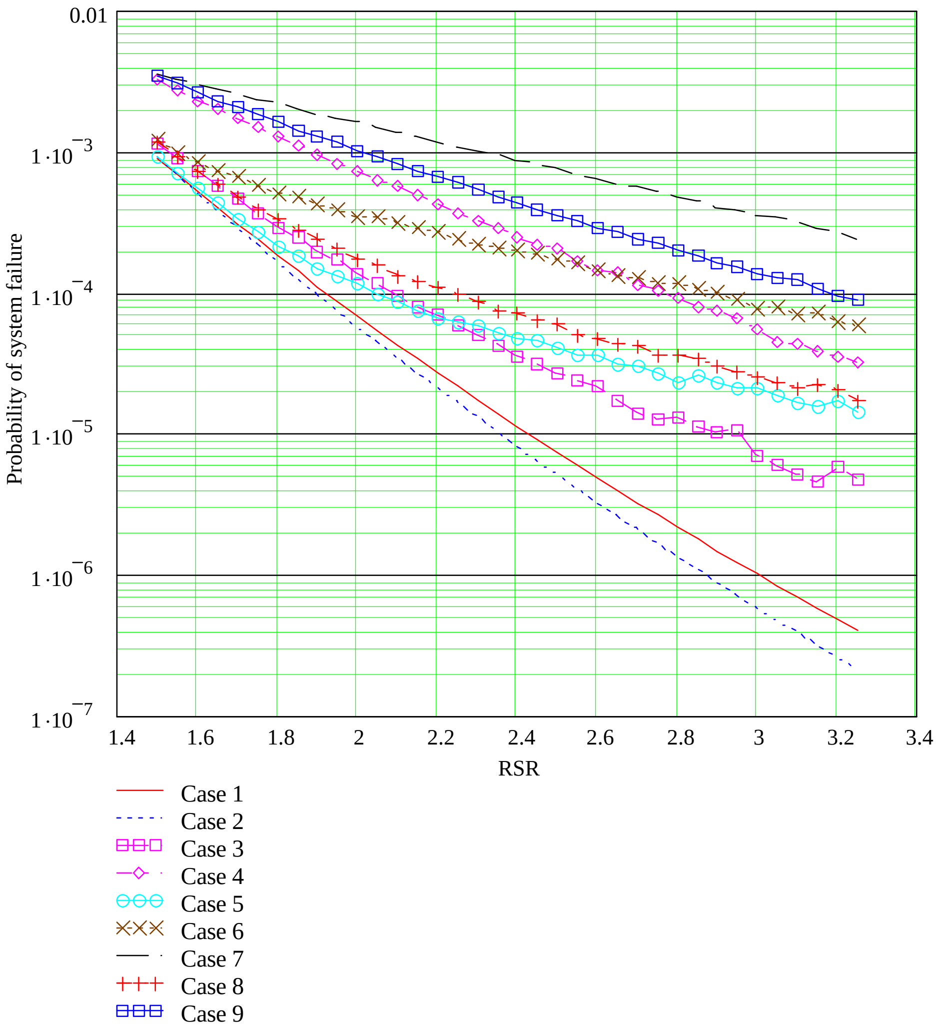

Figure 3: Comparison between probabilities of failure for the different cases

In order to obtain the normalised coefficients, the construction costs is related to field development cost and substructure cost to construction cost. The resulting coefficients are shown in Table 4.

Table 4: Relative economic data for recent jacket projects

	$\rho_{CC} = C_C / C_{FD}$	$\rho_{SS} = C_{SS} / C_C$	C_{SS}/C_{FD}
Kvitebjørn	0.680775	0.192717	0.131197
Grane	0.619269	0.168991	0.104651
Ringhorne	0.444513	0.161655	0.071858
Huldra	0.38893	0.310038	0.120583

Income

From the same data, the net present value (NPV) of income (sales minus operational cost) seems to be estimated to a value around 1.5 to 2 times the total investments in the field development (CAPEX). These estimates are based on a rather pessimistic oil price, and it is likely that these are meant as conservative decision support estimates. The real income is in most cases expected to be higher. In this study a mean value for ρ_I (I_{NPV}/C_{FD}) is assumed to be 3.0, and a variation between 1 and 5 is studied.

The income function of an offshore facility can be subdivided into three phases, namely the *build-up*, the *plateau* and the *decline* phases. In the *build-up* phase, the producers are installed and set on stream. During the *plateau phase*, the production is limited by the processing or transport capacity, and the maximal annual income is obtained, from which the processing and transport costs have to be subtracted. The *decline phase* succeeds the plateau phase and during this phase, the oil and/or gas production decreases exponentially, which is described by a decline factor. For water injected processing, this factor lies in the range from 0.03 to 0.22 and depends on several reservoir and production specific parameters (Almlund 1991). For this example, the decline factor was assumed to be 0.22, and the time for the start of the declining phase was set to 6 years. The *decline phase* ends with the decommissioning of the structure. This point in time is assumed to be 25 years.

Consequences due to fatalities

In order to account for possible fatalities, Nathwani et al. (1997) established the Life Quality Index (LQI). From this index, Skjong and Ronold (1998) derived the amount of money, which should be invested to avert a fatality *ICAF*. According to Rackwitz (2001), the societal loss due to losses of lives, can and should be taken into account in the design decision problem by including its cost equivalent C_{FF}, i.e. the expected number of fatalities NF multiplied with ICAF.

The ICAF value is estimated to 20 MNOK, based on Skjong (2001). For the present study the implied cost of avoiding a fatality as a ratio of the field development costs is used. Table 5 shows this ratio for the four jacket structures used in this example.

Table 5: Relative economical numbers for recent jacket projects

	ρ_{ICAF} = ICAF / C_{FD}
Kvitebjørn	0.002637
Grane	0.001329
Ringhorne	0.002452
Huldra	0.004942

Cost of material losses of a failure

The material cost of failure consists of (Nilsen 2002):

- Environmental losses
- Removal of wreck
- Production loss as a result of downtime after the incident
- Loss of reputation
- Liability expenses
- Long term effects

The cost of environmental losses will be dependent on the amount of environmental spill and the medium (oil or gas) that is produced at the actual installation. For the installation mentioned in this paper, both gas production and oil production is relevant, and cost of the environmental losses will be significantly different. Actual numbers are available from shipping accidents (Exxon Valdez, Braer), and can to some extent be relevant. According to Nilsen (2001) the environmental related costs of the Exxon Valdez accident has been estimated to 30000 MNOK, and for Braer and Sea Empress in the order of 2000 MNOK. The Exxon Valdez case may be viewed as an extreme case and not fully representative for an accident on the Norwegian continental shelf. As a result an estimate of 1000 MNOK is used in this evaluation.

The removal of the wreck can be estimated based on the costs for removing installations, and a cost between 1000 MNOK and 2000 MNOK may be a good estimate.

Income from production for a Norwegian field can be as high as 50 MNOK per day. The downtime will be dependent on the availability of facilities that can be used as replacements, and whether construction of new facilities is needed. In cases where a replacement facility can be used, downtime can be estimated to 50 – 100 days. In cases where a new facility is necessary, a downtime of more than a year seems appropriate. At an average it is assumed that production can be restarted after 200 days, and the cost of production loss can be estimated to 10000 MNOK.

The cost of lost reputation is difficult to assess. This will be different for a small, unknown operator and a worldwide energy consortium. The marked value of Exxon was reduced by approximately 24000 MNOK as a result of the Exxon Valdez incident (Nilsen 2001). A total collapse of a jacket structure may not result in the same amount of environmental spill, and it could be argued that a reduced number is relevant. However, the possible loss of lives would to some extent have the same effect on marked value. As the uncertainty around this is rather large, it is here assumed that a loss of 10% of the marked value of a medium to small-scale company may be relevant, resulting in an estimate of 10000 MNOK.

As a simplification, liability expenses (damage to third party property and personnel, e.g. the cost of the rescue operation) and long-term effects (consequences that emerge after normalisation that would not be there without the accident, e.g. the possibility of future loss in income due to a reduced willingness to continue this type of activities after an accident) are not evaluated. In Nilsen (2001) more details are given to model such expenses.

Finally, total material cost as a result of an accident then adds up to be in the order of 22500 MNOK, resulting in a factor ($\rho_{FM}=C_{FM}/C_{FD}$) of 2.6 times the field development costs. In Pinna et al. (2001), the ratio of failure costs to construction costs for a Monopod are indicated to lie in the range of 3 to 7. Pinna et al. (2001) further indicates that with the condition of a short reconstruction period, a ratio of 10 is indicated to be appropriate. In case of extraordinary severe failure consequences, including both complete failure and clean up costs, the ratio is indicated to be as high as 20. Relative to the field development cost, the ratio between failure costs and field development cost is indicated to be as high as $\rho_{FM}=10$ as a maximum value.

In this study an expected value of 2.6 times the field development cost is assumed, and a range from 0.2 to 5 times the field development cost is studied.

<u>*Cost of strengthening*</u>

The cost of construction of the substructure is assumed to be dependent of the strength of the structure. If the structure is built with an average strength (assumed to be represented with $RSR_0=2.0$) the cost is assumed to be C_{SS_FIX}. If the RSR of the structure is chosen different from the average, the cost is assumed to be linearly varying according to the following formula:

$$C_{SS} = C_{SS\_Fix} \cdot (1 + \rho_{St} \cdot (RSR - RSR_0)) \qquad (16)$$

The factor ρ_{St} is representing the ratio of increase in cost to an increase in the RSR.

$$C_{SS\_Fix} = \rho_{SS} \cdot C_C = \rho_{SS} \cdot \rho_{CC} \cdot C_{FD} \tag{17}$$

For a well-balanced structure (a structure where all members are optimized, and to increase the global strength would result in strengthening of all individual member and joint), the whole structure has to be strengthened in order to improve the RSR. However, for most jacket structures, other load situations (transport with barge to field, lifting or launching, fatigue, earthquake, etc) will also be critical and dimensioning for parts of the structure.

To increase the RSR, the frame braces will have to be strengthened, and also the legs to some extent. This part of the structure represent between 25 – 50% of the total weight, as indicated in Table 6. Based on this assumption, a 50% increase of the RSR (from 2 to 3) will lead to a weight increase of 12.5 to 25%, when the weight increase is assumed to be linearly. Due to buckling being critical for some members, the relationship will not be fully linear, and some additional weight increase should be expected. As a result a 25% increase is assumed (ρ_{St} = 0.25), and a variation from 10% to 50% is studied.

Table 6: Weight distribution in a jacket

	Portion of weight
Frame braces	0.16
Legs	0.20
Plane braces	0.10
Leg nodes	0.19
Plane nodes	0.06
Mudmat, pile sleves etc.	0.29

Summary of economical coefficients used in the study and their range

The economic optimal analysis is based on the expected values for the coefficients as presented in Table 7. As the chosen values for the study will not apply for all structures, the variables are also studied over a range as indicated in the last column.

Table 7: Chosen value for coefficients and the range of the coefficients

Coefficient	Base case	Studied range
ρ_{CC}	0.5	(0.4 – 0.7)
ρ_{SS}	0.2	(0.15 – 0.3)
ρ_{ICAF}	0.003	(0.001 – 0.005)
ρ_{I}	3	(1 – 5)
ρ_{St}	0.25	(0.1 – 0.5)
ρ_{FM}	2.6	(0.5 – 5)
NF (number of fatalities)	50	(1 – 100)
T (Lifetime of field)	25	
r (interest rate)	0.07	
RSR_0	2.0	

Numerical investigations / Case study

Based on the presented framework for decision theory and the economic data for 4 recent Norwegian jacket field developments, the introduced cases are evaluated and the optimal RSR is calculated for each case.

Sensitivity to the applied economic parameters

The coefficient in Table 7 is studied with respect to the influence on the optimal RSR. In Figure 4 the influence on the optimal RSR when each ρ coefficient is changed by +/- 50% from the base case is shown. It can be seen that the influence of the changes in the consequence of loss of lives (NF=50), and the ratio between income and field development cost is very small. The influence of cost of loss of lives will be increased if the ratio of ICAF and field development cost is larger. It should however be mentioned that the income factor ρ_I has a significant impact on the optimal time of possible reconstruction t_0. Further, it can be seen that the material cost of failure and the cost of strengthen the structure influences the optimal RSR significant.

The influence of the ICAF ratio and the number of possible fatalities is shown in Figure 5. The effect of the number of persons on the installation is relatively small with the relevant values of the ratio of ICAF to field development costs (ρ_{ICAF}= 0.001 – 0.01). This indicates that avoiding fatalities does not influence the optimal safety for an offshore jacket structure using this method of estimating the optimal safety. However, if the ICAF values are increased to a value closer to the field development cost, the cost of avoiding a statistical fatality makes significant impact on the optimal RSR and the associated probability of failure, as shown in Figure 5. It should be noted that this figure is applicable only for an increase in the ICAF value, not a decrease in the field development cost, as all the remaining parameters are kept constant. These parameters will also be influenced by a decrease in field development cost.

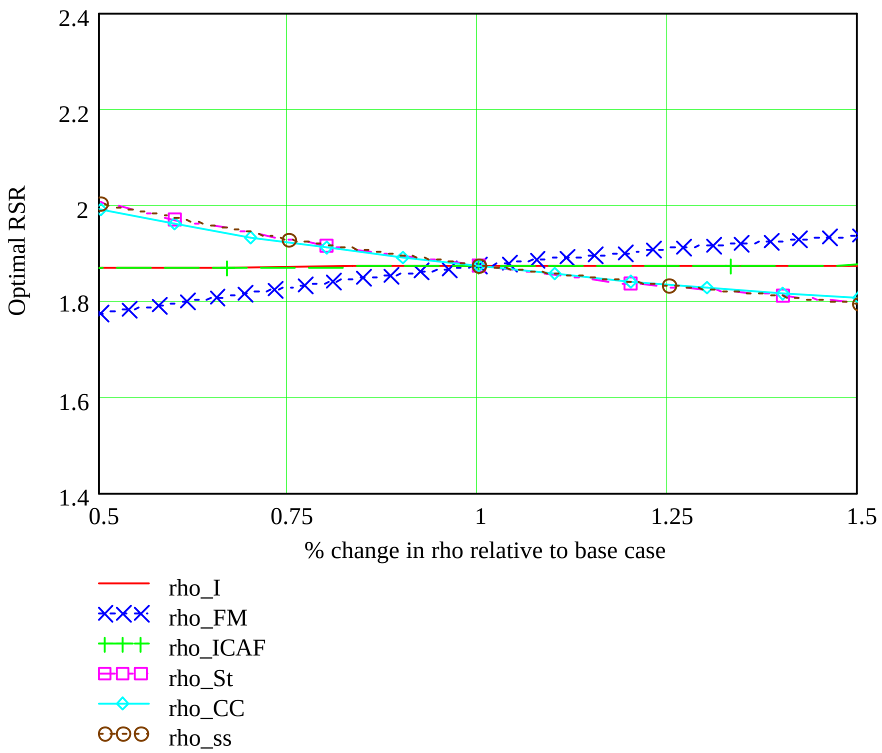

Figure 4: Sensitivity to the economical parameters for the optimal RSR (Case 1)

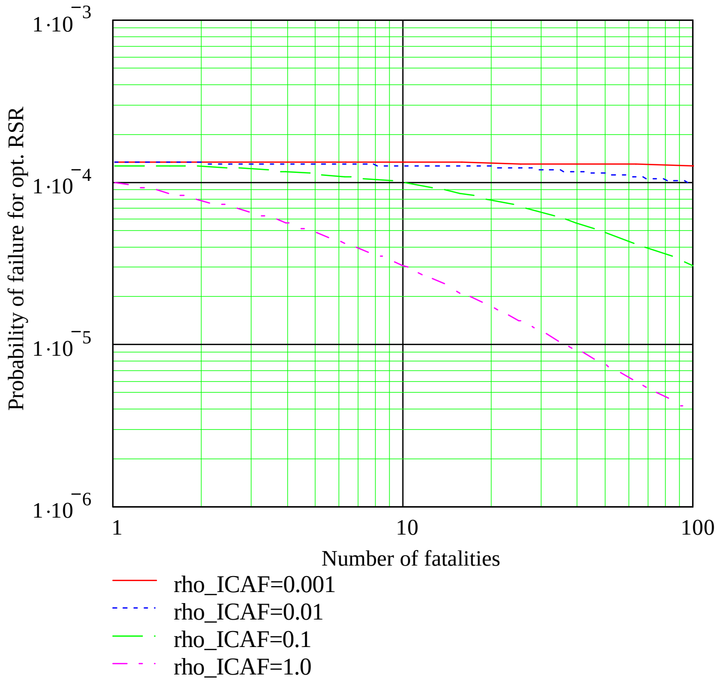

Figure 5: Calculated probability of failure for the optimal RSR with various ICAF values

Sensitivity to epistemic uncertainty

When removing the epistemic uncertainty, the probability of failure is slightly reduced (see Figure 3). However, this does not change the optimal RSR significantly, as the expected benefits are mainly parallel adjusted as shown in Figure 6. Both based on Figure 3 and Figure 6 it seems reasonable to conclude that a relatively large change in the epistemic uncertainty does not result in significant change in the optimal RSR decision (and it's related probability of system failure).

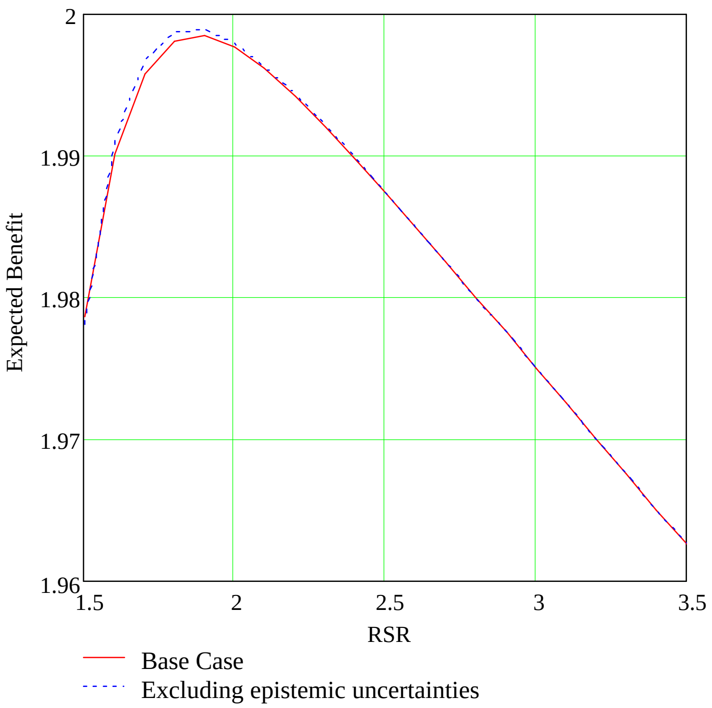

Figure 6: Expected benefit for Case 1 and Case 2

Sensitivity to possible member failures

When possible member failure is introduced, the effect of shifting for the maxima of the expected benefit is more significant. This results in an increasing optimal RSR with increasing probability of member failure, as shown in Figure 7.

The model for the effect of member failure in this study is too simple to draw general conclusions. However, the results clearly show an increasing trend when the probability of a possible member or node failure increases. Figure 7 indicates that this should be addressed when estimating an optimal RSR, at least in cases where the probability of any member or node failure exceeds $1{\cdot}10^{-3}$, which is a likely number for many jacket structures on the Norwegian continental shelf.

Sensitivity to wave in deck loading

Also the possibility of wave in deck loading when reducing the deck height influences the optimal point of the expected benefit, as shown in Figure 8. However, the shift in the expected benefit is moderate for air gaps of 22 m and 20 m.
It is important to note that wave in deck loading does not seems to be needed in the optimisation when air gap is greater than the 10^{-4} wave crest elevation (approximately 24 m in this case). Designing for a 10^{-4} wave crest elevation is a normal requirement on the Norwegian continental shelf.
The optimal air gap could be estimated in a parallel optimisation.

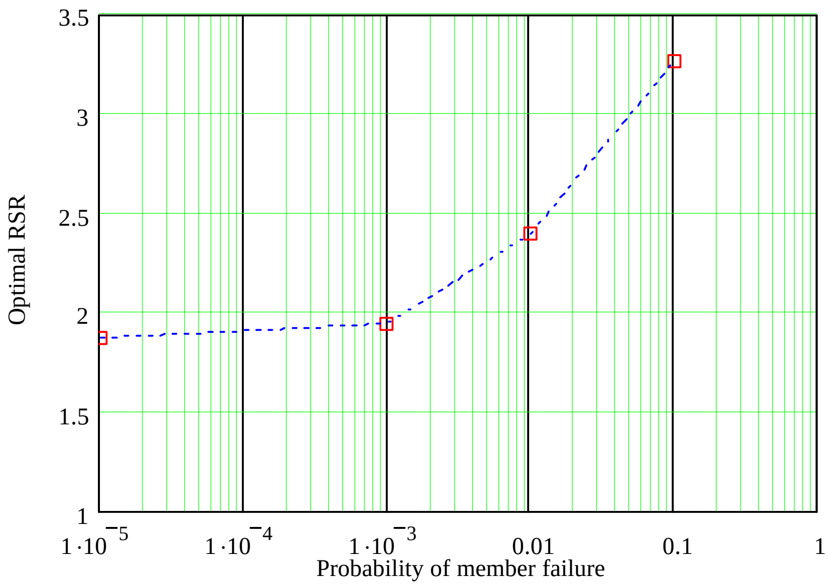

Figure 7: Optimal RSR with increasing probability of member failure

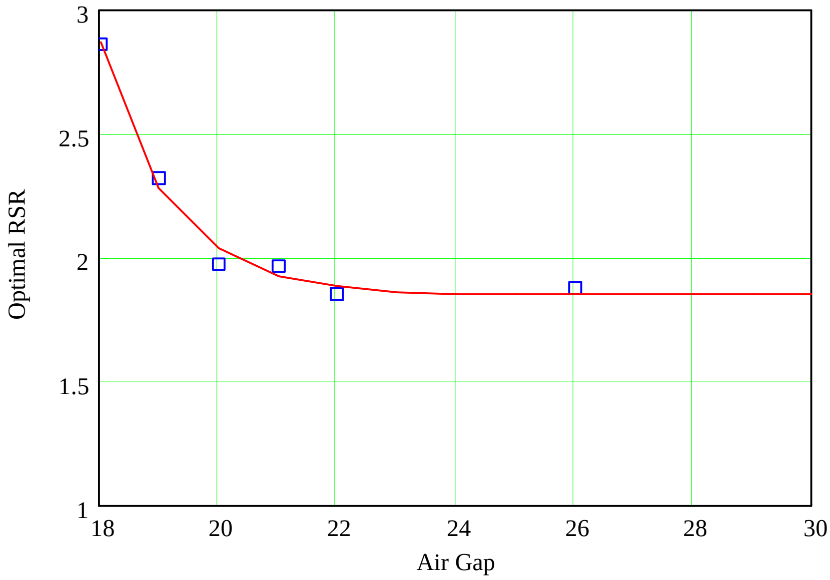

Figure 8: Optimal RSR with increasing air gap (calculated points and curve fit)

Discussion

Common design practise is in general deterministic with fixed requirements to the strength of the structure, or in some cases probabilistic with given acceptance criteria. The implicit requirement in such deterministic regulations indicates that RSR values should be 2.0 and higher depending on the ratio of permanent and live loads to the environmental loads (Ersdal 2004). Permanent loads and live loads are not included in this study, so the comparable RSR value will be 2.0. Also, based on a probabilistic acceptance criterion in the range of 10^{-4} to 10^{-5}, the results of reliability analysis in this paper would indicate a requirement to the RSR around 2.0.
Based on the method presented in this paper, and with input values applicable for a quarter platform (ρ_{FM} = 1.0, ρ_{CC} = 0.9, ρ_{SS} = 0.6, NF= 300 and with the rest of the economic factors as shown in Table 7), an optimal RSR as low as 1.5 is found to be the optimal value based on case 1 (wave in deck loading and member failure not included). By this it follows that the result of an economical optimisation may lead to a less conservative design and a reduction of the safety for the persons aboard. It should also be noted that when member failures are include in the optimisation, a somewhat higher optimal RSR is found. In order to explore whether this is acceptable or not, the underlying thinking behind the methods is briefly discussed with basis in the two following approaches for decision making under uncertainty and risk:

1) If all members of the society have an equal right to life and bodily security, then the same amount of expenditure should be implied in all parts of the society in order to avoid a statistical fatality (Nathwani et al. 1997, Skjong and Ronold 1998). This approach is based on Bayesian Decision Theory and the Life Quality Index (LQI).
2) If all members of the society have an equal, prima facie right to life, and therefore to bodily security, as the most basic of human rights, then allowing one group of

persons to be put at greater risk, without compensation and for no good reason, amounts to violating their rights to life and bodily security (Shrader-Frechette, 1991). This approach is often understood such that the probability of fatality for the individual workers should not be significantly increased as a result of working in the offshore industry, resulting in a *maximum allowable risk approach*.

The mathematical evaluations with cost consequences of fatalities in this paper are clearly based on *approach 1*. An ethical discussion is needed prior to accepting that these methods can be used alone to define the safety level. This discussion will not be taken in this paper. However, the following observations are made.

Based on the *approach 1* the following observation can be made: 1) A maximum allowable risk criteria is irrational with the Bayesian decision approach as basis (can not be logically derived from this principle). 2) As the present method in many cases indicates lower RSR values compared to requirements in regulations and standards (e.g. Norwegian Standards), the application of an RSR which is based on an economic optimization will lead to less conservative design. 3) The method will indicate the most economically optimal solution for the society.

Based on the *approach 2* the following observation can be made: 1) Similarly as mentioned above, approach 1 will be irrational based on a maximum risk approach. 2) The result presented in this paper in many cases indicates lower RSR values compared to requirements in regulations and standards (e.g. Norwegian Standards), the application of an RSR which is based on an economic optimization will lead to unacceptable high risk. 3) The results of economic optimal RSR, with a fixed amount of monetary value to avoid a statistical fatality, will be an unequal safety levels at different installations, as the safety level at individual installation will be governed by the material cost of failure and cost of substructure strengthening. As a result, the method will be in conflict with the belief that an equal safety level (with improvement over time) should be the goal for risk related activities. 4) Within the maximum risk approach it would also be ethical unacceptable to have the total value of human life measured in monetary terms or as an implied cost of avoiding a fatality.

Conclusions

A decision analysis of optimal RSR values based on Cost Benefit Analysis is clearly dependent of the additional failure modes investigated in this paper, as the optimal RSR is highly sensitive to the possibility of member failures and a reduced air gap. It is also seen that if the air gap is sufficient to allow for the wave crest elevation with an annual probability of exceedance of 10^{-4} to pass without hitting the deck, the optimal RSR is not influenced by the air gap. With regards to the influence of possible member failure, more detailed analysis and models should be investigated in order to make general conclusions. The optimal RSR is slightly sensitive to the inclusion of epistemic uncertainties, but this is small compared to the uncertainty with regards to the cost of failure.

Material cost of failure, costs for substructure, and strengthening of the substructure are the most important factors for the evaluation of optimised RSR. With the presented cost of field development and ICAF values the loss of lives has very little impact on the optimal RSR.

The consequence of optimisation will lead to a varying RSR value for different installations. The safety level is decreased by the optimisation compared to deterministic regulations and standards, and economic optimisation will lead to a less conservative design. This as the Cost Benefit Analysis (CBA) method used alone for estimating the optimal RSR will in many cases contradict with the national safety regulations in some countries, e.g. Norway, when applied to the safety of human life.

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