NORSOK STANDARD

N-006

6. draft, May 2008

Assessment of structural integrity for existing offshore load-bearing structures

Standards Norway
Strandveien 18, P.O. Box 242
N-1326 Lysaker
NORWAY

Telephone: + 47 67 83 86 00
Fax: + 47 67 83 86 01
Email: petroleum@standard.no
Website: www.standard.no/petroleum

Foreword

The NORSOK standards are developed by the Norwegian petroleum industry to ensure adequate safety, value adding and cost effectiveness for petroleum industry developments and operations. Furthermore, NORSOK standards are, as far as possible, intended to replace oil company specifications and serve as references in the authorities' regulations.

The NORSOK standards are normally based on recognised international standards, adding the provisions deemed necessary to fill the broad needs of the Norwegian petroleum industry. Where relevant, NORSOK standards will be used to provide the Norwegian industry input to the international standardisation process. Subject to development and publication of international standards, the relevant NORSOK standard will be withdrawn.

The NORSOK standards are developed according to the consensus principle generally applicable for most standards work and according to established procedures defined in NORSOK A-001.

The NORSOK standards are prepared and published with support by The Norwegian Oil Industry Association (OLF), The Federation of Norwegian Industry, Norwegian Shipowners' Association and The Petroleum Safety Authority Norway.

NORSOK standards are administered and published by Standards Norway.

Introduction

The principle standard for offshore structures is NORSOK N-001, structural design which especially refers to ISO 19900, *Petroleum and natural gas industries- General requirements for offshore structures.* This standard gives additional requirements for assessment of the structural integrity of offshore structures in-service and for life extension.

Drafting Note:

The standard is prepared by Oljeindustriens Landsforening (OLF) and will be proposed to be issued as a NORSOK standard.
The fifth draft of this standard is revised according to comments received to the fourth draft by PSA and concrete specialist in Aker Engineering and Technology. The revised text in the fifth draft is shown in blue. There is not shown other revision marks meaning that text deleted from the fourth draft is not marked. Corrections to figures are not marked. Minor corrections such as misprints are not marked.

1 Scope

This NORSOK standard specifies general principles and guidelines for assessment of the structural integrity of existing offshore structures as a supplement to NORSOK standard N-001 and should be used in conjunction with NORSOK standards N-003, N-004 and N-005. The present standard serves as an alternative to NORSOK N-001 for cases where structures are to be operated beyond original design requirements and structural resistance is not easily verified through ordinary design calculations, and where use of additional information gained through the life of the structure can be used to demonstrate structural adequacy.

This NORSOK standard is applicable to all types of offshore structures used in the petroleum activities, including bottom founded structures as well as floating structures. As the majority of ageing facilities are fixed structures of the jacket type, the detailed recommendations given are most relevant for this type of structure.

This NORSOK standard is applicable to different types of materials used including steel, concrete, aluminium, etc.

This NORSOK standard is applicable to the assessment of complete structures including substructures, topside structures, vessel hulls, foundations, mooring systems and subsea facilities.

2 Normative and informative references

2.1 General

The following standards include provisions and guidelines which, through reference in this text, constitute provisions and guidelines of this NORSOK standard. Latest issue of the references shall be used unless otherwise agreed. Other recognized standards may be used provided it can be shown that they meet the requirements of the referenced standards.

2.2 Normative references

ISO 19903	Petroleum and natural gas industries. - Fixed concrete offshore structures
NORSOK N-001	Integrity of offshore structures
NORSOK N-003	Actions and action effects
NORSOK N-004	Design of steel structures
NORSOK N-005	Condition monitoring of load bearing structures

2.3 Informative references

BS 7910	Guidance on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures
DNV-RP-C203	Fatigue Design of Offshore Steel Structures
DNV-RP-C206	Fatigue Methodology of Offshore Ships
ISO 19900	Petroleum and natural gas industries. General requirements for offshore structures
ISO 19901-7	Petroleum and natural gas industries. Specific requirements for offshore structures Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units
ISO 19902	Petroleum and natural gas industries. - Fixed steel offshore structures
ISO 19904-1	Petroleum and natural gas industries. - Floating offshore structures - Part 1: Monohulls, semi-submersibles and spars
PSA, SFT and NSHD	Regulations relating to material and information in the petroleum activities. “The duty of information regulations”
PSA, SFT and NSHD	Regulations relating to design and outfitting of facilities etc. in the petroleum activities. “The facility regulations”
PSA, SFT and NSHD	Regulations relating to conduct of activities in the petroleum activities (the activities regulations)
Royal Decree (Norway)	Royal Decree 31 August 2001: Regulations relating to health, the environment and safety in the petroleum activities. “The framework regulations”

3 Terms, definitions, abbreviations and symbols

For the purposes of this NORSOK standard, the following terms, definitions and abbreviations apply.

3.1 Terms and definitions

3.1.1
shall
verbal form used to indicate requirements strictly to be followed in order to conform to this NORSOK standard and from which no deviation is permitted, unless accepted by all involved parties

3.1.2
should
verbal form used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required

3.1.3
may
verbal form used to indicate a course of action permissible within the limits of this NORSOK standard

3.1.4
can
verbal form used for statements of possibility and capability, whether material, physical or casual

3.1.5
design service life
assumed period for which a structure is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary

3.1.6
original design service life
design life premised at the design stage

3.1.7
extended design service life
assumed period the structure is to be used in addition to its original design service life

3.1.8
total design service life
the sum of the original design service life and the extended design service life

3.1.9
unmanning criterion
the environmental condition (e.g sea state, wind speed) at which the facility should temporarily be unmanned

3.2 Abbreviations

ALS	accidental limit states
API	American Petroleum Institute
BSI	British Standards Institution
DFF	design fatigue factor
DFI	design, fabrication and installation
DNV	Det Norske Veritas
FE	finite element
FLS	fatigue limit states
IMO	International Maritime Organisation
ISO	International Organisation for Standardisation
MPI	magnetic particle inspection
NDT	non-destructive testing
NSHD	Norwegian Social and Health Directorate
PSA	Petroleum Safety Authority Norway

SFT	Norwegian Pollution Control Authority
SCF	stress concentration factor
SLS	servicability limit states
ULS	ultimate limit states

3.3 Symbols

C_{lim}	the maximum wave crest elevation for the structure calculated according to the principles for ALS checks
C_{max}	maximum characteristic wave crest elevation
D_{LCF}	accumulated damage from low cycle fatigue during the considered storm period using Palmgren – Miner accumulation rule
D_{HCF}	accumulated damage from high cycle fatigue during service life using Palmgren – Miner accumulation rule
H_{lim}	the maximum wave height that the structure can resist calculated according to the principles for ALS checks
H_{max}	maximum characteristic wave height
H_s	significant wave height
T	chord thickness, time
d_f	directional wave factor
P_{ann}	annual probability of exceedance
$p_{f\text{-}con}$	probability level for a conditional characteristic wave
$p_{f\text{-}dhv}$	probability level for failure of down-hole-safety valves
$p_{f\text{-}target}$	required probability level for an environmental action
t_d	time when a fatigue crack can be detected
t_T	time when a fatigue crack has grown through thickness

4 Assessment Process

4.1 General

Assessment of existing structures shall be undertaken if any of the initiators specified in Section 4.2 are triggered. The purpose of such an assessment is to demonstrate that the structure is capable of carrying out its intended functions in all phases of their life cycle.

The assessment process shall include or be based on:

- documentation of as-is condition,
- planned changes and modifications of the facility,
- calibration of analysis models to measurements of behaviour if such measurements exists,
- the history of degradation and incidents,
- prediction of future degradations and incidents based on earlier history,
- the effect of degradation on future performance of the structure,
- a documentation of technical and operational integrity,
- planned mitigations,
- a plan or strategy for the maintenance.

The assessment for life extension shall conclude on a safe life extension period with respect to technical and operational integrity of the facility. The assessment shall further identify the circumstances that will limit the life of the facility without major repairs or modifications, and specify criteria defining safe operation (e.g. permissible cracks lengths, permissible corrosion or remaining thickness, remaining anodes, degrading of paint protection, subsidence), including appropriate factors of safety.

The Assessment Process is illustrated in the enclosed flow sheet in Figure 1. This flow sheet may be followed for assessment of all groups of limit states (ULS, SLS, ALS and FLS).

Data collection is an important part of an assessment process. Reference is made to Section 5.2. A further collection of data should be considered if significant data are missing. The feasibility of data should be considered. An update of the design basis may be part of the assessment of data. If data are missing, one solution to this may be to make assumptions to the safe side.

Structural analyses for assessment may be performed provided that a sufficient data basis for performing reliable analysis is available. Then the safety level of the structure can be assessed and it can be decided if mitigation is required. By analysis is understood an engineering process that can imply assessment based on simple hand calculation or more refined structural analysis.

A plan for mitigation as indicated in the flow sheet in Figure 1 involves:

- A plan for the mitigation itself.
- Documentation.
- Plan for maintenance after mitigation.

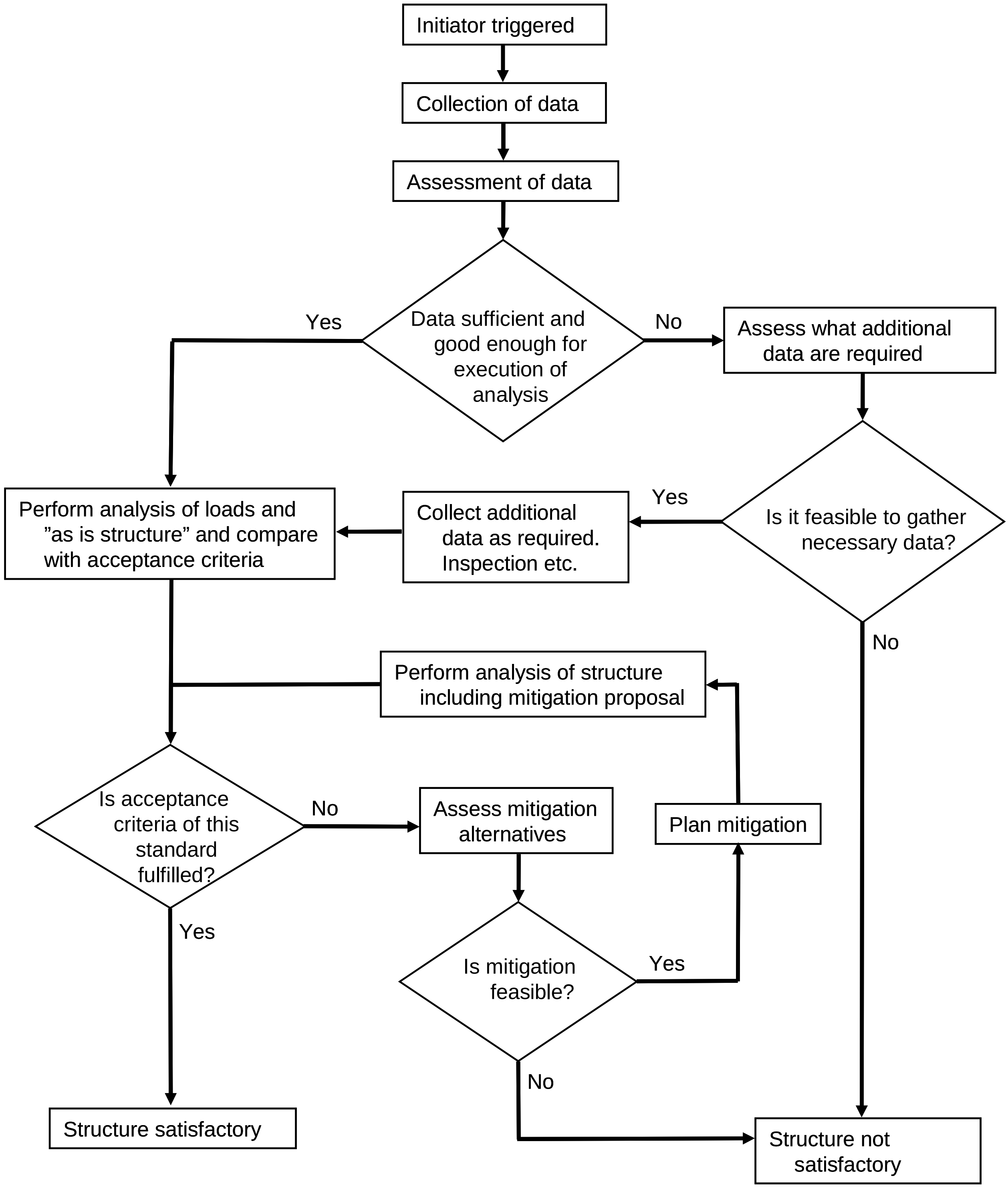

Figure 1 Flow sheet of the assessment process

4.2 Structural assessment initiators

An existing structure shall be assessed to demonstrate its fitness-for-purpose if one or more of the following conditions exist.

a) Changes from the original design or previous assessment basis, including
   1) modification to the facilities such that the magnitude or disposition of the permanent, variable or environmental actions on a structure are more onerous,
   2) more onerous environmental conditions and/or criteria,
   3) more onerous component or foundation resistance data and/or criteria,
   4) physical changes to the structure's design basis, e.g. excessive scour or subsidence, or relocation of mobile offshore units to a new location and,
   5) inadequate deck height, such that waves associated with previous or new criteria will impact the deck, and provided such action was not previously considered.

b) Damage or deterioration of a primary structural component: minor structural damage can be assessed by appropriate local analysis without performing a full assessment; however, cumulative effects of multiple damages shall be documented and included in a full assessment, where appropriate.

c) Exceedance of design service life, if either
   - the remaining fatigue life (including design fatigue factors) is less than the required extended service life, or
   - degradation of the structure due to corrosion beyond any design allowances has occurred, or is likely to occur within the required extended service life.

   Existing design documentation can be applied as basis for the assessment if inspection of the structure shows that time-dependent degradation (i.e. fatigue and corrosion) has not become significant and that there have been no changes to the design criteria (any changes to the original design basis are assessment initiators, see a) above). This requires that in-service inspection has been performed to document a proper safety level.

A structure which has been totally decommissioned (e.g. an unmanned facility with inactive flowlines and all wells plugged and abandoned), or a structure in the process of being decommissioned (e.g. wells being plugged and abandoned) generally does not need to be subjected to the assessment process unless its failure has consequences for nearby facilities.

Note: On the Norwegian Continental Shelf the design life will be limited to the planned design service life stated in the Plan for Development and Operation and the DFI résumé, as defined in the Information Duty Regulation.

5 Data collection

5.1 General

The information needed to perform the assessment shall have sufficient accuracy for its purpose. In case of lack of data or insufficient information, assumption to the safe side may be made.
Requirements to data management for offshore structures can be found in NORSOK N-001 and in ISO 19902.

In the assessment process, information about past performance of the structure and maritime systems shall be collected. This shall also include gathering of experience from similar facilities if available.

5.2 Collection of data

The following information shall be available for assessment:

- As built drawings of the structure.
- New information on environmental data (if relevant).
- Permanent actions and variable actions.
- Functional requirements.
- Design and fabrication specifications.
- Design, fabrication, transportation and installation reports which should include information about material properties (such as material strength, elongation properties and material toughness test values), weld procedure specifications and qualifications, modifications and weld repairs during

fabrication, non-destructive testing (extent and criteria used), pile driving records (action effects during pile driving and number of blows).

- Weight report that is updated during service life.
- In-service inspection history including information on marine growth, corrosion, cracks, dents and deflections, scour, damages due to frost, impact, dents, erosion/abrasion, chloride intrusion, leakages, sulphate attacks.
- Information on in-place behaviour (possible measurements and observations)
- Information and forecast for seabed subsidence.
- Information on modifications, repair and strengthening to the structure during service.
- Experience from similar structures during service life.

5.3 Requirements to in-service inspection to assess as-is condition

5.3.1 General

General requirements to in-service inspection are given in NORSOK N-005. Requirements for various types of facilities are given in 5.3.3 to 5.3.5.

Special attention should be made to details in the splash zone. It is difficult to analyse the structure in this area as a number of different phenomena add together in terms of fatigue damage: wave action, variation in buoyancy due to waves and wave slamming. This area may also be exposed to damages from ship impacts.

5.3.2 Corrosion protection

Ordinary inspection procedures according to N-005 will reveal when protection effect from coating or cathodic protection will no longer suffice.

Mitigation shall be implemented if cathodic protection no longer gives satisfactory protection. Mitigations can be made in the form of:

- Addition of anodes that are clamped or otherwise attached to the structure and electrically connected.
- Installation of a separate structure with anodes that is placed in the vicinity of the facility and that is electrically connected to the structure.
- Installation of system for impressed current.

Coated surfaces can be protected against corrosion by proper maintenance. If corroded structural parts are detected, the capacity of the structural members may be assessed according to recommendations given in NORSOK N-004.

5.3.3 Steel structures

Requirements to in-service inspection planning and structural integrity management of jacket structures in general are presented in Chapter 23 of ISO 19902.

For steel structures it is important to control degradation mechanisms related to corrosion and fatigue. Reference is also made to Chapter 7 of this standard for assessment of the fatigue limit state.

5.3.4 Concrete structures

Requirements to inspection and condition monitoring are given in Chapter 14 of ISO 19903.

5.3.5 Floating structures

Requirements to in-service inspection, monitoring and maintenance are given in Chapter 18 of ISO 19904-1. Requirements are given in Chapters 12 and 14 of ISO 19901-7 for station keeping systems.

6 Assessment principles for existing structures

6.1 General

Existing structures shall meet the requirements of NORSOK N-001. Existing facilities where the primary structure does not meet the criteria for ULS or ALS related to environmental actions that can be forecast like wave and wind actions may continue to be used if the following four requirements are fulfilled:

1) Shut-down and unmanning procedures are implemented. The procedure for shut down and unmanning should meet criteria given in 6.3.
2) Requirements to unmanned facilities according to NORSOK N-001 are satisfied.
3) The environmental actions will not jeopardize any other main safety function (other than structural integrity) relevant for the facility during the storm. Detailed requirements are given in 6.4.
4) The risk of significant pollution is found acceptable. Procedure to check this is given in Chapter 6.5.

With primary structures, in this connection, is understood structural parts where a failure in case of a storm situation, may lead to loss of life, significant pollution or loss of main safety functions needed for safe operation of the facility during the storm.

Existing facilities where structural details do not satisfy the criteria for FLS may continue to be used if requirements in Section 7 and 9 of this standard are fulfilled.

Requirements to assessment of existing concrete structures are presented in Chapter 15 of ISO 19903. Here assessment of fatigue and corrosion is specially mentioned to be considered.

Floating structures shall be checked in the as is condition, with data as described in 5.2, for watertight and weatherproof integrity.

6.2 Assessment of maritime systems

The inspection and maintenance program shall be updated to reflect the as is condition with planned modifications, and if relevant the planned life extension period. Inspection and maintenance program shall include:

- leak detection system,
- watertight and weatherproof closing appliances,
- ballasting and stability, included seawater intake,
- mooring and positioning systems, and
- related safety systems which depend on emergency power or hydraulics.

6.3 Shut-down and unmanning criteria related to structural integrity

A shut-down and unmanning procedure shall be implemented for facilities not satisfying ULS or ALS requirements to manned facilities with respect to environmental conditions and hence need to be shut-down and unmanned during storms.

The shut-down and unmanning procedure shall be implemented in order to ensure that there is less than $5*10^{-4}$ annual probability of the facility being exposed to environmental actions exceeding the structural capacity determined according to the principle of ALS, with personnel onboard.

Note: See Commentary in Annex A for an example of how an unmanning criterion for waves expressed as sea-state thresholds (Hs) can be determined.

6.4 Shut-down and unmanning criteria related to Main Safety Functions

The threshold for environmental conditions in 6.3 relate to structural capacity of the facility and presumes that topside equipment is properly protected or secured against waves or wind actions and that necessary operational safety procedures are in place. This is to ensure that Main Safety Functions other than structural integrity that are relevant for the facility during the storm are not jeopardized. If topside equipment is not properly protected or secured against waves or wind actions, the facility shall be shut-down and unmanned based on a threshold which gives an annual probability of $1*10^{-4}$ of the Main Safety Functions being impaired.

Note: See Commentary in Annex A for an example of how a criterion for waves expressed as sea-state thresholds (Hs) can be determined.

Note: For requirements to Main Safety Functions see PSA, SFT, NMHD: “The Facility Regulation” Section 6 and 10

6.5 Structural requirements due to pollution risk

For well-head facilities and facilities with large oil storage tanks, requirements to unmanned structures in NORSOK N-001 may not be sufficient to satisfy required safety levels for major pollution events (e.g. blow-out). In such cases it should be shown that the combined probability of a structural failure and leakage that could lead to significant pollution is less than 10^{-4} per year.

6.6 Determination of directional wave criteria

When directional wave criteria are used, it may be necessary to use modified probability levels for the characteristic actions in each direction compared to omnidirectional criteria. It should be ensured that the sum of probability of failure for all directions for structures assessed by use of directional criteria is not larger than what would be obtained by using omnidirectional design values for a structure with the same resistance characteristics regardless of directions. The values of the characteristic waves to be used will be a function of the directionality of the waves and the properties of the structure for the various directions. There is consequently, no general answer to this and it can be necessary to develop criteria in each case. If specific criteria are not derived, the following method for determining directional criteria can be used:

Characteristic directional waves are calculated as waves with a probability of being exceeded equal to $p_{f\text{-}target}$ / d_f. Here d_f is a wave directional factor and $p_{f\text{-}target}$ is the exceedance probability level for the characteristic wave in question. Proposed value of d_f is given in Table 1. The directional wave criterion for a sector is defined as the minimum of the characteristic directional wave for the sector and the omnidirectional wave.

Directional wave criteria may be used for the following groups of limit states SLS, ULS and ALS as found appropriate.

Table 1 Values for the directional factor

Number of directions	d_f
4	2
8	4
12	6

7 Check of Fatigue Limit States

7.1 General

The fatigue life is considered to be acceptable and within normal design criteria if the calculated fatigue life is longer than the total design service life times the design fatigue factor (DFF) (NORSOK N-001). Otherwise a more detailed assessment including results from performed measurements of action effects and/or inspections throughout the prior service life as shown in Figure 2.

Well documented in-service records of joints with the shortest calculated fatigue lives can be used to document the fatigue reliability.

A fatigue assessment should take into account all available information, which includes the following:

1) In-service history of the structure. This shall include changes and modifications to the facility (weight and weight distribution, ballasting etc), and assessment of any reported damages (including fatigue damages). This shall also include measurements from structural monitoring, if available.

2) Planned future changes to the facility.

3) Consider the structural analysis models required to obtain a reliable assessment. Different structural models can be required for different phases, subsidence stages or modifications. When different analysis models are used, the total fatigue damage should be calculated by adding together the damages from the different models representing a defined time period.

4) A detailed and consistent fatigue analysis of the structure based on best practise for such analysis and use of S-N data.
Note: Guidance can be found in Annex A of this standard and in NORSOK N-001.

5) For structures where sufficient fatigue lives cannot be documented based on analyses and when it is considered difficult to document sufficient structural reliability by inspection only, it is recommended to perform measurements of action effects of the global force flow in the structure (e.g. axial forces in brace elements). These can then be used to calibrate the calculated forces and/or the analysis model to arrive at more precise fatigue lives.

6) The calculated fatigue lives should be compared with results from in-service inspections with respect to fatigue cracks. If fatigue cracks are found in primary details in the structure, it should be checked that this can be expected based on calculated lives.

   If calculated lives are not in agreement with observed fatigue cracking, it is recommended to look into the remaining uncertainties related to calculation of hot spot stress and fatigue capacity. This means assessment of relevance of stress concentration factors used in case of complex connections and S-N data for the actual fabrication.

   Then a further calibration of data may be performed based on a total assessment of the most significant parameters contributing to uncertainty in calculated fatigue life.

   Finally a revised fatigue assessment shall be performed as basis for planning further in-service inspection for fatigue cracks that fulfils target safety level or the intended safety level in NORSOK N-001.

7) For planning of in-service inspection for fatigue cracks it is recommended to develop crack growth characteristics; i. e. calculated crack length/depth as function of time/number of cycles (this depends on type of joint, type of loading, and possibility for redistribution of stress during crack growth).
Note: Reference is also made to Annex A.

8) The crack growth analysis based on fracture mechanics should be calibrated to that of the S-N data in such a manner that the crack growth characteristics will not be non-conservative when it is used for assessment of inspection intervals.
Note: Reference is made to /24/.

9) The acceptance criterion shall be linked to redundancy or consequence of failure as is implicit in the requirement to Design Fatigue Factors presented in NORSOK N-001.

10) Assessment of maximum allowable crack size (corresponding to some defined maximum action effect that gives satisfactory reliability) should be made. This can be based on BS 7910. If BS 7910 is used for the assessment, characteristic environmental actions with 100 year return period and load and resistance factors equal to unity can be used.
Note :Reference is also made to the Commentary section 7.7 in Annex A.

11) The inspection interval should be made dependent on the reliability of the inspection method that is being used.

12) Elapsed time from earlier inspections should be accounted for in this assessment.

In-service inspection is an integral part of structural integrity management, which is an ongoing process for ensuring the fitness-for-purpose of an offshore structure or of a group of structures.

For steel structures it is recommended to use Electromagnetic NDT methods, i.e. Eddy Current (EC) or Magnetic Particle Inspection (MPI) for the detection of surface cracks (e.g. at weld toe hot spots) in high consequence welded connections. In addition, it is recommended to periodically verify the condition of low consequence members by means of flooded member detection (FMD).
For floating structures the condition may be controlled by leak detection systems. In order to rely on such systems “leak before failure” should be documented.

If an EC/MPI surface crack detection program based on RBI analysis is performed without findings, the time to next inspection may be reassessed based on this information. However if significant fatigue cracks are found, it is necessary to review the inspection intervals for similar important joints which have not required previous EC/MPI inspection.

For structures where several primary connections show short calculated fatigue lives and/or the inspection history of these connections indicates that significant fatigue damage may have been accumulated, it is recommended to consider that more than one connection can fail due to fatigue in combination with a severe storm loading. This can be performed by an assessment of consequence of failure of the relevant connections. Such an assessment may lead to a reclassification of the considered structural component from being “Without substantial consequence” to that of “Substantial consequences”. See NORSOK N-001 for definitions.

This means that requirements to Design Fatigue Factors will be enhanced following NORSOK N-001 when going from “Without substantial consequence” to that of “Substantial consequences”.

This also implies that the target safety level for in-service inspection will be enhanced. For example when planning in-service inspection based on RBI methods the target safety level is normally linked to the DFF required for the considered connection. Reference is also made to Section 9.

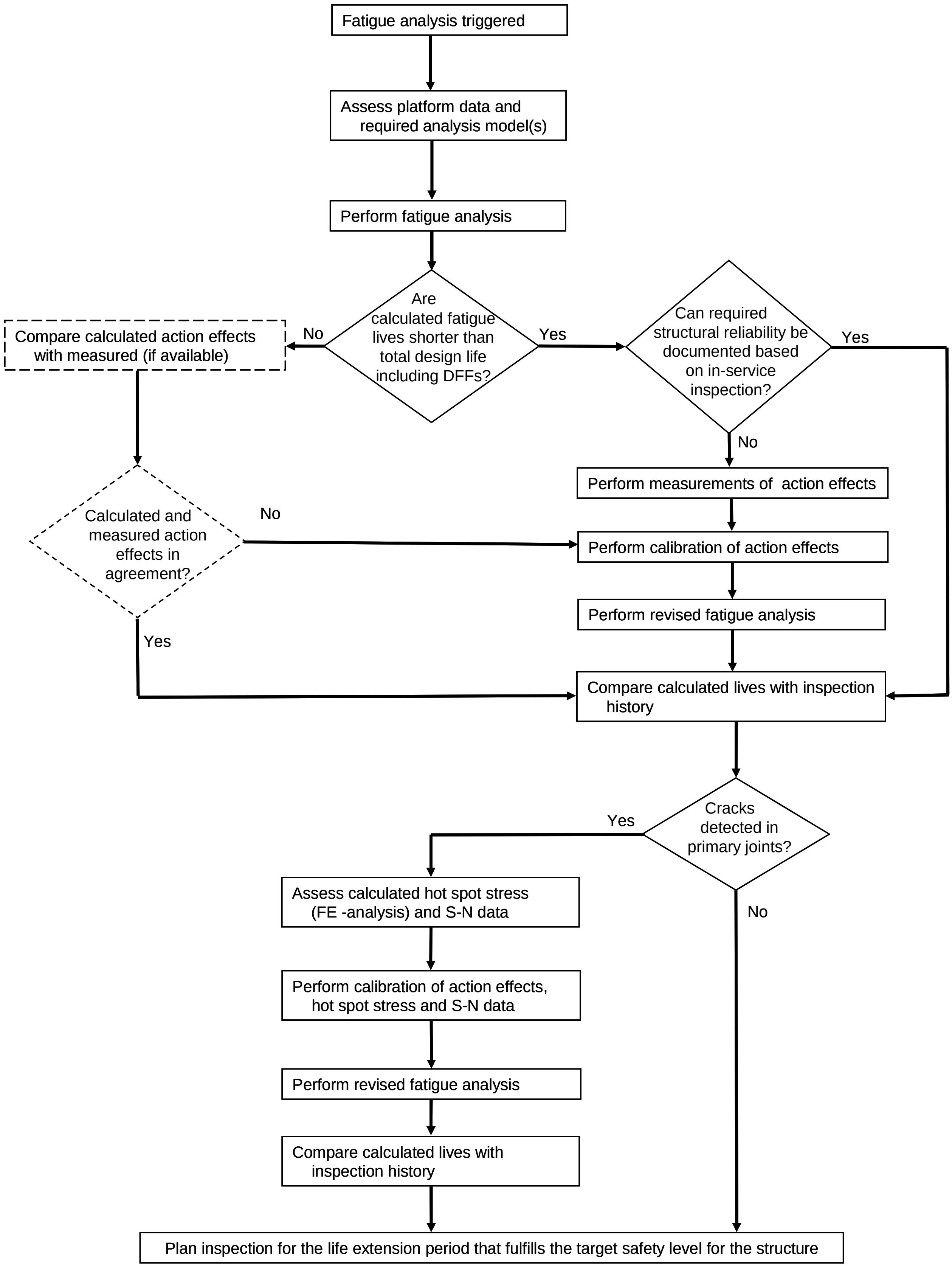

Figure 2 Assessment with respect to fatigue

7.2 Fatigue analysis procedure

The determination of fatigue life is of great importance in an assessment situation. A fatigue analysis procedure implies selection of a number of different parameters, where each of these parameters may have significant influence on calculated fatigue life of the considered structure. These parameters include:

- Environmental data (wave heights and associated wave periods and directionality).
- Wave theory.
- Marine growth.
- Hydrodynamic coefficients.
- Joint flexibility and structural modelling of members.
- Stress concentration factors as function of geometry and loading.
- Combination of stresses into a hot spot stress taking into account direction of loading and multiplanar joints.
- S-N curves.
- Corrosion protection.
- Palmgren Miner rule.

The largest uncertainties for fatigue analysis of offshore structures are associated with the parameters leading from environmental data to a hot spot stress, especially for joints in the splash zone area of jackets.

7.3 Assessment of details that can not be inspected

7.3.1 General

Items that can not be inspected for fatigue cracks and corrosion can be critical issues in an assessment of offshore steel structures for life extension.

Details that can not be inspected and that do not fulfil the original requirements to calculated accumulated fatigue damage including Design Fatigue Factors should be subjected to further assessments. Assessment for this is shown as flow sheet in Figure 3. Fatigue assessment for details defined to be without substantial consequences can be performed as described below. Reference is made to NORSOK N-001 for definition of “without substantial consequences”. Thus, a redundancy analysis may be required for this assessment.

1) Perform an assessment of the in-service history of the structure to check control of corrosion protection systems like potential readings including check of consumed anodes and condition of coating in and above the splash zone area. Based on status of this, assess likely condition of corrosion protection in areas that can not be inspected.

2) Perform a consistent fatigue analysis of ”as-is structure” and actions based on design standards of today. See also Annex A for explanation of consistent fatigue analysis. This may also imply use of refined finite element models of the actual hot spot stress areas.
   Note: For determination of hot spot stresses with element methods general guidance can be found in DNV-RP-C203 and for tubular joints in ISO 19902.

3) Check if calculated accumulated fatigue damage during service life is in accordance with NORSOK N-001. If not, perform measurements of stresses that are strongly correlated with the nominal stress ranges at the actual hot spot.

4) Perform a calibration of the analysis procedure.

5) Perform a consistent fatigue analysis based on calibrated action effect data.
   Note: See also Annex A.

6) The reliability of the considered detail can alternatively be controlled by in-service inspection of areas close to the considered hot spot. The calculated fatigue life of the considered non-inspected detail should be at least 3 times longer than the calculated fatigue life at the hot spot that is inspected.
   Note: Reference is made to /25/.

   Provided that fatigue cracks are not detected at the region with a short calculated life, it is also likely that the considered hot spot has sufficient fatigue capacity (due to correlation in stresses). It should be noted that the use of this methodology requires that analyses are performed accurately and that these analyses include the same degree of bias in assumptions and analysis basis (such as different conservatisms in calculated SCFs for the different hot spots). Assumptions and analysis performed should be documented.

If it is not possible to document sufficient safety by these measures, mitigation is required. Reference is made to Section 9.

7.3.2 Design fatigue factors for piles

The piles in jacket structures are an example where a reliable in-service inspection of welds is hardly realistic.

An assessment of uncertainties involved in calculation of stress ranges during pile driving has been performed, ref. /26/. It has been shown that the uncertainty in action effects based on driving records is lower than that estimated based on soil data. Thus, provided that reliable driving records from pile installation can be provided, where the number of blows and the hammer characteristics have been recorded, a DFF = 3 can be used for assessment for calculation of fatigue damage during driving. However, DFF for the pile for the in-service life shall be selected according to NORSOK N-001.

The total accumulated fatigue damage from in-service life should be added to that from pile driving for calculation of accumulated damage.

The acceptance criterion is that the total accumulated calculated damage is less than 1.0.

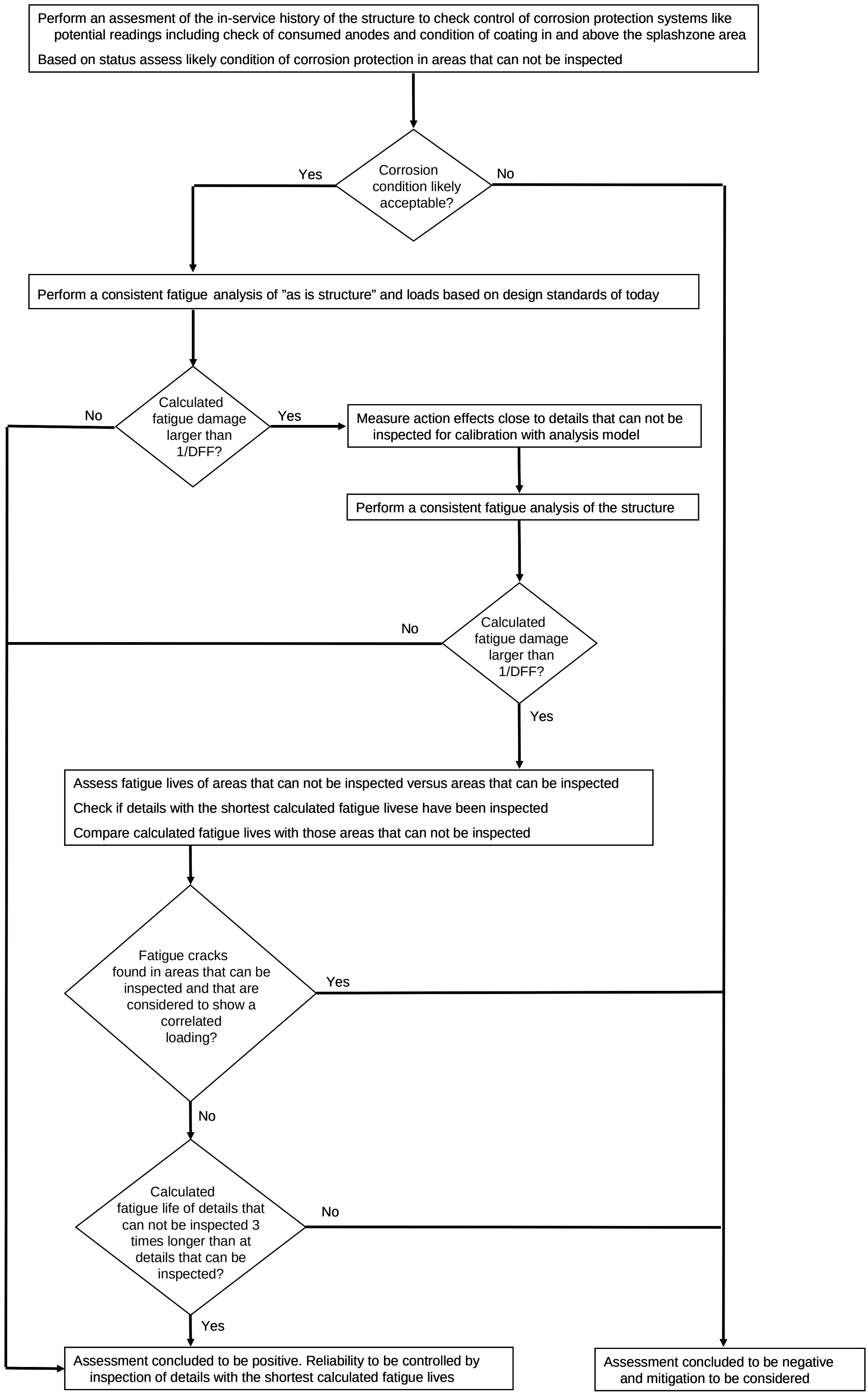

Figure 3 Assessment of details that can not be inspected for fatigue cracks. The procedure is applicable for details where failure will be without significant consequences.

7.4 Specific issues for fatigue assessment

7.4.1 General

The requirements for accurate fatigue assessments are even higher for assessment of a structure during its service life compared to the design stage. Therefore some specific issues related to fatigue analyses and fatigue assessment are given in this section.

Note: Guidance on fatigue assessment of concrete structures is given in ISO 19903

Note: Guidance on fatigue assessment of floating structures is presented in Chapter 10 of ISO 19904-1. Reference is also made to DNV-RP-C206 “Fatigue Methodology of Offshore Ships”. Reference is made to NORSOK N-001 for recommended Design Fatigue Factors.

7.4.2 Measurement of action effect

The fatigue load mechanisms for jacket structures are complex. Therefore, instrumentation of members in the structure may be performed for measurement of action effect to reduce uncertainties. This can be measurements of global action effects and also local action effects. The latter is considered to be the most challenging to analyse; as local action effects are sensitive to modelling of joint flexibility, actual length of members etc.

Measurements should be performed over an interval of at least a year to provide representative action effects. However, measurements of long term stress range distribution for one year are also directly of value for assessment of accumulated fatigue damage. The accuracy of action effect measurements is reliant on simultaneous measurements of directional metocean data.

If measured data are to be used for assessment purposes, redundant instrumentation shall be used.

7.4.3 Stress concentration factors for tubular joints

It is observed that the calculated fatigue lives are sensitive to how the tubular joints are defined in terms of geometry and load path dependence. It is therefore important that this is properly included in the computer program used for fatigue analysis.

7.4.4 Joint flexibility

The effect of including joint flexibility in the analysis can significantly reduce uncertainties on calculated fatigue lives at tubular joints especially for joints in horizontal frames. It is therefore important that this property can be used in an efficient manner for fatigue analysis.

7.5 Acceptance criteria for fatigue crack growth

The acceptance criteria for fatigue crack growth should be based on the actual connection considered. The assessment of crack size at fracture can be based on BS 7910. The fracture toughness for the base material may be used provided that it is likely that the fatigue crack tips grow into the base material. Then the fracture toughness may be derived from Charpy V values for the base material. The fracture toughness should be assessed using a relevant operational temperature for the considered connection.

Note: For simple tubular joints reference can also be made to NORSOK N-004 and ref. /35/.

The acceptance criteria for fatigue cracks are highly dependent on the type of connection considered and the maximum loading the connections can be subjected to. It is also dependent on structural redundancy. Also the following items are of significance for requirements to in-service inspection:

- An inspection method with sufficient probability to detect fatigue cracks should be used.
- The inspection intervals shall not be longer than that the cracks can be detected in due time before they grow to a critical size.

Note: Some examples are presented in the Annex A for illustration purpose.

7.6 Improvement methods

Improvement methods can be important for reassessment of some type of structures. For some types of connections there is a potential for significant improvement of fatigue life by use of these methods. The potential for improvement is largest for connections where a fatigue crack is growing from a weld toe into the base material where fatigue cracks are less likely to initiate from internal defects in welds. For connections like butt welds it can be difficult to achieve and to document significant improvement in life using improvement methods due to limitations in detecting defects in the weld by NDT.

Note: Use of improvement methods is discussed more in detail in Annex A. Reference is also made to DNV-RP-C203.
However, in special situations larger improvements can be achieved than that given in this recommended practice. Proper testing is recommended for documentation as the efficiency is dependent of type of detail considered.

Grinding of cracks can be performed to remove cracks that are up to 60 % of the plate thickness as long as the grinding is performed within a limited area and completely removes the crack. The fatigue life and the ultimate capacity after grinding should be documented by proper analyses.

The calculated fatigue damage at a welded hot spot may be reset to zero provided that proper grinding of weld toes and/or hammer peening is performed. However, it should be remembered that the fatigue damage for cracks growing from internal defects are not reset by improvement of the surface. Hammer peening may also be used once at any specific location to reset the fatigue life to zero provided there is no evidence of cracking.

A reliable system for quality assurance should be established for documentation of performed work when improvement methods are being used.

7.7 Mitigations for fatigue

The following mitigations for fatigue cracks may be considered if short fatigue lives are calculated:

- Reduce loading (e. g. remove members, remove inactive conductors, appurtenances, marine growth).
- Reduce stress level by strengthening (e. g. install new members, clamps)
- Reduce stress concentrations (e. g. internal grouting a tubular joint).
- Improve fatigue capacity by improvement methods.
- Perform controlled in-service inspections such that cracks are detected before they are through the wall thickness such that they can be removed by grind repair methodology.

If through thickness cracks are detected during inspection, other mitigations should be considered such as installation of bolted or bolted and grouted clamps.

8 Check of Ultimate- and Accidental Limit States

8.1 General

The same principles for check of ULS and ALS as for design of structures as given in NORSOK N-001, N-003 and N-004 apply to assessment of existing structures.

Effects of degradation of the structure such as corrosion, wear or damages from impacts need to be properly monitored and accounted for in the assessments. Resistance of damaged steel members can be calculated according to NORSOK N-004.

As it is considerably more expensive to increase the capacity for the existing structure than at the design stage, the demand for more accurate results can be higher when doing assessment of an existing facility. For this reason advanced analysis methods like non-linear analyses are foreseen to be more frequently used, and therefore some guidelines are given in this standard.

8.2 Action and material factors for assessment of existing structures

The action and material factors according to N-001 shall be used for structures that are assessed according to this standard.

Manned facilities that do not meet these criteria need to fulfil the requirements to unmanned facilities in NORSOK N-001 and to implement unmanning and shut down criteria in accordance with Section 6.3.

8.3 Requirements to assessment based on analysis with non-linear methods

When the ultimate structural capacity is assessed by use of non-linear methods, the analysis software and the methods used to represent the structure should be tested against known cases.

All relevant failure modes like yielding of cross-sections, local and global buckling, soil failure etc should be adequately represented in the analysis.

The parameters of the selected analyses model should be calibrated against the resistance determined according to the governing code.

The analyses may be carried out as quasi-static analyses (push-over) or as dynamic time-domain analyses.

When non-linear analyses methods are used to determine the ultimate strength of a structure the following shall be considered:

- The software used shall be documented and tested for the purpose.
- The user shall know the theory behind the methods applied as well as the features of the software.
- The stiffness of the structure should be modelled as the expected value (best estimate).
- All relevant failure modes such as tensile failure, buckling, instability or soil failure should be modelled. The resistance should be modelled as the characteristic values as per the governing code. In general that means 5% fractile in case a low resistance is unfavourable and 95% fractile in case a high resistance is unfavourable.
- The finite elements and the modelling techniques (element type, mesh, material parameters, imperfections etc.) should be calibrated against a known case (e.g. from the design code) in order to show that the failure mechanisms are adequately represented. The calibration should be documented. All factors that influence the resistance shall be addressed. Such factors are material non-linearity, imperfections, residual stresses, hydrostatic pressure etc.
- Care should be made to ascertain that all relevant failure modes are addressed either directly by the analysis or by additional checks.
- The analysis results should be reviewed to reveal if more unfavourable structural behaviour may exist than that provided by the selected modelling of the stiffness as well as the resistance of the various structural elements. In such cases the sensitivity of the structural resistance should be assessed.
- The loading sequence in the analyses should be selected such that the result represents the conditions to be analysed in a safe way.
- It is necessary to carry out a check of the cyclic storm resistance of the structure if the structure, when loaded by the ultimate load, is weakened against subsequent cycles. See 8.4.

8.4 Resistance to cyclic storm actions

8.4.1 General

Structures that are checked in ULS and ALS by use of linear analyses need normally not to be checked for cyclic failures during a storm. If the capacity is determined by non-linear methods, it shall be checked that the structure does not undergo deformations that can weaken its ability to resist subsequent load-cycles. Such changes may be due to plastic (permanent) deformations, redistribution of stress-resultants due to local buckling (cross-section) or member buckling, slip in friction grip joints with prestressed bolts, etc.

Further cyclic checks are usually not required in cases where the structural resistance is restricted to all of the following requirements:

- no structural components will experience local or global buckling determined according to N-004,
- tubular joints are not utilized above the capacity in N-004 (first crack limit),
- no plastic mechanism is formed,
- no part of the foundation has reached the ultimate soil capacity, and
- joints are, by inspection, proven to be free from fatigue cracks or the calculated fatigue loading is negligible.

The cyclic check of the dimensioning storm should be made on low probability characteristic actions and 5% fractile resistance according to NORSOK N-001. No design fatigue factor (DFF) should be applied when checking the cyclic storm actions.

The acceptance criterion for low cycle fatigue reads:

$$D_{LCF} \leq 1.0 - D_{HCF} \tag{1}$$

where
D_{LCF} = Accumulated damage from low cycle fatigue during the considered storm period using Palmgren – Miner accumulation rule

D_{HCF} = Accumulated damage from high cycle fatigue during service life using Palmgren – Miner accumulation rule

Design Fatigue Factors given in NORSOK N-001 shall be accounted for in the calculation of fatigue damage for high cycle fatigue.

8.4.2 Load history for check of storm cyclic resistance

All the remaining cycles in the storm of the maximum wave action may be assumed to come from the same direction as the dimensioning wave.

The load-history for the remaining waves in the storm may be assumed to have a maximum value equal to 0.93 of the dimensioning wave, a duration of 6 hours and a Weibull shape parameter of 2.0 .This applies for check of failure modes where the entire storm will be relevant, such as crack growth. When checking failure modes where only the remaining waves after the dimensioning wave e.g buckling need to be accounted for, a value of 0.9 of the dimensioning wave may be used.

Note: See comments included in Annex A.

8.4.3 Storm cyclic failure modes

The following possible failure modes that shall be assessed as part of the storm cyclic check:

1. Accumulated plastic deformation that can cause instability or tension failure.
2. Crack growth at details with strain concentration such as in tubular joints.
3. Crack growth at details that are deformed due to buckling.
4. Crack growth at butt welds at yield hinges. (An example can be circumferential welds made from one side at thickness transitions.)

5. Changed behaviour in bolted connections due to loss of pretension, slip or plastic deformation of contact areas.

6. Permanent deformations due to slip in grouted connections.

8.4.4 Check of storm cyclic resistance for steel frame structures

The check for the above failure modes should be performed in the following ways:

1. The accumulated plastic deformation can be assessed by analysing the structure for the equivalent load history given above. The structure should remain stable and all strain values should be within limits for tension failure.

   Note: Limits for tension failure are given in NORSOK N-004 Annex A.

2. Tubular joints that are analysed with assumption of appropriate joint and member flexibilities and are loaded within the code values for joint strength (first crack criterion) may be assumed to survive the dimensioning storm criteria defined above without further checks. Otherwise a low cyclic fatigue check shall be made as given in 8.4.5.

3. Members that may buckle either locally or globally during the first storm cycle or the subsequent waves shall be assessed for cyclic loading. If the buckled member is loaded in tension for the reverse load, the cyclic check should be carried out with the assumption that the member have failed. Buckled member that only obtain compression forces during the subsequent cycles can be assumed to maintain the capacity corresponding to the deformed shape of the member.

4. Cyclic check of bolted connections should be made by appropriate representation of the cyclic behaviour of the connections.

5. The cyclic check of grouted connections should include appropriate representation of the cyclic behaviour of the grouted connection.

8.4.5 Check of low cycle fatigue of joints of steel structures

8.4.5.1 General

Joints that are loaded by cyclic loads beyond their yield limit at stress concentrations should be checked for the danger of a crack growing in the storm to a size that will impact the load carrying capacity of the joint.

Note: A procedure for carrying out such checks is given in the Commentary in Annex A. The procedure in Annex A is used to develop a low cycle design curve for tubular joints which is given in 8.4.5.2, in order to ease the low cycle check of such joints.

8.4.5.2 Tubular joints

Low cycle fatigue checks for tubular joints encountered during a storm can be assed by carrying out a fatigue check based on the S-N-curve defined by Equation (2). The low cycle fatigue check may be made similar to ordinary fatigue checks as given in DNV-RP-C203.

$$\log N = \log \overline{a} - m \log \Delta\sigma \tag{2}$$

Values for $\log \overline{a}$ and m is given in Table 2.

Table 2 S-N data for low cycle fatigue analysis of tubular joints

Environment	m	$\log \overline{a}$
Air	5.834	19.405
Seawater with cathodic protection	4.927	16.084

The low cycle S-N-curve is valid up to 10^5 cycles where it coincides with the ordinary high cycle S-N curve. This is shown in Figure 4 for tubular joints in seawater with cathodic protection.

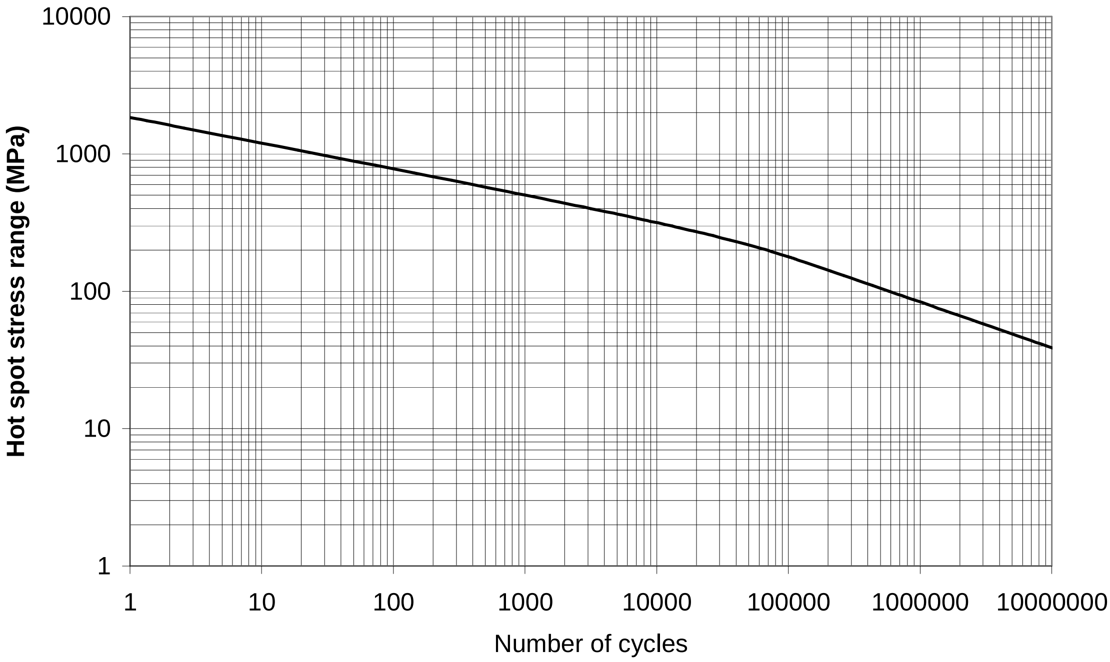

Figure 4 S-N curve for low cycle fatigue for tubular joint in seawater with cathodic protection

8.4.5.3 Other joints

Reference is made to the procedure in the Commentary section in Annex A for derivation of low cycle S-N data for other types of connections.

8.5 Wave-in-deck actions

In case the characteristic wave-crest reaches the underside of the deck, the wave-in-deck actions shall be considered. The actions may be found by use of computational fluid dynamics (CFD) or by simplified methods.

Note: Recommendations for CFD analyses can be found in DNV-RP-C205. Simplified methods can be found in ISO 19902 and in DNV-RP-C205.

When simplified methods are used, it is necessary to ascertain that the simplification of the wave actions and effects of the phase difference between vertical and horizontal forces are made to the safe side.

The dynamic effects caused by the rapid rise time for the wave-in-deck actions should be accounted for either by use of a dynamic amplification factor in case of a quasi-static analysis or by performance of a dynamic time domain analyses.

Simplified methods do not treat the effect on the water particles from the presence of the deck itself. When a wave hits the deck, the kinematics in the wave beneath the deck is strongly influenced by the deck itself. The increased fluid particle velocities should be accounted for when assessing the actions on members or equipment located in the zone closest to the underside of the deck in cases where the wave impact height is 1 m above the lowest deck level.

The wave-in-deck actions should be determined on a wave impact height based upon statistics of wave crest elevation.

8.6 Effect of corrosion and wear

Recommendations for capacity of corroded members are given in NORSOK N-004.

It should be ensured that the condition of the considered corroded structural element is sufficiently surveyed in order that the various failure modes can be properly addressed.

Structures that are not sufficiently protected against corrosion need to be assessed with their net thicknesses at the end of the assumed total design service life. The corrosion rate should be based on relevant experience and appropriate inspection plans need to be implemented.

Structural parts that can be subjected to abrasion from normal use or by accidents need to be inspected to determine the extent of the abrasion. Structural assessments should be made on the basis of forecasted values for the net sections of the structural parts.

8.7 Suggested mitigation possibilities

One or more of the following mitigations may be selected in case the assessment of ULS or ALS has failed:

- Reinforcement of the structure in the form of grouting of members to increase buckling capacity.
- Grouting of joints to increase joint capacity.
- Installation of additional braces.
- Reinforcement of steel structures by stiffeners, brackets etc.
- Reduction of wave actions by regular removal of marine growth or anti-fouling protection.
- Instrumentation of the structure to better calibrate the actions, responses, pore pressure etc.
- Use of material certificate or material testing in order to better estimate the structural resistance.
- Implement storm unmanning preparedness for the facility.

Other mitigations may be selected.

9 Requirements to in-service inspection after assessment

9.1 General

General requirements on in-service inspection planning are given in NORSOK N-005.

Assessment shall include additional considerations and requirements with respect to inspection and maintenance, taking into account the as is condition, planned modifications and if relevant the extended life, for:
- structures with respect to fatigue, corrosion, erosion and thickness measurement,
- critical areas of the structure,
- subsidence,
- scour,
- marine growth.

In-service inspection of structures in extended service life shall take into account that parts of the structure has passed, or will pass the design service life. Hence, inspection intervals shall be adjusted to take into account an increased likelihood of fatigue cracks as more fatigue damage is being accumulated.

Consideration of the likelihood of cracks should be based on calculated design life and inspection findings for the actual detail and similar details in the structure.

Details in structures with increased likelihood of fatigue cracks should be reclassified with respect to consequence of failure as the residual strength need to be assessed on the basis that more than one joint can fail. Reference is made to NORSOK N-004 for definition of classification of structural joints and components based on their substantial consequences and residual strength.

The probability of detecting potential fatigue cracks is a function of crack growth characteristics, reliability of the inspection method used and the time for inspection. The time interval for inspection shall be planned such that potential fatigue cracks can be detected with a large certainty before they grow so large that the integrity of the structure is endangered.

In assessments for life extensions a deterministic approach for in-service inspection for fatigue cracks can be used based on relevant cracks growth characteristics of the considered details. See 9.3.

Note: Different components may show different crack growth characteristics as shown in Annex A (Comm. 7.7).

Components where a failure can lead to substantial consequences and have passed their fatigue design life shall be inspected by an appropriate NDT method with an interval of maximum 5 years.

The extent and rate of corrosion shall be determined for components of primary structural integrity or importance for maritime integrity that have experienced significant corrosion. If there is less than 5 years left of corrosion allowance for such components, corrosion inspections are required at intervals not exceeding 2 years. For all other components, the extent and rate of corrosion shall be determined no later than once the corrosion allowance has been exhausted and a revised inspection plan or remedial action specified accordingly.

Alternatively, risk based inspection (RBI) can be used for planning of in-service inspection for fatigue cracks, corrosion and wear, provided these methods sufficiently take into account that increased probability of fatigue cracks and material loss are likely in extended life. Requirements related to use of probabilistic analysis for assessment of structural capacity are given in NORSOK N-001, Section 7.2.2.

Structures where it is likely that more than one connection may fail due to fatigue during one winter season a fatigue capacity check in damaged condition shall be performed as input to the inspection plan to ensure that the fatigue cracks are detected prior to accelerated fatigue cracking in the remaining structure due to redistribution of loads. The probability of having more than one connection failing due to fatigue cracking shall be assesed based on an overall consideration of: calculated fatigue life, results from performed inspections, experience from similar details in other structures, the number of connections with expected short fatigue lives and the structural configuration.

9.2 Risk based inspection

9.2.1 General

Risk based inspection (RBI) may be recommended for planning of in-service inspection for fatigue cracks. The basis for this methodology is described in the following. The Commentary section is referred to for more details where also some more detailed literature is listed.

In general large uncertainties are associated with fatigue analyses of offshore structures. A design of offshore structures with respect to fatigue is normally based on S-N data (fatigue test data). In-service inspections for fatigue are normally performed in order to assure that the fatigue cracks in the structure do not exceed a critical size.

The reliability of a non-destructive examination is described by the ability to detect an existing crack as a function of the crack size and by the uncertainty associated with the sizing of an identified crack. Regardless of the inspection outcome (detection or no detection), each inspection provides additional information to that available at the design stage, which can be utilised to update the estimated fatigue reliability.

9.2.2 Probability of detecting fatigue cracks

The time to the first inspection is based on the results from the S-N approach, while the inspection intervals are based on the fracture mechanics approach (conditional probability of failure given no-find in all previous inspections. However, if cracks are found, this may also be accounted for in a reliability analysis for planning of inspection intervals). This requires information about probability of detecting cracks. Probability of detection should be established as function of crack depth or crack length depending on method used and working condition such as

- Methodology used
- Above or below water
- Access to hot spot area

- Light condition at hot spot areas

Note: Some guidance on probability of detection curves for Eddy Current and Magnetic Particle Inspection can be found in Annex A of this document for controlled working conditions above water and for inspection under water.

9.2.3 Calibration of analysis model for fracture mechanics

For the S-N fatigue approach, the inspection results can not be used directly to update the estimated fatigue reliability, as no direct relationship between the crack size and the damage accumulation in the S-N approach is available. Therefore a fracture mechanics approach (FM) involving integration of the crack growth is used for this purpose. Due to the nature of the fatigue phenomena minor changes in basic assumptions can have significant influence on the calculated fatigue lives. Thus the calculated fatigue lives are highly dependent on a reliable assessment of the input parameters used in a deterministic approach. In order to achieve reliable results using a somewhat theoretical fracture mechanics model with a number of input parameters it is recommended to perform a calibration of the FM fatigue approach to that of fatigue test data (S-N data).

The resulting amount of required in-service inspection is dependent on how this calibration is performed. Due to uncertainties in the input parameters, probabilistic analyses are found attractive for assessment of reliability of fatigue failures of structures. Here a distribution of each of the input parameters can be used as input data to the analysis.

Probabilistic methods are found useful for planning in-service inspection for fatigue cracks taking the reliability of the inspection method into account. However, significant differences in calculated results can occur when performing probabilistic analyses due to differences in analysis models and/or distributions used for the variables as input. Thus, the difference in RBI analysis results provided by different analysts is likely to increase from that of deterministic fatigue analyses.

Note: Examples of planning inspection using this methodology are presented in the literature, ref. also Annex A. At present there is a lack of common basis for establishing in service inspection planning using probabilistic methods. The methodology presented here is meant as input to establish a more uniform basis for such analysis that better can assure a sound operational life regarding safety and economy.

9.2.4 Acceptance criteria

The acceptance criterion when planning in-service inspection for fatigue cracks based on RBI is depending on consequence of failure. Reference is also made to Section 7.1 for assessment of consequence of failure when the ageing process is considered to have effect on more than one connection.

The risk of a structural failure due to fatigue cracks should not be larger than risk for other failure modes. Probabilities of failures are normally presented on an annual basis. Fatigue is a gradual process where fatigue damage is accumulated. Thus, the annual probability of a fatigue failure will increase with time. The probability of a fatigue failure can then be expressed in terms of accumulated probability of fatigue failure during service life until the time considered. In order to calculate annual probability of failure one can calculate the difference in accumulated probability of failure within a time interval of one year and define this as an annual probability of failure.

An acceptance criterion for a connection with a large consequence of failure can be derived from a requirement that DFF = 10 is required in case that there is no access for in-service inspection and inspection is not possible. From this a target probability level is derived that also depends on distributions that are used in the probabilistic analyses. For a connection where the consequence of a fatigue failure is less i. e. corresponding to use DFF = 3, this value can similarly be used as basis for establishing target reliability level for such connections.

9.3 Effect of different crack growth characteristics on inspection interval

Different types of connections show different crack growth characteristics. The crack growth characteristics are important information with respect to planning inspection.

Note: This is described more in detail in Annex A (Comm.7.7 and comm. 9.6).

Note: For floating structures reference is made to Chapter 18 of ISO 19904-1.

10 Documentation of structural reassessment

The general requirements to documentation as given in NORSOK N-001 applies also for assessment of existing structures. In addition the following aspects should be documented if relevant:

- Reason for the assessment (assessment initiator).

- Basis for the condition assessment.
    - Performance history.
    - As-is condition.
    - Expected future development based on experience.

- Reference documents for the assessment including how the integrity of maritime systems and structures relates to regulations and standards..

- Assessment analyses and results

- Maintenance plans for ensuring sufficient integrity including how to monitor and identify degradation and ageing, and the necessary future mitigations as a result of such degradation.

- Description of necessary mitigations, including plan for replacement and need for future repairs of structures and maritime systems.

- Plans for how to ensure sufficient competence being in place to operate and maintain the facility.

Annex A

Commentary (informative)

This informative annex provides additional guidance and background to selected clauses of the normative standard.

Comm. 4.2
The list is based on ISO 19902.

Comm 6.3

The forecasted environmental conditions at which shut-down and unmanning shall be completed may be determined by the following procedure:

1) Determine the *maximum environmental action* (e.g. described with crest elevation (C_{lim})) the structure can resist according to the principles of ALS. If C_{lim} is lower than the crest elevation C_{2000} with a return period of 2000 years, proceed with steps 2-4. If C_{lim} for the structure is higher than C_{2000} the C_{lim} shall be set to C_{2000}.

2) Define the set of governing parameters that are provided by the forecast (e.g. H_s, sea level, wind speed). Assume H_s in the following.

3) By integration over all sea states below the significant wave height $H_{s,thr}$, determine $H_{s,thr}$ such that the probability that the annual maximum crest exceeds C_{lim}, is $5 \cdot 10^{-4}$

4) The forecasted environmental conditions at which shut-down and unmanning shall be completed, are for $H_s \geq H_{s,thr}$.

An example of how the shutdown and unmanning threshold expressed as sea-sate (H_s) can be established is shown in the following for Southern part of the North Sea (south of 58°N):

The forcasted seastate where unmanning should be effectuated may be determined by the following procedure:

First the maximum wave the structure can resist should be determined. This should be calculated according to the principles of ALS in NORSOK N-001 and N-003. The relation between wave crest C_{LAT}, wave height H_{max} and wave period T_{Hmax} to be used should be as follows:

For Southern Part of the North Sea (south of 58°N)

$$H_{\max} = 1.476\, C_{LAT} \quad \textbf{(3)}$$

$$T_{H\max} = 2.94\sqrt{H_{\max}} \quad \textbf{(4)}$$

With the capacity of the structure determined, the forecasted sea state where unmanning should be effectuated can be found from Figure 5.

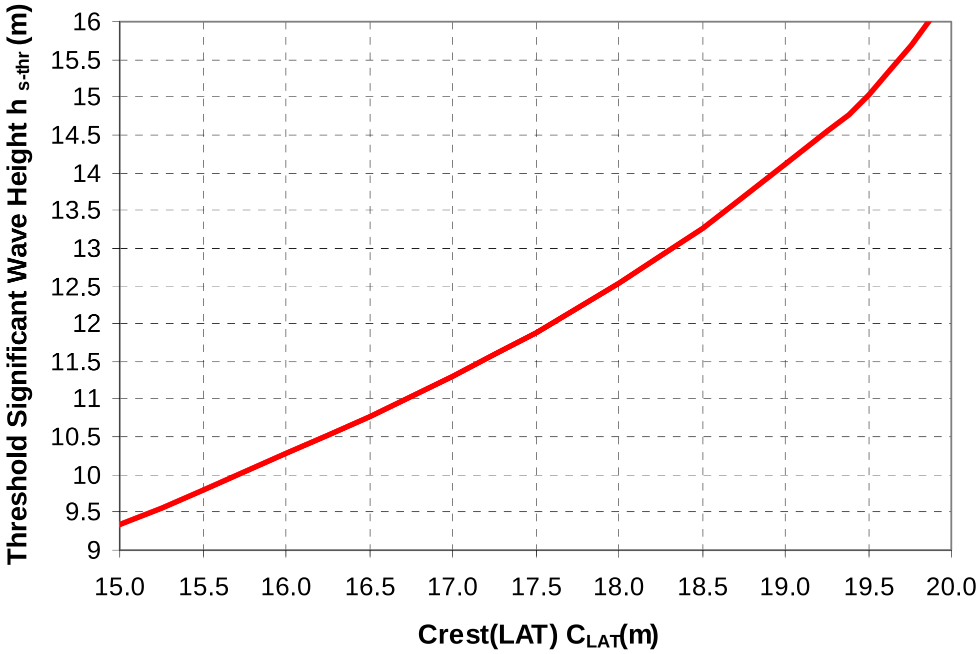

Figure 5 Limiting sea-state for unmanning (Southern part of the North Sea)

The given values assume that reliable weather forecasts are provided in which the inherent forecast uncertainty is accounted for. Alternatively the uncertainty and/or bias in weather forecasts should be accounted for by reducing the threshold H_s.

Unless other data is available the following safety margins to account for uncertainties in weather forecast may be used:

- 72 hour forcast 1.0 m reduction of H_s
- 48 hour forcast 0.8 m reduction of H_s
- 24 hour forcast 0.5 m reduction of H_s

Also the threshold H_s should be reduced if the forecasted sea level is exceptionally high.

Typical shutdown and unmanning frequencies (expected interval between shutdown/unmanning events) for the above example of the structural capacity thresholds is shown in Figure 6.

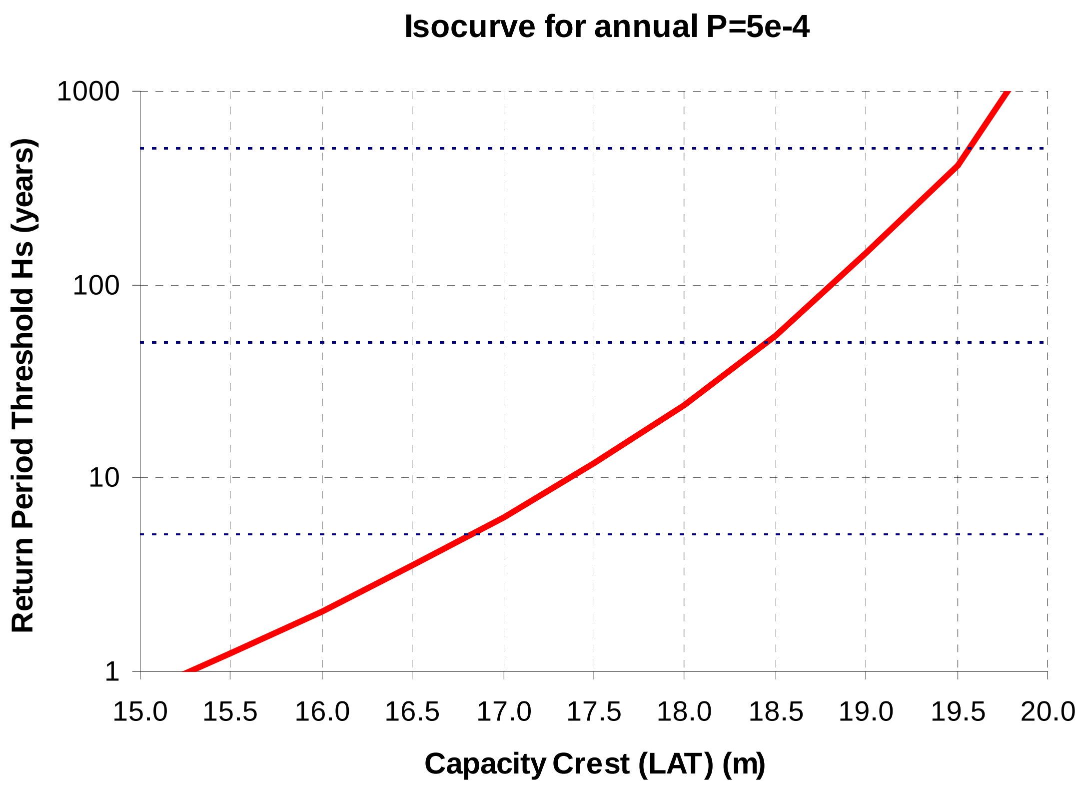

Figure 6 Return period for the sea–state triggering the unmanning criterion.

Comm 6.4
The following is a list of non-structural main safety functions which shall be secured for the duration of the storm (ref. PSA, SFT and NSHD Facility Regulation Section 6 and 10):

- Unmanning routes (i.e. helicopter deck, bridges or a sufficient number of lifeboats).
- Escalation prevention (i.e. loose equipment, walls, pipes, vessels etc. that can impair load bearing structure functions, release of significant volumes of hydrocarbon, or other accidents that can threaten personnel.
- Storm survival functions (e.g. control room, lighting, fire fighting equipment).
- Safe areas (i.e. personnel shall be relocated to areas not affected by extreme waves).

If this is not the case, the facility shall be evacuated based on the $1*10^{-4}$ threshold given in Figure 7. The given values assume that reliable weather forecasts are provided in which the inherent forecast uncertainty is accounted for. Alternatively the uncertainty and/or bias in weather forecasts should be accounted for by reducing the threshold H_s.

Also the threshold H_s should be reduced if the forecasted sea level is exceptionally high.

Using well-head facilities as an example, the combined probability of a structural failure due to wave actions that may potentially be damaging to the well integrity and leakage due to failure of the down-hole safety valves may be checked as follows:

1. Determine the probability $p_{f\text{-}dhv}$ that one or more of the down-hole safety valves fails.
2. Determine the probability level for a conditional characteristic wave as $p_{f\text{-}con} = 10^{-4} / p_{f\text{-}dhv}$. The characteristic wave should be determined as a directional wave with the actual probability using no more than 8 directions.
3. Check the capacity of the structure for this characteristic wave according to the principle of ALS.

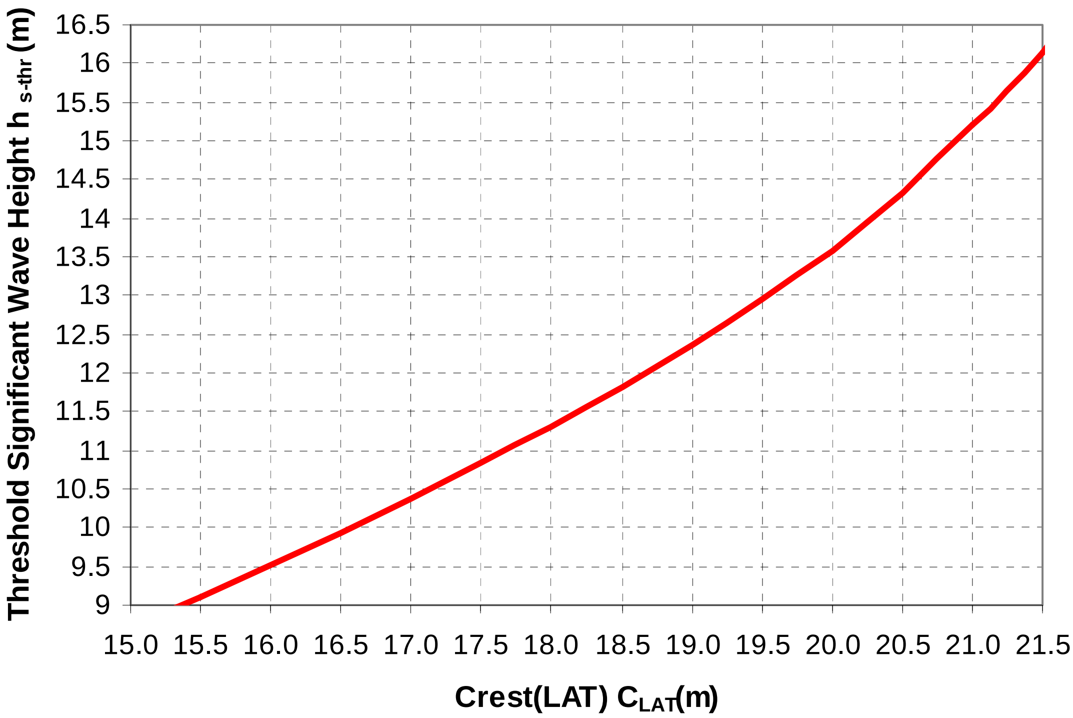

Figure 7 Limiting sea-state governed by main safety functions (Southern part of the North Sea)

Comm. 6.6

The concept of directional criteria should be used with caution. If directional information is used in a reliability analysis of a marine structure, it is important to ensure that the overall reliability is acceptable. For structural analysis, there is a long tradition to select the omni-directional wave for the climatologically worst direction. The wave criteria from other directions are selected based upon the wave statistics in the actual direction, selecting a somewhat conservative value with a longer return period.

The directional values to be used for structural analysis should be determined by due consideration of the actual structure, its load carrying ability when loaded in different directions, and the specific failure modes that are to be investigated. As the number of waves that can practically be analysed is limited, the selected waves need to cover different failure modes, which may imply that a value to the safe side will be selected in certain cases. In order to avoid that the annual failure probability exceeds the target safety level inherent in the NORSOK N-001 it is necessary to consider using a longer return period for the directional waves than the general requirement to characteristic actions in NORSOK N-001. The proposed directional factors are considered to give reliable structures when directional criteria are use.

Comm. 7.1 Consistency in analysis results

For a best possible effect of an in-service inspection of a joint, the selection of joint to be inspected should be based on a fatigue analysis that shows consistency in terms of analysed fatigue lives. By consistency is here understood that the calculated fatigue lives are considered to be based on a similar methodology for all the considered joints. This means that the calculated fatigue lives are considered to be associated with a similar bias relative to the actual fatigue life of the considered joint. A bias on fatigue life may be defined as a ratio between the calculated fatigue life and the actual life. An example of non-consistency could be a jacket with different types of joints such as cast joints analysed in detail with finite element analysis and stiffened tubular joints which probably might be assessed in a conservative manner.

In a design situation it is normal practice to make different assumption of analysis for different types of joints to document a fatigue life. Conservative simplifications are allowed as long as sufficient fatigue capacity can be demonstrated.

For planning of joints to be inspected by detailed NDT one would normally select a joint with a short calculated fatigue life. The consequence of a fatigue failure can also be part of such an assessment. The result might be that joint A in Figure 8 is selected for inspection as this joint shows the shortest calculated fatigue life. However, if a more refined fatigue analysis was performed, the calculated fatigue life may be moved to B, as an example. This means that this joint in reality has a long fatigue life; the probability of a fatigue crack is small, and one would more likely expect to find a crack at other joints shown in Figure 8. Thus, in order to learn as much as possible from an in-service inspection of a joint in a structure, the selection should be based on fatigue analysis that is made for this purpose and which are consistent as far as possible.

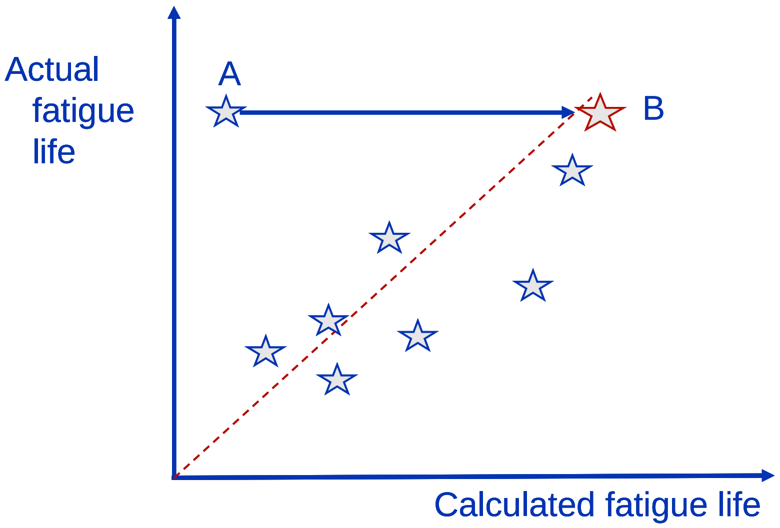

Figure 8 Example of calculated life versus actual life

Comm. 7.4.1 Full scale measurements of jackets reported in literature

General
Since the mid 1970's a number of measurement programs on jacket structures have been performed. Calculation of wave actions are associated with large uncertainties, e. g. uncertainties due to wave theory used, effect of marine growth/roughness, shielding, analysis methods, etc. Thus it is difficult to derive consistent conclusions from the measurements when they are compared with analyses.

The relation between the drag and the inertia coefficient depends on water velocity and thus on wave height and wave periods. For space frame structures with slender members the inertia term in Morison is important for small wave heights while the importance of the drag term increases with increasing wave height. Due to the total complexity of the loading with many parameters involved it is difficult for the researchers to get clear conclusions from measurements as compared with analysis data. It should also be added that the largest focus on results from measurements presented in literature has been on maximum forces which is relevant for the ultimate limit state. Here the drag term in Morison equation is most important for slender members. Less attention has been made to the fatigue limit state in this respect which is considered even more complex due to the combined action effects resulting both from the inertia and the drag term in Morison equation.

Some relevant references are considered more in detail in the following.

Forties Bravo platform
A major instrumentation project on the Forties Bravo Platform is described in ref. /22/. The hydrodynamic coefficients used in the analyses have not been stated in the paper. It is said that there are in general acceptable agreement between measured and calculated stresses (within 30 %). However, the measured overall displacements were about a factor 3 smaller than those predicted.

Valhall QP platform
The Valhall platform is a four leg jacket structure. It is bridge-connected to the neighbour platform and it is used for living quarter. The platform has no conductors or risers. It had two measuring devices. The platform was instrumented with 12 accelerometers and 16 strain gauges. Data were analysed for the period 1982-

1984. The analysed sea states varied from 5.5 m to 10.8 m significant wave height, refs. /23/ and /34/. It was found that the measured displacements were less than that calculated. Quoting from ref. /34/: "From the measured data non-Gaussian responses for the larger sea states were observed. The fatigue damage calculated on the basis of sample cycle distribution is higher than for the Rayleigh distribution for all sea states. The difference is, however, not significant compared to other sources of uncertainty in fatigue analysis, and it may for all practical purposes be neglected".

Fulmar A platform

The Fulmar A platform is an eight legged jacket placed in 82 metres of water depth in the Central North Sea. The geometry of the platform and member dimensions is comparable to the platforms in the Ekofisk area. The fatigue design was performed using a frequency response analysis with C_D = 0.6 and C_M = 2.0, ref. /17/. A marine growth thickness equal to 51 mm was used for the upper part of the jacket in the fatigue analysis. Eight members were instrumented by strain gauges and strain measurements were reported for a period of 2 years. It was found that the measured stresses were substantially lower than that predicted by the analysis.

Quoting from the paper: "Thus the original predicted life of 50 years for the hot spot location on the chord side of the weld would appear to be 500-2500 years based on the measured stress results".

The ratios of average stress ranges were presented by γ as ratio between the measured and the calculated stress. The following quotation from the paper is also made: "Overall the results for γ indicate that the predictions are very conservative and that the ordinates of the predicted stress transfer function would have to be reduced by between 40 % and 60 % to obtain agreement between fatigue damage values obtained from the measured and the predicted stress range distributions".

In the discussion of the reasons for the differences it is said that it is likely that the marine growth profile may have been less than that used in the analysis. However, marine growth could not alone explain the differences. Therefore, it is also concluded that the inertia coefficient used in the analysis is too high.

Magnus platform

The Magnus platform is a jacket structure in 182 metres water depth in the North Sea. The legs are made with large diameters. Thus the platform is considered to be dominated by the inertia term in Morison when fatigue loading is assessed.

Webb and Corr, ref./39/, concluded that C_M = 1.6 gives a conservative value for the loading when used together with drag coefficients of 0.8 / 0.65 when considering the overall forces on the platform. No further details are given on the load calculation procedure. (From this it is believed that C_D = 0.8 is used below water level and C_D = 0.65 above). From figure 21 in their paper it is observed that also C_M = 1.2 would provide conservative results. The platform had no conductors.

Ekofisk 2/4 A platform

During the winter season 1993-94 measurements on Ekofisk 2/4 A were performed in order to reduce uncertainties related to long term fatigue load calculation. Based on these measurements combined with analysis it was concluded that C_M =1.6 is applicable for force calculation together with C_D = 0.8 for global response analysis.

The following conclusions are copied from the calibration in 1999, ref. /20/.

Water particle velocity:

The water particle velocity was overestimated by 19% in average when a wave crest passes, using Stoke's 5th order wave theory. This observation corresponds well with the results reported by Bea, ref. /2/.
It was shown that this overestimation compensates the low drag coefficient used in design practice prior to the new wave load analysis recipe developed by API, and adopted by NORSOK and ISO.
When a wave trough passes, the calculated water particle velocity (using Stoke's 5th), corresponds well with the measurements.

Hydrodynamic coefficients:

Based on global force calibration the following hydrodynamic coefficients were derived based on marine growth equal 20 mm:
C_D = 0.8
C_M = 1.6

For fatigue analysis it was recommended to increase the factors by 7 % in order to account for anodes.
It is noted that the marine growth profile used in this calibration is rather low as compared with some measurements of marine growth performed during the eighties. A larger marine growth thickness would imply even smaller values for the hydrodynamic coefficients.

Kvitebjørn jacket
The Kvitebjørn jacket is a slender steel structure installed in 190 m water depth, ref. /21/. The measured response of the Kvitebjørn jacket is compared with predicted response. The overall conclusion is that the experienced response is less than that used for design. During design the largest natural period was estimated to be around 5 secs. Platform measurements have shown that the largest natural period is significantly lower and a period of 4 secs has been estimated. This can partly be explained by a less deck mass present during measurements than that assumed for the design analysis and a somewhat stiffer foundation than that assumed in design due to uncertainty related to scour.

Instrumentation of members for measurement of load effect
Instrumentation of members may be performed for measurement of load effect to reduce uncertainties. This can be measurements of global load effects and also local load effects.
Measurements should be performed over an interval of at least a year to provide representative load effects. The measurements should also include measurements of the environment. However, if the situation is constant over time with respect to expected long term hot spot stress ranges one may directly use the measured stress range for assessment of fatigue damage during life as shown in Equation (5).

$$D_{tot} = D_{accumulated,1\,year} T_{Life} \tag{5}$$

Comm. 7.4.2 Stress concentration factors and derivation of hot spot stress for space frame structures

Jacket structures
The stress concentration factors by Efthymiou (ref. /15/) are the basis for all hot spot stress calculation in ISO 19902 and also DNV-RP-C203. The stress concentration factor is based on geometry; but it should also be based on a proper force flow through the joint. Otherwise there will not be correspondence between the hot spot stress calculated as basis for the equations for stress concentration factors and the actual stress in a real structure.

For example, so called “balanced forces” (equilibrium of axial forces in the two braces normal to the chord) were used to derive stress concentration factors in a K-joint. If the forces on such a joint were acting in the same direction, the actual physical behaviour would be more that of a Y-joint.
There are different ways to include this in computer programs. Efthymiou in his paper from 1988 has proposed some different methods for this. The same methods are also described by SESAM. However, when making an actual analysis program there are possibilities for introduction of differences during program developments from the same code basis, especially when considering load path dependent SCFs.

The following definitions are used in SESAM:

Geometry:
The SCFs are determined based on geometry only.
Thus if the geometry of the joint is that of e. g. a K-joint, the Efthymiou's SCF equations for a K-joint is used independent of load path behaviour.

Loadpath dependent SCFs:

The SCFs are determined from instantaneous load path in joint.
Taking the load path behaviour into account may change the considered joint into some percentage of other joint type behaviour as in Methods C, B and A; see definitions below.

Method C:
This method is denoted “the conventional approach”. This method involves the following assumptions:
- Axial load in K, KT & X joints is assumed to be balanced.
- Out-of-plane bending in K &KT joints is assumed to be unbalanced.
- Out-of-plane bending in X joints is assumed to be balanced.

- In-plane bending in K joints is assumed to be unbalanced.

Method B:
This method is called influence function method. It is based on use of Efthymiou's equations for stress concentration factors. In principle this is a way to include SCFs that are properly linked to the load path in the considered joint. Method B does not include braces in other planes than that considered.

Method A:
Method A is similar to Method B except that Method A also includes multiplanar effects; i. e. braces in other planes are also included when the load path is calculated.

Based on some performed analysis it is observed that Method C can be conservative as well as non-conservative versus Method A. The difference using the two methods is typical within a factor 2 on fatigue life. Reference is also made to Efthymiou (ref. /15/).

Relevant theory for stochastic fatigue analysis has not been properly developed to incorporate Methods A and B. Therefore, Method C is being used in stochastic fatigue analysis.

Comm. 7.4.3 Effect of joint flexibility
The effect of including joint flexibility can significantly improve the calculated fatigue lives at tubular joints as shown for joints in a horizontal frame in Figure 9. (The joint flexibilities are calculated based on Buitrago et al, ref. /4/). It is also observed that for some joints it can also be non-conservative to not include the joint flexibility in the fatigue analysis.

It is therefore important that this property can be used in an efficient manner for fatigue analysis.

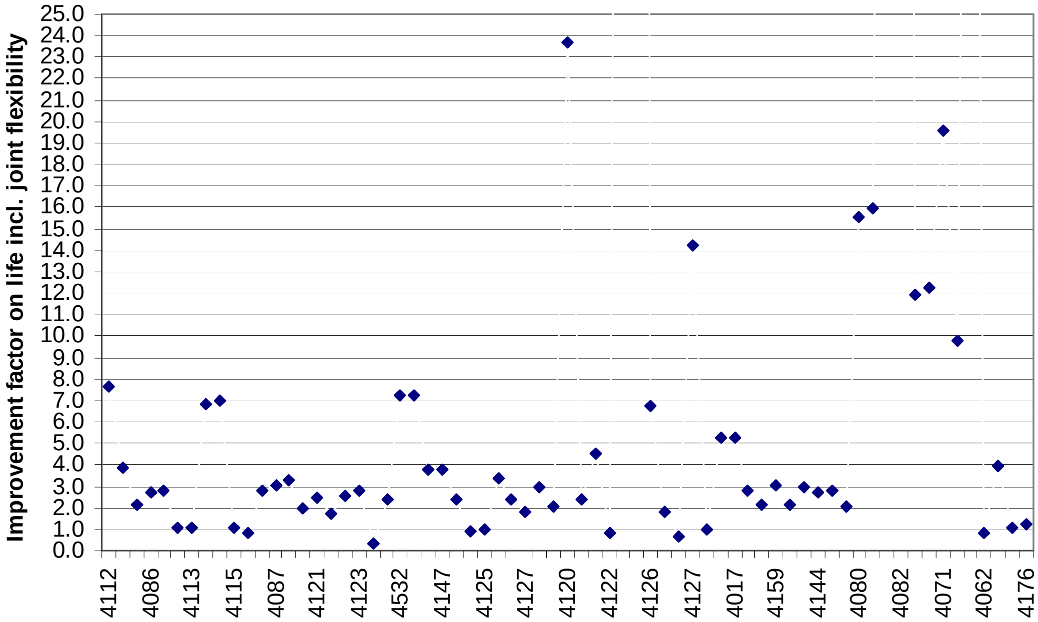

Figure 9 Factor on calculated fatigue life when including joint flexibility at horizontal elevation approximately 20 meters below water level in a jacket

Comm. 7.5 Requirements to in-service inspection with regard to detection of fatigue cracks

Background
The effect of in-service inspection is very much dependent on the crack growth characteristics. This is schematically illustrated by the following graphs derived from fracture mechanics analyses.

<u>Analysis methodology</u>
For simplicity of analysis it is assumed that the defect at the hot spot is going through the plate such that crack growth can be integrated in one dimension. An initial crack size equal 20 mm is assumed (= 2 a_i as an internal crack growing in two directions is assumed).

The fatigue life is calculated based on the following crack growth equation from BS 7910:

$$\int_{a_i}^{a_f} \frac{da}{\left(Y\sqrt{\pi a} - \frac{\Delta K_{th}}{\Delta\sigma}\right)^m} = C(\Delta\sigma)^m N \qquad \textbf{(6)}$$

where
a_i = initial crack size
a_f = final crack size
m and C are material parameters
K_{th} = threshold stress intensity factor
$\Delta\sigma$ = nominal stress range in member outside area with defects
N = number of stress cycles
The following material parameters from BS 7910 are used for the analyses:
C = 5.21*10^{-13} (Units: N and mm) (Data representative for mean plus 2 standard deviations)
m = 3.0
Threshold stress intensity factor: K_{th} = 63 $Nmm^{-3/2}$.
Y = geometry function.
The stress intensity for a through thickness crack can be calculated as

$$K_I = \sigma_\infty \sqrt{\pi a} \qquad \textbf{(7)}$$

where
σ_∞ = nominal stress in a region outside the considered area
a = half crack length (internal crack)

<u>Different types of joints and possibility for redistribution of stress during crack growth</u>
The crack growth development in welded joints can be very different in different types of joints even if the hot spot stresses are equal. The calculated fatigue life is normally derived based on S-N curves where the failure criterion is crack growth through the thickness. Some connections may show significant crack growth life also after a crack has grown through the thickness at the hot spot. This depends on type of welded connection that is considered. The crack growth characteristics are very much dependent on possibility for redistribution of stress during crack growth. Part of the redistribution of stress can also be explained by reduced crack driving stress as the crack is growing out of the hot spot region. This is the situation for a hot spot with significant stress concentration factor such that the nominal stress is significantly lower than at the hot spot stress initiating the crack. Simple tubular joints without stiffeners are example of such connections. Girth welds in tethers of tension leg platforms are an example of a connection where the hot spot stress is close to that of the nominal stress. The fatigue crack leading to failure of the Alexander Kielland platform is another example of this /36/.

Three cases with the same hot spot stress, but with very different stress distribution during crack growth is considered in the following:

- Typical defect as in the Alexander Kielland platform where the stress is of a nominal type during crack growth as illustrated in Figure 10.
- Defect in tubular joint with SCF = 2.0, where the stress is reduced as the crack grows out of the hot spot region, ref. Figure 10.
- Defect in tubular joint with SCF = 3.0, where the stress is reduced as the crack grows out of the hot spot region, ref. Figure 10.

It should be noted that the assumptions made here are simplified and mainly made for illustration purpose.

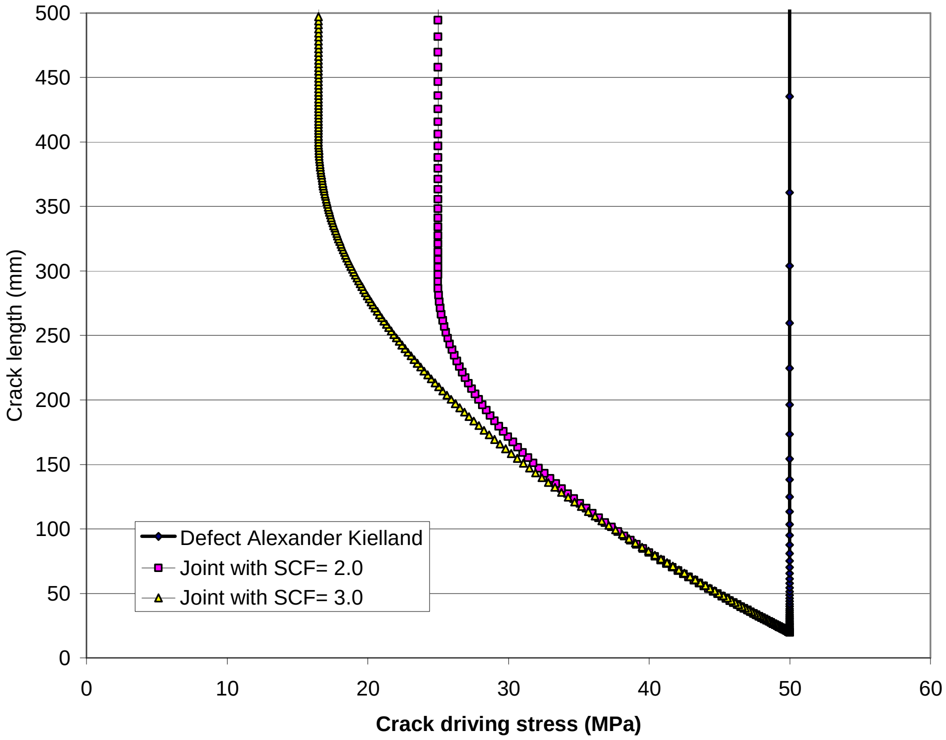

Figure 10 Crack driving stress as function of crack length (for illustration)

<u>Calculated crack growth characteristics</u>
The range of stress intensity is a function of the crack driving stress range and the size of the crack. (The geometry function Y is here for simplicity put equal 1.0).

The ranges of the stress intensity for the different types of joints are shown in Figure 11.

The increments in crack growth for the different types of joints are shown in Figure 12. It is seen that the calculated increments are very much dependent on type of joint considered.

The integrated crack growth for the different types of joints is shown in Figure 13. Even if the crack growth may be similar in the first stage with crack growth at the hot spot, it is observed that the crack growth characteristics thereafter may be significantly different.

The difference in crack growth characteristics may have significant consequences for time interval for a detectable crack size until the crack grows into an unstable fracture as shown in Figure 14. A short interval is observed for an Alexander Kielland type connection while the interval becomes significantly longer in joints with possibility for redistribution of stress.

The time to failure is dependent on acceptance criterion for unstable fracture due to external tension loading that should be assessed in each situation.

Depending on acceptance criterion one may find that inspection can be very efficient for joints showing possibility for redistribution of stress as compared with a situation without redistribution possibility (type Alexander Kielland).

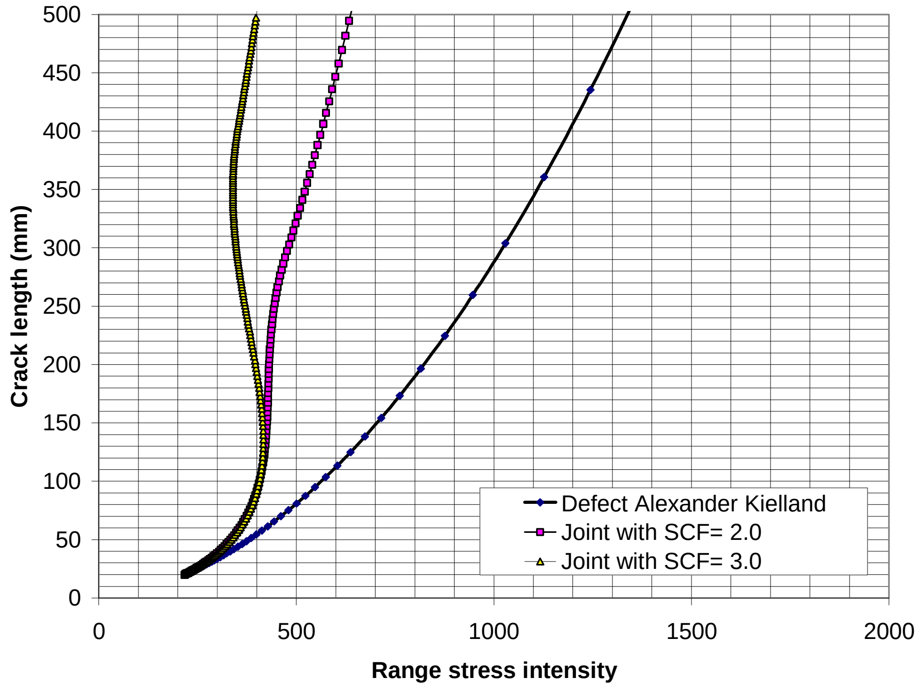

Figure 11 Range of stress intensity as function of crack length

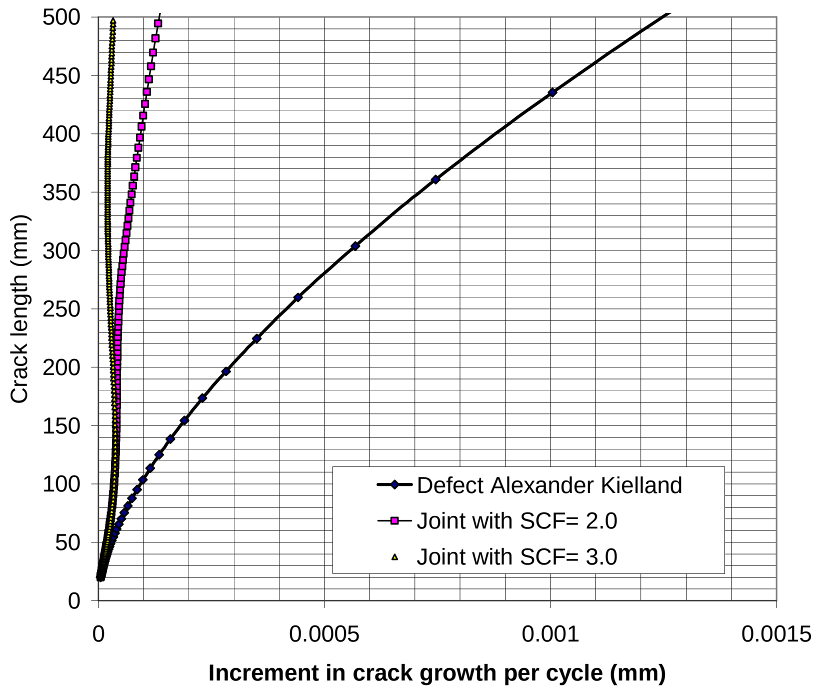

Figure 12 Increment in crack growth for different types of joint and crack length

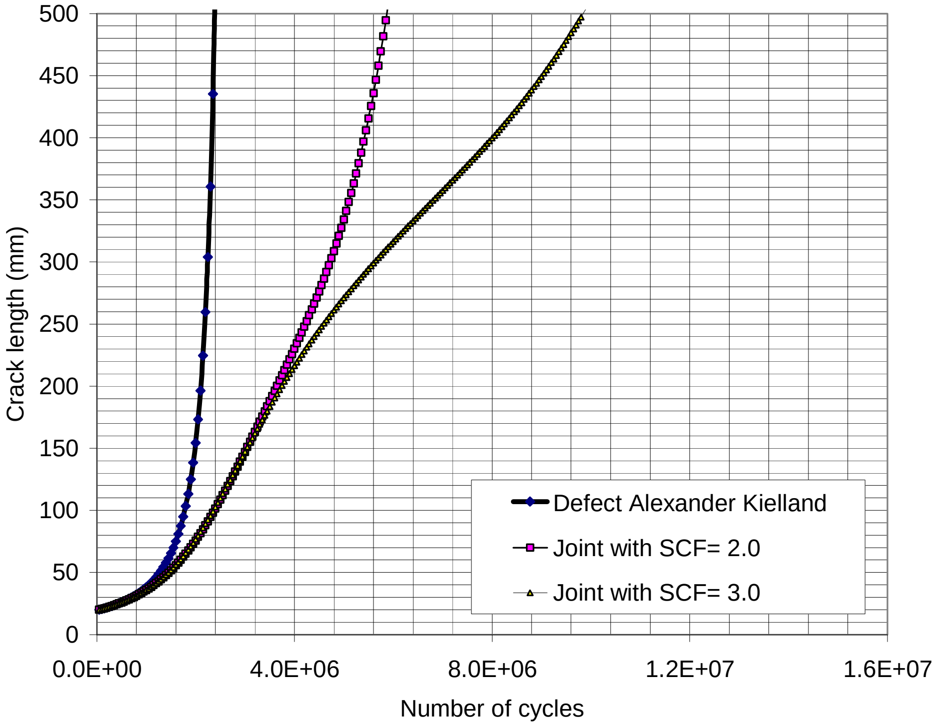

Figure 13 Crack growths as function of number of cycles

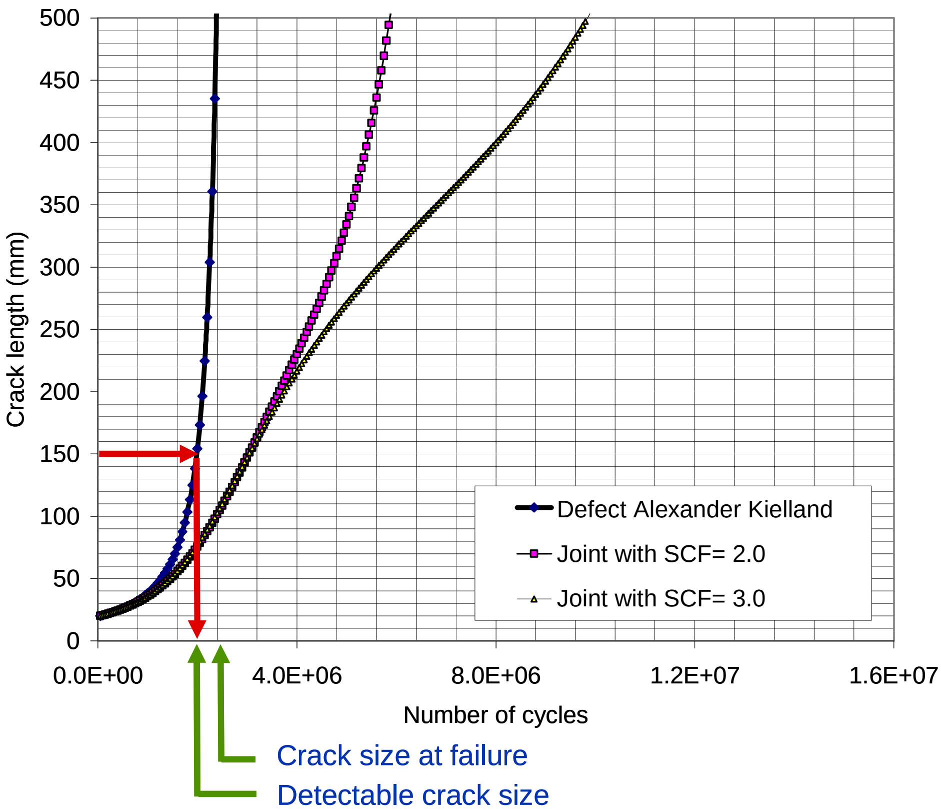

Figure 14 Illustration of inspection interval from detectable crack size to failure

<u>Acceptance criteria for crack growth in tubular joints</u>

The acceptance criteria for crack growth will very much depend on tension loading that need to be transferred through the connection that includes the crack. This depends very much on the actual structure.

For an assessment of tension loading from environment where the wave do not hit the deck it is considered to be conservative to use the 100 year loading as basis. A lower loading might be considered if the probability of such a loading is considered in combination with probability of presence of a fatigue crack.
From acceptance criteria in Ultiguide, ref. /37/, rather large cracks may be accepted for these loads.
A wave loading on deck may imply large tension loads and if the load effect implies significant yielding, only small fatigue cracks can be accepted.

Thus, the combination of probability of fatigue cracks, inspection frequency and maximum loading should be considered for assessment.

Resulting acceptance criteria are illustrated schematically in Figure 15 for different type of joints and different types of maximum loadings. (For purpose of a simple calculation it is assumed that the circumference of the tubular is large as compared with the calculated crack length). It is assumed that the fatigue cracks are growing into the base material. It is further assumed that the fracture toughness is derived from knowledge of Charpy V notch testing only as CTOD values for thicknesses less than 50 mm were not established for the considered connection. BS 7910:1999 is used for assessment of fracture toughness values (K_{Ic}) from Charpy V values. It is also assumed that the temperature in the seawater is around 0^{o} C. A K_{Ic} value equal 3500 $Nmm^{-3/2}$ is used to establish the crack sizes at failure shown in Figure 15.

It is observed that only very small cracks can be accepted if members are grouted to achieve increased resistance under tension loading (using a von Mises yield criterion).

From this figure it is also observed that rather long fatigue cracks might be accepted for other loading conditions.

Thus, the acceptance criteria for fatigue cracks and requirement to in-service inspection have to be seen in relation to what is required from the connection at an ultimate capacity assessment.

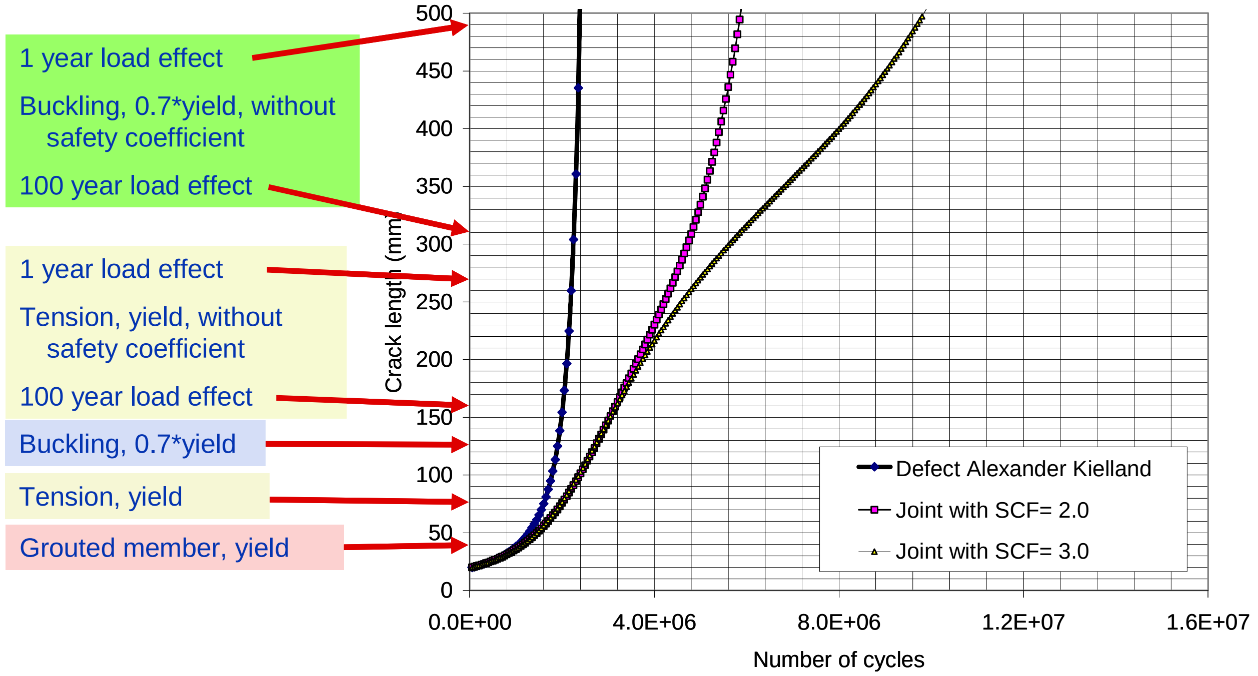

Figure 15 Illustration of acceptance criteria depending on assumed load effect

Comm. 7.6 Fatigue life improvement

<u>Methods</u>
Fatigue lives may be improved by different methods such as grinding and peening of the weld toes. The expertise and confidence in peening methods has grown in recent years. There are different peening methods such as

- Hammer peening
- Needle peening
- Ultrasonic peening

Grinding of fatigue cracks can also be performed as a permanent repair. This repair methodology has been documented to be efficient for simple tubular joints in a joint industry project performed by DNV. Reference is made to refs. /5/ - /12/. Example of a grind repair geometry is shown in Figure 16.

Effect of improvement
The effect of improvement on fatigue life can be significant. However, this depends on welded detail that is considered as indicated in Figure 17. (The S-N curves in Figure 17 are derived using an F-curve as basis and then increasing the fatigue strength by an improvement factor on stress at 2 mill cycles and rotating the S-N curves about 10000 cycles). Normally it may be considered difficult to achieve significant improvement of a fillet welded connection denoted 1 in Figure 17. The reason for this is crack growth from the weld root as indicated in Figure 18. However, there are also fillet welded connections where weld toe improvement can lead to significant increase in fatigue life such as detail denoted 3 in Figure 17. This illustrates a ship side plate welded to a longitudinal stiffener. The plate is subjected to transverse side pressure giving bending stress at the weld toe.

In general the most reliable improvement can be obtained for full penetration welds. However, if the weld toe is improved by grinding, this region becomes a less likely initiation point for fatigue cracks. Then it becomes important that crack initiation will not occur from internal defects as indicated by detail number 2 in Figure 17. Therefore, control of defects by NDT is important in order to achieve a sound fatigue life for these details, ref. /40/.

It has been shown that a significant amount of fatigue life improvement of tubular joints can be made even if a significant amount of material if removed by grinding.

It should also be mentioned that peening may close defects/cracks that are 2 mm deep. This may be useful in areas of undercuts where there might be uncertainty about small defect that may be left undetected after an inspection by eddy current (EC) or magnetic particle inspection (MPI).

Experience
Needle peening was used successfully on the Veslefrikk semisubmersible in 1999. Fatigue cracks have not been observed in areas that were peened. However, a number of cracks have been observed outside these areas. This shows that careful attention should be given to areas that need improvement during life extension assessment, ref. also Chapter 7 in DNV-RP-C203.

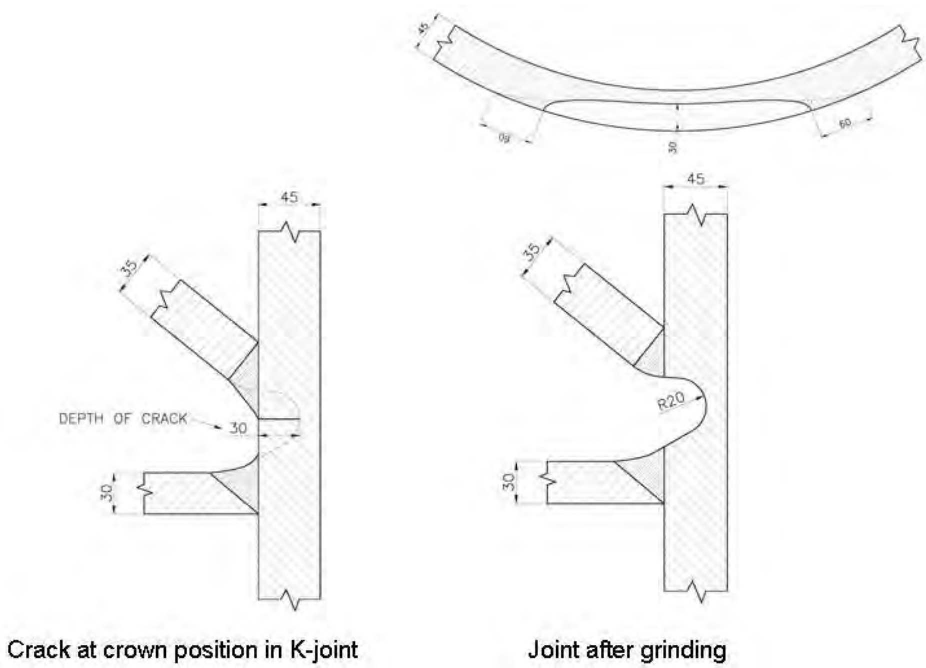

Figure 16 Example of grind repair of deep fatigue crack in tubular K-joint

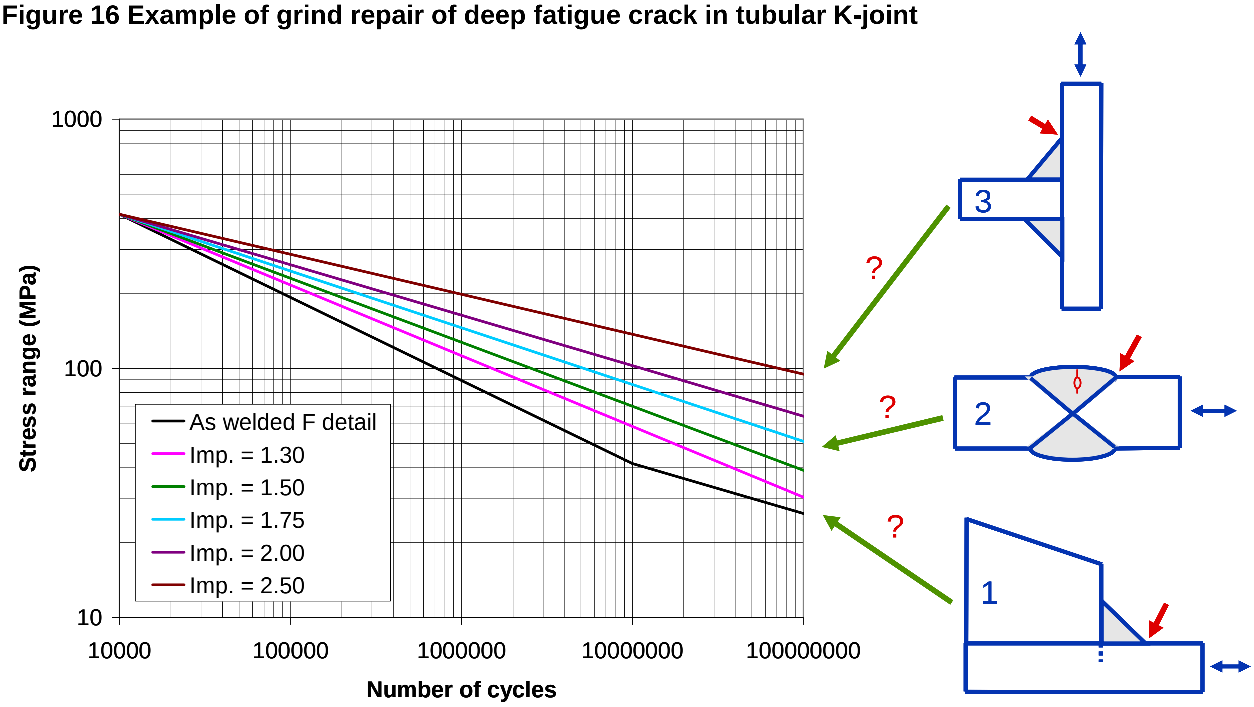

Figure 17 Sketch illustrating effect of improvement of welds

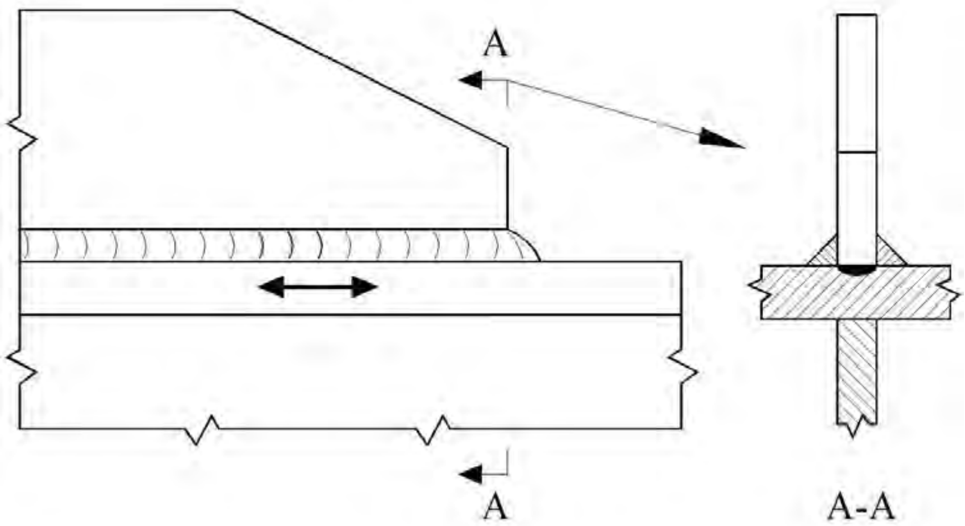

Figure 18 Illustration of fatigue crack growing into the member below the weld root (ref. detail 1)

Comm 8.3
Guidence on how to use non-linear methods for check of offshore structures can be found in Ultiguide /37/ and in /33/.

Comm 8.4.2
The 2nd highest wave in 10.000 year is equal to the 5.000 year value. For the Ekofisk field the 10.000 year wave height is $H_{10.000}$=32.96m, and the 5.000 year wave height is 0.964 $H_{10.000}$. However, the 10.000 year wave height and the 5.000 year wave height are estimated by integrating over all storms, whereas the second highest wave applicable for cyclic loads should be selected from one storm (the storm where the 10.000 year wave height occurs). Thus the expected 2nd highest wave in the storm where $H_{10.000}$ occurs is lower than the 2nd highest wave during 10.000 years (the 5.000 year wave).

The statistics of the second highest wave in a typical storm for a given location can be established from the history of storms from the location. For Ekofisk, the statistics for the 2nd highest wave is documented in /13/. From the report the median value of the second highest wave in the storm is 0.96H_{max}. This is actually quite close to the ratio reported above. The median value of the second highest wave after the maximum wave height is 0.93H_{max}.

It can be argued that the statistics for the 2nd highest wave for the storm where the 10.000 year occurs would be different from what is observed in the 26 years Ekofisk Reference Data Set. By modelling the extreme storm by a rectangular storm profile, simple order statistics for stationary sea states can be applied.

The wave heights are assumed Weibull distributed,

$$F(z) = 1 - \exp\left[-z^{\beta}\right] \quad \textbf{(8)}$$

where $z = \dfrac{h}{\alpha h_s}$.

For the Forristall wave height distribution, $\alpha = 0.681m$ and $\beta = 2.126$.

For the Ekofisk field, $H_{s,10000}$ = 17.71m, T_z = 13.55s and $H_{10.000}$ = 32.96m. For the rectangular storm of duration τ hours, the statistical results shown in Table 3 for the maximum wave height H_{max} in the storm are obtained.

Table 3 Statistics for the maximum wave in a rectangular storm

Maximum Wave H_{max} in Storm	$\tau = 6\,hours$	$\tau = 3\,hours$
Characteristic largest H_{max}; $H_{ch,max}$ / $H_{10.000}$	0.937	0.894
Expected H_{max} ; $E(H_{max})$/ $H_{10.000}$	0.967	0.927
Median(H_{max})/ $H_{10.000}$	0.958	0.917
Probability H_{max}< $H_{10.000}$	0.718	0.847

It can be observed that the 10.000 year wave $H_{10.000}$ corresponds to the 72% fractile for the distribution of the maximum wave in a 10.000-year rectangular storm of duration 6 hours.

For the 2nd highest wave H_2 from a sample of n waves, assuming $H = h_1$ for the highest wave, the distribution function is as follows:

$$F_{H_2|H_1}(z_2 \mid z_1) = \left(\frac{1-e^{-z_2^{\beta}}}{1-e^{-z_1^{\beta}}}\right)^{n-1} \quad \text{where} \qquad z_i = \frac{h_i}{\alpha h_s} \qquad i = 1,2 \tag{9}$$

This corresponds to the highest wave in a sample of n-1 waves, truncated at h_1.

For the second highest wave in the design storm, assuming that the maximum wave is equal to the characteristic largest wave $H_{ch,max}$ in the 6 hour storm, the results are shown in Table 4 and Figure 19.

For a storm duration of 6 hours, the mean value for the 2nd highest wave height in the storm is $0.96H_{ch,max}$. For the 2nd highest wave height in the storm after the maximum wave height, the results for the 3 hour storm are applicable for the 6 hours design storm. Thus for a storm duration of 6 hours, the mean and median values for the 2nd highest wave height after the maximum wave height, are $0.94H_{ch,max}$. These results correspond well with the results reported above for the measured Ekofisk data.

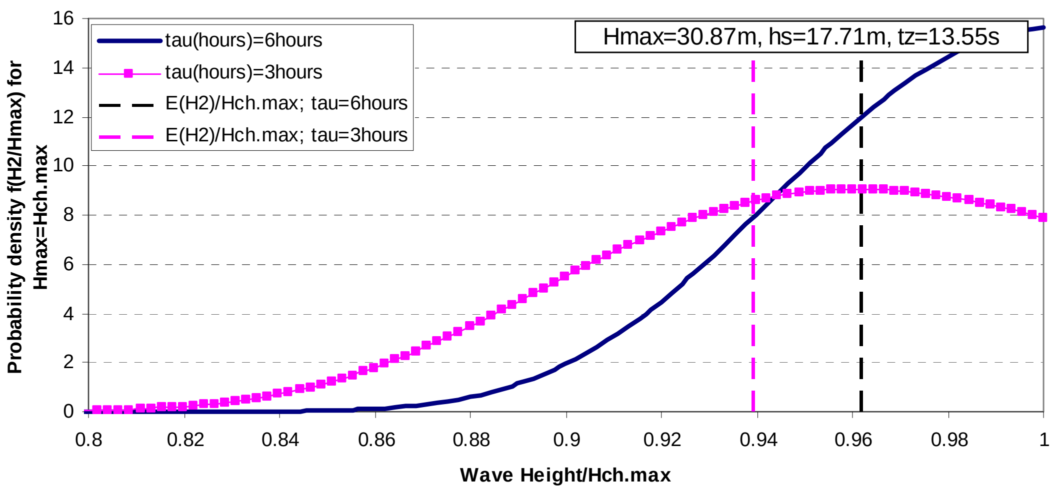

Figure 19: Probability density function $f(h_2/H_{ch,max})$ for the second highest wave in the design storm assuming H_{max}= $H_{ch,max}$ = characteristic largest maximum

Table 4: Statistics for the 2nd highest wave in the design storm, assuming H_{max} = $H_{ch,max}$

2nd highest wave in Storm	$\tau = 6\,hours$	$\tau = 3\,hours$
Expected value $E(H_2 \mid H_{ch,max})/H_{ch,max}$	0.962	0.939
Median value $(H_2 \mid H_{ch,max})/H_{ch,max}$	0.966	0.943

For the second highest wave in the design storm, assuming that the maximum wave is equal to the 10.000 year wave, the results are shown in Table 5 and Figure 20.

For a storm duration of 6 hours, the mean value for the 2nd highest wave height in the storm is $0.93H_{10000}$. For the 2nd highest wave height in the storm after the maximum wave height, the results for the 3 hour storm are applicable for the 6 hours design storm. Thus for a storm duration of 6 hours, the mean and median values for the 2nd highest wave height after the maximum wave height, are $0.90H_{10000}$.

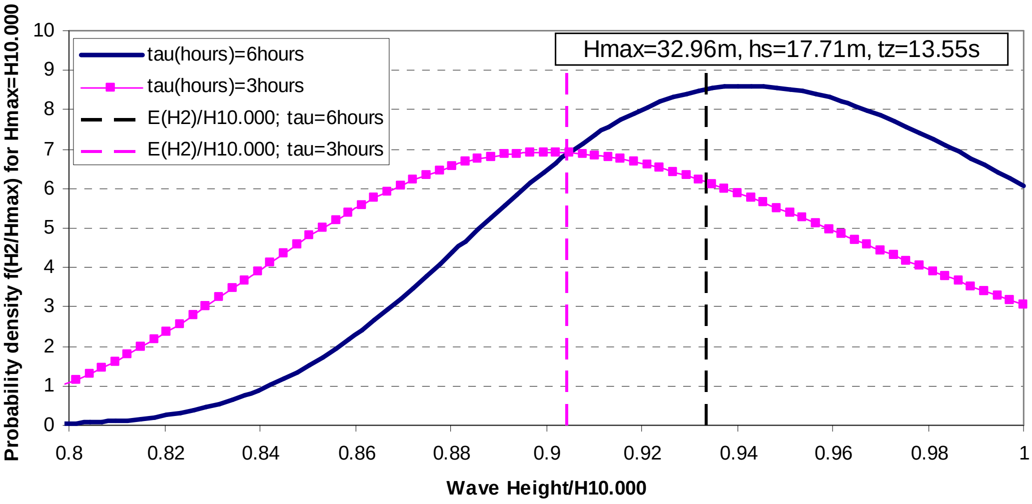

Figure 20: Probability density function $f(h_2/H_{10\,000})$ for the second highest wave in the design storm, assuming $H_{max} = H_{10\,000}$

Table 5: Statistics for the 2nd highest wave in the design storm

2nd highest wave in Storm	$\tau = 6\,hours$	$\tau = 3\,hours$
$E(H_2 \mid H_{10.000}) / H_{10.000}$	0.934	0.904
$Median(H_2 \mid H_{10.000}) / H_{10.000}$	0.936	0.903
$F_{H_{\max}}(E(H_2 \mid H_{\max} = H_{10000}))$	0.340	0.172

In the order statistics applied above it is assumed that the wave heights are independent random outcomes of Weibull distributed wave heights. Thus the 10.000 year wave height and the 2nd highest wave height in the storm giving the 10.000 year wave are random outcomes in the 10.000 year design storm. It is not explicitly accounted for correlation effects and the possibility that due to some physical mechanisms the storm might be prone to give several exceptionally severe wave heights.

Comm. 8.4.3
There is a need to account for the cyclic nature of the loads also for the ULS and ALS conditions. The relation to FLS is that whereas FLS covers failure due to the long term effects and is based upon expected load history, the check of ULS and ALS are based on the use of fractile loads and are related to survival of one extreme storm.

Comm 8.4.5
The following analysis procedure for low cycle fatigue during a severe storm requires that a history of action effects corresponding to this storm profile is established (Values of action effects related to number of wave cycles).

For assessment of low cycle fatigue it should be noted that the cyclic stress-strain curve is somewhat different from that of a stress-strain curve derived under monotonic loading, see Figure 21.

The hot spot stress ranges are assumed to be derived from linear elastic analysis. The hot spot stress range during a severe storm may imply local yielding at the hot spot. Thus, a correction of the elastic stress range is needed in order to derive a stress range that is representative for the actual strain range taking the non-linearity in material behaviour into account. To account for this the fatigue capacity for low cycle fatigue can be derived by one of the following methods:

1) Prepare a finite element model of the considered detail and perform a cyclic nonlinear analysis based on a cyclic stress-strain curve as shown in Figure 21. This provides the actual strain range at the hot spot.

2) Alternatively use the cyclic stress-strain relation combined with the Neuber's rule for derivation of actual strain. This procedure is illustrated in Figure 22.

If the cyclic stress-strain relation is combined with the Neuber's rule, the Neuber's formula (refs. /31/ and /38/) can be written as follows:

$$\frac{\sigma_n^2 \cdot SCF^2}{E} = \sigma_{actual\,HSS}\left[\frac{\sigma_{actual\,HSS}}{E} + \left(\frac{\sigma_{actual\,HSS}}{K'}\right)^{1/n}\right] \qquad \textbf{(10)}$$

where

σ_n = nominal stress
SCF = stress concentration factor from linear elastic analysis (the same as used for high cycle fatigue)
$\sigma_{actual\ HSS}$ = the actual stress at the considered hot spot from a non-linear finite analysis using a cyclic stress-strain curve.
E = Young's modulus
n, K' = material coefficients

K' and n can be obtained by experiments for the actual material, weld and heat effected zone. For assessment of magnitude of low cycle fatigue the following values may be used for a first assessment of criticality with respect to low cycle fatigue:

K' = 582 (in MPa if this value is used for stress).
n = 0.111.
Some coefficients of n and K' for base metal of different steel grades and for welded metal are given in ref. /16/. For the heat affected zone, it is recommended to assume welded metal, if non-linear analysis is carried out to obtain the strain range.

The equation for actual stress based on Neuber's formula can be solved by iteration. Then the strain is calculated from the Ramberg-Osgood relation as

$$\varepsilon_{nl} = \frac{\sigma_{actual\,HSS}}{E} + \left(\frac{\sigma_{actual\,HSS}}{K'}\right)^{1/n} \qquad \textbf{(11)}$$

Then a pseudo elastic stress can be calculated as

$$\sigma_{pseudo} = E\,\varepsilon_{nl} \qquad \textbf{(12)}$$

This hot spot stress range (pseudo elastic stress range) should be combined with the hot spot stress S-N curve T for tubular joints in DNV - RP - C203 before fatigue damage is calculated.

For other types of welded connections the S-N curves in DNV-RP-C203 should be used.

The procedure for low cycle fatigue presented here is used for a tubular in seawater with cathodic protection. This gives results as shown in Figure 4.

An S-N curve for a tubular joint in air environment has been constructed following this procedure. The S-N curve shows a similar shape in the region less than 100 000 as proposed by Berge et al. (2007). However, the present procedure provides a slightly more conservative S-N curve in this region. (Berge et al. (2007) has linked their proposal to a slightly different tubular joint S-N curve: HSE (1995)). Thus, the proposed procedure for low cycle is in agreement with recent test data also for tubular joints.

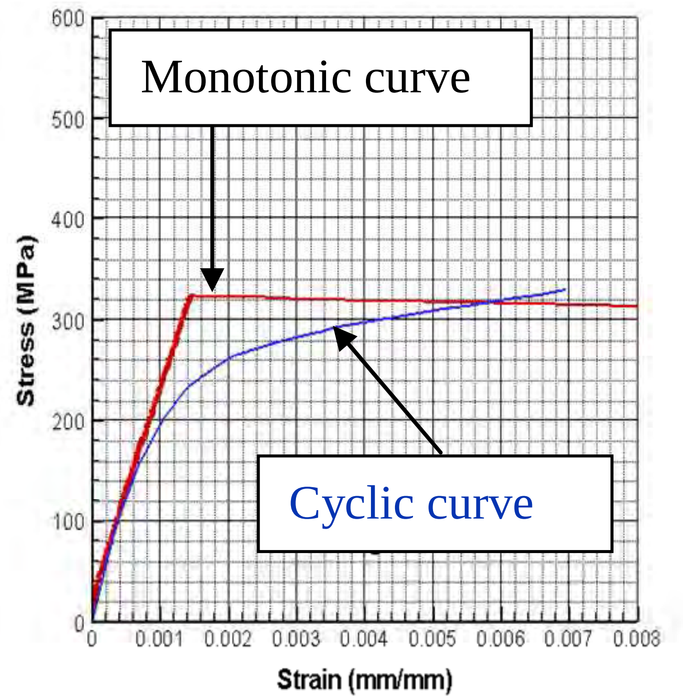

Figure 21 Monotonic and cyclic stress-strain curve

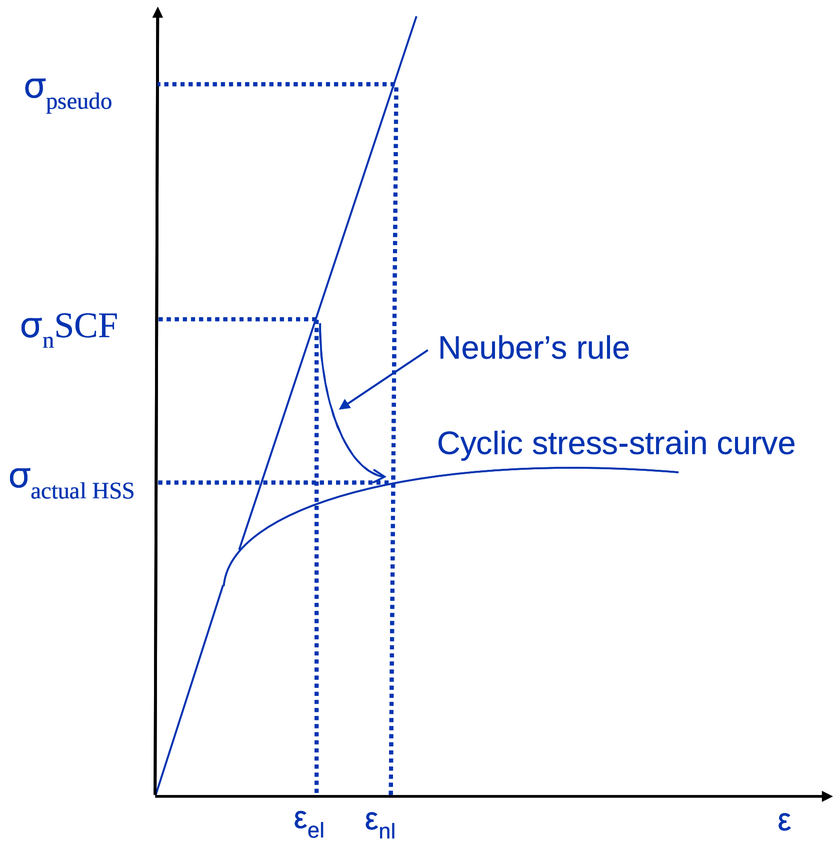

Figure 22 The Neuber approach and use of pseudo-elastic stress

Comm. 8.5

In case of wave-in-deck loads it is recommended that the structural analyses are carried out as dynamic time domain analyses in order to adequately account for the phase differences between the vertical and horizontal wave-in-deck loads and other wave loads and to account for dynamic effects.

Comm. 8.7
Guidance on possible repair techniques can be found in /28/.

Comm. 9.2 Use of probabilistic analysis for planning in-service inspection
Use of probabilistic analysis methods for planning in-service inspection is not properly included in design standards. Reference is e. g. made to refs. /24/, /25/, /27/ and /32/ for use of probabilistic analysis for planning in-service inspection for fatigue cracks.

One may find that it is a significant amount of engineering resources needed to use detailed probabilistic methods for planning of inspection. However, the largest amount of work is related to a proper fatigue analysis of the structure. In order to reduce the work a method may be to analyse some joints and then derive guidelines based on this.

A first step to use probabilistic analysis for planning in-service inspection for fatigue cracks is to calculate accumulated probability of failure based on S-N data as is shown in Figure 23. Normally the accumulated Palmgren Miner damage less than 1.0 in Figure 23 is of interest. This part of the figure for accumulated probability of failure based on S-N data is shown in Figure 24.

The different curves are derived for different assumptions on uncertainty. All the curves include uncertainty in S-N data. The distribution of S-N data is assumed to be normal distributed in a logarithmic S-N diagram with standard deviation equal 0.20.

Four of the curves in the figures include uncertainty with respect to the Palmgren Miner sum as failure criterion.

The Palmgren Miner is assumed log normal distributed with median 1.0 and CoV = 0.3.

Another uncertainty used in the figures is due to environment, structural modelling and calculation of nominal load effect in the structure. This uncertainty is described by CoVnom.

Also some uncertainty on hot spot stress calculation is included. This uncertainty is described by CoVhs.

Figure 24 may be used to assess time interval to first inspection as shown in Figure 25 for one high consequence joint and one low consequence joint. For a high consequence joint the accumulated fatigue damage is d_{acc} = 0.18 for accumulated probability of a fatigue crack equal 10^{-3} using the lowest curve in Figure 25. Thus, if the calculated fatigue life for the considered detail is $T_{\text{detail calculated}}$ which corresponds to a Miner Palmgren sum equal 1.0, the time until first inspection can be calculated as

$$T_{insp} = d_{acc} T_{\text{detail calculated}} \qquad \textbf{(13)}$$

Provided that fatigue cracks are <u>not found</u> during this inspection using a reliable inspection method such as eddy current or magnetic particle inspection of a joint with possibility for significant redistribution of stress during crack growth. This will be the case for a simple tubular joint in a jacket structure, hence the time interval to the next inspection can be derived as:

$$\Delta T_{insp} = T_{\text{insp}} \qquad \textbf{(14)}$$

Using the same inspection methodology for a joint with less possibility of redistribution of stresses during crack growth a shorter time interval should be used. The following equation is proposed to account for possibility of redistribution of stresses during crack growth and efficiency of inspection method to detect fatigue cracks:

$$\Delta T_{insp} = \lambda T_{\text{insp}} \qquad \textbf{(15)}$$

Based on consequence of a fatigue failure, contribution to residual strength of the structure with failure of other members, possibility for redistribution of stresses during crack growth and efficiency of inspection

method to detect fatigue cracks values for d_{acc} and λ may be proposed as shown in Table 6. The guidance in Table 6 is presented based on experience from a number of RBI analysis of different structures by DNV.
It should be noted that the calculated efficiency of the inspection is dependent on probability of detection, but it is also dependent on fatigue crack growth functions (also denoted geometry functions in fracture mechanics).

It should also be mentioned that the results from RBI are dependent on uncertainty related to action effects. The table is based on an assumption of normal uncertainty in calculation of hot spot stress. Thus, rather than using the results in Table 6, RBI analyses are recommended for the considered structure. However, it should be stressed that the output from RBI is not better than that of the reliability of the input data. Thus, the significance of consistent fatigue analysis as basis for the RBI analysis is emphasized.

Updating of reliability after an inspection should be based on relevant data for probability of detection (POD). The POD curves in Figure 26 may be used if other data are not available. These curves are applicable for Eddy Current and Magnetic Particle Inspection. By “controlled working condition” is understood easy access above water. By “underwater working conditions” is understood working conditions below water that a diver will likely meet. This curve is derived based on an assessment of test data derived by divers below water. (Test data have also been reported in conditions in test tanks. However, these data have not been included in the derivation of this curve). Mathematically the curves can be expressed as follows

$$p = 1 - \frac{1}{1 + \left({}^{x}\!/_{x_0} \right)^{b}} \qquad \textbf{(16)}$$

where
x_0 = 0.1608 and b = 1.0092 for controlled working conditions and
x_0 = 1.1007 and b = 0.9123 for underwater working conditions.

It is stressed that the inspection intervals derived in the examples listed above are derived based on an assumption that fatigue cracks are not found during an inspection. The time to next inspection will then depend on the crack growth characteristics and the POD curve used. This may lead to reduced or increased time interval to the next inspection depending on the crack growth characteristics and the POD curve. It should also be noted that if a fatigue crack is found, this will likely reduce the time to the next inspection even if the crack is repaired.

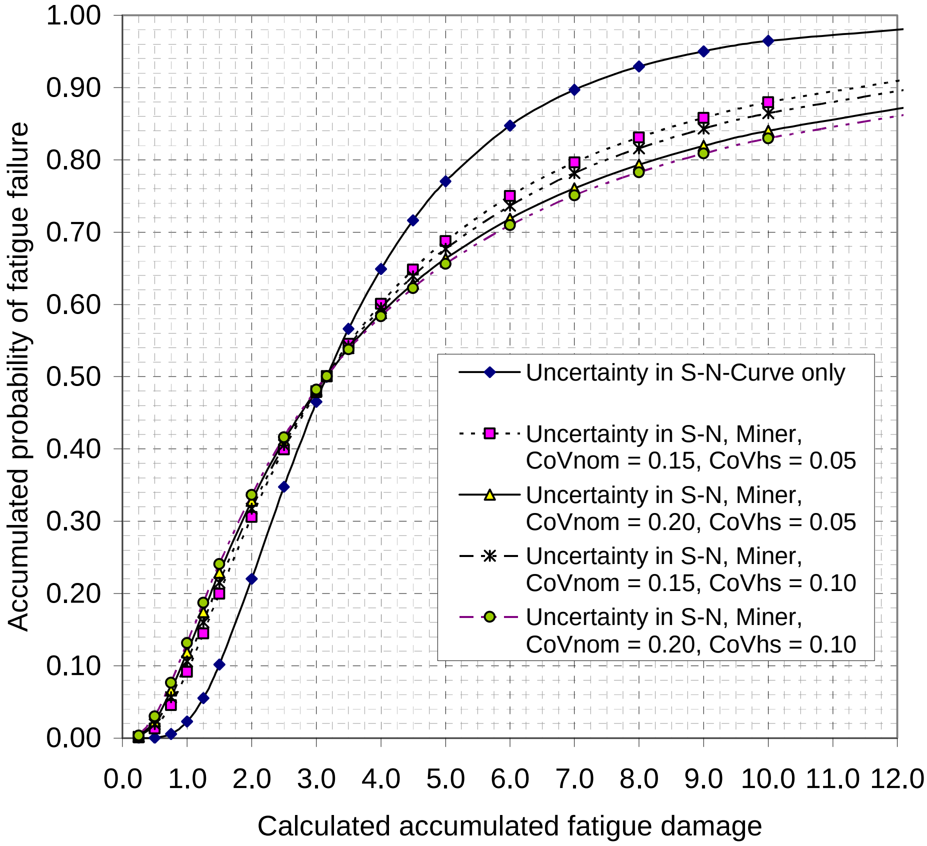

Figure 23 Calculated accumulated probability of fatigue failure as function of calculated accumulated fatigue damage

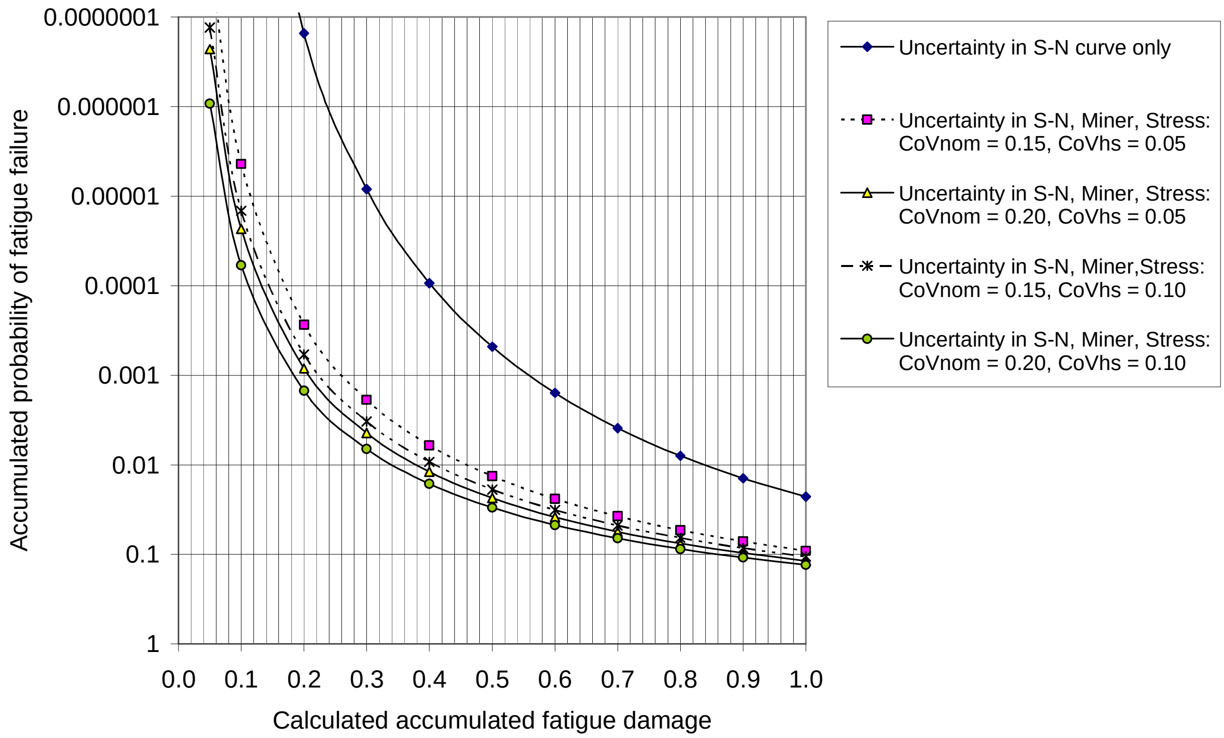

Figure 24 Calculated accumulated probability of fatigue failure as function of calculated accumulated fatigue damage

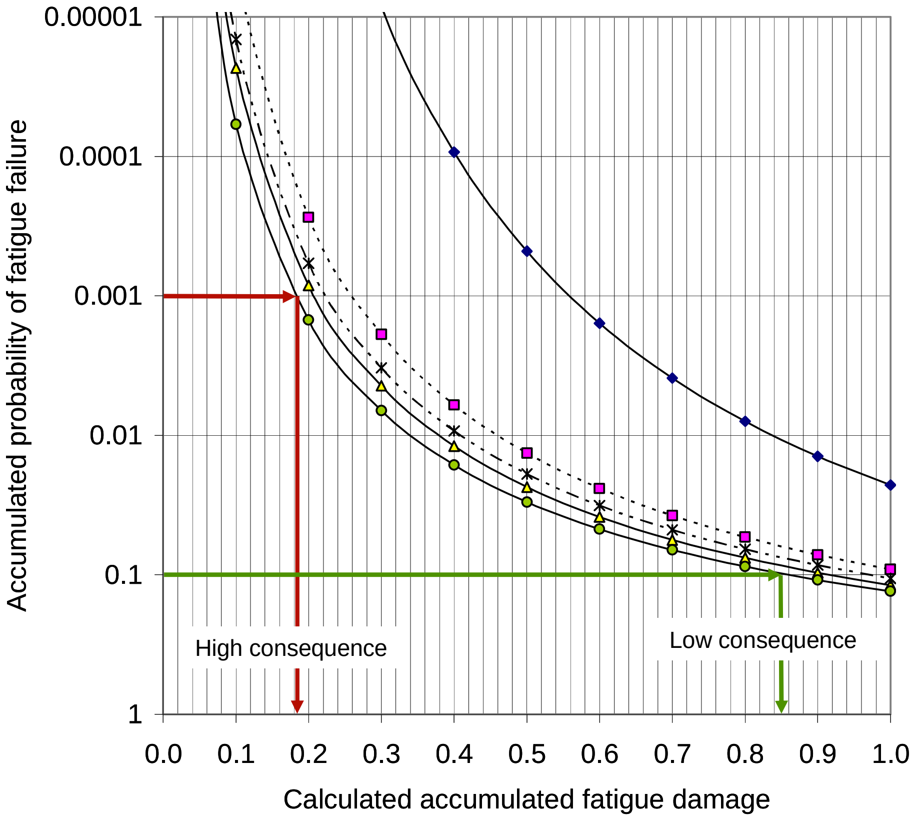

Figure 25 Example of accumulated damage at planned first inspections depending on consequence of fatigue failure

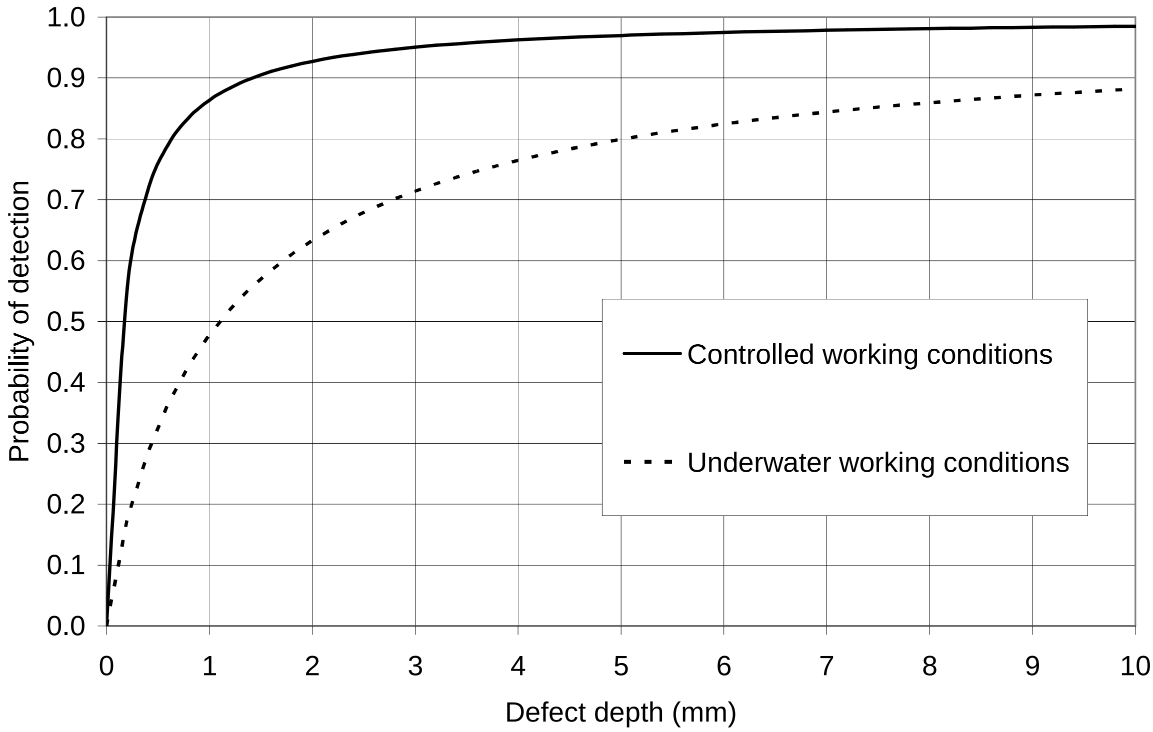

Figure 26 Probability of detection curves for Eddy Current and Magnetic Particle Inspection

Table 6 Time to inspection and time between inspections

Consequence	Working conditions (EC and MPI)	d_{acc}	λ
High	Good	0.15	1.3
	Underwater		1.0
Medium	Good	0.30	2.0
	Underwater		1.7
Small	Good	0.50	-
	Underwater		-

Comm. 9.6 Effect of different crack growth characteristics on inspection interval

Examples of connections with different crack growth characteristics with the same calculated fatigue lives are shown in Figure 27. (Failure criterion is crack growth through the thickness). It is seen from Figure 27 that the time interval for a reliable inspection is dependent on the crack growth characteristics which again is dependent on type of connection. Crack growth characteristics for a simple tubular joint in “as welded” condition is indicated in Figure 27a. It is observed that there is a significant time interval (t_d to t_T) for detection of the crack before it grows through the chord thickness (T).

In some situations it is difficult to achieve sufficient calculated fatigue without weld improvement such as grinding of the weld toe. This means that the hot spot stress range is larger than if an acceptable fatigue life could be documented without grinding. After grinding the crack the initiation period becomes longer, but the crack growth period is shortened due to increased stress range, ref. Figure 27b. This reduces somewhat the time interval for detection of cracks.

Other details show a less possibility of redistribution of stresses during crack growth. A butt weld subjected to pure axial loading is an example of this as shown in Figure 27c in “as welded” condition. It is observed that due to higher membrane stress the crack growth is faster and the time interval for detecting the fatigue cracks is reduced as compared with a simple tubular joint. If the nominal stress normal to a butt weld is so large that machining/grinding of the weld flush with the base material as shown in Figure 27d is required, the initiation time will likely be longer, but the crack growth will be even faster than for “as welded” condition. This should be kept in mind when planning in-service inspection of such connections which e. g. is used in tethers of tension leg platforms.

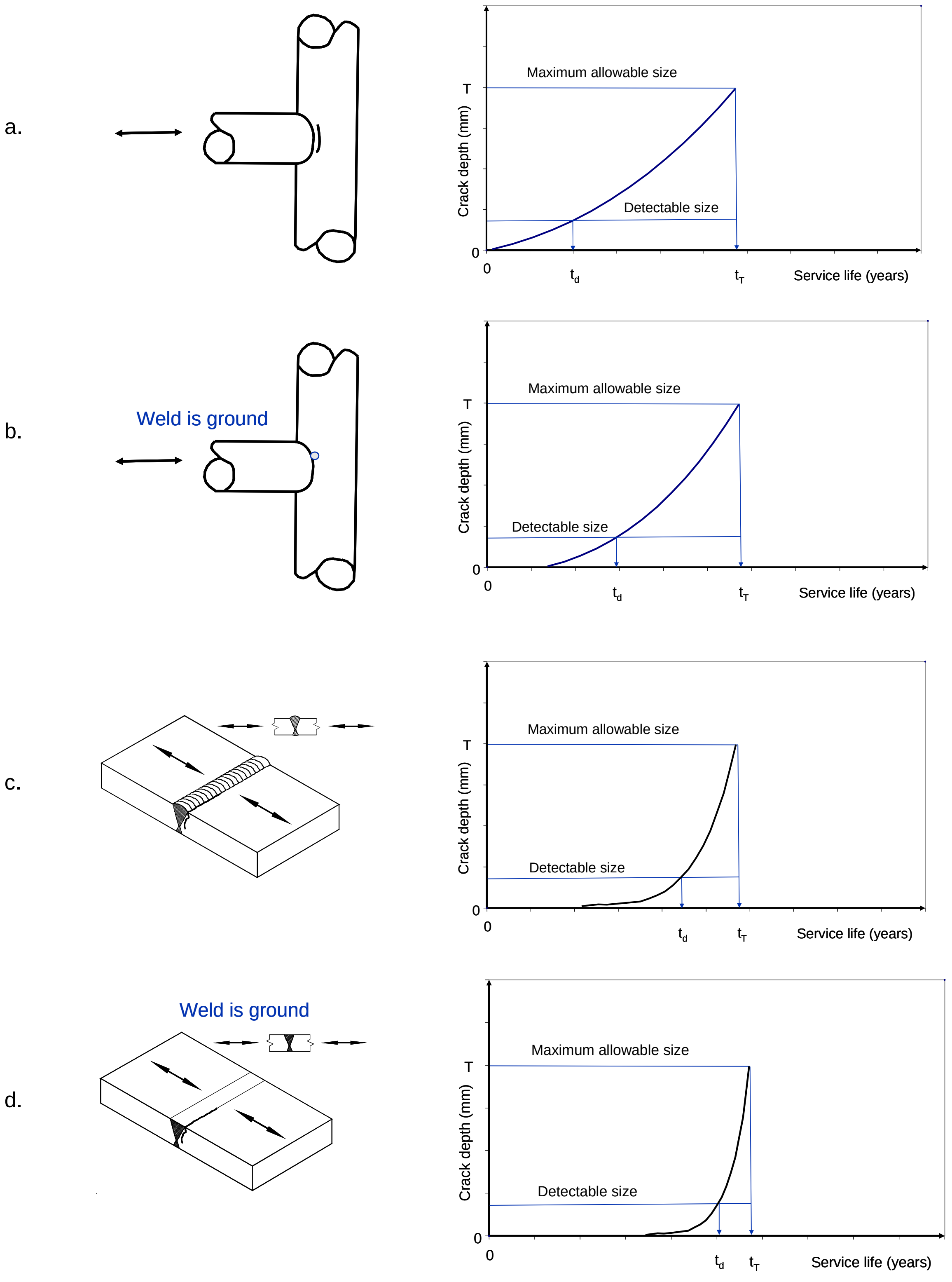

Figure 27 Sketch showing difference in crack growth for as welded and ground connections

Bibliography

/1/ API RP 2A-WSD Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, 21st edition December 2000.

/2/ Bea, R. G., Pawsey, S. F., and Litton, R. W.: Measured and Predicted Wave Forces on Offshore Platforms. OTC Paper no 5787. 1988.

/3/ BS 7910:1999 “Guide on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures”. British Standard.

/4/ Buitrago, J., Healy, B. E. and Chang, T. Y.: Local Joint Flexibility of Tubular Joints. OMAE, Glasgow 1993.

/5/ DNV Report no 85-3175 Grind repairs of welded structures. Phase I Plate structures. DNV 05.05.1985.

/6/ DNV Report no 85-3713 Grind repairs of welded structures. Phase II Tubular joints. DNV 12.12.1985.

/7/ DNV Report no 86-3183 Grind repairs of welded structures. Phase II Tubular joints. Technical summary report Task 3 Brace repair. DNV 20.03.1986.

/8/ DNV Report no 86-3447 Grind repairs of welded structures. Phase II Tubular joints. Technical summary report Task 7. DNV 29.08.1986.

/9/ DNV Report no 86-3452 Grind repairs of welded structures. Phase II Tubular joints. Technical summary report Task 4 Y-joints. DNV 03.09.1986.

/10/ DNV Report no 87-3093 Grind repairs of welded structures. Phase II Tubular joints. Final report on tubular joints fatigue tests. DNV May 1987.

/11/ DNV Report no 87-3503 Grind repairs of welded structures. Phase II Tubular joints. Stress analysis of grind repaired tubular joints. DNV 06.08.1987.

/12/ DNV Report no 87-3680 Grind repairs of welded structures. Executive summary of results. DNV 23.11.1987.

/13/ DNV Report no 2007-2187 Ekofisk; Update to Metocean Criteria; Phase 3

/14/ DNV-RP-C203 Fatigue Design of Offshore Steel Structures. DNV August 2005.

/15/ Efthymiou, M.: Development of SCF Formulae and Generalised Influence Functions for use in Fatigue Analysis. Recent Developments in Tubular Joint Technology, OTJ’88, October 1988, London.

/16/ Heo, H. J., Kang, J. K., Kim, Y. I., Yoo, I. S., Kim, K. S., and Urm, H. S. (2004), “A Study on the Design Guidance for Low Cycle Fatigue in Ship Structures,” Published in PRADS2004, Hamburg, Germany.

/17/ Inglis, R. B. and Kint, T. E.: Predicted and Measured Long Term Stress Range Distributions for the Fulmar A Platform. Behaviour of Offshore Structures, Delft, 1985.

/18/ ISO 19900 General Requirements for Offshore Structures.

/19/ ISO 19902 Fixed Steel Offshore Structures. November 2007.

/20/ Kanter, P. A., Scherf, I., Pettersen, Ø., Osnes, J., Grigorian, H. Yu, W. G. and Christensen, Ø. Instrumentation of Ekofisk Platforms. OTC Paper No 13193, 2001.

/21/ Karunakaran, D. and Haver, S. Dynamic Behaviour of Kvitebjørn Jacket Structure – Numerical Predictions versus full-Scale Measurements. http://www.ivt.ntnu.no/imt/courses/tmr4195/ 17 October 2007.

/22/ Kenley, R. M.; Measurement of Fatigue Performance of Forties Bravo. OTC Paper No 4402, 1982.

/23/ Langen, I., Spidsøe, N., Bruce, R. L. and Heavner, J. W.: Measured Dynamic Behaviour of North Sea Jacket Platforms. OTC Paper No 4655, 1984.

/24/ Lotsberg, I. and Sigurdsson, G., “Assessment of Input Parameters in Probabilistic Inspection Planning for Fatigue Cracks in Offshore Structures”, Icossar Rome, June 2005, Proceedings of the ninth International Conference on Structural Safety and Reliability. ICOSSAR’05. Safety and Reliability of Engineering Systems and Structures. ISBN 905966 0404.

/25/ Lotsberg, I., Sigurdsson G. and Wold, P.T.: Probabilistic Inspection Planning of the Åsgard A FPSO Hull Structure with respect to Fatigue. OMAE’99 St. John’s, New Foundland, July 1999. Also in Journal of Offshore and Arctic Engineering, Vol. 122, May 2000, pp. 134 - 140.

/26/ Lotsberg, I., Sigurdsson, G. Arnesen, K. and Hall. M.: Recommended Design Fatigue Factors for Piles subjected to Dynamic Actions from Pile Driving. OMAE 2008-57251. To be presented at OMAE 2008 Estoril, Portugal.

/27/ Moan, T., (2004) “Reliability-based Management of Inspection, Maintenance and Repair of Offshore Structures”, Vol. 1, no. 1, Journal of Structure and Infrastructure Engineering.

/28/ MSL Engineering Limited, Doc ref. C237R001 rev. 1 Nov. 2004. Assessment of repair techniques for ageing or damaged structures.

/29/ N-001 Structural Design. NORSOK. Rev. 4. February 2004.

/30/ N-004 Design of Steel Structures. NORSOK Standard, rev. 02, 2004.

/31/ Neuber, J (1961), “Theory of Stress Concentrations for Shear-Strained Prismatical Bodies with Arbitrary Nonlinear Stress-Strain Law,” J. of Applied Mechanics (ASME), Vol.28.

/32/ Sigurdsson, G., Lotsberg, I., Myhre, T. and Orbeck-Nilssen, K.: Fatigue Reliability of Old Semisubmersibles, OTC 11950 May 2000, Houston.

/33/ Skallerud and Amdahl, Nonlinear Analysis of Offshore Structures, Research Studies Press ltd, Baldock, Hertfordshire, England

/34/ Spidsøe, N., Brathaug, H. P. and Skjåstad, O.: Nonlinear Random Wave Loading on Fixed Platforms. OTC Paper No 5101. Houston, 1986.

/35/ Stacey, A., Sharp, J. V. and Nichols, N. W.: The Influence of Cracks on Static Strength of Tubular Joints. OMAE, Firenze, 1996.

/36/ The Alexander Kielland Accident. NOU 1981:11. (In Norwegian with summary in English).

/37/ Ultiguide, 1999. DNV, Sintef, Bomel: Best practice guidelines for use of non-linear analysis methods in documentation of ultimate limit states for jacket type offshore structures.

/38/ Urm, H. S., Yoo, I. S., Heo, J. H., Kim, S. C. and Lotsberg, I.: Low Cycle Fatigue Strength Assessment for Ship Structures. PRADS, Lubeck – Travemunde, Germany, 12 – 17 September 2004, 9th International Symposium on Practical Design of Ships and Other Floating Structures. Edited by H. Keil and E. Lehman, Seehafen Verlag. Vol. 2. ISBN 3-87743-808-3.

/39/ Webb, R. M. and Corr, R. B.: Full Scale Measurements at Magnus. Proceedings of E&P Forum Workshop on Wave and Current Kinematics and Loading, Paris, 25-26 October, 1989.

/40/ Wästberg, S.; Salama, M. Fatigue Testing and Analysis of Full Scale Girth weld Tubulars. OMAE2007-29399.